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# Minimum requirements for durable concrete

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Carbonation- and chloride-induced corrosion,  
freeze-thaw attack and chemical attack

**Edited by D.W. Hobbs**

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Built: 1915.

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### **Corrections:**

Rear cover : Bryan

Fig. captions. Chapter 5:  $MgSO_4$   
not  $Mg_2SO_4$

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## PREFACE

Within the forthcoming European Standard pr EN 206, *Concrete - Performance, production and conformity*, durability of concrete will rely on prescriptive specification of minimum grade, minimum binder content and maximum water-binder ratio for a series of defined environmental classes. Minimum cover to reinforcement is specified in DD ENV 1992-1-1 (Eurocode 2: *Design of concrete structures*. Part 1).

In the development of pr EN 206, it has not proved possible within the European Committee for Standardization (CEN) to agree common values for the specification parameters to cover the wide range of climates and wide range of cements in use in the EU Member States. The standard will, therefore, almost certainly include indicative values, and it will be left to individual Member States to specify national values where they require them to differ from the indicated values. In the present draft, the indicative values are based on the mean of the range of values currently in use across Europe, and apply only to Portland cements (CEM I cements). No procedure for modifying these values for other cements has yet been agreed.

It is thus clear that the prescriptive specification of durable concrete for construction in the UK is entirely dependent upon the establishment of UK national requirements for minimum grade, minimum binder content and maximum water-binder ratio based on values of minimum cover given in DD ENV 1992-1-1. It is not acceptable to base UK national requirements on existing British Standards for two reasons:

- pr EN 206 uses a new system of exposure classification based on deterioration mechanisms. Existing British Standard exposure classes do not use this system and cannot be related satisfactorily to the new classes.
- There has been no recent establishment of a UK consensus on durability requirements as the necessary revision of BS 8110 has not been made because of the higher priority of work on development of the European Standards. There is strong belief that some recommendations in BS 8110 will not provide adequate durability, particularly where resistance to chloride ingress is required, whereas other recommendations may be unnecessarily onerous.

The objective of the work presented in this book is to review the UK literature and, where required, the international literature to establish the minimum specifications for concrete necessary to achieve durable concrete in the exposure conditions defined by the pr EN 206 for the broader range of binders used in the UK. This necessitated the making of a number of assumptions, and consideration of the minimum requirements presented in this book should include an assessment of these assumptions, to check that they provide an appropriate safety margin for structures in service. The work is being published to stimulate wider informed debate.

Due to lack of published data, it was not possible to establish the minimum specification for concrete prepared using all the common binders for each of the pr EN 206 exposure classes.

The work presented was carried out by the staff of the British Cement Association and the Building Research Establishment and was part-funded by the Department of the Environment, Transport and the Regions under their Partners in Technology programme.

# CONTENTS

## Preface

1.	<b>Common cements and concrete exposure classes</b>	1
	D. W. Hobbs and J. D. Matthews	
1.1	Common cements	1
1.1.1	Types of cements (Spooner, 1995)	1
1.1.2	Constituents of cement (Spooner, 1995)	2
1.1.3	Strength classes (Corish, 1994)	2
1.1.4	Sulfate resisting cements to pr ENV 197-X	4
1.1.5	Additions (Harrison, 1994)	5
1.2	Exposure classes	5
1.2.1	pr EN 206 (BSI 1997c, d)	5
1.2.2	BS 8110 (BSI: 1997e) and BS 5328 (BSI: 1997f)	8
1.3	References	10
2.	<b>Minimum requirements for concrete to resist carbonation-induced corrosion</b>	11
	D. W. Hobbs, B. K. Marsh, J. D. Matthews and S. Petit	
2.1	Summary	11
2.2	Introduction	12
2.3	Methodology	14
2.4	Carbonation-induced corrosion in the UK	14
2.5	Measurement of carbonation depth	15
2.6	Concretes subject to moderate humidity (XC3)	16
2.6.1	CEM I concretes (PC)	16
2.6.2	CEM I/fly ash concretes	19
2.6.3	CEM I/slag concretes	24
2.6.4	Minimum specification for concrete subject to exposure class XC3	26
2.7	Concretes subject to cyclic wetting and drying (XC4)	27
2.7.1	CEM I concretes	27
2.7.2	CEM I/fly ash concretes	28
2.7.3	CEM I/slag concretes	30
2.7.4	Minimum specification for concrete subject to exposure class XC4	30
2.8	Minimum specification for concrete subject to exposure classes XC1 and XC2	31
2.9	Discussion	31
2.9.1	Minimum binder content	31
2.9.2	Influence of curing	32
2.9.3	Exposure classification	32
2.9.4	Assumptions made during analysis of available data	33
2.9.5	Discussion of the assumptions	34

2.10	Minimum requirements for concretes to resist carbonation-induced corrosion	34
2.11	References	36
	Appendix 2.1: Silica fume concretes subject to moderate humidity (XC3)	40
	Appendix 2.2: Unpublished BCA and BRE data on the carbonation of CEM I, CEM I/fly ash and CEM I/slag concretes subject to sheltered external exposure	41
3.	<b>Minimum requirements for concrete to resist chloride-induced corrosion</b> D. W. Hobbs and J. D. Matthews	43
3.1	Summary	43
3.2	Introduction	44
3.3	Chloride-induced corrosion in the UK	48
3.4	Chloride ingress: marine exposure class XS3	50
3.5	Service life and the propagation period: exposure class XS3	55
3.6	Minimum specification for concrete subject to exposure class XS3	58
	3.6.1 Assumptions based on field performance: CEM I concretes	58
	3.6.2 Minimum specification: CEM I concretes	58
	3.6.3 Assumptions based on field performance: CEM II/B-S, II/A-D, II/B-V, III/A and III/B concretes	60
	3.6.4 Minimum specification: CEM II/B-S, II/A-D, II/B-V, III/A and III/B concretes	60
3.7	Chloride ingress: marine exposure class XS1	62
3.8	Minimum specification for concrete subject to exposure Class XS1	66
3.9	Chloride ingress into submerged concrete: marine exposure class XS2	66
3.10	Minimum specification for concrete subject to exposure class XS2A	66
3.11	Chloride ingress: de-icing salt exposure class XD3	67
	3.11.1 CEM I concretes	67
	3.11.2 CEM II/B-V and CEM III/A concretes	71
3.12	Service life: exposure class XD3	74
3.13	Minimum specification for concrete subject to external class XD3 exposure	76
	3.13.1 Assumptions based on field performance: CEM I concretes	76
	3.13.2 Minimum specification: CEM I concretes	76
	3.13.3 Minimum specification: non-CEM I concretes	78
3.14	Chloride ingress: exposure class XD1	78
3.15	Chloride ingress: permanently submerged concrete: exposure class XD2A	78
3.16	Assumptions made during analysis of available data.	79
3.17	Minimum requirements for concrete to resist chloride-induced corrosion	80
	3.17.1 Chlorides other than sea water	80
	3.17.2 Chlorides from sea water	82
3.18	Acknowledgement	83
3.19	References	83

4.	<b>Minimum requirements for concrete to resist freeze-thaw attack</b>	91
	D. W. Hobbs, B. K. Marsh and J. D. Matthews	
4.1	Summary	91
4.2	Introduction	92
4.3	Freeze-thaw attack in the UK	94
4.4	Testing	97
	4.4.1 Tests for freeze-thaw resistance of concrete	97
	4.4.2 Tests for scaling resistance of concrete due to freeze-thaw attack in the presence of salt.	98
	4.4.3 Tests on aggregates	99
4.5	Air-entrainment and freeze-thaw resistance	99
	4.5.1 Dependence on volume concentration of air	99
	4.5.2 Dependence on air-void spacing	100
4.6	Air-entrained concretes saturated without salt: XF3	103
	4.6.1 CEM I (and CEM I/SR) concretes	103
	4.6.2 CEM I/fly ash concretes	104
	4.6.3 CEM I/slag concretes	104
	4.6.4 Portland limestone cement concretes, CEM II/A-L, B-L	105
	4.6.5 CEM I/silica fume concretes	105
	4.6.6 Minimum specification for air-entrained concrete subject to exposure class XF3	106
4.7	Air-entrained concretes saturated with salt: XF4	106
	4.7.1 CEM I and CEM I/SR concretes: marine exposure	106
	4.7.2 CEM I/slag concretes: marine exposure	108
	4.7.3 CEM I/fly ash concretes: marine exposure	109
	4.7.4 CEM I concretes: de-icing salt exposure	109
	4.7.5 CEM I/fly ash concretes: de-icing salt exposure	110
	4.7.6 CEM I/slag concretes: de-icing salt exposure	112
	4.7.7 CEM I/silica fume concretes: de-icing salt exposure	114
	4.7.8 Minimum specification for air-entrained concrete subject to exposure class XF4.	114
4.8	Non-air-entrained concretes	115
	4.8.1 Performance of high quality concretes	115
	4.8.2 Minimum specification for non-air-entrained concrete subject to exposure classes XF3 and XF4	118
4.9	Minimum requirements for concrete to resist freeze-thaw attack	119
	4.9.1 50 years vs. 100 year required working life	119
	4.9.2 Comparisons with existing specification	120
	1 Relevant exposure classes in British Standards.	120
	2 Comparison of recommendations for minimum concrete quality.	120
	4.9.3 Minimum requirements	121
4.10	References	122

5.	<b>Minimum requirements for concrete to resist chemical attack</b>	131
	D. W. Hobbs and J. D. Matthews	
5.1	Summary	131
5.2	Introduction	132
5.3	Chemical attack in the UK	136
	5.3.1 Sulfate attack	136
	5.3.2 Acid attack	137
5.4	Sulfate attack	138
	5.4.1 General	138
	5.4.2 Criteria for assessment	138
	5.4.2 Portland cements: CEM I and CEM I/SR	139
	5.4.3 Minimum specification: CEM I and CEM I/SR concretes	143
	5.4.4 Portland limestone cements: CEM II/A-L and B-L	144
	5.4.5 Minimum specification: CEM II/A-L and B-L concretes	145
	5.4.6 Cements containing fly ash: CEM II/A-V, B-V, IV/A and IV/B	145
	5.4.7 Minimum specification for cements containing fly ash: CEM II/A-V, B-V, IV/A and IV/B	150
	5.4.8 Cements containing slag: CEM II/A-S, B-S and III/A, B, C	150
	5.4.9 Minimum specification for cements containing slag: CEM II/A-S, B-S and III/A, B, C	153
	5.4.10 Portland silica fume cements: CEM II/A-D	153
5.5	Sea water attack	154
5.6	Acid attack	156
	5.6.1 Influence of cement type	156
	5.6.2 Minimum specification for concretes subject to sulfate and acid attack	157
5.7	Minimum requirements for concrete to resist sulfate and acid attack	158
	5.7.1 Sulfate attack	158
	5.7.2 Acid attack	159
5.8	References	159
	<b>Author index</b>	163
	<b>Subject index</b>	167
	<b>Glossary of cement types: equivalence of British and European cements</b>	back cover



## CHAPTER 1

# Common cements and concrete exposure classes

by D. W. Hobbs, British Cement Association

J. D. Matthews, Building Research Establishment

## 1.1 Common cements

### 1.1.1 Types of cement (Spooner, 1995)

The European prestandard for common cements was published in the UK in 1995 (BS DD ENV 197-1). Since DD ENV 197-1 was published, it has been developed further towards a full European standard, the latest draft of which appeared in March 1997 (pr EN 197-1). Five types of cements are covered, ranging from CEM I (Portland cement) to CEM V (composite cement). Within each of these five types are subdivisions indicating the proportion of Portland clinker, the second main constituent, the standard strength class and rate of strength gain. For example, Portland fly ash cement may be denoted by CEM II/B-V 32.5N where:

- CEM II is the type of cement
- /B indicates a medium proportion of clinker. A would be higher and C lower;
- V is the subtype; indicates the second main constituent, in this case fly ash;
- 32.5 is the standard strength class;
- N is the sub-class: indicating normal early strength. R indicates rapid early strength. If left blank, it implies a normal early strength.

The cement types in DD ENV 197-1 (BSI, 1995a) and the British Cement Standards are compared in Table 1.1, whilst Table 1.2 gives the cement types covered by British Standards. British Standards do not exactly copy all the chemical requirements of DD ENV 197-1 and an important exception is the requirement for sulfate (as  $\text{SO}_3$ ). British Standards retain an effective limit of 3.5 %  $\text{SO}_3$  by mass of cement for all cements.

Table 1.1: Types of cements and their compositions in DD ENV 197-1.

Nomenclature	Cement designation to DD ENV 197-1	Clinker content (%)	Content of other main constituents (%)	British Standard cement
CEM I CEM I/SR	Portland cement	95 - 100 95 - 100	- -	BS 12: 1996 BS 4027: 1996 <sup>+</sup>
CEM II/A-S CEM II/B-S	Portland-slag cement	80 - 94 65 - 79	6 - 20 21 - 35	BS 146: 1996
CEM II/A-D	Portland-silica fume cement	90 - 94	6 - 10	None
CEM II/A-P CEM II/B-P CEM II/A-Q CEM II/B-Q	Portland-pozzolana cement	80 - 94 65 - 79 80 - 94 65 - 79	6 - 20 21 - 35 6 - 20 21 - 35	None None None None
CEM II/A-V CEM II/B-V CEM II/A-W CEM II/B-W	Portland-fly ash cement	80 - 94 65 - 79 80 - 94 65 - 79	6 - 20 21 - 35 6 - 20 21 - 35	BS 6588: 1996 BS 6588: 1996 None None
CEM II/A-T CEM II/B-T	Portland-burnt shale cement	80 - 94 65 - 79	6 - 20 21 - 35	None None
CEM II/A-L CEM II/B-L	Portland-limestone cement	80 - 94 65 - 79	6 - 20 21 - 35	BS 7583: 1996 None
CEM II/A-M CEM II/B-M	Portland-composite cement	80 - 94 65 - 79	6 - 20 21 - 35	None None
CEM III/A CEM III/B CEM III/C	Blastfurnace cement	35 - 64 20 - 34 5 - 19	36 - 65 66 - 80 81 - 95	BS 146: 1996 <sup>§</sup> None <sup>§</sup> None <sup>§</sup>
CEM IV/A CEM IV/B	Pozzolanic cement	65 - 89 45 - 64	11 - 35 36 - 55	None BS 6610: 1996
CEM V/A CEM V/B	Composite cement	40 - 64 20 - 39	36 - 60 61 - 80	None None

<sup>+</sup> Cement to BS 4027: 1996, *Sulfate resisting Portland cement* is included here but will eventually be covered specifically in a future part of DD ENV 197-1.

<sup>§</sup> In BS 4246: 1996 high slag blastfurnace cement covers a cement with a blastfurnace slag content of 50 - 85%.

Note: ASTM Type I to V cements contain no minor additional constituent.

ASTM Type I (no  $C_3A$  limit) and ASTM Type II ( $C_3A \leq 8\%$ ) are equivalent to CEM I/N<sup>7</sup>

ASTM Type III ( $C_3A \leq 15\%$ ) is equivalent to CEM I/R

ASTM Type IV ( $C_3A \leq 7\%$ ) is equivalent to what may be CEM I/42.5 MH

ASTM Type V ( $C_3A \leq 5\%$ ) is equivalent to CEM I/SR (ASTM C 150-96).

**Table 1.2:** Cement types in British Standards.

BS 12	Portland cement
BS 146	Portland-slag cement
BS 146	Blastfurnace cement
BS 4027	Sulfate-resisting Portland cement
BS 4246	High slag blastfurnace cement
BS 6588	Portland pulverized-fuel ash cements
BS 7583	Portland-limestone cement
BS 6610	Pozzolanitic pulverised-fuel ash cement
BS 5224	Masonry cement

### 1.1.2 Constituents of cement (Spooner, 1995)

The possible constituents of factory-produced cements, corresponding to the various cement types listed in Tables 1.1 and 1.2, are as follows:

- Portland cement clinker
- Calcium sulfate as a set-regulator
- Other main constituents (eg. fly ash, slag, limestone)
- Minor additional constituents (eg. fly ash, slag or limestone [to optimise properties])
- Additives (eg. grinding aids, air-entraining improvers, pigments).

The code letters used in the DD ENV 197-1 to indicate the type of second main constituent are:

- S - blastfurnace slag
- D - silica fume
- P - natural pozzolana
- Q - industrial pozzolana (the term 'artificial' is used in pr EN 197-1, BSI, 1997)
- V - siliceous fly ash (ie. UK fly ash)
- W - calcareous fly ash (ie. high-lime fly ash)
- T - burnt shale
- L - limestone
- M - a composite of two or more of the above

The term 'filler' used in DD ENV 197-1 (BSI, 1995a) has been deleted in pr EN 197-1 (BSI, 1997a).

### 1.1.3 Strength classes (Corish, 1994)

Whilst the UK takes a more conservative view than DD ENV 197-1 in terms of cement types and sulfate levels, it takes a far more liberal view in terms of cement strength classes, and this is summarised in Table 1.3. British standards generally adopt the DD ENV 197-1 strength classes but add several more. These extra strength classes in British cement standards will probably lapse when DD ENV 197-1 progresses to a full European Standard. Much of the test work on concrete referred to in the chapters of this book is restricted to class 42.5 CEM I and CEM I/SR cements and to combinations containing class 42.5 CEM I cements.

**Table 1.3:** Cement strength classes in DD ENV 197-1 and British cement standards.

Standard	Cement class	Minimum (N/mm <sup>2</sup> )			Maximum (N/mm <sup>2</sup> )
		2d	7d	28d	28d
ENV 197-1	32.5		16	32.5	52.5
	32.5R	10		32.5	52.5
	42.5	10		42.5	62.5
	42.5R	20		42.5	62.5
	52.5	20		52.5	
	52.5R	30		52.5	
BS 12	32.5 to 52.5 as above and				
	62.5	20		62.5	-
BS 146	32.5 to 52.5 as above and				
	52.5L	10		52.5	-
	42.5L		20	42.5	62.5
BS 4246	32.5L		12	22.5	-
BS 6610	22.5		12	32.5	-
BS 6699	All the classes for BS 146 and				
	37.5		16	37.5	57.5
	47.5		20	47.5	67.5

#### 1.1.4 Sulfate-resisting cements to pr ENV 197-X

Sulfate-resisting cements conforming to the latest draft of the prestandard ENV 197-X (BSI, 1997b) are divided into different cement types (see Table 1.4) belonging to the three main types:

- CEM I SR Sulfate-resisting Portland cement.
- CEM III SR Blastfurnace cement (CEM III/B and CEM III/C only).
- CEM IV/B SR Pozzolanic cement.

**Table 1.4:** Sulfate-resisting cement types and composition: Proportion by mass.

Main cement type	Designation	Notation	Main constituents (%)			Minor additional constituent (%)
			Clinker (K)	Blastfurnace slag (S)	Natural pozzolana (P) or siliceous fly ash (V)	
CEM I/SR	Sulfate-resisting Portland cement	CEM I/SR	95-100	-	-	0-5
CEM III/SR	Blastfurnace cement	CEM III/B SR	20-34	66-80	-	0-5
		CEM III/C SR	5-19	81-95	-	0-5
CEM IV/B SR	Pozzolanic cement	CEM IV/B SR	45-64	-	36-55	0-5

The values of the table refer to the sum of the main and minor additional constituents.

CEM I/SR cement complies with all the chemical requirements for CEM I of EN 197-1 and also complies with either of the following additional requirements:

Tricalcium aluminate (C <sub>3</sub> A)	Sulfate (as SO <sub>3</sub> )
≤ 3.0 (%)	≤ 3.5 (%)
≤ 5.0 (%)	≤ 2.5 (%)

In BS 4027: 1996, the maximum C<sub>3</sub>A content is 3.5 (%) by mass and the maximum sulfate as SO<sub>3</sub> is 2.5 (%) by mass.

CEM III/B SR and CEM III/C SR comply fully with all the chemical requirements for CEM III/B and CEM III/C respectively of DD ENV 197-1.

CEM IV/B SR complies fully with all the chemical requirements for CEM IV of DD ENV 197-1.

### 1.1.5 Additions (Harrison, 1994)

'Additions' are materials such as ground granulated blastfurnace slag (ggbs), fly ash and silica fume which can be added as separate powders at the concrete mixer. In the UK, the resulting cement is often referred to as a combination. The materials used as additions are also used as main constituents for cements. The main difference is that when they are used as part of a cement, the complete cement has to conform to DD ENV 197-1 and is likely to have a higher SO<sub>3</sub> level than a comparable combination. However, additions, being separate products, have to comply with a standard which includes clauses for the evaluation of conformity.

In pr EN 206, additions are split into Type I additions which are nearly inert and Type II additions which are latent hydraulic or pozzolanic.

The UK foreword to the European standard for fly ash, BS EN 450 (BSI, 1995b), notes that BS 3892: Part 1: 1997 fly ash has been used in a different way in the UK than in other European states. It is therefore possible for the UK to retain BS 3892: Part 1. The limit on loss-on-ignition for a BS 3892: Part 1 fly ash is higher than that for a BS EN 450 fly ash but a BS 3892: Part 1 fly ash has a lower limit on the proportion of coarse particles than an EN 450 fly ash.

Slag will be outside the scope of pr EN 206, and BS 6699: 1992: *Ground granulated blastfurnace slag for use with Portland cement* will continue.

## 1.2 Exposure classes

### 1.2.1 Pr EN 206 (BSI, 1997c, d)

The informative examples of the pr EN 206 exposure classes (BSI, 1997c) applicable to the UK which appear in the complementary UK concrete standard to pr EN 206 (BSI, 1997d) are given in Table 1.5. Table 1.6 gives limiting values for exposure classes for chemical attack. In total there are 23 exposure classes, split into six groups covering:

- 1) No risk of corrosion
- 2) Carbonation-induced corrosion
- 3) Chloride-induced corrosion resulting primarily from de-icing salts

- 4) Chloride-induced corrosion resulting from seawater exposure
- 5) Freeze-thaw attack
- 6) Chemical attack.

**Table 1.5:** Exposure classes.

Class designation	Description of the environment	Informative examples applicable in UK
<b>1 No risk of corrosion or attack</b>		
X0	No risk of corrosion of reinforcement or attack of concrete	Unreinforced concrete surfaces inside structures Unreinforced concrete completely buried in non-aggressive soil Unreinforced concrete permanently submerged in non-aggressive water
<b>2 Corrosion induced by carbonation</b>		
XC1	Dry	Reinforced and prestressed concrete surfaces inside structures except areas of structures with high humidity
XC2	Wet, rarely dry	Reinforced and prestressed concrete completely buried in non-aggressive soil Reinforced and prestressed concrete surfaces permanently submerged in non-aggressive water
XC3/XC4	Moderate humidity or Cyclic wet and dry	External reinforced and prestressed concrete surfaces sheltered from direct rain External exposed reinforced and prestressed concrete surfaces Reinforced and prestressed concrete surfaces inside structures with high humidity (eg. bathrooms, kitchens) Reinforced and prestressed concrete surfaces exposed to alternate wetting and drying
<b>3 Corrosion induced by chlorides other than from sea water</b>		
XD1	Moderate humidity	Reinforced and prestressed concrete surfaces in parts of bridges away from direct spray containing de-icing agents
XD2A	Wet, rarely dry - totally immersed on all sides	Reinforced and prestressed concrete surfaces totally immersed in water containing chlorides
XD2B	Wet, rarely dry - immersed on one side only	Reinforced and prestressed concrete elements where one surface is immersed in water containing chlorides
XD3	Cyclic wet and dry	Reinforced and prestressed concrete surfaces directly affected by de-icing agents (eg. walls, abutments and columns within 10m of the carriageway, parapet edge beams and buried structures less than 1m below carriageway level)

Table 1.5 (contd.....)

<b>4 Corrosion induced by chlorides from sea water</b>		
XS1	Exposed to airborne salt but not in direct contact with sea water	Reinforced and prestressed concrete surfaces in saturated salt air External reinforced and prestressed concrete surfaces in coastal areas more than 100m from the sea
XS2A	Submerged - totally immersed on all sides	Reinforced and prestressed concrete completely submerged more than 1m below lowest low water level
XS2B	Submerged - immersed on one side only	Reinforced and prestressed concrete elements where one surface is exposed to air (eg. immersed tunnels)
XS3	Tidal, splash and spray zones	Reinforced and prestressed concrete surfaces in sea water tidal, splash and spray zones down to about 1m below lowest low water level External reinforced and prestressed concrete surfaces less than 100m from the sea
<b>5 Freeze-thaw attack</b>		
XF1	Moderate water saturation, without de-icing agent	Walls and columns exposed to occasional freezing whilst wet without de-icing agent
XF2	Moderate water saturation, with de-icing agent	Walls and columns close to carriageways
XF3	High water saturation, without de-icing agent	Slabs subject to severe freezing without de-icing agent Elements subject to severe freezing whilst wet without de-icing agent Elements subject to severe freezing and frequent splashing or spray without de-icing agent Elements subject to severe freezing and significant capillary suction without de-icing agent
XF4	High water saturation, with de-icing agent	Elements subject to severe freezing with wet water containing de-icing agents Elements subject to severe freezing together with frequent splashing or spray with water containing de-icing agents
<b>6 Chemical attack</b>		
Where concrete is exposed to chemical attack as given in Table 1.6 which occurs in natural soils, groundwater and sea water, the exposure shall be classified as given below.		
XA1A	Non-aggressive chemical environments according to Table 1.6.	
XA1B	Slightly aggressive chemical environments according to Table 1.6.	
XA2A	Moderately aggressive chemical environments according to Table 1.6 or exposure to sea water.	
XA2B	Moderately aggressive chemical environments according to Table 1.6.	
XA3A	Highly aggressive chemical environments according to Table 1.6.	
XA3B	Highly aggressive chemical environments according to Table 1.6.	

**Table 1.6:** Limiting values for exposure classes for chemical attack.

The aggressive chemical environments classified below are based on natural environment at water/soil temperatures between 5°C and 25°C and mobile groundwater.							
The most onerous value for any single chemical characteristic determines the class.							
When two or more aggressive characteristics lead to the same class, the environment shall be classified into the next higher class.							
Chemical characteristic		XA1A	XA1B	XA2A	XA2B	XA3A	XA3B
SO <sub>4</sub> (mg/l) in water		≥200 and ≤400	>400 and ≤600	>600 and ≤1440	>1440 and ≤3000	>3000 and ≤6000	
SO <sub>4</sub> in soil	2:1 water/soil extract (g/l)	≥0.6 and ≤1.2	>1.2 and ≤1.8	>1.8 and ≤2.3	>2.3 and ≤3.7	>3.7 and ≤6.7	>3.8 and ≤6.7
	Acid extract (mg/kg)	≥2000 and ≤3000		>3000 and ≤12000		>12000 and ≤24000	
pH of groundwater or water extract		≤6.5 and ≥5.5		<5.5 and ≥4.5		<4.5 and ≥4.0	
CO <sub>2</sub> (mg/l) aggressive in water		≥15 and ≤40		>40 and ≤50		>100	
NH <sub>4</sub> (mg/l) in water		≥15 and ≤30		>30 and ≤60		>60 and ≤100	
Mg (mg/l)	Groundwater	<1000					>1000
	Brackish groundwater	>1000 and ≤3000				>3000	

**1.2.2 BS 8110 (BSI, 1997e) and BS 5328 (BSI, 1997f)**

The current UK exposure conditions described in BS 5328 (1997f) are reproduced in Table 1.7. Here there are six exposure classes, one of which is when the exposure is abrasive. It could be argued that broadly each of the BS 5328 exposures combines several of the BS EN 206 exposure classes. However, it is not possible to give precise equivalence.

The classification of the exposure conditions for sulfate and acid attack taken from BS 5328: 1997 are given in Tables 1.8 and 1.9. Here, there is similarity in the sulfate classes between the modified pr EN 206 chemical exposure classes and BS 5328: 1997 sulfate exposure classes, but for acid attack, the minimum pH in Table 1.6 is 4.0 (BSI, 1997c) as compared with 2.5 given in Table 1.9 (BS 5328: 1997).



**Table 1.7:** Classification of exposure conditions in BS 8110: 1997 and BS 5328: 1997.

Environment	Exposure conditions
Mild	Concrete surfaces protected against weather or aggressive conditions
Moderate	Exposed concrete surfaces but sheltered from severe rain or freezing whilst wet Concrete surfaces continuously under non-aggressive water Concrete in contact with non-aggressive water Concrete in contact with non-aggressive soil (see class 1 of Table 7a) Concrete subject to condensation
Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing or severe condensation
Very severe	Concrete surfaces occasionally exposed to sea water spray or de-icing salts (directly or indirectly) Concrete surfaces exposed to corrosive fumes or severe freezing conditions whilst wet
Most severe	Concrete surfaces frequently exposed to sea-water spray or de-icing salts (directly or indirectly) Concrete in sea water tidal zone down to 1m below lowest low water
Abrasive	Concrete surfaces exposed to abrasive action eg. machinery, metal-tyred vehicles or water carrying solids

**Table 1.8:** Sulfate exposure classes (BS 5328: 1997).

Sulfate class	Exposure conditions - Concentration of sulfate and magnesium				
	In groundwater		In soil or fill		
	SO <sub>4</sub> (g/l)	Mg (g/l)	By acid extraction	By 2:1 water/soil extract	
SO <sub>4</sub> (%)			SO <sub>4</sub> (g/l)	Mg (g/l)	
1	< 0.4	-	< 0.24	< 1.2	-
2	0.4 to 1.4	-	Classify on the basis of a 2:1 water/soil extract	1.2 to 2.3	-
3	1.5 to 3.0	-		2.4 to 3.7	-
4A	3.1 to 6.0	≤ 1.0		3.8 to 6.7	≤ 1.2
4B	3.1 to 6.0	> 1.0		3.8 to 6.7	> 1.2
5A	> 6.0	≤ 1.0		> 6.7	≤ 1.2
5B	> 6.0	> 1.0		> 6.7	> 1.2

**Table 1.9:** Modification to Table 1.8 for concrete exposed to attack from acids in natural ground (BS 5328: 1997).

pH <sup>+</sup>	Mobility of water	Change in classification with respect to minimum cement content and maximum free water/cement ratio for the cement group recommended on the basis of sulfate class in Table 1.8
5.5 to 3.6	Static	No change
	Mobile	Raise by one sulfate class
3.5 to 2.5	Static	Raise by one sulfate class
	Mobile	Raise by one sulfate class

<sup>+</sup> Determined by the method given in clause 9 of BS 1377: Part 3, 1990

### 1.3 References

- BRITISH STANDARDS INSTITUTION (1995a). *Cement - Composition, specifications and conformity criteria. Part 1. Common Cements*. DD ENV 197-1: 1995.
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## CHAPTER 2

## Minimum requirements for concrete to resist carbonation-induced corrosion of reinforcement

by D. W. Hobbs, British Cement Association

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### 2.1 Summary

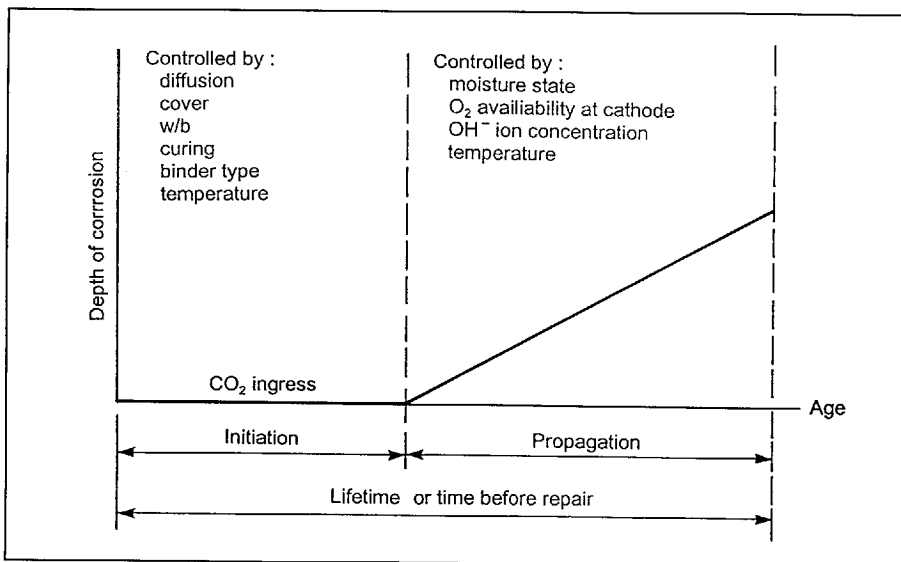
The draft European standard pr EN 206 (BSI, 1997a) classifies concretes subject to a risk of carbonation-induced corrosion according to four exposure classes:

XC1 Dry	XC2 Wet rarely dry	XC3 Moderate humidity	XC4 Cyclic wet and dry
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The object of the work described in this chapter is to propose the minimum concrete qualities for these exposure classes which are necessary for design working lives of 50 and 100 years when the minimum cover to reinforcement is 30 mm. This is the minimum cover given in DD ENV 1992-1-1 (BSI, 1992) for exposure classes judged as encompassing similar conditions to XC1 to XC4. It is assumed that the cover concrete is well compacted. The cements considered are CEM I (Portland cement) and binders containing pulverized fuel ash, slag and silica fume. The minimum concrete qualities proposed for exposure classes XC3 and XC4 are applicable to the UK. In the case of CEM I and CEM I/pfa concretes, they are based **primarily** on UK studies of the performance of concretes exposed externally after a 24-hour cure and for CEM I/slag concretes, after a three-day cure. For exposure classes XC1 and XC2, it is proposed that a nominal minimum quality of concrete will be suitable for both.

## 2.2 Introduction

When carbon dioxide from the atmosphere diffuses into concrete, it combines with water, forming carbonic acid, which then reacts with alkali hydroxides forming carbonates. In the presence of free water, calcium carbonate is deposited in the pores of the concrete at the depth at which carbonation is occurring. As a consequence of carbonation, the pH of the pore fluid drops from a value greater than 12.6, in the uncarbonated region, to a value of about 8 in the region of complete carbonation. If this reduction in alkalinity occurs close to the steel, it can cause its depassivation. In the presence of moisture and oxygen this can lead to corrosion followed by corrosion-induced cracks parallel to reinforcement, followed by spalling. Such deterioration can occur because the corrosion product which forms can be up to 2 to 4 times the volume of steel before it oxidized. A model of the effect of carbonation-induced corrosion upon service life is shown in Figure 2.1 (Tuutti, 1982). There is an initiation period, which is the period taken for the carbonation front to reach or approach the steel. This is followed by the propagation period which is the period between initiation of corrosion and cracking.



**Figure 2.1:** Corrosion model. After Tuutti (1982)

In practice, carbonation-induced corrosion is generally regarded as a minor problem compared with chloride-induced corrosion (Parrott, 1987). However, the bulk of structural concrete is used in residential, industrial and office buildings, rather than engineering structures. Consequently only a small proportion of structures are at risk due to chloride-induced corrosion. Thus the potential service life of most reinforced concrete is governed by the rate of carbonation, the cover to reinforcement and the rate of reinforcement corrosion. Most research on carbonation has been restricted to plain concrete. Relatively few studies have been made of carbonation-induced corrosion (Parrott, 1994; Dhir et al, 1992; Schiessl et al, 1994).

The draft European standard pr EN 206 classifies concretes subject to a risk of carbonation-induced corrosion according to four exposure classes (BSI, 1997a) as follows:

XC1 Dry	XC2 Wet rarely dry	XC3 Moderate humidity	XC4 Cyclic wet and dry
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Examples of concretes which fall into each of these exposure classes are given in pr EN 206: 1997 as:

- XC1 Concrete inside buildings with low air humidity.
- XC2 Parts of water-retaining structures. Many foundations.
- XC3 Concrete inside buildings with moderate or high humidity.  
External concrete sheltered from rain.
- XC4 Surfaces subject to water contact, not within exposure class XC2.

The informative examples applicable in the UK, which appear in the complementary UK concrete standard to pr EN 206 (BSI, 1997b) are as follows:

- XC1 Reinforced and prestressed concrete surfaces inside structures, except areas of structures with high humidity.
- XC2 Reinforced and prestressed concrete completely buried in non-aggressive soil.  
Reinforced and prestressed concrete surfaces permanently submerged.
- XC3/XC4 External reinforced and prestressed concrete surfaces including those sheltered from direct rain.  
Reinforced and prestressed concrete surfaces inside structures with high humidity (eg. bathrooms, kitchens).  
Reinforced and prestressed concrete surfaces exposed to alternate wetting and drying.

The object of the work described in this chapter is to propose the minimum concrete qualities for these exposure classes which are necessary for design working lives of 50 and 100 years ie. Classes 3 and 4, respectively of ENV 1991-1: 1994 (European Committee for Standardization, [1994]) when the actual minimum cover to reinforcement is 30 mm. This is the minimum cover given in DD ENV 1992-1-1: 1992 (BSI, 1992) for exposure classes judged as encompassing similar conditions to XC1 to XC4. It is assumed that the cover concrete is well compacted.

The cements considered are CEM I (Portland cement) and binders containing pulverized fuel ash and ground granulated blastfurnace slag. Binders containing silica fume are briefly dealt with in Appendix 2.1. Throughout the remainder of this chapter, pulverized fuel ash is referred to as fly ash or pfa, and ground granulated blastfurnace slag is referred to as slag. The minimum concrete qualities proposed are applicable to the UK and are based **primarily** on UK studies of the performance of concretes exposed externally after a 24-hour cure or, in the case of concretes containing more than 35% slag by mass of cement, after a three-day cure. Such differences in curing compensate for slag concrete's slower strength development (BSI, 1997c). The minimum concrete qualities proposed are not necessarily applicable to the temperatures and humidities of exposure experienced by concrete structures elsewhere in Europe.

## 2.3 Methodology

In this chapter, the limiting mix parameters for concrete quality to provide adequate resistance to carbonation-induced corrosion are based on experimental results reported in the published literature. UK literature was searched for carbonation depth data derived from experiments using realistic curing and exposure conditions. Where necessary, UK results have been supplemented by results from investigations performed overseas. The text indicates where this has been done. Results are generally available only for carbonation depths up to a few years of age. It has thus been necessary to use established relationships between carbonation depth and age to predict carbonation depths at later ages appropriate to the required design working life given in ENV 191-1: 1994 (European Committee for Standardization, 1994).

Damage due to carbonation-induced corrosion depends not only upon the carbonation front approaching or reaching the reinforcement but also upon the development of sufficient expansion due to corrosion to cause disruption of the concrete cover. Consideration of appropriate concrete qualities has thus necessarily considered rates of both carbonation and corrosion.

Various assumptions have had to be made about the carbonation and corrosion rates in reinforced concrete over long term exposure. These assumptions are detailed in the text, but have generally been conservative to allow for differences in behaviour between test specimens and concrete in structures which will generally be more variable in its properties.

Results have been analysed to find individual relationships between carbonation depth and water-binder ratio and between carbonation depth and compressive strength. Insufficient relevant information is available to establish relationships between carbonation depth and binder content. Results from concretes containing fly ash and slag have each been treated separately from those containing CEM I only.

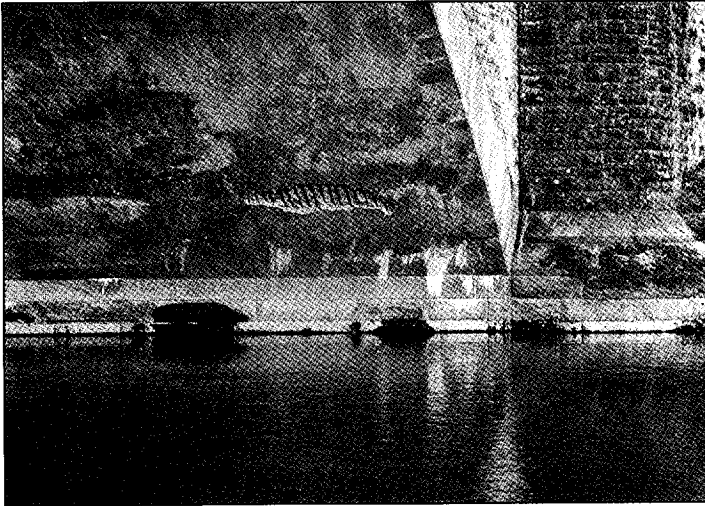
It has been assumed that concrete production and placement has followed reasonable good practice and the properties of concrete in the structure are within normal limits of variability. Nevertheless, it has been felt necessary to assume only minimal levels of curing as it is generally agreed that curing is not given adequate attention for much in-situ concrete construction. Determination of required concrete qualities has been based on minimum values of cover to reinforcement as given in DD ENV 1992-1-1 (BSI, 1992). The implications of this on specification of nominal cover are discussed in the sub-Section 2.9.5 entitled 'Discussion of the assumptions'.

## 2.4 Carbonation-induced corrosion in the UK

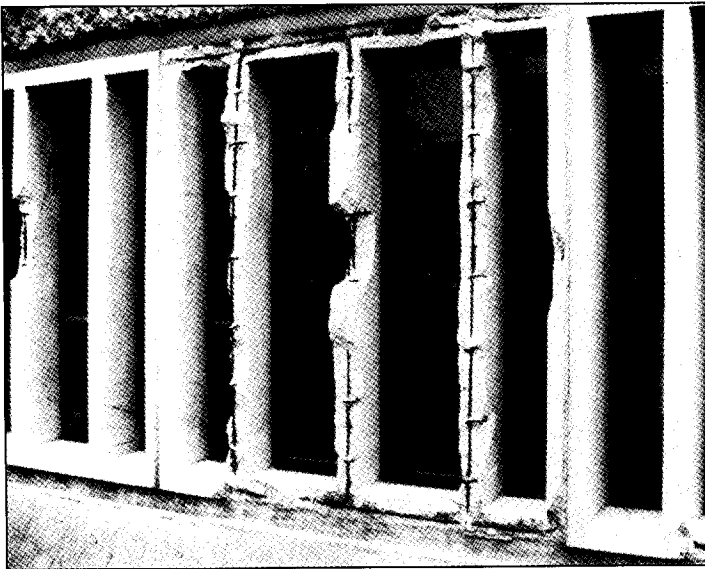
In 1987 Parrott reviewed the reported cases of reinforced concrete members in the UK which had deteriorated due to carbonation-induced corrosion. These included:

- Bridges where corrosion had occurred on soffits, parapets and on stirrups of beams.
- Prefabricated houses built in the 1940s and 1950s.
- A number of industrialized and system-built dwellings dating from the 1960s and 1970s.

Examples of deterioration due to carbonation-induced corrosion are shown in Figures 2.2 and 2.3.



**Figure 2.2:** An example of deterioration in a bridge caused by carbonation-induced corrosion.



**Figure 2.3:** An example of deterioration in a building caused by carbonation-induced corrosion.

## 2.5 Measurement of carbonation depth

The depth of the carbonation front in concrete can be measured in a variety of ways: by X-ray diffraction, infra-red spectroscopy, microscopy, thermogravimetric analysis, chemical analysis or the phenolphthalein test.

The phenolphthalein test is the simplest and most popular test, and is carried out by spraying a freshly split surface of the concrete with a mixture of phenolphthalein, water and alcohol. This pH indicator changes from colourless to magenta as the pH rises from 8.3 to 10.0. On a broken

concrete surface the change in colour becomes readily visible at a pH greater than about 9.0 to 9.5. The test indicates the depth to which the calcium hydroxide has been depleted. The phenolphthalein indicator test is carried out on a split surface of concrete (rather than on a cut surface or on drillings) as this minimises the movement of calcium hydroxide into the carbonated zone.

In most of the papers referred to in this chapter, the depth of carbonation was measured using the phenolphthalein test.

## 2.6 Concretes subject to moderate humidity (XC3)

For concretes exposed to a moderate humidity, the highest risk of carbonation-induced corrosion exists for external exposed concrete sheltered from rain. This section deals with the reported data for this particular exposure regime.

### 2.6.1 CEM I concretes (PC)

For concretes subject to sheltered external exposure in the UK, the average long-term relative humidity to which the concretes are exposed is about 80 to 85% (Parrott, 1995). At a humidity of 80%, Parrott has deduced that the depth of carbonation,  $d_c$ , is related to concrete age,  $t$ , by the expression

$$d_c \propto t^{0.4} \quad \dots \dots \dots \quad (2.1)$$

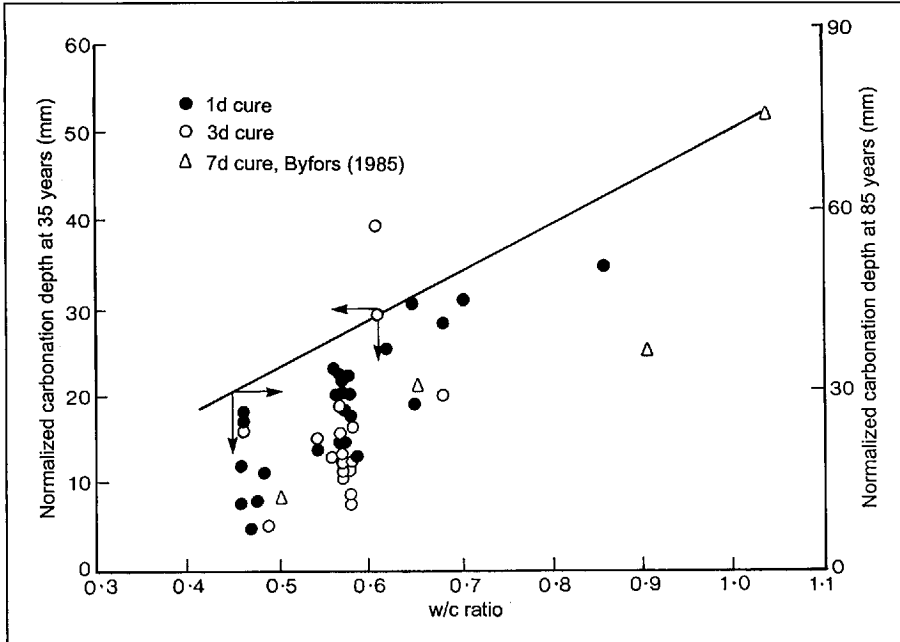
Assuming that corrosion commences when  $d_c$  equals the minimum reinforcement depth, Parrott also deduced that when concrete is subject to long-term exposure at 80% RH, cracking of the concrete, initiated by carbonation-induced corrosion, will commence approximately 20 years after the carbonation front reaches the steel. Data reported by Brown (1987) broadly supports this deduction. Brown examined seven bridges which ranged in age from 51 to 55 years. In nine of the elements examined, carbonation-induced cracking had occurred. The mean ratio of carbonation depth to cover was 1.24 which, from the expression

$$(t_1/t_2)^{0.4} = 1.24 \quad \dots \dots \dots \quad (2.2)$$

gives an upper bound for the corrosion period of 22 years (taking  $t_1$  to be 53 years gives  $t_2$  the age at which the carbonation front reaches the steel as 31 years). In the work described in the current section of this chapter, a more conservative approach than deduced by Parrott is adopted, namely that cracking will be induced 15 years after the carbonation front reaches the steel. Thus for design lives of 50 and 100 years, the carbonation depth should not exceed 30 mm at 35 and 85 years respectively.

Several investigators in the UK have measured the depth of carbonation into laboratory-produced CEM I concretes subject to sheltered external exposure (Hobbs, 1988; Thomas and Matthews, 1992; Parrott, 1990; Barker and Matthews, 1994; Matthews, 1994). Additional unpublished data obtained by BCA and BRE are summarized in Appendix 2.2 (Hobbs, 1995; Matthews, 1995; Brown and Beeby, 1992). The carbonation depths obtained are shown plotted against free water-cement ratio in Figure 2.4, together with some results obtained on concretes exposed to an RH of 80% (Byfors, 1985). The carbonation depths plotted in Figure 2.4 have been normalized to ages of 35 and 85 years using equation 2.1. For concretes subject to a one-day cure, examination of Figure 2.4 indicates that, on the basis of the experimental data, maximum free water-cement ratios of 0.61 and 0.45 should ensure that the depth of carbonation is unlikely to





**Figure 2.4:** Dependence of carbonation depth upon free w-c ratio. CEM I concretes. Sheltered external exposure. UK data.

exceed 30 mm until after ages of 35 and 85 years respectively. Data leading to a similar conclusion has been obtained by overseas investigators (Figure 2.5, Byfors, 1985; Wierig, 1984; Hakkinen, 1993; Fagerlund, 1988; Meyer, 1968).

In Figure 2.6 the measured depths of carbonation into sections of seven UK bridges, are shown plotted against deduced free water-cement ratio (Brown, 1987). The ages of the bridges ranged from 51 to 55 years and the elements were subject to class XC3 or XC4 exposure. The free water-cement ratio was deduced from the capillary porosity and cement content and, according to Brown (1987), has an accuracy of about  $\pm 0.1$ . Also shown plotted in Figure 2.6 is the upper bound curve taken from Figure 2.4, normalized to an age of 53 years. The normalized curve is a reasonable upper bound to the carbonation depths measured by Brown.

In Figure 2.7 the depth of carbonation measured by UK investigators is shown plotted against the 28-day cube compressive strength (Hobbs, 1988; Thomas and Matthews, 1992; Parrott, 1990; Barker and Matthews, 1994; Matthews, 1994, 1997; Hobbs, 1997; Brown and Beeby, 1992). For concretes cured for one day and ignoring one outlier, minimum concrete strengths of 36 and 52 N/mm<sup>2</sup> should ensure that, on the basis of the experimental data, the depth of carbonation is unlikely to exceed 30 mm until after ages of 35 and 85 years respectively. If the outlier is included, then these strengths become 45 and 54 N/mm<sup>2</sup>.

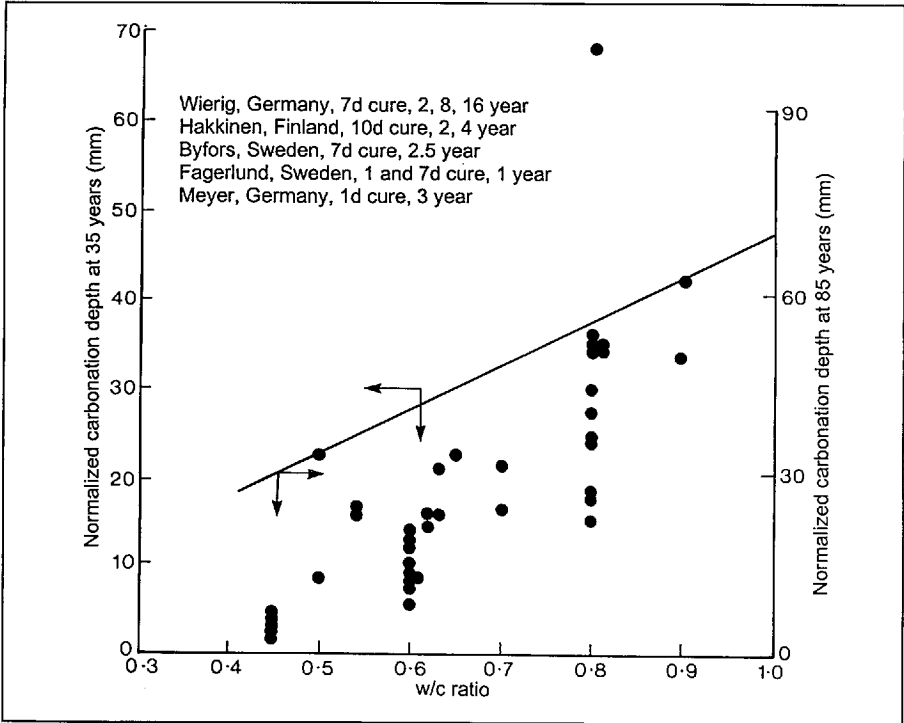


Figure 2.5: Dependence of carbonation depth upon w-c ratio. CEM I concretes. Sheltered external exposure. Non-UK data.

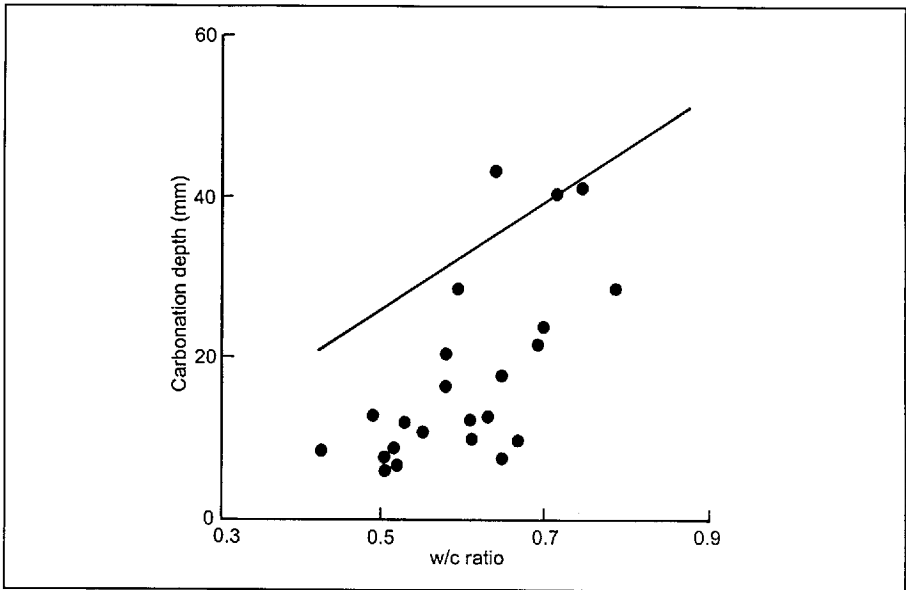
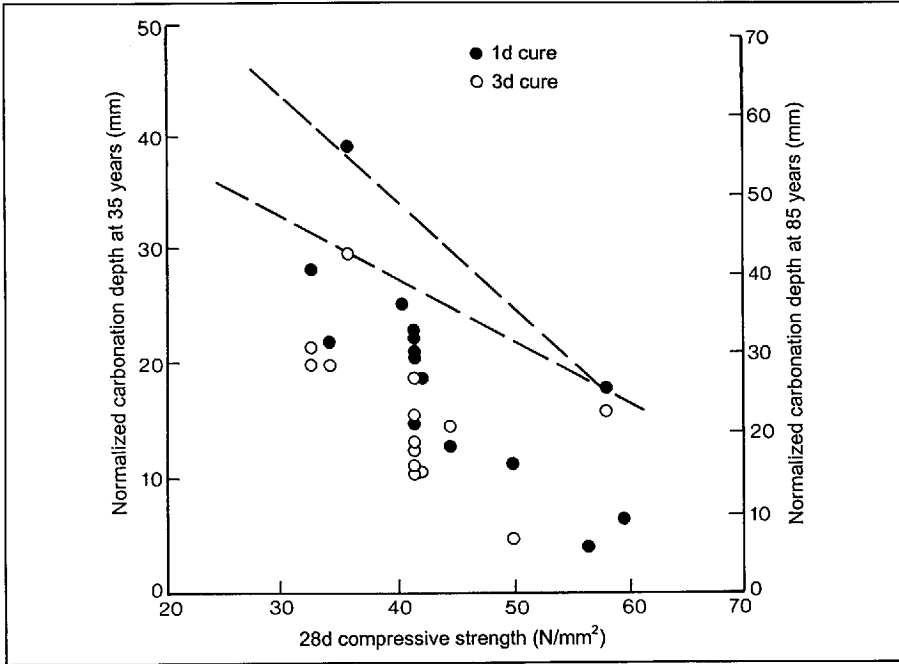


Figure 2.6: Dependence of carbonation depth into sections of seven UK bridges upon deduced water-cement ratio. Ages 51 to 55 years. CEM I concretes.



**Figure 2.7:** Dependence of carbonation depth upon 28d compressive strength. CEM I concretes. Sheltered external exposure. UK data.

**2.6.2 CEM I/fly ash concretes**

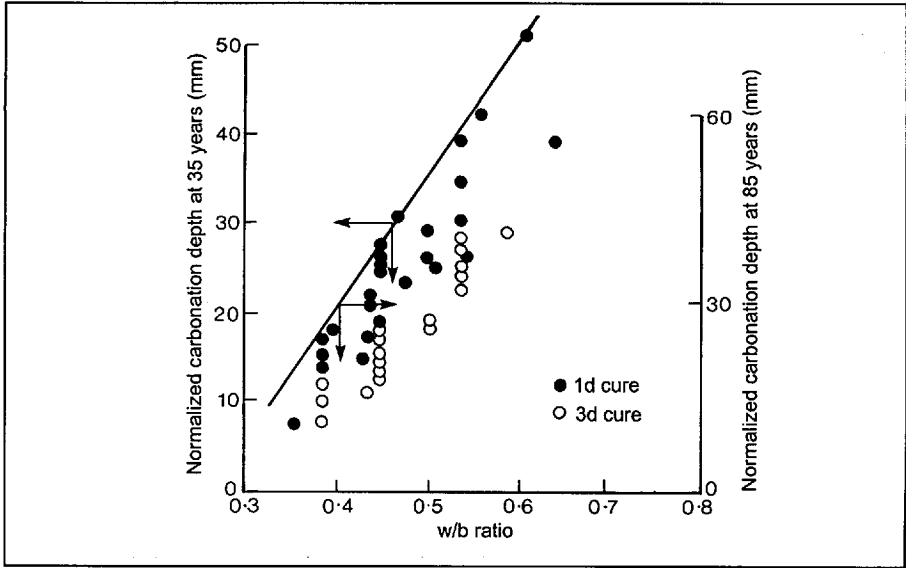
Carbonation data obtained by UK investigators on CEM I/fly ash concretes subject to sheltered external exposure are plotted against free water-binder ratio in Figures 2.8, 2.9 and 2.10 (Hobbs, 1988; Thomas and Matthews, 1992; Parrott, 1990; Barker and Matthews, 1994; Matthews, 1994, 1995; Hobbs, 1995; Brown and Beeby, 1992). Figure 2.8 plots results for concretes containing 30 or 35% fly ash, Figure 2.9a for 0, 30 and 50% fly ash and Figure 2.10 for 50% fly ash. Results obtained by overseas investigators (Byfors, 1985; Fagerlund, 1988) on concretes containing 30% fly ash are shown in Figure 2.11. As in the previous section, the depths of carbonation which are plotted in Figures 2.8, 2.10 and 2.11 have been normalized to ages of 35 and 85 years.

A comparison of Figures 2.8, 2.9a and 2.10 with Figures 2.4 and 2.5 shows that, based on free water-binder ratio, the resistance to carbonation of CEM I and CEM I/fly ash concretes is different. Their differing effects can be estimated by expressing the depth of carbonation at a given age in terms of a water-effective-cement ratio, as follows:

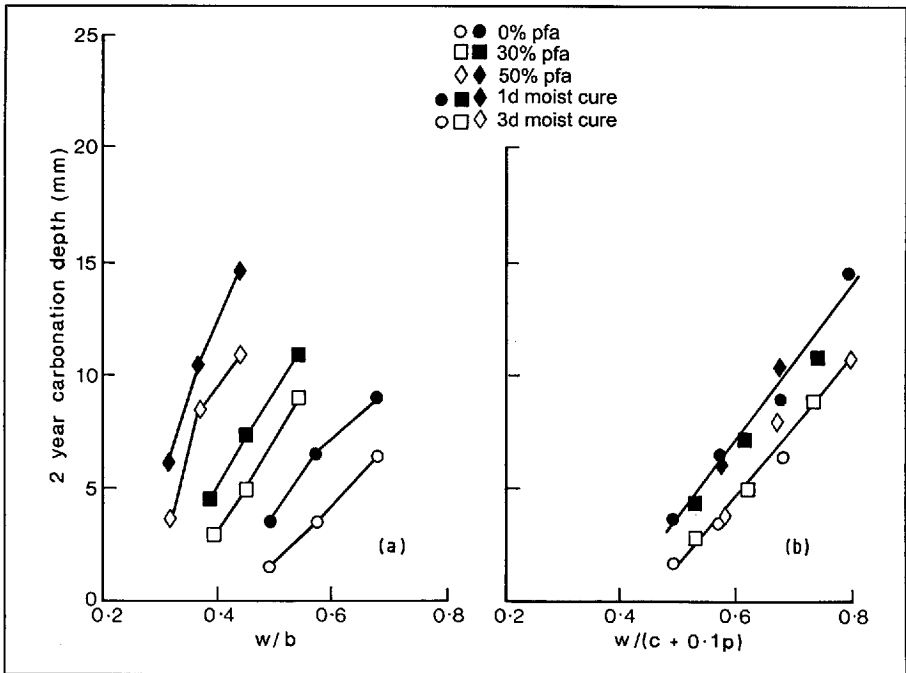
$$d_c = \phi (w/[c + kp]) \dots\dots\dots (2.3)$$

where p is the mass of fly ash and k, the cementing efficiency factor, is the mass of CEM I cement which gives a similar resistance to carbonation as a unit mass of fly ash. In the case of data published by Thomas and Matthews (1992), the depths of carbonation are similar in CEM I concretes and CEM I/fly ash concretes when

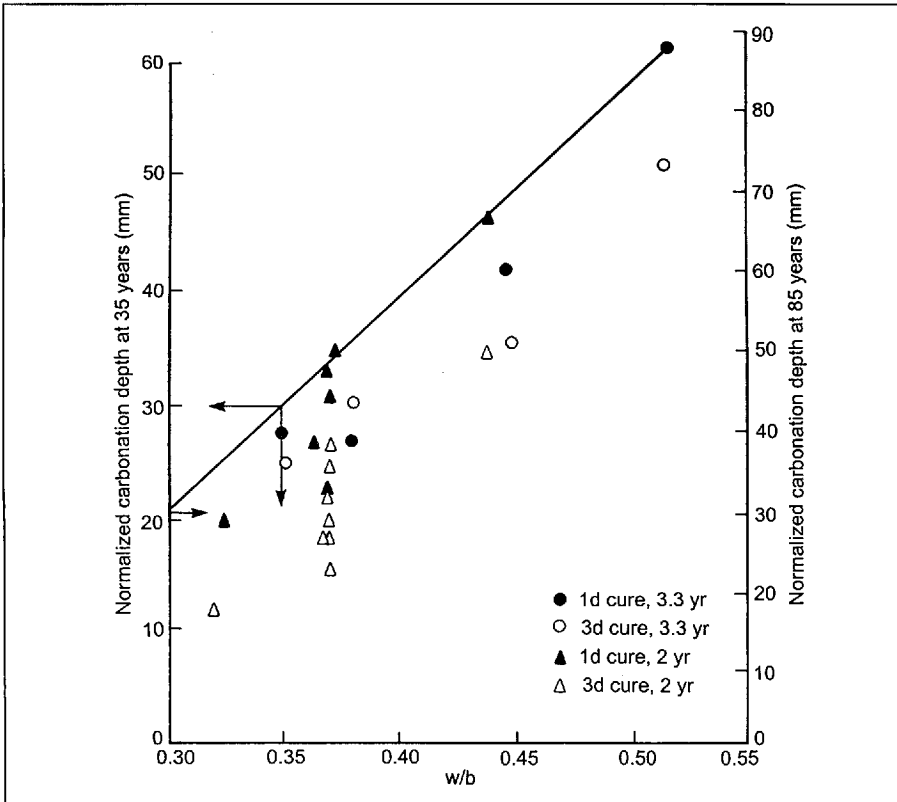
$$(w_1/c_1)_{CEM1} = [w_2/(c_2 + 0.1p)]_{pfa} \dots\dots\dots (2.4)$$



**Figure 2.8:** Dependence of carbonation depth upon free w/b ratio. 30 - 35% fly ash. Sheltered external exposure. UK data.



**Figure 2.9 a & b:** Dependence of carbonation depth on free w/b and free  $w/(c + 0.1p)$  for concretes subject to sheltered external exposure (after Thomas and Matthews, 1992).

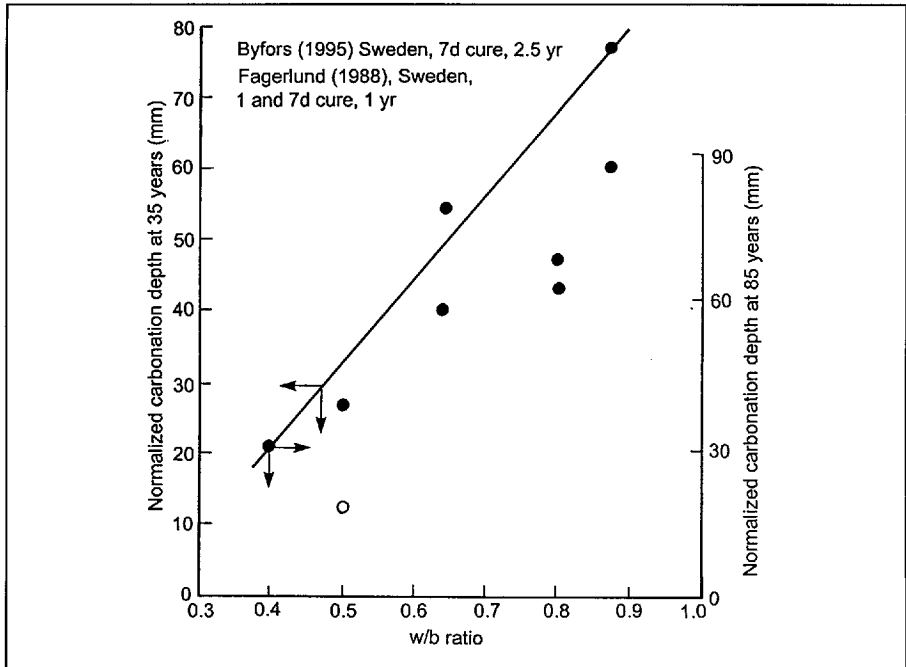


**Figure 2.10:** Dependence of carbonation depth upon free w-b ratio. 50% fly ash. Sheltered external exposure. UK data.

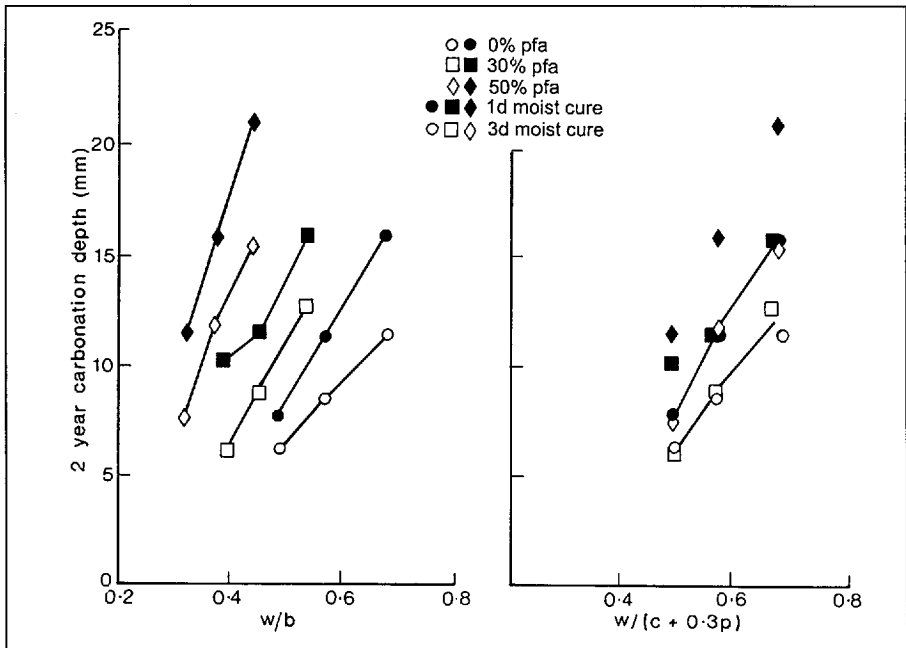
ie. when the  $k$  value is 0.1 (Figure 2.9b). Carbonation data obtained on concretes containing 30% fly ash subject to laboratory exposure (Hobbs, 1988; Thomas and Matthews, 1992) suggest a higher value for  $k$  (see Figure 2.12a & b).

Examination of Figure 2.8 shows that, for a concrete containing 30% fly ash subject to a one-day cure, maximum free water-binder ratios of 0.46 and 0.40 should ensure that the depth of carbonation is unlikely to exceed 30 mm until after 35 and 85 years respectively. Analysis of data from overseas investigations, as shown in Figure 2.11, leads to similar maximum values for water-binder ratios. For concretes containing 50% fly ash, the maximum free water-binder ratios are 0.35 and 0.28 respectively (see Figure 2.10).

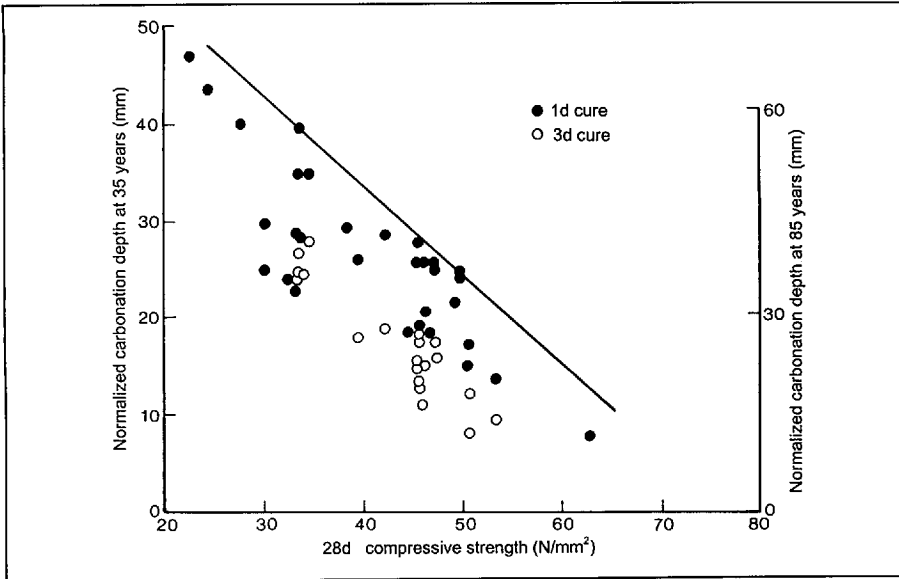
In Figures 2.13 and 2.14 the carbonation data obtained by UK investigators, normalized to ages of 35 and 85 years, is shown plotted against 28-day compressive strength for concretes containing 30 to 35% and 50% fly ash respectively (Hobbs 1988; Thomas and Matthews, 1992; Parrott, 1990; Barker and Matthews, 1994; Matthews, 1994, 1995; Hobbs, 1995; Brown and Beeby, 1992). Examination of these figures shows that, to ensure that the carbonation depth does not exceed 30 mm at 35 and 85 years, minimum 28-day compressive strengths of 44 and 54 N/mm<sup>2</sup> respectively are required for concretes containing 30% fly ash, subject to a one-day cure, and 49 and 58 N/mm<sup>2</sup> respectively for concretes containing 50% fly ash.



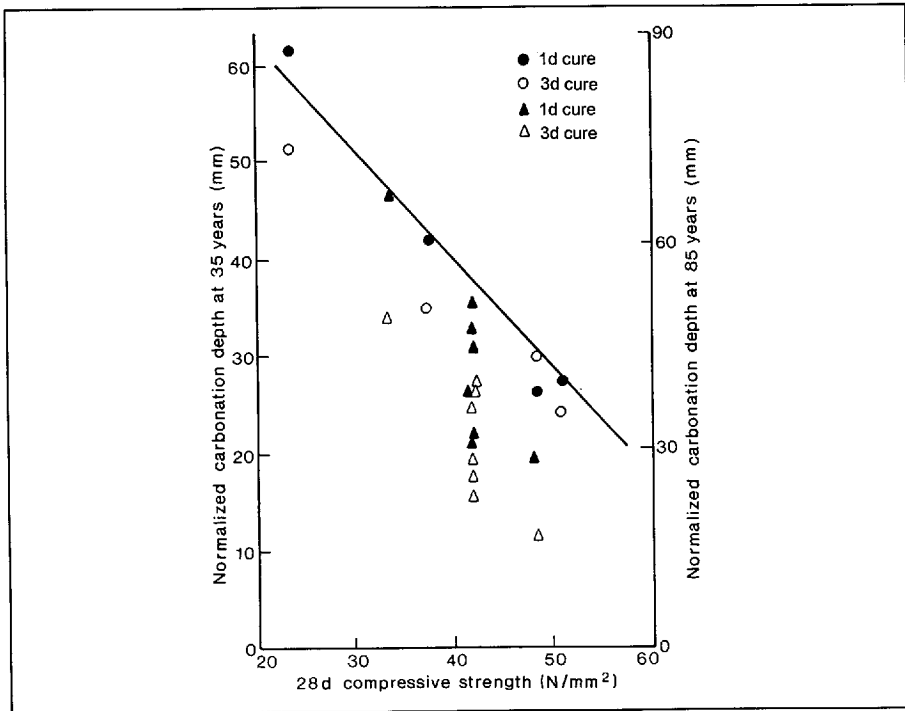
**Figure 2.11:** Dependence of carbonation depth upon w-b ratio. 30% fly ash. Sheltered external exposure. Non-UK data.



**Figure 2.12 a & b:** Dependence of depth of carbonation on free w-b and free w/(c + 0.3p) for concretes stored at 20°C, 65% RH. (After Thomas and Matthews, 1992).



**Figure 2.13:** Dependence of carbonation depth upon 28-day compressive strength, 30-35% fly ash. Sheltered external exposure. UK data.

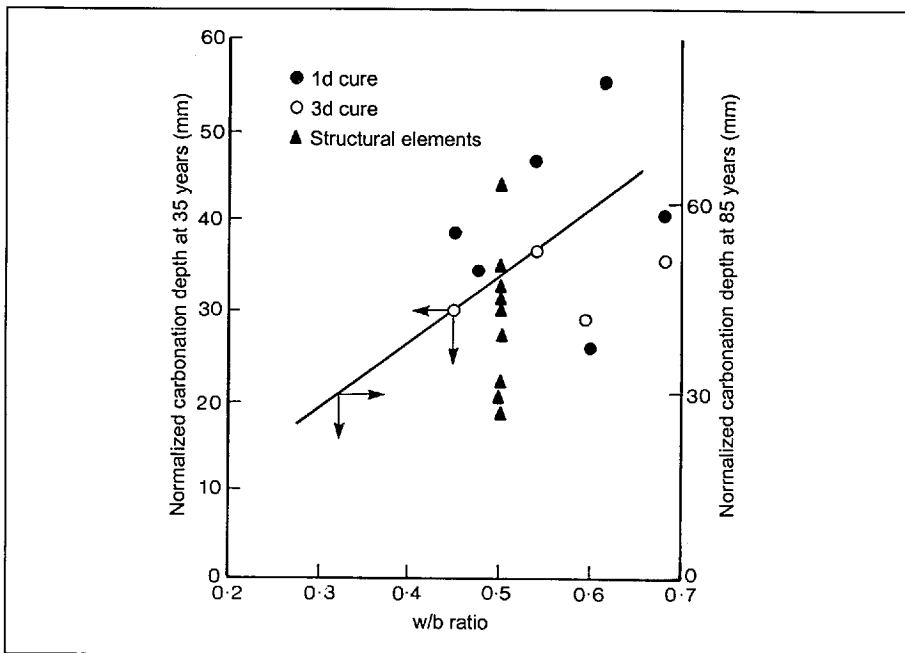


**Figure 2.14:** Dependence of carbonation depth upon 28d compressive strength. 50% fly ash. Sheltered external exposure. UK data. (After Thomas and Matthews, 1992; Hobbs, 1995, Appendix 2.2).

### 2.6.3 CEM I/slag concretes

Carbonation data obtained by UK investigators on CEM I/slag concretes with slag contents of 50 and 70% subject to sheltered external exposure are plotted against free water-binder ratio in Figure 2.15 (Parrott, 1990; Barker and Matthews, 1994; Hobbs, 1995; Matthews, 1995; Osborne, 1994). Also included in Figure 2.15 are data obtained from field structural elements (Osborne, 1994). Results obtained by overseas investigators are shown plotted against water-binder ratio in Figure 2.16 (Wierig, 1984; Hakkinen, 1993; Meyer, 1968; Kleinschmidt, 1965). As in the previous sections, the depths of carbonation are normalized to ages of 35 and 85 years. In Figure 2.17 the depth of carbonation determined by UK investigators is plotted against 28-day cube compressive strength together with data obtained on companion CEM I concretes (Barker and Matthews, 1994; Hobbs, 1995; Matthews, 1995; Brown and Beeby, 1992).

A comparison of Figures 2.15 and 2.16 with Figures 2.4 and 2.5, and of Figure 2.17 with Figure 2.7, shows that, based on both water-binder ratio or 28-day compressive strength, the resistance to carbonation of CEM I and CEM I/slag concretes is different, irrespective of whether the CEM I/slag concretes are cured for one or three days. However, the data obtained on CEM I/slag concretes by UK investigators is less comprehensive than that obtained on CEM I and CEM I/pfa concretes. Examination of Figure 2.15 for a concrete containing 70% slag by mass of binder, subject to a three-day cure, indicates that, on the basis of the experimental data, maximum free water-binder ratios of 0.45 and 0.32 may ensure that the depth of carbonation is unlikely to exceed 30 mm until after ages of 35 and 85 years respectively. This compares closely with water-binder ratios of 0.45 and 0.34 deduced from the overseas data shown in Figure 2.16. However, the validity of these values may be questionable firstly, because of the narrow range of free



**Figure 2.15:** Dependence of carbonation depth upon w-b ratio. 50 and 70% slag. Sheltered external exposure. UK data.



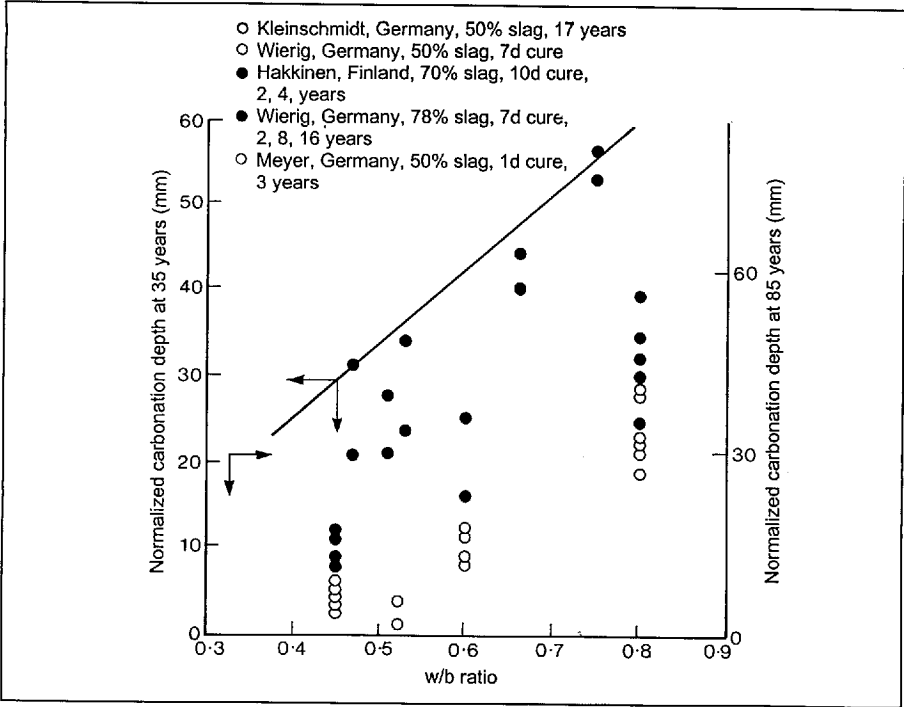


Figure 2.16: Dependence of carbonation depth upon w-b ratio. 50 and 70% slag. Sheltered external exposure. Non-UK data.

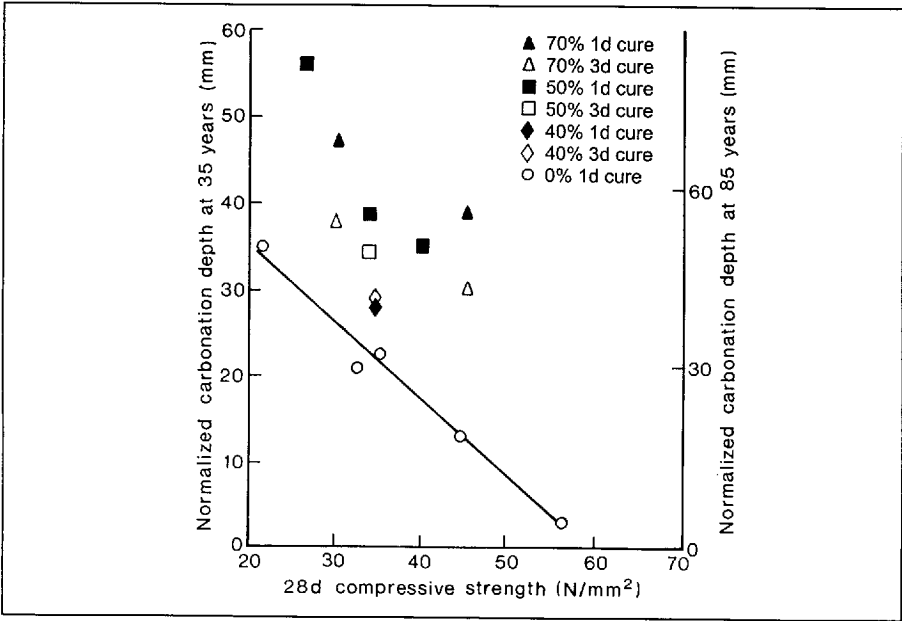


Figure 2.17: Dependence of carbonation depth upon 28d compressive strength. CEM I/slag concretes. Sheltered external exposure. UK data.

water-binder ratios covered in the UK work and secondly, because in the overseas work many of the concretes tested were cured for seven or ten days prior to exposure.

Estimates of the possible maximum free water-binder ratios necessary to ensure that the depth of carbonation does not exceed 30 mm until after ages of 35 and 85 years for concretes containing less (or more) than 70% slag can be made using the k-factor approach.

From the limited data shown in Figure 2.15, it has been deduced that concretes, cured for three days, with 70% slag and water-binder ratios of 0.45 and 0.32 give equivalent performance to CEM I concretes, cured for one day, with free water-cement ratios of 0.61 and 0.45 respectively. Comparing these slag concretes and CEM I concretes indicates a k-factor in the range 0.59 to 0.62<sup>‡</sup>. Assuming a k-factor of 0.60 and a slag content of 40% gives water-binder ratios of 0.51 and 0.38 as being equivalent to water-CEM I cement ratios of 0.61 and 0.45 respectively.

In the case of compressive strength, the data is insufficient to enable estimates to be made of the minimum 28-day compressive strengths necessary to ensure that the carbonation depth does not exceed 30 mm at 35 or 85 years. At equal 28-day compressive strengths, the limited data indicate that the carbonation depth in CEM I/slag concretes, subject to a one- or three-day cure, is greater than the carbonation depth in CEM I concretes subject to a one-day cure.

#### 2.6.4 Minimum specification for concrete subject to exposure class XC3

Table 2.1 gives the minimum qualities for concretes subject to class XC3 exposure necessary for design lives of 50 and 100 years, when the minimum cover to reinforcement is 30 mm. For CEM I and CEM I/pfa concretes, the minimum qualities assume a one-day cure and for the CEM I/slag concretes, a three-day cure.

**Table 2.1:** Minimum concrete qualities from experimental data. Exposure class XC3.

Concrete mix details	Design life (years)	Moderate humidity - XC3				
		CEM I	CEM I/30 (% fly ash)	CEM I/50 (% fly ash)	CEM I/40 (% slag)	CEM I/70 (% slag)
Minimum compressive strength (N/mm <sup>2</sup> )	50	36 (45) <sup>+</sup>	44	49	(45) <sup>§</sup>	
Maximum w-b		0.61	0.46	0.35	0.51	0.45
Minimum compressive strength (N/mm <sup>2</sup> )	100	52 (54) <sup>+</sup>	54	58	(60) <sup>§</sup>	
Maximum w-b		0.45	0.40	0.28	0.38	0.32

<sup>+</sup> When outlier included.

<sup>§</sup> Approximate 28d compressive strength.

<sup>‡</sup> For a one-day cure, the k-factor for slag is approximately 0.4.

## 2.7 Concretes subject to cyclic wetting and drying (XC4)

### 2.7.1 CEM I concretes

Due to the higher moisture content of concrete subject to unsheltered external exposure in the UK, the rate of carbonation will be slower than for similar concrete subject to sheltered external exposure. However, if the carbonation front reaches the steel, and corrosion is consequently initiated, it will occur more rapidly. The corrosion rate, CR, the steel reinforcement corrosion period until cracking,  $t_p$ , and the power  $n$  in the generalised form of equation (2.1), ie.

$$d_c \propto t^n \quad \dots\dots\dots (2.5)$$

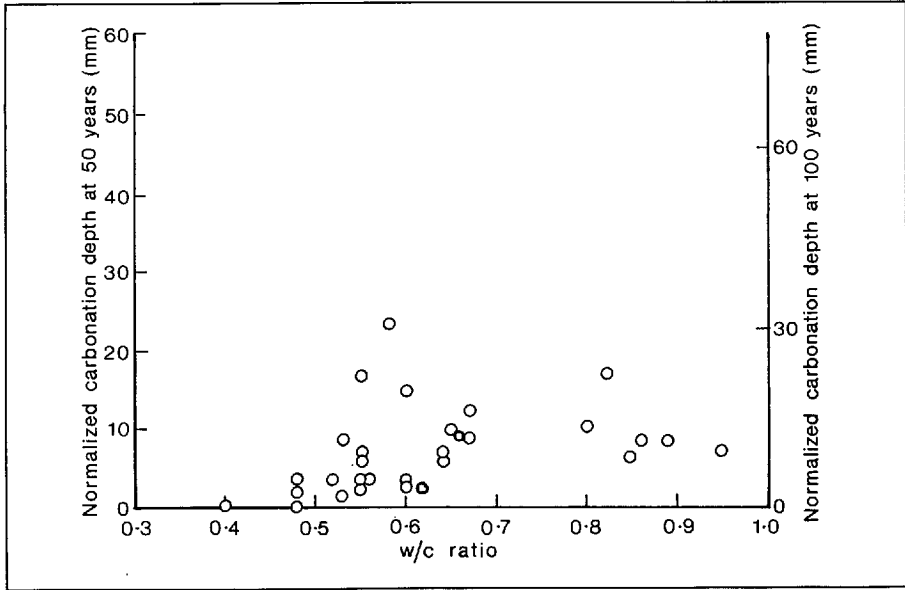
depend upon relative humidity. The estimated values have been determined by Parrott (1995) and are given in Table 2.2, again assuming that corrosion commences when the carbonation depth,  $d_c$ , equals the minimum reinforcement depth.

**Table 2.2:** Values of CR ( $\mu\text{m}/\text{yr}$ ),  $t_p$  (yr) and the power  $n$ .

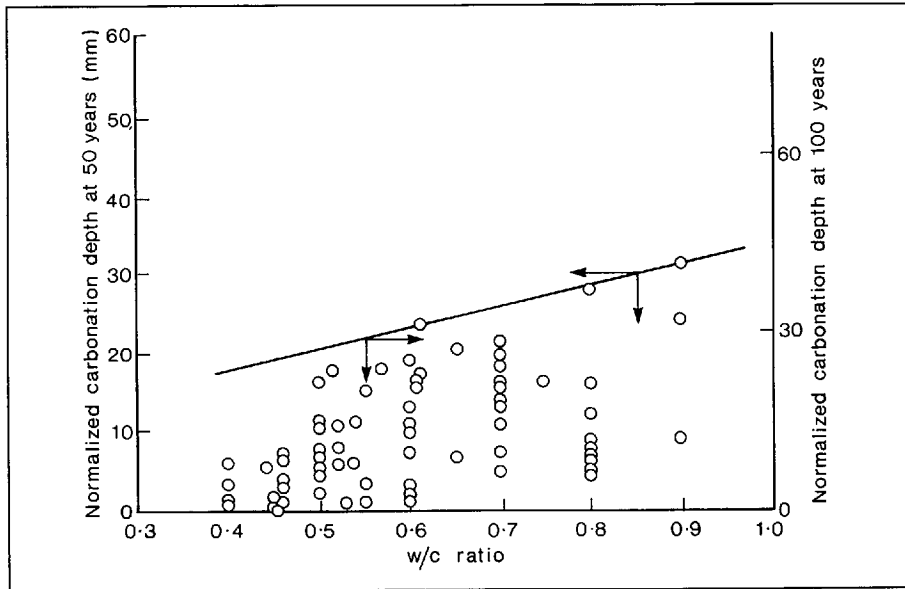
RH (%)	80	90	95	98	100
CR	5	10	20	50	10
$t_p$	20	10	5	2	10
$n$	0.415	0.317	0.256	0.216	0.187

For concretes subject to unsheltered external exposure, the rates of carbonation and subsequent corrosion rates are likely to be greatly influenced by the local microclimate, particularly with respect to moisture condition, and for the purposes of design, onerous values for  $n$  and  $t_p$  should be selected. In the work described in this section of this chapter, it is assumed that  $t_p$  is zero. Thus for a design life of 50 years, the carbonation depth should not exceed 30 mm at 50 years. To normalize the reported carbonation data to a common age of 50 or 100 years, an onerous upper bound value for  $n$  of 0.4 has been chosen (see Table 2.2).

A number of investigators in the UK have measured the depth of carbonation into externally exposed CEM I concretes unprotected from rain (Osborne, 1989; Matthews, 1984, 1995; Baweja et al, 1987; Dunster 1991/92; Thomas and Matthews, 1991; Thomas et al, 1990; Bamforth, 1997). The carbonation depths, normalized to ages of 50 and 100 years are plotted against free water-cement ratio in Figure 2.18. A comparison of the data in this figure with that in Figure 2.4 indicates substantially lower carbonation depths for exposed concretes than for those sheltered from the elements. Similar results have been obtained by overseas investigators (compare Figure 2.19 with Figures 2.4 and 2.5), (Wierig, 1984; Fagerlund, 1988; Nagataki et al, 1986; Yoda, 1982; Kokubu and Nagataki, 1989; Frey, 1993; Nishi, 1962; Hamada, 1969). It follows that XC4 exposure is less severe than XC3 exposure, despite the onerous assumptions that  $t_p$  was zero and the power  $n$  was 0.4. Examination of Figure 2.18 does not show a clear dependence of the carbonation depth upon free water-binder ratio. Even at a high free water-cement ratio of 0.9, it is deduced that the carbonation depth is unlikely to exceed 30 mm at 100 years. However, overseas data do indicate that carbonation depths could exceed 30 mm at 100 years. Examination of Figure 2.19 indicates that on the basis of the overseas experimental data, maximum water-cement ratios of 0.85 and 0.55 should ensure that the depth of carbonation is unlikely to exceed 30 mm until after ages of 50 and 100 years respectively.



**Figure 2.18:** Dependence of carbonation depth upon w-c ratio. CEM I concretes. External exposure. UK data.

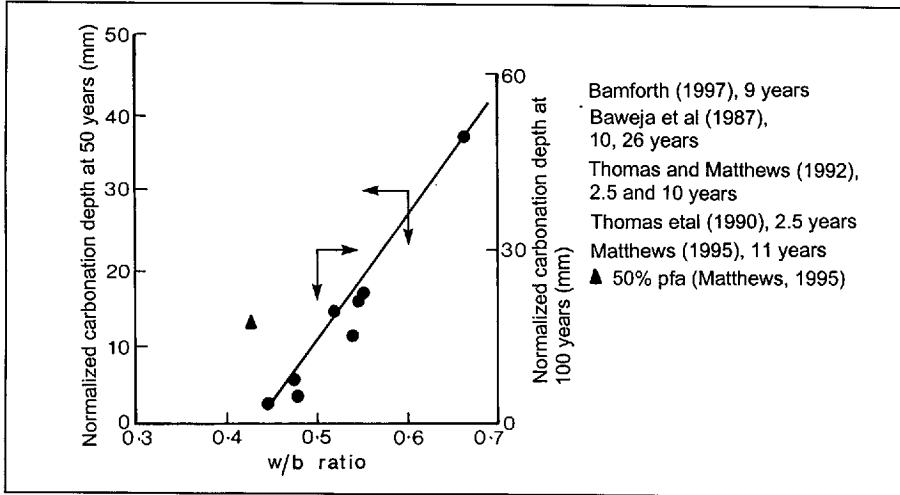


**Figure 2.19:** Dependence of carbonation depth upon w-c ratio. CEM I concretes. External exposure. Non-UK data.

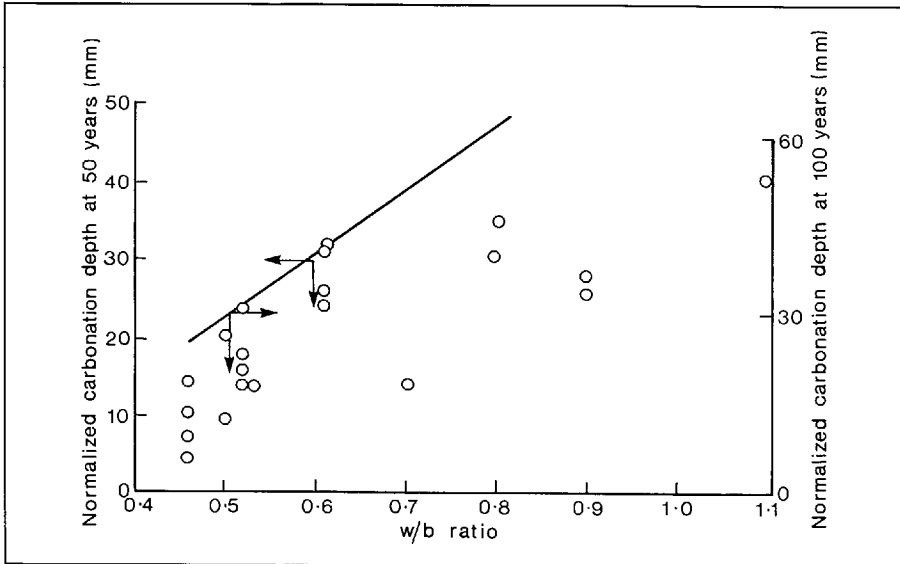
### 2.7.2 CEM I/fly ash concretes

Carbonation data obtained by UK investigators (Bamforth, 1997; Thomas and Matthews, 1992; Baweja et al, 1987; Thomas et al, 1990; Matthews, 1995) and overseas investigators (Nagataki

et al, 1986; Fagerlund, 1988; Kokubu and Nagataki, 1989; Baweja et al, 1987) on CEM I/fly ash concretes subject to external exposure are plotted against water-binder ratio in Figures 2.20 and 2.21 respectively. A comparison of these figures with Figures 2.18 and 2.19 shows that, based on water-binder ratio, the resistance to carbonation of CEM I and CEM I/fly ash concretes is different. For a concrete containing 30% fly ash by mass of binder, examination of both Figures 2.20 and 2.21 shows that, on the basis of the experimental data, maximum water-binder ratios of 0.60 and 0.50 should ensure a minimum design life of 50 and 100 years respectively. The



**Figure 2.20:** Dependence of carbonation depth upon w-b ratio. 24 to 30% fly ash. External exposure. UK data.



**Figure 2.21:** Dependence of carbonation depth upon w-b ratio. 26 to 30% fly ash. External exposure. Non-UK data.

arrows on Figures 2.20 and 2.21 indicate that these water-binder ratios are safe both for UK and overseas exposure conditions.

### 2.7.3 CEM I/slag concretes

Carbonation data obtained by Osborne (1989) on CEM I/slag concretes subject to external exposure are shown plotted against free water-binder ratio in Figure 2.22a. It should be noted that the data are very limited. An examination of Figure 2.22a shows that the resistance to carbonation of CEM I and CEM I/slag concretes is different. For example, concretes containing 40 and 70% slag and with free water-binder ratios of approximately 0.60 to 0.70 and 0.40 to 0.45 respectively, give equal carbonation performance to a CEM I concrete with a free water-cement ratio of 0.85. For CEM I concretes with a free water-binder ratio of 0.55, the comparable free water-binder ratios are in the ranges 0.40 to 0.45 and 0.35 to 0.40 respectively. For concretes subject to XC4 exposure, this indicates a k-factor for slag in the range 0.30 to 0.45 (see Figure 2.22b). This range of k-factors is surprising, as higher values than those for exposure class XC3 would be expected.

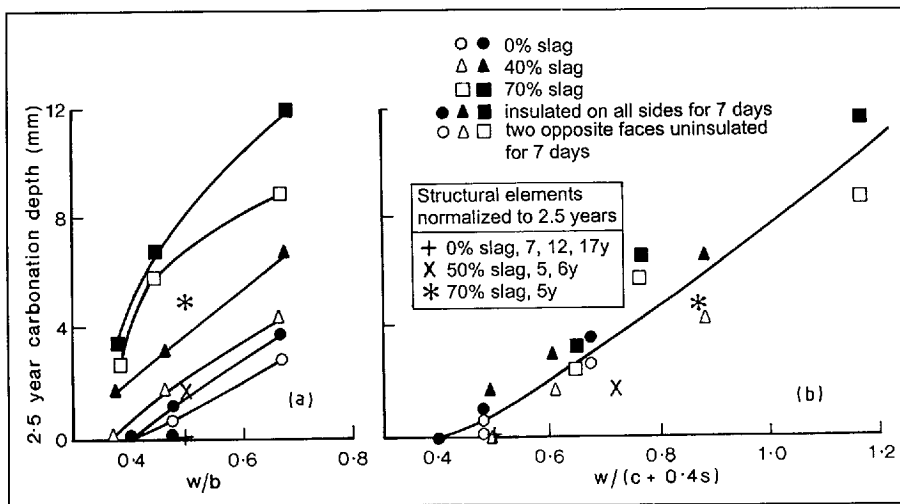


Figure 2.22a & b: Dependence of depth of carbonation on free w-b ratio and  $w/(c + 0.45s)$  ratio for CEM I and CEM I/slag concretes stored externally for 2.5 years. 300 mm cubes top face.

### 2.7.4 Minimum specification for concrete subject to exposure class XC4

From the previous sub-Sections, the maximum free water-binder ratios necessary for design lives of 50 and 100 years, when the minimum cover to reinforcement is 30 mm, have been deduced for concretes subject to exposure class XC4. The maximum free water-binder ratios correspond in some instances to concrete grades which are lower than the minimum specified for reinforced concrete, namely C24/30. Table 2.3 gives the minimum concrete qualities required for design lives of 50 and 100 years, when the concrete is subject to exposure class XC4.

**Table 2.3:** Minimum concrete qualities from experimental data. Exposure class XC4.

Concrete mix details	Design life (yrs)	Cyclic wet and dry			
		CEM I	CEM I/30 (%) fly ash	CEM I/40 (%) slag	CEM I/70 (%) slag
Maximum w/b	50	0.85	0.60	0.60 to 0.70	0.40 to 0.45
Maximum w/b	100	0.55	0.50	0.40 to 0.45	0.35 to 0.40

## 2.8 Minimum specification for concrete subject to exposure classes XC1 and XC2

For concretes subject to exposure class XC1 (dry), carbonation will occur but carbonation-induced corrosion is unlikely. For concretes subject to exposure class XC2 (rarely dry), little carbonation will occur. Consequently a nominal minimum quality of concrete will be suitable for both of these exposure classes.

## 2.9 Discussion

### 2.9.1 Minimum binder content

The binder content of concrete is generally dependent upon the method of mix design employed rather than on a judgement of the minimum amount required to provide a given performance. Little information is available within published literature on the dependence of carbonation rate upon binder content independent of free water-binder ratio. In the absence of any admixtures, the binder content required at a given free water-binder ratio or compressive strength is dependent upon the water demand of the concrete mix to provide the required level of workability. In practice, water-reducing admixtures are often employed to modify the water demand and thus provide the required level of workability at reduced binder contents. The majority of concretes used in the experiments from which the data in this chapter have been obtained, contained no water-reducing admixtures. Binder contents employed may thus have been greater than may actually have been required. The binder contents of the concretes tested which corresponded most closely to the concretes given in Table 2.1 are broadly as follows:

CEM I	c 280 kg/m <sup>3</sup> for w/c 0.60	c 370 kg/m <sup>3</sup> for w/c 0.45
CEM I/30 % pfa	c 350 kg/m <sup>3</sup> for w/c 0.45	c 400 kg/m <sup>3</sup> for w/c 0.40
CEM I/50 % pfa	c 400 kg/m <sup>3</sup> for w/c 0.35	
CEM I/50 % slag	c 360 kg/m <sup>3</sup> for w/c 0.50	
CEM I/70 % slag	c 360 kg/m <sup>3</sup> for w/c 0.50	

where c is the cement or binder content (kg/m<sup>3</sup>).

Recommendations for minimum binder contents given in current British Standards, such as BS 8110 for the structural use of concrete, are generally high compared with many national standards in other CEN (European Committee for Standardization member states). Nevertheless, little information is available on the performance, under UK conditions, of concretes containing lower binder contents than those recommended by British Standards. The experimental results reviewed in this chapter provide no empirical or scientific basis for suggesting minimum binder contents. It has thus not been felt possible to give guidance on minimum binder content in this chapter.

### **2.9.2 Influence of curing**

The relationship between duration of curing and rate of carbonation is not well established for curing under site conditions. Nevertheless, studies have generally shown that carbonation rates for a given type of concrete are lower for well cured specimens than for poorly cured specimens. It is generally recognized that the duration of effective curing achieved in practice is very variable. As stated in the introduction, this chapter has been based primarily upon data obtained from CEM I and CEM I/pfa concretes exposed after one day of curing and CEM I/slag concretes after three days of curing. This may be a conservative assumption, and carbonation depths for concretes that have been subject to longer periods of effective curing will be lower than predicted. The effect of initial curing may be less significant for external sheltered exposure than for concretes subject to internal exposure, whilst for externally exposed concrete the effect may be much less.

### **2.9.3 Exposure classification**

There are significant differences between the exposure classification in pr EN 206 (BSI, 1997a) and that in BS 8110 (1997c). Also, there is some difficulty in the interpretation of the carbonation classes in pr EN 206, particularly XC3 ‘moderate humidity’ and XC4 ‘cyclic wet and dry’. During development of the exposure classification it was intended that the increasing numerical designation should correspond to increasing severity. Literal interpretation of the definitions is difficult due to the absence of any quantification of ‘moderate’ humidity or the severity and frequency of the wetting and drying cycles.

Concrete wets rapidly but dries much more slowly; thus in an outdoor environment in the UK the frequent rain generally means the concrete has a relatively high moisture content and consequent low rates of carbonation. Review of published information has shown that the increased time for the carbonation front to reach the reinforcement in XC4 exposure conditions more than outweighs the shorter corrosion period before damage occurs. It would thus appear that, in the UK, XC3 is a more aggressive exposure class than XC4.

Although the definitions of exposure classes XC1 and XC2 are virtually opposite (ie. ‘dry’ and ‘wet, rarely dry’, respectively) they are both, in effect, nil classes for carbonation-induced corrosion. In the case of XC1, carbonation of the concrete may be relatively rapid but the moisture content of the concrete, by definition, will be insufficient to support significant corrosion of the reinforcement. Conversely, in the case of XC2, the rate of corrosion of the reinforcement might be quite rapid but the high moisture content of the concrete will mean that carbonation of the concrete is unlikely to reach the reinforcement within the lifetimes being considered, ie. 50 years and 100 years.

It is thus clear that the proposed pr EN 206 classification of carbonation-induced corrosion would benefit from review of the definitions of the exposure classes XC1 to XC4. If the system remains unchanged in pr EN 206, it will be necessary to give guidance for use in the UK on how to interpret the classification system.

It is not possible to make a direct comparison with recommendations for concrete quality in current British Standards because of the different systems of exposure classification. Nevertheless, it is possible to make assumptions within the interpretation of those classifications that make broad comparisons possible. Indeed such assumptions have been necessary in the



preceding sections to enable interpretation of the available carbonation depth data in terms of standardised exposure conditions.

Within this report it has been assumed that unsheltered outdoor exposure corresponds to the pr EN 206 exposure condition XC4 (cyclic wet and dry) and that sheltered outdoor exposure corresponds to XC3 (moderate humidity). Dry indoor exposure has been assumed to be equivalent to XC1 (dry). These can be assumed also to correspond to British Standard exposure conditions 'severe', 'moderate' and 'mild' respectively. The British Standard equivalent to XC2 (wet, rarely dry) is probably 'moderate' as this includes concrete surfaces continuously under water. These equivalences are summarised in Table 2.4.

**Table 2.4:** Comparison of pr EN 206 (BSI, 1997a) and BS 5328 (1997c) and BS 8110 (1997d) exposure classes.

pr EN 206 Exposure condition	BS 5328 Exposure condition
XC1	Mild
XC2	Moderate
XC3	Moderate
XC4	Severe

#### 2.9.4 Assumptions made during analysis of available data

A number of assumptions have been necessary within this chapter; some are more firmly based than others. Consideration of the findings of this chapter should include an assessment of these assumptions to check whether the safety margin they provide for carbonation-induced corrosion of structures in service is appropriate. The assumptions made include:

- The minimum cover to reinforcement is as specified in ENV 1992-1-1, (BSI, 1992).
- Corrosion of reinforcement will commence when the carbonation front reaches the minimum reinforcement depth.
- For CEM I and CEM I/fly ash concretes, the curing achieved will be equal in effect to one day's laboratory curing, and for CEM I/slag concretes, the curing period achieved will be equal in effect to three day's laboratory curing.
- Exposure class XC3, moderate humidity, is represented by outdoor sheltered exposure.
- Exposure class XC4, cyclic wet and dry, is represented by outdoor unsheltered exposure.
- The cover concrete is well compacted.
- Future carbonation will occur at the rate predicted from the short term data.
- Concrete in structures will carbonate at a similar rate to the specimens from which the data was obtained.
- Concrete production and placement has followed reasonable good practice and the properties of concrete in the structure are within normal limits of variability.
- Carbonation depth indicated by the phenolphthalein test is the carbonation depth influencing reinforcement corrosion.

- The acceptable limit of serviceability is when the concrete cover cracks.
- The carbonation depth,  $d_c$ , in Portland cement concrete at any time,  $t$ , in exposure classes XC3 and XC4 is given by the expression:

$$d_c \propto t^{0.4}$$

- Cracking of the concrete cover in exposure class XC3 will occur 15 years after the depth of carbonation reaches the minimum reinforcement depth
- Cracking of the concrete cover in exposure class XC4 will occur as soon as the depth of carbonation reaches the minimum reinforcement depth.
- The carbonation depth,  $d_{cs}$ , in concrete made with cement containing 40% slag at any age is related to the free water/effective cement content by the expression:

$$d_c = \phi [w/(c + 0.6s)]$$

The k-factor of 0.6 was deduced from tests on concretes subject to exposure class XC3. For exposure class XC4 a higher value for k may be appropriate.

### 2.9.5 Discussion of the assumptions

Unfortunately, data is generally available only for age spans considerably shorter than those required for design.

There is some evidence (Bakker, 1992) that suggests corrosion of reinforcement in exposure classes XC3 and XC4 may not commence as soon as the carbonation front reaches the minimum depth of reinforcement or may even be delayed indefinitely (Schiessl et al, 1994). Commencement of corrosion will depend upon the depth of concrete cover and its permeability to oxygen and especially moisture, which are both essential for corrosion to occur. Thus, whilst the findings of this report, in terms of the minimum concrete qualities required to ensure given carbonation rates, can be substantiated by experimental data, the link to limits of serviceability depends strongly upon assumptions about initiation times for corrosion and subsequent cracking. The latter is clearly an area in which further research is desirable.

Cover is generally specified as a nominal value in the UK with a negative tolerance of 5 mm effectively defining minimum cover. Studies have shown that the specified 5 mm negative tolerance is commonly not achieved in practice and that a 10 mm negative tolerance might be more appropriate for most in-situ concrete.

## 2.10 Minimum requirements for concrete to resist carbonation-induced corrosion

The minimum requirements for concrete to resist carbonation-induced corrosion for 50 and 100 years, based on this study, when the minimum cover to reinforcement is 30 mm, are summarized in Tables 2.5 and 2.6 respectively. Recently the British Standard Institution committee B/517/1 (BSI, 1997b) has combined exposure classes XC3 and XC4 and this is reflected in Tables 2.5 and 2.6.

**Table 2.5:** Limiting values for composition and properties of concrete exposed to risk of corrosion of reinforcement induced by carbonation for an intended working life of at least **50 years**.

Exposure class	Dry	Wet, rarely dry	Moderate humidity or cyclic, wet/dry		
	XC1	XC2	XC3/XC4		
Minimum strength class	C20/25	C20/25	C30/37	C35/45	C35/45
Maximum w/c	0.65	0.65	0.60	0.35	0.45
Minimum cover to reinforcement (mm)	15	30	30	30	30
Cements	All		I, I/SR, II/A-S II/B-S, II/A-D, II/A-V, II/B-V, IV/A <sup>+</sup>	IV/B <sup>+</sup>	III/A <sup>§</sup> III/B <sup>§</sup>
For combinations					
Slag (%)	≤ 80	≤ 80	≤ 35	-	36-80
Fly ash (%)	≤ 55	≤ 55	≤ 35	36-55	-

\* Siliceous fly ash.

§ Limiting values for CEM III/A and III/B cements based on limited data.

**Table 2.6:** Limiting values for composition and properties of concrete exposed to risk of corrosion of reinforcement induced by carbonation for an intended working life of at least **100 years**.

Exposure class	Dry	Wet, rarely dry	Moderate humidity or cyclic, wet/dry		
	XC1	XC2	XC3/XC4		
Minimum strength class	C20/25	2	C40/50	C50/60	C50/60
Maximum w/c	0.65	0.65	0.45	0.30	0.35
Minimum cover to reinforcement (mm)	15	30	30	30	30
Cements	all		I, I/SR, II/A-S, II/B-S, II/A-D III/A-V, II/B-V IV/A <sup>+</sup>	IVB <sup>+</sup>	III/A <sup>§</sup> III/B <sup>§</sup>
For combinations					
Slag (%)	≤ 80	≤ 80	≤ 35	-	36-80
Fly ash (%)	≤ 55	≤ 55	≤ 35	36-55	-

\* Siliceous fly ash .

§ Limiting values for CEM III/A and III/B cements based on limited data.

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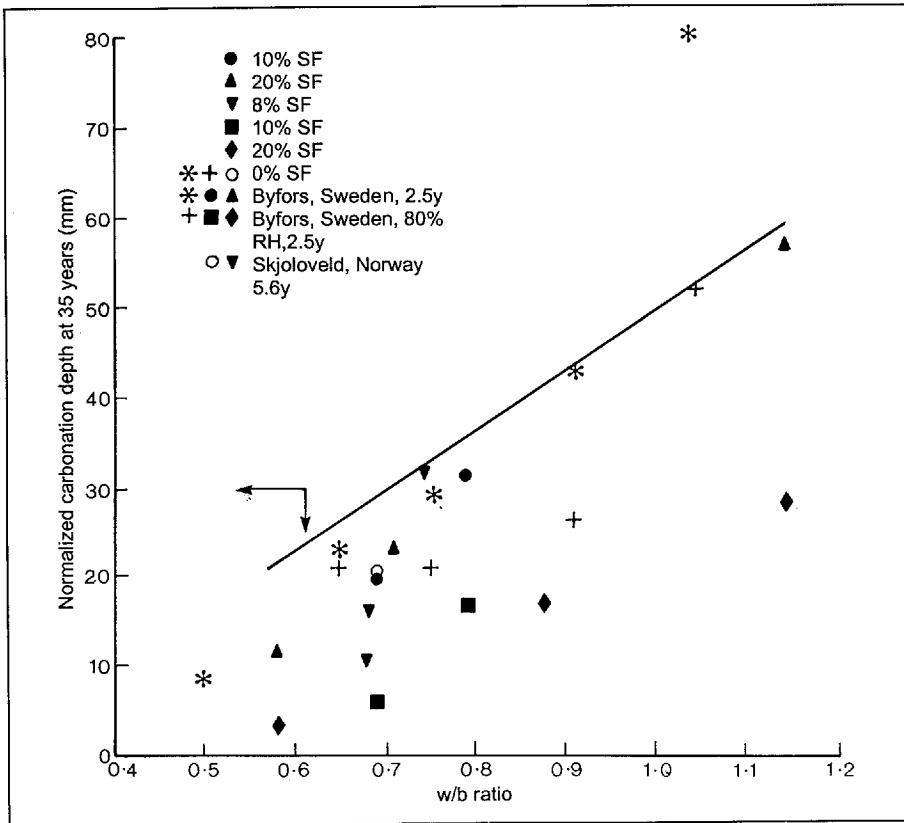
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**APPENDIX 2.1: Silica-fume concretes subject to moderate humidity (XC3)**

In the UK little work has been reported on the effect of silica fume upon rates or depths of carbonation into concretes subject to sheltered external exposure. Some overseas data (Byfors, 1985; Skjolovold, 1986) is plotted in Figure 2.23. On the basis of this data, it is possible that the cementing efficiency factor ( $k$ ) for resistance to carbonation is greater than 1. However, because of the limited data, it is suggested that the  $k$ -factor for silica fume should be taken to be 1. This is consistent with pr EN 206 (BSI, 1997a) in which a  $k$ -factor of 1 for silica fume is specified for XC exposure classes when the water-binder ratio is greater than 0.45. It follows that, for a CEM II/A-D Class 42.5 cement, maximum water-cement ratios of 0.61 and 0.85 (ie. the same values as for CEM I), for sheltered and unsheltered external exposure respectively, should ensure that the depth of carbonation is unlikely to exceed 30 mm until after ages of 35 and 50 years respectively.



**Figure 2.23:** Dependence of carbonation depth upon w-b ratio. Sheltered external exposure. Non-UK data.



**APPENDIX 2.2: Unpublished BCA and BRE data on the carbonation of CEM I, CEM I/fly ash and CEM I/slag concretes subject to sheltered external exposure**

**Table 2.7:** Mix details, 28-day compressive strength and depth of carbonation after three years' exposure. 50% slag. (Hobbs, 1995).

CEM I content (kg/m <sup>3</sup> )	Slag content (kg/m <sup>3</sup> )	Free water-binder ratio	28-day compressive strength (N/mm <sup>2</sup> )	Mean carbonation depth at 3 years <sup>+</sup> (mm)
183	183	0.475	40.6	13
140	140	0.61	26.6	21

<sup>+</sup> 100 mm cubes, 1 day initial cure.

**Table 2.8:** Mix details, 28-day compressive strength and depth of carbonation after 3.25 years' exposure. CEM I and 50% pfa. (Hobbs, 1995).

CEM I content (kg/m <sup>3</sup> )	Fly ash content (kg/m <sup>3</sup> )	Free water-binder ratio	28-day compressive strength (N/mm <sup>2</sup> )	Mean carbonation depth at 3.25 years (mm)	
				1-day cure <sup>+</sup>	3-day cure <sup>+</sup>
275	-	0.60	35.3	15	11
138	138	0.515	23.2	24	20
196	196	0.38	47.8	10	12
350	-	0.46	58.0	7	6
175	175	0.44	37.2	16	14
262	262	0.35	50.6	11	9

<sup>+</sup> Samples sealed for 1 and 3 days. 100 x 100 x 250 mm prisms.

**Table 2.9:** Mix details, 28-day compressive strengths and depth of carbonation after 5 years' exposure. (Barker and Matthews, 1995; Matthews, 1995).

CEM I content (%)	Fly ash content (%)	Slag content (%)	Free water-binder ratio	28-day compressive strength (N/mm <sup>2</sup> )	Mean carbonation depth at 5 years (mm) <sup>+</sup>	
					1d cure	3d cure
100	-	-	0.54	44.3	7.0	6.6
70	30	-	0.54	33.2	12.4	11.3
30	-	70	0.54	30.3	21.7	16.8
30	-	70	0.44	45.5	12.3 (2 yr)	9.4 (2 yr)

<sup>+</sup> 75 x 75 x 250 mm prisms.

**Table 2.10:** Depth of carbonation at 4 years' exposure for a number of concretes with a 28-day compressive strength of 34 N/mm<sup>2</sup>. (Marsh, 1995 from Brown and Beeby, 1992).

CEM I content (%)	Fly ash content (%)	Slag content (%)	Free water-binder ratio	Mean carbonation depth at 4 years (mm) <sup>+</sup>	
				1d cure	3d cure
100	-	-	0.76	9.4	8.5
70	30	-	0.41	11.9	10.6
70	30	-	0.42	12.1	10.5
50	-	50	0.68	16.8	14.4
60	-	40	0.58	11.7	11.9

<sup>+</sup> 250 x 250 X 1000 mm columns.

## CHAPTER 3

## Minimum requirements for concrete to resist deterioration due to chloride-induced corrosion

by D. W. Hobbs, British Cement Association  
J. D. Matthews, Building Research Establishment

### 3.1 Summary

The draft European Standard pr EN 206 (BSI, 1997a) classifies environments giving a risk of deterioration due to chloride-induced corrosion into six exposure classes:

Sea water			Chloride other than sea water		
XS1 Exposed to airborne salt	XS2 Submerged	XS3 Tidal, splash and spray zones	XD1 Moderate humidity	XD2 Wet, rarely dry	XD3 Cyclic wet and dry

This Chapter reviews the results from published studies of the performance of concrete subject to external chlorides in order to establish minimum concrete qualities, for the above exposure classes, necessary to achieve a design working life of 50 and 100 years (classes 3 and 4 of ENV 1991-1:1994, European Committee for Standardization). The cements considered are CEM I (Portland cement) and binders containing fly ash, slag and silica fume.

Minimum concrete qualities applicable to the UK are established, based on international studies of the performance of reinforced and prestressed concretes exposed to sea water or de-icing salt. It was found necessary to base the recommendations on the international literature as extensive field tests on the performance of reinforced concrete subject to sea water exposure and de-icing salt exposure have not been carried out in the UK. Due to lack of published information, no recommendations are given for the minimum qualities of concrete for submerged concrete where drying occurs from one or more faces, or for concrete car parks.

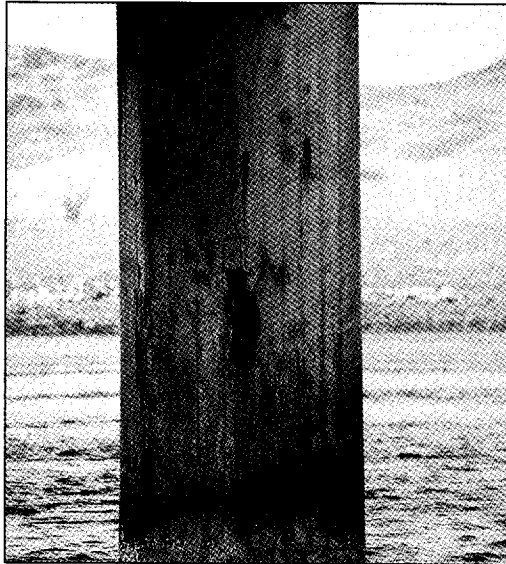
### 3.2 Introduction

Chloride ions can be present in a concrete as a result of the application of de-icing salts, exposure to a marine environment, or airborne salt, or from the concrete constituents. Under field conditions chloride ions can enter the concrete by a number of mechanisms, (i) by diffusion, (ii) by capillary suction under wetting and drying conditions, (iii) under a hydrostatic head and (iv) through cracks or defective joints. In concrete the chloride ions are present both as part of the hydrate structure and as free ions in the pore solution. If present in sufficient quantities, the free chloride ions close to the steel can destroy the passive oxide layer on the steel. This can lead, in the presence of moisture and oxygen, to corrosion, followed by corrosion-induced cracks parallel to reinforcement and then by spalling and loss of concrete section (Figures 3.1 to 3.3). The sequence of steel corrosion can be illustrated by the model shown in Figure 3.4 (Tuutti, 1982).

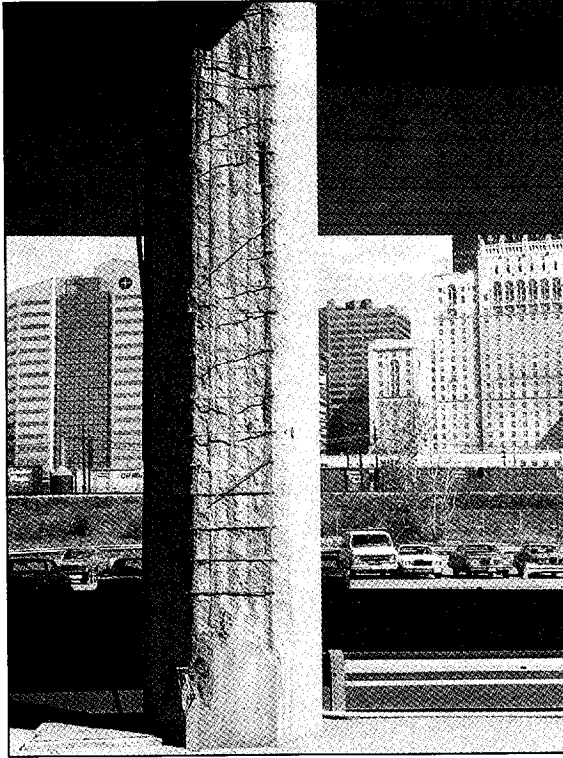
The age at which a critical free chloride level is reached at the steel-concrete interface depends upon several factors including:

degree of exposure to chlorides	pH of the pore solution
water-binder ratio	binder type
curing time	temperature
moisture state	concrete cover
carbonation	uniformity of bond between concrete and steel

(Pettersson, 1992; Hausmann, 1967; Mangat and Molloy, 1992; Pettersson, 1994; Guiguis et al, 1994; Dhir et al, 1993; Rasheeduzzafar, Dakhil and Mukarram, 1987; Rosenberg et al, 1989; Page, 1996; Nilsson et al, 1996; Yonezawa et al, 1988). The propagation period, that is the time taken between initiation of corrosion and visual cracking, will also be influenced by these parameters as well as bar diameter, bar spacing and concrete strength. Availability of oxygen in moist concrete, rather than chloride ion concentration, is probably the major parameter governing the propagation period.



**Figure 3.1:** Cracking and rust staining due to chloride-induced corrosion.

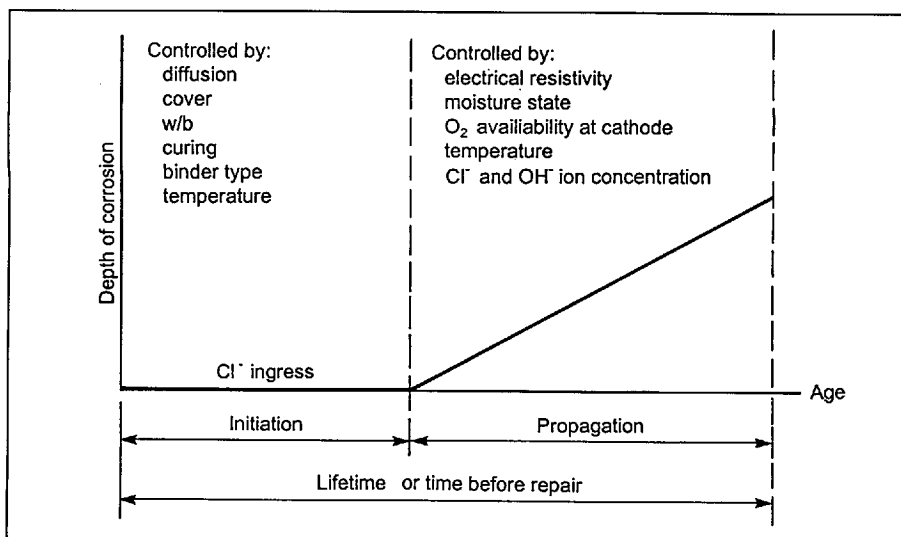


**Figure 3.2:** Loss of reinforcement cover due to chloride-induced corrosion.



**Figure 3.3:** Spalling due to chloride-induced corrosion. Marine exposure.

One of the six exposure classes given in BS 8110 (1997b) is applicable to situations where chloride ingress, without abrasion, may exist. The concrete quality and nominal cover to steel to meet this durability requirement are given in Table 3.1. When the lowest grade of concrete is C50, no air-entrainment is required. However, when the lowest grade of concrete is less than C50, the average total air content by volume of the fresh concrete should, for a nominal maximum aggregate size of 20 mm, be 5.5%.



**Figure 3.4:** Corrosion model. After Tuutti (1982).

**Table 3.1:** Concrete qualities and nominal cover to steel to meet durability requirements (BS 8110: Part 1: 1997, Table 3.3).

Conditions of exposure	Nominal cover (mm)		
Very severe	50 <sup>+</sup>	40 <sup>+</sup>	30
Maximum free w-c ratio	0.55	0.50	0.45
Minimum c (kg/m <sup>3</sup> )	325	350	400
Lowest grade of concrete	C40	C45	C50

<sup>+</sup> Where concrete is subject to freezing whilst wet, air-entrainment should be used and the strength grade may be reduced by 5.

The concrete qualities and minimum cover to steel for concrete subject to marine exposure (BS 6349: Part 1: 1984: AMD 6189) are given in Table 3.2. No air entrainment is required.

**Table 3.2:** Concrete qualities and minimum cover to steel to meet durability requirements (BS 6349: Part 1: 1984, Sections 58.5 and 58.10) Marine exposure.

Exposure	Concrete	Minimum cover (mm)	Maximum water-cement ratio	Minimum cement content (kg/m <sup>3</sup> )
Directly exposed to sea water spray or in tidal zone down to 1 m below lowest level	Reinforced	50	0.45	400
	Plain	NA	0.50	350

The concrete qualities and nominal cover to steel for concrete bridges exposed to salt (BS 5400: Part 4: 1990) are given in Table 3.3. In the *Design manual for roads and bridges BD 57/95 Design for durability*, published in 1995, the requirements have been amended for in-situ concrete as follows:

*“In designing cast in-situ concrete members, the cover used in design and indicated on the drawings, shall be the nominal cover derived from BS 5400: Part 4: Table 13, increased by 10 mm.”*

Thus in-situ concrete is treated more onerously than factory-produced concrete.

**Table 3.3:** Nominal cover to reinforcement under particular conditions of exposure (BS 5400: Part 4: 1990, Table 13). Concrete bridges.

Environment	Examples	Nominal cover (mm)			
		Concrete grade			
		25	30	40	50 and over
<i>Very severe</i> Concrete surface directly affected by: de-icing salts  sea water spray	Walls and structure supports adjacent to the carriageway Parapet edge beams	+	§	50 ‡	40
	Concrete adjacent to the sea				
<i>Extreme</i> Concrete surfaces exposed to: abrasive action by sea water  water with a pH ≤ 4.5	Marine structures	+	+	65 ‡	55
	Parts of structure in contact with moorland water				

+ Concrete grade not permitted.

§ For parapet beams only, grade 30 concrete is permitted provided it is air entrained and the nominal cover is 60 mm.

‡ Air-entrained concrete should be specified where the surface is liable to freezing whilst wet (see Part 7 of BS 5400).

The draft European standard pr EN 206 (BSI, 1997c) classifies concretes subject to a risk of deterioration due to chloride-induced corrosion into six exposure classes as follows:

Sea water			Chloride other than sea water		
XS1 Exposed to airborne salt	XS2 Submerged	XS3 Tidal, splash and spray zones	XD1 Moderate humidity	XD2 Wet, rarely dry	XD3 Cyclic wet and dry

Examples of concretes which fall into each of these exposure classes are given in pr EN 206 (BSI, 1997b) as:

XS1: Structures near to or on the coast

XS2: Parts of marine structures

XS3: Parts of marine structures

XD1: Concrete surfaces exposed to direct spray containing chlorides.

- XD2: Swimming pools. Concretes exposed to industrial waters containing chlorides.
- XD3: Parts of bridges. Pavements. Car park slabs.

Based on the above examples, XD1 is indistinguishable from XD3. In this chapter it is assumed that a better example for XD1 would be concrete surfaces exposed to air-borne chlorides. A British Standards Institution committee (BSI, 1997c) has split exposure classes XS2 and XD2 each into two exposure classes. The informative examples applicable in the UK, which appear in the complementary UK concrete standard to pr EN 206 (BSI, 1997b) are as follows:

- XS1: Reinforced and prestressed concrete surfaces in saturated salt air.  
External reinforced and prestressed concrete surfaces in coastal areas more than 100 m from the sea.
- XS2A: Reinforced and prestressed concrete completely submerged more than 1 m below lowest low water level.
- XS2B: Reinforced and prestressed concrete elements where one surface is exposed to air (eg. immersed tunnels).
- XS3: Reinforced and prestressed concrete surfaces in sea water tidal, splash and spray zones down to about 1 m below lowest low water level.  
External reinforced and prestressed concrete surfaces less than 100 m from the sea.
- XD1: Reinforced and prestressed concrete surfaces in parts of bridges away from direct spray containing de-icing agents.
- XD2A: Reinforced and prestressed concrete surfaces totally immersed in water containing chlorides.
- XD2B: Reinforced and prestressed concrete elements where one surface is immersed in water containing chlorides.
- XD3: Reinforced and prestressed concrete surfaces directly affected by de-icing agents (eg. walls, abutments and columns within 10m of the carriageway, parapet edge beams and buried structures less than 1m below carriageway level).

The object of the work described in this chapter is to propose the minimum concrete qualities and minimum reinforcement covers for exposure classes XS1 to XD3 to give design working lives of 50 and 100 years (ie. Classes 3 and 4, respectively of ENV 1991-1: 1994, European Committee for Standardization). The concrete is assumed to be well compacted. The cements considered are CEM I (Portland cements) and binders containing pulverized fuel ash, ground granulated blastfurnace slag and silica fume. Throughout the remainder of this chapter, pulverized fuel ash is referred to as fly ash and ground granulated blastfurnace slag is referred to as slag. Guidance relating to CEM II/A-L and B-L (Portland limestone cement) concretes is not given due to lack of published information. However, it is important to note that aggregates containing limestone fines, or a proportion of limestone fines, have not been reported to have an adverse effect upon performance of reinforced concretes exposed to chlorides.

### **3.3 Chloride-induced corrosion in the UK**

In the UK, damage to concrete from chloride-induced corrosion of reinforcement is the greatest threat to the durability and integrity of concrete structures. It has been estimated that the annual cost of repairs due to chloride- and carbonation- induced corrosion in the UK is about £500 million (Building Research Establishment, 1994).



The first example of chloride-induced corrosion in UK concrete was observed at the beginning of the century in a concrete element built at Southampton docks in 1902 (Lorman, 1971). Here, a 'wet' concrete was used to ensure better passage of the concrete through the maze of reinforcement but corrosion-induced damage was noted within a few years. This contrasts with the first maritime reinforced concrete structure built in the UK in 1899. This was also built at Southampton docks, using a low w-c ratio concrete and in 1955 was reported to be in excellent condition (Lorman).

Many instances of damage due to ingress of chlorides from sea water into concrete elements have been reported. In the past 20 to 30 years, many of the chloride-induced corrosion problems reported have resulted from the increasing application of de-icing salts to bridges and roads, which has occurred since the 1950s.

There is a widely held belief that most chloride-induced corrosion problems are due to inadequate concrete specification and that specifying lower water-cement ratios, greater cover depths or alternative binders, would greatly reduce the risk of chloride-induced corrosion problems. As a consequence, durability provisions in codes over the past 60 years have become more onerous with minimum cover increasing from 12 mm to 40 mm or more, with minimum concrete quality by grade increasing, since the 1960s, and with maximum water-cement ratios reducing (Beeby, 1993; DSIR, 1933; BSI, 1969; BSI, 1985; Highways Agency, 1995), (see Table 3.4). However, the more onerous durability provisions have not been successful in minimizing chloride-induced corrosion problems in concrete elements.

An examination of the literature leads to the conclusion that most corrosion problems in the field have resulted from construction practices and design procedures which were not adequate for the exposure conditions to which the concretes were to be subjected (Hobbs, 1996). Some examples of weaknesses in structural and material design and construction which have led to premature deterioration are given below:

- Low cover to reinforcement
- Inadequately compacted cover concrete
- Failure of expansion joints above piers and abutments
- The presence of lapped joints, which can restrict compaction, and cold joints at the ground level of bridge piers and columns
- Poorly designed concrete mixes, eg. incorrectly proportioned concrete
- Concretes not adequately designed to resist freeze-thaw attack.

Reinforced horizontal surfaces of concrete, such as tops of pier heads, are particularly vulnerable to chloride-ingress problems, caused firstly by bleeding which increases the water-cement ratio of the surface layer and forms vertical bleeding channels and secondly, by the likely presence of plastic settlement cracks induced by reinforcement placed close to the top of a concrete pour.

In 1995 the UK Highways Agency issued Advice Notes to improve the durability of highways structures by drawing to the attention of designers aspects of design and detailing which are relevant to the durability of structures but are not covered adequately in the existing requirements for the design of these structures. Thus there is a tacit acceptance that changes in material specifications, except by using non-corrodible reinforcement or plain concrete, may not have a major effect on the frequency of chloride-induced corrosion problems.

**Table 3.4:** Increase in minimum cover requirements.

Document	Date	Concrete grade (N/mm <sup>2</sup> )	Minimum cover (mm)	
			Supports adjacent to carriageway	Deck soffits
Memorandum 577	1945	All	38	25 <sup>+</sup>
Memorandum 758	1961	All	38 <sup>+</sup>	38 <sup>+</sup>
Memorandum 785/1	1969	31	51	32
		41	38	25
BE10	1970	30	50	30
BE1.73	1973	30	50	30
		37.5	40	25
BD17.83	1983	30	55 nominal (50 minimum)	45 nominal (40 minimum)
		40	45 nominal (40 minimum)	35 nominal (30 minimum)
BS 5400: Part 4	1984	30	Not permitted	45 nominal (40 minimum)
		40	50 nominal (45 minimum)	35 nominal (30 minimum)
BS 5400: Part 4	1990	30	Not permitted	45 <sup>§</sup>
		40	50 <sup>§</sup> nominal	35 nominal
		50 or over	40 nominal	30 nominal
BD 57/95	1995	50 and over	50 nominal (in-situ)	
			40 nominal (factory)	

<sup>+</sup> Or bar diameter if greater.

<sup>§</sup> Air-entrained if subject to freezing whilst wet.

Although most corrosion problems can be attributed to inadequate construction practices and inadequate design, examples of chloride-induced deterioration which can be attributed to inadequate quality of the concrete employed have been reported in the literature (Hobbs, 1996).

**The remainder of this chapter deals with the material aspects of minimizing chloride-induced corrosion problems.** It should be borne in mind that minimizing such problems by the correct specification of concrete may have only a minor influence upon the overall problem, compared with the improvements which could result from changing construction practices and design to minimize chloride ingress into concrete structures exposed to salt.

### 3.4 Chloride ingress: marine exposure class XS3

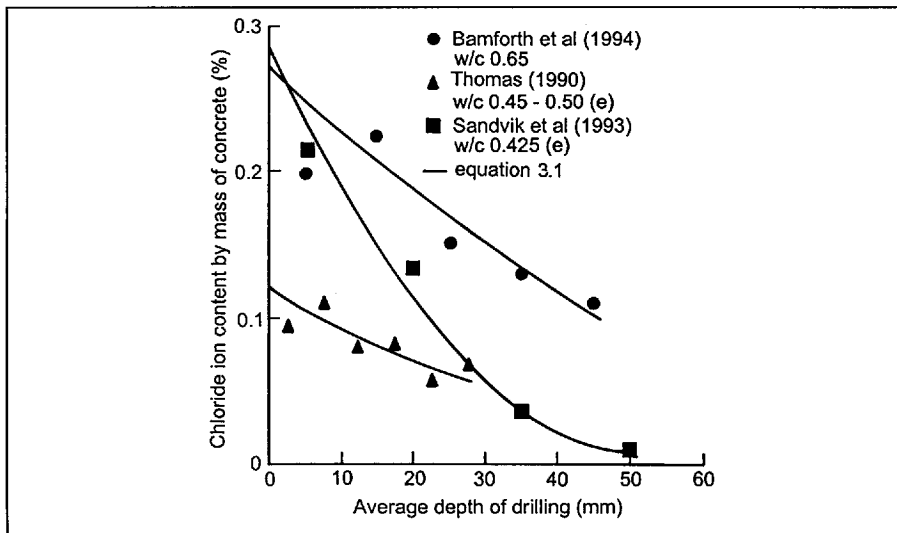
The parts of reinforced and prestressed concretes subject to tidal, splash and spray zone exposure are the most susceptible to chloride-induced corrosion problems. Such concretes are essentially wet throughout their life. For concretes subject to this range of exposures, it is generally assumed

that chloride ingress into concrete can be described by Fick's second law of diffusion:

$$C_x = C_s [1 - \operatorname{erf}(x)/2(D_{cc}t)^{1/2}] \quad (3.1)$$

where  $C_x$  is the total chloride level at depth  $x$  at age  $t$ , and  $C_s$  and  $D_{cc}$  are the surface chloride level and the diffusion coefficient respectively. In reality the transport process is more complex, for example, the transport process of chloride ions into the concrete is reduced by the formation of brucite and calcium carbonate on the concrete surface, by the continuing hydration of the cement and by binding of a proportion of the chloride ions.

Measurements of total chloride ingress into concrete in the tidal and splash zone have been made by a number of investigators. Figure 3.5 shows the fit of equation 3.1 to the measured chloride profile on CEM I concretes of differing water-cement ratio. The influence of water-cement ratio upon the fitted diffusion coefficient is shown in Figures 3.6 and 3.7 for a number of CEM I



**Figure 3.5:** Chloride concentration profiles in concretes subject to marine exposure. 5 years, Bamforth and Chapman-Andrews (1994); about 11 years, Sandvik and Wick (1993); 30 years, Thomas (1990); (e = estimated).

concrete elements cast since about 1960 which have been subject to splash or tidal zone exposure. Figure 3.6 is a linear-linear plot and Figure 3.7 a log-linear plot. Figure 3.8 is similar plot to Figure 3.7 of the more limited data for concretes containing slag, fly ash and silica fume. Here, the data is insufficient to establish trends with increasing slag, fly ash and silica fume contents. The ages of the concretes tested range from 5 to 30 years. In the investigations referred to where the effective diffusion coefficient was not reported by the author(s), a best fitting value for  $D_{cc}$  has been determined for the purposes of this Chapter, using the procedure proposed by Nagano and Naito (1986). The high coefficients observed by Liam et al are probably due to the high exposure temperatures ( $\approx 30^\circ\text{C}$ ). The  $D_{cc}$  for the CEM I concrete with a water-cement ratio of 0.37 was for a concrete pile which had micro-cracked during driving and bending (Gourley and Bieniak, 1983).

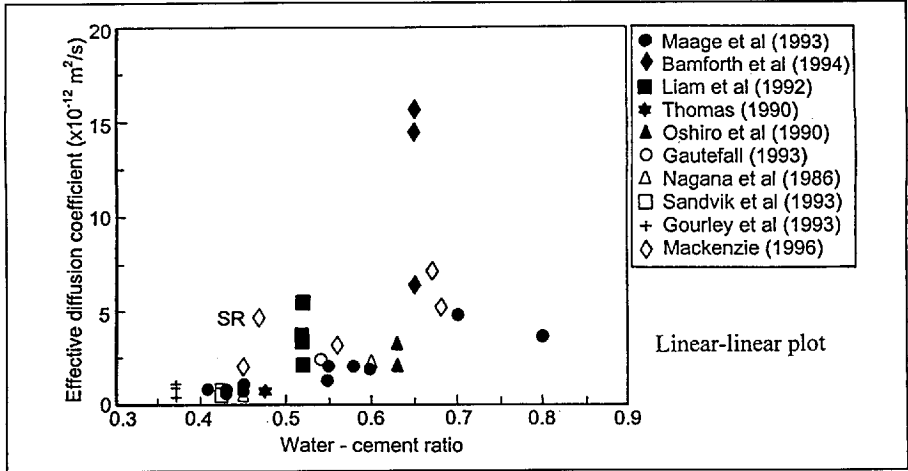


Figure 3.6: Relationship between effective diffusion coefficient and w-c ratio. Marine exposure.

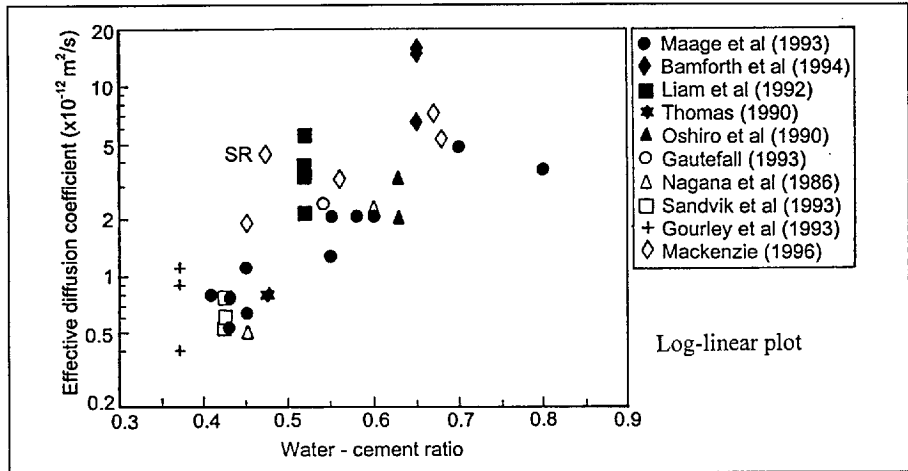
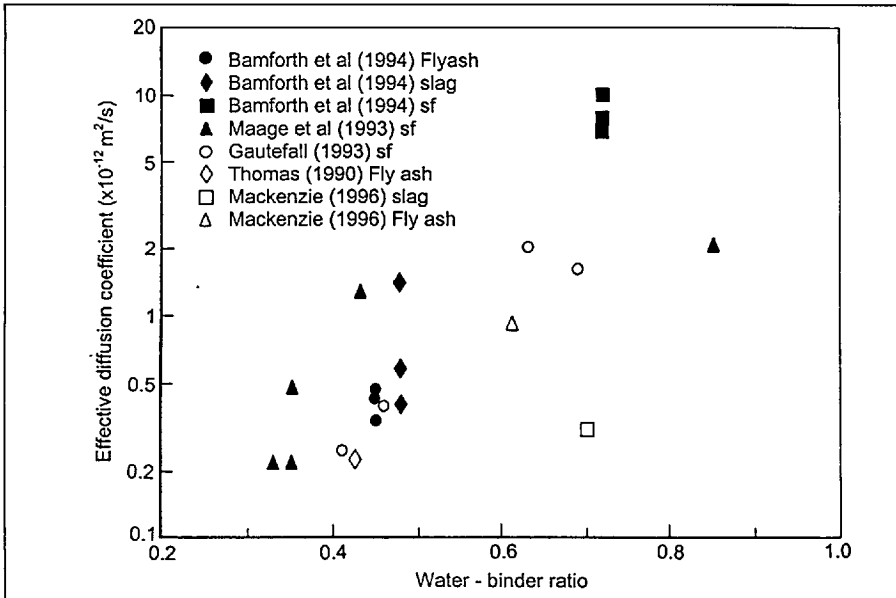


Figure 3.7: Relationship between effective diffusion coefficient and w-c ratio. Marine exposure.

In Figures 3.9 and 3.10, the effective diffusion coefficient is shown again plotted against water-cement ratio for CEM I concretes. Here, the results of Liam et al (1992), Gourley and Bieniak (1983) and Mackenzie (1996) have been adjusted for differences in temperature, on the basis of the Arrhenius equation, by multiplying  $D_{ce}$  by 0.5 (Singapore), 0.75 (Brisbane, Australia) and 0.5 (South Africa) respectively. In the case of Singapore and South Africa, it is assumed that the mean temperature of concrete in the splash zone is 10°C higher than in the UK. The best fitting equation is

$$D_{ce} = 0.04 (1166^{w/c}) \times 10^{-12} \quad (r = 0.693) \quad \dots\dots\dots (3.2)$$

Results obtained by Mackenzie on CEM I/SR Table Bay Breakwater concretes were not included in this calculation because one of the authors' (Hobbs) inspections of these breakwater concretes in 1983 showed that they were of complex shape and were possibly inadequately compacted. At water-cement ratios of about 0.45,  $D_{ce}$  ranges from 5 to 11 x 10<sup>-13</sup> m<sup>2</sup>/s, with a best fitting value



**Figure 3.8:** Relationship between effective diffusion coefficient and w-b ratio. Marine exposure.

of  $9.6 \times 10^{-13} \text{ m}^2/\text{s}$  (at a w-c ratio of 0.45) and  $C_s$  ranges from about 0.1 to about 0.4 % by mass of concrete. For a 0.45 water-cement ratio concretes containing fly ash, slag and silica fume,  $D_{ce}$  ranges from 2 to  $14 \times 10^{-13} \text{ m}^2/\text{s}$  with a best fitting value<sup>+</sup> of  $4.3 \times 10^{-13} \text{ m}^2/\text{s}$  which is less than 50% of that deduced for CEM I concretes of similar water-cement ratio. Larger differences have been reported by Thomas and Matthews (1996) for concretes containing 30% fly ash tested at an age of four years. These concretes were cured for one day, dried until an age of 28 days and then placed in the tidal zone on the BRE marine exposure site. Higher values for  $D_{ce}$  than those quoted above, are applicable to concretes subject to tropical marine exposure (Liam et al, 1992; Roy et al, 1993) and to a hydrostatic head (Gourley and Bieniak, 1983; Roy et al, 1993). According to a recently published review by Bamforth (1996), the surface chloride levels are broadly up to 40% higher for blended cement concretes than for CEM I concretes (compare Figures 3.11 and 3.12), average values being about 0.55 and 0.40% by mass of concrete respectively.

In Figure 3.13,  $D_{ce}$  is shown plotted against age for CEM I concretes with w-c ratios ranging from 0.40 to 0.50. For ages in the range six to 30 years, no clear dependence of  $D_{ce}$  upon age is apparent. Bamforth and Chapman-Andrews (1994) observed no change in  $D_{ce}$  for CEM I concrete, w-c ratio of 0.66, for ages in the range six months to six years. However, for concretes of lower w-c ratio containing fly ash and slag, Bamforth and Chapman-Andrews observed a continuing decline in  $D_{ce}$ .

<sup>+</sup> The best fitting equation to the data plotted in Figure 3.8 is

$$D_{ce} = 0.03 (397.5^{w/c}) \times 10^{-12} \quad (r = 0.564) \quad (3.3)$$

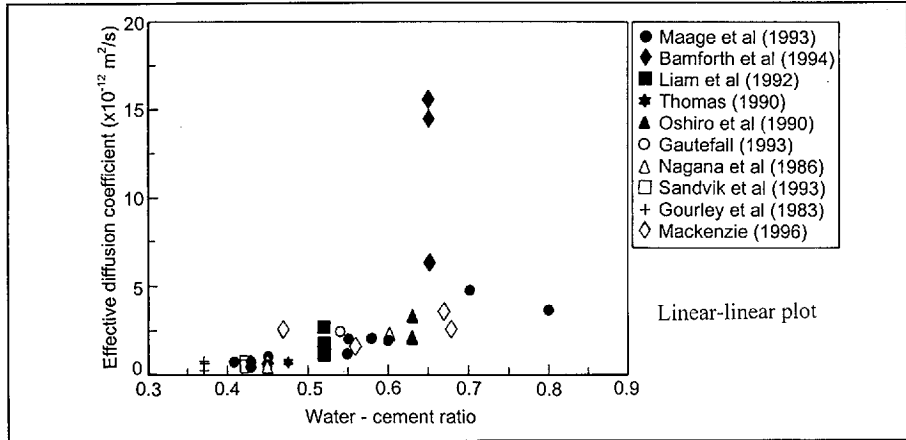


Figure 3.9: Relationship between effective diffusion coefficient and w-c ratio. Marine exposure.

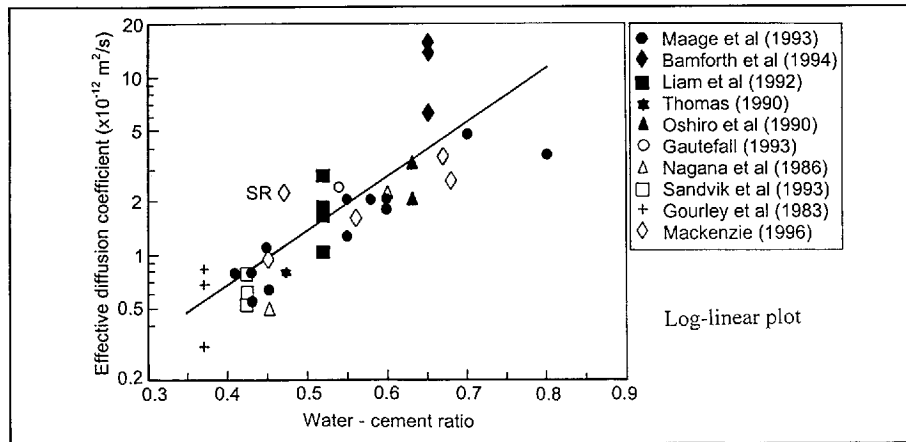


Figure 3.10: Relationship between effective diffusion coefficient and w-c ratio. Marine exposure.

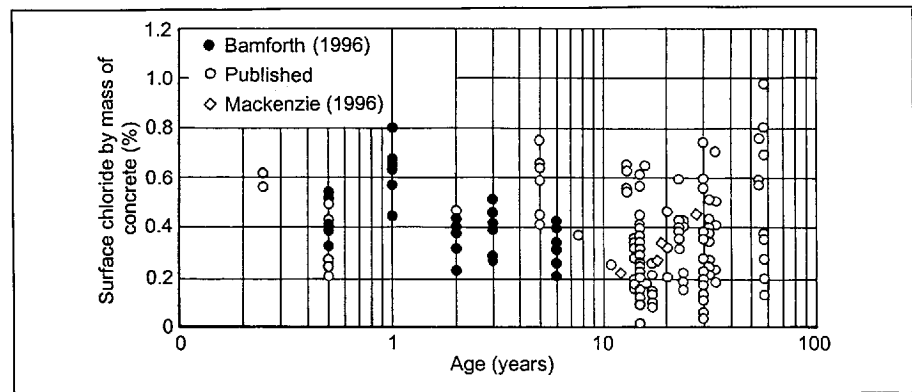
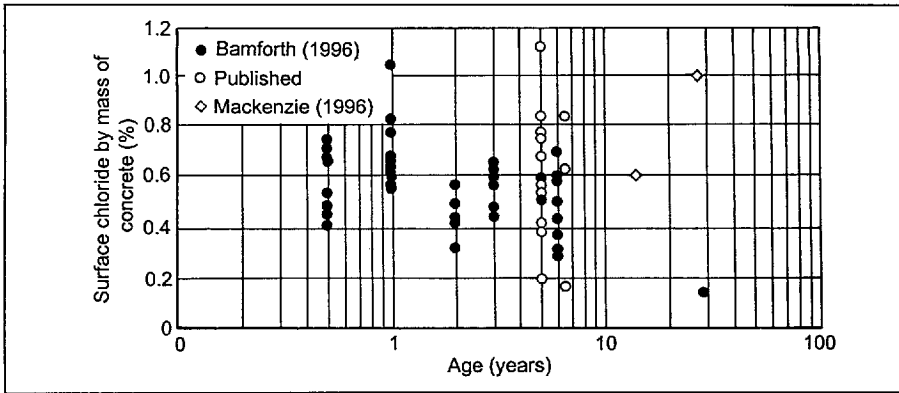
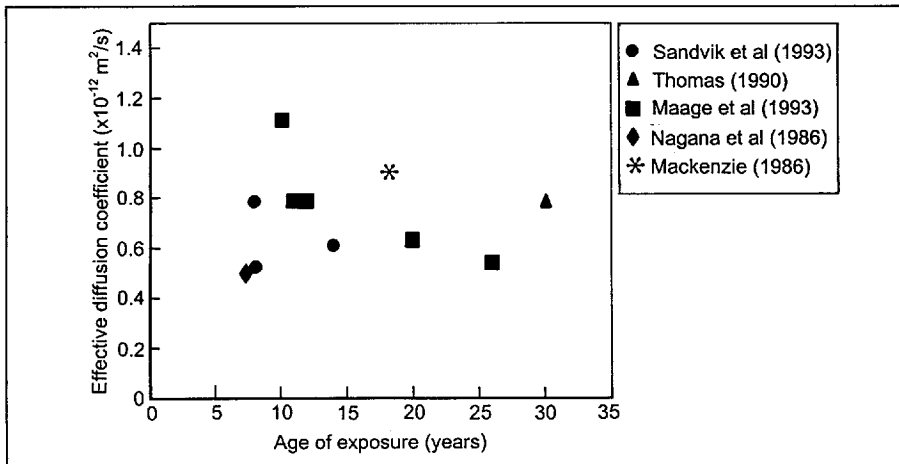


Figure 3.11: Surface chloride concentration versus age for CEM I concretes (Bamforth, 1996).



**Figure 3.12:** Surface chloride concentration versus age for blended cement concretes (Bamforth, 1996).



**Figure 3.13:** Relationship between effective diffusion coefficient and exposure age. Marine exposure. CEM I concretes  $0.4 \leq w/c \leq 0.5$ .

### 3.5 Service life and the propagation period: exposure class XS3

Table 3.5 summarizes the observations made on a number of field studies on reinforced and prestressed concretes subject to marine exposure. All of the concretes referred to are CEM I or CEM I/SR concretes. Examination of the information in the table clearly illustrates that the quality of concrete, as given by w-c ratio, is a major factor influencing the risk of chloride-induced corrosion and cracking. Stark (1995) found in the case of heat-cured prestressed concrete that performance may be related to the  $C_3A$  content of the cement, with the cement of zero  $C_3A$  content exhibiting the best performance. This is rather a surprising observation, as BS 8110 (1997b) requires an increase in cover of 10 mm for concretes prepared using CEM I/SR cements with  $C_3A$  contents below 3.5% by mass.

**Table 3.5:** Performance of concrete elements subject to marine exposure.

Investigator	Structures	When built	Concrete strength (N/mm <sup>2</sup> )	Intended cover (mm)	Water-cement ratio	Condition
Bamforth et al, 1994	1.0 x 0.5 x 0.3 m, precast concrete blocks. UK	1987	39.4	40	0.65	Half cell potential measurements showed corrosion commenced at 2 years
Oshiro et al, 1990	Building. Japan	1984	21	30 & 40	0.63	Half cell potential measurements showed corrosion after 3 years.
Liam et al, 1992	Jetty. Singapore	1968	-	70	0.52(e) <sup>§</sup>	At 24 years cracking and spalling.
Gautefall, 1993, 1995	1.5 x 1.5 x 0.5 m concrete blocks. Norway	1983	-	30 (facing sea), 50 (facing land)	0.54	Surface potentials indicated no corrosion at 9 years.
Thomas et al 1996	100 x 100 x 300 mm concrete prisms. UK	1992	-	20	0.49, 0.57, 0.68	At 4 years significant steel corrosion for 0.57 and 0.68 w-c ratio concretes.
Sandvik et al, 1993	Concrete platforms. North Sea	1971 onwards	-	>50	<0.45 <0.40 splash	Excellent
Somerville, 1995	Precast pretensioned members. UK	Early 1940s onwards	60-80	>21 to 40	0.35 to 0.42	A history of good performance.
Stark, 1995	152 x 152 mm air-entrained precast beams. <sup>†</sup> Los Angeles	1961	-	25 44	0.49 0.49	10 mm bars: good at 32 years. 19 mm bars: cracked by 32 years. Good condition at 32 years.
Stark, 1995	152 x 152 mm air-entrained precast prestressed beams, Los Angeles. 6.35mm prestressed strand heat cured to 60°C	1961	-	25 25	0.35 0.35	High tide, CEM I/SR, C <sub>3</sub> A, 0% & 4.7%; CEM I, 10.1%. Excellent condition at 32 years. Mean tide, CEM I/SR, C <sub>3</sub> A, 0% excellent condition at 32 years; 4.7% one rust spot; CEM I, 10.1% one rust spot + fine crack.
Clark, 1992	Precast, post-tensioned members	Early 1940s onwards	-	-	Probably 0.35 to 0.42	Very few cases reported of reinforcement corrosion.
Antonakis (1972) Anonymous (1997)	Royal Sovereign Light Tower. Reinforced sections. Post-tensioned sections.	About 1968	41 55	50 <sup>†</sup> 75 <sup>†</sup>	0.48 0.37	CEM I/SR, "is performing admirably" at about 29 years.

<sup>†</sup> Cement composition complies with requirements for sulfate-resisting CEM I cement (ENV 197X).

<sup>§</sup> Estimated.

<sup>†</sup> Nominal cover.



Only a few field surveys of concrete exposed to a marine environment, made with blended cements, have been reported. Wiebenga (1980) discusses the 50-year performance of slag cement structures in The Netherlands. The incidence of corrosion damage was observed to be low. Wiebenga concluded that "Regardless of the cement content, corrosion occurred mainly when the concrete cover was relatively small (less than 45 mm)." The compressive strength of the cores quoted by Wiebenga ranged from 61 to 76 N/mm<sup>2</sup>.

Service life predictions require a knowledge not only of the diffusion coefficient, but also of the surface chloride level, of the free chloride level at the concrete-steel interface at which corrosion commences, and of the propagation period (see Figure 3.4). Service life predictions are sometimes based upon the assumption that corrosion is initiated at a single critical total chloride level at the reinforcement. Such an approach is unsatisfactory because there is no unique total (or free) chloride ion concentration which initiates steel corrosion. The literature shows that the total critical chloride ion concentration increases as the binding of chlorides increases, as the hydroxyl ion concentration increases, as the degree of water saturation increases and as the water-cement ratio decreases (Hobbs, 1996). Replacing part of a CEM I by fly ash and slag can, depending upon the alkali levels of the binder components, increase or reduce the hydroxyl ion concentration (Hobbs, 1988). Replacing part of a CEM I by silica fume will reduce the hydroxyl ion concentration (Hobbs, 1988).

In specifications, a maximum allowable total chloride level of 0.4% by mass of binder is sometimes specified for reinforced concrete (BS 8110: 1997b). This chloride level is often taken to be the level at which corrosion commences. However, for low water-cement ratio concretes, total chloride levels close to the steel in excess of 1.0% by mass of CEM I may be required (Sandberg and Pettersson, 1995; Polder and Larbi, 1996). 1.0% by mass of CEM I cement translates into a total chloride level of about 0.17 to 0.20% by mass of concrete.

In the literature, little information exists on the propagation period. However, an approximate estimate can be made from the known performance of concretes subject to marine exposure. Observations which enable a prediction of the propagation period to be made have been published by Stark (1995) (Table 3.5). These observations relate to 152 x 152 x 1220 mm beams stored vertically at mean tide level and above high tide level. In the case of Stark's observations (see Table 3.5), a 32-year service life for Los Angeles exposure conditions, when 10 mm bars are used would, after correcting for mean temperature differences, convert to a 55 year or more service life for UK exposure conditions, but a service life of less than 55 years when 19 mm bars are used.

From Figure 3.10, the effective diffusion coefficient for a concrete with a water-cement ratio of 0.49 is about  $1 \times 10^{-12}$  m<sup>2</sup>/s and, according to Hobbs' (1996) interpretation of the literature, the maximum surface chloride level for a concrete of this water-cement ratio is about 0.4% by mass of concrete. Assuming the total chloride ion concentration at which corrosion commences is 0.20% by mass of concrete ( $\approx 1.0\%$  by mass of cement) gives, for UK exposure, an initiation period of about 25 years and a propagation period of about 30 years for beams reinforced with 19 mm bars and a propagation period greater than 30 years for beams reinforced with 10 mm bars.

### 3.6 Minimum specification for concrete subject to exposure class XS3

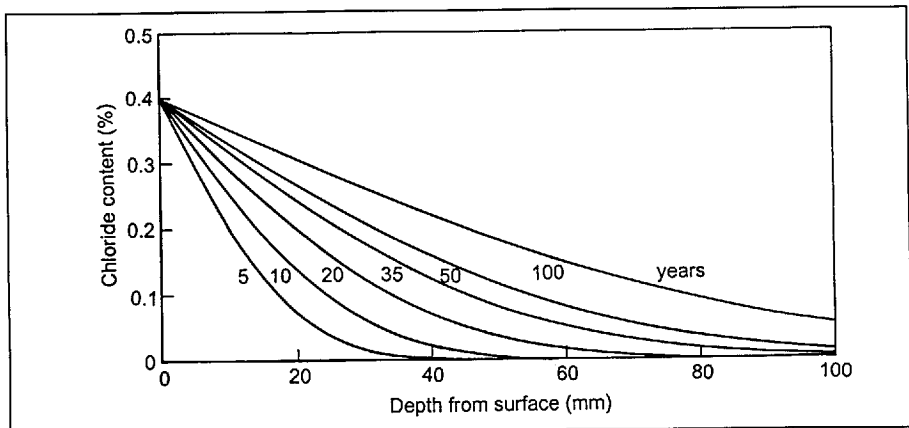
#### 3.6.1 Assumptions based on field performance: CEM I concretes

1. Fick's second law is valid for concretes subject to marine exposure.
2. For factory-produced CEM I (including CEM I/SR) concrete elements of w-c ratio 0.45, subject to tidal or splash zone exposure, the surface chloride concentration is 0.4% by mass of the concrete and the diffusion coefficient is  $10 \times 10^{-13} \text{ m}^2/\text{s}$ . (See Figures 3.10 and 3.11).
3. For a factory-produced CEM I (including CEM I/SR) concrete element with a w-c ratio of 0.40, the surface chloride concentration is 0.4% by mass of concrete and the diffusion coefficient is  $7 \times 10^{-13} \text{ m}^2/\text{s}$  (see Figures 3.10 and 3.11).
4. After the **total** chloride level at the steel reaches 0.20% by mass of concrete, the propagation period to visual cracking is 25 years, a more conservative figure than that deduced earlier. Here it is being assumed that the propagation period is independent of bar size and cover.

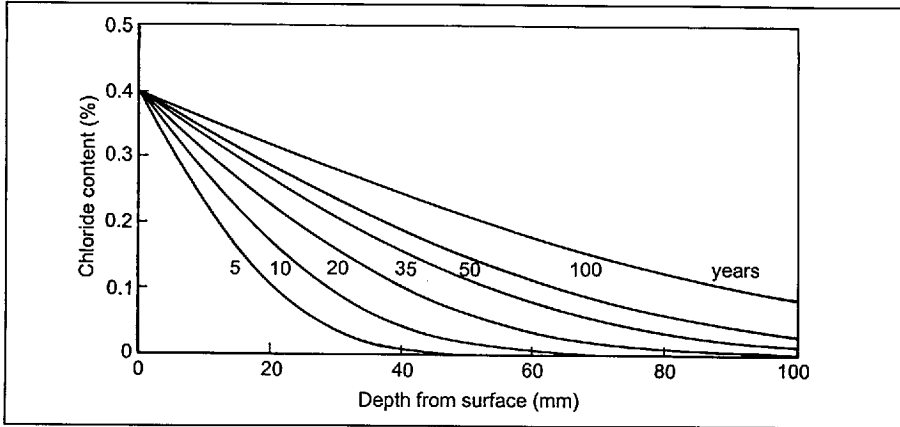
#### 3.6.2 Minimum specification: CEM I concretes

Distributions of total chloride ion calculated from assumptions 1 to 3 are given for precast concretes of w-c ratio 0.40 and 0.45 in Figures 3.14 and 3.15. The likely service lives which follow from assumption 4 are shown plotted against actual cover in Figure 3.16. Clearly when designing for working lives of 50 and 100 years, it is wise to design for longer service lives, eg. 75 years and 125 years respectively. It then follows from Figure 3.16 that design working lives of 50 and 100 years for 0.45 w-c ratio precast concrete elements can be met by minimum covers of 40 and 55 mm respectively. For a 0.40 w-c ratio precast concrete element, these minimum covers become 35 and 45 mm respectively.

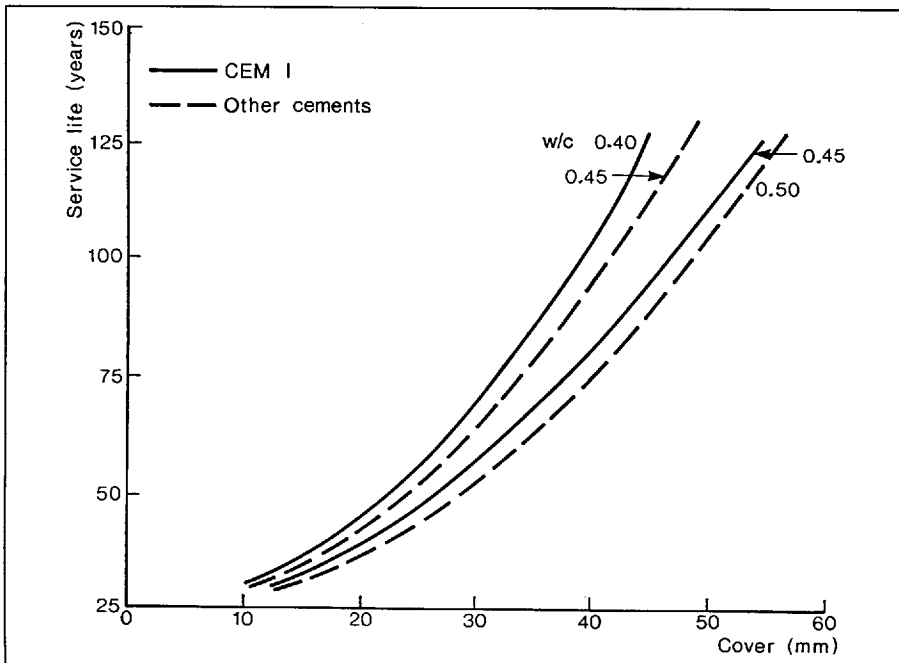
For in-situ concrete, it is proposed that with the exception of unprotected cast horizontal surfaces, the above minimum covers should be increased by 10 mm (Highways Agency, 1995) and that for unprotected in-situ cast horizontal surfaces, the minimum covers, as compared with precast concrete, should be increased by 25 mm. The latter increase is regarded as being necessary due



**Figure 3.14:** Relationship between predicted chloride content and depth - w/c 0.40,  $D_{ce} 7.0 \times 10^{-13} \text{ m}^2/\text{s}$ ,  $C_s$  0.4% by mass of concrete. CEM I concretes.



**Figure 3.15:** Relationship between predicted chloride content and depth - w/c 0.45,  $D_{ce} 1.0 \times 10^{-12} \text{m}^2/\text{s}$ ,  $C_s$  0.4% by mass of concrete. CEM I concretes.



**Figure 3.16:** Dependence of predicted service life upon concrete cover. Factory-produced concrete.

to the likely increase in water-cement ratio of the surface layers resulting from bleeding and because of the possible presence of vertical bleeding channels. The resulting minimum qualities and minimum covers for CEM I precast and in-situ concretes, subject to UK exposure class XS3, are given in Table 3.6 for design working lives of 50 and 100 years. The minimum qualities and minimum covers may also be taken to be applicable to CEM II/A-S and CEM II/A-V concretes.

**Table 3.6:** Minimum qualities and minimum covers for CEM I (including CEM I/SR), II/A-S and II/A-V concretes subject to exposure class XS3.

Concrete	Minimum grade (N/mm <sup>2</sup> )	Maximum water-cement ratio	Minimum cover (mm)	
			50 year design life	100 year design life
In-situ, all surfaces except unprotected horizontal top surfaces	C 40/50	0.45	50	65
	C 50/60	0.40	45	55
In-situ, unprotected horizontal top surfaces	C 40/50	0.45	65	80
	C 50/60	0.40	60	70
Precast	C 40/50	0.45	40	55
	C 50/60	0.40	35	45

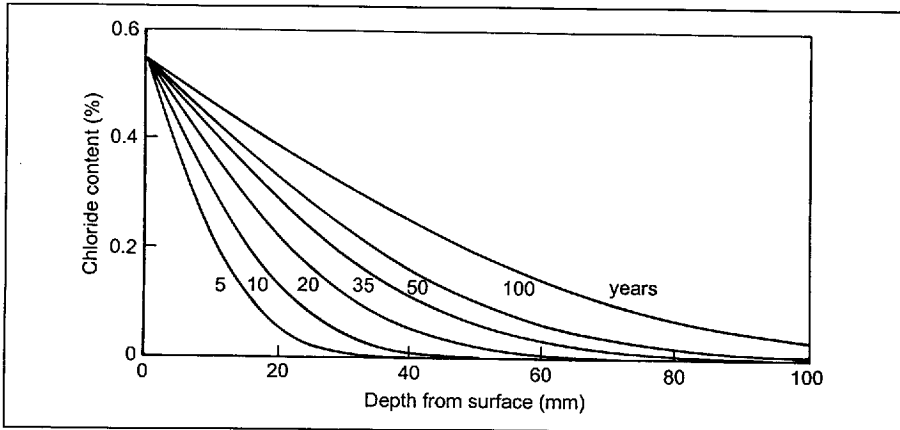
### 3.6.3 Assumptions based on field performance: CEM II/B-S, II/A-D, II/B-V, III/A, and III/B concretes

1. Fick's second law is valid for concrete subject to marine exposure.
2. For factory-produced concrete elements of a w-b ratio 0.45 made using CEM II/B-S, II/A-D, II/B-V, III/A and III/B cements subject to tidal or splash zone exposure, the diffusion coefficient is  $4.5 \times 10^{-13} \text{ m}^2/\text{s}$ . For similar concretes with a w-b ratio of 0.50, the diffusion coefficient is  $6.0 \times 10^{-13} \text{ m}^2/\text{s}$  (see Figure 3.8).
3. For the concretes in assumption 2, the surface chloride concentration is 0.55% by mass of concrete (see Figure 3.12).
4. After the **total** chloride level at the steel reaches 0.20% by mass of concrete, the propagation period to visual cracking is 25 years. Here it is being assumed that neither the total critical chloride level at which corrosion commences nor the propagation period are cement dependent.

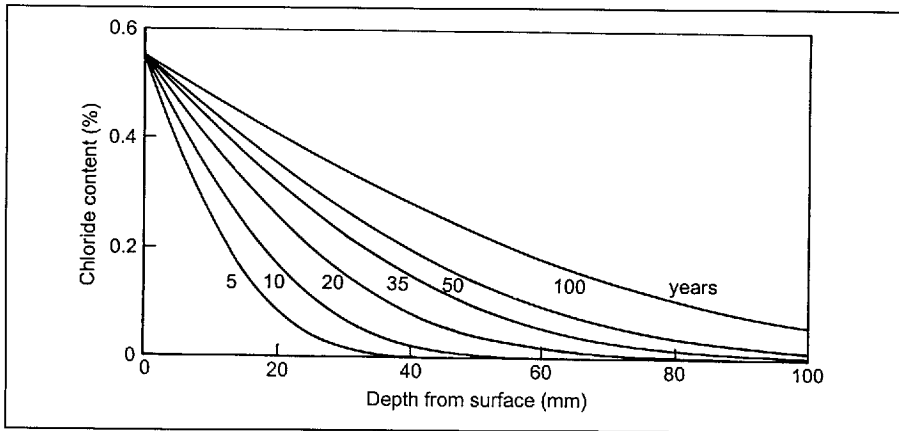
### 3.6.4 Minimum specification: CEM II/B-S, II/A-D, II/B-V, III/A and III/B concretes

Distributions of total chloride ion calculated from assumptions 1 to 3 are given for precast concretes of w-c ratio 0.45 and 0.50 in Figures 3.17 and 3.18. The likely service lives which follow from assumption 4 are shown plotted against actual cover in Figure 3.16. It then follows from Figure 3.16 that design working lives of 50 and 100 years for 0.45 w-c ratio precast concrete elements can be met by minimum covers of 35 and 50 mm respectively. For a 0.50 w-c ratio precast concrete element, these minimum covers become 40 and 60 mm respectively.

For in-situ concrete, it is proposed that the above minimum covers be increased by the same amount as for CEM I concretes. The resulting minimum qualities and minimum covers for CEM II/B-S, II/A-D, II/B-V, III/A and III/B precast and in-situ concretes, subject to UK exposure class XS3, are given in Table 3.7 for design working lives of 50 and 100 years.



**Figure 3.17:** Relationship between predicted chloride content and depth - w/c 0.45,  $D_{cc} 4.5 \times 10^{-13} \text{ m}^2/\text{s}$ ,  $C_s$  0.55% by mass of concrete. Non-CEM I concretes.



**Figure 3.18:** Relationship between predicted chloride content and depth - w/c 0.50,  $D_{cc} 6.0 \times 10^{-13} \text{ m}^2/\text{s}$ ,  $C_s$  0.55% by mass of concrete. Non-CEM I concretes.

**Table 3.7:** Minimum qualities and minimum covers for CEM II/B-S, II/A-D, II/B-V<sup>+</sup>, III/A and III/B concretes subject to exposure class XS3.

Concrete	Minimum grade <sup>+</sup> (N/mm <sup>2</sup> )	Maximum water-cement ratio	Minimum cover (mm)	
			50 year design life	100 year design life
In-situ, all surfaces except unprotected horizontal top surfaces	C 35/45	0.50	50	70
	C 40/50	0.45	45	60
In-situ, unprotected horizontal top surfaces	C 35/45	0.50	65	85
	C 40/50	0.45	60	75
Precast	C 35/45	0.50	40	60
	C 40/50	0.45	35	50

<sup>+</sup> For II/B-V concretes, the minimum grade may be reduced by one grade.

### 3.7 Chloride ingress: marine exposure class XS1

Deterioration of reinforced concrete elements is more severe in coastal areas than away from the coast. This is due to salt-laden air which may extend inland for up to 3 km (Nanni and Lista, 1988). According to data reported by ISO (1983) given by Moringa (1992), the concentration of air-borne salt falls sharply with the distance from the sea (Table 3.8).

**Table 3.8:** Variation of salt concentration with distance from the sea.

Distance from the sea (m)	0	50	100	200	500	1000	2000
Ratio of concentration of air-borne salt to that at m = 0	100	50	25	12	7	5	4

The data in the literature on the effect of salt-laden air upon the performance of reinforced concrete is sparse and is summarized below:

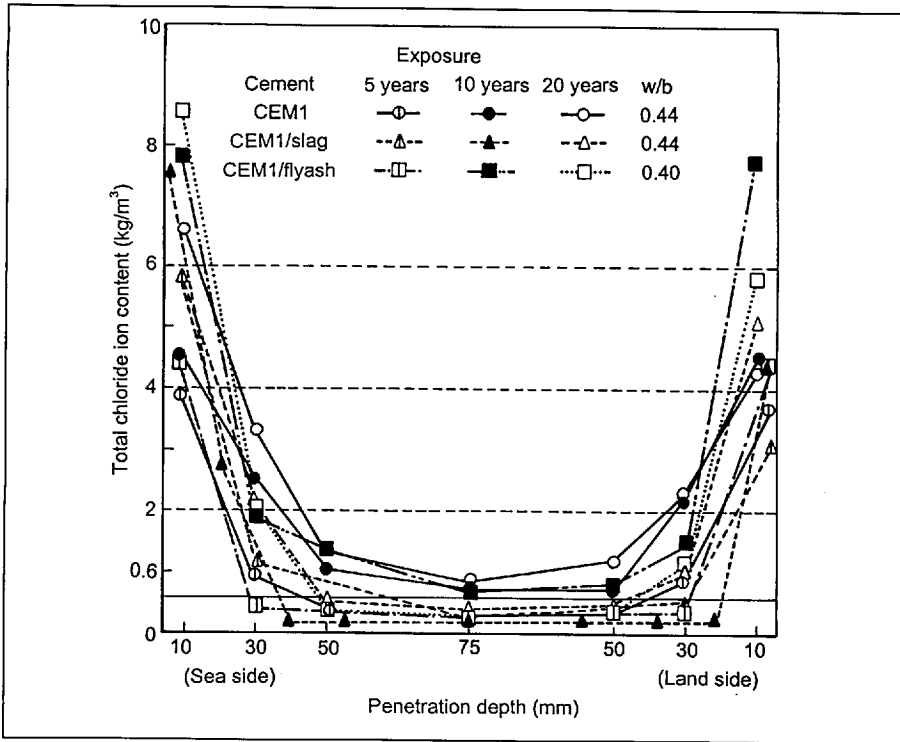
**Whiting and Stejskal (1993).** These authors examined two bridges approximately 30 years of age in a temperate marine site (the Astoria Bridge and Devil's Slide viaducts in Oregon, USA) and note:

"The Oregon coastal bridges are subject to salt-laden fogs which deposit small quantities of chlorides and other salts on the concrete surfaces. In addition, those bridges at Devil's Slide are within a few hundred feet of the coast, where aerosolized salts can be borne by prevailing winds onto the surfaces of the structure. In spite of these exposures, both bridges were in very good condition. While there has been much concern with the condition of the Astoria Bridge, as it is the only land-link between Washington and Oregon for 50 miles (80 km) in this area, most of the problems could be traced to defects encountered during construction, and not to progressive deterioration due to corrosion of the prestressing tendons. A penetrating sealer was applied to the beam surfaces of the Astoria Bridge in the mid-1980s, which helped to slow the ingress of chlorides into the beams. Chlorides at the level of the tendons, approximately 2 in. (50 mm) below the surface, were essentially at baseline level. While the pretensioned box beams at Devil's Slide had not been sealed, here also chlorides were at baseline level at the depth of the strands."

**Nanni and Lista (1988).** These authors discuss the cracking of reinforced concrete structures in coastal zones in the USA where the aggressive action of salt-laden air is the primary cause of concrete deterioration. They attribute the deterioration to a coastal environment in combination with improper cover, misplaced reinforcement, improper handling and placing of the concrete, seawater contaminated aggregates and improper mix proportions.

**Ohta (1991).** Beams were wet cured for a week and dried in laboratory air for three weeks and then cracked. The beams were exposed for up to 20 years at a "height of 1 to 4 m, 30 to 40 m from the beach line in Rumoi which faces the sea of Japan". Three cements were used, a CEM I cement, a CEM II/A-V cement containing between 10 to 20% fly ash and a slag cement containing between 30 to 60% slag. Results reported by Ohta of total chloride ion concentration with depth are plotted in Figure 3.19. The diffusion coefficients quoted by Ohta are:

CEM I	w/c 0.44	$1.8 \times 10^{-8}$ (no units given)
slag	w/c 0.44	$0.5 \times 10^{-8}$
CEM II/A-V	w/c 0.41	$0.6 \times 10^{-8}$



**Figure 3.19:** Rate of chloride ion penetration into concrete (Ohta, 1991).

At depths in excess of 30 mm, the total chloride ion concentration in the CEM I concretes were more than double that in the other two concretes.

**Moringa (1992).** Forty six specimens were exposed to various sea environments around Japan (no further detail given). The mean surface chloride ion concentration and the diffusion coefficient were 0.3% by mass of concrete and  $4 \times 10^{-12} \text{ m}^2/\text{s}$ . Assuming onerous values of  $1.0 \times 10^{-11} \text{ m}^2/\text{s}$  for the diffusion coefficient and 0.72, 0.36 and 0.18% by mass of the concrete for the surface chloride ion concentrations at distances 0, 50 and 100 m from the sea respectively, Moringa gives the maximum water-cement ratios for minimum reinforcement covers of 25, 50 and 75 mm and service lives of 50 and 100 years (see Table 3.9). It is not possible to check the recommendations made by Moringa, as the information is incomplete.

**Jaegermann (1990).** The author reports that many of the exposed concrete façades of buildings along Israel's shoreline show cracking and spalling caused by corrosion of reinforcing steel. In Israel the concrete is exposed to hot dry summers and "rainy winters". The Israel Code of Practice specifies a concrete cover of 30 mm and a maximum water-cement ratio of 0.55 for "buildings near the sea". To meet this water-cement ratio requirement, designers specify a characteristic cube strength of  $30 \text{ N/mm}^2$ . Jaegermann exposed concrete plates of four water-cement ratios to exposure to airborne salt at a height of 15 m and a distance of 30 m from the sea shore. Chloride penetration profiles after three years exposure are given in Figure 3.20. The profiles generally show a peak at about 10 to 20 mm from the concrete surface. The chloride content, either by mass of cement or concrete, decreases sharply as the water-cement ratio is

**Table 3.9:** Maximum water-cement ratios (after Moringa).

Cover thickness (mm)	Required life (years)	Distance from the sea (m)								
		0			50			100		
		Relative humidity (%)								
		90	70	50	90	70	50	90	70	50
25	50	N	N	N	N	N	0.43	N	0.41	0.49
	100	N	N	N	N	N	N	N	N	0.41
50	50	0.47	0.52	>0.60	0.55	>0.60	>0.60	>0.60	>0.60	>0.60
	100	0.40	0.43	0.50	0.45	0.49	>0.60	0.49	0.57	>0.60
75	50	>0.60	>0.60	>0.60	>0.60	>0.60	>0.60	>0.60	>0.60	>0.60
	100	0.53	0.60	>0.60	>0.60	>0.60	>0.60	>0.60	>0.60	>0.60

Note : Cover thickness of 25 mm should not be specified.

reduced. Jaegermann states in the concluding remarks that “it may already be concluded that for a water-cement ratio of 0.5, concrete cover reinforcing steel should be more than 40 mm, and for a water-cement ratio of 0.4, more than 30 mm”.

**Roper, Kirby and Baweja (1986).** 136 buildings were surveyed in Sydney, Australia. Many of these buildings had probably been exposed to airborne salt. Approximately half of the structures were constructed using fly ash. The authors concluded from the results of their inspections that "Cracking appears equally in structures constructed with fly ash and OPC concretes. There appears to be no evidence that buildings which were constructed using fly ash concretes in Sydney are in any better or worse condition than if they had been constructed using OPC concrete".

**Salta (1994).** Concrete prisms, 100 x 100 x 200 mm in size, were prepared with a water-cement ratio of 0.6 and fly ash contents of 0, 15, 30 and 50% by mass. The prisms were moist-cured for seven days and then exposed 50 m from the sea at Roca Cap, Portugal, on the Atlantic coast, an area with many fog banks. The chloride contents at three years are compared in Figure 3.21. The profiles for the prisms subject to airborne salt show a peak at depths between 15 and 25 mm, and below a depth of 30 mm the chloride level increases with increasing fly ash content, a reversal of that observed on similar concretes stored in the tidal zone. At depths greater than 30 mm, the chloride level is lowest for the concretes containing 30% fly ash.



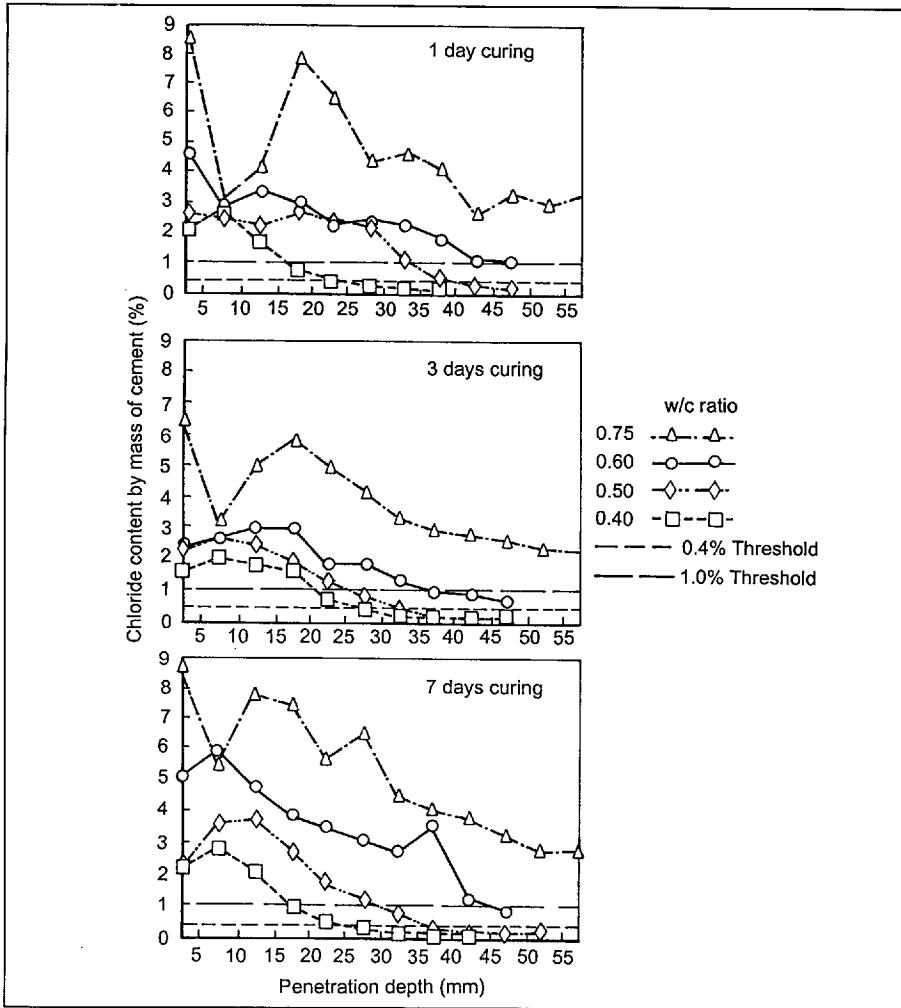


Figure 3.20: Chloride profiles after three years' exposure (Jaegermann, 1990).

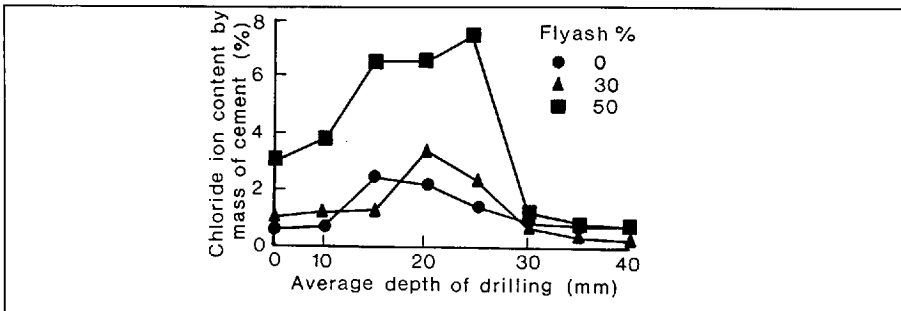


Figure 3.21: Chloride concentration profiles for prisms stored 50 m from sea. w-c ratio 0.6. Age three years (Salta, 1994).

### 3.8 Minimum specification for concrete subject to exposure class XS1

A minimum specification for concrete subject to exposure class XS1 is given in Table 3.10. Due to lack of published data it is not possible to distinguish between the performance of different cements in concretes subject to exposure class XS1. Up to distances of 100 m from the sea it is proposed that the minimum specification should be the same as for XS3 (Tables 3.6 and 3.7) but for sea distances of between 100 and 3000 m it is proposed that both lower covers together with lower concrete grades can meet the desired 50 to 100 year life.

**Table 3.10:** Minimum qualities and minimum covers for CEM I, CEM I/SR, II/A-S, II/B-S, II/A-D, II/A-V, II/B-V<sup>+</sup>, III/A and III/B. Concretes subject to exposure class XS1 with distance from sea ranging from 100 to 3000 m.

Concrete	Minimum grade (N/mm <sup>2</sup> ) <sup>+</sup>	Maximum water-cement ratio	Minimum cover (mm)	
			50 year design life	100 year design life
In-situ, all surfaces except unprotected horizontal top surfaces	C30/37	0.55	45	60
	C35/45	0.50	40	55
In situ, unprotected horizontal top surfaces	C30/37	0.55	65	80
	C35/45	0.50	60	75
Precast	C35/45	0.50	30	45
	C40/50	0.45	25	40
	C50/60	0.40	20	35

<sup>+</sup> For II/B-V concretes, the minimum grades may be reduced by one grade.

### 3.9 Chloride ingress into submerged concrete: marine exposure class XS2

Chloride ingress into concrete permanently immersed in sea water occurs more rapidly than into comparable concrete in the splash or tidal zone (Gourley and Bierniak, 1983; Roy et al 1993; Polder and Larbi, 1996). However, although the steel may lose its passivity, the corrosion rates can be negligible due to the low concentrations of dissolved oxygen in sea water. However, if drying occurs from any face of a submerged concrete, for example a concrete tunnel lining, then attempts should be made to minimize excessive rates of corrosion by design. One possibility could be to ensure that any reinforcement is in a saturated region of the tunnel lining hence ensuring that the corrosion rate is minimized by the low concentration of oxygen in the sea water.

### 3.10 Minimum specification for concrete subject to exposure class XS2A

The minimum specification for concrete subject to submerged marine exposure on all faces (XS2A) is governed by its resistance to sea water attack, rather than its resistance to deterioration due to chloride-induced corrosion. Marine exposure is taken to be equivalent to Class 2 (XS2A) exposure (BSI, 1997c). The resulting minimum specification taken from BS 5328: Part 1: 1997 for Class 2 exposure is given in Table 3.11 (BSI, 1997d). It is recommended that the minimum cover is 40 mm.

**Table 3.11:** Maximum water-cement ratios and minimum cement contents for concretes subject to exposure class XS2A.

Cement	Fully compacted concrete made with 20 mm nominal maximum size aggregate	
	Cement content not less than (kg/m <sup>3</sup> )	Free water-cement ratio not more than
CEM I, II/A-S, II/B-S, II/A-D, II/A-V, II/B-V, III/A, III/B	330	0.50
II/B-V (flyash $\geq$ 25% by mass of nucleus), III/B (slag $\geq$ 74% by mass of nucleus), III/C (slag $\leq$ 90% by mass of nucleus)	300	0.55
CEM I/SR	280	0.55

### 3.11 Chloride ingress: de-icing salt exposure class XD3

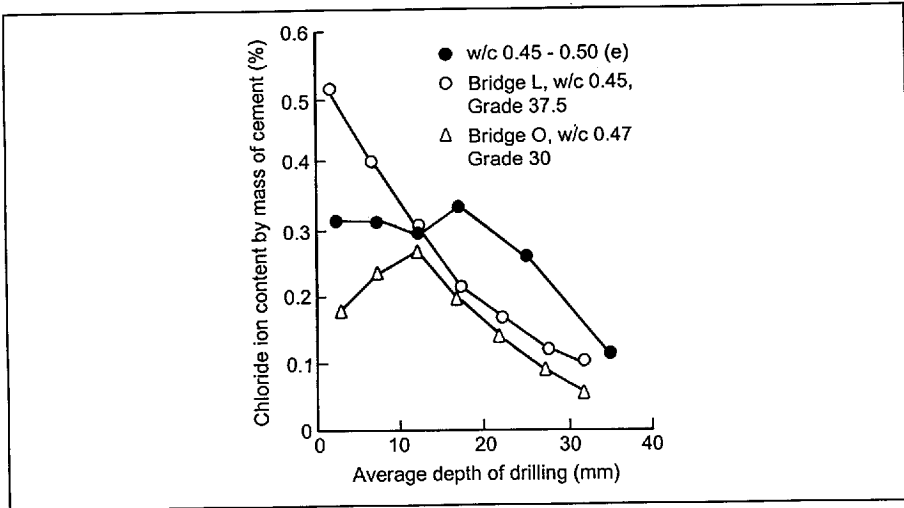
#### 3.11.1 CEM I concretes

Chloride ingress into the surface layers of concrete intermittently exposed to salt-laden water is rarely the result of a simple diffusion process. Chloride ingress into the cover concrete will often occur by capillary suction, and wash-out of some of the chlorides can occur when the concrete is exposed to rain or non-salt-laden vehicle spray. Ingress of chloride ions into the concrete will also be complicated by the process of carbonation which will change the bound chloride level and the permeability of the affected concrete. Carbonation can reduce or increase the permeability of the surface layers, depending upon the binder type (Parrott, 1987). The influence of carbonation will be most marked in parts of structures protected from direct rain. For example, in the case of bridge piers, it may be a year or two before they are subject to their first significant vehicle spray. For such elements, significant drying and carbonation of the cover concrete is possible.

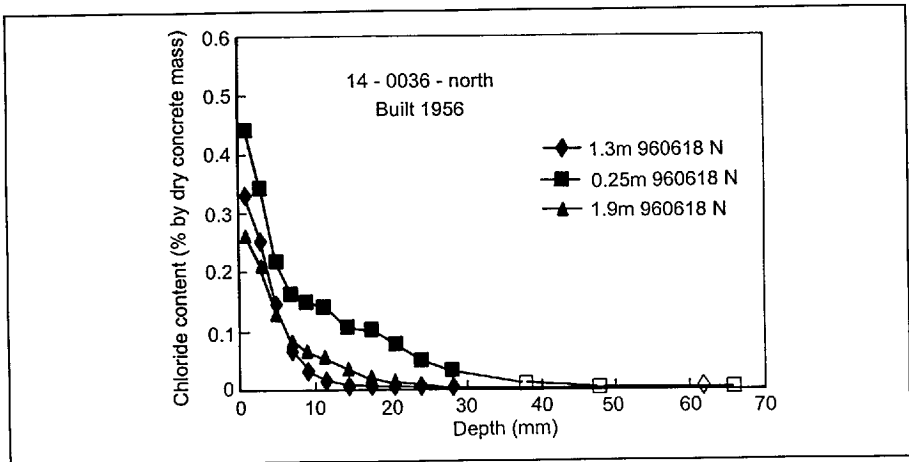
Measurements of total chloride ingress into concrete subject to intermittent exposure to salt-laden water or vehicle spray have been made by a number of investigators: (Brown, 1987; Thomas, 1989; Funahashi, 1990; Thomas, 1990; Henriksen and Stoltzner, 1993; Stoltzner, 1993; Vassie, 1995; Bamforth, 1997; TRL, 1997 and Anderson, 1997). Some of the results obtained on CEM I concretes are plotted in Figures 3.22 and 3.23. At short distances into the concrete, a departure from the chloride ion profile predicted by simple diffusion is sometimes observed.

Fitting Fick's second law of diffusion to the reported chloride levels in 'CEM I bridge elements' gives the following values for the apparent ' $D_{ce}$ ' and  $C_s$ :

- Brown (1987): 10 bridges constructed from 1961/63 to 1971; in sound regions ' $D_{ce}$ ' from  $5.5 \times 10^{-13} \text{ m}^2/\text{s}$  to  $8.0 \times 10^{-12} \text{ m}^2/\text{s}$  and corresponding  $C_s$  of 0.11 and 0.07% by mass of in-situ concrete respectively (Table 3.12). The water-cement ratio of these concretes were estimated to be 0.45 and 0.50. In two deteriorating precast concrete elements, a ' $D_{ce}$ ' of  $2.5 \times 10^{-13} \text{ m}^2/\text{s}$  and  $6.8 \times 10^{-13} \text{ m}^2/\text{s}$  and corresponding  $C_s$  of 0.70 and 0.22% by mass of concrete respectively (Table 3.12). In these latter examples, the concrete elements were exposed to leaks from the motorway bridge deck above. Brown (1987) concluded that all but two of the areas of



**Figure 3.22:** Chloride concentration profiles for bridge piers exposed to de-icing salt. (Vassie, 1995, distance to slow traffic lane 4.75 to 5.0 m, height 1 m, age five years). (Thomas, 1989, age ten years, e = estimated).



**Figure 3.23:** Chloride concentration profiles for bridge piers exposed to de-icing salt. Age 40 years. (Anderson, 1997).

concrete examined had spalled (Figure 3.24) where the chloride ion concentration at the reinforcement level exceeded 0.5% by mass of cement.

- Henriksen and Stolzner (1993): 20 undamaged bridge piers constructed between 1940 and 1990, ' $D_{ce}$ ' from 0.1 to  $1.8 \times 10^{-12} \text{ m}^2/\text{s}$  with a mean of about  $1.2 \times 10^{-12} \text{ m}^2/\text{s}$ , and a corresponding  $C_s$  from 0.02 to 0.17% by mass of concrete with a mean of about 0.09%. Henriksen and Stoltzner (1993) attributed this variability to "varying concrete qualities from bridge to bridge".
- Stolzner (1993): a bridge built in 1970, ' $D_{ce}$ ' of  $1.4 \times 10^{-13} \text{ m}^2/\text{s}$  with  $C_s$  of 0.03% respectively by mass of concrete.

**Table 3.12:** Summary of observations made by Brown (1987).

Concrete	Age (yrs)	Subject to leak	Condition	w-c <sup>†</sup> ratio	Cover (mm)	C <sub>s</sub> (% by mass of concrete)	Cl at bar (% by mass of concrete)	D <sub>c</sub> (m <sup>2</sup> /s)
SA, pier	22	No	Sound	0.57	27	0.072	0.055	5.7 x 10 <sup>-12</sup>
SB, pier	22	No	Sound	0.50	25	0.103	0.054	1.7 x 10 <sup>-12</sup>
TA, pier	22	No	Sound	0.51	28	0.052	0.019	-
TB, pier	22	No	Sound	0.43	18	0.144	0.055	-
GA, crosshead	21	Yes	Spalling	0.52	15	0.338	0.124	4.0 x 10 <sup>-13</sup>
GB, crosshead	21	No	Sound	0.48	25	0.022	0.014	-
GC, crosshead	21	No	Sound	0.49	42	0.195	0.047	8.7 x 10 <sup>-13</sup>
GD, crosshead	21	Yes	Spalling	0.46	48	0.454	0.134	1.0 x 10 <sup>-11</sup>
GE, crosshead	21	Yes	Spalling	0.51	22	0.870	0.316	4.3 x 10 <sup>-12</sup>
HA, crosshead	21	Yes	Sound	0.49	45	0.589	0.155	3.8 x 10 <sup>-12</sup>
HB, crosshead	21	No	Sound	0.50	43	0.071	0.033	8.0 x 10 <sup>-12</sup>
QA, abutment	20	Yes	Sound	0.59	-	0.167	-	6.7 x 10 <sup>-12</sup>
LA, abutment	19	Yes	Sound	0.45	-	0.106	-	5.5 x 10 <sup>-13</sup>
JA, abutment	11	Yes	Spalling (frost?)	0.50	25	0.204	0.123	5.3 x 10 <sup>-12</sup>
JB, abutment	11	No	Sound	0.49	41	0.011	0	-
JC, precast abutment	11	No	Sound	0.32	22	0.112	0.059	1.9 x 10 <sup>-12</sup>
JD, precast abutment	11	Yes	Spalling	0.34	12	0.703	0.134	2.5 x 10 <sup>-13</sup>
JE, abutment	11	Yes	Sound	0.37	31	0.143	0.040	1.7 x 10 <sup>-12</sup>
KA, precast abutment	11	Yes	Spalling	0.34	12	0.220	0.091	6.8 x 10 <sup>-13</sup>
KB, precast abutment	11	Yes	Sound	0.39	28	0.370	0.028	4.7 x 10 <sup>-13</sup>

<sup>†</sup> w-c ratio of original mix is deduced from the capillary porosity and the cement content and according to Brown (1987), the deduced values for w-c ratio are likely to be biased towards low values.

- Thomas (1990): a 10 year old bridge, 'D<sub>cc</sub>' 3.1 x 10<sup>-12</sup> and C<sub>s</sub> 0.06% by mass of concrete.
- TRL (Vassie, 1995): two bridges aged five years, D<sub>cc</sub> of 1.7 x 10<sup>-12</sup> and 2.7 x 10<sup>-12</sup> m<sup>2</sup>/s and C<sub>s</sub> of 0.09 and 0.045% by mass of concrete respectively.
- Funahashi (1990): in the case of car park T-beams, 'D<sub>cc</sub>' from 1.3 to 1.8 x 10<sup>-12</sup> m<sup>2</sup>/s and C<sub>s</sub> from 0.08 to 0.26% by mass of concrete. In this particular case, wash-out of chlorides was unlikely and this may account for the high C<sub>s</sub> of 0.26%.
- Bamforth and Al-Isa (1997): nine year old concrete blocks exposed adjacent to bridge piers in the Midlands of the UK, 'D<sub>cc</sub>' 5.2 x 10<sup>-12</sup> m<sup>2</sup>/s and C<sub>s</sub> 0.33% by mass of concrete, for concrete with a w-c ratio of 0.66 and 'D<sub>cc</sub>' 3.1 x 10<sup>-12</sup> m<sup>2</sup>/s and C<sub>s</sub> of 0.36% by mass of concrete for concrete with a w-c ratio of 0.62. These deductions relate to cores taken in February 1997.
- Anderson (1997): areas of bridge columns supporting four bridges in Denmark rarely exposed to direct rain, three of these bridges had previously been examined by Henriksen and Stolzner (1993):

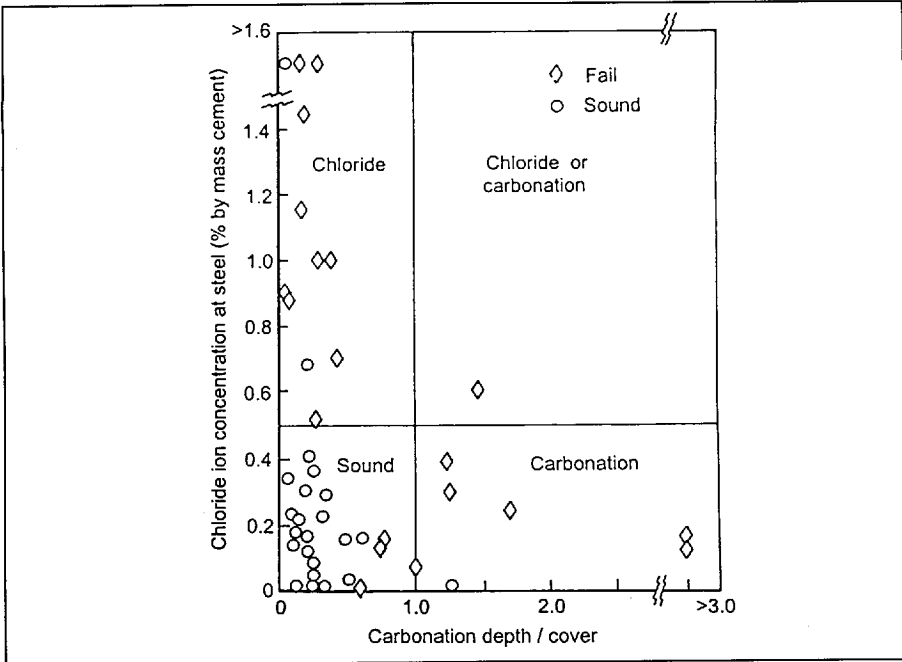


Figure 3.24: Criteria for carbonation or chloride-induced corrosion (after Brown, 1987).

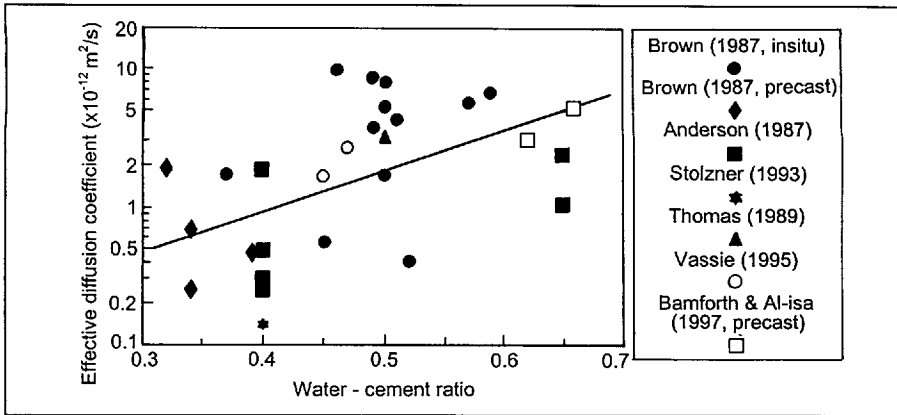
$D_{ce} 2.46 \times 10^{-13} \text{ m}^2/\text{s}$	$C_s 0.19\%$	Bridge 14.0036	Core 3N, 40 years
$D_{ce} 2.61 \times 10^{-13} \text{ m}^2/\text{s}$	$C_s 0.16\%$	Bridge 14.0036	Core S, 40 years
$D_{ce} 3.16 \times 10^{-13} \text{ m}^2/\text{s}$	$C_s 0.04\%$	Bridge 14.0036	Core N1, 40 years
$D_{ce} 2.38 \times 10^{-12} \text{ m}^2/\text{s}$	$C_s .06\%$	Bridge 20.0085	Core 3, 33 years
$D_{ce} 1.02 \times 10^{-12} \text{ m}^2/\text{s}$	$C_s 0.12\%$	Bridge 20.0085	Core 4, 33 years
$D_{ce} 4.90 \times 10^{-13} \text{ m}^2/\text{s}$	$C_s 0.18\%$	Bridge 10.0031	Core 2, 28 years
$D_{ce} 1.86 \times 10^{-12} \text{ m}^2/\text{s}$	$C_s 0.05\%$	Bridge 30.0016	Core 1, 24 years

These deductions relate to cores taken in the spring of 1996 and to measurements at depths greater than 5 mm. It is estimated that the w-c ratio of bridge 20.0085 is about 0.65 and of the others 0.40 (Henriksen and Stolzner, 1993). During the winter months in Denmark, de-icing salt is often applied to road surfaces, sometimes once a day. Consequently, the salt exposure experienced by these bridges is probably more severe than would be the case in the UK.

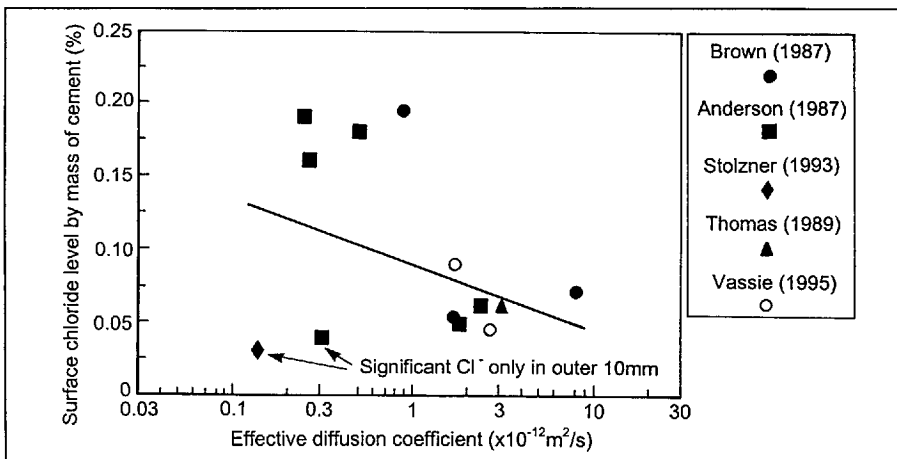
In Figure 3.25, the effective diffusion coefficient is plotted against water-cement ratio for CEM I concretes. The best fitting equation is

$$D_{ce} = 0.06 (906^{w/c}) \times 10^{-12} \quad (r = 0.275) \quad (3.4)$$

At a water-cement ratio of 0.45, the best fitting value for  $D_{ce}$  is  $1.3 \times 10^{-12} \text{ m}^2/\text{s}$ . In Figure 3.26, the surface chloride level is shown plotted against the effective diffusion coefficient for concretes with w-c ratios between 0.40 and 0.50, exposed to de-icing salt spray but not to leaks from a bridge deck. Effective diffusion coefficients above  $1 \times 10^{-12} \text{ m}^2/\text{s}$  correspond with surface chloride levels below 0.1% by mass of concrete.



**Figure 3.25:** Relationship between effective diffusion coefficient and w-c ratio. De-icing salt exposure.



**Figure 3.26:** Relationship between effective diffusion coefficient and surface chloride level. De-icing salt exposure.

### 3.11.2 CEM II/B-V and CEM III/A concretes

Measurements of total chloride ingress into non-CEM I concretes subject to intermittent exposure to salt-laden water or vehicle spray have been made by several investigators (Thomas, 1989; Vassie, 1995; Bamforth and Al-Isa, 1997).

The profiles of chloride ingress into bridge abutments obtained by Vassie (1995) and Thomas (1989), are plotted in Figures 3.27 and 3.28 and into concrete blocks adjacent to a bridge pier, obtained by Bamforth and Al-Isa (1997), in Figure 3.29 together with the profiles obtained on companion CEM I concretes. In the case of concretes containing fly ash or slag, there can, at short distances into the concrete, be a marked departure from the chloride profile predicted by simple diffusion. An examination of Figures 3.27 and 3.28 shows, that at depths less than about 30 and 20 mm, the total chloride ion level in CEM III/A and CEM II/B-V concretes respectively can be higher than for a CEM I concrete, whilst at greater depths, the total chloride ion level can

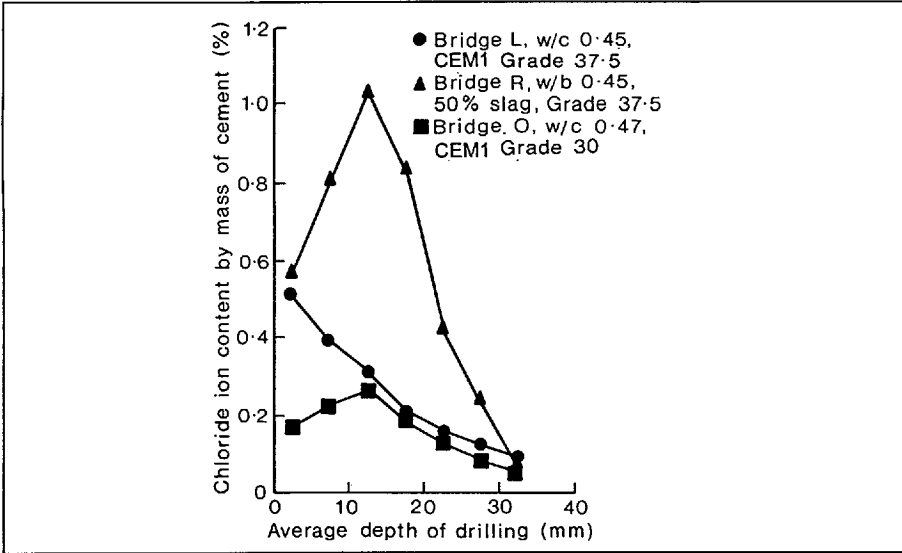


Figure 3.27: Chloride concentration profiles for bridges piers exposed to de-icing salt. Height 1 m. Distance to nearest traffic lane 4.75 to 5.0 m. Age 5 years (Vassie, 1995).

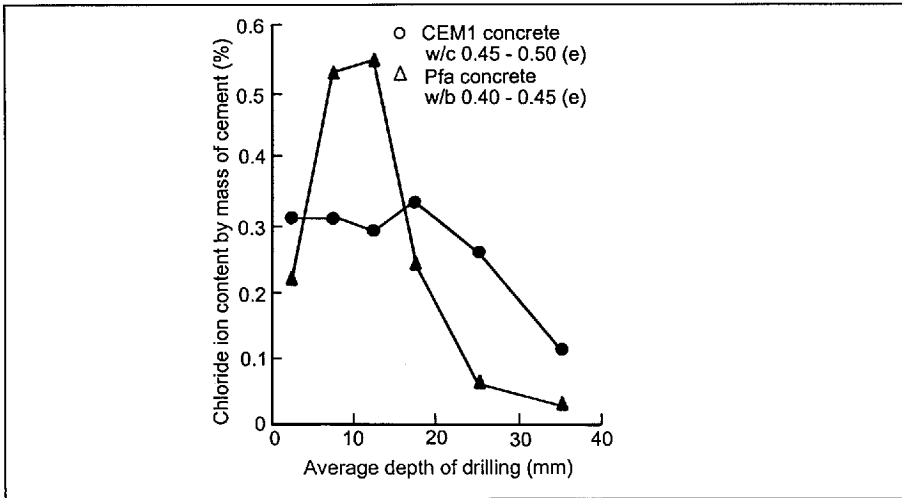
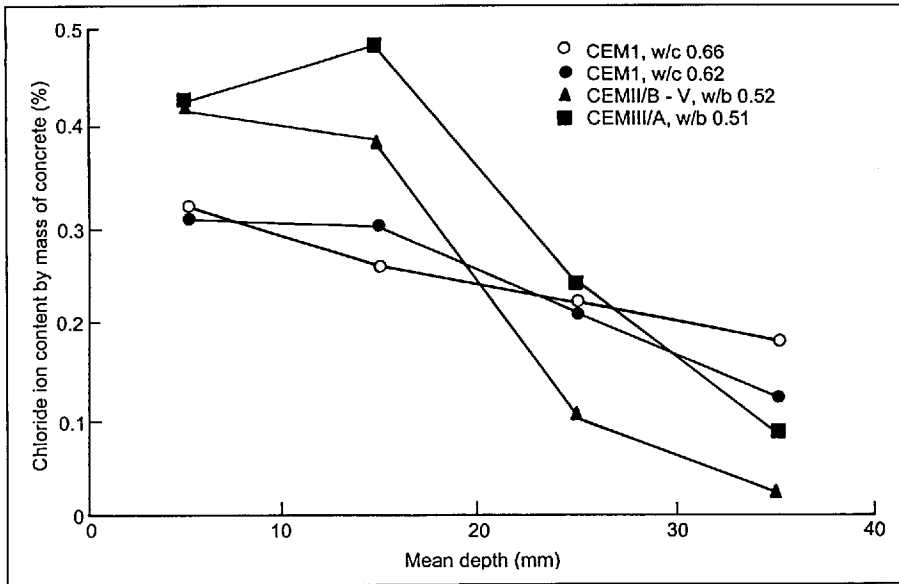


Figure 3.28: Chloride concentration profiles for bridge piers exposed to de-icing salt. Age 10 years (Thomas, 1989). (e = estimated).

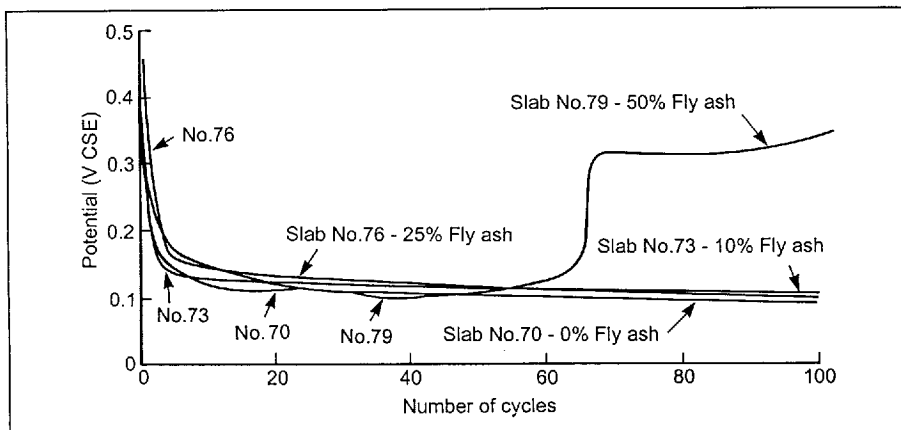
be similar (CEM III/A) or lower (CEM II/B-V) than for a CEM I concrete. In the case of the data shown in Figure 3.29, such judgements are not possible, due to the widely differing w-b ratios of the concretes.



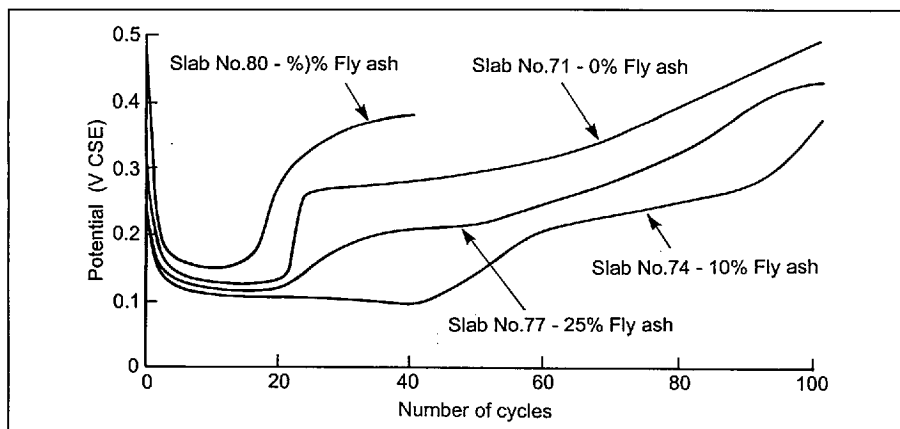


**Figure 3.29:** Chloride concentration profiles for concrete blocks exposed to de-icing salt. Age 9 years. (Bamforth and Al-Isa, 1997).

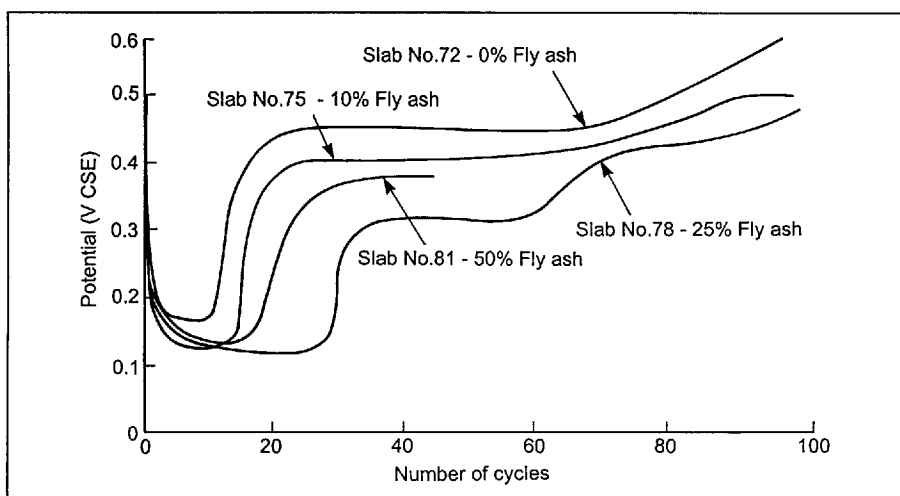
Recently Stark (1995) has published the results of chloride ponding tests which indicate that the effect of fly ash upon chloride-induced corrosion may be complex and may depend upon both w-c ratio and the proportion of fly ash employed (see Figures 3.30 to 3.32). In Stark's tests, concrete slabs of three w-c ratios and a reinforcement cover of 25 mm, were moist cured for 28 days and then subjected to a two week exposure cycle. It consisted of 10 days of drying at 20°C and 50% RH, followed by four days of ponding the top surface of the slabs with 4% sodium chloride solution. The first cycle was begun with the 10-day drying period following immediately after the curing period. Performance was monitored by electrical potential measurements. (Note: this is a particularly severe test as in practice some washout of chlorides will occur.)



**Figure 3.30:** Influence of fly ash on corrosion resistance of mild steel in 0.35 w-c ratio concrete (Stark, 1995).



**Figure 3.31:** Influence of fly ash on corrosion resistance of mild steel in 0.45 w-c ratio concrete (Stark, 1995).



**Figure 3.32:** Influence of fly ash on corrosion resistance of mild steel in 0.60 w-c ratio concrete (Stark, 1995).

### 3.12 Service life: exposure class XD3

Table 3.13 summarizes the observations made in a number of field studies on reinforced and prestressed concretes, excluding concrete elements of car parks subject to de-icing salt exposure. All of the concretes referred to are CEM I concretes. Examination of the information in the table illustrates that, with the exception of concrete exposed to leakage from the bridge deck, the quality of concrete as given by w-c ratio is a major factor influencing the risk of chloride-induced corrosion and cracking.

Service life predictions require a knowledge of the diffusion coefficient, the surface chloride level, the free chloride level at the concrete-steel interface at which corrosion commences, and of the propagation period.

**Table 3.13:** Performance of concrete structures subject to de-icing salt exposure.

Investigator	Structures	When built	Concrete strength (N/mm <sup>2</sup> )	'Intended' cover (mm)	Water-cement ratio	Condition
Bashenini et al (1994)	Desalination plant (SRPC), Middle East	1982	21	50 (25 where spalling)	0.7 <sup>+</sup>	Concentrated brine attack. Cracking and spalling at 10 years
Thomas (1989)	In-situ bridge pier	1978	49	-	0.45-0.50 <sup>+</sup>	At depth 35 mm, low chloride level at 10 years
Vassie (1995)	In-situ bridge piers	1985	C37.5 C30	- -	0.45 0.47	At depth 25 mm low chloride levels at 5 years
Stolzner (1993)	Bridge piers, Denmark	1970	-	30 to 40	0.40-0.45	At 21 years negligible chlorides below 10 mm
Henriksen et al (1993)	Columns of 20 bridges, Denmark	1940-90	-	30 to 40	0.40	'Predicts' 50 year life
Author's examination (1995)	Central bridge piers MI, UK	About 1960	C45?	25 to 28	0.45 (2:1:1 mix)	At 35 years condition good apart from some minor spalling where cover low (probably <10 mm)
Somerville (1995)	Precast UK pretensioned members	From early 1940s	60 to 80	>21 to 40	0.35 - 0.42	A history of good performance
Clark (1992)	Precast post-tensioned members	From early 1940s	-	-	Probably 0.35-0.42	A history of good performance
Highways Agency, UK (1995)	Precast pre-tensioned members	-	C40 C50	>45 >35	Probably 0.35-0.42	Have generally proved to be durable
Brown (1987)	Elements of 8 bridges, UK	1961 to 1972	-	12 to 48	0.32 - 0.57	Spalling where de-icing salt solution drained from deck across crossheads and tops of columns. Otherwise sound
Anderson (1997)	Bridge columns. 5 bridges. Denmark	1956, 1956, 1968, 1972 1963	-	30 to 40 30 to 40 30 to 40 30 to 40 30 to 40	0.40-0.45 0.40-0.45 0.40-0.45 0.40-0.45 ≈0.65 <sup>+</sup>	Good, Cl <sup>-</sup> level at 30 mm, ≤0.02%; ≤0.05%; ≤0.07%; ≤0.04%  Good but Cl <sup>-</sup> up to ≤0.08% at 30 mm
Bamforth and Al-Isa (1997)	Precast blocks	1988	41 48	10 to 40 10 to 40	0.66 0.62	Cl <sup>-</sup> 0.18% at 30 mm depth Cl <sup>-</sup> 0.125% at 30 mm depth

<sup>+</sup> Estimated.

Assuming that:

- the total chloride level at which corrosion commences is 0.06% by mass of concrete (Henriksen and Stoltzner, 1993; Brown, 1987)
- w-c ratio is 0.45
- minimum cover is 40 mm
- $D_{cc}$  is  $2.0 \times 10^{-12}$  m<sup>2</sup>/s
- $C_s$  is 0.10% by mass of concrete
- equation 3.1 is applicable to CEM I concrete bridge elements subject to de-icing salt exposure

gives an initiation period of approximately 50 years before corrosion commences and a service life of possibly 60-75 years. In the authors' view, the assumption of a propagation period prior to cracking and spalling of between 10 to 25 years (Henriksen and Stoltzner, 1993) may be an underestimate for low w-c ratio concretes with covers greater than 30 mm. The performance of low cover bridge pier concrete indicates a possible propagation period of 20 to 30 years.

The above service life prediction was for CEM I concrete. This prediction is based on limited data. In the case of concretes containing fly ash and slag, the data is more sparse. The results obtained by Thomas (1989) and Vassie (1995) indicate higher surface chloride levels but lower effective diffusion coefficients than CEM I concretes. The net effect is broadly similar total chloride levels at depths of about 30 mm for CEM III/A concretes and 15 mm for CEM II/B-V concretes. In the latter case, it should be noted that the w-c ratios of the concretes being compared were different. It follows that firm judgements as to whether or not concretes containing fly ash and slag give longer or shorter service lives for exposure class XD3 than CEM I concretes cannot yet be made.

### 3.13 Minimum specification for concrete subject to external class XD3 exposure

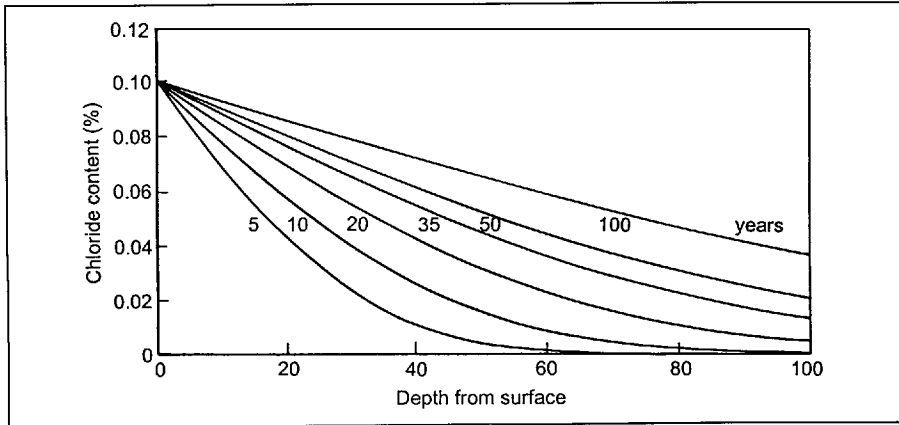
#### 3.13.1 Assumptions based on field performance: CEM I concretes

1. Fick's second law is valid for CEM I concretes subject to external class XD3 exposure.
2. For in-situ CEM I concrete elements of w-c ratio 0.45 subject to external class XD3 exposure the **mean** surface chloride concentration is 0.1% by mass of the concrete and the diffusion coefficient is  $2 \times 10^{-12}$  m<sup>2</sup>/s.
3. After the **total** chloride level at the steel reaches 0.06% by mass of concrete the propagation period to visual cracking is 25 years. Here it is being assumed that the propagation period is independent of bar size and cover.

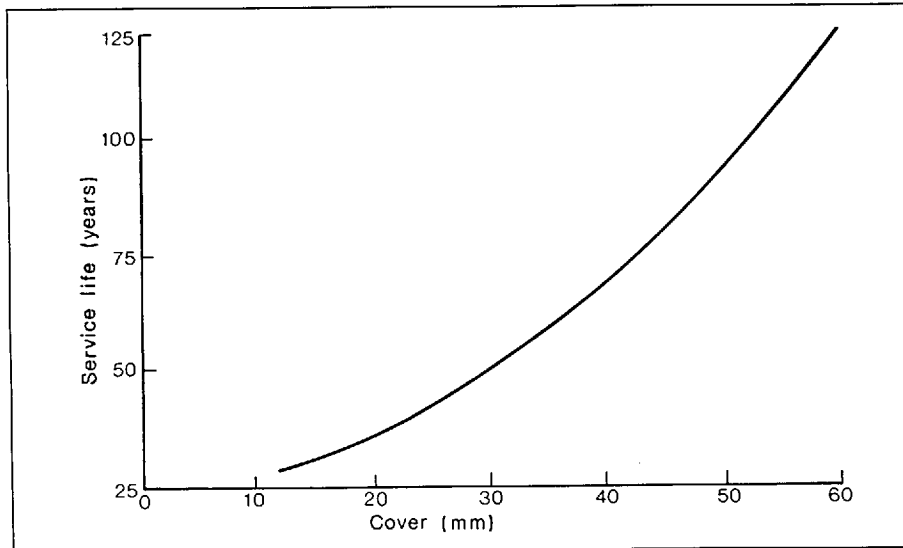
#### 3.13.2 Minimum specification: CEM I concretes

In Figure 3.33 the distributions of total chloride ion calculated from assumptions 1 and 2 are given for in-situ concretes with a w-c ratio of 0.45. The likely service lives which follow from assumption 3 are plotted in Figure 3.34. Clearly when designing for service lives of 50 and 100 years, it is wise to design for longer service lives, ie. 75 and 125 years. It then follows from Figure 3.34 that design working lives of 50 and 100 years for 0.45 w-c ratio concrete elements can be met by minimum covers of 45 and 60 mm respectively.

For precast concrete, it is proposed that the above minimum covers should be reduced by 10 mm, and that for unprotected in-situ cast horizontal surfaces the minimum cover be increased by 15 mm. The resulting minimum quality and minimum covers for CEM I precast and in-situ concretes, subject to UK external class XD3 exposure are given in Table 3.14 for design working lives of 50 to 100 years. Due to the probability of little wash-out of chlorides, the recommendations should not be taken to be applicable to concretes subject to internal class XD3 exposure, eg. multi-storey car parks.



**Figure 3.33:** Relationship between predicted chloride content and depth -  $w/c$  0.45,  $D_{cc}$   $2.0 \times 10^{-12} \text{m}^2/\text{s}$ ,  $C_s$  0.1% by mass of concrete. CEM I concretes. Exposure XD3.



**Figure 3.34:** Dependence of predicted service life upon concrete cover. Exposure XD3. In-situ concrete.

The UK Highways Agency (1995) has reported that precast pretensioned concrete elements have generally proved to be durable. These elements have minimum covers greater than 35 mm when the concrete grade is C40/50. For a 100-year design life, the recommendation for precast concrete proposed in Table 3.14 is more onerous than the current Highways Agency Specification (1995, see Table 3.4). For a 50-year design life, the recommendation for in-situ concrete proposed in Table 3.14 is more onerous than would be deduced from much of the performance data reported in Table 3.13.

**Table 3.14:** Minimum qualities and minimum covers for CEM I concretes subject to external class XD3 exposure.

Concrete	Minimum grade (N/mm <sup>2</sup> )	Maximum water-cement ratio	Minimum cover (mm)	
			50 year design life	100 year design life
In-situ, all surfaces except unprotected horizontal top surface	C40/50	0.45	45	60
In-situ, unprotected horizontal top surfaces	C40/50	0.45	60	75
Precast	C40/50	0.45	35	50

### 3.13.3 Minimum specification: non-CEM I concretes

Due to the limited data on chloride ingress into concretes prepared using cements other than CEM I, and no reports of inferior or superior field performance to CEM I concretes, it is proposed that the limiting values in Table 3.14 are used for concretes prepared using CEM I/SR, CEM II/A-S, II/B-S, II/A-V, II/B-V, III/A and III/B cements.

## 3.14 Chloride ingress: exposure class XD1

The authors are not aware of any reported chloride ingress data into concretes exposed to air-borne chlorides other than sea water.

In the case of air-borne chlorides resulting from de-icing salt application to concrete roads it is proposed that for distances up to 10 m from salted highways, the minimum qualities of concrete should be the same as for XD3. For distances greater than 10 m the exposure to air-borne chlorides is probably minimal and consequently precautions to minimize such risk are unnecessary; a nominal minimum quality of concrete is suitable for this exposure class.

## 3.15 Chloride ingress into permanently submerged concrete: exposure class XD2A

Based on the observations made by Gourley and Bieniak (1983), Roy et al (1993) and Polder and Larbi (1996), chloride ingress into concrete immersed in chloride-bearing waters is likely to occur more rapidly than into concrete above the water level. However, although the steel may lose its passivity, the corrosion rate can be negligible **provided** the dissolved oxygen concentrations are low. Consequently, a nominal minimum quality of concrete will be suitable for this exposure class. However, if drying occurs from any face of a submerged concrete, for

example, a swimming pool, attempts should be made to minimize excessive rates of corrosion by design.

### 3.16 Assumptions made during analysis of available data

A number of assumption have been necessary within this chapter; some are more firmly based than others. Consideration of the findings of this chapter should include an assessment of these assumptions to check whether the safety margin they provide for chloride-induced corrosion of concrete structures in service is appropriate. The main assumptions made include:

- For all concretes subject to exposure class XS3 and for CEM I concretes subject to exposure class XD3, chloride ingress is described by Fick's law.
- The ratio of free chloride ion concentration in the pore solution to the total chloride concentration is independent of cement type and water-cement ratio (XS3 and XD3). This is a simplification; for example, the chloride binding capacity of a CEM I cement according to Schiessl and Raupach (1990) increases with  $C_3A$  content.
- The total chloride ion concentration at which corrosion commences is independent of cement type and water-cement ratio. Glass and Buenfeld's (1995) conclusion from a literature survey, that chloride binding had little effect on the total chloride threshold level, supports this assumption; the conclusions of other investigators do not. For example, according to Pettersson (1992) and Nilsson et al (1996), the chloride ion concentration at which corrosion is initiated is dependent upon the alkalinity of the pore solution. In the case of CEM I/SR cements, the reduced alkalinity (or reduced  $C_3A$  content) has been shown to reduce the initiation time to corrosion and increase the corrosion rate when compared with CEM I cements (Rasheeduzzafar, Entesham Hussain and Al-Saadoon 1992). However, the opposite effect has been observed for CEM I/SR cement of high  $C_4AF$  content (Byfors, 1990). In the case of fly ash and slag, their effects are not clearly established. Bamforth and Chapman-Andrews (1994), Breit and Schiessl (1995) and Thomas, Matthews and Haynes (1990), concluded that the incorporation of slag or fly ash into a cement had no effect on the total chloride level at which corrosion commences. However, in a continuing study by Thomas (1996) and Thomas and Matthews (1996), it was concluded that the total chloride level at which corrosion commences becomes progressively lower with increasing proportions of fly ash. Hansson and Sorrenson (1990) and Pettersson (1993) found that incorporating silica fume into a cement reduced the chloride threshold level.
- The propagation period to visual cracking after the total chloride ion concentration at the steel-concrete interface reaches 0.20% by mass of concrete for XS3 exposure and 0.06% for XD3 exposure, is 25 years for all concretes. According to Andrade and Alonso (1996) and Arya and Xu (1995) the propagation period is shorter for concretes containing fly ash and slag than for CEM I concretes. In the former case when the chloride ion concentration is about 0.5% by mass of cement and in the latter case when the chloride ion concentration is about 1% by mass of cement. Matthews (1994) observed similar corrosion rates at the same chloride level for CEM I, CEM II/B-V (PC/pfa) and II/B-L (PC/limestone) concretes subject to marine exposure when the concretes were prepared to a similar concrete slump and a similar binder content.
- For factory-produced CEM I (including CEM I/SR) concrete elements of w-c ratio 0.45, subject to tidal or splash zone exposure, the surface chloride concentration is 0.4% by mass of the concrete and the diffusion coefficient is  $10 \times 10^{-13} \text{ m}^2/\text{s}$ . Under laboratory conditions,

it has been observed that CEM I concretes prepared using cements of high  $C_3A$  content can have lower diffusion coefficients than CEM I/SR concretes. Interpolation of recent results published by Al-Khaja (1997) indicate that, for the particular concretes tested, which were prepared using cements with  $C_3A$  contents of 10.0 and 2.8% by mass of cement and which had water-cement ratios for both cements of 0.48 and 0.39, the difference in diffusion coefficient was equivalent to a difference in water-cement ratio of 0.03, ie. CEM I concrete w-c ratio 0.45  $\equiv$  CEM I/SR concrete w-c ratio 0.42. A substantially larger difference in water-cement ratio for equivalent diffusion performance has been reported by Rasheeduzzafar and Mukarram (1987) for concretes made using the same CEM I/SR cement and a CEM I of similar  $C_3A$  level to those tested by Al-Khaja. In contrast, Stark's (1995) observations (see Table 3.5) indicate a service life which declines with increasing  $C_3A$  content. In Stark's work, the CEM I/SR of zero  $C_3A$  content had an  $SO_3$  level of 3.0% by mass compared to 1.8% by mass for the CEM I/SR cement employed by Al-Khaja (1997) and Rasheeduzzafar, Dakhil and Mukarram (1987).

- For factory-produced CEM I (including CEM I/SR) concrete elements with a w-c ratio of 0.40, subject to tidal or splash zone exposure, the surface chloride concentration is 0.4% by mass of concrete and the diffusion coefficient is  $7 \times 10^{-13} \text{ m}^2/\text{s}$ .
- For factory-produced concrete elements of w-c ratio 0.45 made using CEM II/B-S, II/A-D, II/B-V, III/A and III/B cements subject to tidal or splash zone exposure, (XS3), the surface chloride concentration is 0.55% by mass of concrete and the diffusion coefficient is  $4.5 \times 10^{-13} \text{ m}^2/\text{s}$ . For similar concretes with a w-c ratio of 0.50, the diffusion coefficient is  $6.0 \times 10^{-13} \text{ m}^2/\text{s}$ .
- For in-situ CEM I concrete elements of w-c ratio 0.45 subject to exposure class XD3, the **mean** surface chloride concentration is 0.1% by mass of the concrete and the diffusion coefficient is  $2 \times 10^{-12} \text{ m}^2/\text{s}$ .
- The difference in quality of in-situ and factory-produced concrete is equivalent to a difference in minimum cover of 10 mm.

### 3.17 Minimum requirements for concrete to resist chloride-induced corrosion

#### 3.17.1 Chlorides other than sea water

Based on this study, the minimum requirements for concretes to resist chloride-induced corrosion from exposure to de-icing salts for 50 and 100 years are given in Tables 3.15 and 3.16. The minimum requirements may not be applicable to the concrete elements of car parks. Minimum requirements are not given for concrete subject to sub-class XD2B exposure.



**Table 3.15:** Limiting values for composition and properties of concrete exposed to risk of corrosion of reinforcement induced by chlorides from sources other than sea water for an intended working life of at least **50 years**.

Exposure class	Moderate humidity	Wet, rarely dry		Cyclic wet/dry
	XD1	XD2A	XD2B	XD3 <sup>+</sup>
Minimum strength class	C20/25	C20/25	No guidance given	C40/50
Maximum w-c ratio	0.65	0.65		0.45
Minimum cover to reinforcement	30 mm	30 mm		45 mm (in-situ); 60 mm (in-situ top horizontal surface). 35 mm (factory)
Cements	All	All		I, I/SR, II/A-S, II/B-S, II/A-V, II/B-V, III/A, III/B, IV/A <sup>§</sup> , IV/B <sup>§</sup>
For combinations: Slag (%) Fly ash (%)	≤80 ≤55	≤80 ≤55		≤80 ≤55

<sup>+</sup> Up to 10 m from salted highways.

<sup>§</sup> Siliceous fly ash.

**Table 3.16:** Limiting values for composition and properties of concrete exposed to risk of corrosion of reinforcement induced by chlorides from sources other than sea water for an intended working life of at least **100 years**.

Exposure class	Moderate humidity	Wet, rarely dry		Cyclic wet/dry
	XD1	XD2A	XD2B	XD3 <sup>+</sup>
Minimum strength class	C20/25	C20/25	No guidance given	C40/50
Maximum w-c ratio	0.65	0.65		0.45
Minimum cover to reinforcement	30 mm	30 mm		60 mm (in-situ); 75 mm (in-situ top horizontal surface). 50 mm (factory)
Cements	All	All		I, I/SR, II/A-S, II/B-S, II/A-V, II/B-V, III/A, III/B, IV/A <sup>§</sup> , IV/B <sup>§</sup>
For combinations: Slag (%) Fly ash (%)	≤80 ≤55	≤80 ≤55		≤80 ≤55

<sup>+</sup> Up to 10 m from salted highways.

<sup>§</sup> Siliceous fly ash.

### 3.17.2 Chlorides from sea water

The minimum requirements for concretes to resist chloride induced corrosion resulting from sea water exposure for 50 and 100 years, based on this study, are given in Tables 3.17 and 3.18. Minimum requirements are not given for concretes subject to sub-class XS2B exposure.

**Table 3.17:** Limiting values for composition and properties of concrete exposed to risk of corrosion of reinforcement induced by chlorides from sea water for an intended working life of at least 50 years.

Exposure class	Moderate humidity	Wet, rarely dry		Cyclic wet/dry	
	XS1 <sup>+</sup>	XS2A	XS2B	XS3 <sup>§</sup>	
Minimum strength class	C35/45 <sup>‡</sup>	See Table 3.11 exposure class XS2A	No guidance given	C40/50 <sup>‡</sup>	C35/45 <sup>‡</sup>
Maximum w-c ratio	0.50			0.45	0.50
Minimum cover to reinforcement	40 mm (in-situ) 55 mm (in-situ, top horizontal surface) 30 mm (factory)			50 mm (in-situ) 65 mm (in-situ, top horizontal surface) 40 mm (factory precast)	50 mm 65 mm 40 mm
Cements	All			I, I/SR, II/A-S, II/A-V, IV/B* *	II/B-S, II/A-D, II/B-V, III/A, III/B, IV/A* *
For combinations: Slag (%)	≤80			≤20	21 to 80
Fly ash (%)	≤55			≤20 and 35 to 55	21 to 35

<sup>+</sup> Distances from the sea ranging from 100 to 3000 m

<sup>§</sup> Distances from the sea ranging from 0 to 100 m.

<sup>‡</sup> For II/B-V, IV/A and IV/B concretes the minimum grades may be reduced by one grade.

\* Siliceous fly ash

\* The authors are aware of only two publications dealing with CEM IV/B fly ash concretes (Thomas, 1996, Thomas and Matthews, 1996). The diffusion coefficients reported were lower than for CEM I concrete but the chloride threshold level for corrosion were also lower.

\* Siliceous fly ash content greater than 20% by mass.

**Table 3.18:** Limiting values for composition and properties of concrete exposed to risk of corrosion of reinforcement induced by chlorides from sea water for an intended working life of at least 100 years.

Exposure class	Moderate humidity	Wet, rarely dry		Cyclic wet/dry	
	XS1 <sup>†</sup>	XS2A	XS2B	XS3 <sup>§</sup>	
Minimum strength class	C35/45 <sup>‡</sup>	See Table 3.11 exposure class XS2A	No guidance given	C40/50 <sup>‡</sup>	C35/45 <sup>‡</sup>
Maximum w-c ratio	0.50			0.45	0.50
Minimum cover to reinforcement	55 mm (in-situ) 70 mm (in-situ, top horizontal surface) 45 mm (factory)			65 mm (in-situ) 80 mm (in-situ, top horizontal surface) 55 mm (factory)	70 mm 85 mm 60 mm
Cements	All			I, I/SR, II/A-S, II/A-V, IV/B* <sup>†</sup>	II/B-S, II/A-D, II/B-V, III/A, III/B, IV/A* <sup>†*</sup>
For combinations: Slag (%)	≤ 80			≤ 20	21 to 80
Fly ash (%)	≤ 55			≤ 20 and 35 to 55	21 to 35

<sup>†</sup> Distances from the sea ranging from 100 to 3000 m.

<sup>§</sup> Distances from the sea ranging from 0 to 100 m.

<sup>‡</sup> For II/B-V, IV/A and IV/B concretes the minimum grades may be reduced by one grade.

\* Siliceous fly ash.

<sup>†</sup> The authors are aware of only two publications dealing with CEM IV/B fly ash concretes (Thomas, 1996, Thomas and Matthews, 1996.) The diffusion coefficients were reported to be lower than for CEM I concrete but the chloride threshold levels for corrosion were also lower.

\* Siliceous fly ash content greater than 20% by mass.

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## CHAPTER 4

## Minimum requirements for concrete to resist freeze-thaw attack

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### 4.1 Summary

The draft European Standard pr EN 206 (BSI, 1997a) classifies environments giving a risk of freeze-thaw attack into four exposure classes:

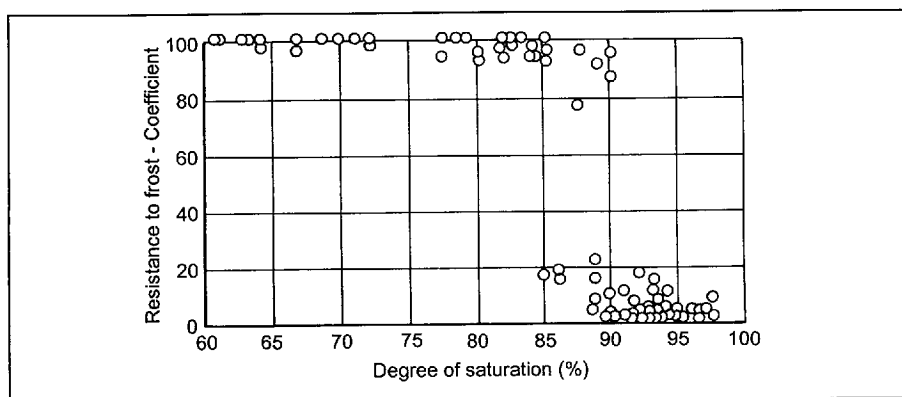
XF1 Moderate saturation, no salt	XF2 Moderate saturation, with salt	XF3 Saturated, without salt	XF4 Saturated, with salt
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This Chapter reviews the results from published studies of freeze-thaw performance of concrete in order to establish minimum concrete qualities, for each exposure class, necessary to achieve a design working life of 50 and 100 years [classes 3 and 4 of ENV 1991-1:1994 (European Committee for Standardization 1994)]. The cements considered are CEM I (Portland cement), CEM II A-L, B-L (Portland limestone cement) and binders containing fly ash, slag and silica fume. Both air-entrained and non-air-entrained concretes are considered.

Minimum concrete qualities, applicable to the UK, have been established for exposure classes XF3 and XF4 based on international studies of the freeze-thaw performance of concretes exposed to water, sea water and de-icing salts. It was necessary to base the recommendations on the international literature, as extensive laboratory and field tests on the freeze-thaw resistance of concrete have not been carried out in the UK. Translation of the published information from laboratory and field studies into minimum concrete qualities to provide a defined level of performance in service necessitated the assumption that the observations were applicable to UK external exposure conditions. For exposure classes XF1 and XF2, it is proposed that a single minimum quality of concrete based on current UK recommendations will be suitable for both. Insufficient information on the effect of binder content on freeze-thaw resistance was found to enable any recommendations to be made for minimum binder content.

## 4.2 Introduction

One of the more common forms of deterioration of concrete is due to ice formation in saturated concrete. In order for internal stresses to be induced by ice formation, about 90% or more by volume of the pores must be filled with water (Figure 4.1, Neville, 1973). This is because the increase in volume when water turns to ice is about 8% by volume. Freezing starts in larger pores in the concrete and can, at lower temperatures, spread to smaller pores. Repeated cycles of freezing and thawing can have a cumulative effect. Concrete can be critically saturated if permanently exposed to water or if exposed to a head of water. Cold concrete can become critically saturated if exposed to warm moist air and the surface layers of a concrete can become critically saturated if exposed to salt.



**Figure 4.1:** Influence of saturation of concrete on its resistance to frost (Neville, 1973).

Concretes which receive the most severe exposure to freezing and thawing are those which are saturated during freezing weather. Examples are hydraulic structures, eg. parts of dams, spillways, tunnel inlets and outlets and piers, and the horizontal surfaces of exposed concrete elements, eg. the tops and bases of walls, gutters, kerbs, driveways and pavements. Vertical concrete faces exposed to rain are rarely affected unless one face is subject to a head of water.

Three main types of deterioration are induced by freezing and thawing:

- Expansion, internal cracking and spalling. Such deterioration is caused by the susceptibility of a paste matrix or a coarse aggregate to freezing and thawing.
- Scaling associated with the application of salt.
- Pop-outs caused by freeze-thaw susceptible coarse aggregate particles.

It has been known since 1941 that air-entrainment in concrete can enable concrete to adequately resist certain types of freezing and thawing deterioration and also the adverse effects of de-icing salts, provided a high quality paste matrix and a frost-resistant aggregate are used (Swayze, 1941). Consequently, air-entrained concretes are often specified in situations where the concrete is likely to be saturated during freezing weather.

Three of the six exposure classes given in BS 8110 (1997b) are applicable to situations where occasional freezing or severe freezing conditions may exist. The concrete qualities and nominal

cover to steel to meet these durability requirements are given in Table 4.1. When the lowest grade of concrete is C50, no air-entrainment is required. However, when the lowest grade of concrete is less than C50, the average total air content by volume of the fresh concrete should, for a nominal maximum aggregate size of 20 mm, be 5.5%. Lower concrete qualities are given for unreinforced concrete subject to the same three exposure classes.

The concrete qualities and nominal cover to steel for concrete subject to marine exposure (BS 6349: Part 1: 1984: AMD 6189) are given in Table 4.2. No air-entrainment is required.

**Table 4.1:** Concrete qualities and nominal cover to steel to meet durability requirements (BS 8110).

Environment	Nominal cover (mm)		
Severe	40	30	25
Very severe	50 <sup>+</sup>	40 <sup>+</sup>	30
Most severe	-	-	50
Maximum free w/c ratio	0.55	0.50	0.45
Minimum c (kg/m <sup>3</sup> )	325	350	400
Lowest grade of concrete	C40	C45	C50

<sup>+</sup> Air-entrained

**Table 4.2:** Concrete qualities and nominal cover to steel to meet durability requirements (BS 6349).

Exposure	Concrete	Minimum cover (mm)	Maximum water-cement ratio	Minimum cement content (kg/m <sup>3</sup> )
Directly exposed to sea water spray or in tidal zone down to 1 m below lowest level	Reinforced	50	0.45	400
	Plain	NA	0.50	350

The draft European standard pr EN 206 classifies concretes subject to a risk of freeze-thaw deterioration into the following four exposure classes (BSI, 1997a):

XF1	XF2	XF3	XF4
Moderate saturation, no salt	Moderate saturation, with salt	Saturated, without salt	Saturated, with salt

It is difficult to fit the freeze-thaw exposure sub-classes to practical situations. The term 'moderate saturation' used in sub-classes XF1 and XF2 presents technical difficulties for two reasons:

- There is no such condition as 'moderate saturation'
- Freeze-thaw attack is significant only when the critical saturation level is exceeded.

It is thus necessary to interpret XF1 and XF2 as being less severe freeze-thaw conditions than

XF3 and XF4 respectively. The distinction between XF1 and XF2 and between XF3 and XF4 is clear, ie. the absence or presence of salt which is known to aggravate freeze-thaw attack.

There is more debate over whether there can actually be two levels of freeze-thaw severity in the presence of salts, as it is argued that the salt will mean that the critical saturation will often be reached in the surface layers. Frequency of freeze-thaw actions is, however, believed by the authors to provide a measure of the severity of exposure.

Within this Chapter, the conditions have thus been interpreted as:

- XF1 Freeze-thaw attack of moderate severity in the absence of salt. Moderate either because of the infrequency of freeze-thaw events or the low likelihood of freezing and thawing whilst saturated.
- XF2 As XF1 but in the presence of salt.
- XF3 Severe freeze-thaw attack in the absence of salt. Severe because of the frequency of freeze-thaw events or the high likelihood of freezing whilst saturated.
- XF4 As XF3 but in the presence of salt.

The examples given in the forthcoming European standard pr EN 206 (BSI, 1997a), given below, are in accord with this interpretation:

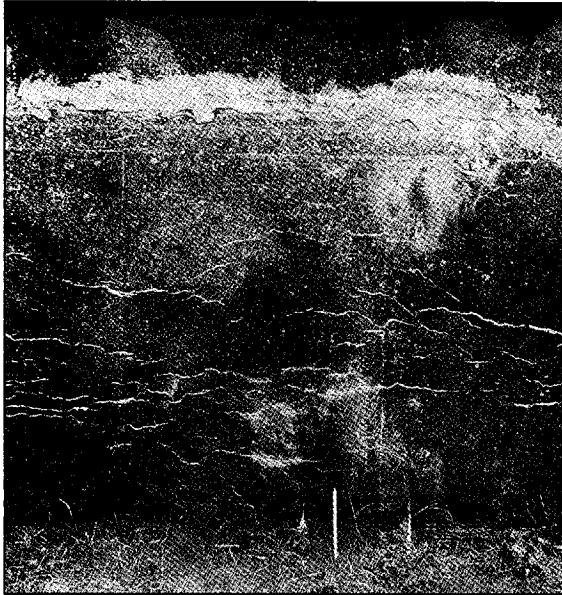
- XF1 Vertical concrete surfaces exposed to rain and freeze-thaw attack
- XF2 Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents.
- XF3 Horizontal concrete surfaces exposed to rain and freezing.
- XF4 Roads and bridge decks exposed to de-icing agents and vertical concrete surfaces exposed to direct spray containing de-icing agents and freezing.

The object of the work described in this Chapter is to propose the minimum concrete qualities for these exposure classes which are necessary for design lives of up to 100 years. It is assumed the concretes are well compacted. The cements considered are Portland cements and binders containing pulverized fuel ash, ground granulated blastfurnace slag, limestone and silica fume. Throughout the remainder of this Chapter, pulverized fuel ash is referred to as fly ash and ground granulated blastfurnace slag is referred to as slag.

### **4.3 Freeze-thaw attack in the UK**

In the UK, freeze-thaw attack is one of the more common causes of concrete deterioration. Examples of freeze-thaw attack of non-air-entrained concrete that have been observed include:

- Expansion, internal cracking, visual cracking and spalling in dams (Hobbs, 1996 a) and pavements (Figures 4.2 and 4.3).
- Severe deterioration in exposed foundation blocks (report of a Working Party, 1992).
- Deterioration in bridges (Wallbank, 1989).
- Spalling and scaling on the top horizontal surface of sea defences (Figure 4.4).
- Cracking and spalling at the tops and bases of walls (report of a Working Party, 1992, Figure 4.5).

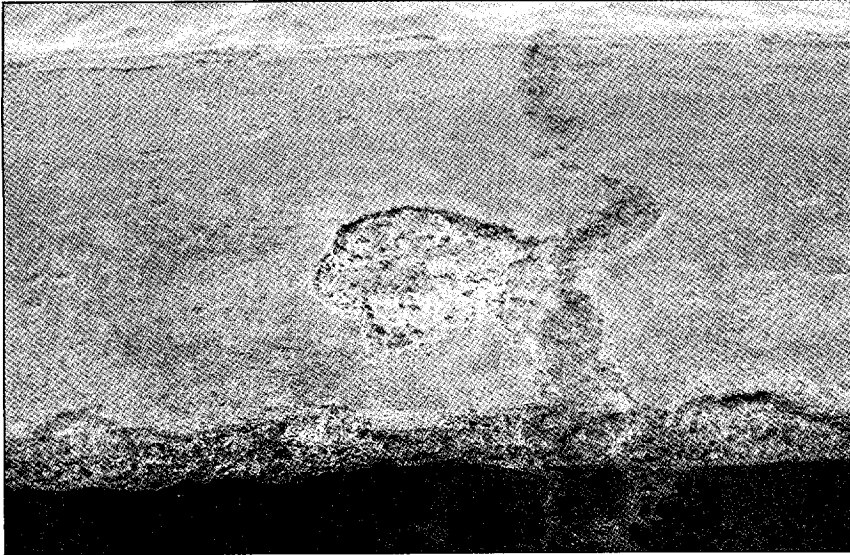


**Figure 4.2:** Cracking due to freeze-thaw attack in a concrete dam.



**Figure 4.3:** Spalling caused by freeze-thaw attack.

- Spalling or pop-outs believed to be associated with a coarse aggregate susceptible to freeze-thaw attack.
- D-line cracking (Cordon, 1966) in pavement slabs (Figure 4.6).



**Figure 4.4:** Deterioration due to freeze-thaw attack at the top of a sea wall.

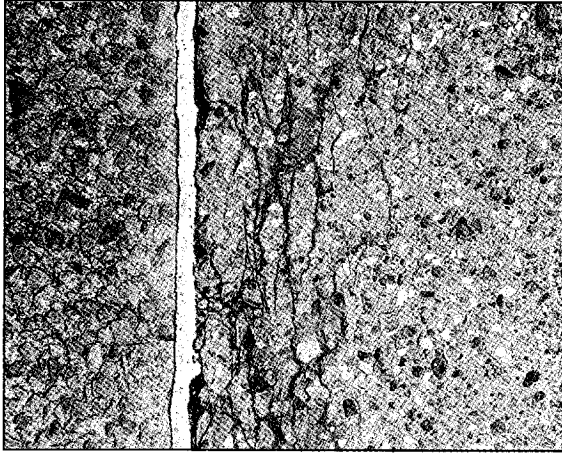


**Figure 4.5:** Deterioration due to freeze-thaw attack at the top of a concrete wall.

- Scaling of pavements and driveways associated with the application of de-icing salts (Figure 4.7).
- Scaling and complete disintegration of kerb stones.

Examples of freeze-thaw attack on air-entrained concrete are much less common.

It is not possible to form clear judgements regarding the quality of the concretes which have performed poorly. Pop-outs, associated with particular coarse aggregates, have been observed with both air-entrained and non-air-entrained concretes and the incidence or risk of pop-outs is not related to concrete quality as defined by grade or water-binder ratio (Hobbs, 1996 b).



**Figure 4.6:** Deterioration due to freeze-thaw attack close to the edge of a pavement slab.



**Figure 4.7:** Freeze-thaw scaling due to de-icing salt exposure.

## 4.4 Testing

### 4.4.1 Tests for freeze-thaw resistance of concrete

Judgements regarding the freeze-thaw resistance of concrete are often based on laboratory freeze-thaw testing rather than an established field performance. The freeze-thaw resistance of concrete has commonly been determined by one of the following laboratory procedures:

**Method 1** (ASTM C666, 1992a, Procedure A) - By rapid freezing and thawing of the concrete whilst it is immersed in water. The cooling rate ranges from 6 to 15°C/hour (Pigeon and Pleau, 1995), with a minimum temperature of -18°C.

**Method 2** (ASTM C666, 1992a, Procedure B) - By rapid freezing of saturated concrete in air and thawing whilst immersed in water. The cooling rate ranges from 6 to 15°C/hour, (Pigeon and Pleau, 1995) with a minimum temperature of -18°C.



**Method 3** (BS 5075: Part 2, 1982) - By freezing and thawing of air-entrained concrete whilst immersed in water. The total air content of the concrete is 5.5% by volume. The cooling rate ranges from 1.4 to 3.3°C/hour with a minimum temperature of -18°C.

In this report the laboratory data discussed were primarily obtained using the above methods. Other test methods are being developed by CEN/TC51.

In Methods 1 and 2, the 'durability' is given by

$$\text{Durability factor} = PN / 300$$

where P is the relative dynamic modulus of elasticity at N cycles and N is the number of cycles at which P falls below 60%. The concretes are subject to a maximum of 300 freeze-thaw cycles. In Method 3 the 'expansion' is measured after 50 cycles and should not exceed 0.050% for satisfactory performance.

The freeze-thaw tests are probably more onerous than UK field exposure. For example:

- The test concretes are normally stored continuously under water or in a fog room prior to test. The concretes are therefore frozen in a saturated state. Concretes in the field are unlikely to be in a similar saturated state when subject to freezing. This is due in part to drying and, particularly with high quality concrete, in part to self-desiccation. When exposed to moisture, such concretes do not re-saturate to the same degree as concretes maintained continuously moist.
- Concretes in the UK rarely experience temperatures as low as -18°C and consequently in laboratory tests water can freeze in smaller pores than is the case with most UK exposed concretes. Also any adverse effects due to differences in coefficients of thermal expansion between the cement paste and aggregate will be more pronounced.
- In Methods 1 and 2, the cooling rates are greater than that experienced by concrete subject to UK natural exposure, which is less than 4°C/hour. As a consequence, higher pressures can be induced in the unfrozen water.

Thus some concretes which are judged to perform poorly in these tests may give, and have given, good performance when subject to UK field exposure.

#### **4.4.2 Tests for scaling resistance of concrete due to freeze-thaw attack in the presence of salt**

Scaling of concrete is the loss of material at the exposed surface of a concrete due to freeze-thaw attack particularly in the presence of salt. No British Standard test exists for determining the scaling resistance of concrete. The most popular test is ASTM C672 (1992b). In this test the concrete is subjected to 14 days moist curing followed by 14 days drying. The finished top surface of the concrete is exposed to a 4% calcium chloride solution and the concrete subjected to a minimum of 50 daily freeze-thaw cycles. The deterioration after 5, 10, 15, 25 and 50 cycles is assessed by visual rating or by the mass of scaled-off material. Concrete is normally considered to have adequate scaling resistance if the mass of scaled-off material does not exceed 1 kg/m<sup>2</sup> after 50 freeze-thaw cycles. The Swedish Standard SS 13 72 44 uses a modified version of this test. (CEN/TC51/WG12/TG4: 6/94).

### 4.4.3 Tests on aggregates

The use of certain coarse aggregates can result in expansion, pop-outs or a reduced scaling resistance. The freeze-thaw resistance of coarse aggregate is sometimes determined using ASTM C682 (1994a). Here, the freeze-thaw resistance of concrete containing the coarse aggregate is assessed by measuring expansion, if any, of air-entrained concrete during the freezing cycle. Alternatively, Methods 1, 2 and 3 given previously may be used. Tests to determine the freeze-thaw resistance of concrete aggregates are under development by CEN.

## 4.5 Air-entrainment and freeze-thaw resistance

In 1941 Swayze showed that the addition to concrete of organic materials such as mineral oils, animal or vegetable fats or natural resins, reduced strength but greatly enhanced the resistance of concrete to freezing and thawing. Research by other organisations showed that this improved resistance was due to air entrainment.

All concrete, after compaction, contains some residual air voids amounting to about 1.0 to 1.5% of the concrete volume. Much of this entrapped air is not considered to be effective in improving resistance to frost attack. This is believed to be because many of the entrapped air pockets are too large and too widely spaced to be effective. Entrained air produces minute bubbles evenly distributed throughout the concrete which do not become filled with the products of hydration. These air bubbles relieve the hydraulic pressure produced when ice forms.

The factors associated with air-entrainment which are believed to be important in enhancing freeze-thaw resistance are:

- Volume concentration of air
- Air-void spacing
- Specific surface area of air-voids (which is related to air-void size)

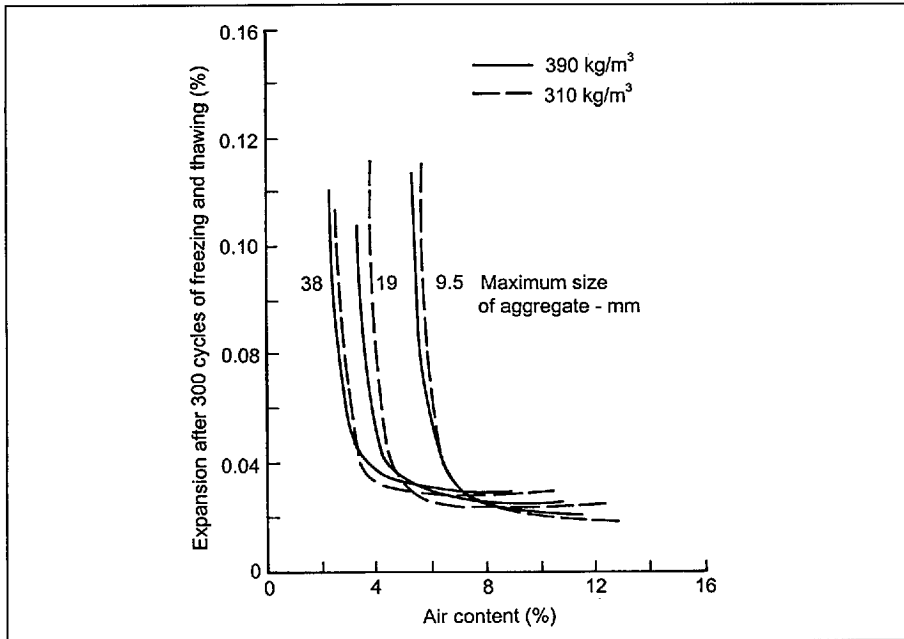
### 4.5.1 Dependence on volume concentration of air

The freeze-thaw resistance of concrete is dependent upon the volume concentration of air. Results obtained by Klieger (1956) on Portland cement concrete, with various maximum sizes of aggregate are plotted in Figure 4.8. Klieger's conclusions included the following:

- In all cases, the entrainment of air increased the resistance of concrete to freezing and thawing and to surface scaling resulting from the use of salts for ice removal.
- The optimum concrete air requirement for frost resistance increased as the maximum size of aggregate decreased (or more correctly as the paste volume increased), for example:
  - optimum air content 6 to 9% for 10 mm maximum aggregate particle size
  - optimum air content 4 to 7% for 19 mm maximum aggregate particle size
  - optimum air content 3 to 6% for 38 mm maximum aggregate particle size.

This conclusion is based on the expansion induced in concrete after 300 freeze-thaw cycles, the concretes being frozen and thawed whilst immersed in water.

- Concretes maintained moist throughout their life required more entrainment for adequate resistance to freezing and thawing than the air-dried concretes.



**Figure 4.8:** Influence of air content upon expansion after 300 cycles of freezing and thawing (Klieger, 1956).

Work carried out by other investigators supports the conclusions obtained by Klieger and recommended minimum air contents for resistance to freeze-thaw cycles given in Standards and Codes of Practice, agree closely with the observations made by Klieger. The recommendations of ACI 201.2R-95 and BS 5328: Part 1: 1997 are given as examples in Table 4.3.

**Table 4.3:** Recommended air contents.

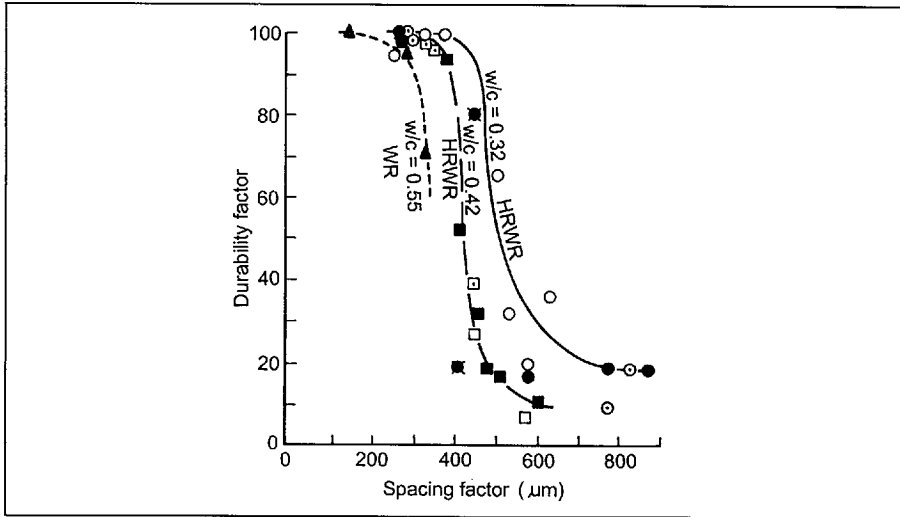
Maximum size of aggregate (mm)	Recommended mean total air content (%)	
	BS 5328 (de-icing salts)	ACI 201.2R-92 saturated or de-icing salts
~ 10	7.5	7.5
14	6.5	-
~ 20	5.5	6.0
~ 40	4.5	5.5

#### 4.5.2 Dependence on air-void spacing

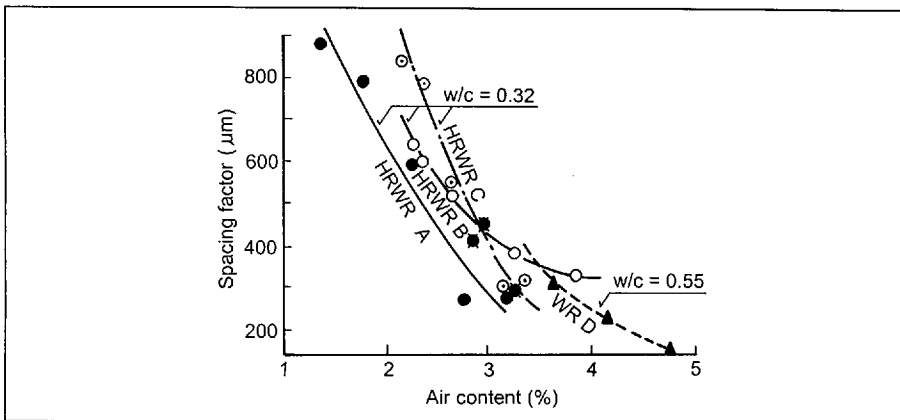
Much of the air entrapped in concrete due to incomplete compaction is unsuitable for enhancing the freeze-thaw resistance of concrete. This is because both the air-void size and spacing are too large. A number of investigators have concluded that for a given air volume concentration, freeze-thaw resistance is related to air-void spacing, the smaller the air-void spacing (or the smaller the air-void size), the greater the freeze-thaw resistance. This is illustrated in Figures 4.9 to 4.11 (Kobayashi et al, 1981). A spacing factor of 200  $\mu\text{m}$  or below will often protect concrete

against scaling due to freezing and thawing in the presence of de-icing salts (Pigeon and Pleau, 1995). A limit of 200  $\mu\text{m}$  on the spacing factor has been specified since about 1958 in the USA (ASTM C260, 1994b). In Germany it has been found that spacing factors between 200 and 250  $\mu\text{m}$  can lead to scaling on roads (Sommer, 1979). Larger spacing factors can give high protection against freeze-thaw attack in the absence of salt (Pigeon and Pleau, 1995).

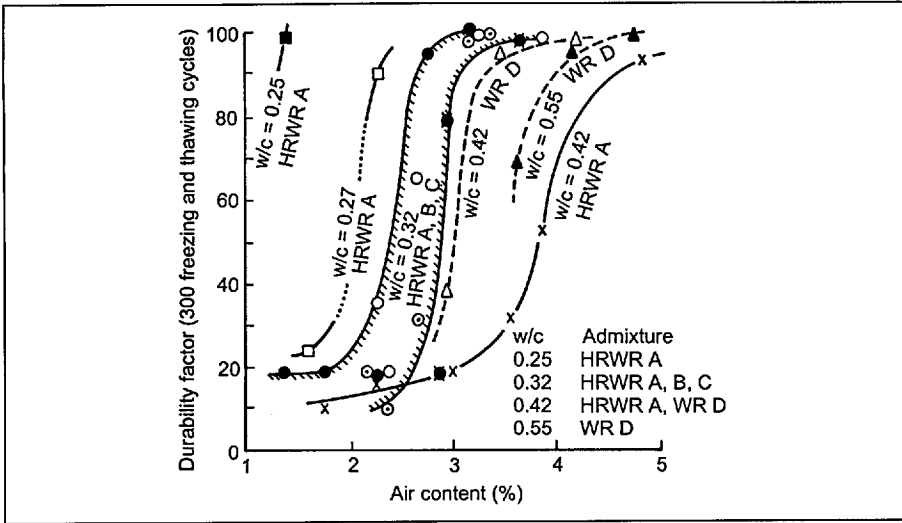
The relationship between air-content and mean air-void spacing is scattered (Saucier, Pigeon and Cameron, 1991) (Figure 4.12), and is influenced by water-cement ratio or cement content and the use of superplasticizers (melamine- or naphthalene-based) (Kobayashi et al, 1981), (see Figure 4.13). The size of the air voids decreases with reducing water-cement ratio (US Bureau of Reclamation, 1955). Superplasticizers can cause an increase in the air-void spacing and a loss



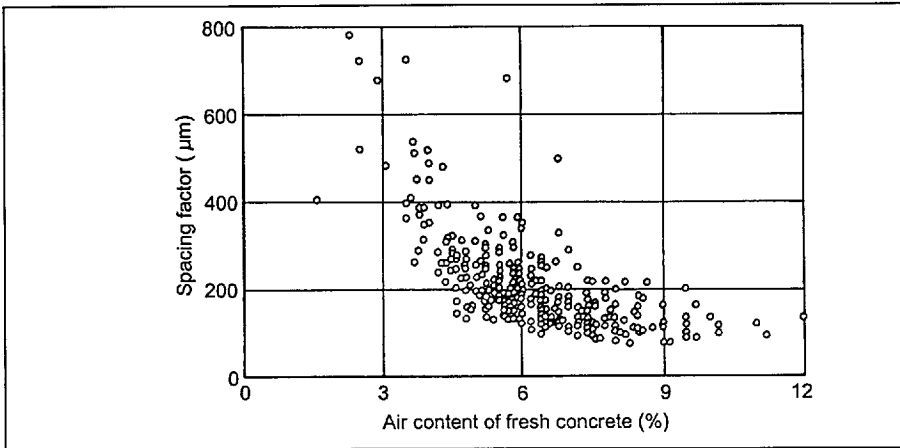
**Figure 4.9:** Relationship between air-void spacing factor and durability factor in concrete containing high-range water-reducing admixture (Kobayashi et al, 1981).



**Figure 4.10:** Relationship between air-void spacing factor and air content for concrete containing high-range water-reducing admixtures (Kobayashi et al, 1981).



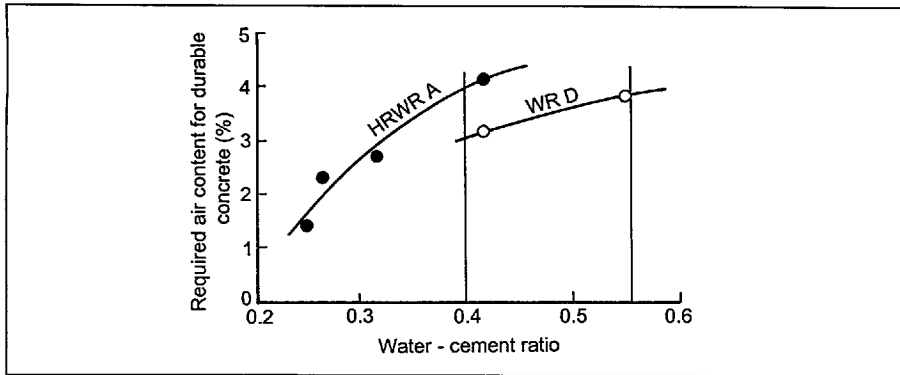
**Figure 4.11:** Relationship between durability factor and air content for concrete containing high-range water-reducing admixtures (Kobayashi et al, 1981).



**Figure 4.12:** Relationship between air content measured on fresh concrete and air void spacing factor in hardened concrete (Saucier et al, 1991).

of air (Mielenz and Sprouse, 1978; Tognon and Cangiano, 1982; MacInnis and Racic, 1986; Plante et al, 1989; Saucier et al, 1990; Robson, 1987).

Current British standards include no requirements for air-void spacing, and for proven and established air-entraining admixtures or combinations of admixtures, the quality of the air-void system is normally judged by the total air content of the concrete.



**Figure 4.13:** Relationship between water-cement ratio and required air content (Kobayashi et al, 1981).

## 4.6 Air-entrained concretes saturated without salt: XF3

### 4.6.1 CEM I and CEM I/SR concretes

As indicated in sub-Section 4.5.1, Klieger (1956) showed from a series of laboratory tests in which a range of concrete samples were subject to 300 freeze-thaw cycles whilst immersed in water, that air-entrainment greatly enhances the freeze-thaw resistance of concrete. In Klieger's work the water-cement ratios were in the range 0.42 to 0.74, the air contents in the range 2 to 16% and the 28-day/6 in. modified cube strengths in the range 13 to 47 N/mm<sup>2</sup>. All the concretes, when properly air-entrained, had high resistance to freeze-thaw attack.

In 1941 the Portland Cement Association in the US (Oleson and Verbeck, 1967) commenced a field study to investigate the performance of cements in concretes exposed to relatively severe natural freeze-thaw cycling. Thin-wall boxes of 0.76 m, kept filled with wet sand to represent retaining walls, were prepared from a number of concrete mixes. The boxes were exposed on a site near Illinois, Chicago. The water-cement ratio of the concretes ranged from 0.41 to 0.77, the 28-day cylinder strengths from 21 to 43 N/mm<sup>2</sup> and the air-contents from 3.2 to 4.9%. After 25 years' exposure it was noted that the performance was 'nearly perfect'. This accords closely with Klieger's laboratory observations.

External exposure tests have also been reported by Sturup and Clendenning (1969). In this work the concretes were exposed on a site on the western outskirts of Toronto. Here, the annual number of freeze-thaw cycles probably lies in the range 45 to 72. Concrete blocks 12 x 12 x 18 in. in size were moist cured for 14 days, exposed to a laboratory atmosphere for 14 days and then partially immersed in water-filled containers. Five year observations are given in Table 4.4. All the concretes which had w-c ratios ranging from 0.5 to 1.0 and air contents ranging from 5.0 to 6.1% exhibited high resistance to freeze-thaw attack. Air-entrained concretes with air contents below 5.0% were not included in the programme. The results are in accord with those by Oleson and Verbeck (1967). From this evidence, for exposure class XF3 a nominal quality of concrete, properly air-entrained, would be suitable.

**Table 4.4:** Condition of outdoor exposure blocks. Age 5 years (Sturup and Glendenning, 1969).

w-c ratio	Air content (%)	Condition
0.5	5.9	No scaling
0.6	6.1	Surface scaling <1 %
0.8	5.0	No scaling
1.0	5.2	No scaling

#### 4.6.2 CEM I/fly ash concretes

Few controlled field performance tests have been carried out on air-entrained concretes containing fly ash. Exposure test observations reported by Sturup and Clendenning (1969) on concrete blocks partially immersed in water for eight years are given in Table 4.5. The concretes contained various proportions of fly ash and had a water-binder ratio of 0.6. The performance was good up to fly ash levels of 30%.

Sturup and Clendenning (1969) comment that air-entrained concretes, in which 30% by mass of the cementing material is fly ash, have been used in exposed locations with excellent results.

**Table 4.5:** Exposure tests on air-entrained fly ash concrete - 12 x 12 x 18 in blocks. Loss on ignition of fly ash 5.4% by mass (Sturup and Glendenning, 1969)

Fly ash content	Air content (%)	Mass loss and degree of scaling below water-line	Expansion
0	5.6	2% slight ravelling	None
15	6.1	2% very light scaling	None
30	6.3	7% light scale	None
45	6.7	9% medium scale	None
60	7.2	23% deep scale	None

Similar deductions have been made from accelerated laboratory tests. For example, Carette and Malhotra (1984) observed good durability performance for concretes containing 20% fly ash, with a w-b ratio of 0.5 and an air content of between 6.1 to 6.5% when the concretes were tested according to ASTM C-666 (1992, procedure A). Virtanen (1983) observed good performance, using a dilation test, for concretes containing 33% fly ash, with a w-b ratio of 0.55 when the air content was 4%. Larson (1964) makes the following observation "When valid comparisons are made with equal strengths and air contents, there are no apparent differences in the freezing and thawing durability of fly ash and non-fly ash concrete". Matthews (1989) observed good performance from concretes containing fly ash cements (28-30% fly ash) using the BS 5075 test with w-b ratios of 0.49 - 0.50 and air contents of 5.5 - 6.2%.

#### 4.6.3 CEM I/slag concretes

Little data has been reported on the freeze-thaw resistance of air-entrained concretes containing slag subject to field exposure. However, Stark and Ludwig (1997) have reported that "cements rich in granulated blastfurnace slag have proved successful in structures subject to frost attack such as dams".

A number of studies have been made on concretes containing slag which have been subject to accelerated freeze-thaw cycling, eg.:

Hogan and Meusel (1981)	ASTM C666-92. Procedure A
Mather (1957)	ASTM C666-92. Procedure A
Klieger and Isberner (1967)	ASTM C666-92. Procedure A
Fulton (1974)	ASTM C666-92. Procedure A
Malhotra (1983)	ASTM C666-92. Procedure B
Virtanen (1983)	A dilation test
Pigeon and Regourd (1983)	ASTM C666-92. Procedure B
Nakamura, Sakai, Koibuchi and Iijima (1986)	ASTM C666-92. Procedure A
Malhotra (1987)	ASTM C666-92. Procedure A

This work has shown that, in accelerated laboratory tests, the durability of concretes containing up to 65% slag is similar to that of Portland cement concretes, provided the concrete is air-entrained and the concretes have the same water-binder ratio.

#### 4.6.4 Portland limestone cement concretes, CEM II/A-L, B-L

The authors are not aware of any controlled field performance tests carried out on air-entrained concretes made with CEM II/A-L, B-L (Portland limestone cements). Also few investigations have been reported of the performance of Portland limestone cement concretes in accelerated freeze-thaw tests. Matthews (1989) observed that Portland limestone cement concretes containing 25% limestone (CEM II/B-L), with total water-binder ratios from 0.54 to 0.58 and air contents from 5.5 to 6.4%, exhibited less than 0.05% expansion when subject to 100 freeze-thaw cycles using the BS 5075 (1982) procedure. However, Matthews observed higher scaling at the 25% limestone level than that observed with the control CEM I concretes.

#### 4.6.5 CEM I/silica fume concretes

Little data has been reported on the freeze-thaw resistance of air-entrained concretes containing silica fume and subject to field exposure. A number of studies have been made on mortars and concretes containing silica fume which have been subject to accelerated freezing and thawing including:

Traetteberg (1980)	Mortars tested using a procedure similar to ASTM C666-92. Procedure B. Approximately 50 freeze-thaw cycles, 0 to 25% sf
Carette and Malhotra (1983)	ASTM C666-92. Procedure A. 0 to 30% sf
Aïtcin and Vezina (1984)	ASTM C666-92. Procedure B. 0 and 8% sf
Virtanen (1983)	A dilation method. 0 and 8% sf
Malhotra, Paintor and Billodeau (1987)	ASTM C666-92. Procedure A. 0 to 20% sf
Pigeon, Pleau and Aïtcin (1986)	ASTM C666-92. Procedure A. 0 and 10% sf

The above work has shown that the freeze-thaw resistance of concretes containing up to 10% silica fume is generally greater than that of CEM I concretes of the same water-binder ratio, whilst at 20 and 30% silica fume, it is generally less.



#### 4.6.6 Minimum specification for air-entrained concrete subject to exposure class XF3

From the above work it is concluded that:

- For exposure class XF3 concretes of similar 28-day strength and air content, prepared using CEM I, CEM II/A-V, CEM IV/A, CEM II/B-V and CEM IV/B, should give similar freeze-thaw durability.
- For exposure class XF3, concretes of similar water-binder ratio and air-content, when prepared using CEM I, CEM II/A-L, CEM II/A-S, CEM II/A-D, CEM II/B-S and CEM/IIIA, should give similar freeze-thaw durability.

Due to lack of published data, judgements are not possible regarding the freeze-thaw performance of concretes prepared using CEM II/B-L, CEM III/B and CEM III/C cements. A minimum concrete specification for air-entrained concrete exposed to freeze-thaw cycling whilst in a saturated state is given in Table 4.6. Due to lack of data, a minimum binder content is not given.

**Table 4.6:** Minimum concrete specification for air-entrained concrete subject to exposure XF3.

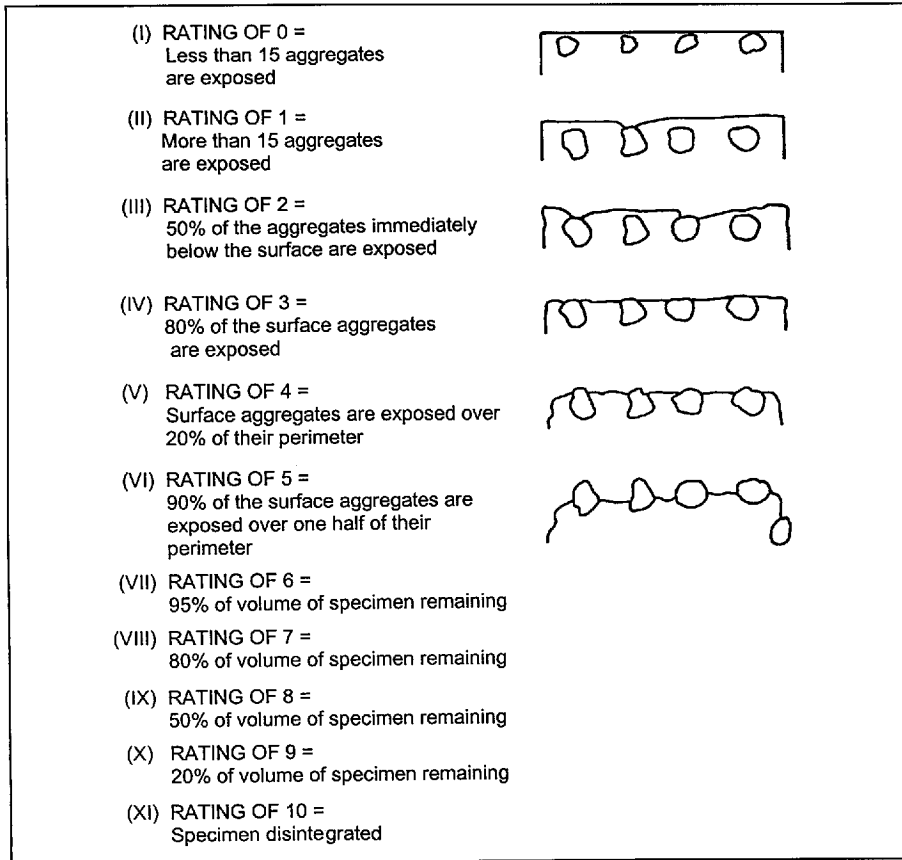
Maximum w-c ratio	0.60
Minimum strength class	C25/30
Minimum air content for aggregate with a nominal size of 20 mm	4 %
Cement types and equivalent combinations	All except CEM II/B-L, III/B and III/C
Other requirement	Freeze-thaw resistant aggregate

Note: Premature exposure to freezing and thawing may result in inadequate performance.

### 4.7 Air-entrained concretes saturated with salt: XF4

#### 4.7.1 CEM I and CEM I/SR concretes: marine exposure

In 1978, Malhotra, Carrette and Bremner (1987 and 1988), commenced a series of long-term exposure tests on concrete prisms, 305 x 305 x 915 mm in size, in a marine environment. All the air-entrained concretes were proportioned to have an air content of  $6 \pm 1\%$  by volume. Prior to marine exposure the prisms were moist cured for 28 days (phase I) or at least 90 days (phase II). The concrete prisms were positioned at mid-tide level on a rack at the entrance to the Bay of Fundy, Treat Island, Maine. This was considered by Malhotra et al to be the most severe marine exposure condition for concrete. At this location the concrete prisms are exposed to repeated cycles of wetting and drying and to an average of about 100 cycles of freezing and thawing per year. The test specimens were monitored at yearly intervals by visual inspection and by taking photographs. The visual rating used to evaluate the test prisms is shown in Figure 4.14. After seven to nine years' exposure, concretes exhibiting visual ratings of 2 or lower may be taken to be concretes of high durability in a marine environment (Malhotra et al 1988). The visual rating of the test prisms in phase 1 after nine years' exposure and phase 2 after eight years' exposure, are given in Tables 4.7 and 4.8.



**Figure 4.14:** Visual rating used to evaluate the test prisms (Malhotra et al, 1987).

**Table 4.7:** Visual rating of CEM I test prisms. Phase I (Malhotra et al, 1987; 1988).

Mix series	ASTM Portland cement Type	w-c ratio	28d cylinder strength (N/mm <sup>2</sup> )	Visual rating at 9 years
A	I	0.4	30	0-1
A	V	0.4	34	1
B	I	0.5	26	0-1
B	V	0.5	32	1
C	I	0.6	24	0-2
C	V	0.6	25	2
SP	I	0.5	27	1-2

Examination of Tables 4.7 and 4.8 shows that all the concretes are performing well and that the performance is independent of CEM I type.

In the mid 1950s, the Waterways Experimental Station, USA, and the Portland Cement Association, USA, commenced a study to investigate the freeze-thaw resistance of air-entrained

**Table 4.8:** Visual rating of CEM I test prisms. Phase II (Malhotra et al, 1987; 1988).

Mix series	ASTM Portland Cement Type	w-c ratio	28d cylinder strength (N/mm <sup>2</sup> )	Visual rating at 8 years
D	I	0.4	32	1
D	V	0.4	32	0
D	II	0.4	30	1
E	I	0.5	28	1
E	V	0.5	29	1
E	II	0.5	24.5	0
F	I	0.6	20	1-2
F	V	0.6	20	1
F	II	0.6	20	1

CEM I concretes exposed to natural weathering at Treat Island, Maine (Mather, 1967; Lamond and Lee, 1990). Eighteen replicate concrete prisms were made from each of a range of ASTM Type I, II and V Portland cements (22 in all), with water-cement ratios of 0.45 to 0.50 and air contents of approximately 6%. At 10 years, that is after about 1,000 freeze-thaw cycles, none had failed (Mather, 1967). At 26 years, and after more than 3,400 cycles, 21% of the ASTM Type I Portland cement concretes had failed, 22% of the ASTM Type II and 33% of the ASTM Type V.

#### 4.7.2 CEM I/slag concretes: marine exposure

In the phase 1 programme of field exposure tests carried out by Malhotra et al (1987 and 1988), a number of the concrete prisms contained various proportions of slag, namely 25, 45 and 65% by mass of cement. The visual rating of these prisms after nine years' exposure are given in Table 4.9. Examination of this table shows that the following concretes performed well:

- Concretes containing 25% and 45% slag and with water-binder ratios  $\leq 0.5$
- Concretes containing 65% slag and with a water-binder ratio of 0.4.

Comparing Tables 4.7 and 4.9 leads to the observation that, for equal performance in a marine environment, lower water-binder ratios are necessary for CEM I/slag concretes than for plain CEM I concretes.

In the mid 1950s, the Waterways Experimental Station, USA, commenced a study to investigate the freeze-thaw resistance of slag concretes exposed at mid-tide level at Treat Island, Maine (Mather, 1956). The concrete prisms, 89 x 114 x 406 mm in size, had a water-binder ratio of approximately 0.46, an air content of 6% and were made with one CEM I control and 11 slag cements with clinker contents ranging from 34 to 75%. After 900 cycles of freezing and thawing (eight years), apart from scaling of the slag concretes, all were showing good performance (Mather, 1965). However, after 25 years of exposure and 3,250 cycles of freezing and thawing, one of the eight control CEM I prisms had failed and 44 of the 50 prisms made using slag cements had failed (Lamond and Lee, 1990).

**Table 4.9:** Visual rating of slag prisms. Phase I (Malhotra et al, 1987; 1988).

Mix series	ASTM Portland Cement Type	Slag content (%)	w-b ratio	28d cylinder strength (N/mm <sup>2</sup> )	Visual rating at 9 years
A	I	25	0.4	30	0
A	I	45	0.4	30	1
A	I	65	0.4	28	1
B	I	25	0.5	27.5	1-2
B	I	45	0.5	27	2
B	I	65	0.5	23	3
C	I	25	0.6	25	1-3
C	I	45	0.6	26	2-3
C	I	65	0.6	20	4

#### 4.7.3 CEM I/fly ash concretes: marine exposure.

Also included in the field programme carried out by Malhotra et al (1987 and 1988) were concretes containing 25% fly ash. These were included in phase II and phase IV of the test programme. The visual ratings after eight years' exposure (phase II) and six years' exposure (phase IV) are given in Table 4.10. Examination of this table shows that concretes containing 25% fly ash perform well when w-b ratio  $\leq 0.5$ .

**Table 4.10:** Visual rating of fly ash prisms. Phase II and IV (Malhotra et al, 1987; 1988).

Mix series	ASTM Portland cement Type	w-b ratio	28d cylinder strength (N/mm <sup>2</sup> )	Visual rating
D	I	0.4	25	1
E	I	0.5	22	1
F	I	0.6	17	2
J	I	0.4	31	0-2
K	I	0.5	28	2
L	I	0.6	20	3

#### 4.7.4 CEM I concretes: de-icing salt exposure

Several investigators have reported observations on the field performance of concrete slabs subjected to de-icing salt exposure. These investigations are briefly outlined below:

**Klieger and Landgren (1969):** Elevated and on-ground concrete slabs were located in an exposure site in the grounds of the PCA laboratories at Skokie, Illinois. The water-CEM I cement ratios were in the range 0.43 to 0.58 and the air contents 4.9 to 6.0%. The slabs were 4 ft x 5 ft x 6 in. Timber was bonded to the top edges of the slabs to retain rain, snow and de-icer solutions. The following conclusions were reached after four to five years' exposure:

- Air-entrained concretes cast in the summer months performed well but there is a tendency towards decreased resistance to de-icer scaling for concretes with a w-c ratio of 0.58.

- Air-entrained concrete cast in the autumn is more susceptible to de-icer scaling, probably because the concrete is of lower strength than concrete cast earlier in the year and thus more vulnerable to freeze-thaw attack.

**Keyser (1968):** In 1955 concrete slabs were cast with water-cement ratios in the range 0.51 to 0.57 and air contents between 6.0 to 7.3%. The slabs were placed in the vicinity of the Control and Research Laboratory, Department of Public Works, Montreal. An examination nine years after construction showed that none of the air-entrained slabs were scaled.

**Weaver and Isabelle (1968):** During the winters of 1948 and 1949, extensive scaling of sidewalks occurred in Montreal due to the use of de-icing salts. As a consequence, a field study was put in hand in which 15 sidewalks, selected at random, were replaced in each of the three consecutive years. The water-cement ratio of test sections were less than 0.5, the air contents 0.5 to 13.5% and the 28d cylinder compressive strengths about 30 N/mm<sup>2</sup>. The slabs which performed well had air contents in the range 3.6 to 13.5%. It was noted that concrete placed and cured at relatively low temperatures subsequently became vulnerable to freezing.

Several investigators have assessed the salt-scaling resistance of CEM I concretes using the ASTM C672-92 procedure, the assessment being made by visual rating or by mass of scaled-off material. A scaling loss of less than 1 kg/m<sup>2</sup> is normally deemed to be acceptable. Some results obtained are given in Table 4.11. The results are in reasonable accord with field observations.

**Table 4.11:** Some de-icer scaling test results. CEM I concretes.

Authors	w-c ratio	Air-content (%)	De-icer scaling or visual rating
Johnston (1995)	0.5	7.2	0.1 to 0.3 kg/m <sup>2</sup>
Langois et al (1989)	0.45	6.0	0.2 kg/m <sup>2</sup>
Bilodeau and Malhotra (1992)	0.39	4.6 to 5.5	Slight to moderate
Bilodeau and Malhotra (1997)	0.39	4.3	Moderate, 2.0 kg/m <sup>2</sup>
	0.31	3.5	Very slight, 0.6 kg/m <sup>2</sup>
	0.27	3.1	Slight to moderate, 0.8 kg/m <sup>2</sup>
Whiting (1989)	0.52	6.2	Slight
	0.59	6.6	Slight
	0.62	6.5	Slight to moderate

#### 4.7.5 CEM I/fly ash concretes: de-icing salt exposure

Few observations have been reported of field studies on Portland/fly ash concretes subject to de-icing salt exposure. Klieger and Landgren (1969) subjected three air-entrained concrete slabs containing 30% fly ash to de-icing salt exposure and observed "that there is little perceptible difference between the surfaces of the slabs with and without fly ash at the same concrete air contents." The water-cement ratio of the control was 0.48 and the water-binder ratio of the fly ash slabs between 0.41 and 0.48.

Several investigators have assessed the salt-scaling resistance of CEM I/ fly ash concretes using the ASTM C672 procedure (Johnston, 1994; Bilodeau and Malhotra, 1992 and 1997; Whiting, 1989). The observations made are summarized in Table 4.12. A comparison of Table 4.12 with Table 4.11 shows that at a common water-binder ratio, de-icer scaling of concretes containing fly ash is more severe than with PC concretes and that the severity of scaling increases with increasing fly ash content. Johnston (1994) concluded that scaling was strongly related to water-Portland cement ratio and was essentially unrelated to water-binder ratio (see Figures 4.15 and 4.16).

**Table 4.12:** Some de-icer scaling results. CEM I/fly ash concretes.

Authors	w-b ratio	Fly ash content (%)	Air content (%)	De-icer scaling (kg/m <sup>2</sup> ) or visual rating
Johnston (1994)	0.48	20	6.7	1.5
	0.49	35	7.1	2.0
	0.43	35	6.8	3.9
	0.41	34	6.8	2.0
	0.37	35	7.7	2.2
	0.45	50	7.5	5.9
	0.42	50	7.1	3.7
	0.39	50	7.4	2.9
	0.36	50	7.8	2.1
Bilodeau and Malhotra (1997)	0.38	58	4.6-5.8	8.7, severe
	0.31	58	4.3-5.1	6.2, severe
	0.27	58	5.3-5.7	5.3, severe
	0.39	58	4.4-6.0	9.1, severe
	0.31	58	4.5-6.6	5.8, severe
	0.27	58	5.2-5.5	4.8, severe
	0.39	58	5.1-5.9	10.6, severe
	0.31	58	4.3-4.9	8.0, severe
	0.27	58	4.1-4.2	8.7, severe
Whiting (1989)	0.45	25	6.3-6.9	Slight to moderate
	0.50	25	5.8-6.1	Slight to moderate, to moderate
	0.60	25	6.0-6.3	Slight to moderate, to severe
	0.45	50	6.1-6.5	Moderate to severe
	0.48	50	6.2-6.6	Severe scaling
	0.57	50	6.3-6.8	Severe scaling

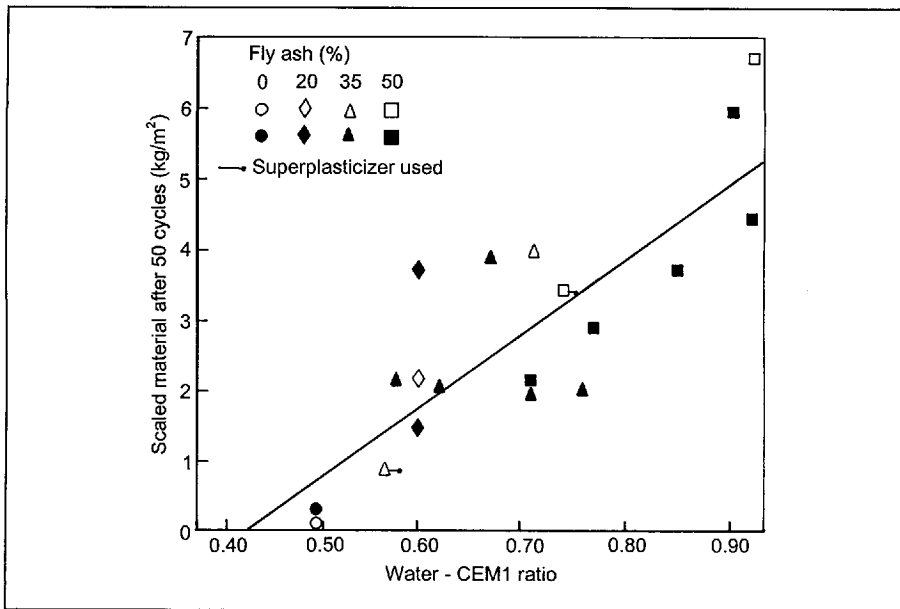


Figure 4.15: Relationship between scaling resistance and water-binder ratio (Johnston, 1994).

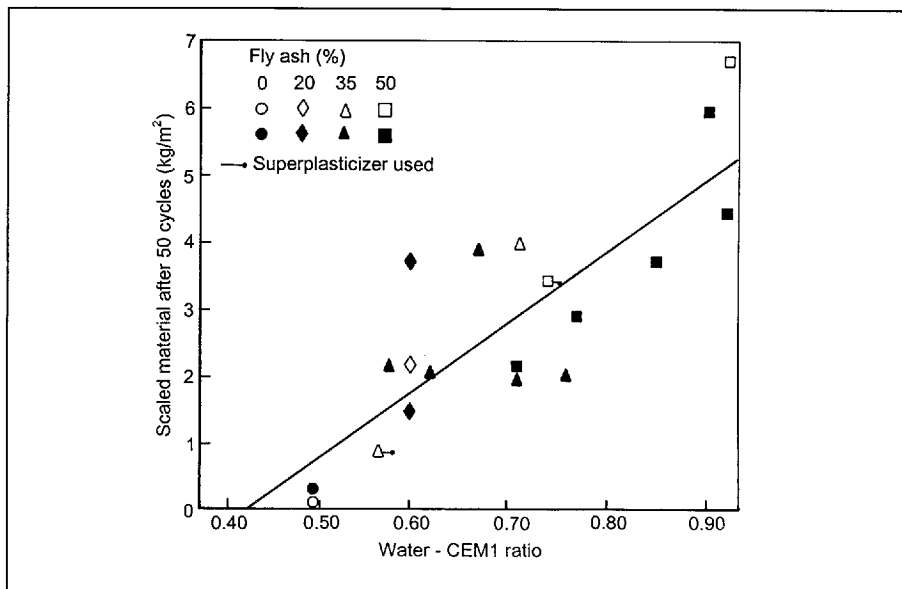


Figure 4.16: Relationship between scaling resistance and water-CEM I cement ratio (Johnston, 1994).

#### 4.7.6 CEM I/slag concretes: de-icing salt exposure

A number of investigations have been published dealing with the scaling resistance of air-entrained slag concretes exposed to de-icing salts (Klieger and Isberner, 1967; Hilsdorf and

Gunther, 1986; Gunther et al, 1987; Bilodeau et al, 1987, Luther et al, 1994; Stark and Ludwig, 1997; Bilodeau and Malhotra, 1997; Bonzel and Siebel, 1997).

Klieger and Isberner (1967) exposed concrete samples with a water-binder ratio of 0.40 to 300 freeze-thaw cycles and to flaked calcium chloride at the end of each freezing cycle. The concretes tested in the main programme had air contents in the range 4.95 to 5.20%. Slight scaling was observed with the CEM I concrete and slight to moderate scaling with the five slag cement concretes. In external exposure tests at a site in Illinois, no scaling was observed on concretes made using two of the slag cements after five winters of exposure to flaked calcium chloride.

Gunter et al (1987) determined the scaling resistance of slag concretes with a water-binder ratios of 0.40, 0.43 and 0.50 to sodium chloride using two test methods; one similar to ASTM C672 and the other to a more severe German Concrete Association test. The slag contents were 0, 30, 73 and 75%. In the case of the high slag content mixes tested according to 'ASTM C672', greater scaling than with the PC concretes was observed, irrespective of the length of the curing period, namely 1 to 48 days. It was also found that curing had no effect on scaling resistance of slag concretes and that carbonation had an adverse effect.

Bilodeau et al (1987) and Bilodeau and Malhotra (1997) determined the scaling resistance of slag concretes containing 0, 25 and 50% slag using three slags from Canada and the USA. The water-binder ratio was 0.55 and the air contents were in the range 6.1 to 6.9%. The concrete slabs were subjected to freezing and thawing in the presence of a 5% sodium chloride solution in accordance with ASTM C672-92. The slabs were subjected to a total of 50 cycles of freezing and thawing. The authors concluded that:

"Regardless of the percentage or type of slag used, the air-entrained concrete incorporating slag exhibited considerably more surface scaling than the reference concrete. According to the ASTM visual rating scale, the scaling of the concrete slabs incorporating slag was classified as moderate. The mass of the scaling residue was in excess of 1.4 kg/m<sup>2</sup>. (The scaling of the control slabs was slight).

The surface of the concrete slabs incorporating 50% slag as replacement for cement, showed somewhat more scaling than the surface of the concrete slabs incorporating 25% slag."

Luther et al (1994) carried out laboratory and field investigations on air-entrained concretes containing 0, 25, 35, 50 and 65% by mass of slag. The ASTM C672-92 test procedure was used; field slabs were placed in "an actively exposed parking lot". The authors concluded that, relative to the CEM I concretes, the laboratory results showed 25 and 35% slag concretes scaled less; 50% was broadly similar and 65% scaled more. The field tests showed that 25% slag concretes scaled less, 35% the same, 50% more and 65% did not scale.

Stark and Ludwig (1997) summarize German work on freezing/de-icing salt resistance of slag cements and also report their own test data. Bonzel and Siebel (1977) "found that the freezing/de-icing salt resistance of concretes rich in granulated blastfurnace slag (content >60%) cannot be improved by air-entraining agents in the same way as comparable cement concretes can." Similar results were obtained by Hilsdorf and Gunter (1986), who found that the freezing/de-icing salt resistance was not improved, even when air-entraining agents resulted in "highly



favourable air void spacing factors of 0.12 mm and 0.04 mm in the hardened concrete." According to the DIN 1045 standard, only grade 45 slag cements may be selected for use under heavy attack by de-icing salts (eg. in concrete road decks). This means that the use of grade 35 blastfurnace slag cements is not permitted in these cases. The slag content of grade 35 slag cements is greater than about 55% by mass of cement. Stark and Ludwig (1997) concluded the freezing/de-icing salt resistance of concretes rich in slag is strongly influenced by the carbonation of the surface layer. According to these authors, the coarser microstructure due to carbonation, as well as metastable carbonates, lead to a freezing/de-icing resistance within the carbonated layer which is lower than that of the concrete core.

#### 4.7.7 CEM I/silica fume concretes: de-icing salt exposure

According to Pigeon and Pleau (1995), "field tests have confirmed that air-entrained concretes containing silica fume can have a good de-icer salt scaling resistance under natural exposure conditions". The resistance to de-icer salt scaling of concrete containing silica fume has been investigated by the following authors:

Sorenson (1983)	RILEM Recommendation CDC2 (1977)	0 to 40% sf
Virtanen (1983)	Thawing in water, freezing in a salt solution	0 and 8% sf
Virtanen (1985)	Thawing in water, freezing in a salt solution	0 to 16% sf
Rasmussen (1985)	ISO 4846	0 and 10% sf
Gagne, Pigeon and Aïtcin (1991)	ASTM C672-92	0 and 6% sf
Langlois, Beaupré, Pigeon and Foy (1989)	ASTM C672-92	0 to 10% sf
Bilodeau and Malhotra (1997)	ASTM C672-92	0 and 8% sf

These authors have shown that the de-icer salt scaling resistance of concrete containing up to 10% silica fume, based on either similar compressive strength or water-binder ratio, is similar to that of comparable CEM I concretes, but at contents of 20 and 30% it can be reduced.

#### 4.7.8 Minimum specification for air-entrained concrete subject to exposure class XF4

A minimum concrete specification for air-entrained concrete exposed to de-icing salts and freeze-thaw attack whilst in a saturated state, is given in Table 4.13. This minimum specification is based on the work reported in the previous sub-Sections. Due to lack of data, guidance is not possible for concretes containing CEM II/A-L and CEM II/B-L cements. Also, due to lack of data, a minimum binder content is not given.

**Table 4.13:** Minimum specification for air-entrained concrete subject to exposure class XF4.

Maximum w/c ratio	0.55
Minimum strength class	C30/37
Minimum air content for aggregate with a nominal size of 20 mm	4%
Cement types and equivalent combinations	CEM I, I/SR, II/A-D, II/A-S, II/A-V, II/B-S, II/B-V, III/A <sup>+</sup> III/B <sup>+</sup> , IV/A, IV/B <sup>+</sup>
Other requirement	Freeze-thaw resistant aggregate

Note: Premature exposure to freezing and thawing may result in inadequate performance.

<sup>+</sup> The literature indicates that CEM III/A, III/B and IV/B cements may not be suitable for the top surfaces of pavements.

## 4.8 Non-air-entrained concretes

### 4.8.1 Performance of high quality concretes

Although it has been shown, both from field performance and laboratory testing, that air entrainment enhances the freeze-thaw resistance of concrete, there are numerous non-air-entrained CEM I concretes exposed to the elements that are performing well. Non-air-entrained concretes do not perform well in accelerated laboratory freeze-thaw tests such as ASTM C666-92 (Procedure A), when the water-CEM I ratio is  $> 0.35$  (Okada et al, 1981; Yamako et al, 1988);  $> 0.5$  (Kobayashi et al, 1981; Foy et al, 1988; Gagné et al, 1990). Similar, or slightly higher, water-CEM I ratios give good resistance to de-icer scaling when the concretes are subject to the ASTM C672-92 test (Foy et al, 1988; Gagné, 1991). Hammer and Sellevold (1990) observed good resistance to de-icer scaling for non-air-entrained CEM I concretes having a water-cement ratio of 0.37 tested according to the Swedish Standard SS 13 72 44). Dhir et al (1987) observed linear expansions of less than 0.3% after 1,200 freeze-thaw cycles, when the water-cement ratio of the non-air-entrained concrete was 0.49, an expansion greater than 0.3% being regarded as failure. In the work of Dhir et al, the concretes were frozen in air and thawed in water, the extremes in temperature being similar to those in EN 480-11: 1995 and ASTM C666.

From the work of the above investigators, Pigeon and Pleau (1995) concluded "It thus seems that non-air-entrained high-strength concretes generally have a good scaling resistance and that the main concern about these concretes is their resistance to internal micro cracking." These investigators also conclude from "the small amount of data that is available", "that the use of fly ash and slag in high strength concrete can cause scaling problems, and the reasons behind these problems remain to be investigated." It has also been observed that non-air-entrained concretes prepared using blended cements containing between 10 and 30% fly ash (Matthews, 1989; Whiting, 1987) and Portland limestone cements, with limestone contents between 5 and 25% (CEM II/A-L, B-L, Matthews, 1989), give inferior freeze-thaw resistance to CEM I concretes. Fagerlund (1988) has reported inferior resistance even at the 5% fly ash level. Table 4.14 gives the results of unpublished work carried out by Hobbs (1988), where the concrete prisms, 75 x 75 x 250 mm in size, containing 0 and 35% fly ash, were frozen and thawed in water. Note that at a w-b ratio of 0.36, equal performance is obtained to the CEM I concrete with a w-c ratio of 0.47.

In the case of non-air-entrained concretes containing less than 10% silica fume, a number of investigators have observed similar or superior de-icing salt scaling resistance and freeze-thaw resistance to the non-air-entrained CEM I controls based on similar compressive strengths or water-binder ratios, for example:

Cheng-yi, Huang and Feldman (1985)	Mortars. ASTM C666-92, Procedure B	10% sf
Yamato, Emoto and Soeda (1983)	ASTM C666-92, Procedure B	5 and 10% sf
Sorenson (1983)	RILEM Recommendation CDC2 (1977)	0 and 10% sf.
Virtanen (1983)	Dilation test also frozen in salt solution, thawed in water	0 and 6% sf
Gagne, Pigeon and Aitcin (1991)	ASTM C672-92	0 and 6% sf

**Table 4.14:** Expansion and reduction in resonance frequency of non-air-entrained concretes subject to 5 and 10 freeze-thaw cycles.

CEM I content (kg/m <sup>3</sup> )	Fly ash content (kg/m <sup>3</sup> )	w-b ratio	14 day 20°C wet cure followed by 5 F/T cycles		91 day 20°C wet cure followed by 10 F/T cycles	
			Expansion (%)	Reduction in resonance frequency (%)	Expansion (%)	Reduction in resonance frequency (%)
367	-	0.47	0.09	17	0.05	12
368	-	0.47	0.09	18	0.04	13
279	-	0.64	0.24	63	0.07	29
278	-	0.65	0.20	36	0.12	32
201	-	0.86	0.33	58	0.32	69
284	152 <sup>+</sup>	0.36	0.09	24	0.01	3
220	118 <sup>+</sup>	0.43	0.24	54	0.06	15
237	129 <sup>+</sup>	0.44	0.14	47	0.05	11
183	93 <sup>+</sup>	0.56	0.31	60	0.09	30
279	150 <sup>§</sup>	0.40	0.18	44	0.14	34
234	127 <sup>§</sup>	0.47	0.32	62	0.16	40
217	117 <sup>§</sup>	0.51	0.37	60	0.20	53
181	97 <sup>§</sup>	0.61	0.43	62	0.22	55

<sup>+</sup> BS 3892: Part 1 fly ash

<sup>§</sup> BS 3892: Part 2 fly ash

The extent to which the laboratory test data are applicable to field concrete is open to question. For example, Litvan et al (1980) concluded from their field tests on precast concrete elements that "conventional laboratory tests", "yielded predictions which show little, if any, agreement with actual field performance." A critical observation relating to laboratory test data has been made by Sturup and Clendenning (1969), namely "we have been puzzled", "when all non-air-entrained concretes broke down rapidly in our rapid water-freeze test despite excellent

performance of many hydraulic structures built before the advent of air entrainment". This latter comment is also valid for non-air-entrained concretes in the UK. The authors of this Chapter are not aware of incidences of freeze-thaw attack in high grade concretes subject to UK external exposure. For example, factory-produced pretensioned concrete members and factory-produced post-tensioned concrete members have a history of satisfactory use in situations where the members are likely to be exposed to pr EN 206 exposure classes XF3 and XF4. Visual cracking, spalling and scaling attributable to freeze-thaw attack has not been reported. These concretes normally have water-cement ratios below 0.45.

A number of field exposure tests have been carried out on non-air-entrained concretes. These are summarized below:

**Oleson and Verbeck (1967):** Thin-walled concrete boxes, filled with saturated sand, were exposed to relatively severe natural freeze-thaw cycling on a site near Chicago, Illinois. The performance of the boxes was based on a visual rating ranging from 1 (no deterioration) to 6 (practically complete deterioration). After five years exposure, non-air-entrained concretes with w-c ratios in the range 0.4 to 0.45 and a slump of 75 mm, exhibited similar good performance to air-entrained concretes (3.7 to 6.9% air) with w-c ratios in the range 0.4 to 0.6. The cements tested were ASTM Types I, II, III, IV and V.

**Sturrup and Clendenning (1969):** Concrete blocks were partially immersed by placing them in filled containers. The exposure site was on the western outskirts of Toronto. The concrete blocks had w-c ratios of 0.5, 0.6, 0.8 and 1.0. The concretes were non-air-entrained (1.3 to 1.7% air) and air-entrained (5.0 to 6.1% air). After five years' exposure, the non-air-entrained concretes with w-c ratios of 0.5 and 0.6 exhibited similar high durability to the air-entrained concretes.

**Axon, Murray and Rucker (1969):** Concrete beams were stored on wet sand in an outdoor exposure pit in Missouri. Assuming a bulk density for the Portland cement of 1500 kg/m<sup>3</sup> gives w-c ratios in the range 0.45 to 0.53. After 20 years' exposure, it was concluded that the freeze-thaw performance was dependent upon coarse aggregate type and not air-content, except when coarse aggregates of low resistance to freeze-thaw cycles were used, and here air entrainment had an adverse effect.

**Sturrup, Hooton and Clendenning (1983):** In one section of the Otto Holden dam in Ontario, constructed in 1950, two non-air-entrained blocks were cast, one contained the normal CEM I concrete used on the job and the other 30% by mass of fly ash. The binder content was 312 kg/m<sup>3</sup>. Examination after 20 years revealed that, at the water-line on the upstream face, the fly ash concrete was showing only slightly more surface spalling than the adjacent plain concrete.

**Malhotra, Carette and Bremner (1988):** Non-air-entrained CEM I concretes with a water-cement ratio of 0.5 exhibited poor resistance to freeze-thaw attack when exposed to mid-sea tide level, Bay of Fundy, Treat Island, Maine.

**Klieger and Landgren (1969):** In a study of the resistance of concrete slabs to de-icing salt exposure, a non-air-entrained CEM I slab with a w-c ratio of 0.53 was cast in the laboratory during the winter, cured for seven days, and then stored outdoors. After five years' exposure, inspection showed that the slab was only slightly scaled by de-icing salt exposure. The air-entrained control slab exhibited marginally better performance.

**Weaver and Isabelle (1968):** An examination of sidewalks in the Montreal area, prepared from non-air-entrained CEM I concretes with a w-c ratio of about 0.6, showed extensive scaling after about 15 years' exposure to de-icing salts.

**Keyser (1968):** In 1955 non-air-entrained CEM I concrete slabs with water-cement ratios in the range 0.61 to 0.63 were placed at the Research Laboratory, Department of Public Works, Montreal. Within one year the surfaces of the slabs had scaled.

**Osborne (1992):** CEM I and slag cement concretes were placed as 100 mm cubes in a tidal marine environment at an exposure site near Southend in the UK. Three CEM I cements and five slags were used. The nominal binder content was 380 kg/m<sup>3</sup>. The free water-binder ratio was approximately 0.45. The slags were used in the proportions 0, 60, 70 and 80 per cent by mass. At an age of five years, it was observed that "no frost damage has occurred in the Portland cement concretes". It was concluded that "four Portland cement concretes with a nominal cement content of 380 kg/m<sup>3</sup> and total w-c of 0.5 were suitable for use without air entrainment in conditions of severe freezing and thawing which occurred at this exposure site". It was also observed that "All blastfurnace slag cement concretes in the tidal zone had suffered frost damage to some degree on their surfaces, in the form of 'pop-outs' and spalling, characteristic of freeze-thaw attack".

**Wiebenga (1980):** An examination of concrete structures along the coast of the Netherlands lead Wiebenga to conclude that slag cement concrete in a marine environment "appears to be more susceptible to surface damage (up to 5-10 mm) due to frost than Portland cement concrete".

**Matthews (1995):** CEM I and fly ash cement concretes were placed, as 100 mm cubes, in a tidal marine environment at the same exposure site used by Osborne (1992). Three Portland cements and four different fly ashes were used. The binder contents were mainly 300 and 400 kg/m<sup>3</sup> and the fly ash contents were mainly 0, 20 and 40%. At an age of 10 years, it was concluded that "in the tidal zone, pfa concretes suffered greater effects of freezing and thawing than the Portland cement concretes, which exhibited little or no evidence of freeze-thaw damage". The latter observation was also valid at a CEM I content of 270 kg/m<sup>3</sup> (w/c  $\approx$  0.6) but not at a CEM I content of 200 kg/m<sup>3</sup> (w/c  $\approx$  1.0).

**Bijen (1996):** Performance of concretes in the Netherlands has led to the view that concretes with w-c ratios below 0.45 are resistant to de-icer scaling. However, Bijen notes that for road structures, mostly CEM I concrete or fly ash cement concrete is used to avoid an early high rate of surface scaling, which can occur with blastfurnace slag cement concretes, indicating that for the latter the scaling was viewed as excessive.

#### **4.8.2 Minimum specification for non-air-entrained concrete subject to exposure classes XF3 and XF4**

It is clear from the reported field studies that non-air-entrained concretes with water-cement ratios below or equal to 0.45 can be highly resistant to the pr EN 206 exposure classes XF3 and XF4, provided the coarse aggregates employed are not susceptible to freeze-thaw attack. This observation may only be applicable to CEM I (Portland) and CEM II/A-D (Portland silica fume cement) concretes. In the case of non-air-entrained concretes containing fly ash or slag, field observations are sparse but both field observations and laboratory performance tests indicate lower freeze-thaw resistance than CEM I concrete, particularly for concretes subject to exposure class XF4. As a consequence of the sparse data it is not possible to make recommendations for non-air-entrained concretes containing fly ash or slag subject to exposure class XF4. Due to an absence of a long-term performance record and the limited laboratory test performance data, it is also not possible to recommend non-air-entrained concretes prepared using CEM II/A-L and

II/B-L (Portland limestone cements). The experimental data that does exist indicate lower freeze-thaw resistance than CEM I concretes (Matthews, 1989).

A minimum concrete specification, based on the work discussed above, for non-air-entrained concrete subject to exposure classes XF3 and XF4, is given in Table 4.15. Due to lack of data, a minimum binder content is not given.

**Table 4.15:** Minimum concrete specification for non-air-entrained concrete subject to exposure classes XF3 and XF4.

Exposure class	XF3, saturated without salt	XF4, saturated with salt
Maximum w-c ratio	0.45	0.45
Minimum strength class	C40/50	C40/50
Cement types and equivalent combination	CEM I, CEM I/SR, II/A-D, II/A-S, II/A-V, II/B-S, II/B-V, IV/A	CEM I, CEM I/SR, CEM II/A-D
Other requirement	Freeze-thaw resistant aggregate	

Note 1: Premature exposure to freezing and thawing may result in inadequate performance.

## 4.9 Minimum requirements for concrete to resist freeze-thaw attack

### 4.9.1 50 years vs. 100 years required working life

There is no direct evidence available concerning the very long term (ie. up to 100 years) freeze-thaw performance of concretes, plain or air-entrained. Freeze-thaw attack is, however, an event-dependent process rather than a continuous process such as carbonation or ingress of chlorides. Freeze-thaw damage can occur whenever certain critical conditions are fulfilled, eg. degree of saturation. Fulfilment of these critical conditions is not time dependent, thus damage could occur as easily in the twentieth year of exposure, say, as it could in the second year. This is not to say that failure of the concrete will necessarily occur after a single event: damage is likely to be cumulative, thus the extent of the damage, should any occur, will increase with the number of damaging events.

Concrete that is freeze-thaw resistant in practice can eventually fail under accelerated testing, which would imply that freeze-thaw attack is time-dependent. This apparent conflict with the performance described above is probably the result of the unrealistic way in which accelerated testing is performed compared with conditions existing in practice. There is evidence to suggest that the repeated immersion in water and subsequent freezing causes the degree of saturation to rise in a way that would be unlikely to occur in practice. Such is the extent of this effect that it can eventually overcome the normal resistance offered by air-entrainment.

If the concrete is designed and constructed sufficiently well to be able to resist the attack of the freezing and thawing conditions to which it is exposed, then it is likely to continue its satisfactory performance unless the conditions of attack become critical (eg. the degree of saturation of the concrete becomes critical). It is thus likely, at least in theory, that concrete that is freeze-thaw resistant for 50 years will continue to be resistant for 100 years.

## 4.9.2 Comparisons with existing specifications

### 1 *Relevant exposure classes in British Standards*

Comparison with existing specifications in British Standards is very difficult due to the limitations of the existing BS method of exposure classification in which the effect of freezing and thawing is considered together with other actions. The BS also does not specifically consider the role of salt in the aggressivity of the freeze-thaw action. The relevant BS 8110 (1997b) exposure conditions are:

Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing or severe condensation.
Very severe	Concrete surfaces occasionally exposed to sea water spray, or de-icing salts (directly or indirectly). Concrete surfaces exposed to corrosive fumes or severe freezing conditions whilst wet.
Most severe	Concrete surfaces frequently exposed to seawater spray or de-icing salts (directly or indirectly). Concrete in seawater tidal zone down to 1 m below lowest water level.

Tables 3.4 and 4.8 of BS 8110 (1997b) for reinforced and prestressed concrete, respectively, and clause 6.2.3.2 give recommendations for quality which, for concrete exposed to freezing whilst wet, include air-entrainment if the concrete grade is less than C50. Air-entrainment is not mentioned in 'severe' conditions. Recommendations for unreinforced concrete are given in Table 6.2, but there is no direct reference to air-entrainment to resist freezing and thawing, although its use appears to be implied.

The design code for concrete bridges (BS 5400: Part 8, 1978) uses an exposure classification system that is similar to that in BS 8110, except that no mention is made of freezing in the definitions. Nevertheless, a note in Table 4.13 says, for 'severe', 'very severe' and 'extreme' conditions, that air entrainment should be specified where the surface is liable to freezing whilst wet.

### 2 *Comparisons of recommendations for minimum concrete quality*

XF1 (moderate saturation, no salt) can be taken as being approximately equivalent to the BS 8110 condition 'severe'. The minimum concrete quality recommended in BS 8110 is C40, with a 0.55 w-c ratio for reinforced and prestressed concrete - but C35 with a 0.60 w-c ratio is recommended for unreinforced concrete. This recommendation also relates to normal outdoor exposed reinforced concrete as it is included in the same exposure class. The recommendation of C30 given in BS 5400 (Part 8: 1978) is regarded by the authors as not reflecting current specifications for concrete bridges. This Chapter recommends that the minimum concrete quality should be the equivalent of the BS 8100 (1997b) recommendation for reinforced and prestressed concrete. This is an increase in quality for unreinforced concrete and for that given in BS 5400 (but which is believed to be unrepresentative of concrete used in bridges in areas classed as exposed to 'severe' conditions).

XF2 (moderate saturation, with salt) can be taken as being equivalent to the BS 8110 'very severe' exposure because of the presence of salt but, because of the 'modest saturation' exposure condition, there should be no need for air-entrainment. Some members of the BSI task group on freeze-thaw have argued that there can be no such condition on the basis that

the presence of salt will mean the surface is always saturated. The authors of this report believe that condition XF2 is valid and that BS 5400 examples (for 'very severe') of 'walls and supports adjacent to the carriageway' and 'adjacent to the sea' relate to this condition and not XF4. The relevant minimum recommendation in both BS 5400 (Part 8: 1978) and BS 8110 (1997b) for 'very severe' is C40 with 0.55 w-c ratio. This Chapter recommends that the minimum concrete quality should be the equivalent of the current BS recommendation.

XF3 (high saturation, no salt) can be taken as equivalent to 'very severe' because of the 'severe freezing whilst wet' with the high level of saturation leading to the need for air-entrainment. The relevant recommendation for minimum concrete quality in both BS 5400 and BS 8110 is C40; 0.55 w-c ratio with air-entrainment. BS 5328 (1997c) allows the concrete grade to be dropped to C35 because of the strength loss that accompanies air-entrainment; the water-cement ratio requirement, however, remains at 0.55. Non-air-entrained C50 concrete can be used as an alternative to air-entrained concrete. The recommendations in this Chapter (C 25/30, 0.6 w-c ratio for Portland cement, air-entrained) thus represent a reduction in minimum concrete quality compared with that currently recommended but with the same requirement for air-entrainment.

XF4 (high saturation, with salt) must, as XF3, also be taken as equivalent to 'very severe' because of the lack of acknowledgement in British Standards of the increased severity of freeze-thaw when salt is present. The equivalent BS requirements are thus the same as those for XF3. The recommendations in this Chapter (C 30/37, 0.55 w-c ratio for Portland cement, air-entrained) represent a similar minimum strength grade but at the same minimum water-cement ratio and requirement for air-entrainment.

### 4.9.3 Minimum requirements

Table 4.16 gives the minimum requirements for concretes to resist freeze-thaw attack for 100 years. The minimum concrete qualities are applicable to the UK, and are based on UK and International studies of the laboratory and field performance of CEM I, I/SR, II/B-S, II/A-D, II/B-V and III/A concretes subject primarily to classes XF3 and XF4 exposure.



**Table 4.16:** Limiting values for composition and properties of concrete exposed to freezing and thawing for an intended working life of up to 100 years.

Exposure class	Moderate saturation without salt	Moderate saturation with salt	High saturation without salt		High saturation with salt	
	XF1	XF2	XF3		XF4	
Minimum strength class	C30/37	C35/45	C25/30	C40/50 <sup>+</sup>	C30/37 <sup>+</sup>	C40/50 <sup>+</sup>
Maximum w-c ratio	0.55	0.55	0.60	0.45	0.55	0.45
Minimum air content for nominal 20 mm maximum aggregate size	-	-	4%	-	4%	-
Other requirements			Freeze-thaw resistant aggregate			
Cements	All <sup>§</sup>		All <sup>§</sup>	I, I/SR II/A-S II/B-S II/A-D II/A-V II/B-V IV/A	I, I/SR II/A-S II/B-S II/A-D II/A-V II/B-V III/A <sup>+</sup> IV/A IV/B <sup>+</sup>	I, I/SR II/A-D
For combinations:						
Slag (%)	≤ 80		≤ 65	≤ 35	≤ 65 <sup>+</sup>	
Fly ash (%)	≤ 55		≤ 55	≤ 35	≤ 55 <sup>+</sup>	

<sup>+</sup> Due to limited published data, a minimum specification is not given for other common cements.

<sup>§</sup> Excluding CEM II/B-L, CEM III/B and CEM III/C cements. For these cements, judgements were not possible due to lack of published data.

<sup>+</sup> The literature indicates that III/A and IV/B cements may not be suitable for the top surfaces of pavements.

## 4.10 References

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## CHAPTER 5

## Minimum requirements for concrete to resist chemical attack

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### 5.1 Summary

The draft European Standard pr EN 206 classifies environments giving a risk of chemical attack into three exposure classes (BSI, 1997a):

XA1	XA2	XA3
Slight aggressive chemical environment	Moderately aggressive chemical environment	Highly aggressive chemical environment

This Chapter reviews the results from published studies, primarily by BRE, of the performance of concretes in sulfate and acid environments, the objective being to establish the minimum qualities of concrete for each exposure class necessary to achieve a design working life of 100 years (class 4 of ENV 1991-1: 1994, European Committee for Standardization). To align with current UK practice, exposure classes XA1, XA2 and XA3 are each split into two sub-classes. The cements considered are CEM I (Portland cements), CEM II/A-L, B-L (Portland limestone cement) and binders containing fly ash, slag and silica fume.

The recommendations made are applicable to well-compacted cast-in-situ concrete, 140 to 450 mm in thickness and exposed on all faces to aggressive groundwaters.

## 5.2 Introduction

All concretes are vulnerable to varying degrees to attack by salts present in solution and to attack by acids. Flowing waters are more aggressive to concrete than static waters. Sulfates and acids in groundwater are fairly common in the UK and consequently precautions are often necessary to minimize deterioration of buried concrete.

In soil and natural groundwater, sulfates of sodium, potassium, magnesium and calcium are common. Sulfate in groundwater can also derive from industrial fertilizers or from industrial effluents. Sulfates generally enter the concrete by diffusion. Groundwater affects concrete as follows:

- Calcium sulfate attacks calcium aluminate hydrate forming ettringite.
- Sodium sulfate attacks calcium hydroxide and calcium aluminate hydrate forming gypsum and ettringite respectively.
- Ammonium sulfate attacks hydrated paste forming gypsum.
- Magnesium sulfate attacks calcium hydroxide, calcium aluminate hydrates and calcium silicate hydrates.

In the affected parts of concrete members the attack can sometimes result in expansion and/or loss in strength. Magnesium sulfate attack, at a given sulfate ion solution concentration, is recognised to be more severe than the attack from other sulfates.

The susceptibility of mortars and concrete elements to attack by sulfate-bearing soils and groundwaters has been known for many years. This susceptibility is generally regarded as being related to the tricalcium aluminate content of the cement and the water-cement ratio of the concrete or mortar.

Acid attack on concrete causes erosion of the surface by dissolving the hydraulic binders and some aggregates (eg. limestones). For inorganic acids such as sulfuric acid, the higher their concentration the lower the pH and the greater the attack. Sulfuric acid forms from the oxidation of iron sulfide minerals in soil. In the case of organic acids such as fulvic and humic acid, the higher the concentration the greater the attack, but they may produce only moderately low values of pH.

The concrete qualities given by cement content and water-cement ratio for the common cements used in the UK, which are recommended in BS 5328. Part 1 (BSI, 1997b) are reproduced in Tables 5.1 to 5.4. Table 5.1 deals with in-situ concretes 140 to 450 mm thick subject to exposure from sulfate in soils with permeability above  $10^{-5}$  m/s. Table 5.3 deals with other types of exposure and types of construction. Recommendations for minimum qualities of concrete exposed to acidic conditions in natural ground are given in Table 5.4. Table 5.2 classifies UK common cements into three groups: 1, 2 and 3.

The draft European standard pr EN 206 (BSI, 1997a) classifies concretes subject to a risk of chemical attack into three exposure classes (Table 5.5). Exposure class XA1 is equivalent to the UK sulfate class 1 when the  $\text{SO}_4$  in groundwater is below 0.4 g/l and the UK sulfate class 2 when the  $\text{SO}_4$  in groundwater is above 0.4 g/l. Exposure class XA2 is equivalent to the UK sulfate classes 2 and 3, and exposure class XA3 is equivalent to the UK sulfate classes 4A and 4B. In

order to align with current UK practice, the exposure classes XA1, XA2 and XA3 are each split into two sub-classes as shown in Table 5.6.

The UK established practice is to measure  $\text{SO}_4$  in soil using the 2:1 water:soil extract method (BS 1377:1990: Parts 1 and 3) and in the context of Mg in water to specify differently for groundwater, brackish water and sea water. For groundwater this adds two further rows to Table 5.5 as given in Table 5.6. Sea water and brackish water can lead to high Mg ion contents, but most of the Mg ions are in combination with the chloride ion and not the sulfate ion, and are not damaging.

**Table 5.1:** Recommendations for concrete exposed to sulfate attack (BS 5328: Part 1: 1997: Table 7a; BRE Digest 363, 1991, Table 1).

Sulfate class	Exposure conditions: concentration of sulfate and magnesium					Recommendations		
	In groundwater		In soil or fill			Cement group (from Table 5.2)	Dense fully compacted concrete made with 20 mm nominal maximum size aggregates conforming to BS 882 or BS 1047	
	$\text{SO}_4$ (g/l)	Mg (g/l)	By acid extraction $\text{SO}_4$ (%)	By 2:1 water/soil extract $\text{SO}_4$ (g/l)	Mg (g/l)		Cement content not less than (kg/m <sup>3</sup> )	Free water-cement ratio not more than
1	< 0.4	–	< 0.24	< 1.2	–	1, 2, 3	–	–
2	0.4 to 1.4	–	Classify on the basis of a 2:1 water/soil extract	1.2 to 2.3	–	1 2 3	330 300 280	0.50 0.55 0.55
3	1.5 to 3.0	–		2.4 to 3.7	–	2 3	340 320	0.50 0.50
4A	3.1 to 6.0	≤ 1.0		3.8 to 6.7	≤ 1.2	2 3	380 360	0.45 0.45
4B	3.1 to 6.0	> 1.0		3.8 to 6.7	> 1.2	3	360	0.45
5A	> 6.0	≤ 1.0		> 6.7	≤ 1.2	As for class 4A plus surface protection		
5B	> 6.0	> 1.0		> 6.7	> 1.2	As for class 4B plus surface protection		

Note: Classification on the basis of groundwater samples is preferred. The limit on water-soluble magnesium does not apply to brackish water (chloride content between 12 g/l and 18 g/l).

The object of the work described in this Chapter is to propose the minimum concrete qualities for these exposure classes which are necessary for working lives of 100 years (ie. Class 4 of ENV 1991-1:1994). The recommendations are applicable to well-compacted cast-in-situ concrete 140 to 450 mm in thickness and exposed on all faces to soils with a permeability above  $10^{-5}$  m/s. The cements considered are CEM I cements and binders containing fly ash, ground granulated blastfurnace slag, limestone and silica fume.

**Table 5.2:** Types of cement (BS 5328: Part 1: Table 7b).

Group	Description
1	<ul style="list-style-type: none"> <li>a) Portland cement conforming to BS 12</li> <li>b) Portland blastfurnace cements conforming to BS 146</li> <li>c) High slag blastfurnace cement conforming to BS 4246</li> <li>d) Portland pulverized-fuel ash cements conforming to BS 6588</li> <li>e) Pozzolanic pulverized-fuel ash cement conforming to BS 6610</li> <li>f) Portland limestone cement conforming to BS 7583<sup>+</sup></li> <li>g) Combinations of Portland cement conforming to BS 12 with ggbs conforming to BS 6699</li> <li>h) Combinations of Portland cement conforming to BS 12 with pulverized-fuel ash conforming to BS 3892: Part 1</li> </ul>
2	<ul style="list-style-type: none"> <li>a) Portland pulverized-fuel ash cements conforming to BS 6588, containing not less than 26% of pfa by mass of the nucleus or combinations of Portland cement conforming to BS 12 with pfa conforming to BS 3892: Part 1, where there is not less than 25% pfa and not more than 40% pfa by mass of the combination.</li> <li>b) High slag blastfurnace cement conforming to BS 4246, containing not less than 74% slag by mass of nucleus or combinations of Portland cement conforming to BS 12 with ggbs conforming to BS 6699 where there is not less than 70% ggbs and (not) more than 85% ggbs by mass of the combination.</li> </ul> <p>Note 1: For group 2b cements, granulated blastfurnace slag with alumina content greater than 14% should be used only with Portland cement having a tricalcium aluminate (C<sub>3</sub>A) content not exceeding 10%.</p> <p>Note 2: The nucleus is the total mass of the cement constituents excluding calcium sulfate and any additives such as grinding aids.</p>
3	Sulfate-resisting Portland cement conforming to BS 4027

<sup>+</sup> Portland limestone cement should only be used in class 1 sulfate conditions.

**Table 5.3:** Modifications to Table 5.1 for other types of exposure and types of construction.<sup>+</sup>  
(BS 5328: Part 1: Table 7c; from BRE Digest 363, 1991, Table 2).

Static groundwater <sup>§</sup>	For classes 2, 3 and 4 the requirements for cements, cement content and free water-cement ratio given in Table 7a may be lowered by one class
Basement, embankment or retaining wall	If a hydrostatic head greater than five times the thickness of the concrete is created by the groundwater, the classification in Table 7a should be raised by one class. This requirement can be waived if a barrier to prevent moisture transfer through the wall is provided
Cast-in-situ concrete over 450 mm thick. Precast ground beams, walls units or piles with smooth surfaces which after normal curing have been exposed to air but protected from rain for several weeks	For classes 2, 3 and 4 the requirements for cement group, cement content and free water/cement ratio given in Table 7a may be lowered by one class  For cast-in-situ reinforced concrete, special consideration should be given to the need to maintain adequate cover to the reinforcement.
Cast in-situ concrete (other than ground floor slabs) less than 140 mm thick or having many edges and corners	The classification in Table 7a should be raised by one class

<sup>+</sup> Any reductions in sulfate class allowed by this table only apply if other durability and structural considerations permit.

<sup>§</sup> Nominally dry sites or soils with permeability less than  $10^{-5}$  m/s as given in Figure 6 of BS 8004:1986 (eg. unfissured clay) where it is decided that the groundwater is essentially static (see BRE Digest 363).

**Table 5.4:** Modifications to tables 5.1 and 5.3 for concrete exposed to attack from acids in natural ground. (BS 5328: Part 1: Table 7d).

pH	Mobility of water	Change in classification with respect to minimum cement content and maximum free water-cement ratio for the cement group recommended on the basis of sulfate class in Tables 5.1 and 5.3
5.5 to 3.6	Static	No change
	Mobile	Raise by one sulfate class
3.5 to 2.5	Static	Raise by one sulfate class
	Mobile	Raise by one sulfate class

Note: If a cement from group 1 has been selected during the classification for sulfate, when raising by one class in accordance with this table, the cement type may still be used taking as minimum cement content the requirement for group 2 cements.

**Table 5.5:** Limiting values for exposure classes for chemical attack (pr EN 206 Table 2).

The aggressive chemical environments classified below are based on natural environments at water/soil temperatures between 5°C and 25°C and a water velocity sufficiently slow to approximate to static conditions. The most onerous value for any single chemical characteristic determines the class. When two or more aggressive characteristics lead to the same class, the environment shall be classified into the next higher class. Determination of the aggressive characteristics should use the reference test methods given in this table.				
Chemical characteristics	Test method	XA1	XA2	XA3
SO <sub>4</sub> <sup>2-</sup> mg/l in water	EN 196-2	≥200 and ≤ 600	> 600 and ≤ 3000	> 3000 and ≤ 6000
SO <sub>4</sub> <sup>2-</sup> mg/kg in soil <sup>†</sup>	EN 196-2 <sup>§</sup>	≥ 2000 and ≤ 3000 <sup>‡</sup>	> 3000 <sup>‡</sup> and ≤ 12000	> 12000 and ≤ 24000
pH of water	DIN 4030-2	≤ 6.5 and ≤ 5.5	< 5.5 and ≥ 4.5	< 4.5 and ≥ 4.0
Acidity of soil	DIN 4030-2	> 20 Baumann Gully		
CO <sub>2</sub> mg/l aggressive in water	pr EN WWV	≥ 15 and ≤ 40	> 40 and ≤ 100	> 100
NH <sub>4</sub> <sup>+</sup> mg/l in water	ISO 7150-1 or ISO 7150-2	≥ 15 and ≤ 30	> 30 and ≤ 60	> 60 and ≤ 100
Mg <sup>2+</sup> mg/l in water	ISO 7980	≥ 300 and ≤ 1000	> 1000 and ≤ 3000	> 3000
<sup>†</sup> Clay soils with a permeability below 10 <sup>-5</sup> m/s may be moved into a lower class. <sup>§</sup> The test method prescribes the extraction of SO <sub>4</sub> <sup>2-</sup> by hydrochloric acid; alternatively, water extraction may be used, if experience is available in the place of use of the concrete. <sup>‡</sup> The 3000 mg/kg limit is reduced to 2000 mg/kg, where there is a risk of accumulation of sulfate ions in the concrete due to drying and wetting cycles or capillary suction.				

**Table 5.6:** Additional limiting values for exposure classes and sub-exposure classes for chemical attack.

Chemical characteristics	XA1		XA2		XA3	
	Class 1 = XA1A	Class 2 = XA1B	Class 2 = XA2A	Class 3 = XA2B	Class 4A = XA3A	Class 4B = XA3B
SO <sub>4</sub> in ground-water (mg/l)	≥200 & ≤400	>400 & ≤ 600	>600 & ≤1440	>1440 & ≤3000	>3000 & ≤6000	
SO <sub>4</sub> by 2:1 water: soil extract (g/l)	≥0.6 & ≤1.2	>1.2 & ≤1.8	>1.8 & ≤2.3	>2.3 & ≤3.7	>3.7 & ≤6.7	>3.7 & ≤6.7
Mg in natural groundwater (g/l)	≤1.0					>1.0

## 5.3 Chemical attack in the UK

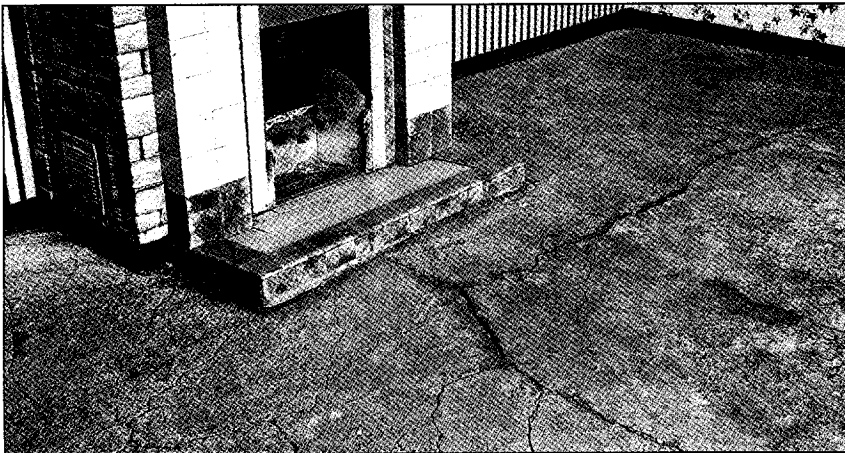
### 5.3.1 Sulfate attack

In the UK sulfate attack is believed to be a relatively rare cause of concrete deterioration. Examples of sulfate attack which have been observed include:

- The expansion of ground floor slabs placed on sulfate-containing fill without the use of a waterproofing membrane (Lea, 1968; BRE Digest 363, 1991, Figure 5.1).

- Sulfate attack on strip foundations and underpinning concrete when groundwater levels rose following construction on an adjacent site that altered ground drainage (Lea, 1968).
- Severe deterioration in the tops of in-situ CEM I/SR concrete piles below a four-year-old housing estate (Crammond and Halliwell, 1995). These piles had been subject to Class XA2B exposure.
- A CEM I concrete strip foundation beneath a 12-year-old garage subject to Class XA2A exposure (Crammond and Halliwell, 1995).

Crammond and Halliwell concluded that in the latter two examples the deterioration was caused by thaumasite formation with limestone from the aggregate participating in the reaction. To establish the extent to which thaumasite formation can lead to deterioration, two field studies are being planned by BRE: (i) a field exposure trial under Class 2 exposure (XA1B, XA2A), and (ii) a survey of foundations. Laboratory test data reported by Crammond and Halliwell (1996) on limestone-containing concretes stored for two years at 5°C under XA3B (Class 4B) exposure conditions, indicated good sulfate resistance as measured by wear rating (see sub-Section 5.4.2) for all but one of the concretes tested with free water-cement ratios below 0.50. However, visual evidence indicated significant surface erosion, suggesting that, in this case, wear rating may not be the best damage indicator. Under Class 5 exposure conditions, low sulfate resistance (as measured by wear rating) was observed for some concretes with free water-cement ratios below 0.50.



**Figure 5.1:** Deterioration of a ground floor slab placed on sulfate-containing fill (Photo by BRE).

### 5.3.2 Acid attack

In the UK, acid attack is not a common cause of concrete deterioration. Examples of acid attack on concrete which have been observed are:

- Erosion of the surface of concrete tunnels by the soft water flowing from a dam catchment area in Yorkshire (Halstead, 1951).
- Erosion of concrete surfaces exposed to moorland waters and with specialised uses of concrete in agriculture or chemical works (Harrison, 1987).

In a survey of the behaviour of concrete in acid soils and groundwaters, Eglinton (1976) found no evidence of structural weakening of mass concrete exposed to the more common types of naturally occurring acid waters.

## 5.4 Sulfate attack

### 5.4.1 General

Although poor performance of concrete in the field has in the past sometimes been attributed to sulfate attack, judgements are difficult as the cement used, the concrete mix proportions and sometimes the ground conditions are not documented. As a consequence, in this Chapter recommendations on cements and the minimum qualities of concrete necessary to resist sulfate in groundwaters are based on laboratory observations.

Due to the short-term nature of laboratory exposure tests, judgements are sometimes made from immersion tests in solutions of higher sulfate concentrations than those likely to be experienced in practice. Generally the tests are carried out at about 20°C. This approach probably ranks concretes and cements correctly but, for concretes exposed to much lower sulfate concentrations, the differences may not be large enough to be of any practical significance. The results are taken to be applicable to concretes cast directly against sulfate soil<sup>†</sup>. If the concretes are allowed to carbonate prior to immersion, then the results may be taken to be applicable to precast concrete.

### 5.4.2 Criteria for Assessment

Deterioration of concrete and mortar due to sulfate attack has been assessed in a number of ways:

- Visual deterioration (Kalousek et al, 1972; Stark, 1989; Al-Amoudi et al, 1992).
- Wear rating (Harrison and Teychenne, 1981; Osborne, 1991; Matthews, 1979; 1991, 1994, 1995a; Osborne, 1994; Crammond, 1996).
- Compressive strength (Harrison and Teychenne, 1981; Osborne, 1991; Matthews, 1979, 1991, 1994, 1995a; Crammond, 1996; Al-Amoudi et al, 1992; Mehta, 1986a; Osborne, 1994).
- Tensile strength (Locher, 1966).
- Expansion (Kollek and Lumley, 1990; Tikalsky et al, 1992; Kalousek et al, 1992; Mather, 1982; Wong and Poole, 1987; Hartmann and Mangotich, 1987).
- Mass change (Tikalsky et al, 1992; Kalousek et al, 1972).

The most comprehensive series of investigations on the sulfate resistance of concretes has been carried out by BRE (Harrison and Teychenne, 1981; Osborne, 1991; Matthews, 1979, 1991, 1994, 1995a; Osborne, 1994; Crammond, 1996). As a consequence, the guidance given in this report on sulfate attack is based largely on BRE's work which includes both laboratory and field

<sup>†</sup> *This is not strictly correct as, prior to immersion in a sulfate solution, the mortars and concretes are often immersed in water for 14 or 28 days. BRE observations have shown that for concretes stored at 20 °C, this increases the degree of sulfate attack on Portland cement concretes. This is to be expected as some of the pore solution sulfate ions will leach out from the surface layers of the concrete into the water, causing the surface layers to become more susceptible to sulfate attack when the concretes are subsequently immersed in a sulfate solution. This effect will depend on specimen size, becoming more marked as the specimen's surface area-volume ratio increases.*



studies. The recommendations in BS 5328: Part 1: 1997 are based on the BRE findings. In BRE's work, concretes were exposed to the following three solutions:

- 1) Magnesium sulfate, 4.2g/l SO<sub>4</sub>. (Note: Class XA3 ranges from 3.0 to 6.0 g/l SO<sub>4</sub>.)
- 2) Sodium sulfate, 18g/l SO<sub>4</sub>. (Note: Class XA3A ranges from 3.0 to 6.0 g/l SO<sub>4</sub> with < 1.0 g/l Mg.)
- 3) Magnesium sulfate, 18g/l SO<sub>4</sub>. (Note: Class XA3B ranges from 3.0 to 6.0 g/l SO<sub>4</sub> with > 1.0 g/l Mg.)

Solutions 2) and 3) give SO<sub>4</sub> concentrations three times as high as the maximum for Class XA3 exposure.

Much of the test work on the sulfate resistance of concrete has been restricted to Class 42.5 CEM I and CEM I/SR cements and Class 32.5 CEM I/slag, CEM I/fly ash and CEM II/A-L, B-L (Portland limestone) cements.

The criterion of assessment used by Harrison (1992) and Matthews (1991,1995a) is that the sulfate resistance of concretes subject to exposure 1) is good when the wear rating at three or five years is less than 15 or 21 mm respectively, where the wear rating is given by

$$\Sigma \frac{\text{(wear on 8 corners of each of 3 x 100 mm cubes)}}{6}$$

In this Chapter a further criterion is used, namely that the sulfate resistance of concretes subject to Class XA3A and XA3B exposure is assumed to be high when the wear rating after three or five years' exposure to 18 g/l SO<sub>4</sub> as sodium sulfate and 18 g/l SO<sub>4</sub> as magnesium sulfate, is less than 40 and 60 mm respectively.

### 5.4.3 Portland cements: CEM I and CEM I/SR

It has been clearly established that, for a given water-cement ratio, the sulfate resistance of a CEM I and CEM I/SR concrete is broadly related to the C<sub>3</sub>A content of the cement employed (Lawrence, 1988 and 1990).

The five-year results obtained in BRE's work on 100 mm cubes are given in Table 5.7 and are plotted in Figures 5.2 to 5.4 for the three main exposure conditions employed. The aggregate used was Thames Valley sand and gravel. Additional data obtained at three years by Harrison and Teychenne (1981) are also given in Table 5.7. Examination of Figures 5.2 to 5.4 and the data in Table 5.7 shows that CEM I cements with C<sub>3</sub>A contents below about 9 to 10% by mass give markedly higher sulfate resistance than CEM I cements with C<sub>3</sub>A contents above 9 to 10% by mass. Other publications support this conclusion (Al-Amoudi et al, 1992; Mehta, 1986; Wong and Poole, 1987; Hartmann and Mangotich, 1987). In a separate field study of the sulfate resistance of concrete subject to groundwater exposure, where the concentration of SO<sub>4</sub> was close to the upper limit for Class XA 2B, high 15-year resistance was exhibited by 0.6 w-c ratio concretes prepared using a CEM I cement with a C<sub>3</sub>A content of 9.4% by mass (Harrison and Teychenne, 1989).

**Table 5.7:** Five-year and three-year wear rating of 100 mm cubes prepared using CEM I and CEM I/SR cements (BRE).

CEM I or CEM I/SR C <sub>3</sub> A content (kg/m <sup>3</sup> )	Cement content (kg/m <sup>3</sup> )	Free water-cement ratio	5-year wear rating (mm)		
			Magnesium sulfate (4.2 g/l SO <sub>4</sub> )	Sodium sulfate (18 g/l SO <sub>4</sub> )	Magnesium sulfate (18 g/l SO <sub>4</sub> )
14.1	400	0.46	152	254	185
14.1	380	0.45	-	>200	216
14.1	360	0.49	137	208	168
14.1	300	0.61	283	283	283
14.1	270	0.67	283	283	283
13.1	300	0.58	282	282	282
10.3	300	0.57	107	282	260
8.8	380	0.45	-	35	74
8.6	300	0.57	5	7	29
8.5	300	0.56	28	12	31
7.1	300	0.58	3	5	49
7.0	400	0.44	8	15	28
7.0	360	0.48	21	19	46
0.8	500	0.38	10	19	32
0.8	450	0.40	10	16	32
0.8	400	0.44	9	13	31
0.8	360	0.48	13	12	46
0.8	300	0.57	11	13	30
0.8	270	0.63	13	11	22
0.8	200	0.97	13	12	40
0.8	180	1.08	29	19	45
0.3	380	0.45	-	19	6
			3 year wear rating (mm)		
9.4	375	0.43	42	51	39
9.4	375	0.47	45	57	51
9.4	375	0.50	39	66	63
9.4	335	0.47	30	51	39
9.4	335	0.52	39	69	51
9.4	335	0.51	48	96	48

Table 5.7 (contd.....)

9.4	335	0.54	63	141	69
9.4	285	0.57	48	114	63
9.4	285	0.62	48	141	57
9.4	225	0.73	21	54	51
0.8	375	0.43	3	3	3
0.8	375	0.47	9	6	9
0.8	335	0.47	3	6	9
0.8	335	0.51	3	3	9
0.8	335	0.52	6	6	6
0.8	335	0.55	6	9	9
0.8	285	0.54	3	3	3
0.8	285	0.61	9	9	6
0.8	225	0.63	3	3	27

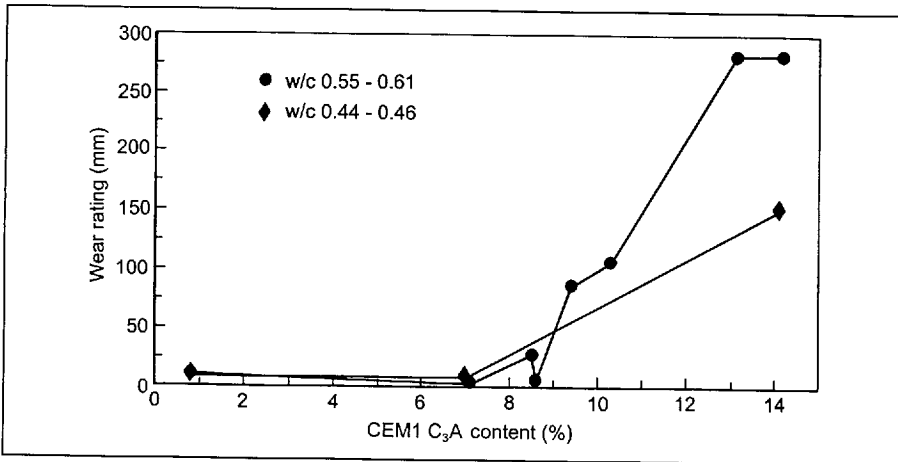


Figure 5.2: Relationship between wear rating and CEM I C<sub>3</sub>A content. Mg<sub>2</sub>SO<sub>4</sub> at 4.2 g/l SO<sub>4</sub>.

An examination of the data in Table 5.7 indicates that CEM I cements with C<sub>3</sub>A contents below 10% by mass may be suitable for use in concretes subject to EN 206 Sub-Class XA2B exposure conditions ( $c \geq 360 \text{ kg/m}^3$ , w-c ratio  $\leq 0.45$ ) and below, or BS 5328 Class 3 exposure. CEM I cements with C<sub>3</sub>A contents above 10% by mass are suitable for use in concretes subject to up to pr EN 206 sub-Class XA2A exposure conditions or BS 5328 Class 2 exposure (Matthews, 1979; 1991; 1994; 1995a). In the case of CEM I/SR cements, the data in Table 5.7 indicates that they are suitable for all ground conditions up to and including pr EN 206 sub-Class XA3B exposure or Class 4B exposure. (Osborne, 1989, 1991; Matthews, 1979, 1991, 1995a; Mather, 1992; Wong and Poole, 1987). In the case of both pr EN 206 sub-Classes XA3A and XA3B exposure conditions (BS 5328 Classes 4A and 4B), the data show that concretes made with CEM I/SR

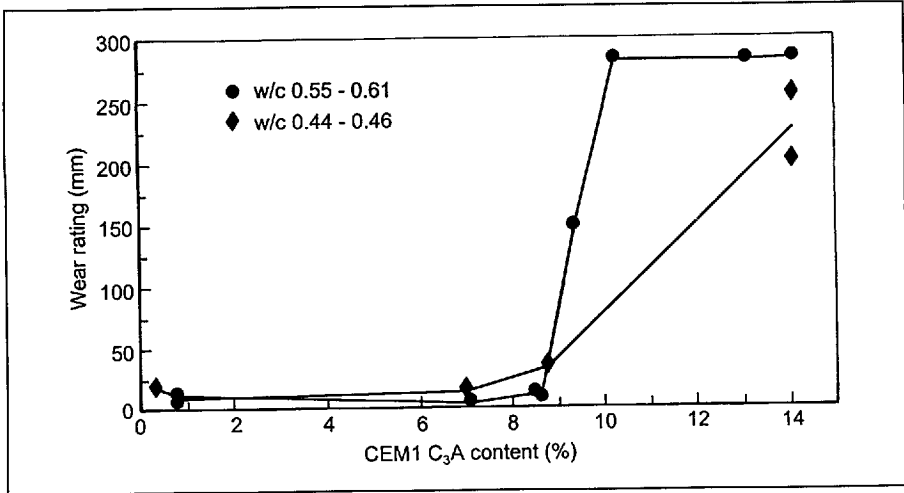


Figure 5.3: Relationship between wear rating and CEM I C<sub>3</sub>A content. Na<sub>2</sub>SO<sub>4</sub> at 18 g/l SO<sub>4</sub>.

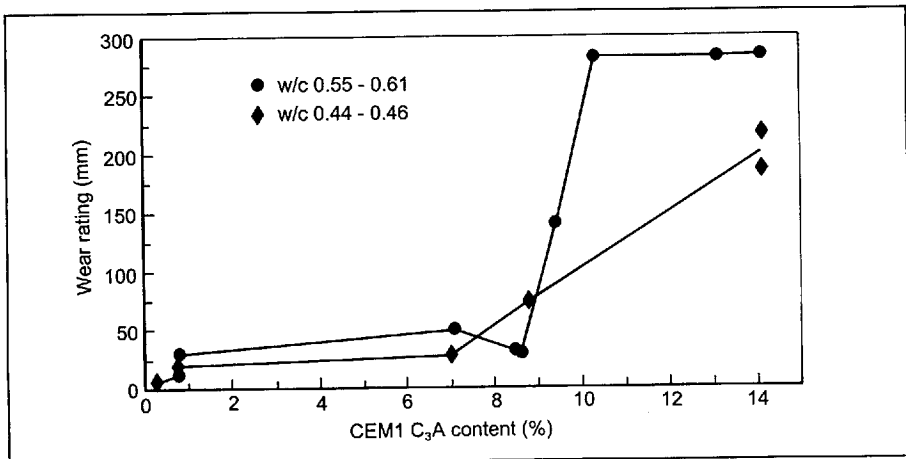
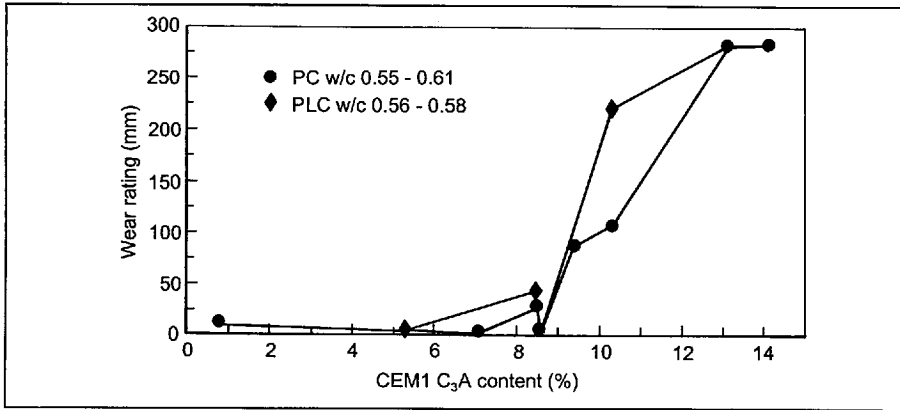
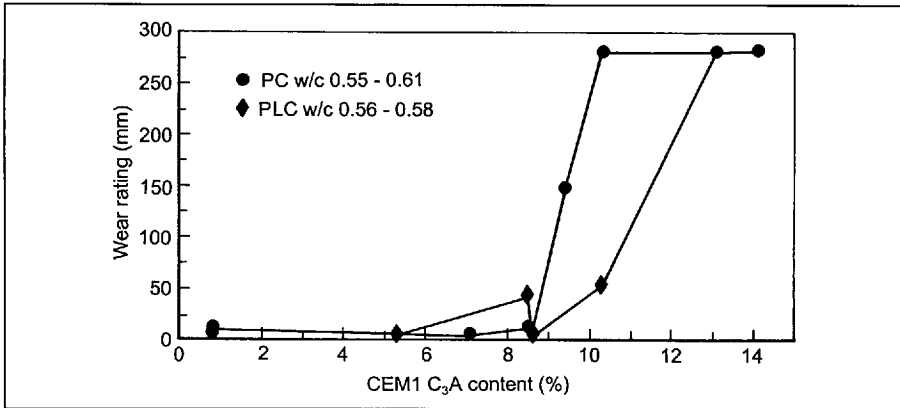


Figure 5.4: Relationship between wear rating and CEM I C<sub>3</sub>A content. Mg<sub>2</sub>SO<sub>4</sub> at 18 g/l SO<sub>4</sub>.

content below 300 kg/m<sup>3</sup> give good performance (see Figures 5.5 and 5.6 Harrison and Teychenne, 1981; Osborne, 1991; Matthews, 1979; 1991; 1995a). Fiskaa (1973) observed high 20-year sulfate resistance for CEM I/SR concrete specimens of w-c ratio 0.50 placed in Oslo's alum shale region where the mobile groundwater contains up to 4.8 g SO<sub>4</sub>/l and the pH varies from neutral to 2.5 (Sellevoid and Nilsen, 1987).



**Figure 5.5:** Relationship between wear rating and CEM I C<sub>3</sub>A content. CEM I and CEM II/B-L concretes. Mg<sub>2</sub>SO<sub>4</sub> at 4.2 g/l SO<sub>4</sub>.



**Figure 5.6:** Relationship between wear rating and CEM I C<sub>3</sub>A content. CEM I and CEM II/B-L concretes. Na<sub>2</sub>SO<sub>4</sub> at 18 g/l SO<sub>4</sub>.

#### 5.4.3 Minimum specification: CEM I and CEM I/SR concretes

Minimum qualities for CEM I and CEM I/SR concretes exposed to sulfate-bearing groundwaters are given in Table 5.8. This minimum specification is based on the work reported in the previous sub-Section. In any future revision of the UK national durability provisions in the complementary BS to pr EN 206, consideration could be given to lowering the minimum cement requirement and raising the maximum w-c ratio for CEM I/SR concretes.

**Table 5.8:** Minimum qualities of CEM I and CEM I/SR concretes exposed to sulfate-bearing groundwater. pH > 5.5.

Chemical characteristics and minimum concrete qualities	XA1		XA2		A3	
	XA1A	XA1B	XA2A	XA2B	XA3A	XA3B
SO <sub>4</sub> in groundwater (mg/l)	200-400	400-600	600-1440	1440-3000	3000 - 6000	
SO <sub>4</sub> by 2:1 water soil extract (mg/l)	600-1200	1200-1800	1800-2300	2300-3700	3700 - 6700	
Mg in groundwater (mg/l)	NA	NA	N/A	N/A	<1000	>1000
CEM I. Maximum w-c ratio	-	0.50	0.50	-	-	-
CEM I. Minimum cement content (kg/m <sup>3</sup> )	-	340	340	-	-	-
CEM I/SR. Maximum w-c ratio	-	0.55	0.55	0.50	0.45	0.45
CEM I/SR. Minimum cement content (kg/m <sup>3</sup> )	-	280	280	320	360	360

#### 5.4.4 Portland limestone cements: CEM II/A-L and B-L

Little data has been published dealing with the sulfate resistance of CEM II/A-L and B-L concretes. Results obtained in a BRE/BCA/Cement Industry Working Party Programme on CEM II/B-L concretes are given in Table 5.9 and the results are plotted in Figures 5.5 and 5.6 for concretes exposed to the lower magnesium sulfate solution (4.2 g/l SO<sub>4</sub>) and the sodium sulfate solution (18 g/l SO<sub>4</sub>) respectively. The aggregate employed was Thames Valley sand and gravel. The concretes had free water-cement ratios of between 0.56 and 0.63, a cement content of 300 kg/m<sup>3</sup> and were subject to the same three solutions, at 20°C, as detailed earlier. Because all the w-c ratios were higher than those currently specified for CEM I concretes subject to sulfate exposure (BS 5328: Part 1: 1997) judgements of the performance of CEM II/B-L concretes are made relative to CEM I concretes of the same w-c ratio.

**Table 5.9:** Five-year wear rating of 100 mm concrete cubes containing 25% of a selected limestone powder.

CEM I C <sub>3</sub> A content (%)	Cement content (kg/m <sup>3</sup> )	Free water-cement ratio (estimated)	Wear rating (mm)		
			Magnesium sulfate (4.2 g/l SO <sub>4</sub> )	Sodium sulfate (18 g/l SO <sub>4</sub> )	Magnesium sulfate (18 g/l SO <sub>4</sub> )
13.1	300	0.59	282	282	282
10.3	300	0.59	220	54	270
8.6	300	0.63	5	4	59
8.5	300	0.58	176	43	190
5.3	300	0.59	8	5	51

From Figures 5.5 and 5.6 and examination of Table 5.9 it can be seen that the sulfate resistance of CEM II/B-L concretes is dependent upon the C<sub>3</sub>A content of its CEM I component and that at the same w-c ratio as the control CEM I concrete the sulfate resistance is lower. As a consequence it is concluded that CEM II/B-L cements may not be suitable for use in concretes

subject to pr EN 206 sub-Class XA1B or XA2A exposure or BS 5328 Class 2 exposure. Due to lack of performance data on CEM II/A-L concretes subject to sulfate exposure, judgements regarding their suitability for sub-class XA1B to XA2A exposure cannot, as yet, be made.

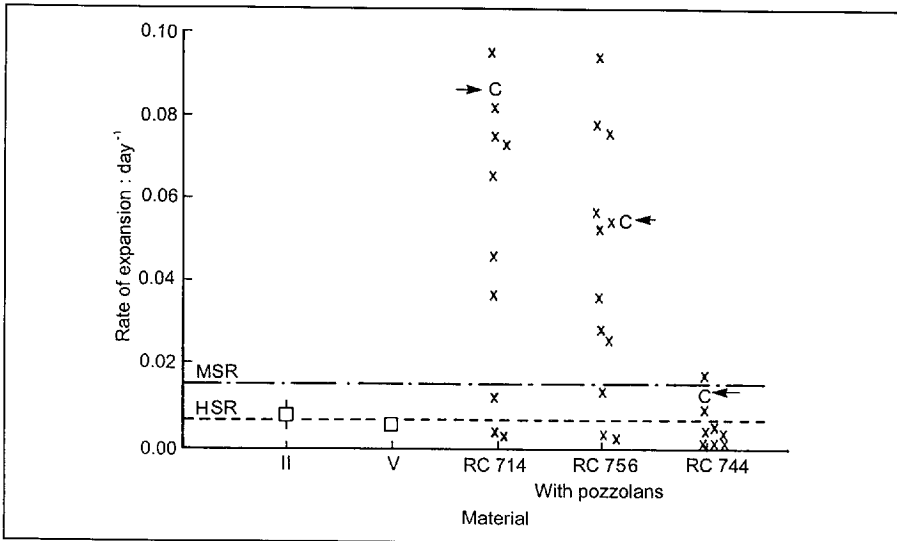
#### 5.4.5 Minimum specification: CEM II/A-L and B-L concretes

Further research is required before recommendations can be made for the minimum concrete qualities for CEM II/A-L and CEM II/B-L concretes exposed to sulfate attack. The field trials and survey of foundations planned by BRE referred to earlier should assist in this respect.

#### 5.4.6 Cements containing fly ash: CEM II/A-V, B-V, IV/A and IV/B

Some of the literature dealing with the effect of partial replacement of a CEM I cement by siliceous fly ash upon sulfate resistance is summarized in Table 5.10. At high sulfate concentrations the performance of CEM I/fly ash cements is dependent upon the  $C_3A$  content of the CEM I cement (Figure 5.7, after Wong and Poole, 1988). Different behaviour has been observed for concretes and mortars containing fly ashes from different sources, but the compositional changes in the fly ashes resulting in these differences have not been established (Matthews, 1991, 1995a; Mehta, 1986; Hartmann and Mangotich, 1987; Dunstan, 1987).

The five-year results obtained in BRE's work are given in Table 5.11 and are plotted against CEM I  $C_3A$  content in Figures 5.8 to 5.10 for the three main exposure conditions employed (Harrison and Teychenne, 1981; Matthews, 1979, 1991, 1994 and 1995a). The aggregate employed was Thames Valley sand and gravel. All concretes containing 30 and 40% fly ash, and the 20% fly ash concretes of cement content  $400 \text{ kg/m}^3$ , exhibited good sulfate resistance after five years' exposure to a magnesium sulfate solution with  $4.2 \text{ g/l SO}_4$  and a sodium sulfate solution with  $18 \text{ g/l SO}_4$  but not to a magnesium sulfate solution with  $18 \text{ g/l SO}_4$ .



**Figure 5.7:** Rates of expansion in 5%  $\text{Na}_2\text{SO}_4$  solutions of mortars bars made with ASTM Type II and V cements, and blended cements using three ASTM Type 1 cements with 13, 9 and 15%  $C_3A$  (RC 714, RC 744, RC 756). C marks the expansion of the Type I mortars. MSR and HSR are moderate and high sulfate resistance.

**Table 5.10:** Summary of performance of Portland /fly ash cements.

Reference	CEM I-cement C <sub>3</sub> A (%)	Fly ash (%) (no. of ashes)	Pre-curing	Exposure (SO <sub>4</sub> g/l)	Comments
Harrison & Teychenne, 1981	9.4	0, 20 (1)	28d water	MgSO <sub>4</sub> (4.2 & 18); Na <sub>2</sub> SO <sub>4</sub> (18)	20% fly ash results in high sulfate resistance, suitable for sub-Class XA3A. CEM I cement gives high sulfate resistance in 4.2 g/l SO <sub>4</sub> solution, suitable for sub-Class XA2B.
Matthews, 1979	1.2, 11.3	0, 25 (1)	28d water	MgSO <sub>4</sub> (4.2 & 18); Na <sub>2</sub> SO <sub>4</sub> (18)	25% fly ash results in high sulfate resistance, suitable for sub-Class XA3A.
Matthews, 1991, 1995a	0.8	0, 20 (4) 40 (4)	28d water 28d water	MgSO <sub>4</sub> (4.2 & 18); Na <sub>2</sub> SO <sub>4</sub> (18)	All concretes showed good sulfate resistance. All concretes suitable for sub-Class XA3B.
	7.0	0, 20 (4) 0, 40 (4)	28d water 28d water		All concretes showed good sulfate resistance. All concretes suitable for sub-Class XA3A.
	14.1	0, 20 (4) 30 (1) 40 (4)	28d water 28d water 28d water		All CEM I concretes performed poorly. Not suitable for sub-Class XA2B. Some of the 20% fly ash concretes gave inferior performance to zero fly ash concretes prepared using 7.0% C <sub>3</sub> A cement; 30 and 40% fly ash, but not fly ash 4 (Part 2 ash) and perhaps 3, gave similar or better performance to 7.0% C <sub>3</sub> A cement. Performance was not related to water-cement ratio. 30 and 40% fly ash suitable for sub-Class XA3A.
Tikalsky et al, 1992	6.0	0, 28 (6)	Saturated lime bath until 24 MPa	Na <sub>2</sub> SO <sub>4</sub> (67.5)	After 540 days, 3 fly ashes gave similar performance to the CEM I cement and 3 gave better. Not all fly ashes suitable for sub-Class XA3A.
Kalousek et al, 1972	1.8 - 4.1	0, 28 (?)	14d moist, 14d dried at 50% RH	Na <sub>2</sub> SO <sub>4</sub> (14.2)	Fly ash improved sulfate resistance. Concretes dried - results applicable to precast members.
	4.6 - 7.3	0, 28 (?)			
Dikeou, 1967	8 cements - Types 1, 11 and V	Up to 28 (12)	14d moist 14d dried at 50% RH	Cycled Na <sub>2</sub> SO <sub>4</sub> (14.2) for 16 hr, dried 8 hr	All fly ashes improved resistance. Concretes dried - results applicable to precast members.
Stark, 1989	8 4,13	0, 20 (1) 40 (1) 0	28d moist, dried at 50% RH	At 1 yr Na <sub>2</sub> SO <sub>4</sub> (56) wetting and drying	For 8.2% C <sub>3</sub> A CEM I cement, fly ash has detrimental effect on resistance. Concretes dried - results applicable to precast members.
Al-Amoudi et al, 1992	8.5	0, 20 (1)	14d water	MgSO <sub>4</sub> plus Na <sub>2</sub> SO <sub>4</sub> (21) + 15.7% Cl <sup>-</sup>	The sulfate resistance of plain and blended cements was similar. Fly ash suitable for sub-Class XA2B.



Table 5.10 (contd.....)

Mehta, 1986a	11	0, 25 (5) 40 (5)	14d moist cured at 40°C	Na <sub>2</sub> SO <sub>4</sub> (27) pH7	Two of the five fly ashes did not improve sulfate resistance. Not all fly ashes suitable for sub-Class XA3A or sub-Class XA2B.
Mather, 1982, Wong & Poole, 1987	9,13,15 15 Type II cements 8 Type II cements 2 Type V cements	Up to 28 (2) 0 0 0	Saturated lime bath until required strength reached	Na <sub>2</sub> SO <sub>4</sub> (34)	With C <sub>3</sub> A 13 and 15% cements, one fly ash resulted in equal performance to ASTM Type II cement, the other did not. With C <sub>3</sub> A 9%, a cement of moderate sulfate resistance, there was general improvement in sulfate resistance to Type II. Both Type V cements had high sulfate resistance. 9% C <sub>3</sub> A cement suitable for sub-Class XA2B. One of the 2 fly ashes, not suitable for sub-Class XA3A.
Matthews, 1994	8.5 8.6 5.3	0, 28 0, 30 0	28d water	MgSO <sub>4</sub> (4.2 & 18) Na <sub>2</sub> SO <sub>4</sub> (18)	All concretes showed good sulfate resistance. All cements suitable for sub-Class XA3B.

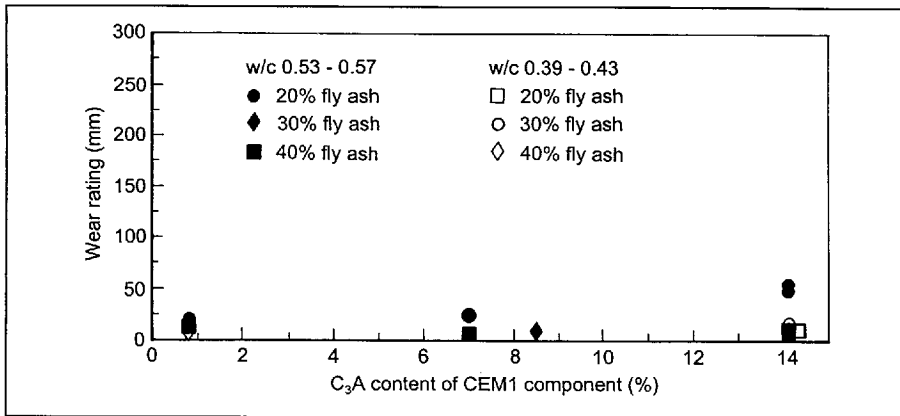


Figure 5.8: Relationship between wear rating and CEM I C<sub>3</sub>A content. CEM I/fly ash concretes. Mg<sub>2</sub>SO<sub>4</sub> at 4.2 g/l SO<sub>4</sub>.

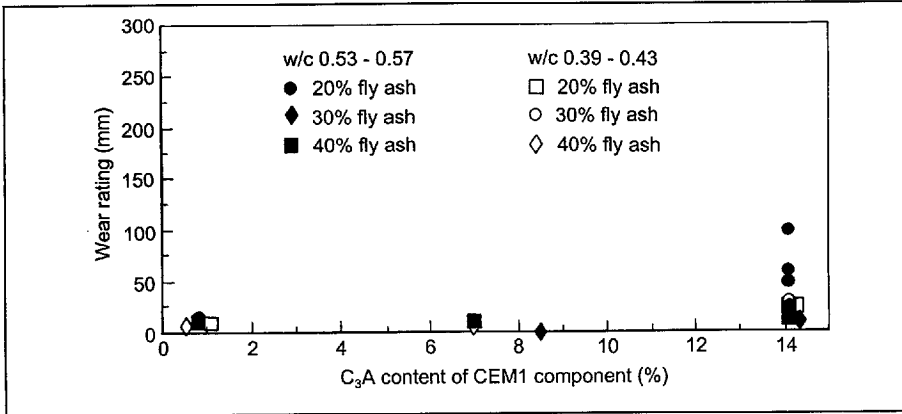


Figure 5.9: Relationship between wear rating and CEM I C<sub>3</sub>A content. CEM I/fly ash concretes. Na<sub>2</sub>SO<sub>4</sub> at 18 g/l SO<sub>4</sub>.

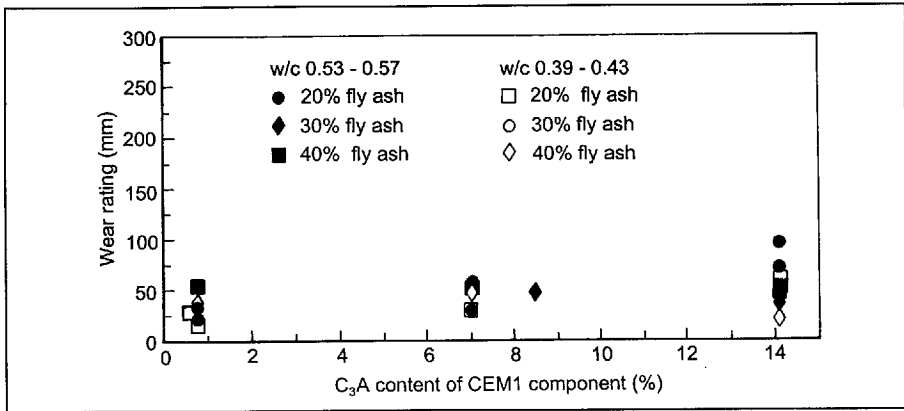


Figure 5.10: Relationship between wear rating and CEM I C<sub>3</sub>A content. CEM I/fly ash concretes. Mg<sub>2</sub>SO<sub>4</sub> at 18 g/l SO<sub>4</sub>.

**Table 5.11:** Five-year wear rating of 100 mm concrete cubes containing 20, 30 or 40% fly ash. Mixer combinations (BRE).

Proportion of fly ash (%)	CEM I or CEM I/SR C <sub>3</sub> A content (%)	Cement content (kg/m <sup>3</sup> )	Free water-cement ratio	5 year-wear rating (mm)		
				Magnesium sulfate (4.2 g/l SO <sub>4</sub> )	Sodium sulfate (18 g/l SO <sub>4</sub> )	Magnesium sulfate (18 g/l SO <sub>4</sub> )
20	14.1	300	0.57	54	48	95
20	14.1	300	0.57	12	59	51
20	14.1	300	0.57	49	98	71
20	14.1	400	0.43	11	28	54
20	14.1	400	0.43	13	16	64
30	14.1	300	0.54	14	9	35
30	14.1	350	0.46	14	34	56
30	14.1	400	0.42	20	31	41
40	14.1	300	0.52	17	12	63
40	14.1	300	0.55	8	12	45
40	14.1	300	0.53	11	25	54
40	14.1	350	0.46	21	18	62
40	14.1	400	0.41	8	11	20
40	14.1	400	0.41	12	13	68
30	8.6	300	0.50	4	5	39
28	8.5	300	0.53	10	3	47
20	7.0	300	0.55	25	10	57
20	7.0	300	0.57	5	6	28
20	7.0	400	0.42	3	7	51
20	7.0	400	0.42	3	9	29
40	7.0	300	0.51	24	8	89
40	7.0	300	0.54	6	11	51
40	7.0	400	0.39	2	4	48
40	7.0	400	0.40	2	5	28
20	0.8	300	0.56	12	14	35
20	0.8	300	0.56	11	15	34
20	0.8	300	0.55	18	13	24
20	0.8	400	0.42	11	14	17
20	0.8	400	0.42	10	12	30
40	0.8	300	0.51	12	12	63
40	0.8	300	0.54	12	12	53
40	0.8	300	0.52	28	13	51
40	0.8	400	0.40	12	12	39
40	0.8	400	0.40	7	10	26

**5.4.7 Minimum specification for cements containing fly ash: CEM II/A-V, B-V, IV/A and IV/B**

Minimum qualities for CEM II/A-V, B-V, IV/A and IV/B concretes exposed to sulfate bearing groundwaters, based on the work reported in the previous sub-section, are given in Table 5.12.

**Table 5.12:** Minimum qualities of CEM II/A-V, B-V, IV/A and IV/B concretes exposed to sulfate bearing groundwater. pH > 5.5.

Chemical characteristics and minimum concrete qualities	XA1		XA2		XA3	
	XA1A	XA1B	XA2A	XA2B	XA3A	XA3B
SO <sub>4</sub> in groundwater (mg/l)	200-400	400-600	600-1440	1440-3000	3000 - 6000	
SO <sub>4</sub> by 2:1 water soil extract (mg/l)	600-1200	200-1800	1800-2300	2300-3700	3700 - 6700	
Mg in groundwater (mg/l)	N/A		N/A	N/A	≤ 1000	>1000
Maximum w-c ratio	-	0.55 <sup>+</sup> 0.50 <sup>§</sup>	0.55 <sup>+</sup> 0.50 <sup>§</sup>	0.50 <sup>+</sup>	0.45 <sup>+</sup>	-
Minimum cement content (kg/m <sup>3</sup> )	-	300 <sup>+</sup> 340 <sup>§</sup>	300 <sup>+</sup> 340 <sup>§</sup>	340 <sup>+</sup>	380 <sup>+</sup>	-

<sup>+</sup> CEM II/B-V and IV/A cements containing not less than 26% fly ash by mass of the nucleus or CEM IV/B containing not more than 42% fly ash by mass of the nucleus or CEM I with fly ash conforming to BS 3892; Part 1 where there is not more than 40% fly ash and not less than 25% fly ash by mass of the combination.

<sup>§</sup> CEM II/A-V, II/B-V and IV/A cements containing less than 26% fly ash by mass of the nucleus or combinations of CEM I with fly ash conforming to BS 3892: Part 1 where there is less than 25% fly ash by mass of the combination.

**5.4.8 Cements containing slag: CEM II/A-S, B-S and III/A, B, C**

Cements containing slag can be produced in a factory or can be used as concrete mixer blends. These production methods give cements which may have different properties. In the case of factory-produced cement, the SO<sub>3</sub> level is often enhanced and optimized. In the case of a concrete mixer blend the SO<sub>3</sub> content is controlled by the SO<sub>3</sub> level of the CEM I component. This difference is often overlooked when judgements are made regarding field performance. European field experience is largely confined to factory-produced slag cements which have SO<sub>3</sub> levels considerably higher than those of mixer combinations. This increased SO<sub>3</sub> level has been shown to increase the sulfate resistance of slag cements (Kondo, 1960; Kollek and Lumley, 1990). In some European countries, high percentage slag cements, namely CEM III/B, made in the cement factory, are regarded as equivalent to CEM I/SR cements.

The influence upon sulfate resistance of the partial replacement of a CEM I cement by slag is complex. The performance depends upon the proportion of CEM I cement replacement by slag (Osborne, 1991 and 1994; Kollek and Lumley, 1990), the C<sub>3</sub>A content of the CEM I cement (Harrison and Teychenne, 1981; Osborne, 1991 and 1994; Kollek and Lumley, 1990); the composition and mineralogy of the slag (Osborne 1991; Locher, 1966; Kollek and Lumley, 1990), the SO<sub>3</sub> level of the slag cement (Kondo, 1960; Kollek and Lumley, 1990) and the sulfate concentration in the groundwater to which the concrete is exposed. Magnesium sulfate in groundwater is more damaging than sodium sulfate. At low and medium replacement levels of CEM I cement by slag (20 to 50%) and without enhancement of the SO<sub>3</sub> level relative to the CEM I component, the sulfate resistance can be reduced (Harrison and Teychenne, 1981;

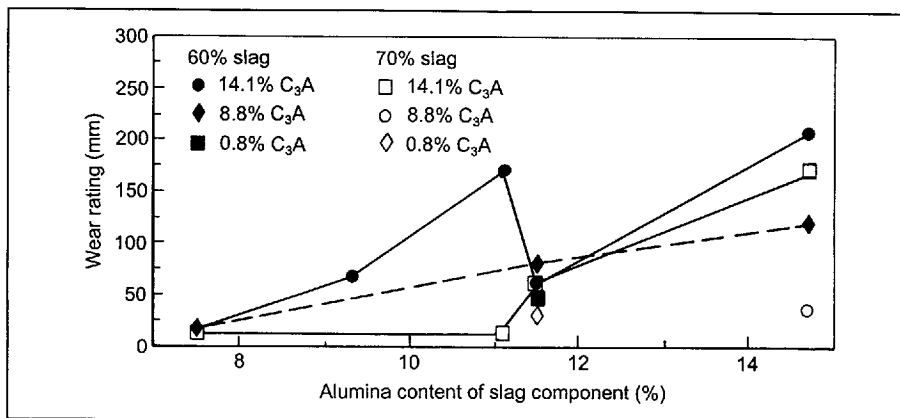
Osborne, 1991; Kollek and Lumley, 1990) or increased (Osborne, 1991; Kollek and Lumley, 1990). This is also the case at high replacement levels (60 to 70%), the effects being more marked for specimens immersed in a magnesium sulfate solution than in a sodium sulfate solution (Osborne, 1991 and 1994; Kollek and Lumley, 1990; Al-Amoudi et al, 1992). In both sodium sulfate and magnesium sulfate solutions, it has been found that 60 to 70% replacement of a CEM I cement by slag, in mixer combinations in concretes and mortars, often does not give equal performance to a CEM I/SR cement, nor in some instances to a CEM I cement with a  $C_3A$  content below 10% by mass (Osborne, 1991 and 1994; Kollek and Lumley, 1990).

It is generally considered that the alumina content of a slag is a major parameter which influences the sulfate resistance of a slag concrete (Osborne, 1991; Locher, 1966; Kollek and Lumley, 1990). Figures 5.11 and 5.12 show the broad relationship between wear rating and slag alumina content for concretes immersed in a sodium sulfate and a magnesium sulfate solution respectively (Osborne, 1991 and 1994).

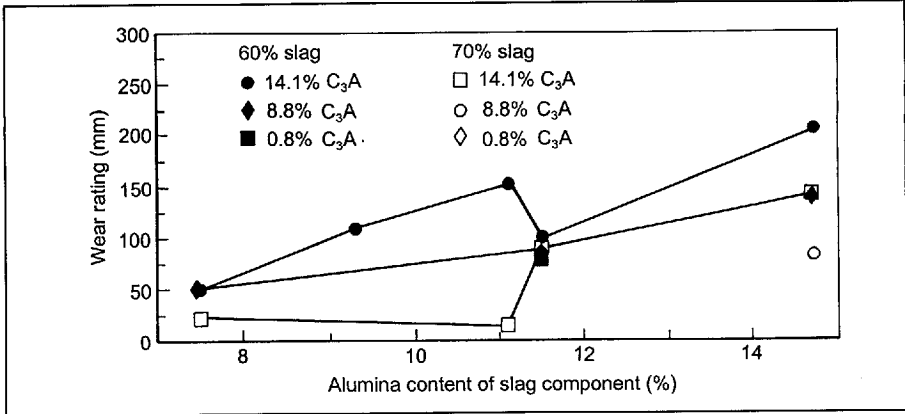
The five-year results obtained in BRE's work are given in Table 5.13 for the two exposure conditions employed (sodium sulfate at 18 g/l  $SO_4$ ; magnesium sulfate at 18 g/l  $SO_4$ ; Osborne, 1991 and 1994). The concrete had a cement content of 380 kg/m<sup>3</sup> and a free water-cement ratio of 0.45. The aggregate employed was Thames Valley sand and gravel.

Examination of the data in Table 5.13 shows that sulfate resistance generally increases with decreasing alumina content of the slag, with decreasing CEM I  $C_3A$  and with increasing slag content from 60 to 70%. In solutions of similar high  $SO_4$  concentration the sulfate resistance is lower in a magnesium sulfate solution than in a sodium sulfate solution.

From the results in Table 5.13 it is concluded that 70% slag results in satisfactory resistance in the 18 g/l  $SO_4$  sodium sulfate solution, except for combinations of high alumina content slags and high  $C_3A$  content CEM I cements. In this case, the maximum alumina content for satisfactory resistance is between 11.5 and 14.7% by mass. It is recommended that further work is carried out on the effect of the alumina content of slag upon the sulfate resistance of CEM III/B concretes.



**Figure 5.11:** Relationship between wear rating and alumina content of slag. w-c ratio 0.45.  $Na_2SO_4$  at 18 g/l  $SO_4$ .



**Figure 5.12:** Relationship between wear rating and alumina content of slag. w/c 0.45. Mg<sub>2</sub>SO<sub>4</sub> at 18 g/l SO<sub>4</sub>.

**Table 5.13:** Five-year wear rating of 100 mm concrete cubes containing 60 to 80% slag. Mixer combinations. No enhancement of SO<sub>3</sub> (BRE).

Slag alumina content (%)	Proportion of slag (%)	CEM I or CEM I/SR C <sub>3</sub> A content (%)	Wear rating (mm)	
			In sodium sulfate (18 g/l SO <sub>4</sub> )	In magnesium sulfate (18 g/l SO <sub>4</sub> )
14.7	60	14.1	208	204
14.7	70	14.1	171	141
14.7	80	14.1	102	35
14.7	60	8.8	119	138
14.7	70	8.8	37	81
14.7	80	8.8	12	37
11.5	60	14.1	62	99
11.5	70	14.1	64	89
11.5	60	8.8	81	87
11.5	60	0.3	49	78
11.5	70	0.3	30	80
11.1	60	14.1	170	151
11.1	70	14.1	12	9
9.3	60	14.1	67	109
9.3	70	14.1	51	84
7.5	60	14.1	15	49
7.5	70	14.1	12	22
7.5	80	14.1	5	16
7.5	60	8.8	15	17

#### 5.4.9 Minimum specification for cements containing slag: CEM II/A-S, B-S and III/A, B, C

Minimum qualities for CEM II/A-S, B-S and CEM III/A, B, C concretes exposed to sulfate-bearing groundwaters, based on the work reported in the previous sub-Section, are given in Table 5.14.

**Table 5.14:** Minimum qualities of CEM II/A-S, B-S and CEM III/A, B, C concretes exposed to sulfate-bearing groundwater. pH > 5.5.

Chemical characteristics and minimum concrete qualities	XA1		XA2		XA3	
	XA1A	XA1B	XA2A	XA2B	XA3A	XA3B
SO <sub>4</sub> in groundwater (mg/l)	200-400	400-600	600-1440	1440-3000	3000-6000	
SO <sub>4</sub> by 2:1 water-soil extract (mg/l)	600 - 1200	1200 - 1800	1800-2300	2400-3700	3700-6700	
Mg in groundwater (mg/l)	N/A	N/A	N/A	N/A	≤ 1000	>1000
Maximum w-c ratio	-	0.50 0.55	0.50 0.55	0.50	0.45 -	
Minimum cement content (kg/m <sup>3</sup> )	-	340 300	340 300	340	380 -	
Permitted CEM cements containing slag	All	II/A-S II/B-S III/A II/B	III/B <sup>+</sup> III/C <sup>+</sup>	II/A-S II/B-S III/A III/B	III/B III/B <sup>+</sup> III/C <sup>+</sup>	III/B <sup>+</sup> III/C <sup>+</sup>

<sup>+</sup> CEM III/B cement where slag ≥ 74% by mass of nucleus and III/C cement where slag ≤ 90% by mass of nucleus. For combinations of CEM I cement and slag conforming to BS 6699 the slag content should be ≥ 70% and ≤ 85% by mass of the combination. If the slag has an alumina content greater than 12 to 14% it should only be used with CEM I with C<sub>3</sub>A ≤ 10% by mass.

#### 5.4.10 Portland silica fume cements: CEM II/A-D

The literature includes a number of investigations of the sulfate resistance of mortars and concretes containing silica fume. BRE's studies included no tests on concretes containing silica fume. Early investigations by others showed that expansions in sodium and magnesium sulfate solutions were reduced by blending up to 20% silica fume with a CEM I cement of intermediate C<sub>3</sub>A level, but were ineffective when a CEM I cement of high C<sub>3</sub>A level was used (Thorvaldson et al, 1929). The incorporation of up to 15% silica fume in pastes and concretes exposed to a sodium sulfate solution has been found by Cohen and Bentur (1988), Carlson and Vennesland (1982) and Bernhardt (1952) to enhance performance but the resistance of paste specimens to a strong magnesium sulfate solution is reduced; the samples losing 20 to 25% of their mass (Cohen and Bentur, 1988). Sellevold and Nilsen (1987) have reported good 30 year field performance for concrete specimens of w/c 0.62 containing 15% silica fume exposed to mobile groundwater in Oslo's alum slate region. This groundwater contains up to 4.8g SO<sub>4</sub>/l and has a pH varying from 7 to 2.5.

The authors are not aware of published data on the sulfate resistance of concretes prepared using CEM II/A-D cements containing between 6 and 10% by mass of silica fume. Consequently, it is recommended that the minimum specification be taken to be the same as for CEM I concretes (see Table 5.8).

### 5.5 Sea water attack

Concrete exposed to a marine environment can deteriorate due to the combined effect of the chemical constituents of sea water on the cement hydration products, crystallization pressure of salts within the concrete if one face is wetted and others are subject to drying conditions, frost action in cold weather (the tidal zone), physical erosion and chloride-induced corrosion of reinforcing steel. In this Section of the report only chemical attack is considered.

In a UK marine environment, concrete generally has a satisfactory history of performance. Only a few concretes subject to a long-term marine exposure have shown major deterioration. The concrete qualities and nominal cover to steel recommended in BS 6349: Part 1 for concrete subject to marine exposure are given in Table 5.15. All cements in common usage in the UK are deemed to be suitable.

**Table 5.15:** Concrete qualities and nominal cover to steel to meet durability requirements (BS 6349:Part 1:1984).

Exposure	Concrete	Minimum cover (mm)	Maximum water-cement ratio	Minimum cement content (kg/m <sup>3</sup> )
Directly exposed to sea water spray or in tidal zone down to 1 m below lowest level	Reinforced	50	0.45	400
	Plain	NA	0.50	350

Sea water contains about 3.5% by mass of soluble salts, the concentration of sodium and chloride ions being highest, typically 11 g/l and 20 g/l respectively. The aggressive ions are magnesium and sulfate with a concentration of approximately 1.4 g/l and 2.7 g/l respectively (Mehta, 1986b). The pH of sea water varies between 7.5 and 8.4 (Mehta, 1986b). Thus on the basis of Table 5.5 the exposure class would be classified as XA2B. However, in a marine environment, expansion of high C<sub>3</sub>A Portland cement concretes does not occur (Mehta, 1986b). The attack which occurs is leaching of calcium hydroxide and ettringite from the surface layers, part of this leaching being due to carbonic acid attack. Below the low tide level, leaching and sulfate attack are considered to be reduced by the rapid formation of magnesium hydroxide (brucite) and magnesium silicate hydrate on the concrete surface. Above the low tide level these reaction products are removed by wave action.

Mehta (1986b) has reviewed the literature dealing with the field performance of concrete in a marine environment. Permeable concretes in sea water can show considerable loss of mass associated with magnesium ion attack, carbonic acid attack and leaching. Except for high water-cement ratio concretes, cement composition has a small influence compared with the permeability of the concrete (Mehta). However, durability data for 100 mm concrete cubes following ten years' exposure to a UK marine environment, reported by Matthews (1995b), do not fully support this conclusion. The CEM I cements employed had C<sub>3</sub>A contents of 0.8, 7.0 and 14.1%. CEM I concretes prepared using the high C<sub>3</sub>A content CEM I cements performed poorly



in fully immersed conditions, irrespective of cement content. CEM I cements of such high  $C_3A$  content are no longer manufactured in the UK. Matthews also concluded:

"For unreinforced pfa concrete specimens exposed in fully immersed conditions there was little evidence of deterioration, specimens with 40% pfa continuing to gain strength up to 10 years. Under these conditions, where the principal form of deterioration is sulphate attack, concretes with 20% or 40% pfa out-performed the Portland cement concretes tested, including those made with SRPC."

"In the tidal zone, pfa concretes suffered greater damage from the effects of freezing and thawing than the Portland cement concretes which exhibited little or no evidence of freeze-thaw damage."

Osborne (1994) has reported durability data for 100 mm concrete cubes following five years' exposure to a UK marine environment (see Table 5.16). Only CEM I cements and combinations of CEM I cements and slag were tested. The concretes had a cement content of  $380 \text{ kg/m}^3$  and a free water-cement ratio of 0.45. Osborne (1994) comments on these results as follows:

"The data showed that in general terms all of the Portland and blastfurnace slag cement concretes had good resistance to sea water attack after 5 years of exposure in tidal and full immersion zones at BRE's marine exposure site at Shoeburyness. However, the SRPC and high  $C_3A$  PC concretes in the full immersion zone had somewhat higher strength losses and some of the slag cement concretes had suffered superficial surface frost damage in the form of 'pop-outs' and spalling, characteristic of freeze-thaw attack."

**Table 5.16:** Durability data for 100 mm concrete cubes following five years' exposure to a marine environment. (Osborne, 1994).

Cement CEM I or CEM I/SR (%)	Slag content (%)	$C_3A$ content (%) of CEM I or CEM I/SR	Strength retained %		Wear rating (mm)	
			Tidal	Full immersion	Tidal	Full immersion
100	0	0.3	80	63	<4	<4
60	40	0.3	66	69	18	26
40	60	0.3	77	69	16	21
30	70	0.3	85	88	15	15
100	0	8.8	80	83	4	4
40	60	8.8	83	82	21	22
100	0	14.1	72	58	10	51
40	60	14.1	85	90	21 <sup>+</sup>	17
30	70	14.1	80	82	30 <sup>+</sup>	20

<sup>+</sup> Frost damage

From the above it is concluded that exposure to sea water should be classified as exposure class XA2A. A similar judgement has been made by Committee B/517/1/1 (BSI, 1997a). The minimum concrete specification for concrete subject to XA2A exposure is given in Tables 5.8, 5.12 and 5.14.

## 5.6 Acid attack

### 5.6.1 Influence of cement type

In a review of concrete behaviour in acidic soils and groundwater, Eglington (1975) noted that although the national standards of a number of European countries recommended the use of pozzolanic or blast furnace cements in acid conditions, the literature generally indicated that in determining the resistance of concrete to acids, the quality of the concrete was more important than the type of cement used. Campus and Dzulynski (1969) and Efes (1984) showed an improvement in performance for a high slag cement over a CEM I cement when comparisons were made of mortar prisms exposed to carbonated water. Halstead (1954) observed that Portland blastfurnace cement, CEM I and CEM I/SR had similar resistance to acid attack. Biczik (1964) reports that Kind (1955) found that CEM I concretes behaved better than pozzolanic or slag concretes. Harrison (1987) concluded that the consensus of published data seemed to indicate a slower rate of attack on concretes with reduced lime contents. However, Harrison (1987) noted that for the particular requirement of acid resistance the limited reported data did not seem to justify any specific recommendation on the type of cementitious binder for use in concrete subject to acid attack.

Recent results obtained by Matthews (1995c) and Osborne (1994) on 100 mm concrete cubes immersed in moorland water in Yorkshire are given in Table 5.17. The pH of the water was found to range from 3.5 to 4.5. The criterion for assessment was the compressive strength retained at two, five and ten years compared with the compressive strength of companion tap-water-stored concrete cubes. Results obtained by Halstead (1953) on '100 mm' cubes immersed in moorland water in Yorkshire (pH 3.9 to 4.6) are also included in Table 5.17. Although the data are too sparse to enable firm judgements to be reached regarding the effect of cement type upon acid resistance, examination of the data in Table 5.17 indicates that the differences in performance between concretes made using different cements are probably too small to be of practical significance.

**Table 5.17:** Compressive strength retained.

Author	Cement	Mix details w-c ratio; cement content (kg/m <sup>3</sup> )		Compressive strength retained (%)				
				2 yr	4 yr	5 yr	10 yr	
Halstead (1954)	CEM I	0.6	-	-	55	-	-	
	CEM I/SR	0.6	-	-	55	-	-	
	CEM I	0.6	-	-	56	-	-	
	Blastfurnace (BS 4246)	0.6	-	-	53	-	-	
Matthews (1995c)	CEM I/SR, 0.8% C <sub>3</sub> A	0.44 - 0.59	300 - 400	83	-	63	55	
	+ 20% fa.	0.42 - 0.56	300 - 400	77	-	64	58	
	+ 40% fa.	0.44 - 0.60	300 - 400	71	-	59	56	
	CEM I, 7.0% C <sub>3</sub> A	0.44 - 0.48	360 - 400	81	-	60	53	
	+ 20% fa.	0.42 - 0.57	300 - 400	80	-	57	55	
	+ 40% fa.	0.39 - 0.54	300 - 400	73	-	59	52	
	CEM I, 14.1% C <sub>3</sub> A	0.46 - 0.61	300 - 400	82	-	70	51	
	+ 20% fa.	0.43 - 0.57	300 - 400	81	-	72	56	
	+ 30% fa.	0.42 - 0.54	300 - 400	78	-	61	62	
	+40% fa.	0.45 - 0.61	300 - 400	74	-	65	53	
	Osborne (1994)	CEM I/SR, 0.3% C <sub>3</sub> A	0.45	380	-	-	61	-
		+ 40% slag	0.45	380	-	-	61	-
+ 60% slag		0.45	380	-	-	62	-	
+ 70% slag		0.45	380	-	-	68	-	
CEM I, 8.8% C <sub>3</sub> A		0.45	380	-	-	55	-	
+ 60% slag		0.45	380	-	-	80	-	
+ 70% slag		0.45	380	-	-	82	-	

### 5.6.2 Minimum specification for concretes subject to sulfate and acid attack

The statement in BS 5328: Part 1: 1997 that "Concrete made with cements containing slag or pulverized-fuel ash can have some acid resistant properties" is not entirely justified by the published literature.

For concretes exposed to mobile groundwater of pH in the range 4.0 to 5.5 and sulfate exposure classes XA1A to XA3B, it is recommended that the classification is raised by one sulfate class or that the minimum cement contents given in Tables 5.8, 5.12 and 5.14 are increased by 40 kg/m<sup>3</sup> and the corresponding maximum water-cement ratios given in Tables 5.8, 5.12 and 5.14 are reduced by 0.05.

## 5.7 Minimum requirements for concrete to resist sulfate and chemical attack

### 5.7.1 Sulfate attack

Based on this study, the minimum requirement for concrete to resist sulfate attack for at least 100 years is given in Table 5.18. The requirements are applicable to well-compacted cast-in-situ concrete, 140 to 450 mm in thickness, and exposed on all faces to groundwaters containing sulfates, with a pH above 5.5.

**Table 5.18:** Limiting values for composition and properties of concrete exposed to aggressive sulfate environments for an intended working life of at least 50 years. pH > 5.5.

Exposure class	Slightly aggressive			Moderately aggressive or sea water exposure			Moderately aggressive		Highly aggressive		
	XA1B			XA2A			XA2B		XA3A		XA3B
Maximum w-c ratio	0.50	0.55	0.55	0.50	0.55	0.55	0.50	0.50	0.45	0.45	0.45
Minimum cement content (kg/m <sup>3</sup> )	340	300	280	340	300	280	340	320	380	360	360
Cement Group <sup>+</sup>	1 <sup>§</sup>	2	3	1 <sup>§</sup>	2	3	2	3	2	3	3

<sup>+</sup> See Table 5.19

<sup>§</sup> Excluding CEMII/A-L

**Table 5.19:** Types of cement.<sup>+</sup>

Group	Description
1	CEM I, CEM II/A-D, CEM II/A-L, CEM II/A-S, CEM II/B-S, CEM II/A-V, CEM II/B-V, CEM III/A, CEM III/B, CEM IV/A Combinations of CEM I cement with slag <sup>§</sup> Combinations of CEM I cement with fly ash <sup>§</sup>
2	CEM II/B-V where fly ash ≥ 25% by mass of nucleus CEM III/B where slag ≥ 74% by mass of nucleus <sup>‡</sup> CEM III/C where slag ≤ 90% by mass of nucleus <sup>‡</sup> CEM IV where 25% ≤ fly ash ≤ 40% by mass of nucleus Combinations of CEM I cement with not less than 70% and not more than 85% slag by mass <sup>‡</sup> Combinations of CEM I cement with not less than 25% and not more than 40% fly ash by mass
3	CEM I/SR

<sup>+</sup> See Glossary of cement types on back cover.

<sup>§</sup> Other than those combinations in group 2.

<sup>‡</sup> Blastfurnace slag with an alumina content greater than 14% should be used only with a CEM I cement having a tricalcium aluminate (C<sub>3</sub>A) content not exceeding 10%.

### 5.7.2 Acid attack

For concretes exposed to mobile groundwater of pH in the range 4.0 to 5.5 and a sulfate environment (exposure classes XA1B to XA3B) the classification in Table 5.18 should either be raised by one sulfate class or the minimum cement contents given in Table 5.18 should be increased by 40 kg/m<sup>3</sup> and the maximum water-cement ratio reduced by 0.05.

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## AUTHOR INDEX

Page numbers in brackets refer to pages in which the full references appear.

- Abduljanwad, S. W. 138, 139, 146, 151, (159)  
 Aïtcin, P. C. 105, 114, 116, (122), (124), (127)  
 Al-Amoudi, O. S. B. 138, 139, 146, 151, (159)  
 Al-Khaja, W. W. 80, (83)  
 Al-Isa, M. 69, 71, 73, 75, (84)  
 Al-Saadoon, S. S. 79, (88)  
 Alonso, C. 79, (83)  
 Anderson, A. 67, 68, 69, 71, 75, (83)  
 Andrade, C. 44, 79, (83), (88)  
 Anonymous 56, (84)  
 Antonakis, C. J. 56, (84)  
 Arya, C. 79, (84)  
 Ashworth, V. 44, (89)  
 ASTM. 97, 98, 99, 101, (122), (123)  
 Axon, E. O. 117, (123)
- Bakker, R. F. M. 34, (36)  
 Bamforth, P. B. 27, 28, 29, (36), 51, 52, 53, 54, 56, 67, 69, 71, 73, 75, 79, (84)  
 Bantia, N. 115, 124  
 Barker, A. P. 16, 17, 19, 21, 24, (36), 41  
 Bashenini, M. S. 75, (84)  
 Baweja, D. 27, 28, 29, (36), 64, (86), (88)  
 Beaupre, D. 114, (125)  
 Beeby, A. W. 16, 17, 19, 21, 24, (36), (42), 49, (84)  
 Benton, E. J. 138, (160)  
 Bentur, A. 153, (159)  
 Bernhardt, C. I. 153, (159)  
 Biczik, I. 156, (159)  
 Bieniak, B. T. 51, 52, 53, 54, 66, 78, (86)  
 Bier, Th. 113, (124)  
 Bilodeau, A. 110, 111, 113, 114, (123), (126)  
 Bonzel, J. 113, (123)  
 Breit, W. 34, (38), 79, (84)  
 Bremner, T. 106, 117, (126)  
 BSI. 1, 4, 5, 8, 10, 11, 12, 13, 14, 32, 33, 34, 40, 43, 47, 48, 49, 66, (84), (85), 91, 93, 94, 120, (123), 131, 132, (159)  
 BSI Committee B/516. 4 (10)  
 BSI Committee B/516/6. 3, (10)  
 BSI Committee B/517. 5, 8, (10), 11, 12, 13, (36), 43, 47, (85), 91, 93, 94, (123), 131, 132, 155, (159)  
 BSI Committee B/525/2/WG1. 13, (36)  
 Brown, J. H. 16, 17, 19, 21, 24, (36), 42, 67, 69, 70, 71, 75, 76, (85)  
 Buenfeld, N. R. 79, (85)  
 BRE. 48, (85), 136, (159)  
 Byfors, K. 16, 17, 18, 19, 22, (36), 40, 79, (85)
- Cameron, G. 101, (128)  
 Campus, F. 156, (159)  
 Cangiano, S. 102, (128)  
 Cao, H. T. 44, (86)  
 Carette, G. G. 104, 105, 106, 117, (123)  
 Carlson, R. 153, (159)  
 Carrasquillo, R. L. 138, 146, (162)  
 Chapman-Andrews, A. F. 51, 52, 53, 54, 79, (84)  
 Cheng-yi, H. 116, (124)  
 Chye, L. K. 53, (88)  
 Clark, L. A. 56, 75, (85)  
 Clendenning, T. G. 103, 104, 116, 117, (128)  
 Cohen, M. D. 153, (159)  
 Cook, D. J. 27, 28, 29, (36)  
 Cordon, W. A. 95, (124)  
 Corish, A. T. 3, (10)  
 Crammond, N. J. 137, 138, (159), (160)  
 Cripwell, J. B. 27, (38)
- Dakhil, F. 44, 80, (87)  
 De Maio, A. J. 113, (126)  
 Dhir, R. K. 12, (36), 44, (85), 115, (124)  
 Dikeou, J. T. 146, (160)  
 Dransfield, J. 115, (124)  
 DSIR. 49, (85)  
 Dunstan, E. R. 145, (160)  
 Dunster, A. M. 27, (37)  
 Dzulynski, M. 156, (159)
- Efes, Y. 156, (160)  
 Eglinton, M. S. 138, 156, (160)

- Emoto, Y. 116, (129)  
Entesham Hussain, S. 79, (88)  
European Standardisation. 13, (37), 43, 48, (85), 91, (124), 131, (160)
- Fagerlund, G. 17, 18, 19, 22, 27, 29, (37), 115, (124)  
Fiskaa, O. M. 142, (160)  
Foy, C. 114, 115, (124), (125)  
Frey, R. 27, (37)  
Fulton, F. S. 105, (124)  
Funahashi, M. 67, 69, (85)
- Gagnè, R. 114, 116  
Gautefall, O. 52, 53, 54, 56, (85)  
Glass, G. K. 79, (85)  
Gourley, J. T. 51, 52, 53, 54, 66, 78, (86)  
Guirguis, S. 44, (86)  
Günter, M. 113, (124)
- Hakkinen, T. 17, 18, 24, 25, (37)  
Halliwell, M. A. 137, (159), (160)  
Halstead, P. E. 137, 156, 157, (160)  
Hamada, M. 27, (37)  
Hammer, T. A. 115, (125)  
Hansson, C. M. 79, (86), (88)  
Harrison, T. A. 5, (10)  
Harrison, W. H. 137, 138, 139, 142, 145, 146, 150, 156, (160)  
Hartmann, C. 138, 139, 145, (160)  
Hattoti, K. (127)  
Hausmann, D. A. 44, (86)  
Haynes, C. A. 79, (89)  
Helland, S. 52, 53, 54, 55, (86)  
Highways Agency. 49, 58, 75, 78, (86)  
Hilsdorf, H. K. 112, 113, (124), (115)  
Hisaka, M. 115, (127)  
Hobbs, D. W. 16, 17, 19, 21, 23, 24, (37), 41, 49, 50, 52, 57, (86), 94, 96, 115, (125)  
Hogan, F. J. 105, (125)
- Iijima, Y. 105, (127)  
International Standards Organisation. 62, (86)  
Isabelle, H. L. 110, 117, (129)  
Isberner, A. W. 105, 112, 113, (125)
- Jaegermann, C. 63, 65, 65, (86)  
Johnston, D. C. 110, 111, 112, (125)  
Jones, M. R. (10), (36), (37), (38), (84), (85), (87), (161)
- Kalousek, G. L. 138, 146, (160)  
Kazama, Y. 115, (127).  
Keyser, J. H. 110, 118, (125)  
Kim, E. 28, 29, (38)  
Kind, V. V. 156, (160)  
Kirkby, G. 64, (88)  
Kleinschmidt, H. J. 24, 25, (37)  
Klieger, P. 99, 100, 103, 105, 109, 110, 112, 113, 117, (125)  
Klinghoffer, O. 44, 79, (87)  
Kobayashi, M. 100, 101, 102, 103, 115, (125)  
Kodama, K. 100, 101, 102, 103, 115, (125)  
Koibuchi, K. 105, (127)  
Kokubu, M. 27, 29, (37)  
Kollek, J. J. 138, 150, 151, (160)  
Kondo, R. 150, (160)
- Lamond, J. F. 108, (125)  
Langlois, M. 114, (125)  
Landgren, R. 109, 110, 117, (125)  
Larbi, J. A. 57, 66, 78, (87)  
Larson, T. D. 104, (125)  
Lawrence, C. D. 139, (161)  
Lea, F. M. 136, 137, (161)  
Liam, K. C. 51, 52, 53, 54, 56, (86)  
Lista, W. L. 62, (87)  
Locher, F. W. 138, 150, 151, (161)  
Lorman, W. R. 49, (86)  
Ludwig, H-M. 104, 113, 114, (128)  
Lumley, J. S. 138, 150, 151, (160)  
Luther, M. D. 113, (126)
- Maage, M. 52, 53, 54, 55, (86)  
MacInnis, C. 102, (126)  
Mackenzie, J. R. 52, 53, 54, 55, (86)  
Maslehuddin, M. 138, 139, 146, 151, (159)  
Mangat P. S. 44, (87)  
Mangotich, E. 138, 139, 145, (160)  
Marsh, B. K. (37), 42  
Mather, B. 105, 108, (126)

- Mather, K. 138, 141, 147, (161)  
 Malhotra, V. M. 104, 105, 106, 107, 108,  
 109, 110, 111, 113, 114, 117, (123),  
 (124), (126)  
 Matthews, J. D. 16, 17, 19, 20, 21, 22, 23,  
 24, 27, 28, 29, (36), (37), (38), 41, 53,  
 79, 82, 83, (87), (89), 104, 105, 115, 118,  
 119, (126), 138, 139, 141, 142, 145, 146,  
 147, 154, 155, 156, 157, (161)  
 McCarthy, M. J. 12, (36), 44, (85)  
 Mehta, P. K. 138, 139, 145, 147, 154,  
 (161)  
 Meusel, J. W. 105, (125)  
 Meyer, A. 17, 18, 24, 25, (37)  
 Mielenz, R. C. 102, (127)  
 Mikols, W. J. 113, (126)  
 Molloy, B. T. 44, (87)  
 Moringa, S. 62, 63, 64, (87)  
 Mukarram, K. 44, 80, (87)  
 Murray, L. T. 117, (123)
- Nagataki, S. 27, 28, 29, (37), (38)  
 Nagano, H. 51, 52, 54, 55, (87)  
 Naito, T. 51, 52, 54, 55, (87)  
 Nakakuro, E. 100, 101, 102, 103, 115,  
 (125)  
 Nanni, A. 62, (87)  
 Negami, S. 100, 101, 102, 103, 115, (125)  
 Neville, A. M. 92, (127)  
 Nilsen, T. 142, 153, (162)  
 Nilsson, L. O. 44, 79, (87)  
 Nishi, T. 27, (38)  
 Northwood, D. O. 51, 52, 53, 54, 56, (86).
- Ohga, H. 27, 28, 29, (38)  
 Ohta, T. 62, 63, (87)  
 Okada, E. 115, (127)  
 Oleson, C. C. 103, 117, (127)  
 Osborne, G. J. 24, 27, 30, (38), 118, (127),  
 138, 141, 142, 150, 151, 155, 156, 157,  
 (161)  
 Oshiro, T. 52, 54, 56, (87)
- Page, C. L. 44, (87)  
 Parrott, L. J. 12, 14, 16, 17, 19, 21, 24, 27,  
 (38),  
 Paul, I. S. 75, (84)
- Pettersonn, K. 44, 57, 79, (87), (88)  
 Pigeon, M. 97, 101, 105, 114, 115, 116,  
 (124), (125), (127), (128)  
 Plante, P. 102, (127), (128)  
 Pleau, R. 97, 101, 105, 114, 115, (127)  
 Polder, R. B. 57, 66, 78, (87)  
 Poole, T. 138, 139, 141, 145, 147, (162)  
 Porter, L. C. 138, 146, (160)  
 Poulsen, E. 44, 79, (87)  
 Procter, R. P. M. 44, (89)
- Racic, D. C. 102, (126)  
 Rasheeduzzafar. 44, 79, 80, (87), (88),  
 138, 139, 146, 151, (159)  
 Rasmussen, T. H. 114, (127)  
 Raupach, M. 34, (38), 79, (88)  
 Regourd, M. 105, (127)  
 Robson, G. 102, (128)  
 Roper, H. 29, (36), 64, (88)  
 Rosenberg, A. 44, (88)  
 Roy, S. K. 51, 52, 53, 54, 56, (86)  
 Rucker, R. M. 117, (123)
- Sakai, M. 105, (127)  
 Salta, M. M. 64, 65, (88)  
 Sandberg, P. 57, (87), (88)  
 Sandvik, M. 51, 52, 54, 55, 56, (88)  
 Saucier, F. 101, 102, (127), (128)  
 Schiessl, P. 12, 34, (38), 79, (84), (88)  
 Sellevold, E. J. 115, (125), 142, 153, (162)  
 Siebel, E. 113, (123)  
 Skjolsvold, O. (38), 40  
 Snow, P. G. 138, 146, (162)  
 Soeda, M. 116, (129)  
 Somerville, G. 56, 75, (88)  
 Sommer, H. 101, (128)  
 Sorenson, B. 79, (86)  
 Sorensen, H. E. 114, 116, (128)  
 Spooner, D. C. 1, 3, (10)  
 Sprouse, J. H. 102, (127)  
 Stark, D. 55, 56, 57, 73, 74, 80, (88), 104,  
 113, 114, (128), 138, 146, (162)  
 Stejskal, B. G. 62, (89)  
 Stoltzner, E. 67, 68, 71, 76, (86)  
 Sturup, V. R. 103, 104, 116, 117, (128)  
 Swayze, M. A. 92, 99, (128)

- Tanikawra, S. 52, 54, 56, (87)  
Teychenne, D. C. 138, 139, 142, 145, 146,  
150, (160)  
Tham, K. 115, (124)  
Thomas, M. D. A. 16, 17, 19, 20, 21, 22,  
23, 27, 28, 29, (38), 51, 52, 53, 54, 55,  
56, 67, 68, 69, 71, 72, 75, 76, 79, 82, 83,  
(88), (89)  
Thorvaldson, T. 153, (162)  
Tikalsky, P. J. 138, 146, (162)  
Tognon, G. 102, (128)  
Traetteberg, A. 105, (128)  
Tuutti, K. 12, (39), 44, 46, (89)
- Vassie, P. 67, 68, 69, 71, 72, 75, 76, (89)  
Verbeck, G. 103, 117, (127)  
Virtanen, J. 104, 105, 114, 116, (128)  
Vezina, D. 105, (122)
- Weaver, W. S. 110, 117, (129)  
Wallbank, E. J. 94, (128)  
Whiting, D. 62, (89), 110, 111, 115, (129)  
Whitlinger, J. E. 113, (126)  
Wick, S. O. 51, 52, 54, 55, 56, (88)  
Wiebenga, J. G. 57, (89), 118, (129)  
Wierig, H. 17, 18, 24, 25, 27, (39)  
Wong, G. 138, 139, 141, 145, 147, (162)
- Xu, Y. 79, (84)
- Yamato, T. 116, (129)  
Yoda, A. 27, (39)  
Yonezawa, T. 44, (89)

## SUBJECT INDEX

### Acid attack 156-157

- deterioration, caused by 132
- from natural ground water. 132, 156-157
- influence of cement type on 156
- influence of pH on 132, 135, 153, 157, 159
- specification to minimize 157, 159

### Additions 5

### Air content

- entrapped 99, 100
- minimum 99, 100, 122

### Air-entrainment

- freeze-thaw resistance, effect of 92, 103-115
  - air content. 45, 46, 47, 99-100, 103
  - air-void size. 99, 100
  - air-void spacing. 100-102
- testing for effectiveness of 97, 98, 103-115

### Alumina content of slag

- sulfate resistance, effect of 151-152

### ASTM

- freeze-thaw tests 97, 98

### Binder content

- minimum 31

### Blastfurnace cement 2, 3, 4

- see CEM I/slag

### Bleeding

- chloride ingress, effect of 49, 59

### BSI

- freeze-thaw test 98

### Carbonation 11-35

- binder content, effect on 31,
- cement type, effect on
  - CEM I concretes. 16-19, 27-28, 40, 41
  - CEM I/fly ash concretes. 19-23, 28-30, 40, 41
  - CEM I/silica fume concretes. 39
  - CEM I/slag concretes. 24-26, 30, 40, 41
- compressive strength effect on 17-18, 21, 23, 24-26
- curing, effect on 32

- measurement of depth of 15, 16
- normalizing 16, 27
- permeability, effect of 67
- water-cement ratio, effect on 16-22, 24-26, 27-30, 39

### Carbonation-induced corrosion 11-35

- assumptions made. 33-34
- corrosion rate for 27
- cover, influence on 16, 27, 34
- deterioration due to 14, 15
- exposure classes for 6, 11, 13, 32-33
- initiation period to 12
- minimizing deterioration due to 26, 30-31, 34-35
- propagation period due to 12, 16, 27

### CEM I cements 1-3

- strength classes. 3, 4

### CEM I concretes

- acid attack on 137-138, 156-157
  - minimizing 135, 157, 159
- carbonation in 16-19, 27-28, 40, 41
- carbonation-induced corrosion in 16, 34
- chemical attack on 132
- chloride-induced corrosion in 44-45, 55-57, 74-76, 79
- chloride ingress into
  - de-icing salt exposure, effect on 67-71
  - marine exposure, effect on 51-55
  - temperature, effect on 51
- corrosion rate for 16, 27, 79
- freeze-thaw attack in 94-97
  - air content, effect on 99-100, 103-104, 106-108, 109-110, 115
  - degree of saturation, effect on 92, 99
  - de-icing salt exposure, effect on 92, 96, 97, 109-110, 117, 118
  - marine exposure, effect on 94, 106-108, 117, 118
  - minimizing 92-93, 106, 114-115, 118-122
  - water exposure, effect on 92, 94, 95, 96, 99-104, 115-118
- marine exposure, effect on 154-155
- minimizing corrosion in
  - carbonation-induced. 26, 30-31, 34-35
  - de-icing salt-induced. 47, 49, 76-78, 80-81

- marine-induced. 47, 58-60, 66-67, 82-83
- sulfate attack on 132, 136-137, 139-143
- minimizing 143-144, 157-158

**CEM I/fly ash cements** 1-3

- strength classes 3, 4

**CEM I/fly ash concretes**

- acid attack on 156-157
- minimizing 135, 157-159
- carbonation in 19-23, 28-30, 40, 41
- chemical attack on 132
- chloride ingress into
- de-icing salt exposure. 71-74, 76
- marine exposure. 51, 53-55
- chloride-induced corrosion in 79
- corrosion rate in 79
- freeze-thaw attack in
- de-icing salt exposure, effect on 110-112, 115
- marine exposure, effect on 109, 118
- minimizing 93, 106, 114-115, 118-122
- water exposure, effect on 104, 115, 117
- marine exposure, effect on 155
- minimizing corrosion for
- carbonation-induced. 26, 30-31, 34-35
- de-icing salt-induced. 78, 80-81
- marine-induced. 60-61, 66-67, 82-83
- sulfate attack on 132, 145-149, 154-155
- minimizing 132-135, 150, 158

**CEM I/limestone cements** 1, 3

- strength classes 3, 4

**CEM I/limestone concretes**

- chemical attack on 132
- chloride-induced corrosion in 79
- freeze-thaw attack in
- water exposure, effect on 105, 115
- minimizing 106
- marine exposure, effect on 79, 122
- minimizing carbonation-induced corrosion in 34-35
- sulfate attack on 132, 144
- minimizing 145, 158

**CEM I/silica fume cement** 1, 2

**CEM I/silica fume concretes**

- acid attack on
- minimizing 158-159
- carbonation in 40
- chloride ingress into

- marine exposure, effect on 51, 53
- freeze-thaw attack into
- de-icing salt exposure, effect on 114, 116
- minimizing 106, 104-105, 118-122
- water-exposure, effect on 105, 116
- minimizing corrosion in
- carbonation-induced. 34-35, 39
- marine-induced. 60-61, 6-67, 82-83
- sulfate attack on 153-154
- minimizing 154, 158-159

**CEM I/slag cements**, 1-3

- strength classes 3, 4

**CEM I/slag concretes**

- acid attack on 156-157
- minimizing 135, 157-159
- carbonation in 24-26, 30, 40, 41
- chemical attack on 132
- chloride-induced corrosion in 57, 79
- chloride ingress into
- de-icing salt exposure. 71-72
- marine exposure. 51, 53-55
- corrosion rate for 79
- freeze-thaw attack in
- de-icing salt exposure, effect on 112-114, 115, 118
- marine exposure, effect on 108-109, 118, 155
- minimizing 106, 114-115, 118-122
- water exposure, effect on 104-105
- marine exposure, effect on 155
- minimizing corrosion in
- carbonation-induced. 26, 30-31, 34-35
- de-icing salt-induced. 78, 80-81
- marine-induced. 57, 60-61, 66-67, 82-83
- sulfate attack in 150-152
- effect of aluminate in slag on 151-152
- minimizing 132-135, 153, 157-158

**CEM I/SR cements** 2, 4, 5

- composition 5

**CEM I/SR concretes**

- acid attack on 132, 156, 157
- minimizing 157, 159
- chemical attack on 132
- chloride ingress into
- marine exposure, effect on 52, 54, 55-56
- chloride-induced corrosion in 55-57, 79-80

- freeze-thaw attack in
  - water exposure, effect on 103, 117
  - marine exposure, effect on 106-108
  - minimizing 93, 106, 114-115, 119-122
- marine exposure, effect on 154, 155
- minimizing corrosion in
  - carbonation-induced. 34-35
  - de-icing salt-induced. 76-78, 81-82
  - marine-induced. 58-60, 66-67, 82-83
- sulfate attack on 139-143, 154-155
  - minimizing 133, 143-144
- Cements**
  - additions within 5
  - constituents of 2, 3
  - equivalence of British and European 2, inside back cover
  - glossary of 2
  - strength classes for 3
  - sulfate levels in 1
  - sulfate resisting 4
  - types of 1, 2
- Cement content**
  - minimum 31
- Chemical attack** 131-162
  - acid attack. 8, 132
    - deterioration due to 132, 137-138
    - cement type, effect on 156-157
    - exposure classes for 9, 133-136
    - minimizing deterioration due to 135, 157-159
  - marine exposure. 154-155
    - minimizing deterioration due to 133, 135, 143-144, 145, 150, 153, 154, 155, 157-159
  - exposure classes for 7-10, 132-136
  - sulfate attack. 8, 132, 136-137, 138-154
    - deterioration due to 132, 136-137
    - minimizing deterioration due to 133, 135, 143-144, 145, 150, 153, 154, 157-159
- Chlorides**
  - bound 44, 67, 79
  - $C_3A$  content, effect on bound 79
  - corrosion threshold level for 44, 47, 48, 57, 58, 60, 74, 76, 79, 80
  - pH influence on 4, 57, 79
  - free 44, 79
- Chloride ingress** 43, 44, 50, 67
  - carbonation, effect on 67
  - de-icing salt exposure, effect on
    - for CEM I concretes. 67-71
    - for CEM I/fly ash concretes. 71-74
    - for CEM I/slag concretes. 71-73
    - diffusion coefficient for 67-71
    - surface chloride level for 67-71
  - exposure classes for 6, 7, 45-48
  - Fick's second law for 50-51
    - diffusion coefficient, from 50-55, 57, 58, 60, 67-71, 76, 80
    - surface chloride level, from 50-55, 57, 58, 60, 67-71, 76, 80
  - marine exposure, effect on 50-55
    - for CEM I concretes. 50-57
    - for CEM I/fly ash concretes. 50-55, 57
    - for CEM I/silica fume concretes. 50-55, 57
    - for CEM I/slag concretes. 50-55, 57
    - for CEM I/SR concretes. 50-57
    - diffusion coefficient for 50-55, 57, 58, 60
    - surface chloride level for 50-55, 57, 58, 60
  - temperature, effect on 51, 52, 53, 54
- Chloride-induced corrosion** 43-89
  - airborne salt exposure, effect on 62-66, 66-67
  - assumptions made for 79-80
  - cover, effect on 55-56, 58-61
  - de-icing salt exposure, effect on 74-76
    - in ponding tests. 73-74
    - service life for 74-76
  - examples of deterioration due to 44, 45, 48-50
  - exposure classes for 6, 7, 43, 45-48
  - initiation period for 4, 46, 57, 76
  - marine exposure, effect on
    - cement type. 50-55, 57
    - service life for 55-57, 76
  - minimum concrete specification for
    - airborne salt exposure. 66, 80-83
    - de-icing salt exposure. 44, 76-79, 80-81
    - marine exposure. 44, 58-61, 66-67, 82-83
  - propagation period for 44, 46, 55-57, 66, 76, 79
  - threshold level for 44, 57, 58
- Combinations** 5
  - proportions for 2

**Compressive strength**

- carbonation, effect of 17, 19, 21, 23, 24, 25, 26
- chloride ingress, effect of 56, 75
- chloride-induced corrosion, effect of 56, 75
- freeze-thaw attack, effect of 104, 106, 107, 108, 109, 115-117

**Concrete grade**

- minimizing carbonation-induced corrosion by 26, 30-31, 34-35
- minimizing chloride-induced corrosion by airborne salt exposure. 63, 66, 80-83  
de-icing salt exposure. 78-79, 80-81  
marine exposure. 59-61, 66-67, 82-83
- minimizing freeze-thaw attack by 93, 106, 114-115, 118-122

**Corrosion**

- carbonation-induced 16, 27, 70
- chloride-induced 44-46, 48-50, 56, 70, 75, 78-79

**Cover to reinforcement**

- minimum requirements for 49-50  
carbonation. 26, 30-31, 34-35  
de-icing salt exposure. 70, 75, 76-78, 80-81
- marine exposure. 56, 58-59, 66-67, 82-83

**Curing**

- carbonation, effect of 17, 19-25, 32, 40

**De-icing salt exposure**

- chloride ingress resulting from carbonation, effect on 67  
for CEM I concretes. 67-71  
for CEM I/fly ash concretes. 71-74  
for CEM I/slag concretes. 71-73  
water-cement ratio, effect on 70-71
- diffusion coefficient for 67-71, 76, 79, 80
- surface chloride level for 67-71, 76, 80
- examples of deterioration resulting from 45, 49, 97, 110-114, 117, 118
- exposure classes for 6, 43, 45-48
- service life for 74-76

**Deterioration due to**

- acid attack. 132, 137-138
- carbonation-induced corrosion. 12, 14-15
- chloride-induced corrosion. 44-45, 56, 69, 70, 73-74, 79
- freeze-thaw attack. 92, 94-97

- inadequate design 48-50
- inadequate construction 49-50
- sulfate attack. 132, 136-137

**Diffusion coefficient**

- for airborne salt exposure. 62-63
- for de-icing salt exposure. 67-71, 76, 79-80
- for marine exposure. 50-55, 57, 79-80
- from Fick's second law. 50-55, 57, 67-71, 76, 79-80
- temperature, effect on 51, 52

**Expansion**

- freeze-thaw induced 92, 99, 100, 105, 116
- sulfate induced 132, 138

**Exposure classes** 5-10

- for acid attack. 10, 135, 136
- for carbonation-induced corrosion. 6, 11, 13
- for chemical attack. 7, 131, 133-136
- for chloride-induced corrosion. 6, 7, 43, 45-48
- for freeze-thaw attack. 7, 91, 93-94
- for sulfate attack. 7, 133

**Factory-produced concrete**

- de-icing salt exposure, minimum requirements for 46-47, 76-79, 80-81
- marine exposure, minimum requirements for 46-47, 58-61, 66-67, 80, 82-83

**Fick's second law of diffusion** 51

**Fly ash**

- in cements (see CEM I/fly ash cements)
- in concretes (see CEM I/fly ash concretes)

**Freeze-thaw attack**

- aggregate, effect on 92, 95, 96, 99
- air-entrainment, effect on 92, 103-115
- air content, effect on 45, 46, 47, 99-100, 103
- air-void size, effect on 99, 100
- air-void spacing, effect on 100-102
- cement type, effect on  
for CEM I concretes. 92, 96, 97, 99-104, 106-108, 109-110, 115-118  
for CEM I/fly ash concretes. 104, 109, 110-112, 115, 117, 118  
for CEM I/limestone concretes. 105, 115  
for CEM I/silica fume concretes. 105, 114, 116



- for CEM I/slag concretes. 104-105, 108-109, 112-114, 115, 118, 155
  - compressive strength, effect on 104, 106, 107, 108, 109, 115-117
  - deterioration mechanisms caused by
    - expansion. 92, 94, 95, 96, 97
    - pop-outs. 92, 95
    - scaling. 92, 94, 96, 97
    - spalling. 92, 95, 96
  - examples of deterioration due to 94-97
  - exposure classes for 7, 91, 92-94
  - minimum concrete specification for
    - de-icing salt exposure. 93, 114-115, 118-122
    - marine exposure. 93, 114-115, 118-122
    - saturated exposure. 93, 106, 118-122
  - testing for
    - with exposure to salt. 98
    - without exposure to salt. 97-98, 99
  - visual rating for 106, 107, 108, 109, 110
- Ground granulated blastfurnace slag**
- in cements (see CEM I/slag cements)
  - in concretes (see CEM I/slag concretes)
- Groundwater**
- acids in 8, 10, 132, 135, 136, 156
  - sulfates in 8, 9, 132, 133, 135, 136
- In-situ concrete**
- de-icing salt exposure, minimum requirements for 46-47, 76-79, 80-81
  - marine exposure, minimum requirements for 46-47, 58-61, 66-67, 80, 82-83
- k-factor for carbonation** 19, 20, 22, 26, 30, 34
- Limestone**
- in cements (see CEM I/limestone cements)
  - in concretes (see CEM I/limestone concretes)
- Marine exposure** 50-55, 106-109, 154-155
- chemical attack due to 154-155
  - chloride ingress resulting from
    - CEM I concretes. 50-57
    - CEM I/fly ash concretes. 50-55, 57
    - CEM I/silica fume concretes. 50-55, 57
    - CEM I/slag concretes. 50-55, 57
    - CEM I/SR concretes. 50-57
  - diffusion coefficient for 50-55, 57, 58, 80
  - Fick's second law applied to 50-55
  - water-cement ratio, effect on 50-57, 58, 60
  - examples of deterioration due to 45, 94, 96, 154-155
  - exposure classes for 7, 46-48, 155
  - freeze-thaw attack due to 96, 106-109
  - minimum concrete specification for 58-61, 66-67, 82-83, 154-155
  - service life for 55-57
  - submerged concrete in a 66-67, 154-155
  - surface chloride level for 50-55, 57-58, 79-80
- Minimum specification**
- for acid attack. 135, 157, 159
  - for carbonation-induced corrosion. 26, 30-31, 34-35
  - for chloride-induced corrosion.
    - de-icing salt exposure. 76-79, 80-81
    - marine exposure. 58-61, 82-83
  - for freeze-thaw attack. 93, 106, 114-115, 118-122
  - for sulfate attack. 133-135, 143-144, 145, 150, 153, 154, 157, 158
- Permeability** 34, 67, 132, 133, 135, 136, 154
- pH**
- acid attack, effect of 8, 10, 132, 153, 159
  - chloride threshold level, effect of 44, 57, 79
- Phenolphthalein test** 15
- Plastic settlement cracks**
- chloride ingress, effect of 49
- Portland cement (see CEM I cement)**
- Portland limestone cement (see CEM I/limestone cement)**
- Pop-outs** 92, 95, 96, 99, 118
- Pozzolan cement** 2, 4
- see CEM I/flyash cement
- Propagation period**
- for carbonation exposure. 12, 16, 27
  - for de-icing salt exposure. 74, 76, 79
  - for marine exposure. 55-57, 58, 60, 66, 79
- Pulverized fuel ash**
- in cements (see CEM I/fly ash cements)
  - in concretes (see CEM I/fly ash concretes)

**Reinforcement cover**

- for minimizing carbonation-induced corrosion. 11, 13, 34-35
- for minimizing chloride-induced corrosion. 58-61, 66, 76-79, 80-83

**Salt**

- concretes subject to air-borne salt. 62-67, 78, 80-83
- concretes subject to carbonation and salt. 67-78, 80-81
- concretes subject to de-icing salt exposure. 46-48, 49, 67-79, 80-81, 92, 96, 109-114, 115, 117
- concretes subject to marine exposure. 46-48, 49, 50-67, 82-83, 94, 106-109, 118, 154-155

**Scaling**

- de-icing salt exposure, effect on 92, 96, 109-114, 115, 117
- marine exposure, effect on 94, 118, 154-155

**Service life**

- for de-icing salt exposure. 74-76
- for marine exposure. 5-57

**Silica fume (see CEM I/silica fume cement and concrete)**

**Slag**

- in cements (see CEM I/slag cements)
- in concretes (see CEM I/slag concretes)

**Sulfate-resisting cement 4, 5**

**Sulfates**

- in cements. 1, 3, 5
- in groundwater. 8, 9, 132, 133, 136, 137, 153
- in sea water. 154

**Surface chloride level**

- from Fick's second law. 51
- for de-icing salt exposure. 67-71, 74, 76, 80-81
- for marine exposure. 50-55, 57, 58, 60, 82-83

**Sulfate attack**

- by natural ground water. 132, 137
- CEM I, effect on 139-143
- CEM I/limestone, effect on 144
- CEM I/pfa, effect on 145-149

- CEM I/slag, effect on 150-152
- CEM I/silica fume, effect on 153-154
- concrete affected by 132, 136-137
- criteria for assessing 138-139
- deterioration due to 132, 136-139
- exposure classes for 8, 10, 132-136
- mechanisms of 132
- minimizing deterioration due to 133-135, 143-144, 145, 150, 153, 154, 157-159
- testing for 138-139
- water-cement ratio, effect on 140-141, 149

**Thaumasite formation 137**

**Threshold chloride level**

- cement type, effect on 57, 49, 80
- for corrosion 44, 47, 48, 57, 58, 60, 74, 76, 79, 80
- pH, effect on 44, 57, 79

**Tricalcium aluminate**

- bound chlorides, effect of 79
- chloride ingress, effect of 55-56, 80
- corrosion rate, effect of 79
- limits on 2, 5, 134, 158
- marine performance, effect of 55-56, 80, 154-155
- sulfate attack, effect of groundwater exposure. 132
- in laboratory tests. 139-143, 144, 145-149, 150-152
- marine exposure. 154-155

**Water-cement ratio**

- carbonation, effect of 16-22, 24-26, 27-30, 39
- chloride ingress, effect of marine exposure. 51-55
- de-icing salt exposure. 70-71
- freeze-thaw resistance, effect of 103, 104, 106, 107, 108, 109, 110, 111, 112, 115-118
- minimizing carbonation-induced corrosion by 26, 30-31, 34-35
- minimizing chloride-induced corrosion by for de-icing salt exposure. 76-78, 80-81
- for marine exposure. 58-61, 82-83
- minimizing freeze-thaw attack by 92-93, 106, 114-115, 118-122

**GLOSSARY OF CEMENT TYPES: equivalence of British and European cements**

<b>Nomenclature</b>	<b>Cement designation to DD ENV 197-1</b>	<b>Clinker content (%)</b>	<b>Content of other main constituents (%)</b>	<b>British Standard cement</b>
CEM I CEM I/SR	Portland cement	95 - 100 95 - 100	- -	BS 12: 1996 BS 4027: 1996 <sup>+</sup>
CEM II/A-S CEM II/B-S	Portland-slag cement	80 - 94 65 - 79	6 - 20 21 - 35	BS 146: 1996
CEM II/A-D	Portland-silica fume cement	90 - 94	6 - 10	None
CEM II/A-P CEM II/B-P CEM II/A-Q CEM II/B-Q	Portland-pozzolana cement	80 - 94 65 - 79 80 - 94 65 - 79	6 - 20 21 - 35 6 - 20 21 - 35	None None None None
CEM II/A-V CEM II/B-V CEM II/A-W CEM II/B-W	Portland-fly ash cement	80 - 94 65 - 79 80 - 94 65 - 79	6 - 20 21 - 35 6 - 20 21 - 35	BS 6588: 1996 BS 6588: 1996 None None
CEM II/A-T CEM II/B-T	Portland-burnt shale cement	80 - 94 65 - 79	6 - 20 21 - 35	None None
CEM II/A-L CEM II/B-L	Portland-limestone cement	80 - 94 65 - 79	6 - 20 21 - 35	BS 7583: 1996 None
CEM II/A-M CEM II/B-M	Portland-composite cement	80 - 94 65 - 79	6 - 20 21 - 35	None None
CEM III/A CEM III/B CEM III/C	Blastfurnace cement	35 - 64 20 - 34 5 - 19	36 - 65 66 - 80 81 - 95	BS 146: 1996 <sup>§</sup> None <sup>§</sup> None <sup>§</sup>
CEM IV/A CEM IV/B	Pozzolanic cement	65 - 89 45 - 64	11 - 35 36 - 55	None BS 6610: 1996
CEM V/A CEM V/B	Composite cement	40 - 64 20 - 39	36 - 60 61 - 80	None None

<sup>+</sup> Cement to BS 4027: 1996, *Sulfate resisting Portland cement* is included here but will eventually be covered specifically in a future part of DD ENV 197-1.

<sup>§</sup> BS 4246: 1996 high slag blastfurnace cement covers a cement with a blastfurnace slag content of 50 - 85%

Note: ASTM Type I to V cements contain no minor additional constituent

ASTM Type I (no C<sub>3</sub>A limit) and ASTM Type II (C<sub>3</sub>A ≤ 8%) are equivalent to CEM I/'N'

ASTM Type III (C<sub>3</sub>A ≤ 15%) is equivalent to CEM I/R

ASTM Type IV (C<sub>3</sub>A ≤ 7%) is equivalent to what may be CEM I/42.5 MH

ASTM Type V (C<sub>3</sub>A ≤ 5%) is equivalent to CEM I/SR (ASTM C 150-96).

CI/5fb			
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This book will give civil, structural and consulting engineers, and other users and producers of cement, aggregate and concrete, a better understanding of the more common causes of concrete deterioration and their effects, and how deterioration can be minimized.

The topics covered in the book include common cements, concrete exposure classes, carbonation and carbonation-induced corrosion, chloride ingress and chloride-induced corrosion, freeze-thaw attack, sulfate attack, acid attack and the minimum requirements for durable concrete subject to a range of exposure conditions. The minimum requirements are applicable to countries where concrete is subject to similar exposure conditions as in the UK.

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