BETONGKONSTRUKSJONERS LIVSLØP

Et utviklingsprosjekt i samarbeid mellom offentlige byggherrer, industri og forskningsinstitutter

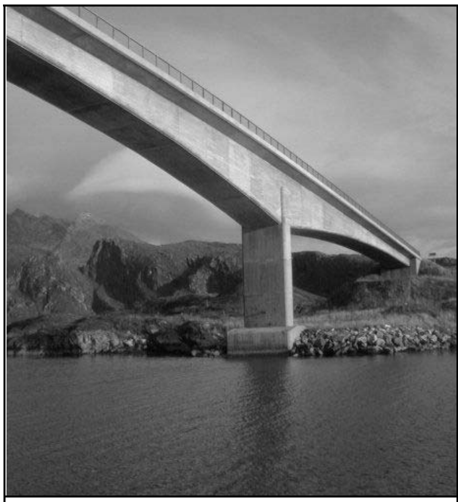
Deltakere:
Statens vegvesen (prosjektledelse), Forsvarsbygg, NORCEM A.S, Selmer Skanska AS, Sika Norge AS, Norges byggforskningsinstitutt, NTNU, SINTEF, NORUT Teknologi as

Rapport nr. 19

Service Life Design of Concrete Structures

Aktivitet DP1 B4

Prosjektet er støttet av BA-programmet i Norges forskningsråd



BETONGKONSTRUKSJONERS LIVSLØP

Rapport nr. 19

Service Life Design of Concrete Structures

Aktivitet DP1 B4

Utgiver:

Statens vegvesen, Vegdirektoratet
Postadresse: Teknologiavdelingen
Postboks 8142 Dep
0033 OSLO
Telefon: 02030
Telefaks: 22 07 38 66

FORFATTER(E):

- Helland, S., Skanska
- Maage, M. og Smeplass, S., Skanska
- Fluge, F., Statens vegvesen

INTERN RAPPORT NR.

GRADERING	ANTALL SIDER OG BILAG
Åpen	12+9+16+22

RAPPORTNR./ ISBN.:	DATO	PROSJEKTLEDERE:
82-91228-25-6	Januar 2003	Finn Fluge og Bernt Jakobsen

KORT SAMMENDRAG

Levetidsmodellering basert på probabilistiske metoder har vært tema for et europeisk nettverkssamarbeid, ”BriteEuRam Thematic Network DuraNet” som ble avsluttet i 2001.

Levetidsprosjektering er ennå ikke brakt frem til et nivå som gjør den moden til å bringes inn i regelverket, men resultater fra ”Betongkonstruksjoners livsløp” er benyttet i standardiseringsarbeid. Prosjektresultatene har gitt grunnlag for valg av statistisk signifikante verdier til bruk ved levetidsberegninger.

Foreliggende rapport omfatter 3 innlegg presentert i Tromsø, juni 2001, under DuraNet møte ”Service Life Design of Concrete Structures – From Theory to Standardisation”.

STIKKORD	NORSK	ENGLISH
	Bestandighet	Durability
	Karbonatisering	Carbonation
	Kloridinntrengning	Chloride ingress
	Levetidsprosjektering	Service Life Design

Rapport **Nr. 19** Service Life Design of Concrete Structures

Prosjekt **Betongkonstruksjoners livsløp**
Et utviklingsprosjekt i samarbeid mellom offentlige byggherrer, industri og forskningsinstitutter.

Aktivitet **DP1 B4**

Deltagere
Statens vegvesen (prosjektledelse),
Forsvarsbygg,
NORCEM A.S,
Selmer Skanska AS,
Sika Norge AS,
Norges byggforskningsinstitutt,
NTNU,
SINTEF,
NORUT Teknologi as

Prosjektet er støttet av BA-programmet i Norges forskningsråd

ISSN 1502-2331
ISBN 82-91228-25-6
50 eksemplarer trykt av
Statens vegvesen, Teknologiavdelingen

Adresse
Vegdirektoratet, Teknologiavdelingen
Postboks 8142 Dep
N-0033 Oslo, Norway

Telefon + 47 02030
Telefax + 47 22 07 39 66

Emneord
Bestandighet
Karbonatisering
Kloridinntrengning
Levetidsberegning

Key words
Durability
Carbonation
Chloride Ingress
Service Life Design

FORORD

Fokus er i løpet av de senere årene flyttet fra bygging av nye konstruksjoner over mot forvaltning hvor det legges større vekt på problemstillinger knyttet til drift, vedlikehold og gjenbruk av eksisterende konstruksjoner.

Prosjektet "Betongkonstruksjoners livsløp" er knyttet opp mot denne typen utfordringer som en samlet bygg- og anleggsbransje står overfor. Kravene til bygg- og anleggskonstruksjoner er at de skal være funksjonelle og kostnadseffektive. Offentlige byggherrer forvalter og vedlikeholder et stort antall konstruksjoner som skal møte samfunnets krav til:

- sikkerhet
- kvalitet/økonomi
- miljø

Det ble de siste årene av 90-tallet lagt ned et betydelig arbeid i prosjektet "Bestandige betong-konstruksjoner". Av resultatene fra dette prosjektet og erfaringene fra prosjektet "OFU Gimsøystraumen" fremgår det klart at beslutningen om å bygge bestandige betongkonstruksjoner må tas tidlig i planleggingsfasen og at det er behov for enkelt å kunne verifisere prosjekteringsforutsetningene.

"Betongkonstruksjoners livsløp" bygger videre på forannevnte prosjekter. Hovedvekten er lagt på klart formulerte forskningsoppgaver som dels konkretiserer eksisterende kunnskap og dels fyller hull i kunnskapsgrunnlaget. Aktivitetene er valgt innenfor en ramme som omfatter alle faser fra planlegging til riving og gjenbruk.

Prosjektets hovedmålsetning har vært:

Kostnadseffektive og miljøgunstige betongkonstruksjoner

med følgende delmål:

- Identifisere hovedparametre i levetidsmodellene og kalibrere dem mot felterfaringer
- System for vurdering av vedlikeholdstiltaks levetid
- System for instrumentell overvåkning av betongkonstruksjoners tilstandsutvikling
- Kunnskapsformidling gjennom normarbeid, kurs og internasjonale nettverk

Prosjektets sluttprodukter er:

- Grunnlag for veiledninger og regler for levetidsprosjektering
- Akseptkriterier for bedømmelse av betongkonstruksjoners bestandighet
- Datagrunnlag til bruk i standardiseringsarbeid og som inngangsdata til europeisk nettverksarbeid
- Kunnskap og kompetanse knyttet til sensorteknologi, måleteknikk, "intelligent" instrumentell overvåkning, katodisk beskyttelse etc., hvor industripartnerne gis mulighet til å utnytte resultatene kommersielt

Prosjektet har bestått av flere større og mindre aktiviteter gruppert i følgende delprosjekter:

- DP1. Levetidsprosjektering
  A. Datainnsamling
  B. Levetidsmodeller
- DP2. Vedlikeholds- og oppgraderingsmetoder
  A. Vedlikeholdsmetoder
  B. Oppgraderingsmetoder
  C. Rustfri armering
- DP3. Måleteknikk

Aktivitetene i prosjektet er basert på enkeltforslag fra prosjektdeltakerne. Hvor aktivitetene hadde fellestrekk, kunne levere resultater til, eller benytte resultater fra andre aktiviteter ble dette identifisert ved oppstarten av prosjektet og nødvendig koordinering foretatt. Ellers er aktivitetene styrt meget selvstendig.

Prosjektet startet høsten 1999 og ble avsluttet høsten 2001. Prosjektet har vært støttet av BA-programmet i Norges forskningsråd med NOK 1 mill i hvert av årene 1999 og 2000.

I tillegg til støtten fra Norges forskningsråd har det vært ytet en betydelig egeninnsats fra deltakerne i form av personalinnsats og kjøp av FoU-tjenester. Prosjektkostnadene per 31-12-00 var NOK 7,25 mill, hvorav NOK 2,7 mill var benyttet til kjøp av FoU-tjenester fra forskningsinstitutter og NOK 0,5 mill fra konsulent. I år 2001 ble det kjøpt tjenester for NOK 1,7 mill som i sin helhet ble finansiert av prosjektdeltagerne. Samlede prosjektkostnader ved avslutningen av prosjektet er ca. NOK 9 mill.

Prosjektet har hatt følgende deltakere:

Statens vegvesen
Forsvarsbygg
NORCEM A.S
Selmer Skanska AS
NTNU
SINTEF
Sika Norge AS
Norges byggforskningsinstitutt
NORUT Teknologi as

I tillegg har prosjektet samarbeidet med Det Norske Veritas og ARMINOX, som alle har bidratt med egeninnsats.

Det er knyttet to dr. gradsstudenter til prosjektet.
Prosjektet mottok i juni 2000 et 3 års dr.grad stipendium. Stipendiat ble tilsatt 01-01-2001.

Prosjektet har vært ledet av Vegdirektoratet. Prosjektledelsen, som har bestått av Finn Fluge Vegteknisk avdeling, Vegdirektoratet og Bernt Jakobsen, Aadnesen a.s, har rapportert til en styringskomite som har bestått av representanter fra prosjektdeltakerne. Styringskomiteen har vært samlet to ganger årlig eller ved behov og har fastlagt mål og hovedstrategier.

SUMMARY

Durability of concrete structures was in the early nineties put on the agenda within European Concrete Standardisation. Initially the work concentrated to the degradation process, important durability factors and how the process was affecting the bearing capacity of structural elements.

At present, probabilistic service life design is on the agenda worldwide. The BritEuRam project DuraCrete (1996-99) played a central role in this development and has been followed by a European network, The BriteEuRam Thematic Network DuraNet, which terminated at the end of 2001.

Durability data, collected from a large number of existing concrete structures, are processed and made available for calibrating probabilistic service life models. Service Life Design is at present not mature for standardisation, but work has been performed, within DuraNet, RILEM, ***fib*** etc., to establish reliable computation methods based on the same principles as for normal Structural Design.

Additionally the results from the project have been used stating the national requirements in the National Annex to the European Standard EN 206-1 “Concrete – Part 1 Specification, performance, production and conformity”. The results have also formed the basis for choosing significant in-put parameters for service life computations.

This report consists of three papers presented at the DuraNet workshop: “Service Life Design of Concrete – From Theory to Standardisation”, Tromsø June 2001.

The papers deal with Life Time Models for computation of time until start of reinforcement corrosion in Concrete Structures exposed for Carbonation and ingress of Marine Chlorides. The models are based on the same probabilistic approach as defined in EN 1990 EuroCode: “Basis of Structural Design”.

Rapport nr.1:	TITTEL**:**	Feltdata for kloridinitiert armeringskorrosjon. Sammenstilling og kvalitetsvurdering av tilgjengelige data.
	Aktivitet:	DP1 A1
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Intern rapport nr. 2197.
	Forfattere:	Skjølsvold, O., Jacobsen, S., Lahus, O., Lindgård, J., Hynne, T.
	ISSN	1502-2331
	ISBN	82-91228-04-3
	Sider:	12+9+7 vedlegg + CD-ROM
	Dato:	Desember 2002
Rapport nr. 2:	TITTEL:	Laboratoriedata for kloridinitiert armeringskorrosjon.
	Aktivitet:	DP1 A1
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A00732.
	Forfattere:	Hynne, T. og Lindgård, J.
	ISSN	1502-2331
	ISBN	82-91228-07-8
	Sider:	13+35+16 vedlegg
	Dato:	Januar 2003
Rapport nr. 3:	TITTEL:	Gimsøystraumen bru. Spesialinspeksjon 1992-kloridprofiler. Vurdering av kloridbelastning og diffusjonskoeffisient
	Aktivitet:	DP1 A1
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Intern rapport nr. 2196.
	Forfattere:	Skjølsvold, O.
	ISSN	1502-2331
	ISBN	82-91228-08-6
	Sider:	14+18+3 vedlegg+CD-ROM
	Dato:	Januar 2003
Rapport nr. 4:	TITTEL:	Kloridinntrengning i ressursvennlig kvalitetsbetong.
	Aktivitet:	DP1 A2
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NORCEM rapport
	Forfattere:	Kjellsen, K.O. og Skjølsvold, O.
	ISSN	1502-2331
	ISBN	82-91228-09-4
	Sider:	14+16+13 vedlegg
	Dato:	Januar 2003

Rapport nr. 5:	TITTEL:	Statistisk beregning av levetid for betongkonstruksjoner utsatt for kloridinntrengning.
	Aktivitet:	DP1 B1
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A01613.
	Forfattere:	Hynne, T., Leira, B.J., Carlsen, J.E. og Lahus, O.
	ISSN	1502-2331
	ISBN	82-91228-10-8
	Sider:	14+59+3 vedlegg
	Dato:	Februar 2003

Rapport nr. 6:	TITTEL:	Dimensjoneringsformat for kloridbestandighet.
	Aktivitet:	DP1 B1
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport STF22 A02601.
	Forfattere:	Leira, B.J.
	ISSN	1502-2331
	ISBN	82-91228-11-6
	Sider:	14+36+ 1 vedlegg
	Dato:	Februar 2003

Rapport nr. 7:	TITTEL:	Pålitelighetsmetodikk ved bruk av FDV og levetidsberegninger.
	Aktivitet:	DP1 B2
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Aas-Jakobsen. Rapp 6943-01.
	Forfattere:	Larsen, R.M.
	ISSN	1502-2331
	ISBN	82-91228-12-4
	Sider:	14 + 67
	Dato:	Februar 2003

Rapport nr. 8:	TITTEL:	Effekt av reparasjon på levetid: Eksempelstudie fra Gimsøystraumen.
	Aktivitet:	DP1 B3
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A01607.
	Forfattere:	Hynne, T. og Leira, B.J.
	ISSN	1502-2331
	ISBN	82-91228-13-2
	Sider:	12 + 22 + 7 vedlegg
	Dato:	Oktober 2006

Rapport nr. 9:	TITTEL:	Bestandighet og levetid av reparerte betongkonstruksjoner.
	Aktivitet:	DP2 A2
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NORUT Teknologi as rapport NTAS F2001-36.
	Forfattere:	Arntsen, B.
	ISSN	1502-2331
	ISBN	82-91228-14-0
	Sider:	14 + 20
	Dato:	Oktober 2006

Rapport nr. 10:	TITTEL:	Restlevetid – Kai Sjursøya.
	Aktivitet:	DP2 A3
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Selmer Skanska AS, rapport nr. B 01-01.
	Forfattere:	Carlsen, J.E.
	ISSN	1502-2331
	ISBN	82-91228-15-9
	Sider:	12 + 15 + 7 vedlegg
	Dato:	November 2006

Rapport nr. 11:	TITTEL:	Feltforsøk Sykkylven bru.
	Aktivitet:	DP2 A4
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. Selmer Skanska AS, rapport nr. B 01-02
	Forfattere:	Carlsen, J.E.
	ISSN	1502-2331
	ISBN	82-91228-16-7
	Sider:	12 + 9 +30
	Dato:	Desember 2006

Rapport nr. 12:	TITTEL:	Strengthening Prestressed Concrete Beams with Carbon Fiber Polymer Plates.
	Aktivitet:	DP2 B1
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NTNU, Institutt for konstruksjonsteknikk.
	Forfattere:	Takacs, P.F. og Kanstad, T.
	ISSN	1502-2331
	ISBN	82-91228-17-5
	Sider:	14 + 46 + 12
	Dato:	Desember 2006

Rapport nr. 13:	TITTEL:	Forsterking av betongsøyler med karbonfiberverv.
	Aktivitet:	DP2 B2
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A00718.
	Forfattere:	Thorenfeldt, E.
	ISSN	1502-2331
	ISBN	82-91228-18-3
	Sider:	14 + 22 + 3 vedlegg
	Dato:	Desember 2006

Rapport nr. 14:	TITTEL:	Forankringskapasitet av CFAP-bånd limt til betong.
	Aktivitet:	DP2 B2
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A01618.
	Forfattere:	Thorenfeldt, E.
	ISSN	1502-2331
	ISBN	82-91228-19-1
	Sider:	14 + 20 + 2 vedlegg
	Dato:	November 2007

Rapport nr. 15:	TITTEL:	Nonlinear Finite Element Analysis of Deteriorated and Repaired RC Beams
	Aktivitet:	DP2 B3
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NORUT Teknologi as rapport NTAS F2001-31.
	Forfattere:	Sand, B.
	ISSN	1502-2331
	ISBN	82-91228-20-5
	Sider:	15 + 34
	Dato:	Desember 2007

Rapport nr. 16:	TITTEL:	Styrkeberegning ved korrosjonsskader.
	Aktivitet:	DP2 B3
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. SINTEF. Rapport nr. STF22 A01619.
	Forfattere:	Stemland, H.
	ISSN	1502-2331
	ISBN	82-91228-21-3
	Sider:	14 + 57
	Dato:	Desember 2007

Rapport nr. 17:	TITTEL:	Korrosjonsegenskaper for rustfri armering.
	Aktivitet:	DP2 C1
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NTNU, Institutt for konstruksjonsteknikk. Rapport R-9-01.
	Forfattere:	Vennesland, Ø.
	ISSN	1502-2331
	ISBN	82-91228-22-1
	Sider:	12 + 16
	Dato:	Januar 2008

Rapport nr. 18:	TITTEL:	Heftforhold for rustfritt armeringsstål.
	Aktivitet:	DP2 C2
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling. NTNU rapport.
	Forfattere:	Hofsøy, A., Sørensen, S.I. og Markeset, G.
	ISSN	1502-2331
	ISBN	82-91228-24-8
	Sider:	12 + 44
	Dato:	Januar 2008

Rapport nr. 19:	TITTEL:	Service Life Design of Concrete Structures
	Aktivitet:	DP1 B4
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling.
	Forfattere:	Helland, S., Maage, M., Smeplass, S., Fluge, F.
	ISSN	1502-2331
	ISBN	82-91228-25-6
	Sider:	12 + 9 +16 + 22
	Dato:	Januar 2008

Rapport nr. 20:	TITTEL:	SLUTTRAPPORT
	Aktivitet:	-
	Utgiver:	Statens vegvesen, Vegdirektoratet, Vegteknisk avdeling.
	Forfattere:	Fluge, F. og Jakobsen, B.
	ISSN	1502-2331
	ISBN	82-91228-26-4
	Sider:	
	Dato:	

INNHOLDSFORTEGNELSE

SAMMENDRAG

Betongkonstruksjoners bestandighet ble, i det europeiske standardiseringsarbeidet, satt på dagsordenen omkring 1990. I den første tiden ble arbeidet konsentrert om å etablere forståelse for hvordan de ulike nedbrytningsprosessene virker.
Levetidsmodellering basert på probabilistiske metoder fikk gjennomslag som følge av BriteEuRam-prosjektet DuraCrete (1996-99). Dette arbeidet er senere fulgt opp gjennom et europeisk nettverkssamarbeid ”BriteEuRam Thematic Network DuraNet” som ble avsluttet i 2001.

Bestandighetsdata fra eksisterende konstruksjoner, innsamlet i prosjektet ”Betongkonstruksjoners livsløp” er gjort tilgjengelig for kalibrering av probabilistiske levetidsmodeller. Levetidsprosjektering er ennå ikke brakt frem til et nivå som gjør den moden til å bringes inn i regelverket. Det pågår imidlertid arbeid, innen så vel DuraNet, RILEM som *fib*, med sikte på å etablere en pålitelig beregningsmetodikk for levetid som en utvidelse av normal konstruktiv prosjektering.

Arbeidet har dessuten bestått i å formidle resultater fra prosjektet til bruk i standardiseringsarbeid i både CEN og NBR, Gjennom dette arbeidet har man kunnet knytte krav til armeringsoverdekning, masseforhold, rissvidder etc. til levetid når konstruksjonen utsettes for en nærmere spesifisert eksponering. Videre har resultatene gitt grunnlag for valg av statistisk signifikante verdier som kan brukes ved levetidsberegninger.

I foreliggende rapport er det samlet 3 innlegg presentert på DuraNet samlingen, “Service Life Design of Concrete Structures - From Theory to Standardisation”, Tromsø juni 2001.

Rapportene behandler modeller for å beregne tiden frem til initiering av armeringskorrosjon for betongkonstruksjoner eksponert for karbonatisering og kloridinntrengning. Beregningsmodellene er basert på de prinsipper for probabilistiske beregninger som er definert i EN 1990 EuroCode: ”Basis of Structural Design”.

DuraNet

Third Workshop, Tromsø 10th – 12th June 2001
On the theme:

"Service Life Design of Concrete Structures
–
From Theory to Standardisation"

Basis of design
Structural and service life design, a common approach

Steinar Helland
Selmer Skanska AS
Oslo, Norway

SCOPE

Probabilistic service life design is presently on the agenda within the concrete community worldwide.
Within organizations like CEN, fib and RILEM various aspects of this technology are discussed.
As the technology and philosophy on the subject gradually mature, we have dedicated this workshop to how such an approach can be anchored in the European Standards and thus being made available to the construction industry.
This paper is meant to provoke discussions in our group by giving some personal remarks and describing some possible scenarios.
The challenge of the author is that ¼ of the audience have been active in writing the Eurocodes, while another ¼ of you have a background and expertise far away from the principles behind structural design. I hope both groups will have patience with me while treating the range from structural philosophy to materials science with a broad pen.

STRUCTURAL DESIGN – CONCRETE STRUCTURES

In 1975 the EU Commission initiated an action plan in the field of construction to eliminate trade barriers and to harmonize technical specifications.

In 1989 EU and EFTA decided to transfer this work to CEN (European Committee for Standardization). Through a series of mandates, the Eurocodes then got the status of European Standards (EN).
The Eurocodes became then de facto linked to the EU Council Directive 89/106/EEC on construction products and 93/37/EEC, 92/50/EEC, 89/440/EEC on public works and services.

For concrete related construction, 3 Eurocodes are essential:

- EN 1990 – Eurocode: Basis of Structural Design
- EN 1991 – Eurocode 1: Actions on Structures
- EN 1992 – Eurocode 2: Design of Concrete Structures

These standards are today available as ENV 1991 “Basis of design and actions on structures” and ENV 1992 “Design of concrete structures”.
The draft version for EN 1990 was made available for inquiry in April this year, while the draft for EN 1992 became available in January.

EN 1990 describes the principles and requirements for safety, serviceability and durability of structures. It is based on the limit state concept used in conjunction with a partial factor method.
EN 1990 also gives guidelines for the aspects of structural reliability relating to safety, serviceability and durability.

EN 1990 implemented in the various European countries will be supplemented with “National Annexes” giving the exact “Nationally Determined Parameters” defining the required level of reliability. These levels of reliability are normally linked to national law.
In Norway, the present parallel document to ENV 1991 is NS 3490:1999 “Design of structures, Requirements to reliability”.

The hierarchy of documents will then be that:

1. EN 1990 “Basis of structural design” defines the general (not material specific) overall level of reliability to fulfill the requirements of the society

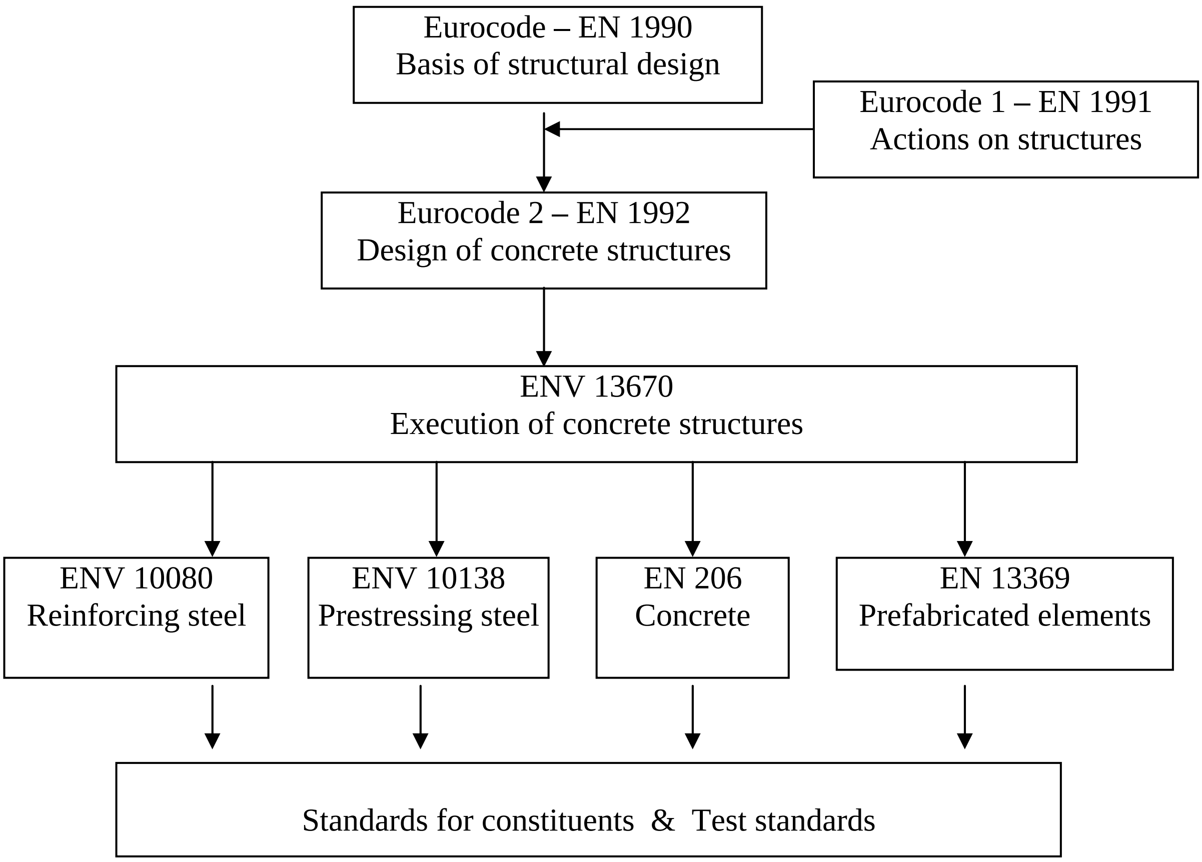
2. EN 1992 “Design of Concrete Structures” gives the material (concrete) specific “basis of design”

3. EN 1992 gives principles and application rules for the verification of the fulfillment of EN 1990 requirements

4. EN 1992 states that *“the design procedures are valid only when requirements for execution and workmanship given in ENV 13670 are also complied with”*

5. ENV 13670 “Execution of Concrete Structures” states that *“concrete shall be specified and produced according to EN 206” – “Reinforcing steel shall conform to EN 10080” – etc*

6. EN 206 states that *“general suitability is established for cement conforming to prEN 197” – “Air content shall be measured according to prEN 12350” - etc*



PARALLELISM IN STRUCTURAL DESIGN AND SERVICE LIFE DESIGN

EN 1990 defines a number of "limit states" defined as *"beyond which the structure no longer fulfils the relevant design criteria"*.

For this demonstration we will deal with the

- "Ultimate Limit State", ULS defined as *"associated with collapse or with other similar forms of structural failure"*
- "Serviceability Limit State", SLS defined *as "correspond to conditions beyond which specified service life requirements for a structure are no longer met"*

These limit states defines the "failure criteria" dealt with in the design.
The "failure criteria" for ULS is linked to structural resistance, while the end of SLS might be characterized by a "Design Service Life" (number of years).

The failure criteria for ULS are fairly well defined in the Eurocodes.

The failure criteria for SLS of a concrete structure should be quantified in EN 1992 (basis of design – material specific).
Such SLS criteria are however only described in a qualitative way not suited as a direct basis for probabilistic calculations.

Reliability management

Different levels of reliability may be adopted for

- structural resistance
- serviceability

The choice of levels of reliability for a particular structure shall take account of the relevant factors, including:

- the possible cause and/or mode of attaining a limit state
- possible consequences of failure in terms of risk of life, injury, potential economical losses
- public aversion to failure
- the expense and procedures necessary to reduce the risk of failure.

Reliability index β

The probability of failure P_f can be expressed through a performance function g such that a structure is considered to survive if $g > 0$ and to fail if $g < 0$.
If R is resistance and E the effect of actions, the performance function is: $g = R - E$
If g is normally distributed, the reliability index β is taken as:
β = (mean value of g)/(its standard distribution)

The relationship between P_f and β for this situation is given as

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}
β	1.28	2.32	3.09	3.72

Target values of reliability index β

From prEN 1990 and NS 3490, reliability indexes might be found for typical cases like

ULS (residual and office buildings where consequences of failure are medium)	β = 3.8 for 50 years reference period
SLS	β = in the range of 1.5 for 50 years reference period

Limit state design – Level A

Limit state design is to verify that the failure criteria (the limit state), is avoided with the required level of reliability.
For ULS, any well-documented procedure for this verification is in principle acceptable.

The verification might be performed by

- a full probabilistic method (level III),
- a First Order Reliability Method (level II) based on certain well-defined approximations, or
- a historical/empirical method.

A standardized verification according to principles and application rules in EN 1992 is however the standard case.

For SLS, also any well-documented verification is in principle acceptable. However, the lack of quantified failure criteria represents a problem.
To be able to perform any calculations, the SLS must be transformed into specific limit states including for instance

- a number of years (service life),
- the limit state itself, for instance 10 % of the surface reinforcement depassivated by carbonation
- a level of reliability to reach the design service life, for instance given by a reliability index

Since the SLS for concrete structures linked to durability must be connected to a possible deterioration process, such specific concrete-related SLS definitions should be found in EN 1992.
One way of performing the verification is simply to avoid the mechanism. This might be done by for instance not applying alkalie-reactive aggregate; to reduce the moisture level in freeze-thaw exposed structures or the use of stainless reinforcement.

All the principles for a probabilistic service life design to fulfill the overall SLS are thus included in the Eurocodes. However, they are not fully operative due to lack of specific failure criteria and corresponding required levels of reliability.

Needed progress to reach level A for service life design

To get the "Basis of design" documents (EN 1990 and the material specific part of EN 1992) operative for service life design, we need:

- to supplement the present qualitative descriptions of SLS with quantified limit states. To enable calculations, these must be linked to specific deterioration mechanisms.
- fine-tune the required levels of reliability for the different deterioration mechanisms depending on the consequences.

Limit state design – Level B

EN 1992 "Design of concrete structures". contains a consistent set of Principles and Application Rules.
The Principles comprise:

- General statements and definitions for which there is no alternative, as well as
- Requirements and analytical models for which no alternative is permitted

Examples of Principles from the ULS are for instance (§ 6.1 bending without axial force)

- *"Plane sections remain plane"*
- *"The tensile strength of the concrete is ignored"*
- *"The stresses in compression are derived from the design stress/strain given in the code"*

The Application Rules are generally recognized rules, which follow the Principles and satisfy their requirements.

An example of Application Rules from the ULS are for instance (§6.2.3 members requiring design shear reinforcement)

- *"The design of members with shear reinforcement is based on a truss model given in the code. Limiting values for the angle of the inclined struts in the web are given in code"*

Verification by the partial factor method

EN 1992 "Design of concrete structures", is based on the partial factor method.
This method is based on given design values for load and resistance combined with a set of partial factors. The design values are either obtained by using characteristic or representative values.
The whole procedure is calibrated to fulfill the required level of reliability in EN 1990.

Based on the "authorized" Principles and Application Rules (though being approximations) and the calibrated set of partial factors, the verification for structural capacity can be carried out according to the standardized procedures in EN 1992.

Verification of the design service life

EN 1992 gives also provisions for the verification of service life (SLS) in § 4 “Durability and cover to reinforcement”. These are, in contrast to the structural parts of the standard, based on “deemed to satisfy” requirements to cover, crack width and concrete composition expected to withstand 18 different classes of environmental conditions.

Although there must be some underlying probabilistic principles behind the provisions given, these are not transparent for the reader.
In contrast to the structural parts, the user is not provided with Principles enabling calculations nor partial factors enabling probabilistic design.

A possible future generation of EN 1990 might incorporate general “authorized” models for service life design. These Principles might according to the DuraNet/CEN workshop in Berlin in 1999 be approximations of deterioration mechanisms like:

- Carbonation given by the expression: $X = k * \sqrt{t}$
- Chloride ingress in a marine environment given by the expression: $C(x, t) = C_i + (C_s - C_i)\mathrm{erfc}\left(\frac{x}{\sqrt{4tD\,(t_o/t)^{\alpha}}}\right)$

Such Principles might, like those applied in structural design, be calibrated according to the principles given in EN 1990 with defined design values for material resistance and environmental load as well as partial factors to achieve the required level of reliability given in EN 1990.
As for structural design, model uncertainties might be incorporated in the partial factors.

Execution aspects

A proper execution is vital to for the structure’s infield performance.
EN 1990 already includes 3 classes for “Inspection Levels”. The standard opens for reduction of partial factors for materials or product property if an increased level of inspection is applied.
ENV 13670 has already taken onboard these 3 classes of inspection, which might be an input to the verification process.
Other aspects in ENV 13670 that might be developed to accommodate service life design calculations might be geometrical tolerances, in particular for placing of reinforcement, and classes of curing.

Needed progress to reach level B for service life design

To get EN 1992, ENV 13670 and EN 206 operative for probabilistic service life verification, we need in addition to those elements listed for level A:

- Consensus on some mathematical models for deterioration mechanisms. Depending on the accuracy of these design expressions, a factor for model uncertainty has to be considered.
- Test methods to derive the material parameters and to map the environmental load
- Defined design values based either on characteristic values or nominal values to support the models

- Sufficient experimental and empirical data to calibrate a consistent set of partial factors and design values to meet the required level of reliability.

Limit state design – Level C

A possible third level for verification might be to keep the present format of "deemed to satisfy" requirements in EN 1992, ENV 13670 and EN 206, but to justify them by a probabilistic based verification. The lack of transparent reasoning behind the choices taken in the different CEN member countries for material composition and cover for a more or less similar environmental action like carbonation has resulted in proposals for w/c requirements ranging from 0.60 to 0.45, the cover being similar. Most of these choices are based on infield experience combined with the well-known probabilistic method named "stomach-feeling".

As will be presented at this workshop, Norway is trying to digest our in-field experience with the present concrete structures through a probabilistic procedure according to the principles given in EN 1990. We hope that this exercise will give us a support to the decisions we have to take in our design code, NS 3473 and national annex to EN 206.
In this process we must keep in mind the economical consequences for the society by being too liberal, and the reduced competitiveness for concrete as a building material if we are too restrictive.

Limit state design – Level D

The last, and least ambitious scenario for the application of probabilistic service life design might be to rely on the present system with "deemed to satisfy" requirements and hope that the wisdom of the code-writers ensures the required level of reliability for the given limiting values for material composition, cover and crack width.
To achieve some flexibility, the code-writers might authorize some benchmarking laboratory tests.
In the informative annex "F" to EN 206, such a provision is already given for freeze-thaw exposed concrete in the footnote 1 "Where the concrete is not air entrained, the performance of concrete should be tested according to prEN FFF-1 in comparison with a concrete for which freeze/thaw resistance for the relevant exposure class is proven".
Such a benchmarking approach needs however a sound correlation between the exposure test and the real long term infield behavior of the structure.

HOW TO INCORPORATE PROBABILISTIC SERVICE LIFE DESIGN IN THE CEN-STANDARDS

The needed elements to get the CEN-standards operative on probabilistic service life design must be anchored in EN 1990, EN 1992, ENV 13670, EN 206 and some test standards.
The present problem is that the revisions of these documents are not in phase. It is somewhat late to influence the coming versions of EN 1990 and EN 1992.

One idea might therefore be to involve the relevant CEN TCs and SCs to produce a CEN Technical handbook comprising all the elements in a consistent way.
Having in mind the somewhat limited practical experience with such a technology, this document might in an introduction period have an informative character and enabling the main standards to import “their” elements during the coming revisions.

Already today we have an ad-hoc committee chaired by CEN TC-104/SC1 (Reinhardt) working on the subject.
This group does today comprise the chairman of TC-250/SC2 (EN 1992) Litzner and the chairman of CEN TC-104/SC2 (ENV 13670) Helland as well as representatives from prenormative bodies like fib and RILEM.

DuraNet

Third Workshop, Tromsø 10th – 12th June 2001
On the theme:

"Service Life Design of Concrete Structures
–
From Theory to Standardisation"

CARBONATION

A probabilistic approach to derive provisions for EN 206-1

Magne Maage and Sverre Smeplass
Selmer Skanska AS
Trondheim, Norway

Introduction

In most existing standards and guidelines, durability of reinforced concrete structures is covered by prescriptive requirements. For carbonation initiated corrosion, this may include requirements on w/c-ratio, cement type and content, compressive strength and concrete cover. Specifications are given as limiting values except for concrete cover, which are given as nominal and minimum values depending on the exposure class.

Future requirements will probably be performance requirements defined according to a probabilistic approach. However, today we don't have the appropriate information on the correlation between carbonation rate and factors like w/c-ratio, cement type and content and environmental situation, especially air humidity and CO_2 *concentration.*

Therefore, the requirements in EN 206-1 are still prescriptive. However, the detailed specifications within the code and the corresponding national documents may be based on a probabilistic approach.

The scope of this paper is to present a probabilistic approach to the process of defining prescriptive requirements for carbonation initiated reinforcement corrosion. The approach is based on the philosophy given in prEN 1990 (Ref. 1), which means that the "failure" reliability Z is calculated as the difference between a resistance against "failure" R (e.g. concrete cover) and an environmental load or action F (e.g. time dependent carbonation depth). Both resistance and load are expressed in a probabilistic way.
Practical examples based on measured carbonation depths and concrete cover in existing structures are presented.

The philosophy presented in this paper may be further developed to a method for probability based durability design for carbonation initiated reinforcement corrosion. However, this is not within the scope of this presentation. Such a method is presented in (Ref. 15).

2. Principle approach for probability design

The "failure" reliability Z (a limit state function) is calculated as the difference between a resistance against "failure", R, (concrete cover) and an environmental load or action, F (time dependent carbonation depth). Failure will in this situation be defined as the end of the initiation period which means that "failure" occur when the largest carbonation depths are equal to the smallest concrete covers.

The probability of failure, p_f*, has to be defined as a maximum target probability,* p_{target}*, depending on safety philosophy. This can be expressed by the equation*

$$p_f = p\{Z = R - F < 0\} < p_{target} \qquad (1)$$

When the functions R and F are normally distributed, Z also is normally distributed.
When the resistance R is normally distributed, it has an average value μ_R *and a standard deviation* σ_R *independent of time. In most other situations the resistance will be time dependent.*

When the load F is normally distributed, it has an average value μ_F *and a standard deviation* σ_F*. Both the F characteristics are increasing with time.*

The failure reliability Z is defined by the limit state function

$$Z = R - F \qquad (2)$$

When normally distributed, the function Z has an average value

$$\mu_Z = \mu_R - \mu_F \qquad (3)$$

and a standard deviation

$$\sigma_Z = (\sigma_R^2 + \sigma_F^2)^{1/2} \qquad (4)$$

The average value μ_Z is reduced and the standard deviation σ_Z increased with time, which means that the probability of failure is increasing with time. In this situation (normal distribution), the failure probability may be expressed by

$$p_f = \Phi(-\mu_Z/\sigma_Z) = \Phi(-\beta) \qquad (5)$$

where β is the so-called reliability index covering safety, serviceability and durability (prEN 1990, clause 1.5.2.17). In a design situation, the calculated β has to be greater than a required reliability level β_0 depending on the safety level.

The relation between failure reliability, resistance and load is demonstrated in Fig. 1.

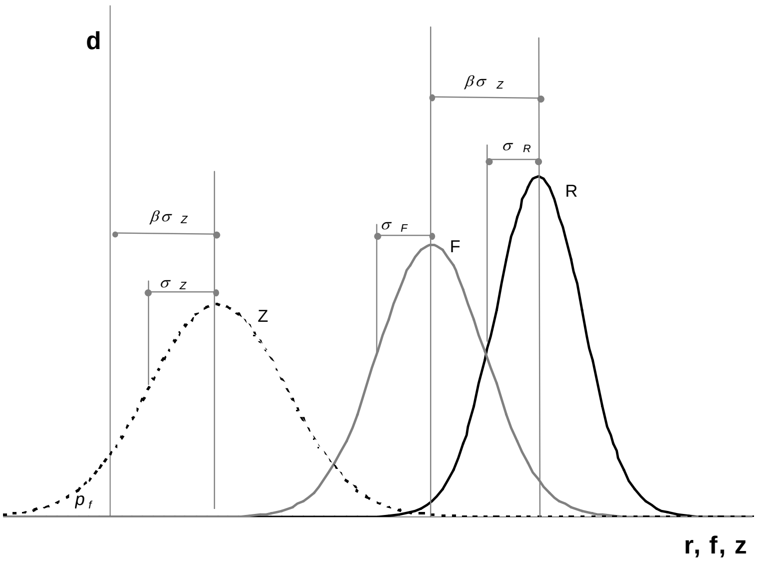


Fig. 1 Relation between failure reliability, Z, resistance, R and load (action), F.

The relation between the reliability index β and the probability of failure p_f, when normal distribution applies, is shown in Table 1.

Table 1. Relation between reliability index β and the probability of failure p_f.

p_f	10^{-1}	$0.668\ 10^{-1}$	$0.359\ 10^{-1}$	$0.227\ 10^{-1}$	10^{-2}	10^{-3}	10^{-7}
β	1.28	1.50	1.80	2.00	2.32	3.09	5.20

According to prEN 1990, the highest value of β (lowest probability of failure) is required when the consequences of failure are high and the reference period for the load is short. Typical examples are failures due to accidents in public structures.

Serviceability limit states (SLS) are applied when the "failure" leads only to economic consequences. This is typical for durability situations where the deterioration will be visible long before any risk of collapse is reached.

This paper is focusing on typical SLS design where the target is to restrict the probability of corrosion initiation due to carbonation. After initiation, it may take many years before possible corrosion results in loss of serviceability or loss of structural safety. Additionally, the reference period for the load is equal to the service life, resulting in an even lower requirement to the limiting β value.

From this point of view, the probability of corrosion initiation due to carbonation may be set as high as 10^{-1}, corresponding to a reliability index β of approximately 1.30. This is a lower β value than proposed in (Ref. 14), where β=0.5 for XC1, β=1.5 for XC2 and XC3, and β=2.0 for XC4.

For corrosion initiation due to chlorides, a somewhat lower probability and a corresponding higher β should be chosen due to the fact that chloride initiated corrosion may result in a faster and often more local corrosion process than in the case where carbonation is initiating the corrosion.

This philosophy is in agreement with prEN 1990, clause C6, but a β equal to 1.30 is somewhat lower than indicated in the informative clause C6 for Reliability Class RC2 structural members. No β values are given for Reliability Class RC1 in prEN 1990 for SLS, but the values should be somewhat lower than for class RC2. A reliability index β of approximately 1.30 should therefore be reasonable for this situation.

4. Durability design

Durability design according to DURACRETE

A design method for new structures is given in chapter 10.3 in (Ref. 15). The design equation, g, stating that corrosion is initiated when the carbonation front reaches the reinforcement is given by:

$$g = x - x_c(t) \tag{6}$$

where x is cover thickness and x_c is penetration depth of the carbonation.

The characteristic value for the cover thickness is defined as the mean value or the nominal value determined through the design process.

The penetration depth $x_c(t)$ is given by:

$$x_c(t) = (2 \cdot c_{s,ca} \cdot D_{ca} \cdot t)^{0.5} \qquad (7)$$

where $c_{s,ca}$ is surface concentration, D_{ca} is the effective coefficient with respect to carbonation and t is time.

D_{ca} is given by:

$$D_{ca} = D_{ca,0} \cdot k_{e,ca} \cdot k_{c,ca} \cdot (t_0/t)^{2 \cdot nca} \cdot \gamma_{Dca} \qquad (8)$$

where $D_{ca,0}$ is diffusion coefficient with respect to carbonation determined on the basis of compliance tests, $k_{e,ca}$ is environmental factor, $k_{c,ca}$ is curing factor, t_0 is the age of the concrete when the compliance test is performed, n_{ca} is aging factor and γ_{Dca} is partial factor for the diffusion coefficient with respect to carbonation.

Numbers for the different factors are given in (Ref. 15) and will not be repeated here.

The model used at existing structures

The model may be used also when carbonation depths are measured after a number of years of exposure in existing structures. However, in this situation the model may be simplified because most of the factors are included in the measured carbonation depths.

In Eq. 7 and 8, all parameters except from time, t, and the aging effect, $(t_0/t)^{2 \cdot nca}$, are included in the measured carbonation depths. The aging factor, n_{ca}, is in (Ref. 15) estimated to be in the range of 0 – 0,16, depending on type of cement and environmental situation. For average relative humidity around 65 %, n_{ca} is estimated to be 0, which means that no aging effect is present. Even with an aging factor up to 0,16, the aging effect will be low, especially when carbonation depths are measured after many years of exposure. The aging effect will therefore be neglected in the following calculations. This is a conservative simplification. Based on these assumptions, Eq. 6 may be written as:

$$g = x - C_{REF} \cdot (t)^{0,5} \qquad (6a)$$

where C_{REF} includes all the parameters given in Eq. 7 and 8.

5. Failure probability in an existing structure.

This example covers the simplest possible situation. An existing structure is examined after t1 years of exposure. The examination includes two types of measurements, the carbonation depth and the concrete cover.

Measurements of carbonation depth results in an average value μ_{CAt1} and a standard deviation σ_{CAt1}.

Measurements of concrete cover results in an average value μ_{CC} and a standard deviation σ_{CC}, which of course are independent of time.

The progression model for carbonation development is given by the following simple equation, see also Eq. 6a:

$$CA_t = CA_{REF} (t)^{0.5} \qquad (9)$$

where CA_t *is carbonation depth at time t and* CA_{REF} *is a constant depending on materials and exposure climate (air humidity and* CO_2 *concentration). The standard deviation for carbonation depth is assumed to be proportional to the average carbonation depth, meaning that the coefficient of variation is constant. Based on this and the assumption that the measurements are normally distributed, the reliability index may be calculated for different exposure times.*

Example

Carbonation depths are measured in a structure after 10 years of exposure. Average and standard deviation are calculated to be: $\mu_{CA10} = 9.0$ *mm and* $\sigma_{CA10} = 3.2$ *mm.*

Concrete covers are measured. Average and standard deviation are calculated to be:
$\mu_{CC} = 27.0$ *mm and* $\sigma_{CC} = 4.2$ *mm.*

The reliability index after 50 years of exposure, corresponding to the specified lifetime of the structure, is calculated as follows:

The average carbonation depth μ_{CA50} *at 50 years is calculated by Eq. 9:*

$$\mu_{CA50} = \mu_{CA10} (50 / 10)^{0.5} = 9.0 (5)^{0.5} = 20.1 \text{ mm}$$

The standard deviation for carbonation at 50 years, σ_{CA50}*, has to be increased proportionally to the average carbonation increase from 10 to 50 years:*

$$\sigma_{CA50} = \sigma_{CA10} (\mu_{CA50} / \mu_{CA10}) = 3.2 (20.1 / 9.0) = 7.1 \text{ mm}$$

The failure reliability function, Z, after 50 years exposure has an average μ_{Z50} *and standard deviation* σ_{Z50} *calculated by Eq. 3 and 4 at 50 years:*

$$\mu_{Z50} = \mu_R - \mu_F = \mu_{CC} - \mu_{CA50} = 27.0 – 20.1 = 6.9 \text{ mm}$$

$$\sigma_{Z50} = (\sigma_R^2 + \sigma_F^2)^{0.5} = (\sigma_{CC}^2 + \sigma_{CA50}^2)^{0.5} = (4.2^2 + 7.1^2)^{0.5} = 8.2 \text{ mm}$$

The reliability index at 50 years, β_{50}, is calculated by Eq. 5:

$$\beta_{50} = \mu_{Z50} / \sigma_{Z50} = 6.9 / 8.2 = 0.84$$

If the evaluation is based on average values for carbonation depth and concrete cover, the service life is found to be longer than 50 years, $\mu_{CA50} < \mu_{CC}$. However, if the evaluation is based on a probabilistic approach, assuming 10% risk of carbonation initiated reinforcement corrosion (β = 1.28 according to Table 1), the requirement of 50 years service life is not met.

6. Failure probability when concrete cover is in agreement with the standards and carbonation values are based on concretes with various w/c-ratios

Concrete cover

Rules for concrete cover are given in prEN 1992-1: 2nd draft January 2001(Ref. 2), Table 4.3 and ENV 13670-1: 1999 (Ref. 3), clauses 6.6 and 10.6. Both will be revised in a few years.

In prEN 1992-1, Table 4.3, the minimum required concrete cover c_{min} for exposure classes XC2 and XC3, corrosion induced by carbonation, is 25 mm. This is valid for normal weight concrete, normal reinforcement and 50 years service life. The minimum cover for a service life of 100 years should be increased by 10 mm.

The nominal cover c_n, defined as the sum of the minimum cover c_{min} and a "minus-deviation", is equal to the average target concrete cover (the dimension of the chairs to be used). The "minus-deviation", $\Delta_{(minus)}$ is, according to ENV 13670-1, equal to 10 mm, which means that the required nominal cover is 35 mm. (The "plus-deviations" given in ENV 13670-1 are of no interest here). These numbers for c_{min} and $\Delta_{(minus)}$ are also in agreement with the Norwegian Standards NS 3473 (Ref. 4), clause 17.1.8 and NS 3420 (Ref. 5), clause L2, d1).

From a statistical point of view, the minimum cover has to be associated with a probability of "failure", meaning that a certain percentage of the reinforcement has a smaller cover than the required minimum. A reasonable assumption may be that 5% of the reinforcement has a lower cover than the required minimum. Assuming that the concrete cover is normally distributed, the distance from the mean value to the 5% percentile is 1.645 times the standard deviation, giving the standard deviation σ_{CC} = 10 / 1.645 = 6.1 mm.

In the calculations in chapter 7, the average concrete cover, μ_{CC}, will be set equal to 35 mm and the standard deviation, σ_{CC}, equal to 6.1 mm.

Carbonation depth

Information on carbonation depths may be taken from different sources, e.g. by examination of existing structures or from in field R&D programmes. The data will vary depending on concrete compositions, cement types, content of pozzolanic materials, age and environmental situations. In order to adjust the measurements to a "normalised" situation, some rules for such adjustments have to be agreed within "concrete families", i.e. groups of concrete types.

In this presentation the adjustments due to varying w/c-ratio and age will be included. Other variables like cement type, type and quantity of pozzolans and environmental situation (exposure *class) will be regarded as different "concrete families". Adjustments have to be carried out within each "concrete family".*

Within each concrete family, the measured carbonation depths will be normalised to a w/c-ratio equal to 0.60 (other w/c-ratios may be chosen) according to Eq. 10, developed on the basis of information from ref. 6, 7, 8, 9 and 10:

$$CA_{0.6,t1} = CA_{w/c1,t1} \cdot (2.5 - 2.5 \cdot w/c1) \qquad (10)$$

Where

- $CA_{0.6,t1}$ *is the carbonation dept at w/c-ratio equal to 0.60 and concrete age t1*
- $CA_{w/c1,t1}$ *is the measured carbonation depth in a concrete with w/c-ratio equal to w/c1 at age t1.*

Eq. 10 is an approximation based on test data and accurate enough for w/c-ratios between 0.50 and 0.70.

Based on the $CA_{0.6,t1}$ *values, the average carbonation depth* $\mu_{0.6,t1}$ *and the standard deviation* $\sigma_{0.6,t1}$ *may be calculated for age t1 and w/c-ratio equal to 0.6.*

The next adjustment will be for age. Service life is defined as 50 years and the adjustment is carried out according to Eq. 9:

$$CA_{0.6,50} = CA_{0.6,t1}\,(50 / t1)^{0.5} \qquad (11)$$

where

- $CA_{0.6,50}$ *is the carbonation depth at age 50 years and w/c-ratio equal to 0.60*

Based on the $CA_{0.6,50}$ *values, the average carbonation depth* $\mu_{0.6,50}$ *and the standard deviation* $\sigma_{0.6,50}$ *may be calculated for age 50 years and w/c-ratio equal to 0.6.*

Uncertainty on w/c-ratio

An individual uncertainty for the w/c-ratio should be introduced for the data from each source. This uncertainty should be higher when the data come from an existing structure compared to a situation where the data come from an in field R&D programme. In this presentation a standard deviation of 0.02 and 0.05 are proposed and will be used for the w/c-ratio for data from in field R&D programmes and for data from existing structures, respectively. The uncertainty of the w/c-ratio, represented by standard deviations, will be included in the calculations of the reliability index.

Calculation of the reliability index $\boldsymbol{\beta}$

Several methods may be used for reliability calculations as described in (Ref. 11):

- *Monte Carlo*
- *Analytical integration*
- *Numerical integration*
- *Numerical approximation (FORM)*

In this presentation, only the FORM method will be used. FORM stands for First Order Reliability Method. The simplest of the FORM methods is the "mean value approach", which can be done by hand. The first step is to linearise the reliability function Z to the form:

$$Z \approx g(X) = a_0 + a_1X_1 + a_2X_2 + ...+ a_nX_n \tag{12}$$

Details of how to do this are not included here, see (Ref. 11).
The average value μ_z is given by

$$\mu_z = g(\mu_{x1}, \mu_{x2}, \mu_{x3},..... \mu_{xn}) \tag{13}$$

and the standard deviation σ_z is given by:

$$\sigma_z^2 = (\partial g/\partial x_1 \cdot \sigma_{x1}^2) + (\partial g/\partial x_2 \cdot \sigma_{x2}^2) + ... + (\partial g/\partial x_n \cdot \sigma_{xn}^2) \tag{14}$$

The reliability index $\beta = \mu_z / \sigma_z$ can then be calculated.

Example

The probability of failure will be calculated for the same example as given in chapter 5 with the addition that also the w/c-ratio is a variable. The w/c-ratio is found to be 0.55, and the purpose of the calculation below is to see if a concrete with w/c-ratio equal to 0.60 is good enough to fulfil the requirements for 50 years service life. In Table 2, the different variables are listed.

Table 2. List of basic variables in the calculation example

X_i	Description	Distribution	$\mu(X_i)$	$\sigma(X_i)$
CC	Concrete cover	Normal	27.0 mm	4.2 mm
$CA_{w/c1,t1}$	Carbonation depth at age t1	Normal	9.0 mm	3.2 mm
w/c1	w/c-ratio	Normal	0.55	0.02

The resistance, R, is defined to be the concrete cover R = CC

The load (action), F, is given by $F = CA_{w/c1,t1} (2.5 - 2.5 \cdot w/c1) \cdot (50/t1)^{0.5}$

The reliability function, Z, is given by Z = R – F:

$$Z = CC - CA_{w/c1,t1} (2.5 - 2.5 \cdot w/c1) \cdot (50/t1)^{0.5}$$

The Z function is not linear, but will be used as it is since the variation of the w/c1 ratio is relatively small. This approximation is good enough for $\sigma_{w/c1}$ up to at least 10% of $\mu_{w/c1}$.

The average of the reliability function is calculated according to Eq. 13:

$$\mu_z = \mu_{CC} - \mu_{CAw/c1,t1} \cdot (2.5 - 2.5 \cdot \mu_{w/c1}) \cdot (50/t1)^{0.5} = 27 - 9 \cdot (2.5 - 2.5 \cdot 0.55) \cdot (50/10)^{0.5}$$
$$= 27.0 - 22.6 = 4.4 \text{ mm}$$

The standard deviation of the reliability function is calculated according to Eq. 14:

$$\sigma_z^2 = (\sigma_{CC} \cdot \partial z/\partial_{CC})^2 + (\sigma_{CAw/c1,t1} \cdot \partial z/\partial_{CAw/c1,t1})^2 + (\sigma_{w/c1} \cdot \partial z/\partial_{w/c1})^2$$

$$= (1 \cdot \sigma_{CC})^2 + (-1(2.5 - 2.5 \cdot w/c1) \cdot (50/t1)^{0.5} \cdot \sigma_{CAw/c1,t1})^2 + (-2.5 \cdot (50/t1)^{0.5} \cdot CA_{w/c1,t1} \cdot 1 \cdot \sigma_{w/c1,t1})^2$$

$$= 4.2^2 + [(5)^{0.5} \cdot (2.5 - 2.5 \cdot 0.55) \cdot 3.2]^2 + [2.5 \cdot (5)^{0.5} \cdot 9 \cdot 0.02]^2 = 83.4$$

$$\sigma_z^2 = 83.4 \quad \Rightarrow \quad \sigma_z = 9.1 \text{ mm}$$

The reliability index $\beta = \mu_z / \sigma_z$ can then be calculated:

$$\beta = \mu_z / \sigma_z = 4.4 / 9.1 = 0.48$$

Compared to the example in chapter 5, both the adjustment of carbonation depth due to w/c-ratio different from 0.60 (lower) and the introduction of a variation of the w/c-ratio, result in a reduced β value.

7. Reliability indexes based on concrete cover requirements in the standards and real carbonation depths for different concrete families

7.1 In situ testing

Carbonation depths have been measured in existing structures located in Oslo, Bergen, Trondheim and Tromsø in Norway. The measurements were carried out during the summer 2000 by two students (Ref. 16).

The age of the structures was from approximately 7 to 13 years. All structures had a w/b-ratio of approximately 0,60 according to the requirements in the Norwegian Standard NS 3420 (1986) for Moderate Exposure Class (“Noe aggressivt”), valid when the structures were built.

Carbonation was measured at locations on the structures classified as XC3 and XC4 according to EN 206-1 (ref. 12). However, the distinction between the two was sometimes difficult.

The cement type used was CEM I in all concretes. Silica fume was used in all concretes tested in Bergen and Trondheim. None of the concretes tested in Oslo and Tromsø included silica fume.

The data for each structure are given in Appendixes 1 and 2. The measured values are normalised to w/b ratio equal to 0.60 and to an exposure age of 50 ad 100 years according to Eq. 10 and Eq. 11 respectively. These values are chosen because we want to control if a concrete quality of w/b ratio equal to 0.60 with CEM I is good enough at service lives of 50 and 100 years with the chosen nominal concrete cover (35 and 45 mm). The normalised values for each structure are also listed in Appendix 1 for 50 years service life and in Appendix 2 for 100 years service life.

It may be discussed how the standard deviation shall be calculated. In this presentation the standard deviation is calculated within each concrete family (with and without silica fume) and exposure classes (XC3 and XC4) based on the average normalised carbonation depth within a location, see Appendixes 1 and 2. This is supposed to give the best expression of the variation within an exposure class. The coefficient of variation is calculated for each concrete family. This coefficient is varying somewhat, most probable due to a limited number of results. To be more general in the calculations, the coefficient of variation is set equal to 40 % for all concrete families. This is an average number based on the data given in Appendixes 1 and 2. A coefficient of variation of 40 % is relatively high due to the fact that the climatic variation within an exposure class may vary quite a lot.

The number of data for exposure class XC3 and concrete with silica fume is so limited (2) that no reliable calculations can be carried out. However, some estimations will be done based on the increased carbonation depth for concrete with silica fume compared to the concrete without silica fume within exposure class XC4, see Tables 3 and 5.

The concrete in exposure class XC3 is expected to have a faster carbonation rate than the same concrete exposed to class XC4 due to dryer conditions in XC3. However, the corrosion rate is expected to be lower for concrete in class XC3 than in class XC4 due to the same effect. For this reason, it is most probable that the Norwegian national requirements will be the same for the two classes XC3 and XC4 just like the proposals in the Netherlands, Denmark and United Kingdom.

Service life of 50 years

The results of the calculations from Appendix 1 are given in Table 3.

Table 3. Average carbonation and standard deviation normalised to w/b = 0.60 and 50 years of exposure measured on existing structures. See Appendix 1.

Concrete Family		**Exposure class**	
		XC3	**XC4**
Concrete without silica fume	Average (mm)	15.9 (4)*	12.7 (7)*
	Std. dev. (mm)	7.0 (45%)	4.3 (34%)
Concrete with silica fume	Average (mm)	*18.8 (est)*	15.0 (12)*
	Std. dev. (mm)	*7.5 (est)*	6.1 (41%)

* Number of data

For exposure classes XC4 and XC3, the reliability index β is calculated based on the method given in chapter 6. The input parameters in the calculations and the results are given in Table 4. Carbonation depths are as given in Appendix 1, but standard deviations are calculated based on a coefficient of variation equal to 40 %.

Table 4. Input parameters and calculated results for exposure classes XC3 and XC4

Concrete family		**X_i**	**Description**	**$\mu(X_i)$**	**$\sigma(X_i)$**	**β**
Concrete without silica fume	XC3	CC	Concrete cover	35.0 mm	6.1 mm	2.11
		$CA_{0.6,50}$	Carbonation depth at 50 years	15.9 mm	6.4 mm	
		w/b1	W/b-ratio	0.60	0.05	

Concrete with silica fume		CC	Concrete cover	35.0 mm	6.1 mm	1.63
		$CA_{0.6,50}$	Carbonation depth at 50 years	*18.8* mm	*7.5* mm	
		w/b1	W/b-ratio	0.60	0.05	
Concrete without silica fume	XC4	CC	Concrete cover	35.0 mm	6.1 mm	2.75
		$CA_{0.6,50}$	Carbonation depth at 50 years	12.7 mm	5.1 mm	
		w/b1	W/b-ratio	0.60	0.05	
Concrete with silica fume		CC	Concrete cover	35.0 mm	6.1 mm	2.28
		$CA_{0.6,50}$	Carbonation depth at 50 years	15.0 mm	6.0 mm	
		w/b1	W/b-ratio	0.60	0.05	

Based on the assumptions on concrete cover given in chapter 6 (standard requirements) and measured carbonation depths in existing structures exposed to classes XC3 and XC4 with and without silica fume, the reliability index β is higher than 1.3 for all these concrete families with w/b-ratio 0.60. This means that for concretes based on CEM I cement with w/b-ratio 0.60, the risk of corrosion initiation will be less than 10%, both with and without silica fume in the concrete the first 50 years.

Service life of 100 years

The results of the calculations from Appendix 2 are given in Table 5.

Table 5. Average carbonation and standard deviation normalised to w/b = 0.60 and 100 years of exposure measured on existing structures. See Appendix 2.

Concrete Family		**Exposure class**	
		XC3	**XC4**
Concrete without silica fume	Average (mm)	22.5 (4)*	18.0 (7)*
	Std. dev. (mm)	10.0 (45%)	6.0 (34%)
Concrete with silica fume	Average (mm)	*26.5 (est)*	21.2 (12)*
	Std. dev. (mm)	*10.6 (est)*	8.6 (41%)

* Number of data

For exposure classes XC4 and XC3, the reliability index β is calculated based on the method given in chapter 6. The nominal concrete cover is increased by 10 mm to 45 mm compared to service life of 50 years. The standard deviation will not be changed since the tolerance is the same. The input parameters in the calculations and the results are given in Table 6. Carbonation depths are as given in Appendix 2, but standard deviations are calculated based on a coefficient of variation equal to 40 %.

Based on the assumptions on concrete cover given in chapter 6 (standard requirements), except that the nominal concrete cover is increased by 10 mm, and measured carbonation depths in existing structures exposed to classes XC3 and XC4 with and without silica fume, the reliability index β is higher than 1.3 for all these concrete families with w/b-ratio 0.60. This means that for concretes based on CEM I cement with w/b-ratio 0.60, the risk of corrosion initiation will be less than 10%, both with and without silica fume in the concrete the first 100 years.

Table 6. Input parameters and calculated results for exposure classes XC3 and XC4

Concrete family		**X_i**	**Description**	**$\mu(X_i)$**	**$\sigma(X_i)$**	**β**
Concrete without silica fume	XC3	CC	Concrete cover	45.0 mm	6.1 mm	2.00
		$CA_{0.6,50}$	Carbonation depth at 100 years	22.5 mm	9.0 mm	
		w/c1	W/b-ratio	0.60	0.05	
Concrete with silica fume		CC	Concrete cover	45.0 mm	6.1 mm	1.46
		$CA_{0.6,50}$	Carbonation depth at 100 years	*26.5* mm	*10.6* mm	
		w/c1	W/b-ratio	0.60	0.05	
Concrete without silica fume	XC4	CC	Concrete cover	45.0 mm	6.1 mm	2.78
		$CA_{0.6,50}$	Carbonation depth at 100 years	18.0 mm	7.2 mm	
		w/c1	W/b-ratio	0.60	0.05	
Concrete with silica fume		CC	Concrete cover	45.0 mm	6.1 mm	2.21
		$CA_{0.6,50}$	Carbonation depth at 100 years	21.2 mm	8.5 mm	
		w/c1	W/b-ratio	0.60	0.05	

7.2 Field exposure R&D programme

Within a research programme started in the mid 1980-ties (Ref. 13), concrete samples with different cement types, silica fume and w/b-ratios were produced in laboratory. The samples were exposed to different conditions with the scope of measure carbonation depths depending on materials combination and exposure conditions.

Results from this research programme will not be included here because such relatively small samples (10 x 10 cm) are not representative for a real structure. A programme like this will primarily give results on the effect of the different variables relative to each other.

7.3 General comments and conclusions

The results from testing existing structures show that concrete with silica fume has somewhat faster carbonation than concrete without silica fume. This is in agreement with earlier reported results when comparison is based on w/b-ratio. When comparison is based on the same strength, the carbonation rate seems to be approximately the same for the two types of concrete.

Results from existing structures are more realistic than results from samples with small cross sections. Hence, only calculations based on results from existing structures are included in this report.

The results show that the requirements for concrete composition exposed to classes XC3 and XC4, according to EN 206-1, and cover requirements according to prEN 1992-1, will be as they are today in Norway for a planned service life of 50 years. This means that the w/b-ratio should not exceed 0.60 for concrete with CEM I cement both with and without silica fume.

For a planned service life of 100 years, the concrete cover should be increased by 10 mm.

Based on this, the Norwegian Council for Building Standardization is proposing the following requirements for concrete composition and concrete cover exposed to carbonation when using cement CEMI both with and without silica fume:

Exposure class (EN 206-1)		**Planned service life**			
		50 years		**100 years**	
		XC1	XC2, XC3, XC4	XC1	XC2, XC3, XC4
Concrete cover	Nominal	Combined with Class X0	35 mm	Combined with Class X0	45 mm
	Tolerance		10 mm		10 mm
	Minimum		25 mm		35 mm
W/b-ratio	Maximum		0.60		0.60

References:

1. Draft prEN 1990. “Eurocode: Basis of Structural Design.” CEN, January 2001.

2. prEN 1992-1 (2nd draft) “Eurocode 2: Design of concrete structures – Part 1: General rules and rules for buildings.” CEN, January 2001.

3. ENV 13670-1 “Execution of concrete structures – Part 1: Common rules.” CEN, 1999.

4. NS 3473: 1998 “Prosjektering av betongkonstruksjoner. Beregnings- og konstruksjonsregler.” NSF / NBR, Oslo 1998.

5. NS 3420-L: 1999 “Beskrivelsestekster for bygg, anlegg, installasjoner – Del L: Betong.” NSF / NBR, Oslo 1999.

6. Maage and Skjølsvold: “Karbonatisering av betong med og uten silikastøv.” SINTEF Rapport STF65 A83090, 1983.

7. Mayer, Wierig and Husmann: “Karbonatisierung von Schwerbeton.” Deutscher Ausschuss für Stahlbeton, Heft 182, Berlin 1967.

8. Nishi: “Outline of the studies in Japan regarding to the neutralization of alkali (or carbonation) of concrete.” RILEM Symposium, Durability of Concrete, Field Report, Prag, 1962.

9. Kishitani: “Dauerhaftigkeit von stahlbeton.” (in Japanese), Dissertation Universität Tokyo 1962. Deutscher Auszug in Zement-Kalk-gips, 17, 1964.

10. Schiessl: “Zur frage der zulässigen rissbreite und der erforderlichen betondeckung im stahlbetonbau unter besonderer berücksichtigung der karbonatisierung des betons.” Deutscher Ausschuss für Stahlbeton, Heft 255, Berlin 1976.

11. Siemes, Vrouwenvelder and Faber: “Probabilistic Methods for Durability Design.” DuraCrete. Project BE95-1347, Document BE95-1347/R0, January 1999.

12. EN 206-1: “Concrete – Part 1: Specification, performance, production and conformity.” CEN, 2000.

13. Maage, Gautefall, Havdal and Rønne: “Betongens funksjonsdyktighet – Karbonatisering. Delrapport 23: Eksponering av betongprøver. Program og blandinger.” SINTEF Rapport STF65 A88086, 1988.

14. K. Zilch and A. Schiessl: “The Determination of the Nominal Concrete Cover by Durability Design.” Background document to chapter 4 in EN 1992-1-1, February 2001.

15. L. J. Ebels, J. t. Haar and F. Tolman: “Durability Design of Concrete Pavements.” DuraCrete. Project BE95-1347, Document BE95-1347/R14B, May 2000.

16. K. E. Gerhardsen and M. A. Helland: In situ measurements of carbonation on existing structures in Oslo, Bergen, Trondheim and Tromsø during summer 2000

DuraNet

Third Workshop, Tromsø 10th – 12th June 2001
On the theme:

"Service Life Design of Concrete Structures
–
From Theory to Standardisation"

Marine chlorides

A probabilistic approach to derive provisions for EN 206-1

Finn Fluge
Directorate of Public Roads
Road Research Laboratory
Oslo, Norway

Introduction

In June 2000 the new European Standard EN 206-1 “Concrete – Part 1: Specification, performance, production and conformity” was launched.
To get the document operational in the various European Countries, a “National Annex” for each of these nations had to be issued. These annexes comprise provisions depending on geography and well-established regional traditions and experience, but also where it was not practical to achieve European consensus.
Among these provisions are the limiting values for concrete composition and related requirements to the cover to the reinforcement in the design codes to ensure the design working life of the structure.

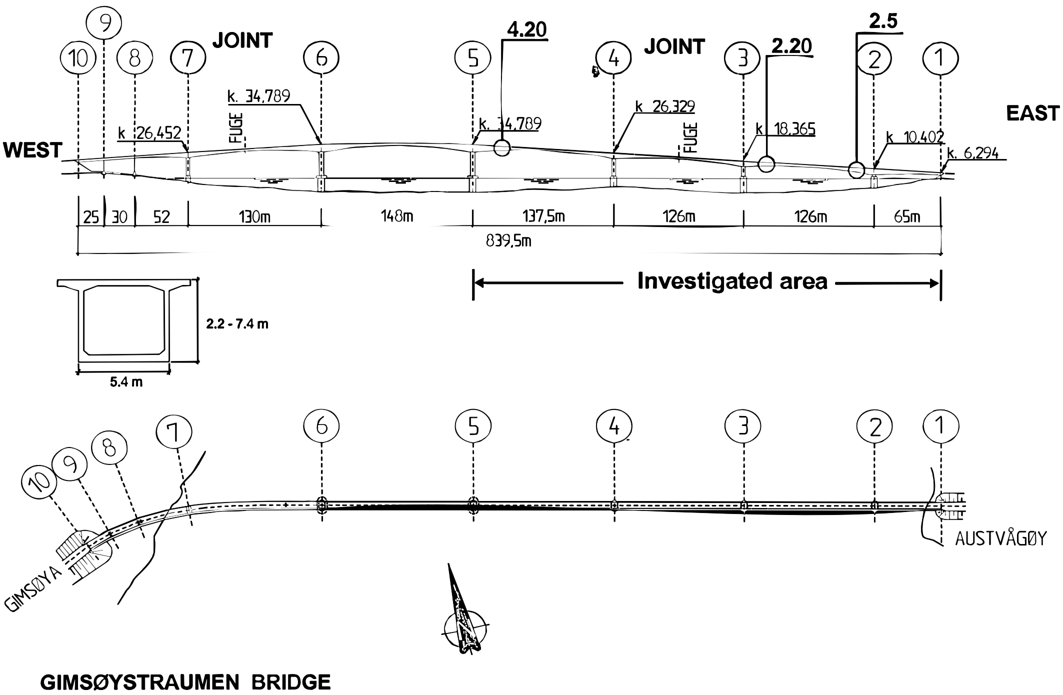
In Norway, the national standardisation body, The Norwegian Council for Building Standardisation – NBR, established a code committee to work out these requirements.

Having in mind Norway’s long coast and numerous marine structures, the provisions needed to achieve a relevant set of requirements to ensure the expected in-field performance of chloride exposed structures (exposure classes XS), was considered as a key issue.

To derive these provisions, the code committee concentrated on an assessment of the performance of existing structures. These assessments were based on in-field observations processed by the means of a mathematical model for ingress of chlorides in concrete. To conclude with the proxy parameters for durability needed for the standard, the probabilistic approach defined in prEN 1990: 2001 “EuroCode: Basis of Structural Design” was applied.

Inspection of existing marine structures

During 1999 to 2001, a Norwegian R&D project named “Lifecycle of Concrete Structures” headed by the Norwegian Public Road Administration, compiled and assessed the work done during the 1990s on field-performance of marine concrete structures.



These activities comprise offshore structures and a great number of coastal bridges and harbour works. In particular was the Gimsøystraumen bridge built in 1979 - 81 and inspected and repaired a decade later, thoroughly inspected and reported.

Real structures normally experience some abnormality in the achieved chloride profile compared with that of the model described in figure 2.
This abnormality is probably due to not continuous exposure to spray/splash of seawater combined with periods of washout due to rain.

Model for chloride ingress

The traditional assumption has been that chloride ingress into concrete obeys Fick's second law of diffusion for a semi-finite medium with constant exposure, and that there is a critical value of the chloride content in the concrete, $C = C_{crit}$, leading to the corrosion of steel:

$$C(x,t) = (C_s - C_i)\,erfc\left(\frac{x}{\sqrt{4tD}}\right)$$

where

- $C(x,t)$ = Chloride content at depth x at time t.
- C_i = Initial chloride content.
- C_s = Chloride content on the exposed surface.
- t = Exposure time.
- x = Depth.
- erfc = Error function.

- D = Diffusion coefficient.

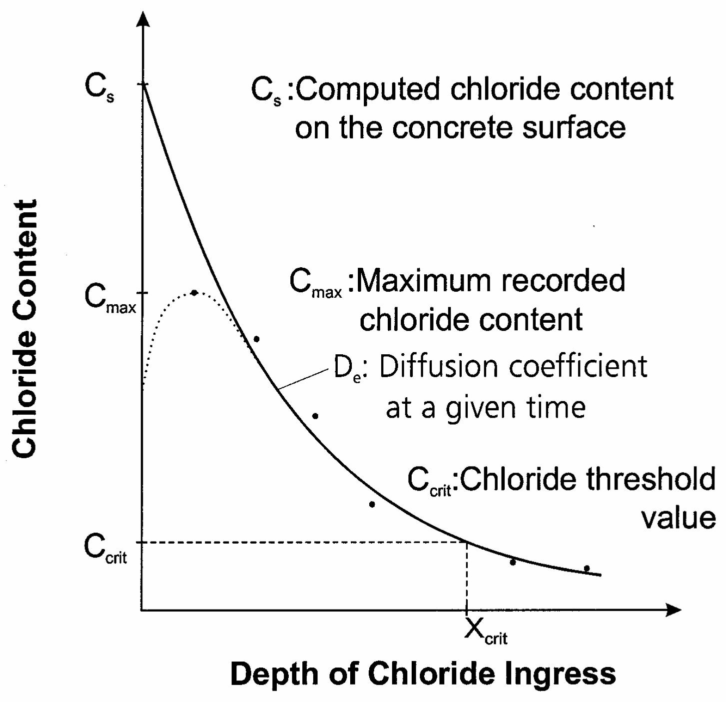


Figure 2 *Typical chloride profile in concrete exposed to sea water determined by regression analyses of data measured in the structure.*

In this equation C_s represents the chloride load, while the chloride diffusion coefficient, D, characterise the materials ability to withstand the ingress of chlorides.

D was earlier considered as a time independent parameter. However, it has been confirmed both in laboratory testing /Helland 1995/, /Mage et al. 1996/ and by observations from existing structures /Maage et al. 1996/, /Maage et al. 1999/ that this resistance is improved over time and obeys the mathematical expression:

$$\frac{D(t)}{D_o} = \left(\frac{t_o}{t}\right)^{\alpha}$$

Where D(t) is the time dependent chloride diffusion coefficient, t is the age of the concrete, and D_o is a measured reference diffusion coefficient at the age of t_o.
α is a parameter to be determined by regression analysis of test results.

The exponent α governs how fast the diffusion coefficient is improved over time. The physical explanation for this effect is two-fold. The cement-water reaction of the concrete is a long going process. As the hydration goes on, the porosity of the paste decreases. This has a well-known beneficial effect on the long-term strength gain, but the reduced porosity also improves the resistance towards ingress of chloride ions. The second effect is the beneficial effect of contact with the seawater itself. For mature material /Maage et al. 1991/ demonstrated that an ion exchange occurred between the seawater and the surface layer. Magnesium and potassium gradually blocked the pore system and then further improved the resistance to chloride ingress.
The α exponent reflects the decrease of the achieved diffusion coefficient with age due to the combined effect of hydration and all other mechanisms acting in-field as ion exchange with the seawater. Thus $\alpha = \beta + \gamma$ where β represents the effect of continued hydration of the cement and γ represents the beneficial effect on the concrete skin by being in contact with the seawater.

Chloride load

The effect of the environment is represented in the C_s in Fick's second law. This parameter identifies the representative chloride concentration at the concrete surface during the time of exposure. The C_s depends both on the salinity of the water, possibly the porosity of the surface layer (and thus the amount of saline pore water) and the length of wetting versus drying in the splash zone.

While calculating the C_s from a measured chloride profile, the C_s is represented as the chloride concentration at the surface (fig 2).

<u>*Observations from the Gimsøystraumen bridge*</u>

Typically the measured C_{max} on a bridge girder is distributed over the section like on figure 3 from the Gimsøystraumen bridge. The variations are obviously a result of difference in the microclimate. The influence of rain on the windward side is clear.

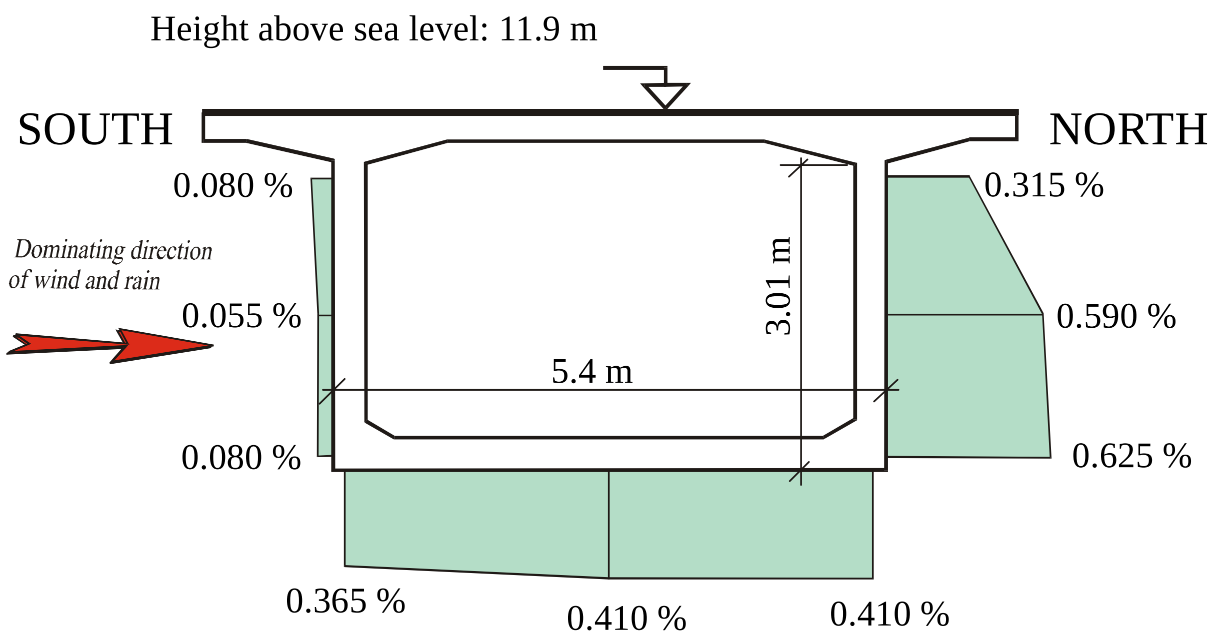


Figure 3 Gimsøystraumen bridge. Influence of microclimate on the environmental load. C_s is given in % of concrete mass
The effect of windward / leeward is clear.

The main inspection/condition survey of the Gimsøystraumen bridge was performed in 1992 at an age of 11 years (4000 days). This included:

- More than 4600 chloride analysis at 920 locations
- 752 of them on the super structure
- 168 on the columns

During the condition survey the following tests were performed:

- Drilled concrete powder (4 holes per test) for chloride analysis
- Measured concrete cover by covermeter
- Chiselling for recording real concrete cover (calibration of the covermeter)
- Evaluation of the level of reinforcement corrosion.

In addition recording of electrical potential and electrical resistance in the cover.

Regression analyses were performed in order to determine:

C_s - chloride load
D_e - diffusion coefficient

In the analyses the initial chloride content C_i was generally set to $C_i = 0.01$ % of concrete mass in the computations, but $C_i = 0.03$ % of concrete mass was also used.
During the analysis we found, for column 3, a better correlation with the other column if using $C_i = 0,03$ % of concrete mass. Going back to the construction diary we could read that some problems had occurred when constructing column 3, due to supply of sea grabbed sand with a too high content of filler.

Influence of height above sea level.

Figure 4 shows maximum recorded chloride content C_{max} at different heights above sea level and include both windward and leeward effects. The data represents, in addition to those from the Gimsøystraumen bridge, also measurements from 35 other coastal bridges representing 850 chloride profiles.

In the figure we distinguish between 4 environmental zones:

- 0 – 3 m above sea level
- 3 – 12 m above sea level
- 12 – 24 m above sea level
- > 24 m

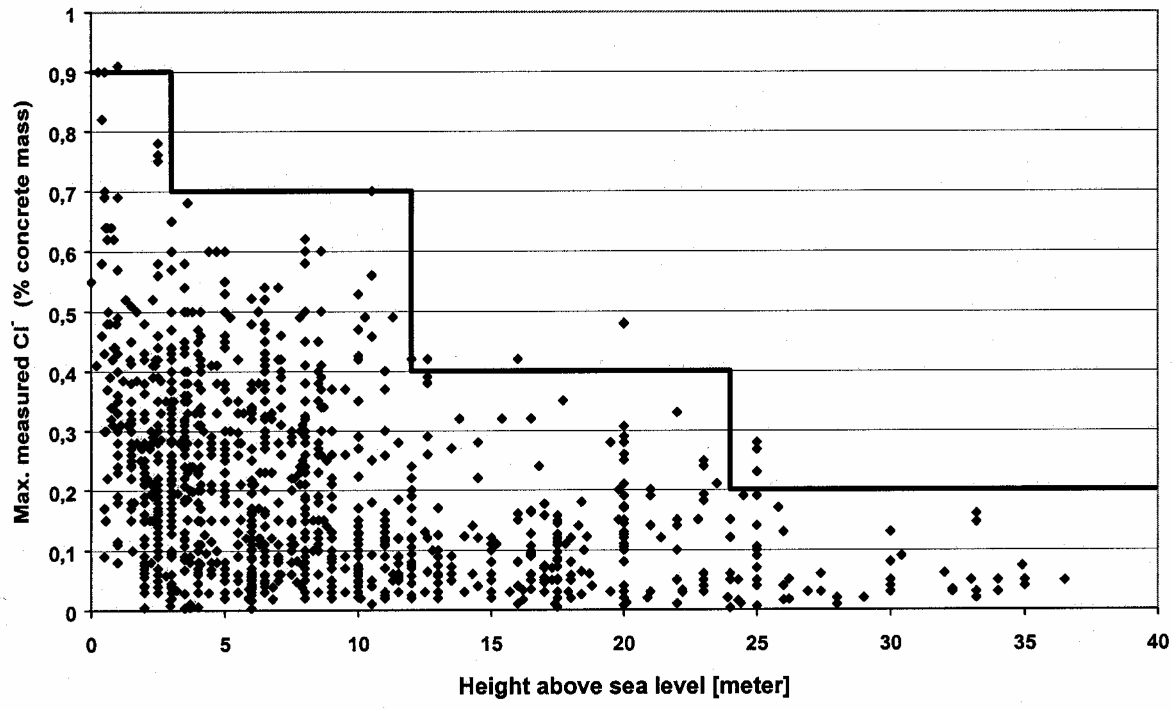


Figure 4 Chloride concentration, C_{max}, as function of height above sea level. Values given in Cl^- (%) of concrete mass.

The measured profiles have been analysed to derive the C_s – values and these computed data have been used as basis for the further discussions in this paper.

In figure 5 the computed C_s – values for the leeward side of the Ginsøystraumen bridge are given.

Make notice to the high C_s values over the massive parts of the structure over the columns.

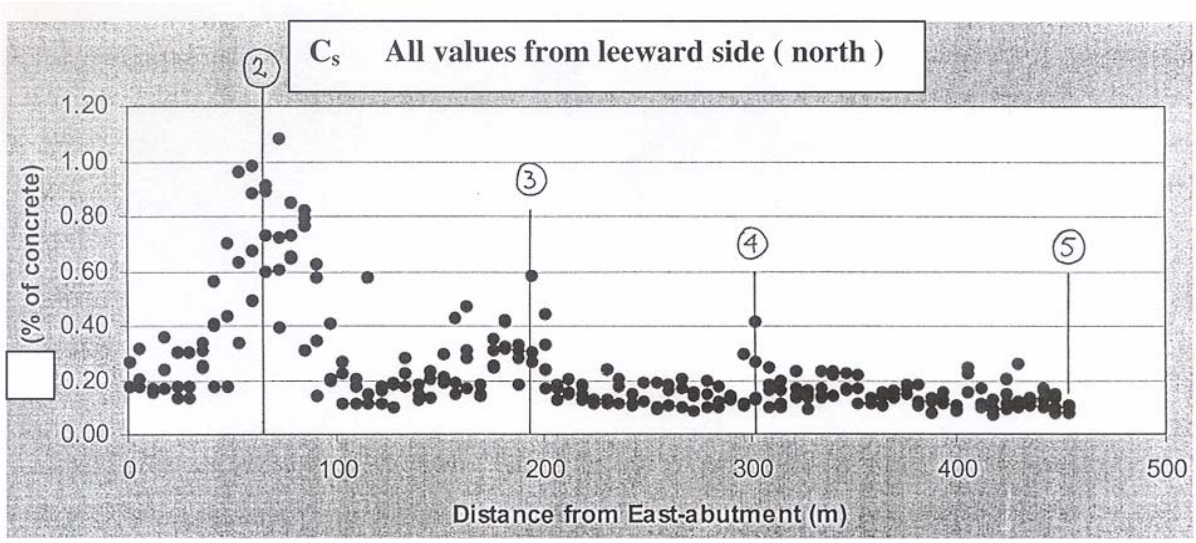


Figure 5 *Computed C_s – values for the leeward side of the Gimsøystraumen bridge*

In table 1 the characteristic C_s is given for 4 zones with different height above sea level. In this presentation, characteristic C_s is defined as $C_{s\ char.} = C_{s\ mean} + 1.3\ \sigma_s$ (10 % of the population has higher concentrations than $C_{s\ char.}$).

Table 1 *Chloride content Cl^- in % of concrete mass*

Zone Height above sea level in meter	Concrete load mean values C_s	Standard deviation σ_s	Design value C_{sn} $C_s + 1.3\ \sigma_s$
0 – 3	0.51	0.23	0.81
3 – 12	0.36	0.24	0.67
12 – 24	0.22	0.19	0.47
> 24	0.17	0.10	0.30

D_e diffusion coefficient

Figure 6 shows the computed D_e at the age of inspection (about 4000 days) along the leeward side of the superstructure of Gimsøystraumen bridge.

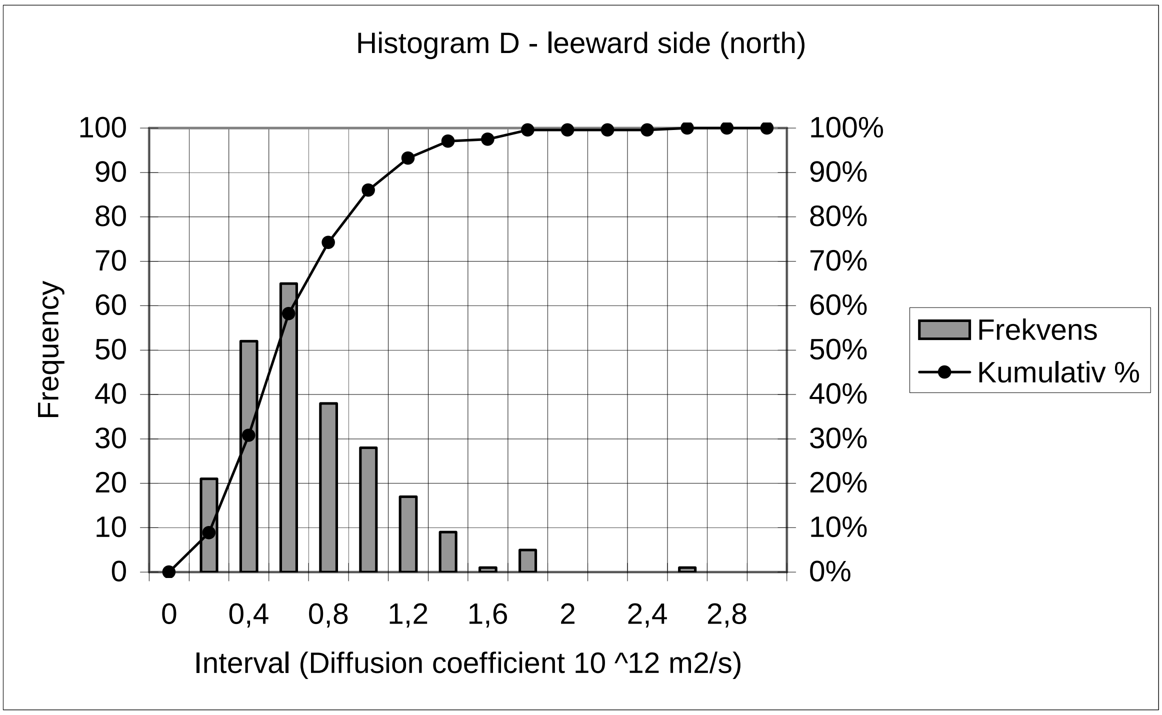


Figure 6 *Histogram of computed D_e from cores taken from the Gimsøystraumen bridge after 11 years*

The statistical distribution of the observed diffusion coefficients have been tested against the following probabilistic models:

Table 2

Model	**Mean value** 10^{-12} m^2/s	**Standard Deviation** 10^{-12} m^2/s	R^2
Normal			0.9452
Gamma			0.9548
Gumbel	0.61	0.37	0.9913
Weibull			0.9879
Lognormal			0.9817

All the models fit well to the recorded data. For simplicity, I will apply Normal Distribution in the following computations. Studies applying other models have demonstrated that the accuracy of the end conclusion is only negligible influenced by this practical approach.

Figure 7 also demonstrates that the diffusion coefficient might be regarded as a material parameter not sensitive to the chloride exposure itself.

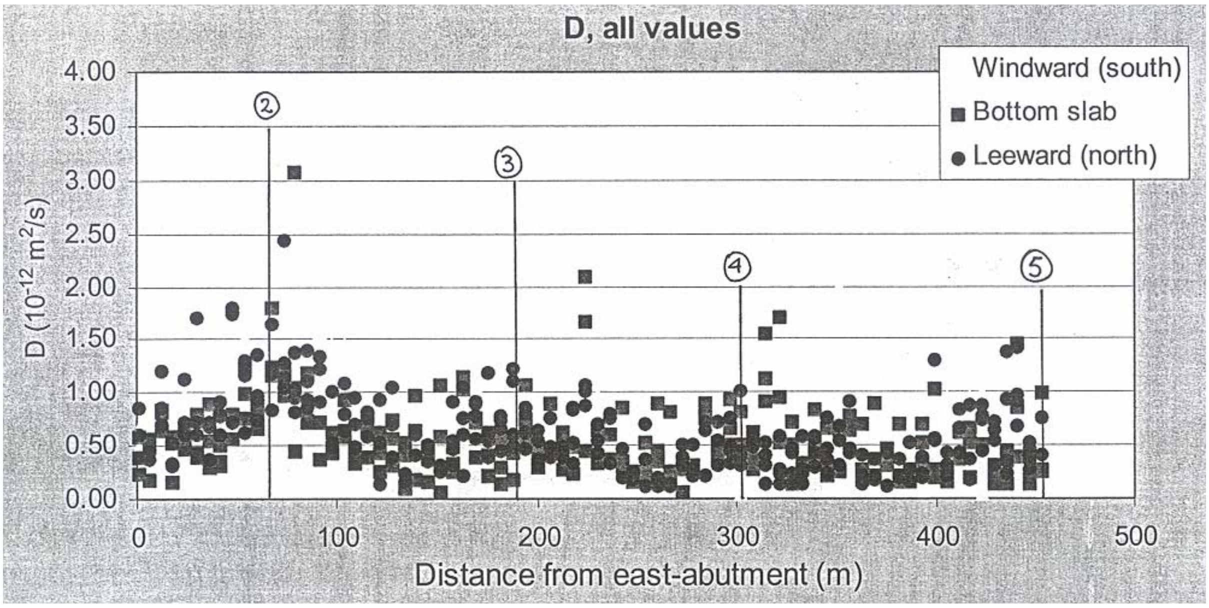


Figure 7 *Diffusion coefficients based on drilled cores from the superstructure 11 years after construction*

The concrete in the superstructure of Gimsøystraumen bridge is of Grade 40 MPa with w/b-ratio $\approx$ 0.52.
Corresponding D_{bulk} tested according to [NT Build 443 – 5 week exposure to 10 % NaCl solution] = $14 \cdot 10^{-12}$ m^2/s . The tests were performed on the inner (virgin) parts of drilled cores from the structure.

<u>*Concrete cover*</u>

Figure 8 shows the distribution of concrete cover recorded in section 2 of Gimsøystraumen bridge.

The specification required 30 mm minimum cover to the reinforcement.
Achieved numbers were a mean of 29 mm with a standard deviation of 5.2 mm.
The data obeys fairly well that of a normal distribution.

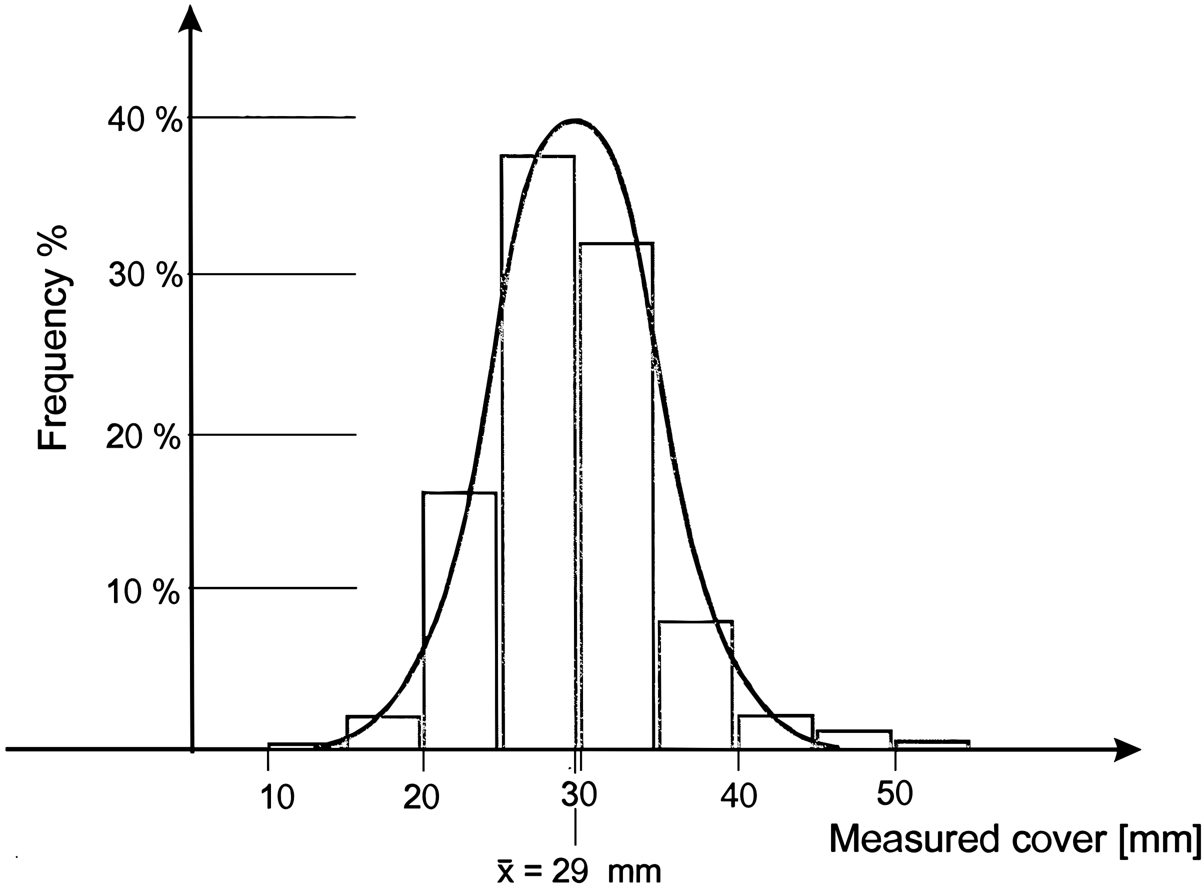


Figure 8 Distribution of cover in area 2 – Gimsøystraumen bridge. Based on 3612 measurements

<u>*Threshold value for initiation of corrosion*</u>

During the inspection of Gimsøystraumen bridge in 1992, concrete cover was chiselled away in 110 locations in order to both measure real concrete cover and to evaluate the level of rebar corrosion.

The evaluation of rebar corrosion was based on the following corrosion levels:

A: No sign of corrosion
B: Signs indicting depassivation
C: Corrosion
D: Heavy corrosion
E: Severe corrosion, pitting etc.

Figure 9 sums up the findings on both Gimsøystraumen bridge and other coastal bridges.

Corrosion level C indicates start of corrosion and is in our work defined as "failure".

Hence a threshold value of C_{crit} = 0.72 % Cl^- of weight of cement or 0.13 % Cl^- of concrete mass has been used in the further computations.

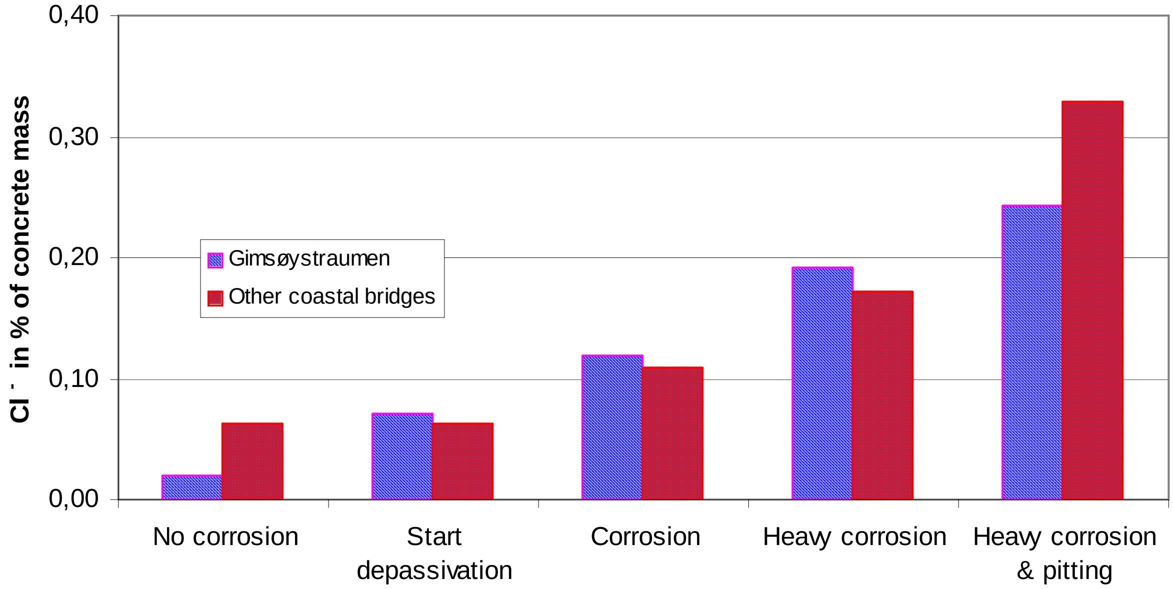


Figure 9 *Corrosion levels observed at the Gimsøystraumen bridge and from the general survey of 35 other Norwegian coastal bridges versus chloride content in the concrete.*
The registrations are based on visual inspection of the rebars after chiselling off the concrete cover at some 300 locations

In broad lines, these levels observed in the Norwegian coastal structures correspond very well with those reported by R. Browne /Browne 1980/ and which are often used for this purpose.

Table 3 *Risk of corrosion depending on chloride content /Browne 1980/*

Cl^- % of cement	Cl^- % of concrete assumed 440 kg cement/m³	Risk of corrosion
> 2.0	> 0.36	Certain
1.0 – 2.0	0.18 - 0.36	Probable
0.4 – 1.0	0.07 - 0.18	Possible
< 0.4	< 0.07	Negligible

Model for a probabilistic approach

The probabilistic approach applied in this paper is that described in draft prEN 1990 “Eurocode: Basis of Structural Design”

The "failure" reliability Z is calculated as the difference between a resistance against "failure", R, (concrete cover) and an environmental load or action, F (time dependent chloride ingress). Failure will in this situation be defined as the end of the initiation period which means that "failure" occur when the depth of the critical chloride concentrations are equal to the smallest concrete covers.

The probability of failure, p_f, has to be defined as a maximum target probability, p_{target}, depending on safety philosophy. This can be expressed by the equation

$$p_f = p\{R - F < 0\} < p_{target}$$

When the functions R and F are normally distributed, Z also is normally distributed.

When the resistance R is normally distributed, it has an average value μ_R and a standard deviation σ_R independent of time. In most other situations the resistance will be time dependent.

When the load F is normally distributed, it has an average value μ_F and a standard deviation σ_F. Both the F characteristics are increasing with time.

The failure reliability Z is defined by the limit state function

$$Z = R - F$$

When normally distributed, the function Z has an average value

$$\mu_Z = \mu_R - \mu_F$$

and a standard deviation

$$\sigma_Z = (\sigma_R^2 + \sigma_F^2)^{1/2}$$

The average value μ_Z is reduced and the standard deviation σ_Z increased with time, which means that the probability of failure is increasing with time. In this situation (normal distribution), the failure probability may be expressed by

$$p_f = \Phi\,(-\,\mu_Z/\sigma_Z) = \Phi\,(-\beta)$$

where β is the so-called reliability index covering safety, serviceability and durability (prEN 1990, clause 1.5.2.17). In a design situation, the calculated β has to be greater than a required reliability level β_0 depending on the safety level.

The correlation between failure reliability, resistance and load is demonstrated in Fig. 10.

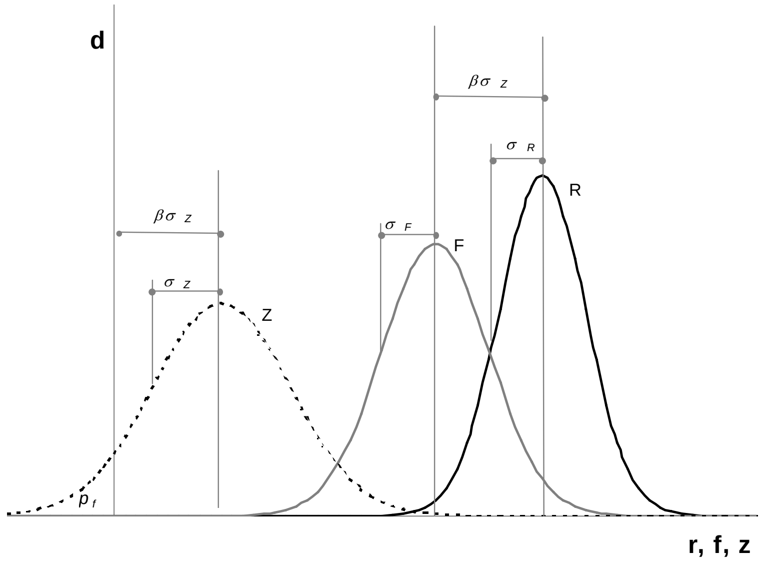


Figure 10 Correlation between failure reliability, Z, resistance, R and load (action), F

The relation between the reliability index β and the probability of failure p_f, when normal distribution applies, is shown in Table 4.

Table 4. Relation between reliability index β and the probability of failure p_f.

p_f	10^{-1}	$0.668\ 10^{-1}$	$0.359\ 10^{-1}$	$0.227\ 10^{-1}$	10^{-2}	10^{-3}	10^{-7}
β	1.28	1.50	1.80	2.00	2.32	3.09	5.20

According to prEN 1990, the highest value of β (lowest probability of failure) is required when the consequences of failure are high and the reference period for the load is short. Typical examples are failures due to accidents in public structures.

Serviceability limit states (SLS) are applied when the "failure" leads only to economic consequences. This is typical for durability situations where the deterioration will be visible long before any risk of collapse is reached.

This paper is focusing on typical SLS design where the target is to restrict the probability of corrosion initiation due to chlorides. After initiation, it may take many years before possible corrosion results in loss of serviceability or loss of structural safety. Additionally, the reference period for the load is equal to the service life, resulting in an even lower requirement to the limiting β value.

From this point of view, the probability of corrosion initiation due to chlorides may be set as high as 10^{-1}, corresponding to a reliability index β of approximately 1.30.

This philosophy is in agreement with prEN 1990, clause C6, but a β equal to 1.30 is somewhat lower than indicated in the informative clause C6 for Reliability Class RC2 structural members. No β values are given for Reliability Class RC1 in prEN 1990 for SLS, but the values should be somewhat lower than for class RC2. A reliability index β of approximately 1.30 should therefore be reasonable for this situation.

Figure 11 is a simple model to derive the failure reliability function Z. The figure demonstrates the influence of the different parameters mentioned before.

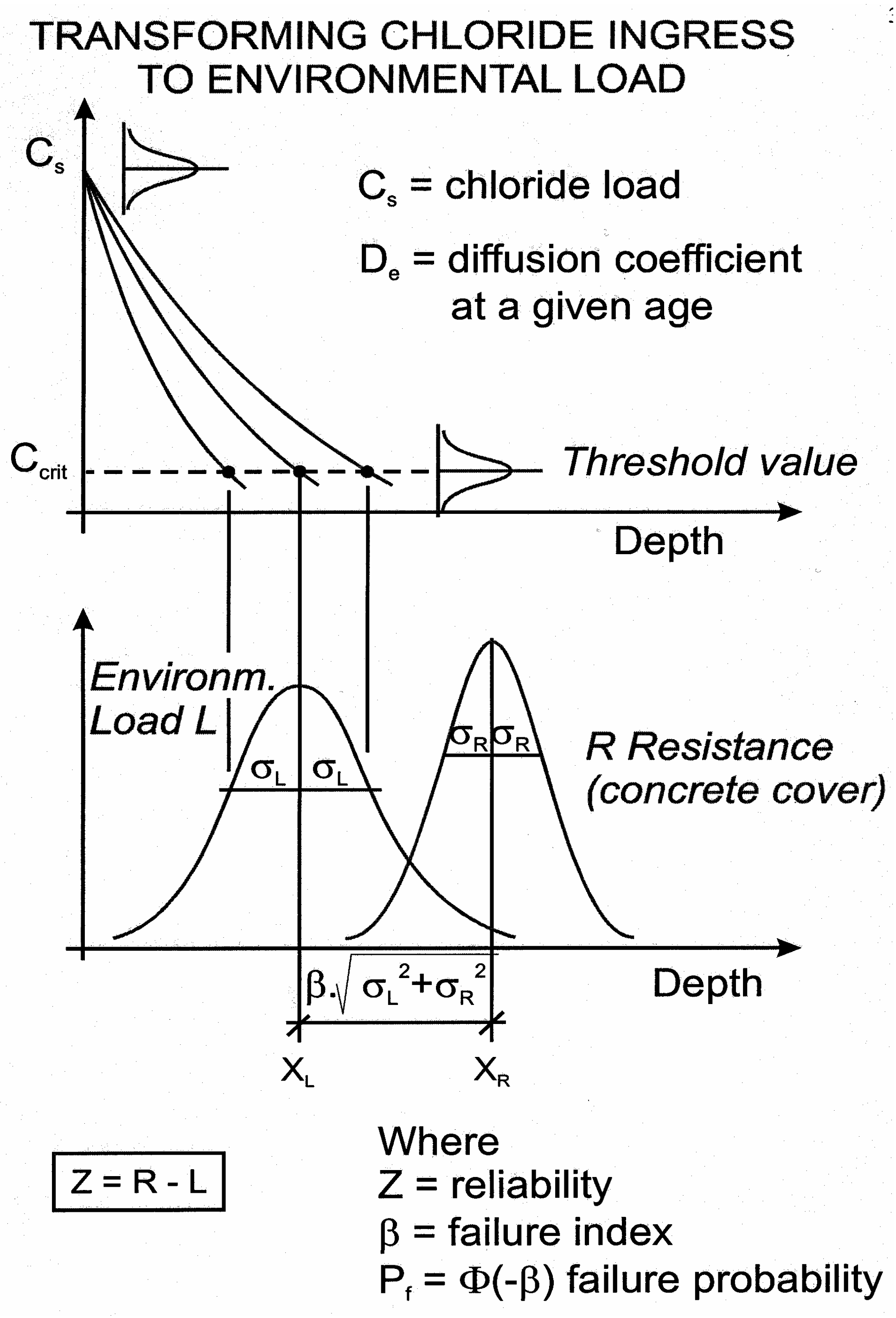


Figure 11 *Transforming chloride ingress to environmental load to derive the failure reliability function Z*

The upper graph shows the chloride ingress on basis of a chloride load C_s and diffusion coefficient D_e with a statistical variation.

The points where the chloride ingress curves are crossing the threshold value is defined as an "Environmental" Load. The Environmental Load has a statistical distribution, here pressed into a normal distribution, see lower graph.

Examples based on data from Gimsøystraumen bridge:

Cross-section 2.5 leeward side, see figure12.

Chloride load	C_s = 0.625 % Cl^- of concrete mass
Diffusion coefficient	D_e (11 years) = $0.61 \cdot 10^{-12}$ m^2/s (mean)
Age 4000 days	$\sigma = 0.37 \cdot 10^{-12}$ m^2/s standard deviation
Threshold value for start depassivation (fig 9)	C_{crit} = 0.07 % Cl^- of concrete mass
Concrete cover	X_R = 29.0 mm mean
	σ_R = 5.2 mm standard deviation

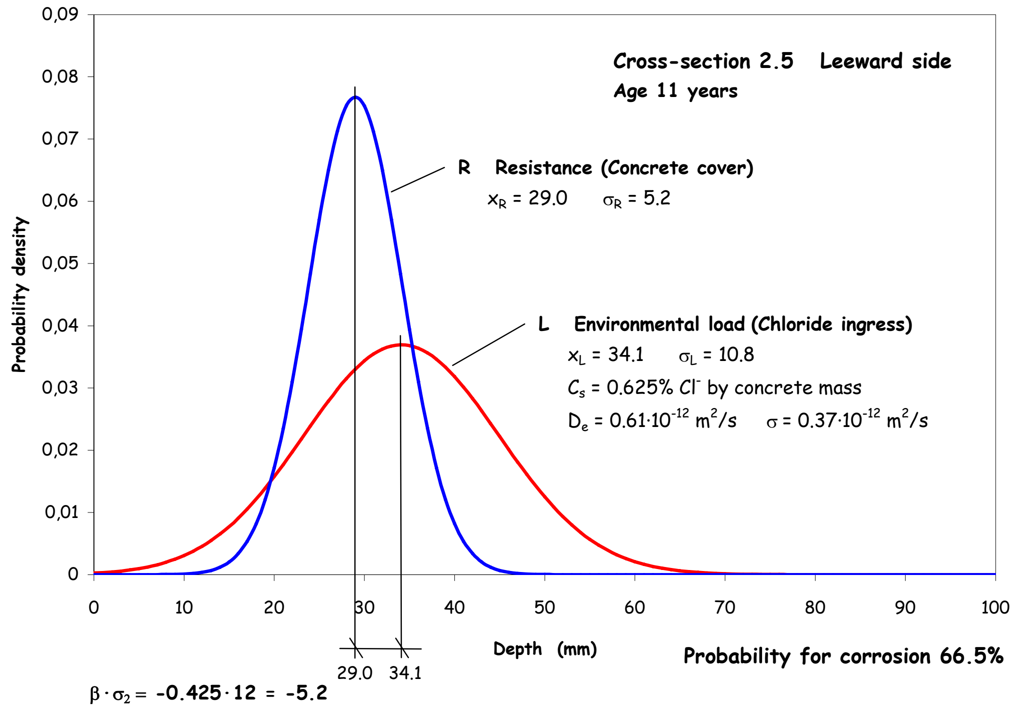


Figure 12 Cross-section 2.5. Leeward side. Age 11 years (see fig. 3)

For this section at the leeward side, observations confirmed that active corrosion actually did take place after 11 years. This conforms that the computed probability of corrosion was realistic. The example indicates that the applied procedures for computing the risk correlates with reality.

Cross-section 2.5 windward side, see figure 3.

As above except:

Chloride load C_s = 0.10 % Cl^- of concrete mass

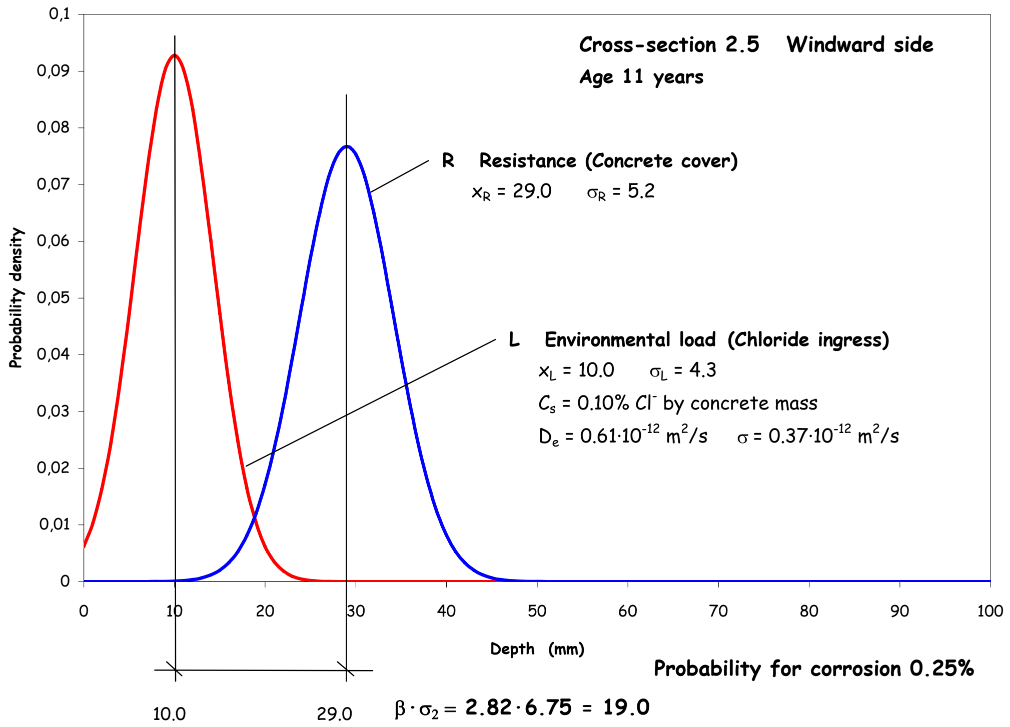


Figure 13 *Cross-section 2.5. Windward side. Age 11 years (see fig. 3)*

For this section at the windward side, observations confirmed that active corrosion actually did not take place after 11 years. This conforms again that the computed probability of corrosion was realistic. This example does also indicate that the applied procedures for computing the risk correlates with reality.

<u>*Cross-section 2.5 leeward side, see figure 3.*</u>

As the first example except:
Concrete cover adjusted to a failure of 10 %.

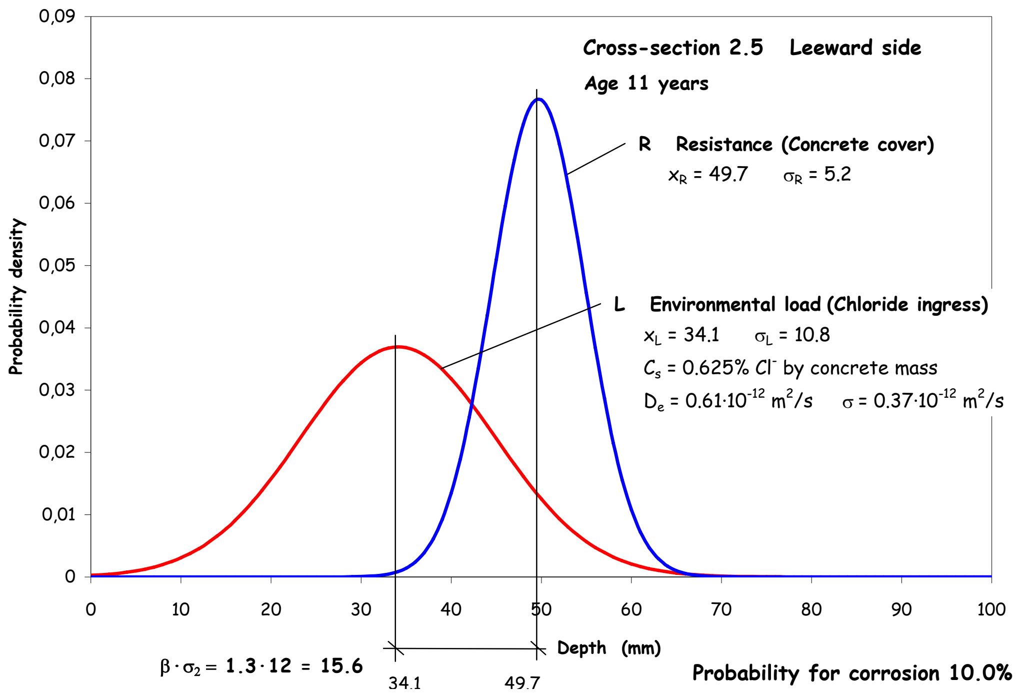


Figure 14 *Cross-section 2.5. Leeward side. Age 11 years*
Concrete cover adjusted to a failure of 10 %.

In this case we have applied the calibrated model to compute the needed nominal cover to achieve a 10 % probability for start depassivation after 11 years based on the chloride load and material quality measured at the Gimsøystraumen bridge.

Relevance to EN 206-1

During the above chapters, the expected distributions of design parameters for a service life calculation have been assessed. In addition, the whole system of design parameters and variation of these has been treated in a chloride ingress model and the results compared with the actual observations on old in-field located structures with age ranging from about one to three decades.
This forms the basis for the calibration of the total system.
These calibrated procedures are then well suited tools for assessing other combinations of concrete quality and cover that might comply with a service life of 50 respectively 100 years with the required degree of reliability.

<u>*Assumption for concrete cover*</u>

Rules for concrete cover are given in NS 3473: 1999 “Design of concrete structures” and NS 3420:1999.

Both are under revision and will be issued in 2002 as NS 3473:2002 and NS 3465:2002.
NS 3465 "execution of concrete structures" will be a 90 % loyal copy of ENV 13670-1: 2000 "Execution of concrete structures"

In NS 3473:1999 table 12, the minimum required concrete cover x_{min} for exposure class "Very Aggressive – Splash zone" (exposure class XS-3according to EN-206-1), corrosion induced by chlorides from sea water, is 50 mm. This is valid for ordinary reinforcement and 50 years service life.

The nominal cover x_n, defined as the sum of the minimum cover x_{min} and a "minus-deviation", is equal to the average target concrete cover (the dimension of the chairs to be used). The "minus-deviation", $\Delta_{(minus)}$ is, according to ENV 13670-1, equal to 10 mm, which means that the required nominal cover is 60 mm. These tolerances are also in agreement with the Norwegian Standards NS 3473 (Ref. 4), clause 17.1.8 and NS 3420 (Ref. 5), clause L2, d1).

From a statistical point of view, the minimum cover has to be associated with a probability of "failure", meaning that a certain percentage of the reinforcement has a smaller cover than the required minimum. A reasonable assumption may be that 5 % of the reinforcement has a lower cover than the required minimum. Assuming that the concrete cover is normally distributed, the distance from the mean value to the 5 % percentile is 1.645 times the standard deviation, as demonstrated at the Gimsøysundet bridge, giving the standard deviation
$\sigma_R = 10 / 1.645 = 6.1$ mm.

In the calculations, the average concrete cover, μ_R, will therefore be set equal to
50 mm + 10 mm = 60 mm and the standard deviation, σ_R, equal to 6.1 mm as the design assumption for new structures executed according to NS 3473 / NS3465.

Assumption for environmental load (Chloride surface concentration)

Due to the big differences in environmental load on the various parts of the structure caused by the microclimatic variations (see for instance figure 3), it is difficult to define representative statistical criteria for the % of the reinforcement that should be linked to the "failure criteria" (depassivation). The considerations have to be related to the worst exposed parts of the structure. It will be difficult to justify that heavy corrosion at one section of the structure might be compensated by greater parts sheltered from seawater spray.
In these computations we have therefore chosen a nominal value for the design surface chloride concentration. This value is chosen from table 1. The principle of "nominal" characteristic value of action is based on prEN 1990 paragraph 4.1.2 : "*(1) P – The characteristic value F_k of an action is its main representative value and shall be specified: - as a mean value, an upper or lower value, or a nominal value (which does not refer to a known statistical distribution) …*"

However, it must be admitted that the procedure with nominal design parameters biases somewhat the statistical concept when computing the overall level of reliability.
In these calculations, design values for the surface chloride concentrations, C_s, have been chosen as 0.81 – 0.67 – 0.47 – 0.30 % by mass of concrete (table 1)

Computation

Based on:

- C_s = 0.81 – 0.67 – 0.47 – 0.30 % by mass of concrete
- Concrete cover to the surface reinforcement with a mean of 60 mm and a standard deviation of 6 mm
- A level of reliability of 10 %
- A failure criteria linked to the onset of corrosion
- Critical chloride concentration for onset of corrosion of 0.13 % of concrete mass
- The calibrated model for chloride ingress as described above and with an aging factor (the exponent α) ranging from 0.50 to 0.67

The needed diffusion coefficients were then derived for 50 years design service life.
This was determined as 7.0 10^{-12} m^2/s corresponding to a referense object subject to 4 weeks curing and 5 weeks exposure to chlorides (NT Build 433)

Relation between water binder ratio, w/b-ratio, and bulk diffusion coefficient D_{bulk}

These diffusion coefficients might again be transformed to the parameters required in the tables in EN 206-1, i.e. water-cement ratio and cement type.
An example of such a correlation is given in figure 15
The graph shows a few data indicting the relation between w/b-ratio and D_{bulk} tested according to [NT Build 443]

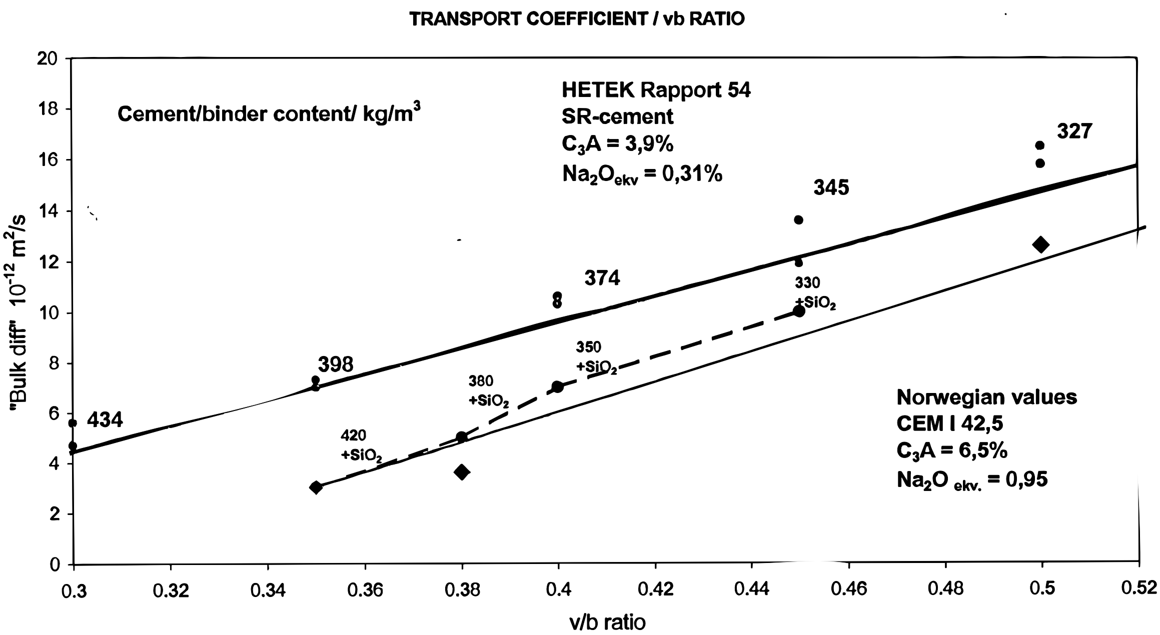


Figure 15 *An example of the correlation between bulk diffusion coefficient and water-binder ratio*

Conclusion

Based on the above field data, models for computation and reasoning, the code committee concluded to prescribe the following combination of concrete composition and cover to the reinforcement in the national Appendix to NS-EN 206-1 to ensure 50 years of service life for marine structures exposed to sea water with the required level of reliability:

Table 5 Needed nominal cover

Durability class according to National Norwegian Annex to NS EN-206-1	Needed nominal cover to reinforcement			
	Zone (height above sea level) according to table 1			
	0 - 3 m	3 - 12 m	12 - 24 m	> 24 m
M-40 or MF-40	63.4 mm	59.3 mm	49.9 mm	38.1 mm

The minimum requirements for mix composition for a “Durability class” M-40 according to the Norwegian National Annex to NS EN-206-1 are in brief:

> A water/cementitious ratio less than 0.40 and one of the following binder types: CEM I cement and > 4 % silica fume, CEM II/A-S, (6-20 slag), CEM II/B-S (21-35 slag), CEM II/A-D, (6-10 silica), CEM II/A-V, (6-20 FA), CEM II/B-V (21-35 FA), CEM III/A (36-65 Slagg).

As might be noted, the Norwegian code committee did not feel comfortable by allowing a concrete mix based on an unblended binder for use in the most exposed marine structures.

Durability class MF-40 has similar minimum requirements except that it also shall stand freeze/thaw actions. By that reason a minimum content of 4 % entrained air is demanded, Since the Nordic countries have little in-field experience with the winter-performance of concrete mixed with slag-cements, these are not covered by the National Annex for structures both exposed to sea water and freeze/thaw.

The final step was then to interpret the exposure classes as they are qualitatively described in EN 206-1.

Table 6 Extract from Table 1 "Exposure classes" from EN 206-1.
Informative examples (right column) from the Norwegian National Annex

4 Corrosion induced by chlorides from sea water		
Where concrete containing reinforcement or other embedded metal subject to contact with chlorides from sea water or air carrying salt originating from sea water, the exposure shall be classified as follows:		
XS1	Exposed to airborne salt but not in direct contact with sea water	Surface near to or on the coast, but not subject to direct spray
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures

The Norwegian code committee did that in the following manner:

Table 7 Required nominal cover (minimum cover + 10 mm tolerance) according to Norwegian design standard NS 3473

Durability class according to National Norwegian Annex to NS EN-206-1	Required nominal cover to reinforcement	
	Exposure class according to NS EN 206-1	
	XS-3	XS-2
M-40 or MF-40	60 mm	50 mm

In the Norwegian code NS 3473 "Design of concrete structures", these requirements are given as requirements for minimum concrete cover. Having in mind the maximum allowed deviation given in the European standard for execution of concrete structures, ENV 13670-1 and as well in its Norwegian counterpart, NS 3465 of 10 mm, these minimum requirements are 50 and 40 mm respectively.

The robustness in the application of the model for chloride ingress is demonstrated in the fact that the input data for the computation were the structural response from actual structures after more than 10 years of exposure. At this age, due to the time-factor in the model, somewhat half of the critical chloride concentration has already been built up at the location of the surface rebars. A possible inaccuracy in the prediction will therefore to a great extent have been reduced.

References:

1 Helland S. «Service life prediction of marine structure»
Proceedings FIP Symposium «Concrete ‘95» Brisbane, Australia 1995

2 Maage M., Helland S., Poulsen E., Vennesland Ø. and Carlsen J.E.:
"Service Life Prediction of Existing Concrete Structures Exposed to Marine Environment." ACI Materials Journal, Vol. 93, No. 6, Nov.-Dec. 1996

3 Maage M., Helland S. Carlsen J. E. «Chloride penetration into concrete with light weight aggregates." Brite EuRam project BE96-3942, CUR P.O.Box 420, NL- 2800 AK Gouda, The Netherlands, May 1999.
The report is downloadable from www.sintef.no/bygg/sement/elcon

4 Maage M., Helland S. «Quality Inspection of «Shore Approach» High strength concrete». Proceedings Second CANMET/ACI International Conference on Durability of Concrete, Montreal 1991, ACI SP 126, Detroit USA

5 Browne R. D. et al. :"Concrete in the Ocean - Marine Durability of the Tongue Sand Tower". CIRIA UEG Technical Report No. 5, C & CA, London UK. 1980