Vegdirektoratet
Trafikksikkerhet, miljø- og teknologiavdelingen
Tunnel og betong
Mars 2015

Statens vegvesen

Fly ash in concrete, Danish experience

Etatsprogrammet Varige konstruksjoner 2012-2015

STATENS VEGVESENS RAPPORTER Nr. 370

Foto: Dirch Bager

Statens vegvesens rapporter

NPRA reports
Norwegian Public Roads Administration

Tittel
Flygeaske i betong, danske erfaringer

Undertittel
State-of-the-art rapport

Forfatter
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Avdeling
Trafikksikkerhet, miljø- og teknologiavdelingen

Seksjon
Tunnel og betong

Prosjektnummer
603242

Rapportnummer
Nr. 370

Prosjektleder
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Emneord
Varige konstruksjoner, tilstandsutvikling bruer, flygeaske, betong, bestandighet, kloridinntrengning

Sammendrag
I rapporten fokuseres det på danske erfaringer med kloridinntrengning i betong som inneholder flygeaske. Generelle erfaringer med produksjon og bestandighet av betong med flygeaske er også beskrevet. Flygeaske har vært i bruk i Danmark siden midten av 1970-tallet, typisk som tre-pulver blandinger som også inkluderer silikastøv. Betonger med flygeaske gir generelt redusert kloridinntrengning sammenlignet med betonger med ren Portlandsement. Effekten av flygeaske blir tydeligere med økende alder.

Antall sider 65

Dato Mars 2015

Title
Fly ash in concrete, Danish experience

Subtitle
State-of-the-art report

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Department
Traffic Safety, Environment and Technology Department

Section
Tunnel and concrete

Project number
603242

Report number
No. 370

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Key words
Durable structures, existing bridges, fly ash, concrete, durability, chloride ingress

Summary
The report focuses on Danish experience regarding chloride ingress in concrete containing fly ash, but general experience with production and durability of fly ash concrete is also included. Fly Ash has been used in Denmark since the mid 1970'es, typically based on a three powder blend also including silica fume. In general, reduced chloride ingress can be observed for concretes with fly ash compared to Portland cement concrete. The impact of fly ash becomes more noticeable after several years.

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Forord

Denne rapporten inngår i en serie rapporter fra **etatsprogrammet Varige konstruksjoner**. Programmet hører til under Trafikksikkerhet-, miljø- og teknologiavdelingen i Statens vegvesen, Vegdirektoratet, og foregår i perioden 2012-2015. Hensikten med programmet er å legge til rette for at riktige materialer og produkter brukes på riktig måte i Statens vegvesen sine konstruksjoner, med hovedvekt på bruer og tunneler.

Formålet med programmet er å bidra til mer forutsigbarhet i drift- og vedlikeholdsfasen for konstruksjonene. Dette vil igjen føre til lavere kostnader. Programmet vil også bidra til å øke bevisstheten og kunnskapen om materialer og løsninger, både i Statens vegvesen og i bransjen for øvrig.

For å realisere dette formålet skal programmet bidra til at aktuelle håndbøker i Statens vegvesen oppdateres med tanke på riktig bruk av materialer, sørge for økt kunnskap om miljøpåkjenninger og nedbrytningsmekanismer for bruer og tunneler, og gi konkrete forslag til valg av materialer og løsninger for bruer og tunneler.

Varige konstruksjoner består, i tillegg til et overordnet implementeringsprosjekt, av fire prosjekter:

Prosjekt 1: Tilstandsutvikling bruer
Prosjekt 2: Tilstandsutvikling tunneler
Prosjekt 3: Fremtidens bruer
Prosjekt 4: Fremtidens tunneler

Varige konstruksjoner ledes av Synnøve A. Myren. Mer informasjon om prosjektet finnes på vegvesen.no/varigekonstruksjoner

Denne rapporten tilhører **Prosjekt 1: Tilstandsutvikling bruer** som ledes av Bård Pedersen. Prosjektet vil generere informasjon om tilstanden for bruer av betong, stål og tre, og gi økt forståelse for de bakenforliggende nedbrytningsmekanismene. Dette vil gi grunnlag for bedre levetidsvurderinger og reparasjonsmetoder. Innenfor områdene hvor det er nødvendig vil det etableres forbedrede rutiner og verktøy for tilstandskontroll- og analyse. Prosjektet vil også frembringe kunnskap om konstruktive konsekvenser av skader, samt konstruktive effekter av forsterkningstiltak. Prosjektet vil gi viktig input i forhold til design av material- og konstruksjonsløsninger for nyere bruer, og vil således ha leveranser av stor betydning til Prosjekt 3: Fremtidige bruer.

Rapporten er utarbeidet av *Mette Geiker, NTNU,* som del i et samarbeidsprosjekt med Varige konstruksjoner.

State-of-the-Art report

FA in Concrete, Danish experience

Date: 31 January 2015

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Project name: Research and development collaboration agreement between NPRA and NTNU
(FoU samarbeid Varige konstruksjoner TB1 - Kloridinntrengning i betong)

Project no.: SVV: 2012081554, NTNU: 25116700

Preface

The report is prepared as part of a Research and development collaboration agreement between NPRA and NTNU (FoU samarbeid Varige konstruksjoner TB1 - Kloridinntrengning i betong, saksnummer: 2012081554, dated 29.6.2012).

The goal of this report is to provide a State-of-the-Art report on Danish experience on fly ash concrete with focus on chloride ingress.

NRPA's motivation for the collaboration is:

> «Statens vegvesen har et stort antall bruer utsatt for meget høy kloridbelastning som følge av plassering langs kysten med værhardt marint klima. Kloridinntrengning inn til armeringen fører til korrosjonskader som nedsetter sikkerheten, øker vedlikeholdskostnadene og forkorter bruenes levetid.
>
> Betongbruene i Norge har tradisjonelt vært bygget av betong med ordinære Portlandsementer (CEM I), men fra begynnelsen av 90-tallet alltid i kombinasjon med silikastøv (SV-40 betong). Sementer med flygeaske har i varierende grad vært på markedet fra midten av 80-tallet, og i løpet av de siste årene har flygeaskesementer med 17-20% flygeaske (klasse CEM II/A-V) blitt dominerende på det norske markedet. Om endringene i betongsammensetning de siste 15-20 årene har hatt den nødvendige effekt er ikke verifisert i særlig grad, og en slik aktivitet er derfor av stor betydning for vurdering av betongbruers levetid i fremtiden.
>
> Betonger med flygeaske og slagg utmerker seg ved å være svært tette og utvikle meget god motstand mot kloridinntrengning, i tillegg utvikler de meget høy elektrisk resistivitet over tid. Det finnes etter hvert en rekke internasjonale erfaringer med både slagg- og flygeaskebetonger, der spesielt de danske erfaringene med flygeaskebetonger er av spesiell interesse»

The overall objective of the collaboration is to improve the basis for selection of binders for reinforced concrete structures in marine exposure. Special focus is placed on how low-carbon binders with high fly ash and slag content resist chloride ingress.

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1. Introduction

Fly ash has been used in concrete production in Denmark since the mid 1970'es. According to Edvardsen [2014], the typical concrete mix for extra aggressive environment ("class E")[1] has during the last 20 years been based on a three-powder blend of Portland cement, fly ash and silica fume.

Experience with the use of fly ash in selected Danish infrastructure project was in 2009 summarized in a survey report prepared by COWI A/S for Emineral a/s [Eriksen et al. 2009]. The report focuses on concrete exposed to harsh environments such as seawater, freeze-thaw, and de-icing salt exposures.

The summary of the report is copied in Appendix 1 and the full report can be downloaded from http://www.emineral.dk/datablade.aspx . On this homepage you can also find certificates and declarations as well as reports on environmental aspects of fly ash.

Based on their survey Eriksen et al. [2009] summarize: "With fly ash addition, benefits as above[2] can be obtained in the fresh, hardening and hardened state for most properties. However, use of fly ash requires special attention for properties such as strength development and freeze-thaw resistance." And "For low variation in concrete properties during production it is of utmost importance that concrete constituents are of uniform quality."

The present report focuses on Danish experience regarding chloride ingress in concrete containing fly ash. However, general experience with production and durability of fly ash concrete is also included. Initially the Danish requirements to concrete with fly ash are described in Chapter 2. Chapters 3, 4 and 5 summarize observations from infrastructure projects, full scale trials, and field exposure sites. Construction related topics are briefly discussed in Chapter 6.

[1] Class E "extra aggressive" corresponds to exposure classes XD2, XD3, XS3, XF4, XA3 according to EN 206. Class E is described as "moist environment where large amounts of alkalis and/or chlorides are added or accumulated at the concrete surface". Typical structures are balconies, parking decks, swimming pools, columns, abutments near driving lane and edge beams on bridges, and marine structures in splash zone (http://www.aalborgportland.dk/media/pdf_filer/cement_og_beton_2012_web.pdf, page 41-42; "Betonbro – Generalnote – GN-P")

[2] low permeability to reduce chloride ingress and depth of carbonation, resistance to chemical attack, no development of alkali silica or other internal reactions, resistant to freeze-thaw attack, workable and compactable to prevent problems with execution, low shrinkage and low heat development during hardening to prevent thermal cracking and reduction of design strength.

2. Danish requirements to concrete with fly ash

The Danish Road Directorate's (VD) current requirements to concrete bridges can be found at http://vejregler.lovportaler.dk/SearchResult.aspx?t=%2fV1%2fNavigation%2fTillidsmandssystemer%2fVejregler%2fUdbud%2fbygvaerker%2f . Copies of the "Betonbro – Generalnote – GN-P" dated 10th April 2014 and "Beton AAB" dated 1st August 2012 are enclosed as Appendices 2 and 3.

VD allows fly ash in a maximum weight ratio to cement at 0.33 (FA/C≤0.33) in combination with CEM I (HS/EA/≤2) or CEM I (HS/LA/≤2)[3]. In addition silica fume in a maximum ratio to cement at 0.06 (SF/C≤0.06) [4] is allowed (but not required). Efficiency factors at 0.5 and 2.0 are used for fly ash and silica fume, respectively, i.e. the equivalent water-to-cement ratios by mass $w/c_{eq}=W/(C+0.5FA+2SF)$. Among others, requirements to frost resistance are given; acceptance criteria to either performance in frost testing or air void structure are to be fulfilled for concrete after normal transport and possible pumping (see Appendix 2, Beton, item 2.2 and notes 15, 16 and 17). To limit alkali silica reactions requirements are given to both aggregates and maximum alkali content in the concrete, independently of possible fly ash and silica fume content, see Appendix 2 "Betonbro – Generalnote – GN-P".

The development of Danish requirements to concrete from 1888 to 1988 is described in [Poulsen 1989]. The development of the requirements to concrete composition and cover in the Danish standards from 1949-1988 is summarized in Table 4, [Kjær et al. 1994] (a copy of the paper is enclosed as Appendix 4). Ten percent fly ash was allowed in the 1973 edition of DS 411, the 1984 edition opened up for use of higher amounts of fly ash and of silica fume (FA+SF≤35%; SF≤10%).

Kjær et al. [1995] also compared specifications for the first Danish marine bridge with fly ash concrete on a large scale (the Farø Bridges), the Great Belt Link and the Øresund Link and other selected medium and large structures in Northern Europe, see Appendix 4.

A summary of the historic development of standardization of fly ash can be found in [Eriksen et al. 2009]; Chapter 7. Additional information can be found on http://www.emineral.dk/datablade.aspx . The first European standard came in 194; EN 450-1:1994 Fly ash for concrete – Definitions, requirements and quality control. In 2005 co-combustion to supplement pulverized coal was allowed which led to additional requirements to the fly ash. In 2007 minor changes were made, and the latest version of the European standard is from 2012. The main changes in that standard were

- Extension of permissible content of solid co-combustion materials
- Extension of permissible content of liquid and gaseous co-combustion materials
- Fixed limits for loss on ignition
- Incorporation of the specifications from EN 450-1+A1:2007.

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[3] Nomenclature: see e.g. http://www.aalborgportland.dk/media/pdf_filer/cement_og_beton_2012_web.pdf, page 16. Type and strength class according to DS/EN 197-1, sulphate resistance, alkali content and chromate content according to DS/INF 135.
CEM I (HS/EA/≤2): Portland Cement (high sulphate resistance/extra low alkali content/chromate content<2 mg/kg)
CEM I (HS/LA/≤2): Portland Cement (high sulphate resistance/ low alkali content/chromate content<2 mg/kg)

[4] MS (for micro silica) in Danish literature

Today's requirements to fly ash for use in Danish concrete are given in DS/EN 450-1, 2012 "Fly as for concrete – Part 1: Definitions, specifications and conformity criteria". Selected requirements are summarized in Table 1. Reference is given to the standard for methods of testing. Regarding durability requirements it is stated that: "…In certain application, particularly for concrete in severe environmental conditions, the choice of fly ash category may have an influence on the durability of concrete, e.g. freeze-thaw resistance and resistance to alkali aggregate reactions. In such cases, the choice of fly ash category shall follow the appropriate standards and/or regulations valid in the place of use." According to DS 2426–EN 206-1:2011 is fly ash for concrete to comply with requirements to category A for loss on ignition and category N for requirements to fineness.

Table 1 – Selected chemical and physical requirements to fly as for use in concrete according to DS/EN 450-1, 2012. Reference is given to the standard for methods of testing.

Property		Unit	Requirement
Chemical	Loss on Ignition	% by mass	Category A: ≤5.0; B: ≤7.0; C: ≤9.0
	Chloride	% by mass	≤0.10
	Sulphate	% by mass	≤3.0
	Free calcium oxide	% by mass	≤1.5
	Reactive calcium oxide	% by mass	≤10.0
	Reactive silicon dioxide	% by mass	≥20.0
	Silicon dioxide+ aluminium oxide + ferro oxide	% by mass	≥70.0
	Total alkalis		≤5.0
	Magnesium oxide	% by mass	≤4.0
	Phosphate	% by mass	≤5.0
Physical	Fineness Retrained on 0.045 mm sieve	% by mass	Category N: ≤40.0 Category S: ≤12.0
	Activity index	%	28 days: ≥75; 90 days: ≥85

3. Danish concrete structures containing fly ash

Eriksen et al. [2009] summarizes data from selected Danish infrastructure projects, examples of mix design are given in Table 2.

Table 2 - Examples of mix design for selected Danish infrastructure projects [kg/m3]. Admixtures not included. [Eriksen et al. 2009] ([a] from [Bager 2001a]; [b] from [Gotfredsen et al. 1985]; [c] from [Storebælt 1998a]; [d] from [Storebælt 1998b]; [e] from [Øresund 2001]).

	Farø bridges		Guldborgsund tunnel		Great Belt bridge		Øresund tunnel	
Year	1980-1984		1986-88		1992-1997[c]	1991-1993[d]	1996-1999[e]	
	Underwater concrete	Structural concrete	Tunnel 1	Tunnel 2	Type A, East Bridge	Mod. Type B, West B. caissons	Tunnel segments	SCC for closure joints
CEM I 42.5 SR	330	330	275	275	320	340	324	380
Fly ash	100	40	80	50	47	75	52	70
Silica fume				15	20	20	24	45
Sand	580	622 (0-2 mm[b])	695	650	575	771	633	750
Stone	1103	1205	1190	1190	1285	918	1154	1000
Water	150	140	128	143	133	155	143	
w/c^f			0.47	0.52	0.42	0.46		
$W/(C+0.5FA+2SF)^g$	0.40	0.40					0.38	0.39

[f] Water-to-cement ratio by mass

[g] Equivalent water-to-cement ratio by mass (w/c_{eq}); assuming efficiency factor 0.5 and 2 of fly ash and silica fume, respectively

For the Alssund bridge fly ash (50 kg/m3) was in 1977 included in two foundations to improve workability in connection with pumping[5]. During the 1970'es and 80'es fly ash and/or silica fume were used in several smaller projects; the Farø bridges were the first infrastructure project where fly ash was used on a large scale. According to Edvardsen [2014] the typical concrete mix for extra aggressive environment ("class E") has during the last 20 years been based on a three-powder blend of Portland cement, fly ash and silica fume. A typical (2014) mix composition with 20% fly ash and 5% silica fume is given in Appendix 5 [Jensen 2014].

Farø bridges

The construction of the Farø bridges is described in [Gotfredsen et al. 1985]. The concrete castings took place from March 1981 to June 1984. The mix designs for foundations and columns are given in Table 2. Fly ash was added to reduce bleeding [Gotfredsen et al. 1985, p 38].

[5] Data from the Alssund bridge are currently not included in the report due to lack of detailed information

Microstructural observations of concrete extracted in 2012 from the submerged part of the Farø bridges revealed micro-structural zoning from the concrete surface and an uneven and scaled surface. Similar observations were made for almost all the investigated structures in a survey on Danish coastal bridges (Vejle Fjord bridge, Alssund bridge, Farø bridges, Storstrøm bridge, Øresund bridges, Vilsund bridge, Oddesund bridge, Hadsund bridge). [DTI 2012]

The following zoning was observed on the core from the Farø bridges, which was extracted from level -1.5 on the southern side on one of the pillars [DTI 2012]:

- 0-3 mm: Partially carbonated and highly cracked paste, and massive ettringite and thaumasite-like phases in voids
- 3-10 mm: Porous paste with ettringite and thaumasite-like phases in air voids.

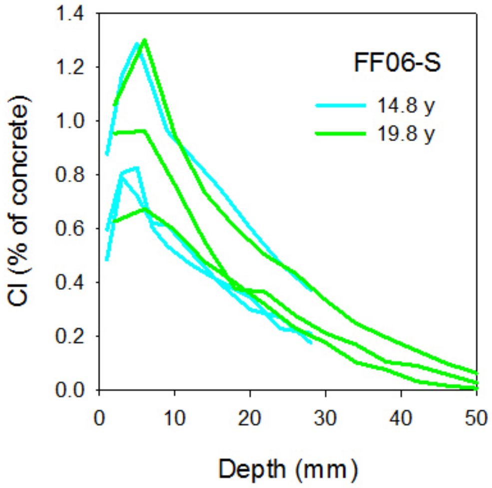
These findings are in agreement with other observations [Eriksen and Buhr 2013]. However, also areas with larger damage degree have been observed. A diver inspection in 2011 of the Farø bridges below sea level revealed that the concrete in one of the piers (sulphate resistant Portland cement and 11% fly ash, an equivalent w/c-ratio, w/c_{eq}=0.40) had partly eroded from the surface to depths up to 20-30 mm after about 30 years in service. And petrographic analysis of cores extracted 2012 showed that the attack had progressed to 10-15 mm from the eroded surface from underwater cores and that the deterioration primarily was caused by sulfate reactions. [Eriksen and Buhr 2013]

A comprehensive report on the long-term performance of the Farø bridges is in print [Jensen et al. 2014]. In addition to the above described erosion due to sulfate reaction, fine, surface parallel cracks were observed to a depth of 20 mm as well as insufficient air entrainment to provide frost resistance. Surface parallel cracks and possible frost damage will facilitate chloride ingress.

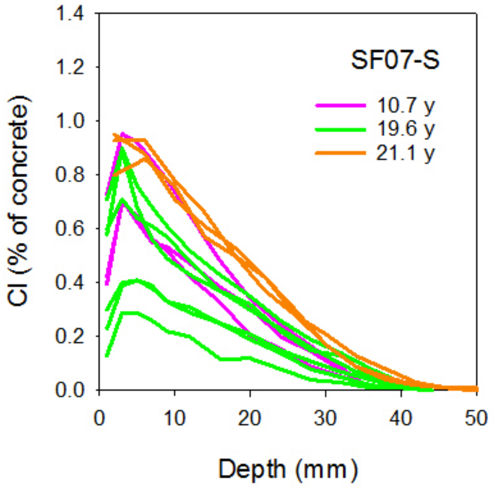
Data on (total) chloride ingress profiles from [Jensen et al. 2006] are presented in Figures 1 and 2:

- Level 0.5 m (splash zone) in pier FF06-S after 14.8 and 19.8 years
- Level 0.35 m (splash zone) in pier SF06- after 7.6 to 15.4
- Level -0.9 to -1.5 m (submerged zone) in pier SF06-S after 10.7 and 22.4
- Level 0.7-0.9 and 1.5 m (splash zone) in pier SF07-S after 15.4 to 21.1.

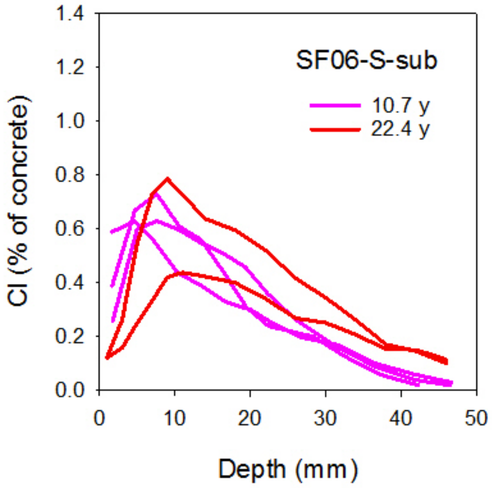
All data are from structural parts of the Farø bridges, i.e. from ingress into “structural concrete” with the concrete mix design given in Table 2 (sulphate resistant Portland cement and 11% fly ash, and an equivalent w/c-ratio, w/c_{eq}=0.40). Generally the profiles illustrate that the maximum chloride concentration is not obtained at the surface, and that both the maximum value and the depth of the maximum chloride content increase with exposure time. The microstructural zoning (phase changes) described above is considered the main course of these observations; see e.g. [De Weerdt et al. 2014].



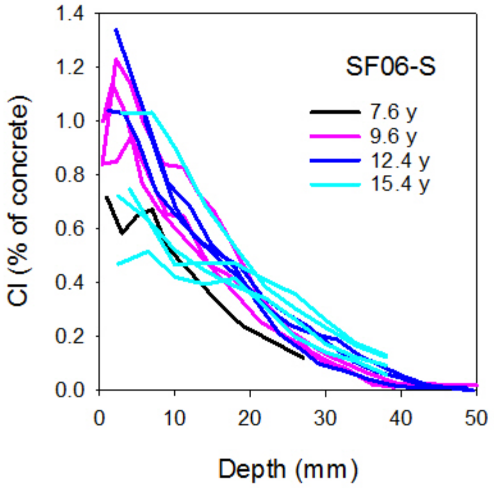
FF06-S: level 0.5 m (splash zone)



SF07-S: 0.7-0.9 and 1.5 m (splash zone)



SF06-S-sub: level -0.9 to -1.5 m (submerged)



SF06-S: level 0.35 m (splash zone)

Figure 1 – Chloride ingress in piers of the Farø bridges measured after 14.8 and 19.8 years of marine exposure (above). All data from [Jensen et al. 2006].

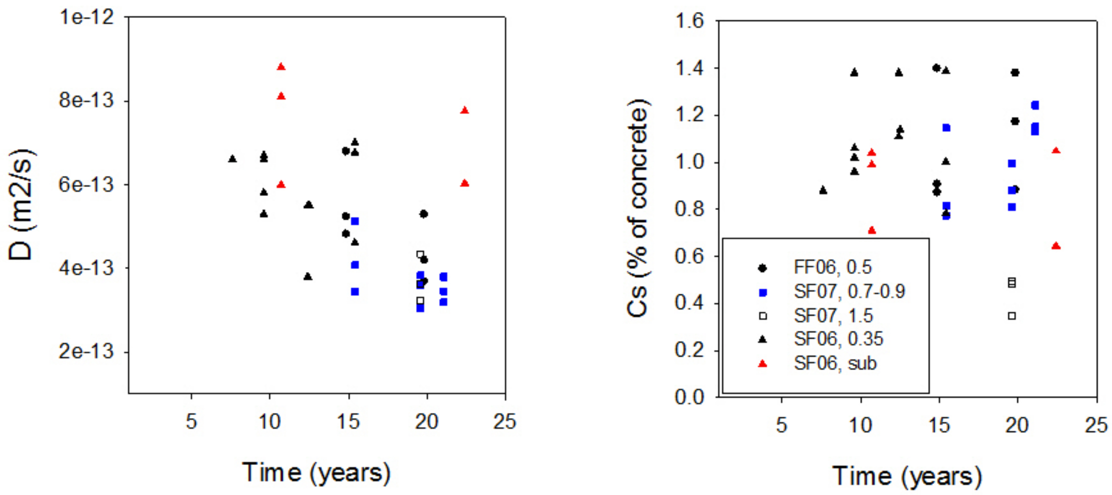


Figure 2 - Calculated apparent chloride diffusion coefficient (D) and surface concentration (Cs) as a function of exposure time (minimum first data point in chloride profiles omitted). Calculated values from [Jensen et al. 2006].

The data illustrate that large variations in chloride ingress might be observed, probably due to combinations of variability in the concrete properties (see profiles in submerged concrete) and the variability of the exposure (splash zone).

Based on the chloride profiles Jensen et al. [2006] calculated the apparent chloride diffusion coefficient and surface concentration using the error-function solution to Fick's second law and omitting surface near data points where reduced chloride content was observed. Changes in the calculated apparent chloride diffusion coefficient (D) and surface concentration (Cs) as a function of exposure time are summarized in Figure 2.

Comparing to data from other Danish costal bridges (total chloride profiles [VD report 198], enclosure 6.1, apparent diffusion coefficient [VD report 198], enclosure 9.1) it can be observed that the apparent diffusion coefficients are comparable, but the total chloride content is higher in the Farø bridges.

Based on investigations undertaken in the period 1988 to 1997 Stoltzner et al. [2000] calculated time to initiation of chloride induced corrosion: 15-40 years from time of construction in the tidal zone (level 0.5 m) and after 25-40 years in the splash zone (level 1.5) assuming a critical chloride concentration at 0.05-0.10% by mass of concrete and 50 mm cover.

To mitigate corrosion, a pilot project using magnetite anodes for cathodic protection was installed. According to Stoltzner et al. [2000] conditions for prevention of corrosion in the submerged and tidal zone were established.

Great Belt Link

In 1998, investigations were undertaken to reassess the service life of the Great Belt Link East bridge with regard to initiation of chloride induced reinforcement corrosion. Based on the measured chloride profiles in cores taken after five years exposure in the tidal zone (concrete type A, see Table 2 for details), ref. Figure 3, and additional statistical data, ref. Table 3, analysis of the probability of initiation of corrosion was carried out. Using the error-function solution to Fick's second law with constant diffusion coefficient, a median time to initiation of corrosion (probability of corrosion at 0.50) of about 150 years was predicted. The DuraCrete model with at time dependent diffusion coefficient and using an aging factor of 0.3 predicted a median time to initiation of corrosion significantly larger than 300 years.

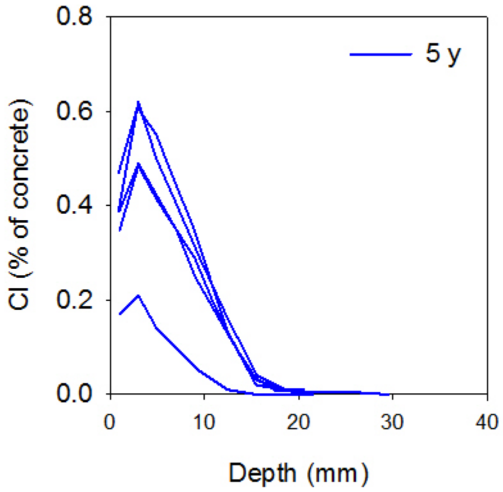


Figure 3 - Chloride content (% by mass of concrete) versus depth from surface after five years exposure in the tidal zone of the Great Belt Link East bridge pier 6. Concrete type A. Data extracted from [Geiker and Vincentsen 1999]

Table 3 - Statistical parameters used [Geiker and Vincentsen 1999].

Property	Unit	Mean	SD
Apparent diffusion coefficient	10^{-13} m^2/s	2.3	0.6
Surface concentration	% by mass of concrete	0.74	0.27
Cover	mm	77.5	5
Critical chloride threshold	% by mass of concrete	0.1	0.25
Ageing factor	-	0.3	0.15

In a presentation from Storebæltsforbindelsen [Laursen 2009] a durability study from 2005 on remaining service life time for different structural parts was presented. Data is also available in [Møller and Andersen 2010]. Data from chloride ingress after 5, 10 and 17 years in the tidal zone of the Great Belt Link East bridge piers are shown in Figure 4. Based on chloride ingress measurements the service life was calculated to *SL*≈100 years for 75 mm concrete cover thickness at level +/-1 m (tidal and splash zone), and 2 m above this zone the service life for the same concrete cover thickness was determined to 200-500 years. The assumptions for these calculations were: chloride ingress by diffusion, data from 0-5 mm from the surface omitted, diffusion coefficient and surface concentration constant, critical chloride concentration, C_{crit}=0.1% of concrete, and constant paste content. For the submerged zone, *SL*≈100 years, was calculated assuming a critical chloride concentration, C_{crit} =0.15% of concrete, however, based on expectations of limited corrosion rate in the submerged parts *SL*≈500 years was estimated. To provide additional protection of the reinforcement in the submerged zone and in the lower part of tidal/splash zone cathodic prevention in form of sacrificial anodes has been installed [Laursen 2014].

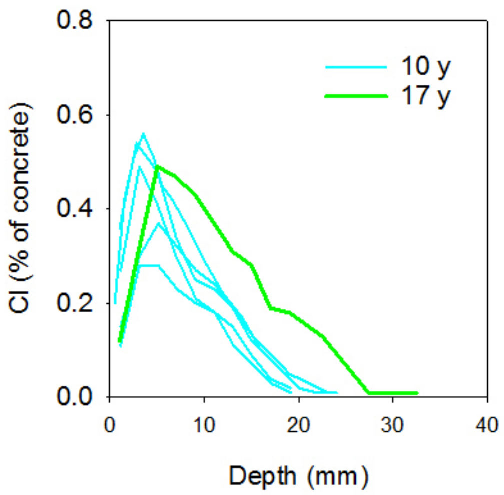


Figure 4 - Chloride content (% by mass of concrete) versus depth from surface after 10 and 17 years exposure in the tidal zone (level 0 to +1) of the Great Belt Link East bridge piers. Concrete type A. Data extracted from [Laursen 2010]

Similar to other marine exposed concrete structures some sea water attack has been observed in concrete in the water line [DTI 2012]. Compared to the Farø bridges, the zoning is limited, 0-3 mm after 17 years. Increased water content (degree of capillary saturation) was measured to a depth of approximately 15 mm at the water line, and in some areas where insufficient air void structure was found frost damage was observed in the outer 0-10 mm [Laursen 2009], [Møller and Andersen 2010].

4. Full scale trials

As part of a survey on Danish "high performance concrete" Bager [2001a] included information on full scale trials with concrete containing fly ash: a fish ladder in Klokkerholm, Madum Å bridge, groynes in Thyborøn, test elements at Aalborg Portland; a road pavement at Aalborg Portland, and a road pavement in Hjørring. The test elements at Aalborg Portland are described in the section on exposure sites. The mix designs of the remaining structures are given in Table 4, and the performance is summarized below. Full scale castings with fly ash containing concretes were also undertaken in connection with the HUA project (Danish Road Directorates bridge no. 470-5019, see Section 6). However, no data on the performance seems to be available.

Table 4 – Mix design of some Danish full scale trials with "high performance concrete" described in [Bager 2001a]

Exposure		Inland, moist	Marine	De-icing	De-icing	De-icing
Structure		Fish ladder, Klokkerholm (one of mixes)	Groynes, Thyborøn	Madum Å	Road pavement, Aalborg Portland	Road pavement, Hjørring
Year [6]		1979	1983		1992	1995
CEM I 42.5 SR	kg/m^3	290	280	320	275	260
Fly ash	kg/m^3	125	80	80	80	55
Silica fume	kg/m^3				28	20
Sand (0-4 mm)	kg/m^3	664	640	490	565	673
Stone	kg/m^3	996 (max 16 mm)	1165 (max 16 mm)	1257 (max 32 mm)	1480 (max 8 mm)	1443 (max 11 mm)
Water	kg/m^3	160	148	128	80	87
w/c	-					
W/(C+0.4FA+2SF)	-	0.47	0.47	0.39	0.22	0.27
W/(C+0.5FA+2SF)	-					
Air	%	4.0	4.7	5.6	1.5	1.5
Compressive strength, 28 days (cylinder)	MPa	45	38	33	99	94

Marine exposure

The project, which included application of fly ash concrete, was initiated by the Danish Coast Authorities in 1983 to improve the durability of Dolos blocks in the groynes at the West Coast of Jutland [Bager in 2001]. According to Bager [2001a] the groynes in Thyborøn were performing that well that fly ash has been used in the production of these Dolos blocks since.

[6] Year of project initiation or construction

Exposure to de-icing salt

Madum Å bridge

The Madum Å bridge was constructed in 1984 for demonstration of the applicability of fly ash in concrete. The construction of the Madum Å bridge is described in [VD Report 5 1985]. Traffic takes place directly on the surface; no membrane is used for protection. According to Bager [2001a] the pavement functioned without any problems in November 2001. A typical concrete composition is given in Table 4; variations are illustrated in Table 5.

Table 5 - Concrete composition, Madum Å bridge [VD Report 7 1993]

		Foundation		Walls				Deck
		South	North	Stage 1, SW	Stage 1, NW	Stage 2, SE	Stage 2, NE	
Cement	kg/m3	250	300	320	310	300	251	320
Fly ash	kg/m3	150	100	80	80	51	51	80
Water	kg/m3	143	143	138	135	133	126	138
Sand	kg/m3	463	482	484	492	576	608	490
Stone	kg/m3	1260	1258	1258	1258	1261	1260	1257
w/c	-	0.57	0.47	0.43	0.44	0.44	0.50	0.43
W/(C+0.5FA)	-	0.44	0.41	0.38	0.39	0.41	0.46	0.38

Investigations undertaken from 1984 to 1993, see Table 6, are reported in [VD Report 1993]. In summary, the investigations indicate a durable concrete. However, an increasing and relatively high amount of micro cracking has been observed on fluorescence impregnated thin sections, see Table 7. It is not known if the development has stabilized; however, already after one year the number of cracks was high. According to [VD Report 1993], the number of cracks observed in 1991 is comparable to the maximum requirements for new concretes.

Table 6 - Investigations (number of samples/tests, Madum Å bridge [VD Report 7 1993]

Structure	1984	1985	1986	1987	1989	1991
Visual inspection	+	+	+	+	-	+
Compressive strength	70				6	2
Chloride ingress, in-situ				3		3
Chloride ingress, lab[7]						3
Micro-structure	10	4	2	2	-	4

Table 7 - Amount of paste and adhesion cracks observed in fluorescence impregnated thin sections (2-4 sections per test). [VD Report 7 1993]

Crack type	1984	1985	1986	1987	1991
Paste cracks	3	64	27	17	66
Adhesion cracks	8	39	22	8	44

[7] 1987 according to [VD report 7 1987], but 1991 according to data sheets obtained from COWI.

The chloride ingress varies, which can be explained by varying exposure to water and de-icing salt. To assess the quality of the concrete, accelerated chloride ingress test (bulk diffusion testing) was undertaken on three cores in 1991. The cores were drilled from the South-West part of the wall (Stage 1 SW, 80 kg/m^3 fly ash); and the cores were extracted under the bridge were de-icing salt had not been used. The original concrete surface was exposed in the laboratory and the chloride ingress profiles measured after 35 days of exposure are shown in Figure 5. Based on the data from 2 mm depth and inwards and assuming no initial chloride content, chloride diffusion coefficients at 1.5-2.0 10^{-12} m^2/s were calculated. [VD Report 7 1993], [Hansen 1993]

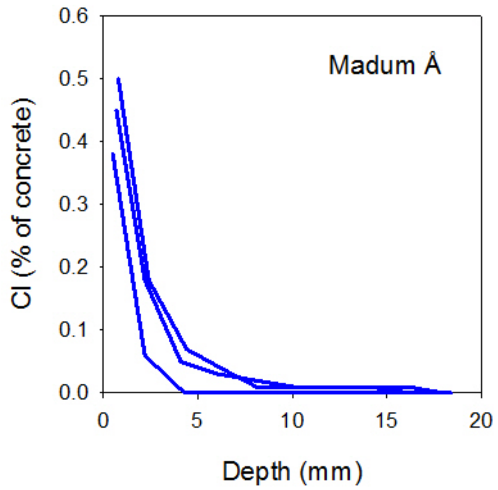


Figure 5 - Madum Å bridge, chloride profiles after accelerated chloride testing for 35 days of cores extracted in 1991 from the South-West part of the wall (Stage 1 SW, 80 kg/m^3 fly ash), data from [Hansen 1993], graphs in [VD Report 1993].

Road pavements

The road pavement at Aalborg Portland road was constructed in 1992 as part of a project on paver compacted concrete for roads. During the years some cracks have appeared in the top layer. This is explained by movements in the underlying layers and the brittleness of the high strength concrete. No other degradation was observed. The road pavement close to Hjørring was constructed in 1995 as a result of the European project ECOPAVE. Similarly to the internal road at Aalborg Portland cracks were observed. [Bager in 2001a]

Inland climate, moist

The project was initiated in 1979 by Aalborg Portland to document durability of concrete with fly ash and silica fume. The fish ladder in Klokkerholm is, among others, cast from a concrete with 30% fly ash and W/(C+0.4FA)=0.47. The fish ladder was in 2001 found to perform well after 22 years exposure to inland conditions and ground water. [Bager 2001a]

5. Field exposure sites

In general, the infrastructure projects do not allow for comparison of the performance of different concrete compositions. In addition to the investigations of edge beams of Karlstrup Mose highway bridge, comparative data can be obtained from other field exposure sites, see Table 8.

Table 8 - Concrete field exposure sites

Field site	Marine	De-icing	Inland	Reference
DTI			9 years	[Jakobsen 2008]
Aalborg Portland				
Hirtshals harbor	25 years			[Bager 2008]
Main road, North Jutland		18 years		[Bager 2001b]
Aalborg Portland			25 years	[Bager 2008]
Karlstrup Mose highway bridge		29 years		[Eriksen 2014]
Rødbyhavn	2 years			[DTI], [Jakobsen 2013]
Träslövsläge	20 years			[Boubitsas et al. 2014], [Tang 2013]

Marine exposure

Träslövsläge field site, Sweden

Approximately 40 concrete slabs differing in binder composition and w/b are exposed to marine environment at Träslövsläge field site, Sweden. The exposure site is shown in Figure 6. Field data up to 20 years are summarized in a recent report [Boubitsas et al. 2014]. The summary of the findings is copied in Appendix 6, and the compositions of the investigated concretes are given in Appendix 7. Cores have been drilled from atmospheric, splash, and submerged zones of the trial panels, see Figure 7.

Figure 6 - Träslövsläge field site, Sweden. Courtesy Dimitrios Boubitsas 2015

Figure 7 – Panel from Träslövsläge field site, Sweden. Position of exposure zones indicated according to [Boubitsas et al. 2014]. Photo courtesy Dimitrios Boubitsas 2015

Combinations of FA and SF were found to reduce chloride ingress. Air entrainment was found to enhance chloride ingress. Some cases of surface damages in form of exposed aggregates were noticed close to the water line (splash zone) [Boubitsas 2014].

Based on combined measurement of corrosion condition and chloride content chloride thresholds of at least 1% of binder were estimated [Boubitsas et al. 2014].

Figures 8-10 summarize chloride ingress data from [Tang 2013] and [Boubitsas et al. 2014] for submerged exposure at Trävslövsläga for concrete mixes with FA as well as other mixes selected for comparison:

- Impact of FA (Aalborg Portland, Denmark) and SF (Elkem Norway), (Figure 8 and 9)
- Impact of water-to-binder ratio (*w/b*) (given as equivalent water-to-cement ratio (w/c_{eq}), efficiency factor 1 for silica fume and 0.3 for fly ash) (Figure 10).

Reduced chloride ingress can be observed in Figure 7 for concretes with FA and/or SF (assuming DK-cement comparable to Swedish Anl-cement (no slabs with mixes with the Danish cement 9-40, 10-40 and 11-35 were left for the investigation after 20 years [Boubitsas 2014]). The impact of FA seems to increase with time. The maximum chloride concentration and depth of maximum (peak) increase (at least) the first 5 years for the two- and three-powder mixes.

11-35
85% DK +10% FA 5% SF
w/c_{eq}=0.35

10-40
78.5% DK +17% FA 4.5% SF
w/c_{eq}=0.40

9-40
95% DK + 5% SF
w/c_{eq}=0.40

1-35 and 7-35
100% AnI
w/c_{eq}=0.35

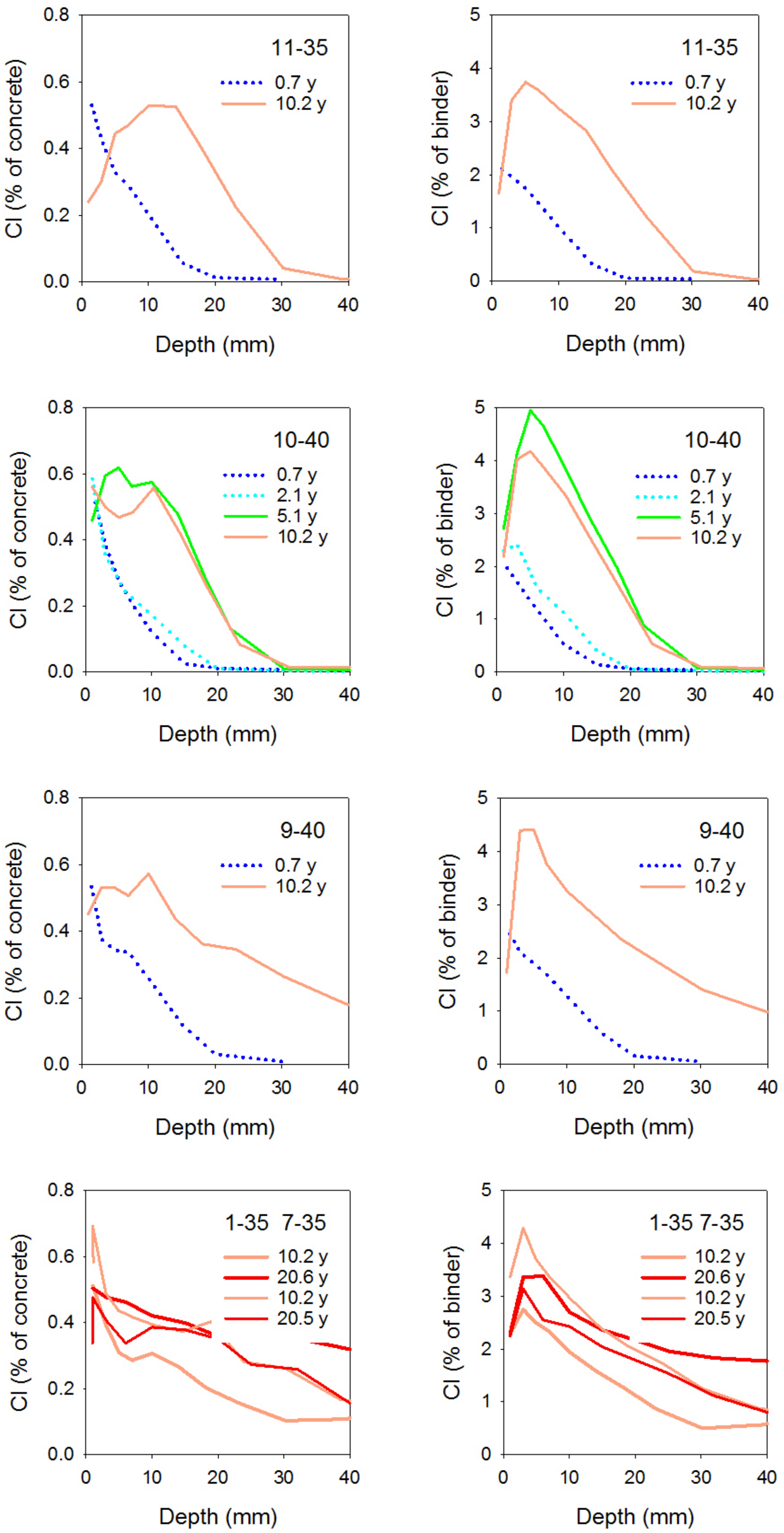


Figure 8 - Impact of FA and SF on chloride ingress. Submerged exposure at Trävslövsläga. Chloride ingress profiles. Chloride in mass % of concrete (left) and binder (right); time in years. w/c_{eq} assuming efficiency factor 1 for silica fume and 0.3 for fly ash. Data from [Tang 2013] and [Boubitsas et al. 2014].

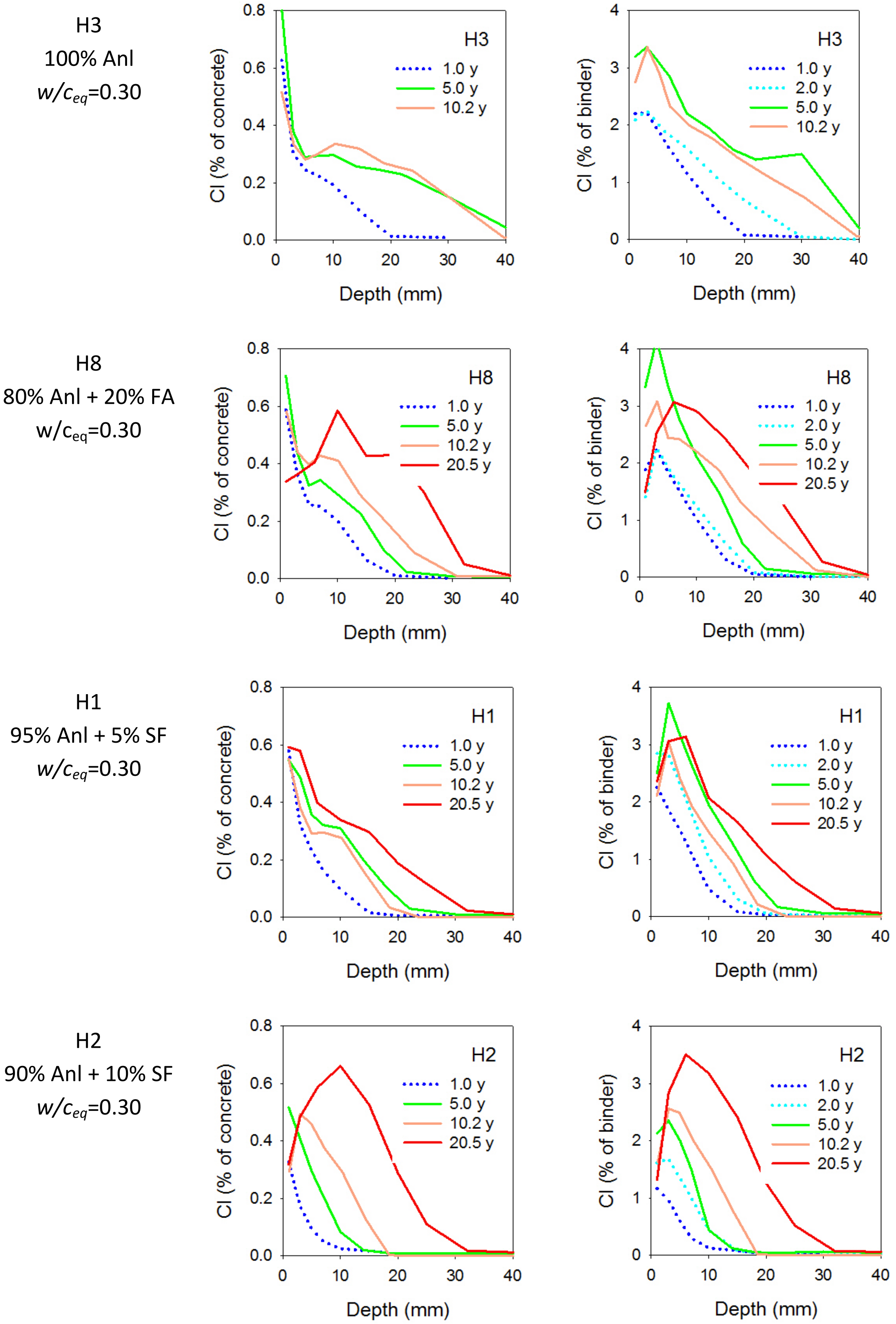


Figure 9 - Impact of FA and SF on chloride ingress. Submerged exposure at Trävslövsläga. Chloride ingress profiles. Chloride in mass % of concrete (left) and binder (right); time in years. w/c_{eq} assuming efficiency factor 1 for silica fume and 0.3 for fly ash. Data from [Tang 2013] and [Boubitsas et al. 2014].

Similar trends for the impact of FA and SF (reduced ingress for concretes with FA or SF; increasing impact of FA with time) can be observed in Figure 8. However, the maximum chloride concentration appears, expect for H2 (90% ANL+10% SF), stable after 2-5 years. The depth of maximum chloride content (peak) seems to increase over a longer period for blends.

For comparison, the impact of *w/b* on chloride ingress is illustrated in Figure 10. As expected increased ingress depth is observed for increasing *w/b*. The maximum chloride concentration appears stable after a few years. The depth of maximum chloride content (peak) seems to increase over a longer period. Note that no long term data are available for w/c_{eq}=0.4.

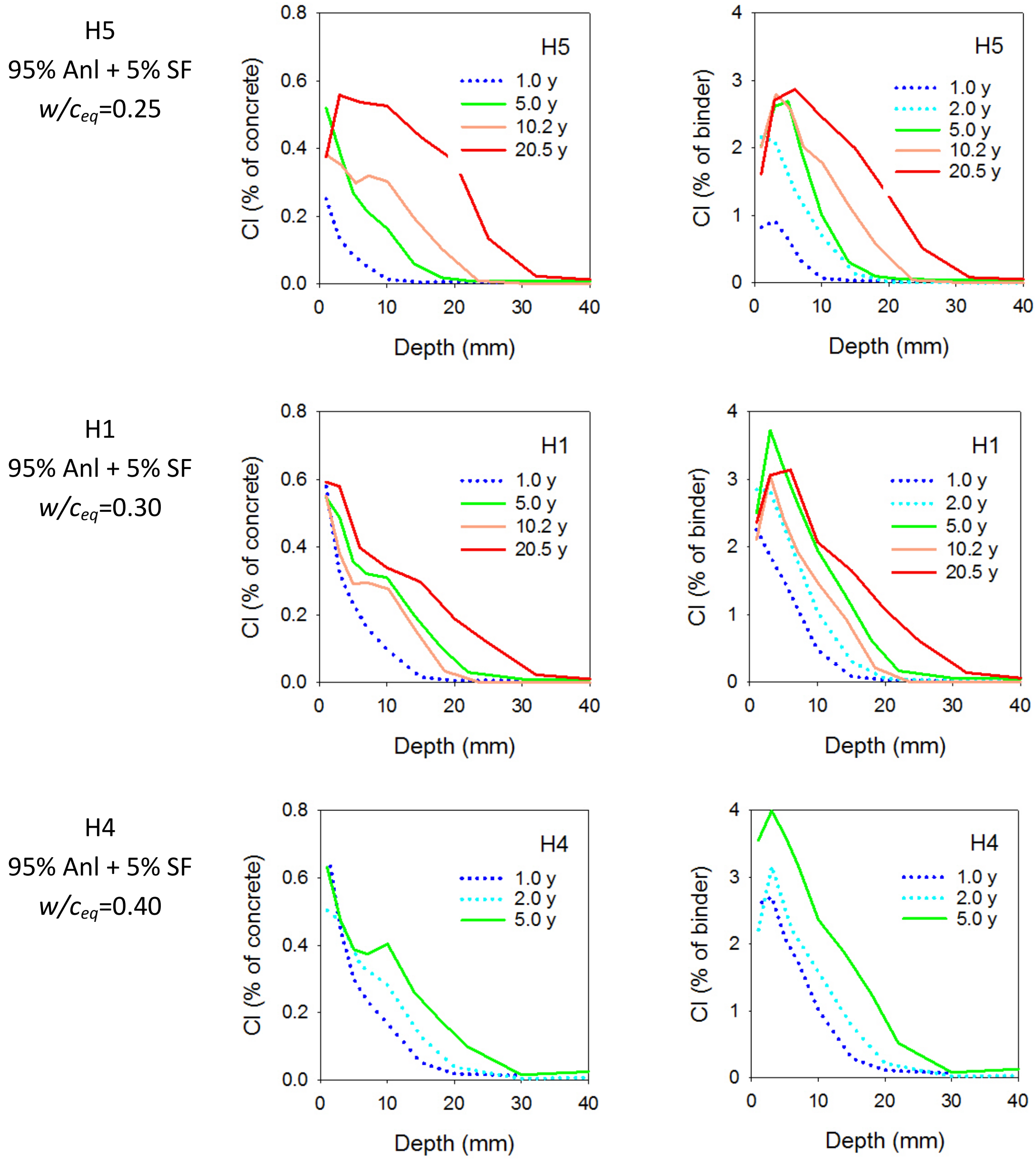


Figure 10 - Impact of w/b on chloride ingress. Submerged exposure at Trävslövsläga. Chloride ingress profiles. Chloride in mass % of concrete (left) and binder (right); time in years. w/c_{eq} assuming efficiency factor 1 for silica fume and 0.3 for fly ash. Data from [Tang 2013] and [Boubitsas et al. 2014].

Hirtshals harbor

The Aalborg Portland field exposure sites cover different environments: marine, de-icing salt, and inland climate. The marine exposure site is illustrated in Figure 11.

Figure 11 – Aalborg Portland exposure site at Hirtshals harbour.
Element being lifted out of rag. Courtesy Dirch Bager 2015

The performance of sixteen concrete mixes exposed for 18 years to marine environment, de-icing salt, and inland climate is described in [Bager 2001]; 25 years' performance in marine environment (and inland climate) is described in [Bager 2008]. One of the 16 mixes contained 17% FA (CEM I 42.5-SR, W/(C+0.4FA)=0.4), six of the mixes contained 25% FA (CEM II/B-V 42.5, based on CEM I 42.5).

The impact of fly ash and silica fume on mitigation of alkali silica reactions was tested indirectly as alkali reactive sand was used unintentionally. Due to ASR damage, no chloride profiles could be measured after 18 years (and 25 years) in concretes with rapid hardening Portland cement (CEM I 42.5) alone.

Chloride ingress after 8.5 years appears comparable in concretes with and without fly ash. However, whereas continued ingress was observed in the (few) concrete with CEM I 42.5-SR, the ingress in the concretes with fly ash (CEM I 42.5-SR plus fly ash and CEM II/B-V 42.5) seems to stagnate.

None of the concrete elements exposed to marine environment suffered from frost damage. (The ASR damaged concretes could have been affected by frost; however, these were made from plain Portland cement which does not indicate frost susceptibility of concrete with fly ash).

Based on the investigations Bager [2008] concluded that FA (and SF) improves the long-term durability with regard to ASR and chloride ingress.

Detailed microstructural investigations of selected concretes were undertaken by Chabrelie et al. [2008]; however, none of the investigated concretes contained fly ash. The investigations showed zoning of the marine exposed concrete: an outer magnesium rich zone, followed by a sulphate rich zone and a deeper ingress of chlorides; leaching was observed to several mm. The zoning is similar to observations made on concrete samples from e.g. Østmarknadset, Trondheim fjord [De Weerdt et al. 2014].

Femern Belt exposure site, Rødbyhavn

The performance of concrete slabs made from 18 different mixes exposed for two years at the Femern Belt exposure site, Rødbyhavn, is described on DTIs homepage[8] and in [Jakobsen 2013]. Three of the concrete mixes contain fly ash, ref. Table 9. Concrete D was a self-compacting concrete.

Table 9 - Concrete compositions Femern Belt exposure site, Rødbyhavn. After DTI's homepage[9]

	Density (assumed) kg/m^3	Unit	Concrete			
			A	B	C	D
Cement CEM I 42.5 N (low alkali sulphate resistant)	3100	kg/m^3	356	322	300	336
Fly ash (EN 450-1 N)	2200	kg/m^3	-	57	100	112
Water	1000	kg/m^3	146	140	140	157
Paste content		m^3/m^3	0.261	0.270	0.282	0.316

The chloride ingress in these concrete after two years is illustrated in Figure 12. Each graph represents one core. The following observations can be made: the maximum total amount of chloride is much higher in concrete mixes with fly ash and the profiles are steeper (indicating a lower transport coefficient), especially for the concrete in the submerged zone. However, it should also be noted that the profiles vary substantially between cores from the submerged zone of the three mixes with fly ash. As the calcium content in the binders varies, the Cl/Ca ratios only indicate the Cl/binder content. Data are pt. not available for the binder compositions, but an estimate is that data for the fly ash would be decreased by 20%. In addition leaching resulting in a decreased Ca content in the outer surface should be taking into account [De Weerdt et al. 2014]. Leaching may cause a substantial reduction of the Ca content in the surface near region. Also, the data from the outermost millimetres might be affected by a calcium carbonate crust [De Weerdt et al. 2014].

The data from the Femern Belt exposure site were obtained using SEM/EDS. De Weerdt et al. recently showed that the actual values obtained are method dependendt, see Figure 13 where Cl/Ca mass ratios obtained by SEM-EDS is compared with ratios obtained using ICP-MS. Both methods indicate the same trend, but there is a difference between the actual values obtained. This might be due to the removal of chlorides during the preparation of the polished sections. [De Weerdt et al. 2013b]

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8 http://www.concreteexpertcentre.dk/32447

9 http://www.concreteexpertcentre.dk/30664,2

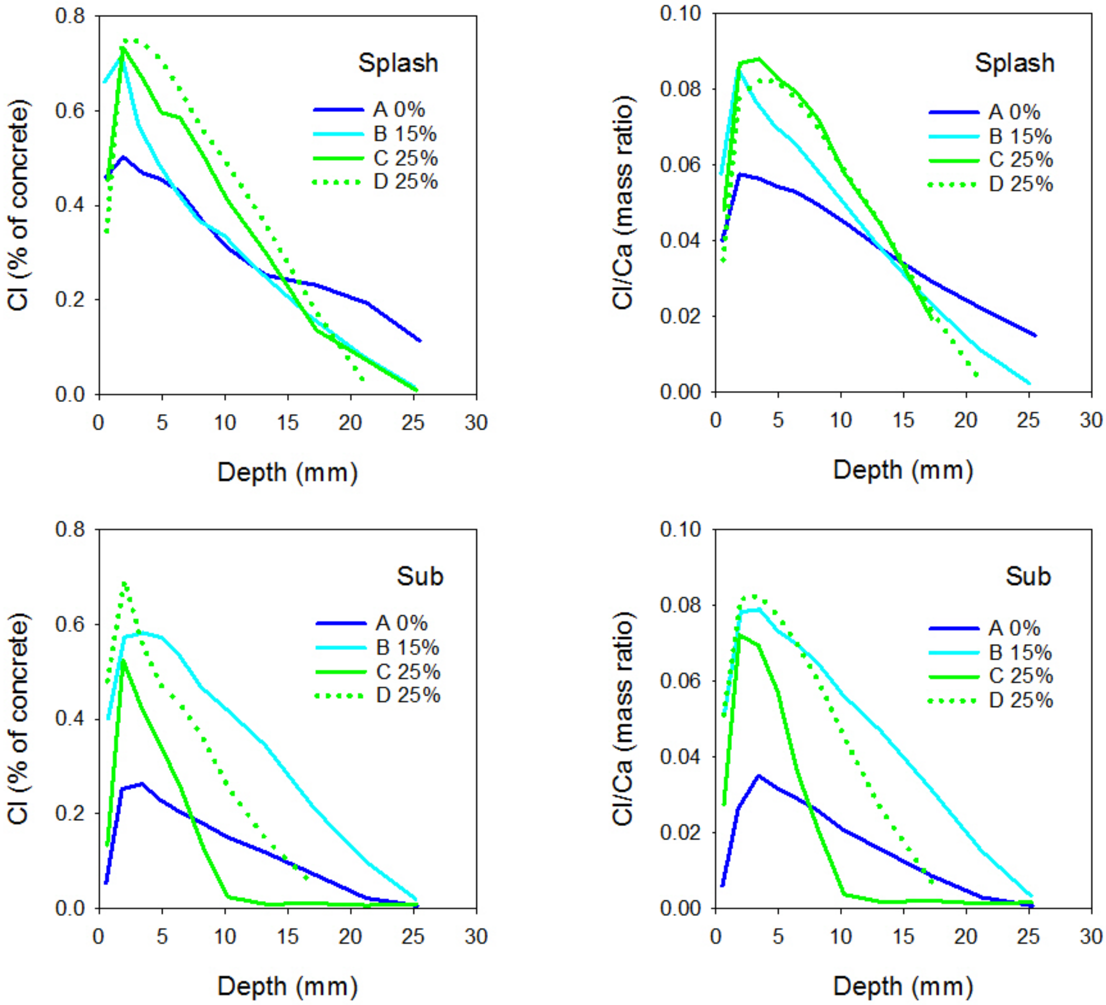

Figure 12 - Chloride ingress after 2 years in splash (upper) and submerged (lower) zone, Femern Belt exposure site, Rødbyhavn. Left: percentage of concrete; right: Cl/Ca mass ratio. Data extracted from DTIs homepage. [DTI 2012b]

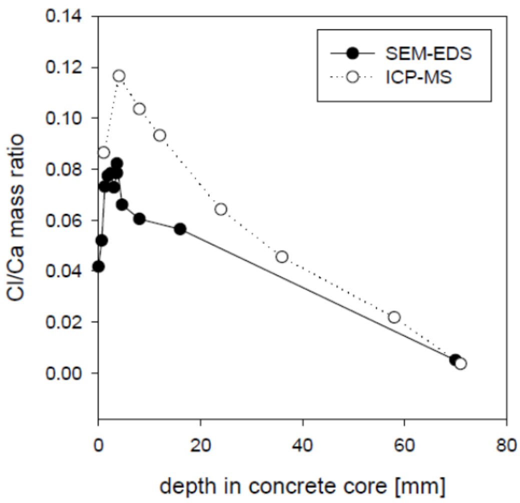

Figure 13: Cl/Ca mass ratio determined by SEM-EDS and ICP-MS as function of the depth in the concrete core. [De Weerdt et al. 2013]

Observation from optical microscopy revealed that the surfaces of the concrete, except for Concrete A and B, all showed signs of weak scaling after 2 years of marine exposure [Jakobsen 2013]. SEM-EDX analysis showed that the surfaces can be divided into three chemical zones: a magnesium rich zone, a sulphur rich zone and a chloride rich zone [Jakobsen 2013]. The zoning is similar to observations made on concrete samples from e.g. Østmarknadset, Trondheim fjord [De Weerdt et al. 2014] and is part of the explanation for the time dependent change in surface concentration and transport coefficient.

Exposure to de-icing salt

Aalborg Portland's exposure site

The performance of sixteen concrete mixes exposed for 18 years to marine environment, de-icing salt or inland climate is described in [Bager 2001b]. One of the 16 mixes contained 17% FA (CEM I 42.5-SR, 17% FA, W/(C+0.4FA)=0.4), six of the mixes contained 20-25% FA (CEM II/B-V 42.5). Of each mix one horizontally placed element was subjected to de-icing salt during winter. The elements were placed on a field at Aalborg Portland together with exposed other elements which were exposed to inland climate (see below). Performance in marine environment is described above.

Surface cracks were observed in three of the concretes without pozzolans and one with pozzolan. These cracks were, however, attributed to ASR. Apparently, none of the concretes suffered from noticeable frost damage; only limited surface scaling was observed on few of the concrete elements exposed to de-icing salt. Except for the ASR affected concretes and one concrete with Portland cement alone, the concretes with more than 3% air appear frost resistant independent of w/b and binder type. [Bager 2001b], [Bager 2014]

Edge beams of Karlstrup Mose highway bridge

In continuation of the Madum Å bridge project (see Section 3) selected fly ash concretes were further tested in de-icing environment. A series of edge beams varying in cement and fly ash content and in equivalent water-to-cement ratio (W/(C+0.5FA)) were cast in 1984 and placed next to the motorway on Karlstrup Mose highway bridge (bridge 10-0054 UF of Karlstrup Mose), see Figure 14. The main purpose was to investigate the freeze/thaw durability of concrete with fly ash. The upper surface of the edge beams is almost horizontal and in level with the road resulting in an extensive exposure to de-icing salt in the winter period from the road which has a high traffic load. Sodium chloride has primarily been used as de-icing salt. [Eriksen 2014]

In total 21 edge beams were produced from seven different concrete compositions with 20 to 60% fly ash of the cement weight and equivalent water-to-cement ratio (W/(C+0.5FA)) from 0.34 to 0.46. The cement types used was similar to the cement used for the Farø bridges (PC(A/L/S) from Aalborg Portland with approx. 0.3% eq. alkali content and 1% C_3A). The mix compositions are given in Table 10. [Eriksen 2014]

Figure 14 - Karlstrup Mose highway bridge. Edge beams containing varying amounts of fly ash. Photo courtesy Kirsten Eriksen

Table 10 – Concrete compositions, edge beams of Karlstrup Mose highway bridge [Eriksen 2014]

		ID							
	Unit	21	44	45	46	37[b]	43	36[b]	Ref[c]
Cement	kg/m3	250	300	350	300	300	250	250	?
Fly ash	kg/m3	150	100	100	150	100	50	50	
Water	kg/m3	131	135	137	135	137	128	123	?
W/(C+0.5FA)[a]	-	0.40	0.39	0.34	0.36	0.39	0.46	0.45	0.42[d]
Air content (fresh con.)	Vol. %	6.4	7.2	6.9	6.5	7.0	6.5	6.5	≈4
Compressive strength at 56 days	MPa	48	42	52	56	44	45	46	?

[a] Calculated based on actual composition

[b] Fly ash from Fynsverket, fly ash from Enstedverket in remaining mixes

[c] Based on thin section analysis the reference mix seems to contain some silica fume

[d] Based on thin section analysis

After approximately 30 years of exposure to de-icing salt the edge beams were investigated. The inspection covered visual inspection, macro- and micro analysis (petrographic analysis) and chloride analysis on drilled cores. The cores were drilled vertically from the upper surface. [Eriksen 2014]

The visual inspection and the petrographic analysis showed that generally only surface scaling (of 'cement skin' or outer mortar layer) was observed, and only on the upper surface. An exception is mix 36, were scaling was observed at one edge towards the road. Lowest damage was observed for the reference mix (which seems to contain silica fume), second lowest for mix 46 (C/FA=2/1). The degree of weathering increased with decreasing strength. There seems not to be a direct correlation between degree of weathering and fly ash content or water-to-binder ratio. The surface scaling appears to be due to freeze/thaw action, no deeper crack development was observed.

Comparing thin sections from investigations in 1995 and 2014 it was found that no apparent change in denseness of the bulk concretes had occurred from 10 to 29 years of age. The apparent water-cement- ratio determined by thin section analysis was in general comparable to the equivalent water-to-cement ratio (W/(C+0.5FA)) calculated from the mix compositions.

Chloride profiles measured after 29 years exposure are shown in Figure 15. One core of each concrete composition was tested. It can be observed that the chloride ingress is deepest for the reference mix and for the two mixes with highest equivalent water-to-cement ratio.

Based on the investigations, Eriksen [2014] concludes that observations not only support the use of FA according to today's regulations (33% FA of cement weight), but also indicates that FA in amounts of 50-60% of cement weight can be used for concrete exposed to de-icing salt. It should, however, be mentioned that the many variations in constituents limit the possibilities for drawing conclusions regarding the impact of fly ash alone. [Eriksen 2014]

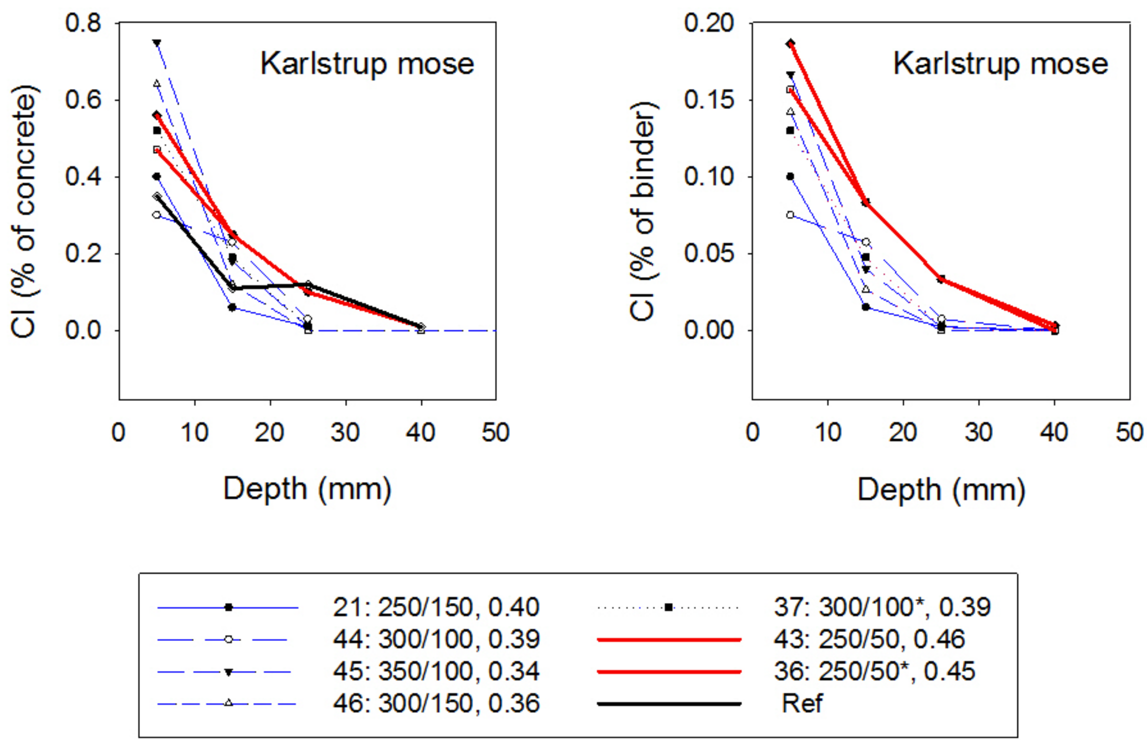

Figure 15 – Chloride ingress after 29 years in edge beams exposed to de-icing salt. Left: % of concrete, right: % of binder, assuming constant paste content. The edge beams vary in cement and fly ash content (C/FA) and equivalent water-to-cement ratio (W/(C+0.5FA)); * FA from different source), see Table 10. Data from [Eriksen 2014]

Inland climate

DTI's exposure site

Concrete blocks from nine different concrete mixes were stored in outdoor water tanks for eight years followed by six years at exposure to inland climate at DTI storage place. After the 14 years exposure the concrete blocks were subjected to visual inspection and cores extracted for petrographic investigation [Jacobsen 2008]. The concrete varied in composition: CEM I 42.5 - SR with and without FA and/or SF, and slag cement. All concretes performed well on a macroscopic level. No signs of deterioration were observed in two of nine concretes: OPC and Great Belt mix design. Concretes with combinations of FA and SF all showed signs of casting defects in form of small surface near cracks; and most had microcracks in the surface resulting in minor scaling. All slag concretes (CEM III/A) had deeper carbonation depth and increased porosity in the carbonated zone.

Aalborg Portland's exposure site

The performance of sixteen concrete mixes exposed for 25 years to marine environment or inland climate is described in [Bager 2008]. One of the 16 mixes contained 17% FA (CEM I 42.5-SR, 17% FA, W/(C+0.4FA)=0.4), six of the mixes contained 20-25% FA (CEM II/B-V 42.5). The elements exposed to inland climate were either placed horizontally or vertically on a field at Aalborg Portland (for each mix one of the horizontally placed elements was subjected to de-icing salt during winter, see above).

Surface cracks were observed in three of the concretes without pozzolans. The surface cracks were, however, attributed to ASR. Apparently, none of the concretes suffered from noticeable frost damage when exposed to moist conditions without de-icing salt. (Also, no frost damage was observed in marine exposed concrete, and only limited surface scaling was observed on few of the concrete elements exposed to de-icing salt.)

The depth of carbonation on vertical surfaces after 25 years inland exposure at Aalborg Portland exposure site is illustrated in Figure 14 for all 16 mixes (all mixes containing fly ash are marked as CEM II/B-V). Water-to-binder ratio appears to be the main controlling parameter for carbonation.

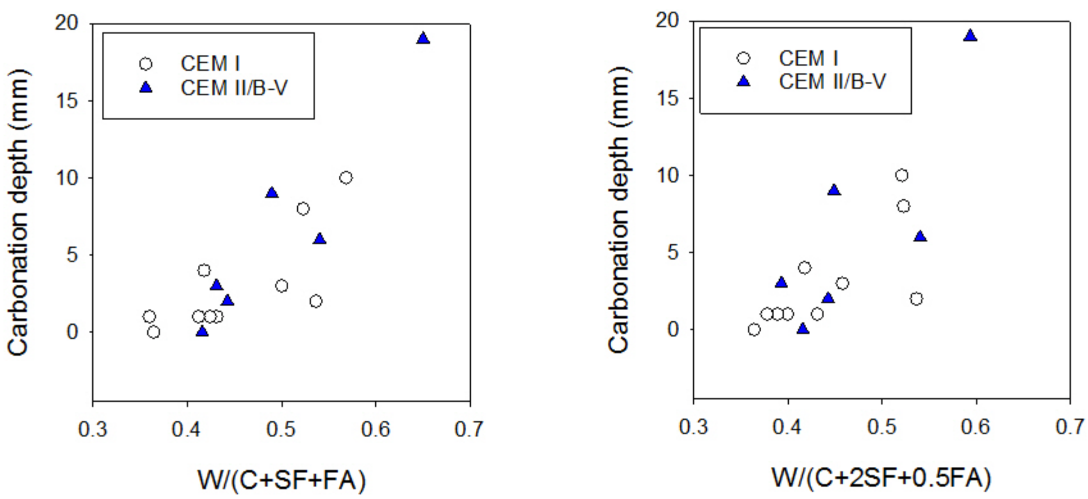

Figure 14 –Depth of carbonation on vertical surfaces after 25 years inland exposure at Aalborg Portland exposure site. CEM I or CEM II/B-V with or without silica fume. Data from [Bager 2008]

6. Construction

Two major projects focusing on the contractor's technology and performance testing were undertaken in the 1990'es in Denmark: HUA, Højkvalitetsbeton til udsatte anlægskonstruktioner and HETEK, Højkvalitetsbeton, entreprenørens teknologi. As part of HUA trial casting were made, among others, to investigate air voids stability and durability using established performance tests supplemented with investigations of macro-, micro and air void structure [Geiker 1995a]. The applicability of the performance tests and the time of testing can be debated, and data will not be summarized here. The analyses of macro-, micro and air void structure revealed that several of the concretes contained a substantial amount of microdefects, especially concretes cast in high walls and with *w/c*<0.4. In addition were the standard requirements to air voids structure in hardened concrete not met (min 8% and max 20% entrained air content cementing matrix and min 25 mm^{-1} specific surface).

Full scale castings were undertaken with three concretes, all three-powder mixes [Geiker 1995b]. The concretes were used for the casting of the Danish Road Directorate's bridge no. 470-5019. Main conclusions from the castings were that trails castings should include optimization of vibration (duration and methods) and investigation of the impact of execution on the air content in the final structure. According to Eriksen [2014] problems with incapability between binders and admixtures, and between different types of admixtures seems to be solved.

Concretes with a high amount of fines and a low w/b are more prone to premature drying and plastic shrinkage cracking. In the HETEK project requirements to moisture curing/protection were set up depending on the amount of fly ash and silica fume and on the w/b. [VD-report 124]

7. Summary

The report focuses on Danish experience regarding chloride ingress in concrete containing fly ash, but general experience with production and durability of fly ash concrete is also included.

Fly ash has been used in concrete production in Denmark since the mid 1970'es. The typical concrete mix for extra aggressive environment ("class E") has during the last 20 years been based on a three-powder blend of Portland cement, fly ash and silica fume (typically 15-20% FA and 5% SF).

In general, reduced chloride ingress can be observed for concretes with fly ash and/or silica fume compared to plain Portland cement concretes. The impact of fly ash becomes more noticeable after several years. Data from the Femern Belt marine exposure site indicate that the fly ash content should be relatively high (25% vs 15%) to reduce chloride ingress. This is supported by data from the Farø bridges (11% FA).

Zoning is observed in marine exposed concrete. Observations from one pier of the Farø bridges (11% FA) indicate that marine exposure might result in substantial scaling due to phase changes. Damage to the same degree has not been reported elsewhere.

The zoning affects the chloride profiles. Data illustrate that the maximum chloride concentration is not found at the outer most surface, and that in general both the maximum value and the depth of the maximum chloride content increase with exposure time. This time dependent behaviour should be taken into account when predicting service life.

Data from concrete elements with reactive aggregates exposed to marine exposure in Hirtshals harbour for 25 years illustrate a mitigating effect of fly ash (17% and 25% FA) on alkali silica reactions.

No frost damage was observed in concrete trial elements with and without fly ash exposed to marine environments for 25 years, and only limited surface scaling was observed on few of the concrete elements exposed to de-icing salt. This is supported from edge beams from Karlstrup Mose highway bridge, where only limited surface scaling was observed on horizontal surfaces after 29 winter's exposure to de-icing salt. The importance of a satisfactory air void structure is illustrate by indications of frost damage in the outer 0-10 mm at the water line in areas with insufficient air void structure. According to Eriksen [2014] earlier experienced problems with incapability between binders and admixtures, and between different types of admixtures seems to be solved.

8. Acknowledgements

The willingness of colleagues in Denmark (Mette E. Andersen, Dirch Bager, Carola Edvardsen, Kirsten Eriksen, Ulla Hjorth Jakobsen, Birit Buhr Jensen, Ib Bælum Jensen, Ulf Jönsson, Erik Stoklund Larsen, Ernst Laursen, Peter H. Møller) and Sweden (Dimitrios Boubitsas, Tang Luping) to share data and experience is greatly acknowledged. Comments from Klaartje De Weerdt, NTNU, are also greatly appreciated.

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Appendix 1

Kirsten Eriksen and Marianne Tange Hasholt: Fly ash in production of high performance concrete in Denmark COWI Report for Emineral a/s, August 2009. The summary of the report is copied here; the full report can be downloaded from http://www.emineral.dk/datablade.aspx

"In Denmark, fly ash has been used as a supplementary cementitious material in concrete production since the 1970es. Fly ash in infrastructure projects dates back to the same decade as the Danish Road Directorate at an early stage offered that selected small bridges and bridge elements like edge beams could be used for performance trials. From ca. 1980 fly ash has also been used in large infrastructure projects, e.g. Farø bridges (1981-1984), Guldborgsund tunnel (1986-1988), and Great Belt Link (1988-1998), alone or in combination with micro silica.

The use of fly ash has developed through the years with varying amounts in different concrete types. This fact makes direct comparisons of results between projects difficult due to many parameters involved. However, in general the experience with use of fly ash is positive, and follow-up investigations in the years after execution point to a satisfactory service life (assuming chloride ingress is the governing parameter). Today, in Denmark and worldwide, use of fly ash and other supplementary cementitious products as micro silica and blastfurnace slag is considered necessary in most projects, when proportioning durable concrete for harsh environments.

In the beginning, the motivation for using fly ash in concrete production was primarily to reduce heat development and risk of cracking, as well as economic aspects in this connection. Also, it was considered to some degree to extend service life by reducing chloride ingress.

Today, environmental awareness is growing, and this adds new lines to the list of arguments in favour of fly ash in concrete. From an environmental point of view, use of fly ash in concrete production is a good alternative to disposal of what would otherwise be a waste product, and when fly ash is used as cement replacement, it lowers the environmental impact from cement production, including CO2 emission.

Experience from full-scale projects, combined with Danish and international studies, form a sound basis for further fly ash use. This publication (in chapter 5) gives a review on how fly ash influences concrete properties and concrete quality, as it is documented in literature. Fly ash affects a variety of fields in concrete technology; from workability of fresh concrete over heat development in the hardening concrete to strength and durability of the hardened concrete.

Today, specifications for high performance concrete often require 100-120 years of service life (or even longer). This implies that such concrete

- has low permeability to reduce chloride ingress and depth of carbonation,
- will withstand chemical attack,
- does not develop alkali silica or other internal reactions,
- is resistant to freeze-thaw attack in cold climates.

Furthermore, the concrete should

- be workable and compactable to prevent problems with execution,
- show low shrinkage and low heat development during hardening to prevent thermal cracking and reduction of design strength.

With fly ash addition, benefits as above can be obtained in the fresh, hardening and hardened state for most properties. However, use of fly ash requires special attention for properties such as strength development and freeze-thaw resistance. Referring to the latter, unburned carbon in fly ash will reduce the effect of most air entraining agents, resulting in problems with obtaining a satisfactory air void system if the unburned carbon is too high.

For low variation in concrete properties during production it is of utmost importance that concrete constituents are of uniform quality. This also applies to fly ash. Fly ash uniformity is influenced by raw material (coal and possibly cocombustion materials), combustion facility at the power plant and production related factors as e.g. load variations.

For larger projects using high performance concrete it is recommended that source and homogeneity of fly ash for the specific project is secured to minimize variations. This can be specified in the design phase, which shall also consider applicable pre-testing methods and time of testing."

Appendix 2

Betonbro – Generalnote – GN-P", 10th April 2014, Danish Road Directorate

http://vejregler.lovportaler.dk/SearchResult.aspx?t=%2fV1%2fNavigation%2fTillidsmandssystemer%2fVejregler%2fUdbud%2fbygvaerker%2f .

Betonbro - Generalnote - GN-P

af 10. april 2014
(Vejdirektoratet)

Se originaldokumentet her:

GRUNDLAG

Broer. Vejledning til belastnings- og beregningsregler, Vejregelrådet - Vejdirektoratet.

Nationale annekser, brospecifikke Eurocodes:

DS/EN 1990/A1 Annex A2 Anvendelse for broer inkl. DK NA

DS/EN 1991-2 Last på bygværker Del 2 Trafiklast på broer inkl. DK NA

DS/EN 1992-2 Betonkonstruktioner Del 2 Betonbroer - Dimensionerings- og detaljeringsregler inkl. DK NA

DS/EN 1997-1 Geoteknik - Del 1 Generelle regler inkl. DK NA

MÅL

Koter (DVR90) stationer, koordinater (DKTM), og vejradier er i meter.

Øvrige ubenævnte mål er i mm.

Vinkler er som hovedregel i nygrader (gon).

DIREKTE FUNDERING

Udrænet forskydningsstyrke		**Friktionsvinkel**	**Effektiv kohæsion**	**Sætninger (forudsatte)**	
Understøtningslinje	$C_{u,k}$	$\varphi_{pl,k}$	c	Totale	Differens
-	kN/m^2	Grader	kN/m^2	mm	mm

PÆLEFUNDERING

Pæledimension	**Trykpæl (+) Trækpæl (-)**	**Regningsmæssig bæreevne**	**Pælekobling**
mm	-	kN	-

FRIKTIONSMATERIALER

	Friktionsvinkel $\varphi_{pl,k}$	**Uensformighedstal U=d60/d10**	**Komprimeringsgrad (Vibration) T**
Gruspudefundering			
Friktionsfyld			

BETON (KONSTRUKTIONSOPDELING)

Konstruktionsdel	**Miljøklasse iht. DS/EN 1992-1-1 DK NA**	**Eksponeringsklasse iht. DS/EN 206-1**
Kantbjælker Søjler ved kørebaneareal Vederlag ved kørebaneareal	E	XD2, XD3, XS3, XF4, XA3
Brodæk Søjler > 3m fra kørebaneareal (> 5 m for sporbærende broer) Vederlag > 3m fra kørebaneareal (> 5 m for sporbærende broer) Vægge Fløje Sætningsplader Pæle Fundamenter	A	XD1, XS1, XS2, XF2, XF3, XA2

SLAP ARMERING

Identifikation[1]	Styrke		Duktilitet		Forankring		Koldbukning[3]		Svejselighed
	f_{yk} (MPa)	f_{yck} (MPa)	ε_{uk} (%)	(f_t/f_y) k	Klasse	f_R[5] relativ ribbeareal	Ø ≤ 16 mm D	Ø > 16 mm D	
R [2]	≥ 235	≥ 235	≥ 5,0	≥ 1,08	B	-	≥ 2Ø (Ø ≤ 12 mm)	≥ 3Ø (Ø > 12 mm)	+
N	≥ 500	≥ 440	≥ 2,5	≥ 1,05	A	≥ 0,056	≥ 4Ø	≥ 7Ø	÷
K	≥ 500	≥ 440	≥ 5,0	≥ 1,08	B	≥ 0,056	≥ 4Ø	≥ 7Ø	÷
Y	≥ 550	≥ 440	≥ 5,0	≥ 1,08	B	≥ 0,056	≥ 4Ø	≥ 7Ø	÷
Z	≥ 500	≥ 400	≥ 7,5	≥ 1,15 <1,35	C	≥ 0,056	≥ 4Ø	≥ 7Ø	÷
F [4]	≥ 550	≥ 440	≥ 5,0	≥ 1,08	B	≥ 0,056	≥ 4Ø	≥ 7Ø	÷
BSV [6]	≥ 500	≥ 400	-	-	-	≥ 0,056	≥ 4Ø	≥ 7Ø	hæfte

[1] Signatur er anvendt på tegninger.

[2] Konstruktionsstål (S235). Værdien gælder for Ø ≤ 16 mm.

[3] For rundjern (R) sker springet i bukkediameter ved 12mm.

Afvigelser fra skemaets bukkediametre, nødvendiggjort af konstruktive hensyn, er angivet på armeringstegningerne.

Tilbagebukning er kun tilladt, hvis det for en given bukkediameter er dokumenteret, at armeringen fortsat opfylder de angivne krav efter tilbagebukning.

[4] Rustfri armering (F) skal leveres som en af følgende kvaliteter: 1.4436, 1.4429, 1.4571 eller 1.4462 iht. DS/EN10088-1.

[5] Relativ ribbe areal f_R beregnes iht. DS/EN ISO 15630-1. Det anførte krav gælder for Ø > 12 mm. For Ø = 5 - 6 mm er kravet 0,035 og for Ø = 6,5 - 12 mm er kravet 0,040.

[6] Svejste armeringsnet iht. DS/EN 10080.

STØBESKEL MM

Støbeskel må kun udføres som vist på tegninger eller efter tilsynets godkendelse.

Støbeskel skal udføres ru iht. DS/EN 1992-1-1.

Alle udadgående hjørner affases 20×20mm.

ARMERINGSSTØD

Stød, der ikke er vist på tegninger, og som ikke angivet i SAB Slap armering, skal udføres med stødlængde som opfylder kravene i DS/EN 1992-1-1 inkl. DK NA, idet det skal forudsættes, at stødet er trækpåvirket.

SPÆNDT ARMERING

I henhold til DSF/FprEN10138-3

Liner pr. kabel	Stålbetegnelse	Areal pr. line	Karakteristisk trækstyrke pr. line	Karakteristisk 0,1 % trækstyrke pr. line	ε_{uk}	Udmattelsesklasse	Spændings korrosionsklasse	Forankringstype		Opspændingskraft
		mm^2	kN	kN	(%)			Aktiv Aktiv	Aktiv Passiv	Se spændliste
7	Y1860S7	150	279	246	≥ 3,5	F1	C1	+		Se spændliste
13	Y1860S7	150	279	246	≥ 3,5	F1	C1	+		Se spændliste
19	Y1860S7	150	279	246	≥ 3,5	F1	C1	+		Se spændliste

Duktilitet: $(f_{pk}/f_{p0,1k}) \geq 1,1$

BETON

Komponent	Ref. i AAB	Egenskab	Enhed	Miljøklasse	
				A	E
Beton	1	Udførelsesklasse	-	3	3
Cement	2.1.1	Type	-	CEM I (HS/EA/≤2) CEM I (HS/LA/≤2)	CEM I (HS/EA/≤2) CEM I (HS/LA/≤2)

		Styrkeklasse, Min.	MPa	42,5	42,5
Tilslag	2.1.2	Miljøklasse	-	A	E
Groft tilslag	2.1.2	Sortering	Kat.	G_c85/20 eller G_c90/15	
		Indhold af finstof	Kat.	$f_{1,5}$[18)]	
		Lette korn under 2400 kg/m^3 Maks. [1)]	%	1,0	Ingen bestemmelse
		Lette korn under 2500 kg/m^3 Maks. [1)]	%	Ingen bestemmelse	1,0
		Kritisk absorption Maks. [2)3)4)]	%	1,1	1,1
		Reaktionsfähiger flint Maks. [2)6)]	%	3	3
		Accelereret mørtelprismeekspansion Maks. [2)5)]	længde %/antal dage	0,1/14 dage	0,1/14 dage
		Stenstørrelse[14)]	mm	Min. 25 og Maks. 32	
Fint tilslag	2.1.2	Sortering	Kat.	GF85	
		Indhold af finstof	Kat.	f_3	
		Humusindhold	-	Lysere end standardfarven	
		Kemisk svind Maks. [4)7)8)]	ml/kg	0,3	0,2
		Reaktive korn Maks. [4)7)8)]	vol-%	2,0	1,0
		Mørtelpri?hyp?smeekspansion Maks. [4)7)8)]	Længde %/antal uger	0,1/8 uger	0,1/20 uger
		Accelereret mørtelpri?hyp?smeekspansion Maks. [5)7)]	Længde %/antal dage	0,1/14 dage	0,1/14 dage
Fibre [20)]	2.1.3	Type	-	-	-
		Trækstyrke min.	kN/mm^2	-	-
		Længde/ diameter forhold, min.	-	-	-
Betonsammen sætning og frisk beton	2.2	v/c-forhold Maks. [9)]	-	0,45	0,40
		Cementindhold, Min.	kg/m^3	300	320
		Betonfillerindhold Min.	kg/m^3	375	375
		FA/C-forhold, Maks. [19)]	-	0,33	0,33
		MS/C-forhold, Maks.	-	0,06	0,06
		Chloridindholdsklasse[10) 11)]	-	Cl 0,1	Cl 0,1
		Indhold af ækv. alkali (ekskl. bidrag fra FA+MS) Maks. [10)]	kg/m^3 v/60 vol-% mørtel	3,0	3,0
		Luftindhold i frisk beton, Min. [12)]	vol-%	4,5	4,5
		Fiberindhold, min.	kg/m^3	-	-
Hærdnet beton	2.2	Styrkeklasse, Min. [13)]	-	C35	C40
		Luftindhold i hærdnet beton, Min. [15)16)]	vol-%	3,5	3,5
		Afstandsfaktor, Maks.	mm^{-1}	0,20	0,20
		Frostprøvning[15) 17)]	-	God	God
Produktionsegenskaber	4.2.2	Konsistensændring	mm	Bestemmelse skal foretages ved forprøvning	
		Luftindholdsændring	%	Bestemmelse skal foretages ved forprøvning	
		Styrkeudvikling	MPa	Bestemmelse skal foretages ved forprøvning	
		Varmeudvikling	kJ/kg	Bestemmelse skal foretages ved forprøvning	
Dæklag[21)]	3.3	Dæklag, Min	mm	40	50

		Foreskrevet dæklag, Min.	mm	45 ± 5	55 ± 5
Efterbehandling	3.8	Udtørringsbeskyttelse	Modenhedstimer	120	180

1) Indhold af lette korn skal bestemmes på groft tilslag med mikroporøs flint. Mikroporøs flint omfatter både porøs chalcedon-flint og opal-flint.

2) En af de tre alternative metoderskal dokumenteres.

3) Kritisk absorption bestemmes for de 10 % af materialet, der er flint med korndensitet over 2400 kg/m^3, og som har den største absorption. Indhold af korn mindre end 4 mm indgår ikke i prøvningen. Hvis korn mindre end 4 mm udgør mere end 5 %, skal fraktionen mindre end 4 mm undersøges for alkalikiselreaktivitet efter de metoder, som er angivet for fint tilslag.

4) Denne metode kan kun anvendes for tilslag med mikroporøs flint. Mikroporøs flint omfatter både porøs chalcedon-flint og opal-flint.

5) Denne metode kan ikke anvendes for tilslag med mikroporøs flint.

6) Denne metode kan kun anvendes for tilslag indvundet i Nordsøen.

7) En af de fire alternative metoderskal dokumenteres. Gennemsnittet af de seneste 3 prøvningsresultater skal opfylde kravene.

8) Indhold af korn større end 4 mm indgår ikke i prøvningen. Indhold større end 4 mm må ikke overstige 5 %.

9) Ved bestemmelse af v/c-forhold medregnes mikrosilica med aktivitetsfaktoren 2,0 og flyveaske med aktivitetsfaktoren 0,5.

10) Ved beregning skal deklarerede maks. værdier for de enkelte delmaterialer anvendes.

11) Der må ikke tilsættes chlorid til armeret beton. For uarmeret beton er chloridklassen Cl 1,0 %, og hvis der anvendes spændarmering i moderat miljøklasse skal chloridklassen skærpes til Cl 0,1.

12) Luftindhold skal etableres ved brug af et luftindblandende tilsætningsstof. Krav til luftindhold kan fraviges for konstruktionsdele, der ifølge SAB ikke udsættes for frost.

13) De anførte styrkeklasser er minimumsværdier for armeret beton. SAB kan indeholde krav om højere styrkeklasser.

14) For konstruktionsdele med meget høj armeringsintensitet fx forankringszoner for spændt armering bør der foreskrives maks. stenstørrelse på 16mm.

15) Krav til enten frostprøvning eller luftporeanalyse skal opfyldes: Kravet kan fraviges for konstruktionsdele, der ifølge SAB ikke udsættes for frost. Kravet skal dokumenteres opfyldt ved normal transporttids afslutning og efter pumpe, hvis en sådan anvendes.

16) Luftporeanalysen omfatter analyse på tre prøveemner (støbte/udborede/udsavede), hvor der fra hvert emne skal fremstilles overflade til prøvningen på min. 7000 mm^2.

17) Frostprøvning iht. SS 137244.

18) Indhold af finstof.

19) Flyveaske-cement forholdet.

20) Fibre må kun tilsættes, såfremt dette er specificeret i udbud?hyp?smaterialet. Eventuelle fibre skal leveres og dokumenteres iht. DS 2426 (inkl. Anneks R) samt DS/EN 14889-1:2006 og DS/EN 14889-1/ZA:2007 (stålfibre) og DS/EN 14889-2:2006 og DS/EN 14889-2/ZA:2007 (polymerfibre).

21) AAB Betonbro Slap armering.

Appendix 3

"Beton AAB", 1st August 2012, Danish Road Directorate

http://vejregler.lovportaler.dk/SearchResult.aspx?t=%2fV1%2fNavigation%2fTillidsmandssystemer%2fVejregler%2fUdbud%2fbygvaerker%2f .

Beton - AAB

(Betonbro)

af 1. august 2012
(Vejdirektoratet)

Se originaldokumentet her

1. Alment

Alt betonarbejde skal udføres i udførelsesklasse 3.

1.1 Referencer

Nedennævnte standarder og beskrivelser er i nævnte rækkefølge gældende for arbejdet med de tilføjelser og fravigelser, som fremgår af arbejdsbeskrivelser samt det øvrige projektmateriale:

- DS 2426:2011 Beton - Materialer. Regler for anvendelse af EN 206-1 i Danmark.
- DS/EN 206-9:2010 Del 9: Yderligere regler for selvkompakterende beton (SCC).
- DS 2427:2011 Betonudførelse - Regler for anvendelse af EN 13670 i Danmark.
- DS/EN 1992-2:2005 Eurocode 2: Betonkonstruktioner - Del 2: Betonbroer - Dimensionerings- og detaljeringsregler inkl. DK NA.
- Tillæg 1: 15-12-2008 EN 1992-1-1 DK NA:2007 Nationalt anneks til Eurocode 2: Betonkonstruktioner - Del 1-1: Generelt - Almindelige regler samt regler for bygningskonstruktioner.
- DS/EN 1992-1-1 + AC:2008 Eurocode 2: Betonkonstruktioner - Del 1-1: Generelle regler samt regler for bygningskonstruktioner inkl. DK NA.
- DS/EN 1504-3:2006 Produkter og systemer til beskyttelse og reparation af betonkonstruktioner. Definitioner, krav, kvalitetskontrol og overensstemmelsesvurdering - Del 3: Konstruktiv og æstetisk reparation.

De i referencerne anførte vejledninger, noter mm. skal betragtes som krav, der kun må fraviges, hvis det er angivet i projektmaterialet.

For anvendelsesstandarder gælder, at det samtidig er en reference til de(n) pågældende standard(er).

2 Materialer

2.1 Delmaterialer

Entreprenøren skal oplyse samtlige materialers oprindelse.

2.1.1 Cement

Cement skal opfylde kravene til CEM I cement.

I miljøklasse A og E skal cementen være klassificeret med høj sulfatbestandighed (HS) og ekstra lavt (EA) eller lavt (LA) alkaliindhold.

2.1.2 Tilslag

Der skal anvendes naturlige tilslag i overensstemmelse med DS/EN 12620 samt kravene i DS 2426, afsnit 5.2.3.

Groft tilslag

Groft tilslag skal opfylde kravene til kategori $G_c85/20$ eller $G_c90/15$ for sortering, jf. DS/EN 12620, tabel 2 og kategori $f_{1,5}$ for indhold af finstof, jf. DS/EN 12620, tabel 11.

Fint tilslag

Fint tilslag skal opfylde kravene til kategori G_F85 for sortering, jf. DS/EN 12620, tabel 2 og kategori f_3 for indhold af finstof, jf. DS/EN 12620, tabel 11.

2.1.3 Fibre

Fibre må normalt ikke tilsættes. Eventuelle fibre skal leveres og dokumenteres i henhold til DS 2426 (inkl. Anneks R) samt DS/EN 14889-1 (stålfibre) og DS/EN 14889-2 (plastfibre).

2.1.4 Andre delmaterialer

Andre delmaterialer må normalt ikke tilsættes.

2.2. Betonens sammensætning

Der er følgende tilføjelser til kravene til betonsammensætning og frisk beton anført i DS 2426:

- Minimum cementindhold skal være henholdsvis 230, 300 og 320 kg/m^3 i miljøklasse M, A og E
- Maksimalt mikrosilica/cement-forhold er 6 %.

2.3 Levering og opbevaring af delmaterialer

Sand- og stenmaterialer skal opbevares i siloer eller bunker, uden at forurening fra omgivelserne kan finde sted. Bunden i siloer og underlaget for bunkerne skal være af stål eller slidstærk beton og være drænet.

2.4 Blander

Betonblanderen skal være en satsblander, der kan blande delmaterialerne til en ensartet masse. Blanderen skal kunne tømmes fuldstændigt.

Det skal fra anlæggets manøvreplads være muligt løbende at vurdere betonmassens konsistens og inden samt efter tømning at foretage en visuel vurdering via fjernsyn af den færdigblandede beton og tømningens effektivitet.

3 Udførelse

3.1 Generelt

Projektmaterialet kan omfatte en generalnote med de vigtigste krav til delmaterialer, betonsammensætning og udførelse.

3.2 Tolerancer

Alle konturlinjer, der er bestemmende for bygværkets udseende (f.eks. dækundersider, kantbjælker) skal udføres så nøjagtigt, at der ikke forekommer synligt skæmmende afvigelser fra den korrekte form.

Følgende tolerancer skal overholdes for det færdige betonarbejde:

1. Placering af fundamenter, piller, søjler, overbygninger m.v. i forhold til systemlinjer:
   Maks. afvigelse ± 20 mm
2. Brodæks højdebeliggenhed gældende for såvel overside som underside:
   Tegningsmålet ± 20 mm
   For planhed, overfladefinish og geometri af overflader for kantbjælker, brodæk, søjler m.v. gælder desuden kravene under pkt. 5.
   For områder med faldopbygning til vandafledning skal følgende opfyldes ved vandprøve for, at faldopbygningen er korrekt udført:
   - På broer med resulterende fald større end 20 ‰ må pytdannelse på brodækkets betonoverside ikke forekomme.
   - På broer med resulterende fald under 20 ‰ må pytter kun undtagelsesvis forekomme (ca. 3 pr. 100 m^2) og kun i lokale områder mindre end 0,5 m^2. Pytternes maksimale dybde må ikke overstige 1 mm.
   - Pytdannelser i dybdelinjer må kun forekomme med en 0,2-0,3 m lang pyt pr. 5 m dybdelinje.
3. Brodæks sidebeliggenhed gældende for alle lodrette flader:
   Maks. afvigelse ± 20 mm
   Afvigelser målt ud fra den forudsatte geometriske form må højst være:
   - 12 mm for målelængder mellem 1 m og 6 m
   - 20 mm for målelængder mellem 6 m og 18 m
   - 30 mm for målelængder over 18 m.

   Disse afvigelser skal være jævnt stigende eller faldende over hele målelængden.
4. Dimension af konstruktionsdele:
   - Ved fundamenter, der støbes uden form dog: Tegningsmålet + 100 mm, - 10 mm.
   - Dimensioner (d) mindre end 200 mm: Tegningsmålet ± 1/20 x d mm
5. Betonoverfladers lokale jævnhed og tekstur:
   - For ikke synlige overflader støbt imod form skal overflader opfylde følgende krav til overfladegeometri:
     i) Planhed, generelt ± 5 mm målt med 1 m retskinne jævnt stigende/faldende.
     ii) Lokal planhedsafvigelse ± 5 mm (kanter/porehuller/grater).
     iii) Porestørrelse og -antal: porer ≥ 20 mm skal udfyldes; for porer < 20 mm er der ingen krav.
   - Dimensioner større end 200 mm: Tegningsmålet ± 10 mm
   - For synlige overflader støbt imod form skal overflader opfylde krav til overfladegeometri:
     i) Planhed, generelt ± 4 mm målt med 1 m retskinne jævnt stigende/faldende.
     ii) Lokal planhedsafvigelse ± 2 mm (kanter/porehuller/grater).
     iii) Porestørrelse og -antal: porer ≥ 20 mm skal udfyldes; porer fra 10 til 20 mm må maks. forekomme med 200 stk. pr. 10 m^2; for porer < 10 mm er der ingen krav.
   - For overflader, hvor der skal udføres fugtisolering og påsvejses bitumenplader skal krav til overfladens geometri svarende til ikke synlige overflader støbt imod form overholdes, dog med skærpede krav til porer og tekstur:
     iv) Porestørrelse og -antal: porer ≥ ø10 mm skal udfyldes; porer fra 5 til 10 mm må maks. forekomme med 50 stk. pr. 10 m^2 og ikke udpræget i samlinger; for porer < 5 mm er der ingen krav.
     v) efter grunding skal betonoverfladens tekstur målt i henhold til DS/EN 13036-1 opfylde følgende krav:
   - alle enkeltværdier skal ligge i intervallet fra 0,4 til 1,3 mm
   - middelværdien skal endvidere ligge i intervallet fra 0,5 mm til 1,0 mm.
     alt Tekstur efter grunding, jf. AAB - FUGTISOLERING 10.3.3
   - For brodæk, hvor der skal udlægges kunststofbelægninger samt uisolerede brodæk, skal ovennævnte for overflader med fugtisolering opfyldes og dertil kommer jævnhedskrav i AAB - BROBELÆGNING, afsnit 1.5.2.

3.3 Støbeprogram

Støbeprogrammet skal forelægges for bygherren senest 5 arbejdsdage før planlagt støbning.

3.4 Materiel

Før en støbning påbegyndes, skal entreprenøren sikre, at han disponerer over det nødvendige, klargjorte reservemateriel til blanding, transport, støbning og bearbejdning af betonen, således at en påbegyndt støbning med sikkerhed kan gennemføres kontinuert.

Der skal ved store betydende støbninger disponeres over to blandeanlæg, der begge skal have tilstrækkelig kapacitet til at gennemføre de planlagte støbninger uden andre støbeskel end de foreskrevne.

3.5 Støbeskel

Støbeskel skal placeres som angivet på tegningerne.

3.6 Klargøring af form og armering

Entreprenøren skal, senest 48 timer før betydende støbninger er planlagt påbegyndt, meddele dette til bygherren.

3.7 Behandling af ikke hærdnede betonoverflader

Oversiden af betonkonstruktionerne skal afrettes til den på tegningerne anførte form og til de krævede højder med de anførte tolerancer ved hjælp af ledere og skabeloner med tilstrækkelig stivhed.

Afretning af oversider, som senere skal fugtisoleres, skal ske med bjælkevibrator efterfulgt om nødvendigt af glitning.

Afretning af oversider, som ikke skal fugtisoleres, skal afrives med bræt. Afrivningstidspunktet skal vælges således, at afrivningen kan gennemføres uden oparbejdning af slam i overfladen. Overfladen skal være jævnt afrevet.

Overfladeforskalling på skrå oversider skal udføres som demonterbare flager af en længde, som tillader omhyggelig udstøbning og afrivning i sektioner.

3.8 Efterbehandling

3.8.1 Krav

Betonens udstøbning og efterbehandling skal planlægges og udføres på en sådan måde, at betonen i hærdeperioden beskyttes mod skadelige påvirkninger fra omgivelserne og hærdevarmen.

Entreprenøren skal tilgodese følgende krav:

- Beskyttelse mod udtørring skal være etableret hurtigst muligt, og inden der er fordampet en vandmængde på 1,5 kg/m^2 fra overfladen, dog senest 1 time efter afbindingstidspunktet. Denne vandmængde svarer til en lagtykkelse større end eller lig med 0,20 m. For tykkelser mindre end 0,20 m skal vandmængden reduceres proportionalt med den mindre tykkelse. Såfremt fordampningsforholdene ikke vurderes på grundlag af de aktuelle forhold, skal beskyttelsen etableres inden 1 time efter støbning. Beskyttelsen skal om nødvendigt etableres midlertidigt inden afretning foretages.
- Beskyttelsen mod udtørring skal opretholdes indtil følgende modenhed (ækvivalent hærdetid ved 20 ^{o}C) er opnået i betonens overfladelag:

Miljøklasse	Modenhedstimer
E	180
A	120
M	36

Hvis afbindingen starter senere end 5 timer efter blanding, øges modenhedskravene tilsvarende.

- Den støbte beton skal beskyttes mod udvaskning.
- Betonens temperatur under hærdningen må ikke overstige 60 ^{o}C. Eftervisning af temperaturforhold skal ske på grundlag af temperaturregistreringer i konstruktionen. Placering af temperaturfølere skal aftales med bygherren.

3.8.2 Foranstaltninger

Efterbehandlingen med hensyn til beskyttelse mod udtørring kan for overflader mod form ske ved, at formen bliver siddende, indtil den krævede modenhed er opnået. Såfremt afformning finder sted, inden betonen har opnået den krævede modenhed, skal der senest 1 time efter afformningen etableres beskyttelse af alle afformede overflader.

Beskyttelsen skal ske ved vandlagring, tildækning med vanddamptætte plastpresenninger eller svær plastfolie. Samlinger skal udføres tætte, og tildækningsmaterialerne skal fastholdes effektivt til betonoverfladen også under vindpåvirkning. Beskyttelse kan også ske ved opretholdelse af høj relativ fugtighed (f.eks. i et tågekammer), hvis det dokumenteres, at temperaturdifferensen mellem luft og beton ikke skaber et højere vanddamptryk på betonoverfladen end i luften.

Efterbehandlingen kan for frie overflader ske ved tildækning som for overflader mod form eller, ved påsprøjtning af betonforseglingsmiddel. For overflader, der efterfølgende skal fugtisoleres, kan grunderen erstatte betonforseglingsmiddel, jf. AAB - FUGTISOLERING, såfremt grunderen opfylder nedenstående krav til vandtilbageholdelsesevne.

Entreprenøren skal dokumentere, at dette ikke medfører skader på betonen, misfarvning af betonen eller reduceret vedhæftning af evt. fugtisolering og maling. Betonforseglingsmiddel på voksbasis må ikke anvendes på flader, der skal fugtisoleres med bitumenplader, uanset om disse sandblæses. Betonforseglingsmiddel må kun anvendes på støbeskel, hvis både beton og armering renses ved sandblæsning, inden støbningen genoptages.

Betonforseglingsmiddel skal have en vandtilbageholdelsesevne i 3 døgn på mindst 75 %, i henhold til TI-B 33.

Der må ikke anvendes forseglingsmiddel, hvor der senere skal overfladebehandles medmindre det ved forprøvning er dokumenteret, at dette ikke påvirker udseendet eller vedhæftningen af overfladebehandlingen.

Beskyttelse mod udvaskning kan ske ved tildækning med presenninger. Tildækning skal udføres således, at afrevne overflader ikke beskadiges.

3.9 Hærdnet beton, overflader

Entreprenøren skal straks meddele bygherren, såfremt der ved afformningen viser sig fejl på betonoverfladerne, eller i bygværkets geometri. Synlige betonoverflader skal fremtræde ensartede i kulør og overfladekarakter, uden misfarvninger og uden skæmmende ujævnheder.

Før ophugninger og reparationer af fejl udføres skal der udarbejdes en procedure, som beskriver omfang og reparationsmetode. Proceduren skal forelægges bygherren til godkendelse. Fejl skal udbedres hurtigst muligt.

3.9.1 Afrevne overflader

Procedure for udjævning af eventuelle fordybninger samt eventuel tætning af betonoverfladen skal forelægges bygherren til godkendelse.

3.9.2 Formstøbte ikke synlige overflader

Alle forankringsjern udtrækkes eller afkortes, og samtlige rør og prophuller udsættes med mørtel i henhold til DS/EN 1504-3. Entreprenøren skal dokumentere, at udsætningen er mindst lige så tæt som den omgivende beton.

3.9.3Formstøbte synlige overflader

På synlige flader skal alle prop- og clampshuller udsættes helt, således at hulranden står skarpt og ligger ca. 2 mm tilbage fra den øvrige betonoverflade. Inden prop- og clampshuller udsættes, oprives siderne f.eks. med en stiv, konusformet stålbørste, således at der skabes en ru overflade.

3.9.4 Overflader, der skal fugtisoleres eller belægges med asfalt.

Betonoverflader, der er direkte underlag for fugtisolering eller belægning, skal gennemmåles omhyggeligt, således at eventuelle afvigelser fra forudsat geometrisk form konstateres. Desuden foretages vandprøve for at konstatere evt. pytdannelse, jf. afsnit 3.2 og 4.5.4.

For at sikre, at den færdige overflade tilfredsstiller det stillede jævnhedskrav, udarbejdes der efter udførelsen af ovennævnte målinger forslag til, hvorledes eventuel afretning udføres, udbedring af betonoverfladen eller afretning med bituminøse materialer i belægningslagene. Forslaget forelægges bygherren til godkendelse.

Såfremt det resulterende fald er større end 10 promille skal eventuelle områder af betonoverfladen (lunker), fra hvilke vandafledning ikke er mulig, samt eventuelle områder, der ikke opfylder tolerancerne i afsnit 3.2, repareres så korrekt profil opnås.

Betonoverflader, der skal isoleres med bitumenplader, renses fuldstændigt for cementslam og cementmørtel, jf. AAB - FUGTISOLERING 3.1.

4 Kontrol

4.1 Generelt

Betonkontrol omfatter forprøvning, prøvestøbning, produktionsprøvning af materialer og udførelseskontrol.

For betydende konstruktioner gælder:

- udarbejdelse af støbeprogram (afsnit 3.3)
- forprøvning af betons produktionsegenskaber (afsnit 4.2.2)
- prøvestøbning, herunder udstøbning af prøvelegeme og udtagning af repræsentative delmaterialeprøver(afsnit 4.3)
- produktionsprøvning af genbrugsvand (afsnit 4.4.1)
- udtagning af prøver på pladsen til måling af frisk betons luftindhold og konsistens (afsnit 4.4.2).

Forprøvning består i at dokumentere, at de stillede krav opfyldes for delmaterialer og betonsammensætning.

Ved prøvestøbningen dokumenteres det, at alle krav til materialer og udførelse kan opfyldes samtidigt under realistiske produktionsvilkår.

De egentlige støbninger må ikke påbegyndes, før bygherren har haft resultaterne fra forprøvning af delmaterialer og betonens sammensætning samt prøvestøbning til godkendelse. Der regnes normalt med 5 arbejdsdage hertil.

Produktionsprøvning foretages løbende i produktionsprocessen for materialer.

Udførelseskontrol foretages løbende i udførelsesprocessen.

4.2 Forprøvning af materialer

4.2.1 Delmaterialer

Følgende dokumentation skal tilvejebringes og indgå i entreprenørens dokumentation:

- Oplysning om delmaterialernes oprindelse
- Gældende certifikater og varedeklarationer samt resultater fra seneste produktionsprøvning for de enkelte delmaterialer.

Groft tilslag

Bjergartsfordeling i henhold til metode DS/EN 932-3 (også for underkorn større end 2 mm) skal dokumenteres.

4.2.2 Betonens sammensætning

For hver betonsammensætning skal der udføres forprøvning. Hvis der foreligger dokumentation fra tilsvarende beton, som er blevet forprøvet inden for de seneste 12 måneder, kan denne dokumentation anvendes som alternativ til forprøvning.

Forprøvningen skal som minimum opfylde følgende krav:

- Forprøvningen udføres på frisk beton med en temperatur svarende til det der forventes at være aktuelt ved brostøbningerne

- Eftervisning af trykstyrken skal baseres på mindst 3 prøvelegemer fra hver af tre satse, jf. DS/EN 206-1, afsnit A4
- Krav til betonsammensætning skal dokumenteres opfyldt ved kontrol af blanderapporter fra hver af de tre satse.
- For andre egenskaber end styrke, som skal eftervises ved prøvning, dvs. konsistens og luftindhold i frisk beton, skal kravene eftervises opfyldt på hver af de tre satse.
- De i DS 2426 tabel 2426-7 angivne produktionsegenskaber skal bestemmes på en af de tre satse.
- For egenskabsrelaterede dimensioneringsmetoder jf. DS/EN 2426 afsnit 5.3.3 skal krav til frostbestandighed eftervises opfyldt på mindst en af de tre satse.

På baggrund af de opnåede resultater ved forprøvningen skal der for tilsætningsstoffer opstilles vejledende grænser for receptvariationer, til brug for styring af produktionen.

Beregningerne på betonsammensætningen og prøvningsresultaterne fra prøveblandinger skal forelægges bygherren til godkendelse.

4.2.3 Levering og opbevaring af delmaterialer

Ved visuel inspektion skal det kontrolleres, at opbevaringen af delmaterialer opfylder de stillede krav.

Tørstofindholdet i slurry skal kontrolleres ved inddampning af en udtaget prøve fra slurrybeholderen. Afvigelser fra den deklarerede middelværdi må højst være 1,0 % (procentpoint).

4.3 Prøvestøbning

Der skal for hver betontype blandes det antal betonsatse med normal opfyldning af blanderen, som mindst svarer til normal fyldning af en rotérbil.

Ved prøvestøbningen skal der anvendes den aktuelle transportmetode samt den maksimale transporttid, der garanteres af producenten for den pågældende betontype.

Ved anvendelse af selvkompakterende beton skal der foretages en demonstration af, at de krævede faldforhold på overside af brodæk/kantbjælke kan etableres med de tilstræbte flydeegenskaber for betonen. Det skal samtidig demonstreres, at armering og afstandsklodser omstøbes tilfredsstillende, og at den anvendte betontype ikke udviser separationstendenser. Herudover skal prøvestøbningen dokumentere i hvor stort omfang vibrering kan tillades ved støbepauser og lign.

Der skal udstøbes et prøvelegeme hvis form og størrelse er beskrevet i SAB eller aftales med bygherren.

Mindst 3 stk. Ø100 mm borekerner skal udbores af prøvelegemet. På hver borekerne udføres:

- En luftporeanalyse i henhold til DS/EN 480-11
- Hvis der anvendes mikrosilica tilsat som pulver, skal fordelingen af mikrosilica, undersøgt på borekernen, opfylde kravet i DS 2426 afsnit 9.8.

Det skal ved prøvestøbningen eller ved beregning dokumenteres, at de planlagte foranstaltninger til efterbehandling er egnede til at sikre opfyldelse af de stillede temperaturkrav i betonen under de vejrforhold, der med rimelighed kan forventes at forekomme.

Beregning af betonens temperaturtilstande og modenhedsudvikling skal foretages på baggrund af målinger af betonens varmeudvikling målt ved adiabatisk eller semiadiabatisk kalorimetri.

Valg af udstyr til måling af betonens temperatur, herunder program for målingens udførelse, antal placeringer af målesteder samt journalføring af måleresultaterne, skal forelægges bygherren til gennemsyn.

Viser prøvestøbningen ikke den forlangte betonkvalitet, skal delmaterialer, blandingsforhold, materiel eller udførelsesteknik ændres, og nye prøvestøbninger udføres, indtil den forlangte kvalitet opnås.

De ved prøvestøbningen opnåede resultater danner referencegrundlag for betonkvaliteten i bygværket.

4.4 Produktionsprøvning af materialer og udstyr

4.4.1 Delmaterialer

Entreprenøren skal i forbindelse med støbning af brodæk, samt for hver påbegyndt 1000 m^3 beton i entreprisen udtage følgende repræsentative delmaterialeprøver:

- 10 kg cement
- 10 kg flyveaske
- 10 kg mikrosilica
- 10 kg sand
- 20 kg sten
- 1 l fra hvert kar med genbrugsvand
- 10 kg af øvrige delmaterialer med undtagelse af drikkevand.

Prøverne opbevares efter aftale med bygherren indtil afleveringen.

Blandevand

Genbrugsvand skal kontrolleres mindst 1 gang per dag, når der produceres beton i henhold til nærværende beskrivelse.

4.4.2 Betonens sammensætning

Luftindhold

Luftindholdet samt konsistensen af den friske beton skal kontrolleres på betonfabrik på de 3 første læs og derefter 1 gang pr. 25 m^3 leveret beton. Ved levering udtages prøver på pladsen i samme omfang. Ved anvendelse af pumpe skal målingerne foretages efter pumpning.

Styrke

Antallet af udtagne prøver til styrkebestemmelse fastsættes af entreprenøren. Dog skal der for hvert støbeafsnit udføres mindst 3 prøvninger og i gennemsnit mindst 2 prøvninger pr. 50 m^3 leveret beton. Udtagning af prøver foretages på betonfabrikken.

Såfremt det på grundlag af de udtagne prøver ikke er muligt at dokumentere, at styrkekravet er opfyldt, er det pågældende kontrolafsnit afvigende, medmindre entreprenøren ved prøvning af udborede cylindre af konstruktionsbeton kan eftervise, at denne har en karakteristisk styrke på mindst 90 % af den krævede karakteristiske styrke. Denne prøvning sker for entreprenørens regning.

Prøvningen udføres ved udboring af 6 kerner senest 3 arbejdsdage efter, at de støbte cylindre viser, at styrkekravet ikke er opfyldt.

Kernerne udbores i henhold til DS/EN 12504-1, idet højde/diameter-forholdet skal være 200/100 mm. Såfremt højden på grund af konstruktionens udformning ikke kan blive 200 mm, tillades undtagelsesvis mindre højder, dog minimum 150 mm. Udboringsstederne fastlægges efter aftale med bygherren.

Afhængigt af konstruktionsbetonens modenhed for det pågældende kontrolafsnit, udføres følgende:

1) Under 28 døgns modenhed: Kernerne trykprøves i vandmættet tilstand, når de har opnået en modenhed svarende til 28 døgn
2) Over 28 døgns modenhed: Kernerne trykprøves straks efter vandmættet tilstand er opnået, jf. kravene til opbevaring inden prøvning i DS/EN 12504-1.

Styrkekravet anses for opfyldt, såfremt trykstyrkernes middelværdi, beregnet i henhold til annex 1, opfylder kravene i projektmaterialet.

Prøvningsprogrammet skal på forhånd godkendes af bygherren.

4.4.3 Levering og opbevaring af delmaterialer

De dage, hvor der leveres beton i henhold til nærværende bestemmelser, skal følgende udføres:

- Kontrol af at samtlige delmaterialer opbevares i henhold til de stillede krav
- Tørstofindholdet i slurry skal kontrolleres ved inddampning af en udtaget prøve fra slurrybeholderen. Afvigelser fra den deklarerede middelværdi må højst være 2,0 % (procentpoint).

Der skal føres journal over ovenstående kontrolprocesser.

4.5 Udførelse

4.5.1 Tolerancer

Entreprenøren skal føre en journal over alle registreringer i forbindelse med kontrol af bygværkets geometri.

4.5.2 Støbning

Entreprenøren skal føre journal for hver støbning. Journalen skal indeholde alle relevante oplysninger angående støbningen. Oplysningerne skal mindst bestå af:

- Tidspunktet for støbningens påbegyndelse og afslutning.
- Støbetakt (støbt betonmængde pr. time).
- Afbrydelser og forstyrrelser under støbningen.
- Indbygningsstedet for den beton, hvoraf der er udtaget prøver.
- Vejrliget under støbningen (nedbør, vind, lufttemperatur).

4.5.3 Efterbehandling

Entreprenøren skal føre journal, der for hvert støbeafsnit angiver alle for efterbehandlingen relevante forhold som:

- Tidspunkt for støbningens påbegyndelse og afslutning
- Formmaterialets art og type
- Vejrliget under efterbehandlingen (nedbør, vind, lufttemperatur, solindfald og relativ fugtighed)
- Den friske betons temperatur
- Afformningstidspunkt
- Efterbehandlingens art og varighed.

Herudover skal betonens temperaturer fra støbningstidspunktet registreres i et antal punkter i hvert støbeafsnit til konstatering af, om de stillede krav til maksimaltemperatur og maksimaltemperaturdifferencer er opfyldt i hele støbeafsnittet.

Såfremt det i hærdningsperioden, på grundlag af den ovennævnte kontrol, viser sig, at hærdningens udvikling afviger fra det på grundlag af beregningerne forudsatte forløb, skal entreprenøren gribe ind med forholdsregler, således at kravene overholdes.

De trufne foranstaltninger skal meddeles bygherren og journalføres.

Arbejdsoperationer (f.eks. opspænding og trafikbelastning), der er afhængige af betonens hærdningsudvikling og styrke, må først iværksættes, når entreprenøren har dokumenteret, at den nødvendige modenhed er til stede.

I journalen skal endvidere noteres, på hvilket tidspunkt og af hvem inspektionerne er udført.

4.5.4 Hærdnet beton, overflader

Kontrol af overfladens planhed generelt og planhedsafvigelser lokalt udføres i henhold til følgende fremgangsmåde, når der er berettiget tvivl hvorvidt kravene i afsnit 3.2 (pkt. 5) er opfyldt:

- Kontrol af planhed, generelt. Der anvendes en 1 m retskinne og en målekile/målestok. Positive værdier af afvigelsen beskriver en hævning af planen og negative værdier beskriver en fordybning.
- Kontrol af lokal planhedsafvigelse (dybdemål). Der anvendes bl.a. måleur og skydelære. Måling af grater/toppe/huller foregår vinkelret på den omgivende planflade. Positive værdier af afvigelsen beskriver en top/grat og negative værdier beskriver et hul.
- Kontrol af porestørrelse (planmål) og -antal. Måling foregår ved at der udpeges 3 områder a 10 m^2. Indenfor hvert område måles ved hjælp af skydelære porestørrelserne i plan (max udstrækning) og antallet indenfor hvert udfaldsrum opgøres for hvert felt. Gennemsnittet af de tre flader skal overholde udfaldskravene.

4.5.5 Overflader, der skal fugtisoleres og/eller belægges med asfalt, eller som skal fungere som uisoleret brodæk

Betonoverflader gennemmåles omhyggeligt ved nivellement og med retskede. Pytdannelse konstateres ved vandprøve. Målingernes omfang skal aftales med bygherren, idet det fremstillede profil er afgørende for omfanget.

Overflader, der skal fugtisoleres med kunststofisolering, skal kontrolleres for aftræksstyrke af overfladen, jf. AAB - KUNSTSTOFBELÆGNING, afsnit 1.3.

Annex 1: Kontrol af betonstyrker på udborede kerner af eksisterende konstruktioner

Styrkekravet anses for opfyldt, såfremt middelværdien af de målte styrker, $f_{c,målt,mid}$, omsat til en ækvivalent normmæssig referencestyrke, $f'_{c,mid}$, opfylder kravet:

$f'_{c,mid} \geq 0{,}90 \times k_n \times f_{ck}$

hvor faktoren k_n beregnes som angivet i DS 2426 anneks X.1,

f_{ck} er den krævede karakteristiske trykstyrke og

$f'_{c,mid}$ beregnes som følger:

$f'_{c,mid} = k_1 \times k_2 \times k_3 \times k_4 \times f_{c,målt,mid}$

hvor

k_1 er en faktor, der omsætter styrken svarende til en cylinder med forholdet 1:2. For udborede cylindre med et andet forhold end 2 mellem cylinderhøjde og diameter, vil omregning til den standardiserede cylinderhøjde kunne ske ved multiplikation med en faktor k_1. Under forudsætning af, at forholdet mellem cylinderhøjde og cylinderdiameter er beliggende mellem 1 og 2, og at cylinderdiameteren er beliggende mellem 70 mm og 150 mm, kan denne faktor regnes at være:

$k_1 = 0{,}2 \times h/d + 0{,}6$

hvor h og d er henholdsvis højde og diameter.

k_2 er en faktor, der omsætter styrken svarende til en refe rencecylinder med højde-diameter-forhold på 300/150 mm.

Højde/diameter	Omregningsfaktor
140/70	0,90
200/100	0,95
300/150	1,00

For de her angivne 200/100 mm kerner er $k_2 = 0{,}95$.

k_3 er en faktor, der omsætter styrken fra udboret styrke til støbt styrke. Hvis der ikke ved forsøg fastlægges omregningsfaktorer, kan der mellem ligedannede prøvelegemer med samme modenhed regnes med efterfølgende omregningsfaktorer for forholdet mellem styrken af det støbte prøvelegeme og det tilsvarende udborede prøvelegeme:

Højde/diameter	Omregningsfaktor
140/70	1,40
200/100	1,25
300/150	1,10

For de her angivne 200/100 mm kerner er $k_3 = 1{,}25$.

k_4 er en faktor, der omsætter styrken til styrken ved 28 modenhedsdøgn. For faktoren benyttes værdierne:

$k_4 = 1{,}00$ ved modenhed på 28 døgn

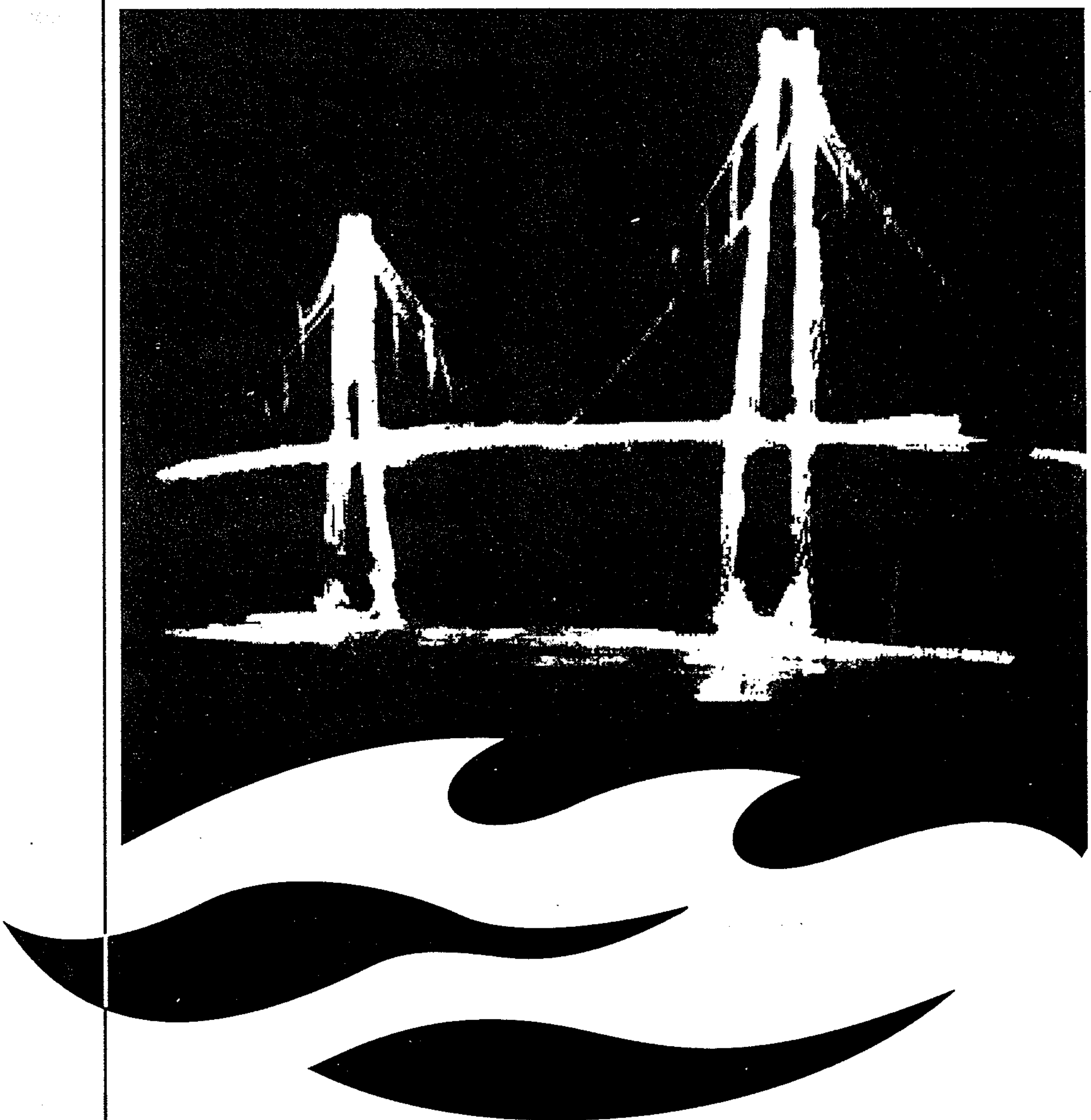
$k_4 = 0{,}95$ ved modenhed på 29-35 døgn

$k_4 = 0{,}90$ ved modenhed på 36-42 døgn.

Appendix 4

Ulla Kjær, Birgit Sørensen and Mette Geiker: "Chloride resistant concrete - theory and practise", Concrete Across Borders, International Conference, Odense, 1994, pp 227-237

CONCRETE ACROSS BORDERS



International Conference 1994

Hotel H.C. Andersen
Odense, Denmark

Organized by Danish Concrete Association in collaboration with
Danish Concrete Institute and supported by
American Concrete Institute (ACI) and the Danish ACI chapter

PROCEEDINGS
VOLUME I

Chloride Resistant Concrete
- theory and practise

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Cowiconsult A/S.

Synopsis. Chloride induced corrosion of the reinforcement is the major problem for the durability of concrete structures today. The existing technology does not yet make it possible to guarantee a specific service life. Actual exposure conditions of concrete in structures differ greatly from that in accelerated test methods. At present there is still no documented relation between performance in structures and in accelerated testing. New structures are well documented but chloride exposure times are still too short to predict service life from measured chloride ingress. Older structures are not sufficiently well documented to allow evaluation of the influence of design parameters on actual chloride ingress. Actual design criteria are compared for selected medium and large structures in Northern Europe, including the Channel Tunnel and the Great Belt project as well as earlier projects. Experience with measured and calculated chloride ingress gained so far from these structures is reviewed.

Introduction

1. Reinforced Concrete Marine Structures in Scandinavia have traditionally performed well. A great number of harbours and bridges exist today, mainly constructed in reinforced concrete. Some of these are now more than 60 years old, and although they of course have been maintained and repaired over the years their performance is still satisfactory. However, a few examples of unsatisfactory durability of marine structures constructed in recent years also exist.

2. Chloride induced corrosion of the reinforcement in motorway bridges etc. has become the major problem for the durability of these structures in Denmark, where de-icing salt is used extensively. Because of the moderate temperatures achieved all year round, and because of the relatively high quality of concrete, service life is still generally satisfactory, as compared with eg. the Middle East, where ingress or contamination with chloride due to the high salt content in ground or water, high temperature sometimes in combinatin with low quality of concrete has resulted in structures having only achieved a service life of a few years.

3. However, where previously the service life of a structure (although not generally specified) was expected to be of the order of 25 to 50 years, nowadays major structures, eg. the Channel Tunnel and the Great Belt project, are designed for a service life of 100 - 120 years.

Requirement	Unit	1980's Oil Platforms	Channel Tunnel segments	Railway Tunnel, Great Belt Link, Denmark (A conc)	
		Norway	France	Segments	In Situ Tunnel
Specif. Service Life	Year	25	120	100	100
Min. Char. Compr. Strength	MPa	55/70	45/65	50	40
Min. Cover	mm	50	35*	35*	50
Max. w/c-ratio, Equiv.	kg/kg	0.40	0.35	0.35	0.35
Max. permeability Coeff.	10^{-13} m/s	1	1	0.25	-
Max. Chlor.Bulk Diff. Coeff.	10^{-13} m^2/s	-	-	6	-
Min. Cement Content	kg/m^3	350	400	300	300
Max. Cement Content	kg/m^3	500	-	500	-
Min. Flyash Content	% of powder	-	-	10	10
Min. Silica Content	% of powder	-	-	4	4
Max. Silica Content	% of powder	-	-	8	8
Max. Flyash + Silica Content	% of powder	-	-	25	25
Aggregates	-		Limestone	Granite	Granite
Aggregates D_{max}	mm	20	12.5	16	32
Cement, Alkali Cont.	%	0.6	0.8	0.4	0.4
Cement, C_3A Cont.	%	5 - 8	5 - 8	3 - 5	3 - 5
Min./Max. Air, Hard. Concrete	%	3 - 5	no air entr.**	no air entr.**	-
Min./Max. Air, Hard. Concrete	% of cement matrix	-	-	-	8 - 20
Min. Spec. Surface	mm^{-1}	25	-	-	25
Max. Spacing Factor	mm	0.25	-	-	-
Reference No.		16	15	12	12

* For structural reasons the cover was only 35 mm. For the segments for the Great Belt Link, additional protection of the reinforcement was introduced in the form of epoxycoated reinforcement. Furthermore, the reinforcement is electrically connected and as such prepared for cathodic protection.

** The segment concrete is not air entrained since it is not exposed to frost in the tunnels.

Table 1. Comparison of Specifications for Oil Platforms (Norway), Channel Tunnel (France) and Great Belt Link (Denmark)

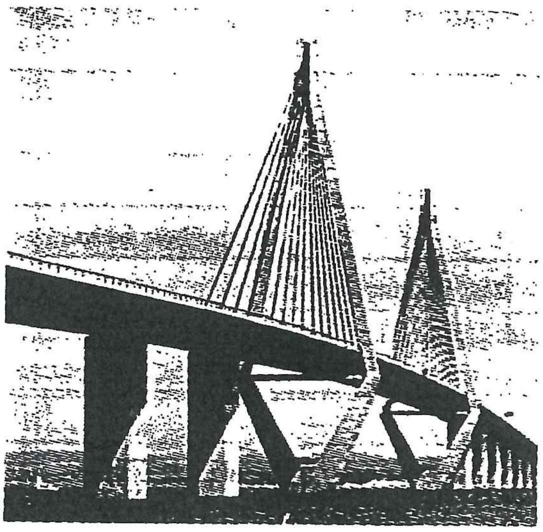
4. The existing technology does not yet make it possible to guarantee a specific service life. There is no documented relation between results achieved by accelerated test methods and real life performance of structures. Although by now some of the early concrete structures are more than 100 years old, the constituent materials, mix composition, execution and loading applied with present day technology, makes it impossible to compare the early structures with present day construction.

5. Because of this, although the intended service life may be the same, the resulting design and specifications for exposed marine structures may be different between nations, as well as within a nation within relatively few years.

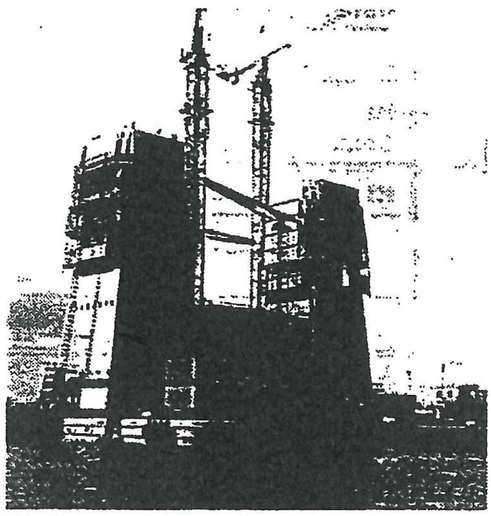
Design Criteria for Marine Structures in Northern Europe

6. Present day attempts to design for a specified service life of a structure under actual loads, both structural as well as environmental. The functional requirements such as eg. Resistance to Frost, Chloride Penetration, Sulphate Penetration, Alkali Aggregate Reaction etc. are generally not quantified, but a number of substitute requirements such as eg. w/c-ratio, cement content, cement type, aggregate reactivity, etc. are specified quantitatively in stead.

7. Table 1 compares selected substitute requirements in specifications for 3 selected recent, large, marine reinforced concrete structures: Oil Platform (Norway), Channel Tunnel (segments, France) and Great Belt Link (Denmark). It can be seen that there are many similarities. The major differences between these national specifications are mainly caused by the different attitude to the replacement of cement by pozzolans. (Other parameters, eg. cover, aggregate D_{max} etc. are determined by the geometry and the structural use).



The cable-stayed Farø Bridge pylons are diamond shaped. The main span is 290 m, and the side spans are 120 m each. The choice of steel after tender for the bridge girder resulted in a reduction in weight to about one third of that of a concrete girder originally planned, and a reduction of cable-stays from 64 to 18 per pylon.



The East Bridge pylon legs are slipformed to a high of 254 m. The lower 21 m of the pylon above the caisson is designed as a monolithic structure with heavily reinforced 1.2 m thick walls to resist impact loads of 400 MN. The pylons are located in water depth of about 20 m

8. Table 2 compares substitute requirements in specifications for 3 selected large, marine reinforced concrete structures in Denmark: "Old" Lillebælt Bridge (1935), "New" Lillebælt Bridge (1968) and Great Belt Link (1995). It can be seen that max. specified w/c-ratio has been reduced. The total min. binder content is more or less constant, but in the specifications for the Great Belt Link part of the cement has been replaced by flyash and microsilica. Air entrainment has been introduced. Requirements to min. wet curing have been reduced, reflecting the rapid hardening cements used presently and our improved ability to calculate the hardening development of the concrete.

Requirement	Value	Old Lillebælt	New Lillebælt	Great Belt Link East Bridge**
Year, end construction		1935	1968	1996
Design Life	years	-	-	100
Concrete Mix		1:2:2	-	-
Min. Compr. Str.	MPa	20	48	45
Min. Cover	mm	40		75
Max. w/c-ratio, Equiv.	kg/kg	-	0.55	0.35
Min. Cement Content	kg/m^3	425	425	300
Min. Flyash Content	% of powder	-	-	10
Min./Max. Silica Content	% of powder	-	-	5-8
Aggregates	-	hard + durable	granite	granite
Cement, Alkali Cont.	%	-	0.4	0.4
Cement, C_3A Cont.	%	-	low heat	3-5
Min. Air, Hard. Concr.	%	-	(4)*	-
Min./Max. Air, Hard. Concr.	% of cement matrix	-	-	8-20
Min. Spec. Surface	mm^{-1}	-	-	25
Curing, min.	days	42	28 wet 14 air	10 wet
Reference No.	-	17	18	12

* Fresh concrete ** Concrete type A

Table 2. Comparison of specifications for 3 large bridges in Denmark: "Old" Lillebælt Bridge (1935), "New" Lillebælt Bridge (1968) and Great Belt Link (1996).

9. Table 3 compares substitute requirements in specifications for 3 fairly recent, large, reinforced structures in Denmark: Øresund Bridge (1993), Great Belt Link (1989) and Farø Bridges (1978). It can be seen that within this relatively short span (15 years) concrete specifications have only been moderately revised.

Requirement	Unit	Øresund	Great Belt East Bridge (A)	Farø
Year, specification	-	1993**	1989	1978
Service Life	years	100***	100	
Min. Char. Compr. Str.	MPa	40	45	35
Min. Cover (splash zone)	mm	70	75	50
Max. w/c-ratio Equiv.	kg/kg	0.42	0.35	0.45
Min. Cement Content	kg/m^3	270	300	-
Min./max. Flyash Content	% of powder	10/20	10	-
Min./max. Silica	% of powder	2/6	5/8	-
Aggregates	-	-	granite	-
Cement, Alkali Cont.	%	0.4	0.4	0.4
Cement, C_3A Cont.	%	min. 3	3-5	-
Min./Max. Air, Hard. Concr.	%	-	-	4-6*
Min./max. Air, Hard. Concr.	% of cement matrix	8/20	8/20	-
Min. Spec. Surface	mm^{-1}	25	25	-
Max. Spacing Factor	mm	-	-	0.25
Min. Curing Before Chloride Exposure	days	365****	10	7
Reference No.		20	12	19

* Characteristic value, fresh concrete
** Landworks in Denmark
*** No maintenance the first 50 years
**** It is recommended not to expose the concrete to chloride before it is 1 year old, if at all possible

Table 3. Comparison of specifications for 3 recent bridges in Denmark: Øresund (1993), Great Belt Link (1989) and Farø (1978)

10. Table 4 compares substitute requirements in the 3 latest revisions of DS 411 (Danish Code of Practise for Reinforced Concrete structures): 1984, 1973, 1949, and Basic Concrete Specification for Building Structures (BBB), 1988. It can be seen that DS 411 has been revised over the years in relation to the experience gained, eg. inclusion of pozzolans, air entrainment and restriction of max. temperature gradient.

DS 411/BBB Requirement*	Unit	1st Ed.	2nd Ed.	3rd Ed.	BBB
Published	year	1949	1973	1984	1988
Max. w/c-ratio	kg/kg	0.55	0.60	0.50	0.45
Min. Cement Content	kg/m^3	225	-	-	-
Min. Char. Compr. Strength	MPa	-	-	30	35
Moist Curing	days	7	**	**	4/5
Min. cover	mm	30-50	30	30	-
Max. flyash	%	-	10	***	***
Max. flyash + microsilica	%	-	-	35	35
Max. Microsilica		-	-	10	10
Max. Temp.Gradient	°C	-	-	15	-
Air Entrained		-	**	**	**
Reference		13	13	13	21

* Aggressive environment. Reinforced concrete
** Recommended where necessary
*** Flyash allowed

Table 4. Comparison of the 3 latest revisions of DS 411, the Danish Code of Practice for Reinforced Concrete Structures, and BBB 1988.

Performance of marine structures

11. As stated above, the actual service life of marine reinforced concrete structures in Scandinavia is generally satisfactory, evaluated on visual inspection.

12. Only recently (within the last 15 years) chloride penetration into existing structures has been tested quantitatively.

13. Table 5 shows measured chloride concentrations for selected structures close to sea level. It can be seen that in 11 - 26 years, the critical chloride concentration has penetrated 32 - 62 mm from the surface, i.e. corresponding to commonly used cover thicknesses.

		Langeland Bridge	Stigsnæs Harbour	Farø Bridge	Nerlandsøy (N)	Halsskov Harbour
Construction End	Year	1962	1967	1982	1967	1962
Tested	Year	1978	1978	1992	1993	1978
Level tested	m	+2.5	+2.2	+.35	+2.7	+2.3
Depth for .06% Cl^-	mm	62	55	30	20-60	32
Surface Conc. Cl^- *	%	0.35	0.24	1.1	0.75	0.15
Interior Conc. Cl^-	%	0.02	0.01	0.01	0.01	0.01
Effective Chloride Diffusion Coeff. *	10^{-13} m^2/s	20	26	6 - 7	340	13
Reference No.		1	1	22	27	1

* Calculated using Fick's 2. law

Table 5. Measured chloride concentrations for selected Danish (and one Norwegian) marine structures.

14. Table 6 shows measured chloride concentrations for a selected structure at different levels above sea water. It can be seen that the "splash zone" reaches as high as + 21 m above sea level.

Langeland Bridge. Built 1962. (Tested 1978)								
Level	+2.5	+3.4	+5.1	+7.0	+10.2	+15.0	+21.2	+40
Interior Cl^- %	0.02	0.02	0.01	0.02	0.03	0.02	0.03	0.01
Surface Cl^- * %	0.35	0.23	0.37	0.34	0.35	0.27	0.09	0.01
Depth for Cl^- = 0.06%, mm	62	74	103	87	105	88	42	-
Effective Chloride Diffusion Coeff. * 10^{-13} m^2/s	20	44	46	32	40	35	13	-
Reference No. 1								

* Calculated using Fick's 2. law

Table 6. Measured chloride concentrations in Langeland Bridge at different levels above sea.

Actual selected mix proportions for recent marine structures in Northern Europe

15. Table 7 shows the selected concrete mix proportions for a number of large, marine structures, reflecting the difference in the requirements as shown in tables 1, 2 and 3. The main difference can be seen to be in the use of pozzolans as cement replacement, and the reduction of the water content and the w/c-ratio made possible by the introduction of plasticizers and superplasticizers, and made necessary by the increase of the specified service life. It is to be expected that the w/c-ratio has reached the practical limit now, and that any increase in service life must be obtained by other measures.

Structure	Great Belt Link			Channel Tunnel	Farø Bridge	New Lillebælt	Old Lillebælt
kg/m^3	East Br.	East Tun.	West Br.				
Concrete	A	A1 segments	B	segment	structural concr.	main towers	1:2:2
cement	315	335	320	400	330	310	415
flyash	40	40	38	-	40	-	-
microsilica	23	20	16	-	-	-	-
water	130	128	138	140	140	170	210
sand	575	585	245	670	622		800
aggr.2/8	450	-	575	1310**	285		
aggr.8/16	478	1360	392		***	*	****
aggr.16/32	347	-	586				
air entr.	0.6	-	1.9	-	+	+	-
plast.	1.5	0.8	1.7	+	+	-	-
superpl.	5.0	3.0	6.5	+		-	-
w/c-ratio	0.34	0.32	0.37	0.35	0.45	0.55	0.50
air, fresh	5.5	1.7	6.0	-	4 - 6	4 - 5	
slump	120	10	140	10	50 - 60		
Remarks				France		not pumped	Estimated
Reference	2	12	4	15	19	18	17

* D_{max} = 55 mm
** D_{max} = 12.5 mm. Total 1310 kg/m^3 aggregate 2/8/12.5 mm
*** total 920 kg/m^3 aggregate 8/16/32 mm
**** total 950 kg/m^3 aggregate 2/8/16/32 mm

Table 7. Comparison of actual concrete mix proportions for selected structures.

Test methods for chloride penetration

16. Recently, a number of test methods for determining concrete resistance to chloride penetration have been developed. There are several types

- electrical, where the concrete resistance to passing an electrical charge is recorded: AASHTO T277-831 (Ref. 10), Resistivity (Ref. 24)

- chemical, where the actual penetration of chloride ions into concrete after exposure to actual sea water or concentrated solutions is measured, the so-called chloride bulk diffusion methods: SBF Eastern Railway Tunnel (Ref. 5), APM 302 (Ref. 6), WBS Annex A (Ref. 9). Based on the measured penetration the effective chloride diffusion coefficient is calculated, using Fick's second law of diffusion.

  Alternatively, a double cell diffusion test method is used, referring to steady-state diffusion and Fick's first law (Ref. 25).

- combined electrical and chemical, presently being developed and tested (Ref. 26).

Laboratory testing

Structures	Great Belt, East Bridge			Great Belt, Eastern Tunnel		Great Belt West Bridge	Farø Bridge
Age at Start of Testing	28 d	28 d	28 d	28 d	>28d		10 y
Test Method, Ref.	5	9	10	5	5	6	9
Concrete Type	A	A	A	A1	A1	B	
Specimens Tested	trial mix	trial mix	trial mix	trial mix	seg-ment	trial casting	FF09 +6.5
Exposure	28d-40°C	35d-23°C	35d-23°C	28d-40°C	28d-40°C	35d-23°C	50d-23°C
Surface Conc.*, Cl^- %	1.95	0.81		.68 - 1.03	.34 - .74	0.67-1.6	.4
Cl Bulk Diff. Coeff. 10^{13} m^2/s	36	16	18	2-33 **	3.8-21	11-34 **	12-18
Reference No.	11	11	2+3	8	47	4	22+23

* % of concrete   **2) the big variation is due to the fact that 4 different trial castings of varying quality were tested

Table 8. Comparison of calculated chloride diffusion coefficients from measured chloride concentration in a structure (Farø Bridge) and after accelerated testing of cores (Great Belt Link).

17. Actual exposure condition of concrete in structures differ greatly from that in accelerated test methods. At present there is still no documented relation between performance in structures and in accelerated testing. Table 8 shows effective chloride diffusion coefficients calculated from accelerated bulk diffusion testing and measured chloride concentrations in trial castings (Great Belt Link) and a structure (Farø Bridges).

Conclusions

18. Although the exposure conditions are comparable, the concrete requirements and the resulting concrete mix composition varies between nations as well as with time within a nation, reflecting the development in technology as well as the still uncertain art of service life design

19. The potential resistance of concrete to chloride penetration can be determined in the laboratory using accelerated test methods, eg. chloride bulk diffusion

20. The actual chloride penetration has at present only been measured for a few structures. Of those, not enough is known about the old structures, and the chloride penetration into the young structures is not yet sufficient to predict service life

21. Accordingly, it is not yet possible to document any relation between concrete quality and resistance against chloride penetration in actual structures

22. These relations should be established by testing of recent structures in order to evaluate the effect of revised specifications, intended to prolong the service life, eg. reduced max. w/c-ratio, use of pozzolans etc.

23. The durability of marine structures is not only determined by the requirements to materials or concrete mix proportions and/or the achieved concrete resistance to aggressive components eg. chlo-ride ions, but also by a number of other factors: cover, execution (cracks, honeycombing etc.), environmental exposure, additional protective measures etc.

24. Furthermore, alternative structural materials, such as steel or wood, require continual maintenance. It may be cheaper in future concrete structures to design for maintenance too.

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Appendix 5

Typical mix composition, Courtesy of Ib Bælum Jensen 2014

Unicon A/S

Betonblanket

v.1.13

Side 1 d. 19-06-2014 kl. 07:34:15

Recept.....: UE40LSFEE25LNKS--- Fabrik: Ejby--1 Version 0133

Miljøklasse	EKSTRA - AGGRESSIV
Styrkeklasse	C40
Kontrolklasse	Skærpet
Største sten	25
Tilstræbt konsistens	140 mm
Forprøvning. attestnr.	040003198560

Sammensætning	Navn	Densitet kg/m³	Mængde kg/m³	Volumen l/m³
Cement	Lavalkali CEM I 42,5N(HS/EA<2)	3.200	346,4	108
Flyveaske	Flyveaske B4	2.300	69,3	30
USILIKA	Silika Finnfjord	2.200	17,3	8
Koldt vand	Koldt vand	1.000	153,5	154
Luftiblanding	UM.AIR 22SB (luftindblanding)	1.010	1,0	1
Plastificering	UM.POZZOLITH 20N (sortplast)	1.160	2,8	2
Superplast	UM.GLENIUM SKY 631 (superplast)	1.050	2,2	2
Sand	E0004 RN Avedøre Metro	2.620	615,1	235
Sten	E0408 Dalby	2.720	217,6	80
Sten	E0816 Dalby	2.720	435,2	160
Sten	E1625 Dalby	2.720	435,2	160
Tilstræbt luftindhold i beton, volumen %	6,0			60,0
			2.296	1.000

Kontrolpunkt	**Værdi**	**Beregning**
Microsilica	5 %	100 * 17 / 346
Flyveaske	20 %	100 * 69 / 346
Ækv. Cementindhold	416 kg/m3	346 + 0,5 * 69,3 + 2,0 * 17,3
Effektivt vandindhold	158 kg/m3	0,98 + 1,83 + 1,63 + 153,51
Ækv.V/Cforhold	0,380	158 / 416
Fillerindhold i beton	563 kg/m3	346 + 69 + 17 + 129.56 (<0.25 mm)
Mørtelindhold	614 l/m3	1000 - 386.35 (>4mm)
Ækv. alkaliindhold 60% mør.	1,75 kg/m3	(1,791 * 1000 * 60 / 100) / 614
Alkaliindhold	1,79 kg/m3	
Max. chloridindhold	0,07 %	100 * 0,310 / (346 + 17 + 69)

Chlorid- og alkaliregnskab

Delmateriale	kg/m3	%(Alkali)	kg/m3(Alkali)	%(Chlorid)	kg/m3(Chlorid)
Cement	346,38	0,40	1,39	0,02	0,069
Flyveaske	69,28	0,00	0,00	0,02	0,014
Mikrosilica	17,32	1,00	0,17	0,30	0,052
Luftiblanding	1,00	0,50	0,01	0,30	0,003
Plastificering	2,81	0,50	0,01	0,02	0,001
Superplast	2,17	2,00	0,04	0,01	0,000
Sand	615,12	0,02	0,12	0,02	0,123
Sten	217,60	0,00	0,00	0,00	0,002
Sten	435,20	0,00	0,00	0,00	0,004
Sten	435,20	0,00	0,00	0,00	0,004
Koldt vand	153,51	0,02	0,04	0,03	0,038
Total	2.296,00		1,79		0,310

Kornkurve, gennemfald i %

Sigte, mm.	0.075	0.125	0.25	0.5	1	2	4	8	16	32	64	128
Sammensat Vægt	0	1	8	17	26	33	38	51	77	100	100	100
UE0004RN-AVD-METRO	1	3	20	45	70	90	99	100	100	100	100	100
UE0408DALBY	1	1	1	1	2	3	12	95	100	100	100	100
UE0816DALBY	0	0	1	1	1	1	3	10	95	100	100	100
UE1625DALBY	0	0	0	0	0	0	1	2	15	100	100	100

Betonblanket

v.1.13

Recept.....: UE40LSFEE25LNKS--- Fabrik: Ejby--1 Version 0133

Materialeegenskaber:

sand	UE0004RN-AVD-METRO	Humus	Lys	Kemisk svind	0,08	Reakt. korn	0,50
		Absorption	0,2	Mørtelprism8	-	Mørtelprism20	0,14
		Acc. mørtel	-				
Stone	UE0408DALBY	Korn<2500	0,00	Korn<2400	-	Korn<2200	-
		Absorption	0,4	Krit. abs.	-	Acc. mørtel	0,02
Stone	UE0816DALBY	Korn<2500	0,00	Korn<2400	-	Korn<2200	-
		Absorption	0,3	Krit. abs.	-	Acc. mørtel	0,02
Stone	UE1625DALBY	Korn<2500	0,00	Korn<2400	-	Korn<2200	-
		Absorption	0,3	Krit. abs.	-	Acc. mørtel	0,02

Konsistensvarianter	**Min.**	**Max.**	**Basis**
Konsistens	60,00	200,00	120,00
V/C-forhold	0,38	0,38	0,38
Effektivt Vandindhold	142,60	165,70	155,00
Ækv cement indh.	375,26	436,05	408,00
Sand	692,86	575,93	629,95
Sten	1.088,00	1.088,00	1.088,00
Luftibland.	0,90	1,05	0,98
Tilsæt.	4,49	5,22	4,89
Chlorid indhold	0,08	0,07	0,07
Ækv. Alkaliindhold	1,65	1,87	1,76
Ækv. Alkaliindhold 60%	1,61	1,83	1,72

Appendix 6

Dimitrios Boubitsas, Tang Luping, Peter Utgenannt: Chloride Ingress in Concrete Exposed to Marine Environment -Field Data Up to 20 Years' Exposure, Final draft rapport, CBI betonginstitut, 2014-02-14

Abstract

"This report presents the results from a research project dealing with chloride ingress in concrete exposed to a marine environment after exposure for over 20 years. In the beginning of the 1990s, some 40 types of concrete slabs were exposed to seawater at the Träslövsläge field site on the west coast of Sweden. Through a number of previous research projects the concrete slabs were periodically sampled for chloride penetration profiles after exposure for 0.5–2, 5 and 10 years. In this project, chloride penetration profiles in all of the available concrete slabs after exposure for over 20 years were measured again. These chloride profiles were used for validation of prediction models for chloride penetration. Two models, one empirical and another mechanism-based, were compared with the measured chloride profiles. In the study the corrosion conditions of the rebars embedded in the concrete slabs were measured using a non-destructive method developed based on the principle of galvanostatic pulse technique. A destructive visual examination was carried out to confirm the results from the non-destructive method.

The results show that the chloride ingress is in general more severe in the submerged zone than in the other zones. Multi-pozzolanic additions such as fly ash and silica fume can effectively reduce chloride ingress. The mechanism-based model gives reasonable prediction of chloride ingress from 1 up to 20 years whilst the empirical model based on short-term field data underestimates chloride ingress in concrete with low water-binder ratios and pozzolanic additions. From the predictions of the mechanism-based model, it has been demonstrated that the best measure to achieve 100 years‟ service life with a cover thickness of for example 60 mm is to use either 5% silica fume or 20% fly ash with reduced water-binder ratio ≤ 0.30, or to use a combination of both fly ash and silica fume (w/b 0.35). It seems that a water-binder ratio lower than 0.30 does not further reduce chloride ingress.

The chloride threshold values were estimated from the analysis of corrosion conditions and chloride contents at the cover depth measured after 10 and 20 years‟ exposure. The results make it reasonable to assume a chloride threshold value of at least 1% by weight of binder for initiation of corrosion of reinforcement steel embedded in the marine concrete structures. This threshold value seems valid for various unitary and binary binders including ordinary Portland cement, sulphate resistance Portland cement and blended cement with 5% silica fume, and with different water-binder ratios in a range of 0.3 to 0.5. For the ternary binder blended with 5% silica fume and 10% fly ash with water binder ratio 0.35, the chloride threshold value can be as high as 2% by weight of binder content. These chloride threshold values were based on the results from thin cover thickness. A thicker cover provides a relatively more stable micro-climate with less variation in moisture and oxygen, implying that a higher chloride concentration is needed to initiate corrosion under such a stable climate condition. Therefore, it is reasonable to assume somewhat higher threshold values for steel embedded in concrete with greater cover thickness

Appendix 7

Femern Belt Exposure site, concrete compositions,
Danish Technological Institute, http://www.concreteexpertcentre.dk/30664,2

Concrete ID			Ren Portland	Lav flyveaske	Høj flyveaske	SCC flyveaske	Mikrosilica	3-pulver	3-pulver uden luft	3-pulver højt v/c
			A	B	C	D	E	F	G	H
Powder composition (%-wt)	Low alkali SR cement	CEM I 42,5 N	100	85	75	75	96	84	84	84
	Cement	CEM I 52,5 N								
	Slag cement	CEM III/B 42,5 N								
	Fly ash	EN 450-1 N		15	25	25		12	12	12
	Microsilica	50 wt% slurry					4	4	4	4
	GG blast furnace slag	EN 15167-1								
Concrete composition	Cement	kg/m^3	365	322	300	336	340	300	310	276
	Fly ash	kg/m^3		57	100	112		43	44	39
	Microsilica, solid matter	kg/m^3					14	14	15	13
	GGBS	kg/m^3								
	Water content	l/m3	146	140	140	157	147	140	145	145
	Aggregate 0/2	kg/m^3	695	671	642	678	695	677	731	700
	Aggregate 4/8	kg/m^3	377	374	367	349	377	377	386	380
	Aggregate 8/16	kg/m^3	266	270	271	704	266	272	266	268
	Aggregate 16/22	kg/m^3	529	538	541	-	529	543	530	534
	Air Entraining Agent	kg/m^3	1,7	1,7	2,3	4,0	0,7	1,6	0,0	1,5
	Superplasticizer 1	kg/m^3							3,8	
	Superplasticizer 2	kg/m^3		2,3	2,2					
	Superplasticizer 3	kg/m^3	2,8			2,9	2,7	2,9		2,6

Concrete ID			3-pulver lavt v/c	SCC3-pulver	Slaggecement	Slaggecement uden luft	SCC slaggecement	Rapid og slagge	Superabsorberende polymer
			I	J	K	L	M	N	O
Powder composition (%-wt)	Low alkali SR cement	CEM I 42,5 N	84	84					96
	Cement	CEM I 52,5 N						30	
	Slag cement	CEM III/B 42,5 N			100	100	100		
	Fly ash	EN 450-1 N	12	12					
	Microsilica	50 wt% slurry	4	4					4
	GG blast furnace slag	EN 15167-1						70	
Concrete composition	Cement	kg/m^3	330	350	360	375	410	108	340
	Fly ash	kg/m^3	47	50					
	Microsilica, solid matter	kg/m^3	16	17					14
	GGBS	kg/m^3						252	
	Water content	l/m^3	135	163	144	150	164	144	147
	Aggregate 0/2	kg/m^3	671	687	689	702	686	689	695
	Aggregate 4/8	kg/m^3	374	354	373	381	353	374	377
	Aggregate 8/16	kg/m^3	270	713	263	269	712	263	266
	Aggregate 16/22	kg/m^3	538		525	535		525	529
	Air Entraining Agent	kg/m^3	2,3	2,2	0,8	0,0	1,6	1,0	0,0
	Superplasticizer 1	kg/m^3							
	Superplasticizer 2	kg/m^3							
	Superplasticizer 3	kg/m^3	3,6	3,4	2,3	2,6	2,9	2,9	3,7

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ISSN: 1893-1162

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