THE DURABILITY OF CONCRETE ELEMENTS EXPOSED IN THE TIDAL ZONE

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ABSTRACT

The objective of this paper is to present the results from an investigation of the durability of concrete elements of different mix design, exposed in the tidal zone.

Ten elements (1.5 x 1.5 x 0.5 m) were made of five different concretes with various w/(c+s) ratios (0.45 - 0.83) and microsilica content (0 - 20 %). Each side of the elements was reinforced with wire mesh with 30 and 50 mm covering respectively.

The risk of corrosion was evaluated from the electrochemical potential, electric resistivity and the chloride content. In addition the general condition of the elements was considered from water absorption tests and compressive strength measurements. The elements were investigated after two and five years of exposure.

The risk of corrosion was very low after two years exposure but the chloride content in some elements was at a critical level as far as 30 mm from the surface. After five years the chloride content had increased considerably and the risk of corrosion initiation in some elements was high.

Key-words: Durability, marine environment, microsilica, strength grade.

1. INTRODUCTION

The use of microsilica (MS) in concrete became common in Norway from 1978. The experience on durability properties from existing MS concrete structures is therefore still limited.

At the time when this research program was started, the strength grade was the main design criteria irrespective of the environment of the concrete structure.

When using microsilica, a certain strength grade may be reached for a lower cement content and a higher w/c ratio than without microsilica. Especially for the lower strength grades, the durability of MS-concrete has therefore been questionable.

The objective of the research program referred to in this paper /1,2,3/ was to investigate various durability parameters on medium and high strength grade concretes (C35 and C65) with and without microsilica, exposed to sea water in the splash zone. The research program was started in 1983, and the durability parameters have been investigated after 2 and 5 years of exposure.

2. EXPERIMENTAL

2.1 Mix design and exposure conditions

2.1.1 Concrete mix and specimen design

Five mixes with two strength grades, referred to as C35 and C65 according to the Norwegian Code NS 3473, were investigated.

The materials used were:

- Rapid hardening portland cement (RP 38)
- Microsilica slurry
- Coarse river aggregate with D_{max} = 27 mm
- Coarse river sand with additions of some fine sand in mixes 1 and 2
- Sulfonated lignin as water reduction admixture (Rescon P)

From each of the five concrete mixes two concrete blocks were cast, 1500x1500x500 mm. The mix proportions are shown in Table 1. Two 8 mm wire meshes (type K335) with centre distance of 150 mm, were cast into each concrete block. The mesh dimensions were 1200x450 mm. One mesh was placed with a concrete cover of 50 mm, and one on the opposite side with a cover of 30 mm. The wire meshes were placed 150 mm from the top and bottom of the blocks.

An insulated wire was connected to each mesh and lead out on the top of the block, as shown in Fig 1. The meshes were to be used to monitor the corrosion potential during time of exposure. In addition 20 cubes 100 mm, and 5 prisms 100x100x500 mm, were cast for strength measurements.

The mixing and the casting of the specimens was done by a ready mixed concrete plant (Trondheim Mørtelverk). The casting was done outdoor at a temperature of 15 - 18 °C. The curing temperature during the following night was 2 - 5 °C. Both blocks and the laboratory samples were cast from the same mix. Each mix was approximately 3 m^3. Demoulding took place the day after and the concrete blocks were immediately wrapped into plastic with an opening of approx. 100 mm between the concrete and plastic sheet. This was done in order to reduce the temperature difference in the specimens and to prevent early age cracking. Some days later the plastic was removed and the specimens were then flushed with water several times a day. The cube and prism were placed into fresh water of 20 °C immediately after demoulding.

Table 1. Mix proportions and results from fresh concrete properties

Materials/properties	Weight of materials (kg/m^3) - results				
	C35-0	C35-10	C35-20	C65-10	C65-20
Cement, RP 38	370	275	234	457	394
Water	198	192	194	204	218
Microsilica	0	27.5	46.8	45.7	78.8
Sand	893	842	847	752	749
Gravel	897	1030	1032	919	905
Plasticizer (Resc. P)	2.0	2.8	4.7	8.0	7.9
Slump (cm)	10	13	12	11	14
Density (kg/m^3)	2360	2370	2360	2390	2350
Air content (%)	2.6	3.2	2.0	1.2	1.3
w/c ratio	0.54	0.70	0.83	0.45	0.55

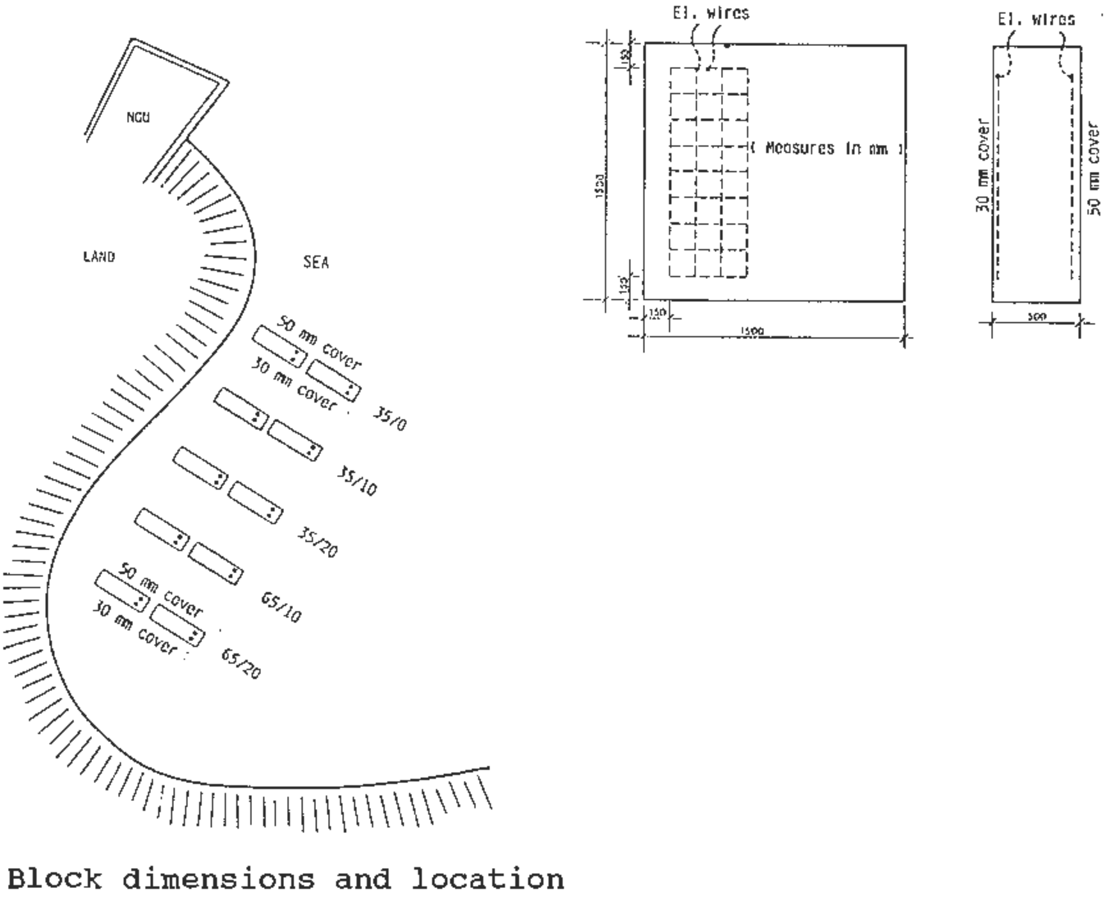


Fig 1. Block dimensions and location

2.1.2 Exposure conditions

The specimens were placed facing north-east in a small bay at Trondheimsfjorden just outside Trondheim. Fig 1 shows the orientation of the specimens and also what concrete cover that is facing the sea.

In the Trondheim area the normal tidal zone is 1.83 m. At low tide the specimens are at dry conditions. At high tide around 0.20 m of the specimens is above water level. However the waves will occur in a splash zone condition at all times. During winter the specimens are periodically exposed to frost.

2.2 Potential mapping and electric resistivity.

Potential mapping has been done on the concrete specimens after 2 and 5 years of exposure, and the results are discussed in the next chapter. These measurements are done in order to find indications of steel reinforcement corrosion. Such measurements give no evidence of the corrosion rate, but give indication if the environment around the reinforcement have been disturbed either by chloride ions penetrating through the concrete, or by carbonation of the concrete. The measurements are based on established electrochemical principals, but the correlation between potential values and corrosion risk is empirical. In addition, the electric resistivity of the concrete has to be taken into consideration.

The electrochemical potential (ELP) of the steel reinforcement has been measured by use of a reference electrode, here the $Cu/CuSO_4$ - electrode. The following threshold values empirically based on years of experience from different types of constructions have been found to be valid:

$E < -350$ mV	High corrosion risk
-200 mV $< E < -350$ mV	Possible risk of corrosion
$E > -200$ mV	Low corrosion risk

These values have to be combined and evaluated together with measurements of electric resistivity, because local variations of the resistivity influence the ELP variations. The electric resistivity influence the corrosion rate. If potential measurements show that corrosion is possible, the following limits of resistivity values (specific resistance) are found to be valid.

$\rho > 12000$ Ohm cm	No risk of corrosion
$5000 < \rho < 12000$ Ohm cm	Possible risk of corrosion
$\rho < 5000$ Ohm cm	High risk of corrosion

The resistivity of concrete is measured by drilling four holes with a centre distance of 100 mm and a depth of 50 mm into the concrete. Steel electrodes are treated with high conductivity gel and placed into the holes. The resistivity is then read out on a conductivity meter (Metraterr 2).

2.3 Chloride penetration

When concrete is exposed to marine environments, the risk of chlorides penetrating into the concrete is always possible. If the penetration of free chloride ions reaches the steel reinforcement, the passive layer on the steel surface can be broken and corrosion can be initiated.

In this test the penetration depths of chlorides have been measured after 2 and 5 years of exposure. The 2 years samples were taken from drilled cores that were sawn into disks of approximately 10 mm thickness. The disks were crushed into fine powder, and the powder was boiled with distilled water for 15 minutes. The solution was then filtered and analyzed by using a specific chloride ion electrode, and the total chloride content of the concrete was read out.

The chloride test after 5 years of exposure was done in a different way. The samples were taken by drilling 12 mm holes in intervals of 15 mm. The powder from each interval was collected and brought to the laboratory for analysis of the total chloride content. The method used here was a rapid chloride test, referred to as the "QUANTAB" - test /4/.

There is some discussion of threshold values due to corrosion risk and chloride content, but in British Standard CP 110, the threshold value is set to 0.35 % chloride ion of cement weight for reinforced concrete. For prestressed concrete the value is lower. In the Norwegian Code NS 3420, the threshold value in the concerete is 0.40 % for reinforced concrete and 0.10 % of cement weight for prestressed concrete, based on chloride addition in fresh concrete.

2.4 Water absorbtion

2.4.1 General

Water absorbtion test was done after 5 years of exposure. The samples were cores of 100 mm diameter. 4 cores were drilled out from one block of each concrete mix. Two cores were taken near the top of the blocks and two cores were drilled out from the bottom part of the specimen. One disk with 20 mm height was sawn from each core, so that one side of the disc was cast skin surface. The test after sawing was carried out by the following procedure:

- Drying at 105 °C for 3 days - Weighing
- Water absorbtion for 4 days - Weighing
- Submerged in water for 3 days - Weighing
- Submerged in water at 5 MPa pressure - Weighing
- Drying in 105 °C until constant weight - Weighing

From the results the following parameters can be determined:

- Resistance number (m)
- Capillarity number (k)
- Total porosity and absorbtion porosity accessible for water

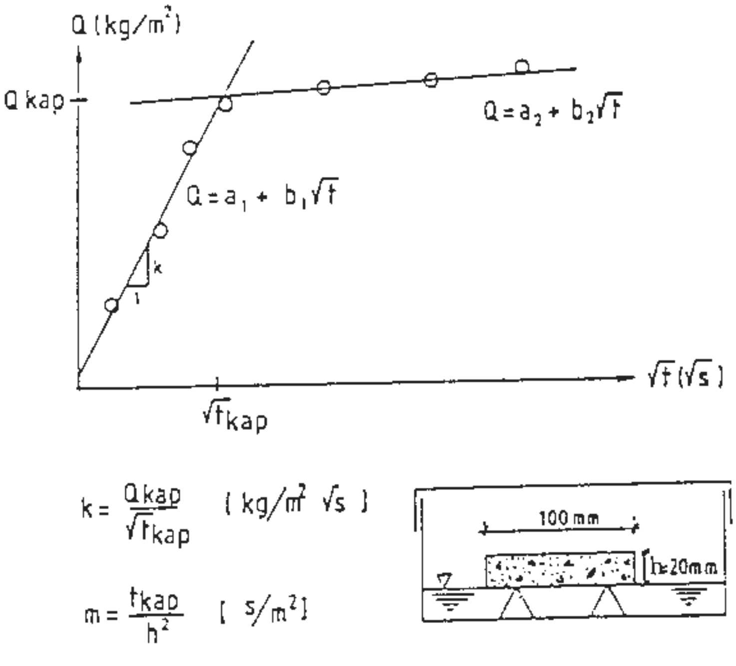
2.4.2 Resistance number and capillarity number

The water absorption in the specimen is approximately proportional to the square root of the time until the point in time which the water has reached the upper surface of the specimen, see Fig 2. The movement of the water can be calculated as:

$$h = \sqrt{(t/m)}$$

Where h is the distance from the water front to the water exposed surface, t is the time and m is the resistance number.

The relation between the amount of water absorbed and the square root of time is expressed mathematically by using linear regression analysis, see Fig 2 /7/.



$$k = \frac{Q_{kap}}{\sqrt{t_{kap}}} \quad [\mathrm{kg/m^2 \sqrt{s}}]$$

$$m = \frac{t_{kap}}{h^2} \quad [\mathrm{s/m^2}]$$

Fig 2. Principle for determination of resistance number, m, and capillarity number, k, based on the relation between water absorption and time

The resistance number m, expresses the time in which the water reaches the top of the specimen, which reflects the fineness of the gel and capillary pore system rather than the total porosity, and therefore a good correlation between m and the w/c ratio is expected. Because of this correlation the resistance number can be used as a quality criterion for concrete.
The capillarity number expresses the amount of water absorbtion per unit time and is in contrast to m, influenced by the total volume of gel and capillary pores in the concrete, and therefore also by the ratio between volume of paste and aggregate. Air voids are assumed not to be filled by water suction.

2.4.3 Total porosity and gel/capillary porosity

The total porosity is calculated from the difference between the weight of the specimen after water saturation at 5 MPa and the weight after drying at 105 °C.

Gel/capillary porosity is calculated from the difference between the weight of the specimen after water saturation at atmospheric pressure and drying at 105 °C. The difference between total porosity and gel/capillary porosity express a measure of the air content of the concrete. This can be used to evaluate the frost resistance of the concrete. The results from this test are discussed in the next chapter.

2.5 Compressive strength

Compressive strength of the concrete have been determined after 4 weeks and after 5 years of exposure. Both cores and cubes were used for these tests.

After 4 weeks of curing, cores with 100 mm diameter and height, were drilled out from one block of each mix. The same drilling was done after 5 years of exposure. The cores were taken out at the centre of each block. The remaining holes were sealed off with mortar. The compressive strength was also determined on 100 mm cubes, stored in water in the laboratory at 20 °C for 4 weeks and 5 years respectively.

3. RESULTS

3.1 Visual inspection

The lower half of the blocks were overgrowed with algae and shells. The overgrowing was removed before drilling of samples. Scaling due to freezing and thawing, was observed on the C35-10 and the C35-20 blocks. The scaling was more extensive after five years than that after 2 years exposure. No damages due to corrosion of the reinforcement could be seen on the element surfaces.

3.2 Evaluation of corrosion risk

3.2.1 General

In general, the electrochemical potential (ELP) and resistivity have decreased considerably from 2 to 5 years of exposure, especially on the side with 30 mm covering.

The chloride content was determined at the top and at the bottom of each side of all mixes after 5 years of exposure. There was no significant relation between the chloride content and the sample location.

In the following the corrosion risk is evaluated with respect to w/c ratio and microsilica content. The evaluation is based on the average ELP, resistivity and the chloride content. The average ELP is the mean value from measurements on the middle of the 30 mm covering side of each block from each mix. The average resistivity is the mean value from two measurements on the middle of the 30 mm covering side of each block from each mix.

The average chloride content after 2 years of exposure is the mean value from two measurements on specimens from the middle of the 30 mm covering side of one block from each mix. The average chloride content after 5 years is the mean value from measurements on the bottom and on the top of the same blocks. Since different methods were used to determine chloride contents, see article 2.3, the increase in chloride content from 2 to 5 years shown in Figs 5-7, are only indications.

3.2.2 Effect of w/c ratio

Figs 5 and 6 shows the chloride profiles in the two mixes with 10% and 20% MS respectively. The cement content i e the paste/aggregate ratio is much higher in the two mixes with the lowest w/c ratio, see Table 1. It is known that the permeability and hence the chloride penetration increase at increasing paste/aggregate ratio. Still the chloride content is considerably lower, and below the critical level, in the two low w/c ratio concretes. Concequently, the positive effect of reducing the w/c ratio, is significant. The chloride content in both 0.70 and 0.83 w/c ratio concretes was higher than the critical limit even after 2 years.

The average ELP, see Fig 3, does not indicate corrosion in the high w/c ratio concretes, in spite of a very high chloride content for at least three years. However, the ELP has locally dropped considerably from 2 to 5 years (C35-10), and also the resistivity has dropped considerably, see Fig 4, in the period. The results therefore indicate that the initiation period is nearly ended and that corrosion is near to begin in the 0.70 w/c ratio concrete in particular.

The results discussed in this article also shows that MS has a favorable effect on the ELP and resistivity since the risk of corrosion seems to be about the same for the three C35 mixes in spite of an increase of the w/c ratio as the MS addition increase.

3.2.3 Effect of microsilica content

The w/c ratio of mix C35-0 (no MS) and C65-20 (20 % MS) is 0.54 and 0.55 respectively, and the cement content is 370 and 394 kg/m^3 respectively, see Table 1. It is therefore possible to determine the effect of adding 20% MS to the concrete, with w/c ratio approx. 0.55, on the corrosion risk is.

As can be seen from Fig 7, the average chloride content in the two mixes is very similar. The average chloride content was approximately doubled from 2 to 5 years, and the average content at 30 mm depth was higher than the critical limit according to NS 3420, see article 2.3.

The average ELP indicates that corrosion is not initiated, see Fig 3. However, the ELP in one of the blocks from the mix without MS, has dropped considerably from 2 to 5 years, and was relatively low after 5 years. The resistivity in this mix was also relatively low at this time, see Fig 4.

Based on the above mentioned, it seems that the corrosion can be initiated within a relatively short time in the mix without MS. 20% MS addition does not seem to effect the total chloride content. This is in contrast to the results presented by Fisher et al and Vennesland /9,10/. The MS seems to have a favorable effect on the ELP and resistivity of the concrete, which also is mentioned in article 3.2.2.

As can be seen from Figs 5 and 6, the total chloride content in the mixes with w/c ratio 0.70 and 0.83 was very high even after 2 years exposure. Still, the ELP indicates that the corrosion is not initiated even after 5 years. It is known that only free chlorides in the concrete are effective with respect to corrosion of steel reinforcement. The results are based on the total chloride content, and the ratio between the amount of free chlorides and the amount of chlorides bound in the paste, is not found. Byfors /5/ found that the chloride binding capacity in OPC paste added MS is higher than that in the OPC paste without MS. One possible explanation for that corrosion seems not to be initiated, in spite of the high total chloride content, is that a relatively high amount of the chlorides are bound and therefore not effective. However, Page and Vennesland /11/ found that the chloride binding capacity in MS conrete was lower than in concrete without MS. Explaination for the conflicting results is not found.

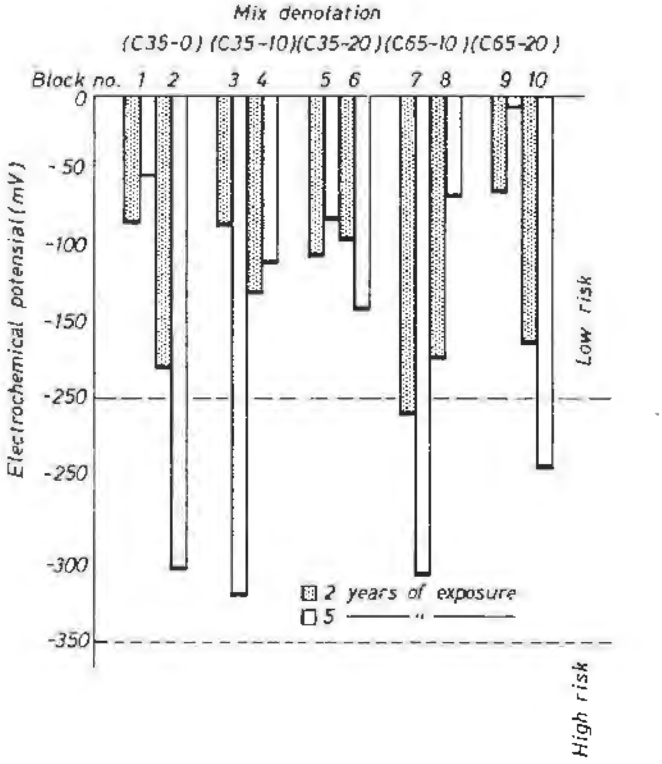


Fig 3. Average electrochemical potentials in the concretes with 30 mm reinforcement covering, after 2 and 5 years exposure in the splash zone

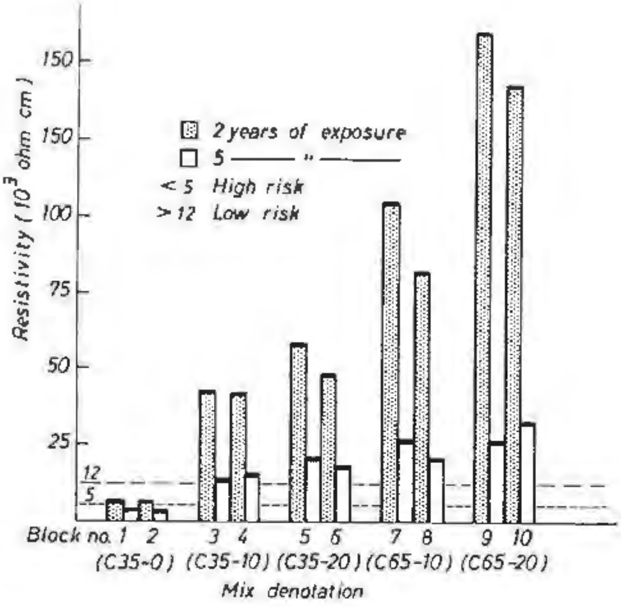


Fig 4. Average electric resistivity in the concretes with 30 mm reinforcement covering, after 2 and 5 years exposure in the splash zone

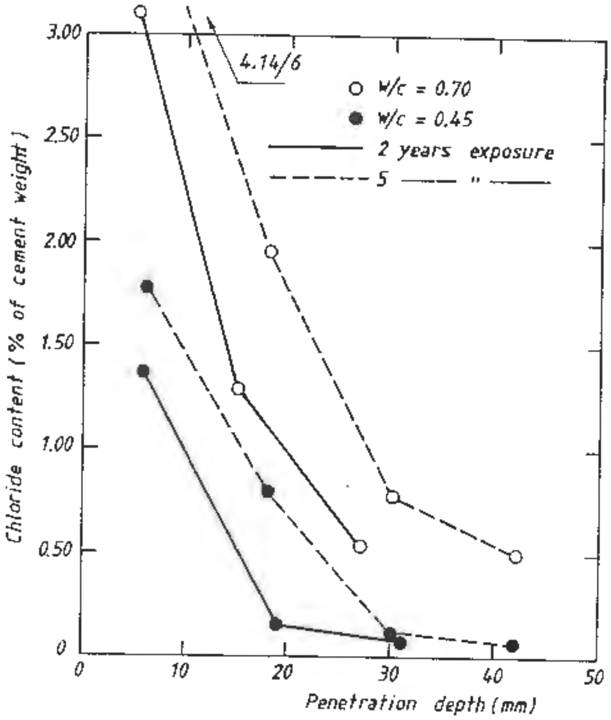


Fig 5. Chloride profiles in concrete with 10% microsilica (MS) and different w/c ratios (C35-10 and C65-10) after 2 and 5 years exposure in the splash zone

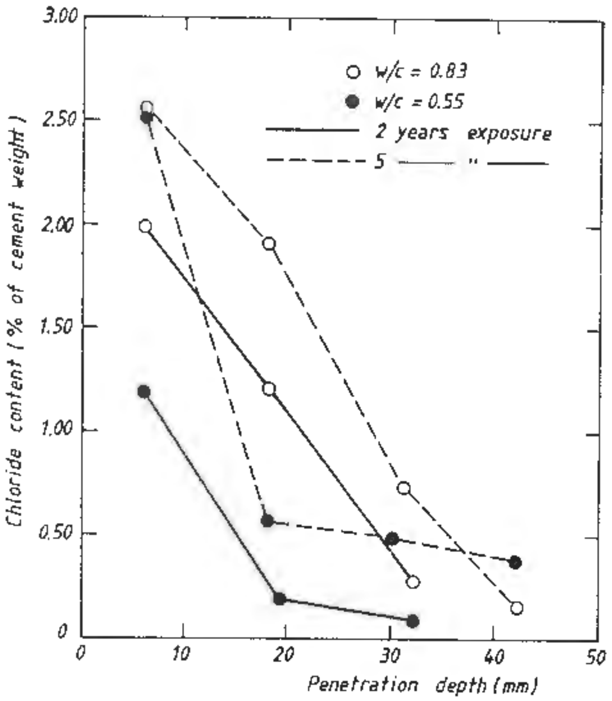


Fig 6. Chloride profiles in concrete with 20% microsilica (MS) and different w/c ratios (C35-20 and C65-20)after 2 and 5 years exposure in the splash zone.

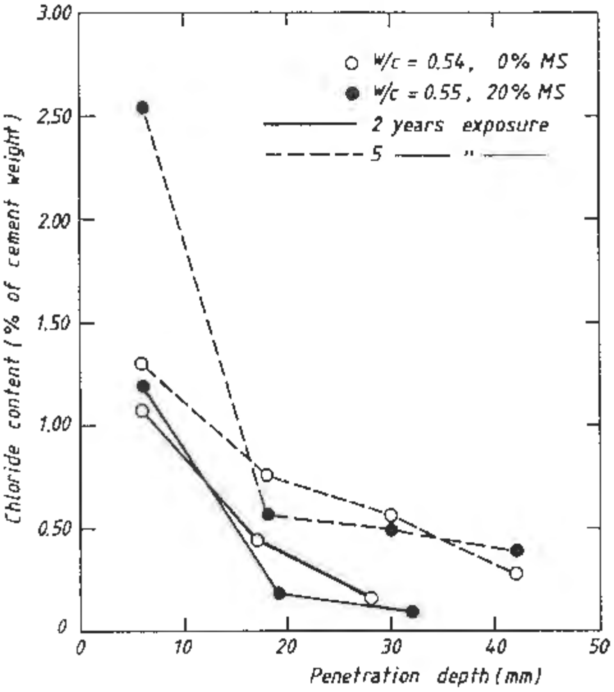


Fig 7. Chloride profiles in concretes with 0% and 20% microsilica (MS) (C35-0 and C65-20) after 2 and 5 years exposure in the splash zone.

3.3 Water absorption properties

As mentioned in article 2.4, the resistance number can be used as a quality criterion for concrete, and the number reflects the fineness and thereby the permeability of the concrete. The following discussion is therefore based on the determined resistance number, m.

The results are shown in Table 2 and Fig 8. The results from testing of the C65 concretes are not significant, because the absorption curves, see article 2.4, deviate from the normal pattern, i e the break point could not be well defined.

As can be seen, the resistance numbers for the C35 concretes are considerably higher at the bottom of the blocks than that at the top of the blocks. This is probably an effect of a tightening of the pore system, due to marine growth and deposition due to a chemical reactions between concrete and sea water elements. Bizcok /6/ has described a model of how sea water attacks the concrete. It can be seen from this model, that the first step is a chemical reaction between mainly magnesium chloride in the sea water and the calcium hydroxide in the concrete. The reaction products contributes to a tightening of the pore system. The bottom of the blocks are submerged in sea water for 80-90% of the time, while the top is submerged a few times a year, when the tide is very high. Consequently, the tightening effect would be much more distinctive at the bottom of the blocks than that at the top. The resistance numbers at the top of the blocks correspond fairly well to the resistance numbers found earlier in approximately similar concretes, cured in fresh water /7/, which confirms a tightening effect. As can be seen the tightening effect is most distinctive in the concretes with lowest resistance number. The resistance number at the bottom is relatively similar for the concretes. Results from an another ongoing project at FCB also show a tightening effect of concrete exposed in sea water /8/. It can also be seen that the gel/capillary porosity in the C35 concretes is lower at the bottom than that at the top, which further confirms the tightening effect.

The results reported in /7/ also show that the resistance number increase at increasing MS content, which also can be seen in the C35 concretes (top), see Fig 8.

A relatively good correlation between the resistance number and the chloride penetration is expected. However, there was no significant difference between the chloride penetration at the bottom and at the top of the blocks, as mentioned in article 3.1, while there was a considerable difference in the resistance number. The reason for this may be that the chloride penetration mechanism at the top and at the bottom is different. The access of sea water at the top is very low, which should contribute to a low chloride penetration. However, the drying/rewetting (due to splashing) will contribute to an accumulation of chlorides in the concrete, because after every sea water suction and the following drying, the chlorides will remain in the concrete.

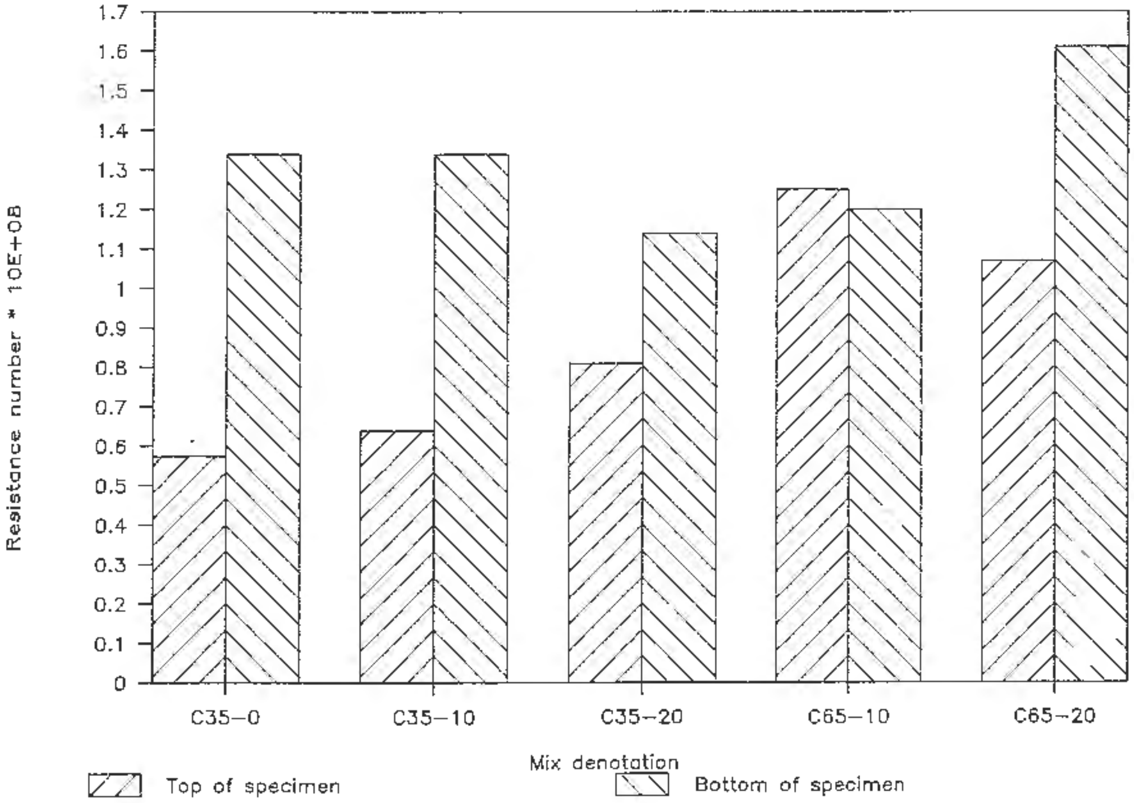


Fig 8. Resistance number in the concretes, determined at the bottom and at the top of the blocks

Table 2. Results from water the absorption tests

Block no and mix denotation		m (s/m^2) x E+08	k $kg/m^2/\sqrt{s}$ x E-02	Total porosity (%)	Gel/capillary porosity (%)	Air Cont. (%)
2	top	0.575	1.46	16.8	15.1	1.7
C35-0% MS	botm	1.34	1.07	15.6	13.8	1.8
3	top	0.64	1.60	16.8	15.0	1.8
C35-10% MS	botm	1.34	1.12	15.7	14.3	1.4
6	top	0.801	1.47	17.5	15.9	1.6
C35-20% MS	botm	1.14	1.18	16.8	15.1	1.7
7	top	1.25	1.06	17.3	14.8	2.5
C65-10% MS	botm	1.20	1.08	15.4	13.9	1.5
9	top	1.07	1.33	18.8	16.9	1.9
C65-20% MS	botm	1.61	1.10	19.8	17.7	2.1

3.2.4 Compressive strength

In general, the sea water exposure seemed not to have any negative effect on the compressive strength, compared to pure water curing in the laboratory, see Table 3. There was no significant relation between strength level or MS content and the increase of compressive strength from 28/30 days to 5 years. Apparently, it seems that an increase of the MS content caused an increase of the relative "in situ strength" (cylinders). This effect is not observed for the lab cured cubes. The reason for this may be that the curing temperature in the blocks was relatively low compared to the lab cured cubes, see article 2.5. It is known that the strength development in MS concretes are more sensitive to the curing temperature than concrete without MS.

Table 3. Results from compressive strength measurements after 28/30 days and 5 years

Mix denotation		Cylinders 1)		Cubes 1)	
		30 d (MPa)	5 yrs (MPa)	28 d (MPa)	5 yrs (MPa)
C35 0 % MS	1	37.7	44.8		47.7
	2	40.5	45.7		51.0
	3	39.0	41.8		54.7
	4		44.2		
	Mean	39.1	44.1 (12.8)	40.2	51.1 (27.1)
C35 10 % MS	1	42.0	51.3		53.2
	2	40.1	47.6		52.4
	3	39.4	46.7		50.2
	4		43.1		
	Mean	40.5	47.2 (16.5)	42.6	51.9 (21.8)
C35 20 % MS	1	36.8	50.5		55.7
	2	38.8	46.2		53.9
	3	39.4			56.3
	4				
	Mean	38.3	48.4 (26.4)	45.2	55.3 (22.3)
C65 10 % MS	1	62.1	75.2		87.9
	2	59.1	70.9		85.5
	3	65.0	73.7		85.0
	4		73.6		
	Mean	62.1	73.4 (18.2)	71.6	86.1 (20.3)
C65 20 % MS	1	55.2	70.1		80.1
	2	55.5	65.4		83.3
	3	58.4	73.7		80.5
	4		69.7		
	Mean	56.4	69.7 (23.6)	67.0	81.3 (21.3)

1) Numbers in brackets represent the increase in percent from 4 weeks to 5 years.

4. CONCLUSIONS

The condition of concrete blocks (1500x1500x500 mm) exposed in the tidal zone for 5 years, has been investigated mainly with respect to corrosion of the reinforcement. The blocks are made of five different concretes with w/c ratios in the range of 0.45 - 0.83, and 0, 10 and 20% microsilica (MS). In general, there was not found any indications that corrosion has been initiated in any of the blocks.

The total chloride content at the reinforcement with 30 mm covering, was found very high, and above a critical level, in the concretes with w/c ratio 0.70 and 0.83 and 10% and 20% MS respectively, even after 2 years of exposure. In spite of this the electrochemical potentials and the electric resistivity indicated that corrosion was not initiated even after 5 years of exposure. However, there was some indications that the time before initiation is relatively short. After 5 years also the chloride content in the concrete with w/c ratio 0.54 and 0% MS was above the critical level at the 30 mm covering. The time before corrosion initiation in this concrete also seems to be relatively short. The chloride content in the two last mixes, i e with w/c ratio 0.45 and 0.55 added 10 and 20% MS respectively, was relatively low.

The results show that the w/c ratio has a considerably effect on the chloride penetration. The MS addition did not seem to have any significant effect on the chloride penetration in the concretes. However, in spite of a relatively high total chloride content, the corrosion risk in the MS concretes seemed to be low. This is probably due to a higher electric resistivity, but a possible increase of the chloride binding capacity (decreased content of effective chlorides) when adding MS, can also be an explaination.

The water absorption tests showed that the permeability at the bottom of the blocks was lower than that at the top. This is probably due to marine growth and deposition due to chemical reaction between concrete and sea water elements.

In general, the sea water exposure seemed not to have any negative effect on the compressive strength, compared to pure water curing in the laboratory.

5. ACKNOWLEDGEMENT

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6. REFERENCES

/1/ Maage, M and Sandvik, M.: Betongblokker for langtidsforsøk. SINTEF-report STF65 A84030, Trondheim, Norway 1984. In Norwegian.

/2/ Hammer, T A and Havdahl, J.: Betongblokker for langtidsforsøk etter ca 1.5 års eksponering. SINTEF-report STF65 A86003, Trondheim, Norway 1986. In Norwegian.

/3/ Hammer, T A and Havdahl, J.: Betongblokker for langtidsforsøk etter ca 5 års eksponering. SINTEF-report STF65 A89034, Trondheim, Norway 1989. In Norwegian.

/4/ BRE publication IS 12/77, Simplified method for determination of chlorides in hardened concrete. England 1977.

/5/ Byfors, K.: Chloride Binding Capacity in Cement. Nordic Concrete Research, publication no 5, Oslo 1986. pp 27-38.

/6 / Biczok, I.: Concrete Corrosion Concrete protection, Akademiai Kiado, Budapest 1972, pp 215-222.

/7/ Smeplass, S.: Kapillærabsorpsjon som kvalitetskriterium. SINTEF-report STF65 A88028, Trondheim, Norway 1988. In Norwegian.

/8/ Smeplass, S, Meland, I and Rønne, M.: Tilstandskontroll av sjøvannseksponert betong. SINTEF-report in preparation. In Norwegian.

/9/ Fisher, K P, Bryhn, O and Aagaard, P: Corrosion of Steel in Concrete: Some Fundamental Aspects of Concrete with Added Silica. Report no 51304-06, The Norwegian Geotechnical Institute, Oslo, Norway 1982.

/10/ Vennesland, Ø: Silika i Betong - Delrapport nr 3 Armeringskorrosjon. SINTEF-report STF65 A81031, Trondheim, Norway 1981.

/11/ Page, C L and Vennesland, Ø: Pore solution composition and chloride binding capacity of silica fume- cement pastes. Materials and Construction, 16, 19, 1983.