LONG-TERM DURABILITY OF CONCRETE

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ABSTRACT

The objective of the present project is to investigate the long-term durability characteristics of various types of concrete exposed to various environments. The project is carried out as a semi-full scale project with a total concrete volume of 2 m^3 for each of the 16 different types of concrete.

The activities with respect to the full-scale specimens include destructive as well as non-destructive testing.

In addition to the larger specimens a number of laboratory specimens were cast in order to provide initial data on the quality of the concrete.

1. INTRODUCTION

The project "Long-term Durability of Concrete" described in the present paper is sponsored by Danalith A/S, Færdigbeton Aalborg A/S, F. L. Smidth & Co. A/S, KH Beton A/S and Aalborg Portland.

These companies together represent vital parts in the production of concrete, e.g. the cement producer, the ready-mixed concrete producer and the contractor.

The aim of the project is to evaluate the long-term durability of various types of concrete exposed to various realistic conditions. The project should be seen in the light of the evolution during the last decade with respect to modern concrete technology. The use of mineral additions plays an important part in most countries today, and in Denmark pulverized fuel ash (flyash) and microsilica are used to an increasing extent.

Flyash has been used for many years in other parts of the world and also microsilica has been used for several years, primarily in Norway.
Wide experience in the use of these materials, therefore, already exists. However, practical experience often suffer from the lack of sufficient documentation and proper references while, on the other hand, laboratory experiences traditionally are based on idealized conditions with very little similarity to reality.

The present project makes it possible to follow the general conditions of a number of concrete specimens which in contrast to most laboratory experiments are produced in large scale and exposed to realistic conditions, and which in contrast to most practical experience are very well documented.

The project started in 1983, and is planned to continue up to the year 2008 which gives a total duration of 25 years.

2. PROJECT DESCRIPTION

2.1 Concrete mix design

Relevant combinations of cement and mineral additions are tested on three levels of quality leading to a total of 16 different concrete mixes.

The quality has been characterized by the cement content which was 260, 340 and 390 kg/m^3 respectively. With an approximately constant water demand of 140 l/m^3 for the various concrete mixes this corresponds to w/c-ratios in the range of 0.35 to 0.55.

The cements used are three Danish cements, a rapid hardening portland cement (PC(R)), a portland flyash cement with 25 percent flyash (PFC(A)) and a low alkali sulphate resistant portland cement (PC(A/L/S)). The mineral additions are a Danish flyash and a microsilica imported from Norway.

When mineral additions were used, a reduction in the cement content was made corresponding to a cementing efficiency factor of 0.3 for flyash and 3.0 for microsilica. The dosages of flyash and microsilica were 20 percent and 10 percent of the cement content, respectively.

Sound aggregates were used to avoid durability problems related to aggregates. A sand free of alkali-reactive material and free of porous limestone was used together with a rounded granite with a maximum particle size of 16 mm.

All mixes were air-entrained with an air content of 5-6 percent as measured on site prior to concreting. The specified slump was 60-80 mm. These properties were achieved by use of an air-entraining agent of the vinsol resin type and a plasticizer based on lignosulphonate.

2.2 Casting

The concrete was mixed at a ready-mixed concrete plant and delivered on site. Before casting the slump and the air content were measured. It was allowed to add one extra dosage of air-entraining agent on site to meet the specified requirements.

When accepted the concrete was cast into 3 slabs each 1.0 x 1.0 x 0.2 m, 2 wall-panels each 1.2 x 1.0 x 0.2 m and a third wall-panel 2.0 x 1.0 x 0.2 m (see figure 1).

The concrete for the slabs was vibrated with a screed-vibrator trowelled by hand and finally broomed with a fibre-broom to achieve a uniform grooved surface. Immediately after finishing, a curing membrane was sprayed onto the surface to prevent loss of water from the concrete.

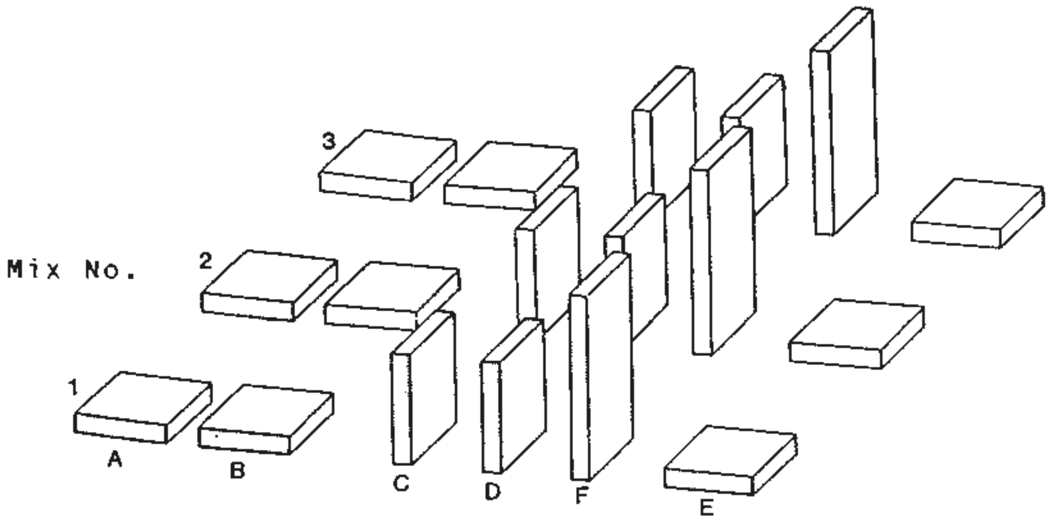

Figure 1. Arrangement of slabs and walls on the outdoor test field

The wall panels were cast vertically and vibrated with a poker vibrator, and the formwork made of plywood resulted in a smooth surface. Loss of water could not take place until the formwork was striked after 7 maturity-days. Immediately after removal of the formwork the concrete surfaces were sprayed with a curing membrane.

During the casting process relevant climatic data were registered. Figure 2 shows a plan of the test field set-up.

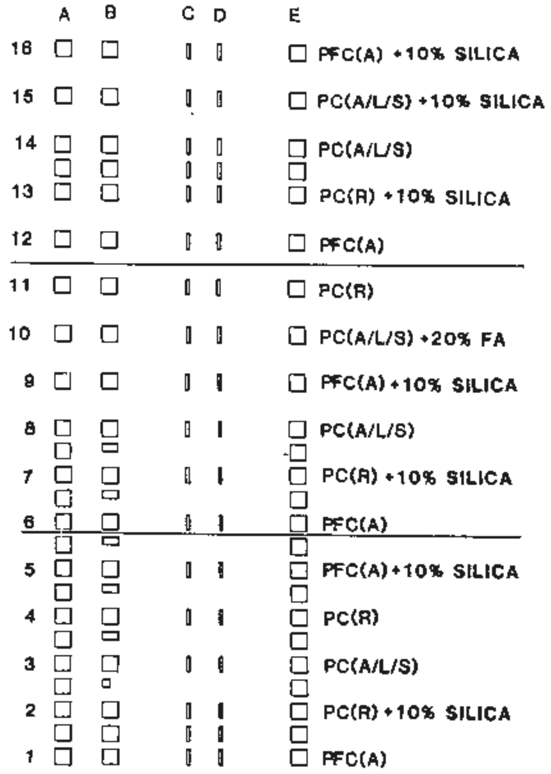

Figure 2. Plan of test field with 16 different mixes

In addition to these large specimens a number of laboratory specimens (beams, cylinders, slabs) were cast on a mobile vibrating table. After one day in a heated (20°C) shed they were transported to the laboratory for further curing.

2.3 Exposure

For each of the concrete mixes three slabs and three wall panels were cast.

Two of the slabs were exposed to normal outdoor conditions at the test field.
The third slab was placed in the same environment except that the surface was treated with de-icing chemicals during winter time.

Two of the wall panels were exposed to normal outdoor environment at the test field and they only differ from the slabs as regards their orientation. The third (and largest) panel was transported to the harbour of Hirtshals at the North Sea, where it was placed in the splash zone to illustrate the influence of chloride and sulphate in combination with freezing, thawing and splash, see figure 3.

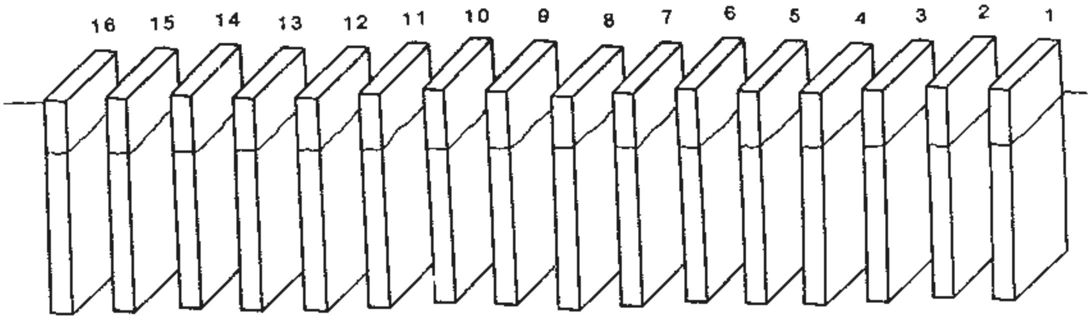

Figure 3. Arrangement of wall panels in splash zone

It is expected that the most aggressive environments will cause durability problems for some of the poorest concrete mixes within a few years.

The laboratory specimens were cured in water at 20°C, except those used for measuring drying shrinkage and freeze/thaw resistence.

2.4 Test programme

During the mixing of the concrete at the ready-mix plant, samples were taken of the cement and the mineral additions used for each particular mix. These samples were tested in the laboratory for chemical composition and physical properties according to Danish standards.

The air content, bulk density and slump were measured at the plant and on arrival at the site prior to acceptance. After casting, the air content and slump were measured again to see if any major changes had occurred during the operation.

During the process of casting the climatic conditions were monitored, i.e. wind, sun and temperature.

While the full-scale specimens were cast, the laboratory specimens were prepared on a mobile vibrating table. A total of 40 specimens were made for each mix and after one day of hardening in a temperature-controlled shed, these specimens were transported to the laboratory for further handling.

Three specimens were used for each of the following tests :

- Compressive strength, modulus of elasticity, pulse velocity and bulk density at ages of 7 and 28 days, 1, 5 and 10 years. Cylinders with a diameter of 100 mm and a height of 200 mm were used.
- Modulus of rupture measured after 28 days, 1 and 5 years in a three-point bending test of beams (100x100x500 mm).
- Drying shrinkage. After 28 days wet curing prisms (100 x 100 x 400 mm) were subjected to a drying environment with 60-65 percent RH at 20° C.
- Freeze/thaw experiments with de-icing chemicals. The slabs (200 x 200 x 50 mm) were cured for 14 days in water and 14 days in air before they were subjected to 25 freeze/thaw cycles (+20/-20° C) at a rate of one cycle per day. After 10, 20 and 25 cycles the amount of scaling was measured.
- Permeability to gas (oxygen) will be measured on well hardened specimens (ø 150 x 50 mm) according to a method proposed by Cembureau /1/.

The full-scale specimens are used for various purposes, as described in the following.

<u>Slab A</u> has in addition to a reinforcement mesh placed in the centre of the specimen three smaller arrangements of reinforcement placed with 10, 20 and 30 mm of concrete cover, respectively.

These are used for electrochemical measurements of the potential of the reinforcement according to a method developed at the Danish Corrosion Center /2/. The method comprises measurement of the potential between the steel and a reference electrode at the point of interest. A saturated calomel electrode (SCE) is used as reference.

For each cover thickness 10 potential measurements are made. According to the programme, measurements shall take place after 1, 2, 5, 10, 15, 20 and 25 years.

Furthermore, slab A is used for characterizing general weathering and freeze/thaw-durability in a moderately aggressive environment. This part is documented with photos during the annual inspection.

Slab B is used for drilled cores. A total of 16 cores with a diameter of 100 mm and a height of 200 mm can be taken from this slab.

Cores are planned to be drilled and tested after 28 days, 1, 5, 10 and 25 years. At each age two cores are used for compressive strength measurements.
The same cores are also used for measuring pulse-velocity, bulk density, modulus of elasticity and depth of carbonation. In addition to these cores one extra core is taken at ages of 28 days, 5 and 25 years for microscopic analyses of air-void characteristics and evaluation of the homogeneity and porosity of the concrete and the cement paste. Various defects, i.e. micro and macro cracks, are observed.

Wall C is used for the same purpose as Slab B, and simply represents different ways of casting and curing as well as differences in exposure due to the different orientation of the specimens. Cores for microscopic analyses, however, are not taken from Wall C.

Wall D is used for the same experimental procedures as Slab A, and the results from these specimens also reflect differences due to their orientation during casting and exposure.

Slab E is identical with Slab A apart from systematic treatment of the surface of Slab E with de-icing chemicals during a 4 months period in winter time.

Wall F was cast on site and after a hardening period of 2 months it was transported to Hirtshals at the North Sea and partly submerged into sea water. The wall is placed in a steel arrangement from which it can be lifted upon the quay for inspection and testing.

The wall is placed at a level where the upper part (1 meter) is exposed to splash and tidal effects while the lower part is continuously under water.

Testing of the upper half of the wall is identical with Wall D (corrosion, freeze/thaw etc.) while testing of the lower part is identical with Wall C and used for cores.

3. RESULTS AFTER TWO YEARS

Two years after casting the results so far are mainly related to documentation of the initial quality of the concrete and field measurements up to one year. As the results are rather comprehensive only part of these will be presented.

3.1. Cement characteristics

During the project only minor variations in quality of each cement type were observed, and the three types of cement are characterized in terms of chemical and physical data given in table 1. Chemical composition of the mineral additions used are also shown in table 1.

Table 1. Data for cements and mineral additions.

		PFC(A)	PC(R)	PC(A/L/S)	FA	Microsilica
SiO_2	%	27	21	24	53	92
Al_2O_3	%	10.3	5.2	2.5	31	2.2
Fe_2O_3	%	3.6	3.0	3.1	7.4	0.3
CaO	%	52	63	66	4.9	0.4
MgO	%	1.1	1.0	0.7	1.4	1.0
SO_3	%	2.5	2.9	1.8	0.7	0.7
Loss on ignition	%	2.3	1.5	0.9	2.4	3.0
Na_2O-eq.	%	0.8	0.6	0.3	1.8	2.6
Insoluble residue	%	15	1.6	0.4	-	-
FA content	%	20	3	-	-	-
C_3S	%	-	-	55	-	-
C_2S	%	-	-	28	-	-
C_3A	%	-	-	1.5	-	-
C_4AF	%	-	-	9.4	-	-
$CaSO_4$	%	-	-	2.4	-	-
Time of set initial	h:min	2:10	1:30	2:30	-	-
Time of set final	h:min	2:45	2:00	3:00	-	-
Fineness Blaine	m^2/kg	450	430	310	-	-
Residue on sieve 200 μm	%	0.3	0.2	0.1	-	-
Residue on sieve 90 μm	%	1.5	0.9	0.4	-	-
Compressive strength (DS 427) 1 day	MPa	17	19	11	-	-
7 days	MPa	40	46	38	-	-
14 days	MPa	44	50	44	-	-
28 days	MPa	51	54	50	-	-
56 days	MPa	58	60	59	-	-

3.2. Concrete quality and compressive strength

Table 2 summerizes the results from the quality control of the fresh and hardened concrete as measured in the laboratory. As it can be seen, the 28 days strength ranges from 25 MPa to 60 MPa.

The gain of strength in the time interval between 28 days and 1 year is strongly dependent on the cement type and the use of mineral additions.

Figure 4 shows the strength development on a relative scale for the three cement types. As expected PFC(A) exhibits the highest strength gain, 50 percent, while PC(A/L/S) and PC(R) gain 40 percent and 25 percent, respectively. When microsilica is used these percentages are reduced for the PFC(A) and PC(A/L/S), however, they are fairly unchanged for PC(R).

Table 2. Mix composition and strength measured on laboratory specimens. A: PFC(A), R: PC(R), A/L/S: PC(A/L/S), FA: Flyash, S: Microsilica

Mix No.	Type of cement	Cement	Mineral addit.	Water	Air cont.	Slump	Compressive strength (MPa)			Modulus of rupture (MPa)	
		kg/m^3	kg/m^3	kg/m^3	%	mm	7 days	28 days	1 year	28 days	1 year
1B	A	261	-	141	5.8	60	20	27	44	4.1	5.1
2	R+S	200	20	125	5.6	50	21	36	49	4.8	5.5
3	A/L/S	264	-	138	5.7	45	17	25	35	4.3	4.8
4	R	261	-	140	5.8	50	21	28	36	4.0	4.2
5	A+S	199	21	143	5.5	40	16	27	40	4.4	6.0
6	A	333	-	146	5.6	55	24	32	50	4.6	5.8
7	R+S	256	26	141	5.5	60	30	51	64	5.7	6.8
8	A/L/S	340	-	142	6.0	60	31	40	57	5.1	5.9
9	A+S	260	26	140	5.9	55	21	34	47	5.0	5.4
10	A/L/S+FA	327	65	141	5.8	65	33	44	70	5.4	6.9
11	R	334	-	144	5.8	50	30	38	47	4.6	5.2
12	A	397	-	165	5.4	50	28	38	54	4.8	6.3
13B	R+S	300	30	140	5.8	70	30	49	57	5.8	6.2
14	A/L/S	390	-	142	5.4	60	38	48	67	6.1	6.7
15	A/L/S+S	300	30	136	5.6	50	39	62	69	6.4	7.3
16	A+S	300	32	143	5.5	40	29	46	57	5.7	6.5

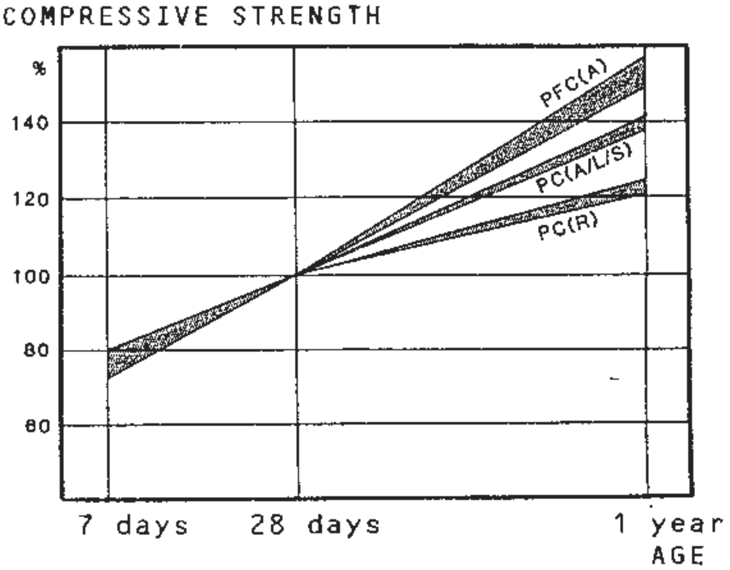

Figure 4. Compressive strength development relative to the strength at 28 days

Apparently, microsilica has a special accelerating effect upon PC(A/L/S) with respect to the 28 days strength which is in agreement with earlier findings /3/.

Mix No. 10 with FA shows the highest strength gain from 28 days to 1 year, viz 60 percent.

The results support the assumption that flyash mainly contributes to the concrete strength at ages beyond 28 days. This applies to separate addition of flyash as well as the use of flyash in portland flyash cements. This is believed to result in concrete of higher quality when flyash is used, due to the fact that requirements normally are related to the strength at 28 days.

However, it is often claimed that the long term strength evolution will be far less under practical conditions due to differences in curing conditions between laboratory experiments and practical conditions. Table 3 shows the results from the testing of cores taken from the full-scale specimens. In the figures 5 and 6 these are plotted against laboratory results.

Table 3. Compressive strength of cylinders and cores after 28 days and 1 year. A: PFC(A), R: PC(R), A/L/S: PC(A/L/S), FA: Flyash, S: Microsilica

		Laboratory results on cast cylinders		Results on cores taken from the structure		Results from seawater
Mix No.	Type of Cement	28 days MPa	1 year MPa	28 days MPa	1 year MPa	1 year MPa
1B	A	27	44	29	41	39
2	R+S	36	49	33	45	39
3	A/L/S	25	35	24	36	43
4	R	28	36	22	32	28
5	A+S	27	40	28	37	42
6	A	32	50	30	45	44
7	R+S	51	64	49	66	50
8	A/L/S	40	57	36	47	48
9	A+S	34	47	38	45	44
10	A/L/S+FA	44	70	42	57	65
11	R	38	47	37	49	43
12	A	38	54	33	47	53
13B	R+S	49	57	47	57	54
14	A/L/S	48	67	51	64	58
15	A/L/S+S	62	69	64	69	65
16	A+S	46	57	49	53	50

At 28 days no difference between strength of cores taken from the structure and the strength of cast cylinders apparently can be seen. This is to some extent surprising since most investigations mentioned in the literature show substantially lower strength of cores than of cylinders.

After 1 year a tendency to lower strength of the cores (0-10 percent) can be seen. The reduction, however, is far less than expected.
The reduction is a little more pronounced for concrete mixes containing flyash than for other mixes. On the other hand, there is hardly any reduction when microsilica is used.

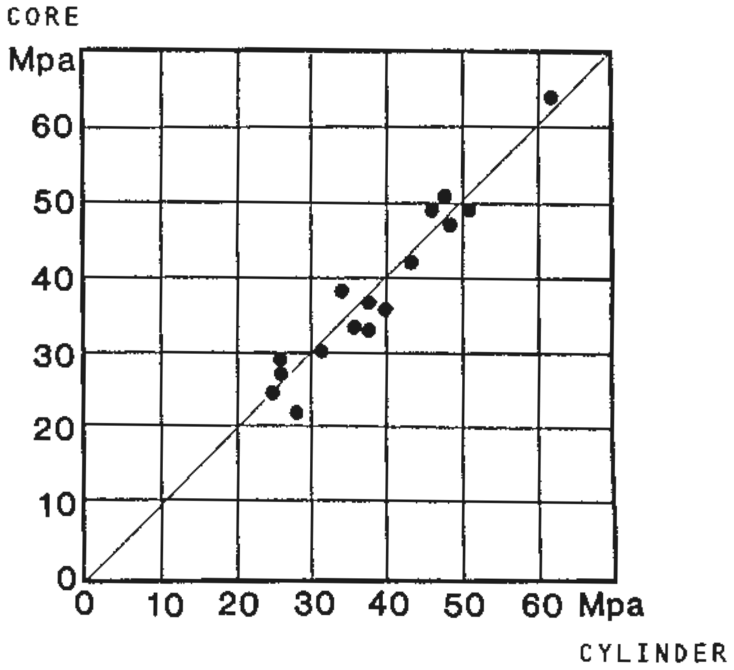

Figure 5. Compressive strength of cores and cast cylinders at 28 days

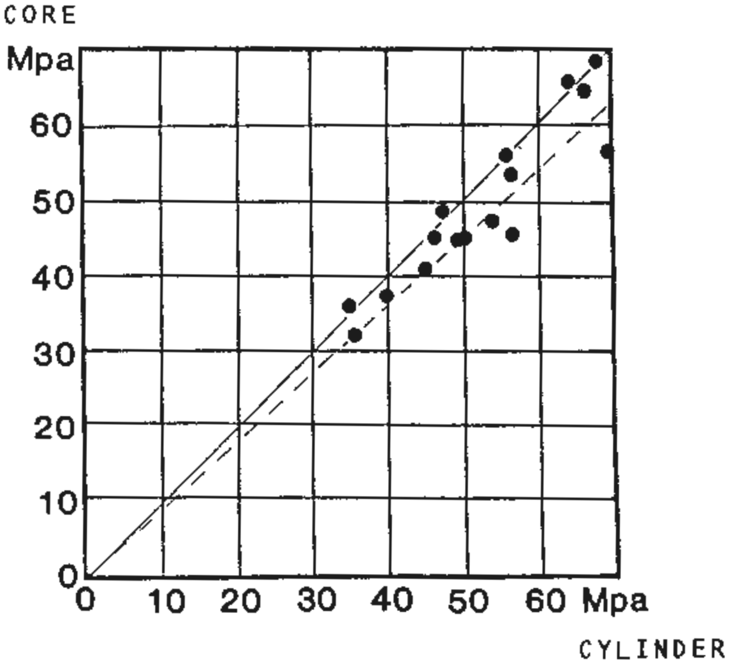

Figure 6. Compressive strength of cores and cast cylinders at 1 year

Figure 7 shows the relationship between the one year core strength and the 28 days cylinder strength. Apparently the relation is fairly unaffected by the cement type and the mineral additions used, although some variation is seen particularly in the low strength region. The strength gain when flyash is used as found in the laboratory seems to be less pronounced in the structure.

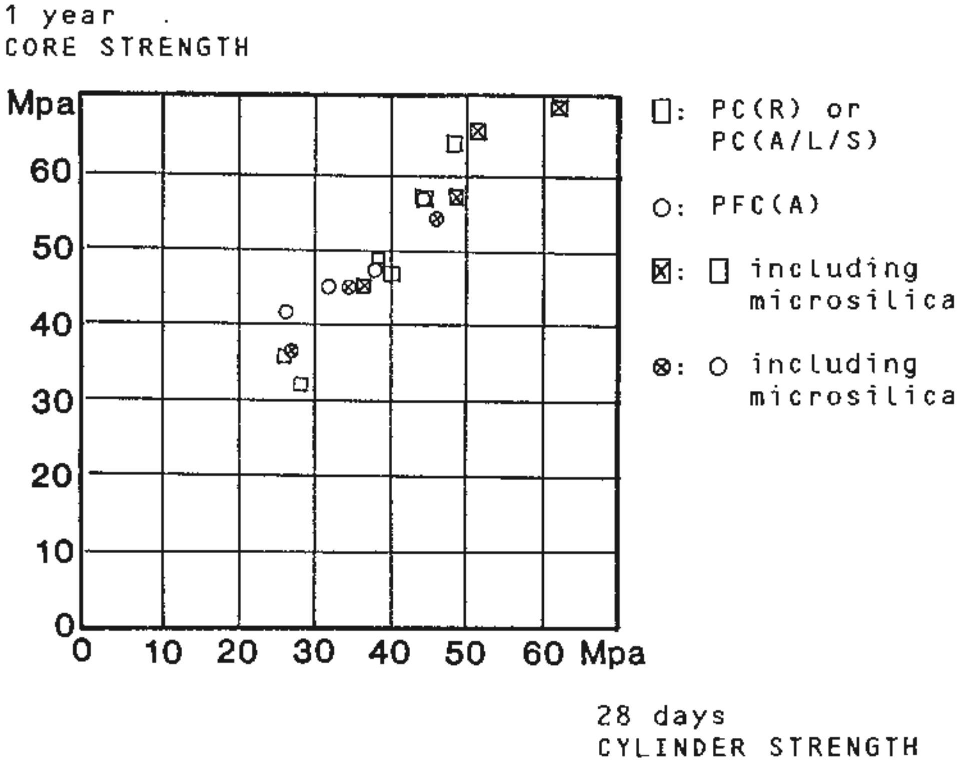

Figure 7. Comparison between cylinder strength at 28 days and core strength after 1 year

Figure 8 shows the relation between the compressive strength and the modulus of rupture. It is seen that this relationship is undependent of the composition of the binder. It is more likely related to the aggregates used and the shape and size of the actual specimens.

After 28 days a core was taken from Slab B for microscopic examination of the pore and micro structure. Table 4 shows the characteristics of the air void system as well as the air content measured on the fresh concrete.

It can be seen from the table, that some of the mixes have lost a great deal of the original air content during the process of casting.
Apparently, no systematic pattern in the way the various combinations of cement and mineral additions loose the air can be seen. However, flyash or microsilica does not seem to improve the conditions in that respect.

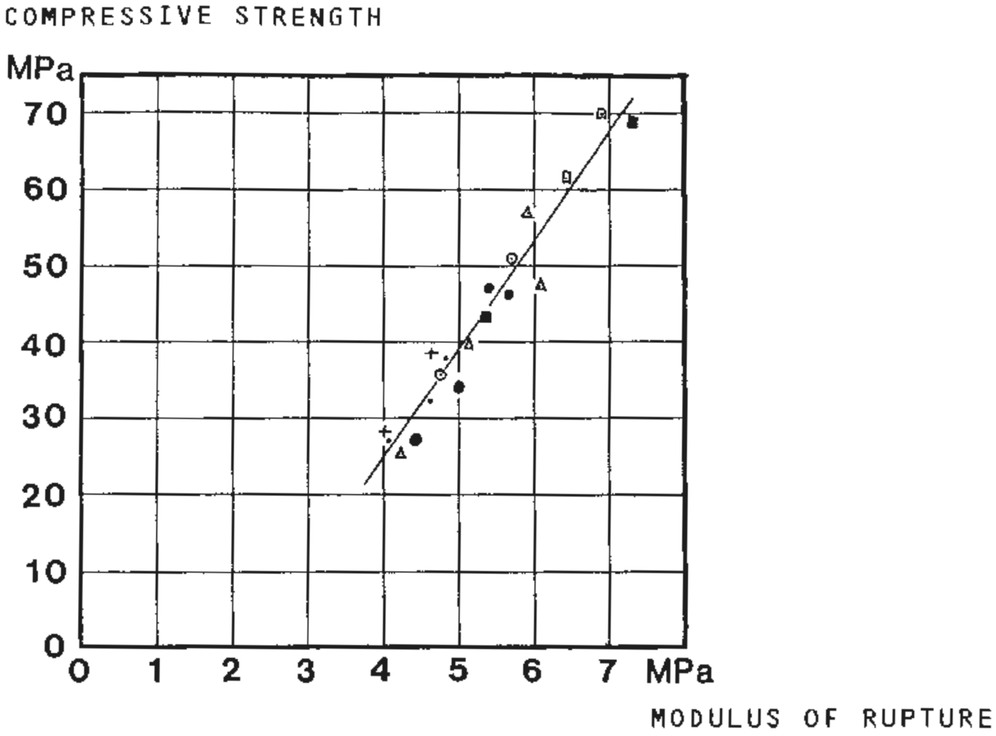

Figure 8. Relationship between modulus of rupture and compressive strength, as found laboratory cured specimens, 28 days.

Table 5 shows the air loss at various stages from the mixing of the concrete until it was placed in the structure. Mixes with PC(A/L/S) tend to loose less air than mixes with other cement types.

Generally speaking the air void system is acceptable, although the air content in the mixes 6 and 7 is too low. The specific surface of the air bubbles is above 30 mm^{-1} for most of the mixes and a spacing factor of less than 0.20 mm is also achieved for the major part of the mixes.
These limits are often used in Denmark as acceptance criteria for air-entrained concrete.

The results for macro and micro structure are rather comprehensive and only a brief description will be given here.

At the macroscopic level all concretes were characterized as being fairly homogeneious. No macro-cracks or other defects were visible and the aggregates showed no sign of being porous or reactive in any way.

At the microscopic level, however, the situation is more complex. All mixes showed micro-cracks in the paste as well as cracks in the bond between the paste and some of the aggregates. There is, however, no significant pattern in the way these cracks occur for the various mixes, although the number of these cracks in one thin section varied between 10 and 100.

Table 4. Results of air content of fresh concrete and of air-void characteristics measured on cores taken from slab B. A: PFC(A), R: PC(R), A/L/S: PC(A/L/S), FA: Flyash, S: Microsilica

Mix No.	Type of cement	Air content of fresh concrete %	Air content of hardened concrete %	Specific surface mm^{-1}	Spacing factor mm
1B	A	5.8	2.8	50	0.13
2	R+S	5.6	2.8	41	0.16
3	A/L/S	5.7	4.7	28	0.18
4	R	5.8	4.5	52	0.10
5	A+S	5.5	2.6	32	0.20
6	A	5.6	1.9	59	0.13
7	R+S	5.5	1.6	35	0.23
8	A/L/S	6.0	5.6	29	0.16
9	A+S	5.9	4.7	37	0.14
10	A/L/S+FA	5.8	4.0	36	0.15
11	R	5.8	4.3	30	0.17
12	A	5.4	4.7	32	0.15
13B	R+S	5.8	5.6	29	0.16
14	A/L/S	5.4	3.5	36	0.16
15	A/L/S+S	5.6	4.7	30	0.17
16	A+S	5.9	2.8	42	0.15

Table 5. Loss of air during transport (1), on site (2) and during casting (3).

Mix No.	Type of cement	Loss of air % 1	2	3	1+2+3 %	Rest %
1B	PFC(A)	1.7	0.5	2.5	4.7	2.8
2	PC(R)+S	1.2	0.7	2.4	4.3	2.8
3	PC(A/L/S)	0.6	0.7	0.2	1.5	4.7
4	PC(R)	2.0	0.8	1.3	4.1	4.5
5	PFC(A)+S	2.1	0.5	2.3	4.9	2.6
6	PFC(A)	2.5	0.0	3.1	5.6	1.9
7	PC(R)+S	2.4	1.2	2.7	6.3	1.6
8	PC(A/L/S)	0.7	0.2	0.2	1.1	5.6
9	PFC(A)+S	2.3	0.2	1.0	3.5	4.7
10	PC(A/L/S)+F	0.8	0.8	1.0	2.6	4.0
11	PC(R)	2.2	0.6	0.9	3.7	4.3
12	PFC(A)	0.0	2.2	0.0	2.2	4.7
13B	PC(R)+S	0.5	0.3	0.0	0.8	5.6
14	PC(A/L/S)	1.1	0.2	1.7	3.0	3.5
15	PC(A/L/S)+S	0.6	0.8	0.1	1.5	4.7
16	PFC(A)+S	2.0	0.4	2.6	5.0	2.8

Thin sections from the surface of the concrete showed a tendency to an increasing number of micro-cracks at right angels to the surface when PC(R) was used in combination with microsilica.

As regards the microscopic observations it is obvious that the small examined area hardly can be representative for the entire structure, and the methods and results have to be used with this in mind.
It was considered important, however, to include microscopic examinations as well as ordinary laboratory activities in achieving a proper documentation of the initial quality of the concrete mixes used in this project.

3.3 Freeze/thaw

The freeze/thaw experiments in the laboratory were carried out according to the Danish standard which is similar to ISO 4846.

The small slabs were subjected to 25 freeze/thaw cycles with the upper surface covered by a 3 percent NaCl-solution and the amount of scaled material was weighed after 10, 20 and 25 cycles. The results are shown in the figures 9 to 11.

The results show that for the lowest cement content the scaling is quite severe for all mixes except for mix No. 4. The acceptance criteria is a scaling of less than 0.5 kg/m^2 after 25 cycles. Mix No. 5 with PFC(A) and microsilica has the highest degree of scaling, but also mix No. 3 and mix No. 2 are severely damaged after 25 cycles. The aggregates in these three mixes were completely exposed.

With a cement content of 340 kg/m^3 only one or two mixes failed to be accepted.
Again the combination of PFC(A) and microsilica (mix No. 9) gave the highest scaling. Mix No. 10 with PC(A/L/S) and flyash was also rejected with only a few grammes beyond the limit for acceptance.

With the highest cement content (390 kg/m^3) all the mixes were accepted with a scaling of approximately 0.1 kg/m^2.

This pattern is in accordance with the expectations with respect to concrete quality (strength) and freeze/thaw resistance. However, problems due to the arrangement of the reinforcement in the first 5 mixes caused a number of severe cracks to occur in the slabs A and E. Therefore these mixes were repeated, and new specimens were cast.

The results of these mixes with respect to freeze/thaw experiments are shown in figure 12. In comparison with the former results for these mixes shown in figure 9 the amount of scaling is greatly reduced. In fact all the mixes fulfil the acceptance criteria except mix No. 5 which is just above the limit.

These results caused some confusion with respect to the reliability of the test method which had become a Danish standard shortly before these tests were carried out.

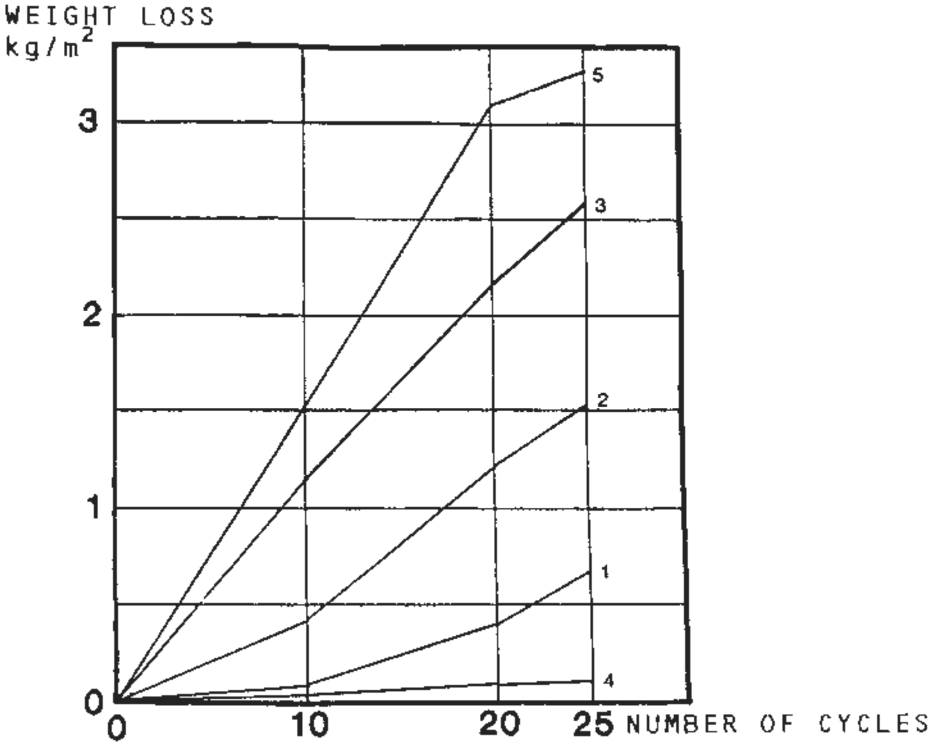

Figure 9. Results from freeze/thaw experiments on concrete with 260 kg of cement per m^3, mix no. 1-5.

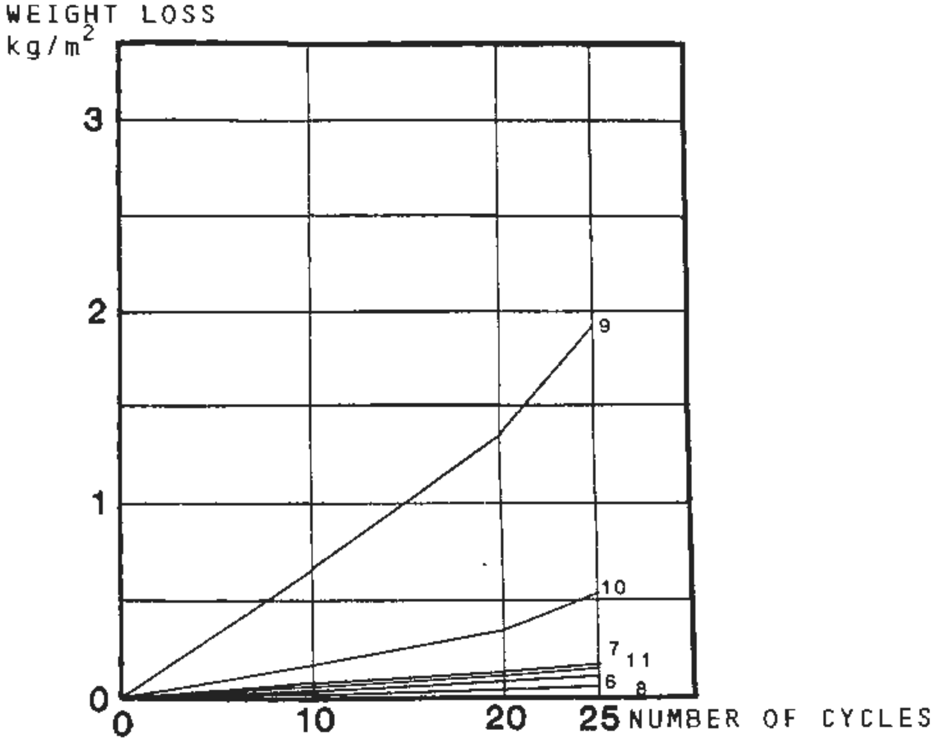

Figure 10. Results from freeze/thaw experiments on concrete with 340 kg of cement per m^3, mix No. 6-11

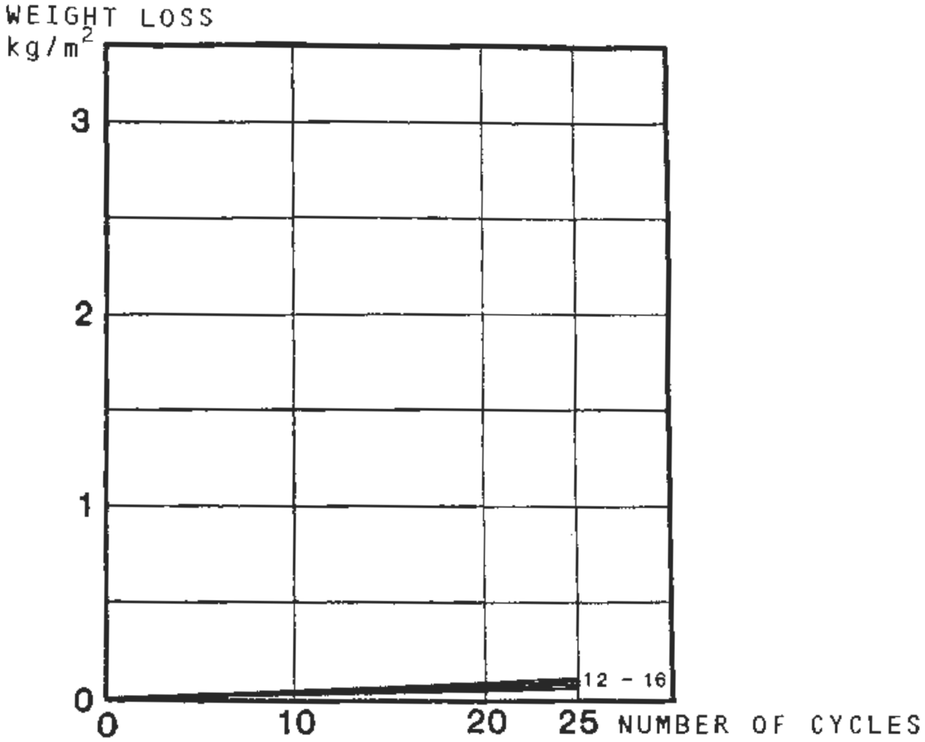

Figure 11. Results from freeze/thaw experiments on concrete with 390 kg of cement per m^3, mix No. 12-16.

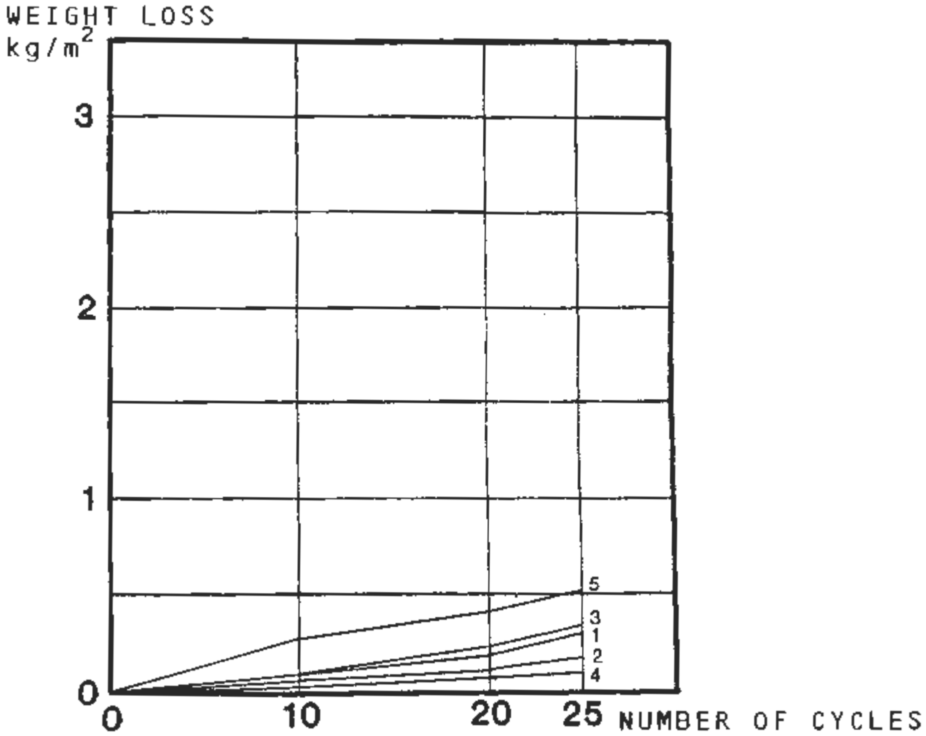

Figure 12. Results from freeze/thaw experiments on concrete with 260 kg of cement per m^3, repetition of mix No. 1-5.

Apparently, there were not any obvious difference between the original series of mixes and the repeated ones that could have caused these diverging results. However it was later discovered that the filling, i.e. the amount of material to be tested of the freezer had differed significantly between the original and the repeated series.

Such variations in the filling of the freezer have later been shown to cause variations in the rate of cooling.

Bager and Sellevold /4/ have shown by low temperature calorimetry that the rate of cooling has a great effect on the amount of ice that is formed and the rate at which the ice front propagates. It is therefore assumed that the diverging results are due to differences in the rate of cooling of the two series.

It is noticed by comparing the figures 9 and 12, that the results lead to almost the same ranking of the mixes in the two series. Again mix No. 5 gives the highest amount of scaled material and mix no. 4 the lowest.

The method has been critisized by several concrete technologists in Scandinavia, among others Petersson /5/ who has shown that even small variations in the test procedure may alter the results dramatically. The rate of cooling is just one of these.

The discussion of whether the freeze/thaw-durability of concrete should be assessed directly by freeze/thaw tests or indirectly by analyses of air void characteristics is outside the scope of this paper. However, a proper freeze-thaw test should be designed and prescribed in such a way that a satisfactory repeatability and reproducibility can be achieved. Obviously, this is not the case with the present test procedure. A working group in Scandinavia is working on an improvement of the procedure.

Inspection of the full-scale specimens after one and two years of exposure showed no signs of any deterioration of the concrete surfaces due to action of frost.

3.4. Corrosion

The potentials of the reinforcement were measured after one year for the Slabs A and E and for the wall F exposed to sea water. A saturated calomel electrode (SCE) was used as reference.

As regards the A slabs all the results of the measurements are within the range of -10mV to +60 mV and the reinforcement in these slabs can therefore be considered as being in the passive state.

The Slab E is similar to A except for the use of de-icing chemicals and this tends to lower the potentials slightly. However, the results showed no potentials less than -100 mV. The major part of the potentials were within the range of 0 to -40 mV, so the reinforcement in these slabs seems to be in good condition as well.

On the wall panels of type F at Hirtshals the results were somewhat different. As shown in table 6 the potentials of the reinforcement with a concrete cover of 10 mm show quite low values, especially for the mixes 1 to 6 which represent the lowest concrete quality. Each figure in the table represents the average of 10 measurements. For each combination of concrete type and cover two figures are shown corresponding to the two surfaces of the panel.

Table 6. Potentials of reinforcement in mV as measured with reference to the SCE

Mix	Type of	Cover thickness					
No.	cement	10 mm		20 mm		30 mm	
1B	PFC(A)	-505	-100	-167	-98	-76	-72
2	PC(R)+S	-257	-243	-73	-82	-28	-48
3	PC(A/L/S)	-344	-196	-139	-93	-64	-32
4	PC(R)	-340	-250	-90	-47	-120	-45
5	PFC(A)+S	-558	-468	-98	-49	-129	-18
6	PFC(A)	-192	-244	-102	-110	-96	-54
7	PC(R)+S	-482	-87	-114	-97	-117	-92
8	PC(A/L/S)	-296	-21	-21	-22	-94	-23
9	PFC(A)+S	-412	-233	-63	-64	-131	-94
10	PC(A/L/S)+FA	-239	-54	-73	-23	-51	-115
11	PC(R)	-77	-46	-125	-111	-84	-123
12	PFC(A)	-78	-35	-103	-35	-93	-127
13B	PC(R)+S	-90	-108	-76	-55	-89	-110
14	PC(A/L/S)	-55	-115	-43	-42	-49	-81
15	PC(A/L/S)+S	-78	-334	-108	-197	-63	-90
16	PFC(A)+S	-111	-365	-104	-190	-70	-144

Steel can be considered passivated at pH = 13.5 when the potential is in the range of -594 mV to +175 mV (SCE) /6/. The potentials measured are all within this range.

However, potentials in the range of -400 to -600 mV might indicate initiation of corrosion.
Lack of oxygen may also result in low potentials as the passive film of oxides cannot be preserved. The fact that the lowest potentials occur in the cases of low cement content and thin cover indicates that lack of oxygen is not the reason for the low potentials /7/.

The results show some scatter. There is, however, a tendency of lower potentials with PFC(A) in combination with microsilica. Furthermore, as expected there is an effect of the thickness of the concrete cover and the concrete quality.

3.5 Carbonation

The depth of carbonation was measured on cores after 28 days and again after 1 year.

The results after 28 days were obtained by microscopic examinations, while the 1 year results were achieved by use of phenolphthaleine. The results are shown in table 7.

Again PFC(A) and microsilica tends to give the highest figures although categorial conclusions should not be drawn on the basis of these early results.

Table 7. Depth of carbonation after 28 days and after 1 year.

Mix No.	Type of cement	Depth of carbonation (mm)	
		28 days	1 year
1B	PFC(A)	1.4-1.8	2-5
2	PC(R)+S	0.9-2.0	1-1 1/2
3	PC(A/L/S)	0.1-1.8	1-3
4	PC(R)	0.5-1.5	1/2-2
5	PFC(A)+S	0.3-1.4	2-6
6	PFC(A)	0.4-0.7	1-4
7	PC(R)+S	0-0.8	1/2-3
8	PC(A/L/S)	0-0.2	1/2-1 1/2
9	PFC(A)+S	0.8-1.6	2-4
10	PC(A/L/S)+FA	0.6-1.0	1/2-2
11	PC(R)	0-0.6	1-3 1/2
12	PFC(A)	0.6-1.6	2-3
13B	PC(R)+S	0.2-0.9	1/2-1 1/2
14	PC(A/L/S)	0-1.2	0-1
15	PC(A/L/S)+S	0.4-1.3	1-2 1/2
16	PFC(A)+S	1.3-1.8	2-3 1/2

4. FINAL REMARKS

The present project makes it possible to monitor the general conditions as well as specific properties of 16 different concrete mixes. These mixes are produced in full scale and exposed to realistic conditions and at the same time they are very well documented due to extensive laboratory activities including testing of most properties related to durability of concrete structures.

It is the intention that the project may be expanded to include other new materials that might gain a footing in future concrete technology.

After two (one) years of exposure all 16 types of concrete seem to be in good condition, although some of the results indicate that combinations of PFC(A) and microsilica should be used with caution.

Five year measurements will be carried out in 1988, and the results from these measurements will be published as soon as possible.

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