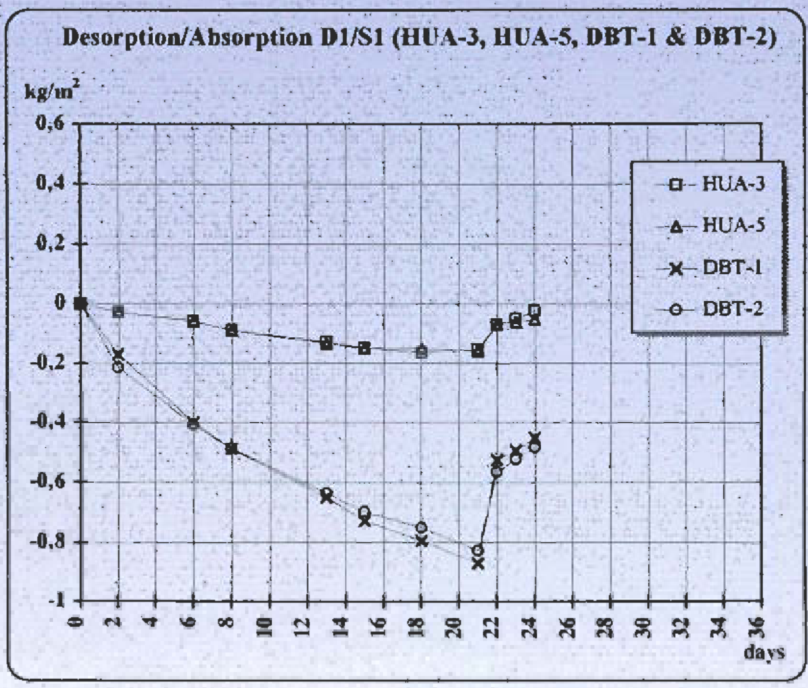
HETEK

Method for test of the Frost Resistance of High Performance Concrete

Supplementary research



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Abstract

The effect of conditioning and freeze/thaw exposure on the moisture profile of high performance concrete has been investigated. Parameters relevant to freeze/thaw testing have been investigated and supplementary methods have been described: Measurement of absorption during conditioning and testing, Conditioning of test specimens, Temperature control, and Measurement of internal damage in concrete specimens exposed to Borås testing. An alternative model of freeze/thaw damage mechanism is described. Two freeze/thaw test methods relevant for scaling and for internal damage of HPC are proposed.

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1 Summary Report

1.1 Introduction

The Danish Road Directorate has made a research and development contract with seven consortia and individual companies. The subject of the contract is the establishment of guidelines for the execution of concrete structures with 100 years' life, focusing on the technology of the contractor. Experience from large construction works proves that the execution phase is important to the achievement of the requested durability of concrete structures exposed to aggressive environment.

Task 2 of the development contract: 'Test Method for determining the Frost Resistance of High Performance Concrete' is awarded to a consortium consisting of Dansk Beton Teknik A/S, Dansk Teknologisk Institut and Dansk Betoninstitut a/s. High performance concrete is defined as concrete with an expected life of 100 years in aggressive environment [VD, 1995]. The consortium has implemented the following definitions of High Performance Concrete:

Concrete that can last for 100 years in an aggressive environment, having an equivalent W/C ratio of 0.35-0.45, and complying with the present Danish specifications regarding materials, mix composition etc.

The first activity of Task 2 was a State-of-the Art Report, including an evaluation of a series of existing freeze/thaw test methods. This formed the basis for the Consortium recommendations for supplementary research [Laugesen, 1996] needed to:

i) increase the knowledge of moisture movements in HPC
ii) choose test methods for the further part of Task 2
ii) strengthen selected parts of the test method(s) chosen

The present report presents the results of the carried out supplementary research.

Background

The State-of-the-Art-Report concluded that two test methods are needed for describing freeze/thaw deterioration, one causing scaling and one causing internal cracking. This is in accordance with the recommendations of CEN TC 51/WD 12/TG 4.

The test methods included in the State-of-the-Art Report can be separated in test methods giving mainly internal cracking:

* ASTM C666
* Critical Dilation
* Dobrolubov/Romer

and test methods giving mainly scaling:

* Borås
* CDF
* Cube Test

Based on the State-of-the-Art Report it has not been possible to isolate one freeze/thaw test method within each group as being evidently superior, since they all have draw backs. The State-of-the-Art Report does, however, identify improvements of each test method, by which they could possibly be upgraded.

With the supplementary research presented below, and considering the experience available, the most likely candidates for recommended test methods for the further work in Task 2, are believed to be the Borås test method as a method for scaling, and the ASTM C666 as a method for internal cracking.

The preliminary results obtained during the supplementary research, will be further monitored and evaluated during the rest of the project.

Objectives

The supplementary research is carried out with the following objectives:

* Increase the knowledge of moisture movements in HPC
  - Moisture Movements During Conditioning and Testing
  - On the Influence of Salt

* Improve selected parts of the freeze/thaw test methods:
  - Absorption During Conditioning and Testing, Method
  - Conditioning of test specimens
  - Temperature Control during testing
  - Measurement of Internal Damage during Borås testing

* Description of a new supplementary model for freeze/thaw deterioration:
  - Temperature induced stresses during heating of the frozen test specimen

* These activities form the basis for the selection of the freeze/thaw test methods to be applied for the remaining part of Task 2:
  - Proposed test methods

The results of the individual activities covered by the supplementary research are summarized and concluded below. The detailed results are presented in chapters 2-5.

1.2 Moisture Movements in HPC

The effect of conditioning (selected drying and re-saturation) and freeze/thaw exposure on the moisture profile have been investigated.

The work is based on the testing of moisture parameters in concrete specimens at various levels from exposed surfaces after various combinations of drying, re-saturation and freeze/thaw testing, mainly according to the Borås method. The samples applied were 117 concrete prisms (30x30x70 mm^3) and 20 slices (50 mm thick), all cut from cast concrete cylinders (Ø:150 mm).

Most tests have been performed on a three powder concrete with an equivalent w/c ratio of 0.39 (HUA-3). Selected testing has also been performed on three other concretes, ranging from 0.37 (HUA-5) to 0.45 (DBT-1 and DBT-2) in equivalent w/c ratio. The latter two are not air entrained.

When initiating the supplementary research, it was expected that the high density and low permeability of HPC would limit the moisture movements taking place during the limited time available for conditioning samples prior to freeze/thaw testing.

This was confirmed by the supplementary research. It was found that generally re-saturation is slow, causing the original water content (from before drying) not to be reached.

The Borås conditioning (21 days drying at 65% RH and 20° C followed by three days capillary suction) had no significant influence on the moisture content of dense concretes (HUA-5: w/c =0.35, HUA-3: w/c= 0.39). Prolonged re-saturation (14 days capillary suction) caused a slightly increased moisture content (HUA-3). Vacuum saturation for two days resulted in only half of the absorption obtained after 14 days capillary suction. That is vacuum saturation does not provide increased saturation of concretes as dense as HUA-3.

The more porous concretes (DBT: w/c=0.45, no air) were, as expected, more affected by the Borås conditioning, but did only absorb less than half of the moisture lost during drying.

Drying of the HUA-3 concrete at 50°C for 14 days caused a significant moisture loss. Re-saturation for 3 days only caused an absorption of 1/4 of the moisture lost, prolonged re-saturation (14 days) a further absorption at 1/8 of the moisture lost. The dried specimens showed a large variation in degree of saturation as a function of depth from the exposed surface (50% in the outer layer (10 mm), 70-75% at further depth). Re-saturation caused an increase in degree of saturation (outer layer 75%-80%), freeze-thaw exposure a further increase to (85-95%).

Drying and re-saturation according to Borås of the DBT-samples are observed to affect the outer layer only, where as the degree of saturation of the entire prisms was increased during testing due to cracking.

Freeze-thaw exposure did not increase the rate of absorption of the HUA-samples. Higher rates of absorption were observed for the specimens most severely dried and prolonged testing may result in a higher degree of water saturation of the dried samples, compared to the initial moisture state before conditioning.

The available data does not confirm information on salt causing increased moisture ingress nor any effect of the salt concentration in the exposure liquid.

All HUA-specimens showed good freeze-thaw resistance: There were no signs of internal cracking in the frost tested HUA-samples. Furthermore, surface cracking was only observed around one sand grain in one sample.

No effect of the conditioning and freeze-thaw exposure was observed on the frost resistance of the tested concrete (HUA-3, Borås conditioning versus drying 14 days at 50°C followed by 14 days re-saturation). This may be due to the above mentioned good performance of this concrete.

No firm recommendations can be given concerning the method of conditioning to be used for high performance concrete to be frost tested.

The method of conditioning should be reproducible and preferably also rapid and non-elaborous. Furthermore, the conditioning should give a similar frost resistance ranking as observed in-situ.

Considering drying at increased temperature as part of the conditioning of concrete to be frost tested, it should be remembered that the degree of hydration of especially concrete with flyash or slag will be increased.

As described above for all 4 concrete types tested, the water loss caused by the initial drying is not fully compensated by the subsequent re-saturation, not even with re-saturation prolonged to 14 days. This may appear to contradict the results obtained by drying of virgin cement paste specimens, observed to increase the amount of freezable water and thus reduce the frost resistance [Bager, 1984]. However, the present work can not exclude, that the Borås conditioning reduces the water content, yet causes an increase in the freezable water. This question may be further enlightened in the further part of Task 2, e.g. by applying both normal Borås conditioning, and the conditioning with a more severe drying effect, on selected samples to be tested.

The Influence of Salt

The analyses of the moisture content of the samples and the chloride penetration profiles do not confirm that salt on the concrete surface causes increased moisture ingress. We do realize however, that the research performed under HETEK on this aspect is very limited.

1.3 Improvement of Test Methods.

Absorption during Conditioning and Testing

A simple test method is described for measuring the amount of water absorbed in test specimens during conditioning (drying and/or re-saturation) and during freeze/thaw testing. The method was applied for the testing described in chapter 2: Moisture Movements in HPC.

The method is based on weighing the test specimens before, during and after conditioning and testing, and an eventual drying at 105°C.

Conditioning of Test Specimens

Considerations concerned pre-conditioning of test specimens prior to Borås freeze-thaw testing are presented, with the following conclusions:

* Pretesting: Directly testing with high water content at the time of the required maturity of the concrete. In the view of the results from the testing moisture movements, ref. chapter 2, this could imply testing without drying as part of the conditioning.

* Specimens from structures: The specimens are - depending on actual exposure - conditioned to a specified degree of saturation.

An example of the influence of pre-conditioning on scaling is presented. Two types of pre-conditioning from 7 maturity days to time of testing were applied: water curing and air curing at laboratory atmosphere. The results showed that for the applied HPC concrete compositions (ref. chapter 3.2), no influence of the pre-conditioning was found.

Temperature Control during Testing

All methods for determining the frost-resistance of concrete, concrete products and aggregates have the common feature that the temperature of the test specimens shall run through a pre-defined course a number of times. This temperature history must be documented, because it influences the result of the test. Inadequate temperature control is probably one of the main causes of the considerable scatter among the results of several Round Robin tests [Laugesen et al, 1996].

Various methods for controlling the temperature during freeze/thaw testing are described. The advantages and disadvantages of these methods are discussed.

Furthermore, a study regarding the influence of the temperature cycles is presented. The effect on Borås testing (Scaling test) of three different temperature cycles were investigated:

1: In the frost phase the temperature is outside the limiting curve, so that the temperature falls very rapidly to a minimum of -25°C and rises even more rapidly afterwards. The maximum gradient during cooling is -3.5°C/h, during warming 7.7°C/h.

2: The temperature is at the middle of the permitted interval. The maximum gradient during cooling is -1.9°C/h, during warming 5.2°C/h.

3: The temperature is within the permitted interval, but the gradients are as high as possible. The maximum gradient during cooling is -2.0°C/h, during warming 6.0°C/h.

The results after 56 cycles show:

* That temperature cycle No. 1 results in an amount of scaled material 40% to 120% higher than cycle No. 2, with a mean increase of 90%.

* That there is no significant difference in the amount of scaled material resulting from the cycles within the permitted temperature interval (Nos. 2 and 3).

* Temperature level and temperature gradient influence the results of the frost-resistance tests investigated.

These results confirm those given in [Petersson and Lundgren, 1996]. In this, table 2 (concrete A,C and S) shows that a change in the minimum temperature results in a 30-100% change in the amount of scaling, and that increasing the gradient from 2°C/h to 4°C/h increases the amount of scaling by 18-56%.

The result of this supplementary research showed that it is important to keep the temperature cycle within the limiting curves if uniformity of effects and comparability of results are to be achieved. The application of correct temperature curves in freeze/thaw testing could be assessed by requiring, that the time/temperature curves applied for the actual testing are enclosed in the reports.

Further studies may be needed to identify if the existing requirements to the temperature curves should be further strengthened.

Measurement of Internal Damage during Freeze/Thaw Testing

Concrete structures show two different types of freeze/thaw - damages:

* Surface scaling
* Internal cracking/delamination

The existing freeze/thaw test methods are generally regarded as representing only one of the two. Thus, the Borås method is applied for surface scaling, whereas ASTM C666 is generally applied for internal cracking.

However, the present knowledge of freeze/thaw testing may reflect how the samples were measured, rather than how they actually responded to the testing.

In order to investigate the possible formation of internal cracks in the concrete samples during Borås testing, 3 different methods were applied together with the normal scaling measurements:

* Length change measurements
* Dynamic E-modulus changes, by ultra sonic measurements
* Microstructural analysis, by thin section petrography.

Two samples were tested from each of 4 different concrete compositions, two with expected high frost resistance, and two with expected low frost resistance.

The results of this supplementary research showed that the internal damage, as seen by length change measurements and by thin section analysis, correlate well to the surface scaling. The frost resistant concrete showed little or no scaling, no length change and no change of microstructure. The low frost resistant concretes showed large scaling, marked length change both at depths of 5 mm and 25 mm from the exposed surface, and also showed marked change of microstructure.

It is, however, not known if the observed clear correlation's of scaling and internal damage would be found for concretes with intermediate frost resistance.

It is thus an open question, if the Borås testing can be applied for measurements of internal damage.

The further work in Task 2, on calibration of the test methods, includes measurement of possible internal damage in Borås tested specimens. This is done by length-change measurements during testing and by petrographic analysis of the samples after testing.

1.4 Temperature Induced Stresses during Heating of Frozen Concrete.

A new composite model for freeze/thaw deterioration is described. The model is based on a series of assumptions, e.g. that the thermal expansion coefficient of ice (in concrete) is 5 times that of concrete [Thaulow, 1996].

Calculations using the composite model has verified that cracking may take place during heating of frozen concrete due to the larger thermal expansion of the ice relative to the cement paste. The model predicts potential damage of frozen concrete when the temperature is increased by 10°C to 20°C.

The implication on test methods of this model is, that a freeze/thaw test method should include a freezing temperature in the range -10°C to -20°C. This is satisfied for both the Borås method and the ASTM C666.

1.5 Proposed Test Methods.

The following methods are proposed used, taking into account the below mentioned proposed revisions:

* Scaling: SS 13 72 44
* Internal cracking: ASTM C 666

The proposed revisions are preliminary and is to be revised based on the outcome of the remaining activities of Task 2.

2 Moisture Movements in HPC

2.1 Introduction

This chapter presents the experiments undertaken to provide improved information on the moisture movements in high performance concrete (HPC).

The following activities, ref. [Laugesen, 1996] are dealt with:

* Activity 2.1 "Moisture Movements During Conditioning and Testing"
* Activity 2.2 "On the Influence of Salt".

Furthermore, the experiments undertaken are basis for

* Activity 3.1 "Absorption during Conditioning and Testing, Method"
* Activity 3.4 "Measurement of Internal Cracking in (Borås) Test Specimens, Method".

2.2 Conclusions and Recommendations

Summary and Conclusion

The effect of conditioning (selected drying and re-saturation) and freeze/thaw exposure on the moisture profile has been investigated.

Most tests have been performed on a three powder concrete with equivalent w/c = 0.39 (HUA-3). Selected testing has also been performed on three other concretes (HUA-5, DBT-1 and DBT-2, ref. below).

The Borås conditioning (21 days drying at 65% RH and 20° C followed by three days capillary suction) had no significant influence on the moisture content of dense concretes (HUA-5: w/c =0.35, HUA-3: w/c= 0.39). Prolonged re-saturation (14 days capillary suction) caused a slightly increased moisture content (HUA-3). Vacuum saturation for two days resulted in only half of the absorption obtained after 14 days capillary suction. That is vacuum saturation does not provide increased saturation of concretes as dense as HUA-3.

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Recommendations

No firm recommendations can be given concerning the method of conditioning to be used for high performance concrete to be frost tested.

The method of conditioning should be reproducible and preferably also rapid and non-elaborous. Furthermore, the conditioning should give a similar frost resistance ranking as observed in-situ.

Drying of virgin cement paste specimens have been observed to increase the amount of freezable water and thus reduce the frost resistance. Drying is likely to occur in-situ. However, the effect on concrete and of in-situ exposure on the amount of freezable water should be investigated further.

Furthermore, the rate of absorption during re-saturation and testing should be measured for a longer period on samples dried according to Borås method (20°C, 65% RH) and 50°C, low RH.

Considering drying at increased temperature as part of the conditioning of concrete to be frost tested, it should be remembered that the degree of hydration of especially concrete with flyash or slag will be increased.

2.3 Background

Conditioning is performed to obtain similar/comparable initial conditions of test specimens or to simulate the effect of similar exposure conditions.

As basis for specifying the conditioning of test specimens before freeze/thaw exposure information on the movement of moisture in HPC is required.

Special considerations regarding HPC, the effect of salt, and ingress of moisture were dealt with in [Laugesen et al, 1996].

Low porosity of HPC not only affects the frost resistance but also the moisture content and the moisture profiles obtained during conditioning and testing.

Denser concretes will typically both have less and a finer porosity. A non-airentrained, dense concrete is likely to have a higher degree of saturation (u_{actual}/u_{total}, u=moisture content) than a more porous concrete at a given relative humidity below 100%, ref. Figure 2.3.1. This is of course only true if the dense concrete can be re-saturated after hydration, where self-desiccation takes place or if the relative humidity (RH) is lower than caused by self-desiccation, and if the effect of hysteresis, causing lower water content upon re-saturation, does not affect significantly.

The degree of saturation is a function of RH and of the degree of filling of larger voids and cracks being emptied at RH < 100%. The effect on degree of saturation of variation in air content is larger than the influence of variation in water/cement-ratio [Fagerlund, 1996].

Drying has been reported to increase the amount of freezable water in cement paste specimens [Bager, 1984]. This may be due to a coarsening of the pore structure and an increased continuity of the pore system, ref. [Fontenay, 1982]. The effect appears to be less pronounced for low w/c, ref. Figure 2.3.2, giving desorption isotherms for virgin (first desorption) and dried (second desorption) mortar specimens varying in w/c. This could be due to these concretes being initially dryer due to self-desiccation.

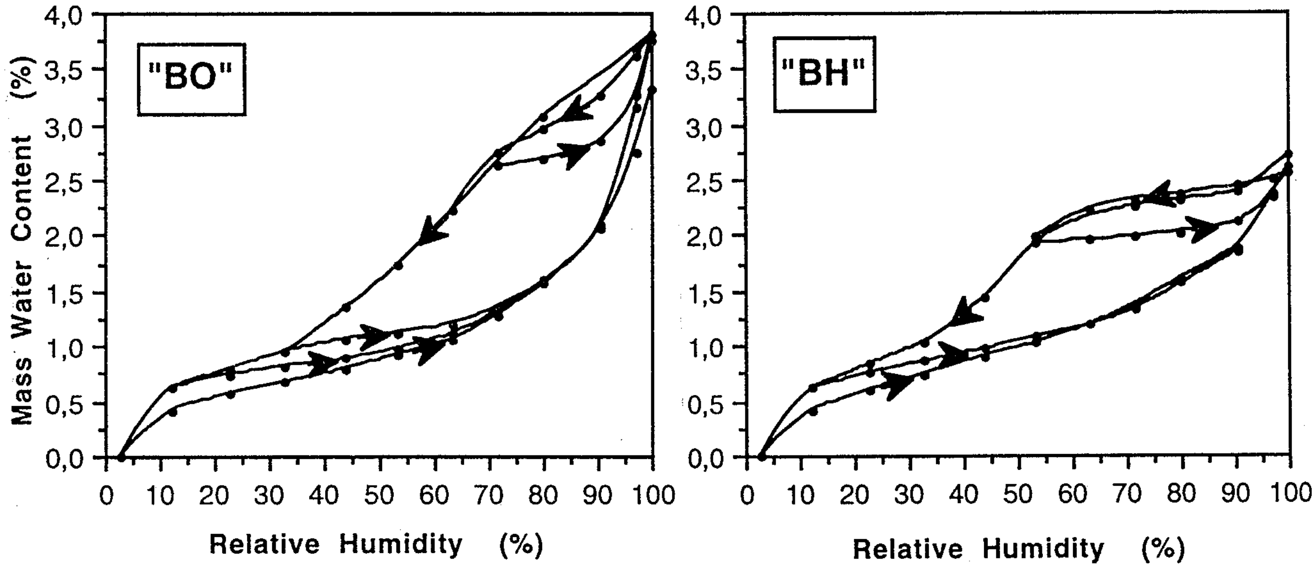


Figure 2.3.1 Selected adsorption and desorption isotherms of concrete (23°C. BO: w/c= 0.48; BH:w/c=0.26, microsilica/cement=0.1; age 1 year. Self-desiccation after 6 months and 2 years: BO: RH=95% and 87%; BH: RH=72% and 64%).The moisture content is given as the amount of evaporable water per weight of binder plus non-evaporable water. The total moisture content is affected by the specimen not having been dried at RH < 3%. [Baroghel-Bouny et. al, 1996].

Moisture Distribution During Conditioning and Testing

Apparently, not much open (published) information exists on the moisture distribution in concrete specimens during drying and none on moisture profiles obtained during re-saturation and testing.

In an ongoing research project "Standard Test Methods for Concrete Permeability Measurements, MAT1-CT93-001" selected methods of conditioning have been investigated [MAT, 1995].

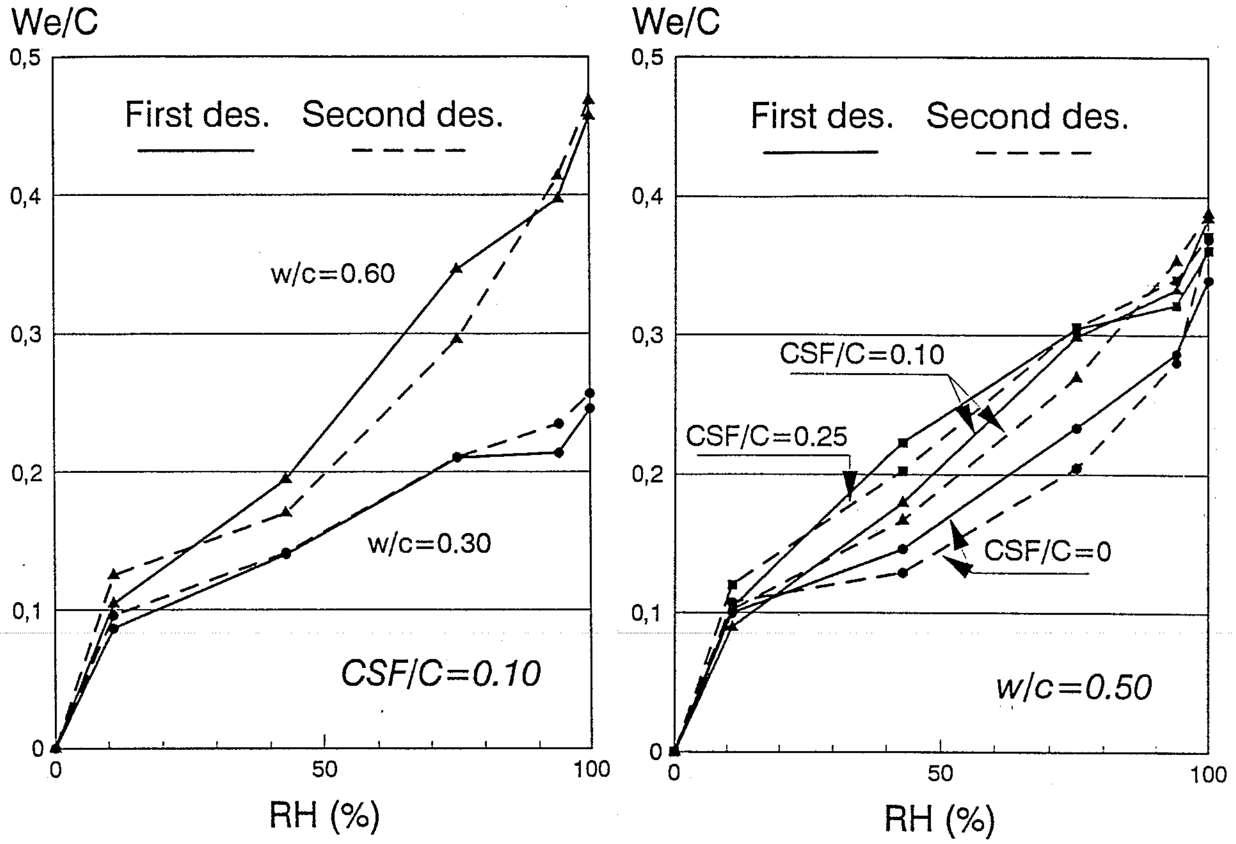


Figure 2.3.2 Desorption isotherms for mortar differing in w/c and microsilica content [Atlassi, 1992].

The applicability of the following drying methods for installment of a pre-set average moisture content has been investigated for discs Ø 150 mm, height 50 mm (OPC, w/c=0.4, 0.7):

1 50°C, 85% RH (the actual concrete having similar moisture content at 20°C, 65% RH)

The method requires control of temperature and humidity, and information on the sorption isotherms is needed.

It was found that an equilibrium moisture concentration corresponding to the moisture content at 20°C, 65% RH could not be obtained within a period of 28 days. However, a difference in RH less than 10% was found.

2 50°C, approximately 10% RH plus a period for redistribution of moisture profiles.

The method requires control of temperature and weight loss (the duration of the drying period depends on the drying rate and will last until a preset weight loss is reached). Furthermore, information on sorption isotherms is needed.

After drying the specimens were sealed and stored at 50°C until age of 28 days. In most of the reported cases the difference between the outer and the inner part of the specimen was less than 10% RH.

The observed 10% difference in RH will not cause a significant influence on the moisture content if the relative humidity is in the area where change in moisture content ($d(w_n)/d(RH)$) is small, this is typically at 65-75% RH for non-virgin specimens, ref. Figure 2.3.1.

For drying method 2, however, the outer layer will adsorbe and the inner part desorbe during the redistribution period, and equal moisture content will never be obtained, only equal RH. The difference between absorption and desorption isotherms varies with the relative humidity, ref. Figure 2.3.1, thus the variation in moisture content within a sample depends on the actual RH.

Moisture Changes During Conditioning and the Effect on Frost Resistance

The influence of variations in test parameters was dealt with in [Laugesen et al, 1996]. Selected parts of Section 8.3 in the reference are given below together with additional information.

The degree of saturation is the single factor that has the largest effect on the result of a freeze /thaw test. The degree of saturation is determined both by the treatment before start of freeze /thaw - i.e. method of drying and re-saturation - and by the 'wetness' of the test, i.e. the period during which the specimen can absorb water [Fagerlund, 1996].

For concrete with low capillary porosity and the size of test specimens used in most of the test methods, the short term re-saturation period of 3-5 days, is expected to influence only the outermost part of the specimen. Defects in microstructure, such as cracks and larger porous systems may, however, cause saturation to a marked depth or even throughout the test specimen. Consequently, variation in microstructure may be directly responsible for variation in frost resistance due to increased absorption during both re-saturation and testing [Laugesen, 1996-b].

For cast test specimens being water cured until the moment of initiating the test, no (marked) water absorption during repeated freeze/thaw cycles has been observed for ASTM C 666 [Pigeon et al., 1986] and for Borås [Jacobsen et al., 1995].

It must be considered, however, that also very small water absorption might cause big frost damage in cores where the concrete was very close to critical saturation when the test started [Fagerlund, 1996].

For the test methods requiring curing in air, and/or if the conditioning includes potential drying of the test specimens, absorption during the freeze/thaw test can be observed which is profoundly larger than the absorption without freeze/thaw cycles. It has been shown for Borås and CDF [Jacobsen and Sellevold, 1994]. In these tests most concretes had a low degree of frost resistance and therefore were severely damaged in the test. In a concrete with high frost resistance the effect of salt on the absorption might be smaller or none [Fagerlund, 1996].

Increased absorption rates during freeze/thaw testing has also been recorded for ASTM C 666 [Laugesen, 1996-b].

Furthermore, it has been shown that the rate and degree of absorption are further increased by use of salt water, for Borås [Jacobsen and Sellevold, 1994], for CDF [Setzer, 1995] and for Cube test.

Guse and Hilsdorf [1994] have investigated the effect of conditioning on the frost resistance of high performance concretes (OPC +/- 10% MS, w/c=0.3, +/- air entrainment). The specimens (145 mm x 145 mm x 80 mm) were conditioned and tested as follows:

- Curing (mould/moist in jute and plastic foil): 1 - 49 days
- Drying (65% RH, 20°C?): 48 - 1 days (curing and storage total 49 days)
- Re-saturation (capillary suction of 3% NaCl-solution): 7 days
- Frost exposure according to CDF-method: From age 56 and two weeks onwards.

In general, the moisture content after re-saturation is less than or equal to the moisture content after curing. The most plausible reason is that water contained in coarser pores and defects is lost irreversibly during the first drying of the concrete (This water can be quite substantial in concrete containing porous aggregate.) ref. [Fagerlund, 1996]. A slight increase in moisture content upon frost exposure is observed. Increased curing up to 7 days followed by min 42 days drying caused less scaling than prolonged curing (and a decreased period of drying), ref. Figure 2.3.3.

Petersson [1994] compared the effect of drying before re-saturation according to:

1 SS 13 72 44 (climate chamber, RH=50 +/-5%, temperature=20 +/-2°C, wind<0.1 m/s)

2 No drying (demoulded after 2 days, then plastic bag)

A larger and linear rate of scaling of specimens not dried was observed. This was explained by these specimens having a critical moisture content from the beginning. The data obtained by the two methods correlated reasonably well (Portland cement concrete, w/c=0.45, 0.50).

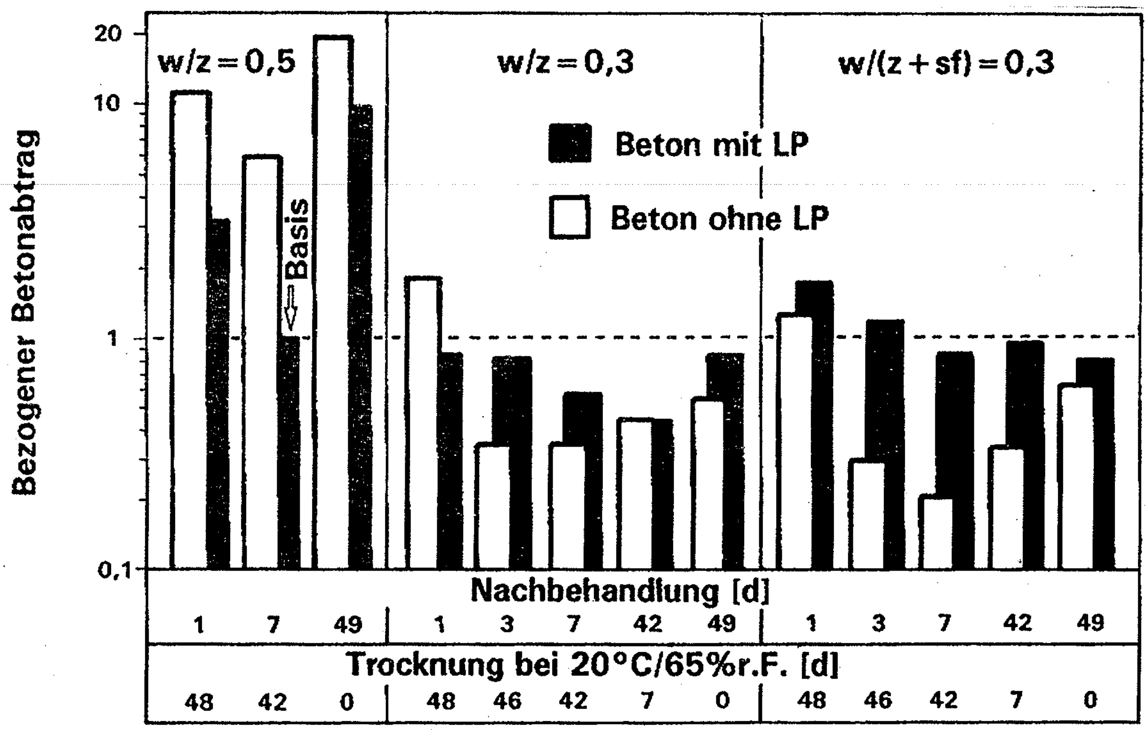


Figure 2.3.3 Scaling after 28 freeze-thaw cycles, 3% NaCl-sol. (1-sf,1:Mix with silica fume, air cured 1 day). [Guse and Hilsdorf, 1994].

[Jacobsen et al., 1995] have measured the weight loss of test specimens stored in climate rooms/cabinets under various conditions as well as the effect of conditioning on frost resistance. It was shown that samples conditioned at 50-65 % RH and moderate wind speed obtained the highest frost resistance measured by the Borås method, ref. Figure 2.3.4. It should be noted that the scaling measured is high and the relevence of the data for high performance concrete can be questioned.

The large effect of the drying procedure on scaling is not clarified. According to [Fagerlund, 1996], this may be due to:

1. Increase in the amount of freezable water after re-saturation, or
2. Decrease in the amount of water in coarser pores, interfaces, aggregate, etc.

Effect 1 is negative, effect 2 is positive. At mild drying, effect 2 is probably dominant. At hard drying, effect 1 is probably dominant. Thus, small changes in the drying climate might have very big effect on the result of the test.

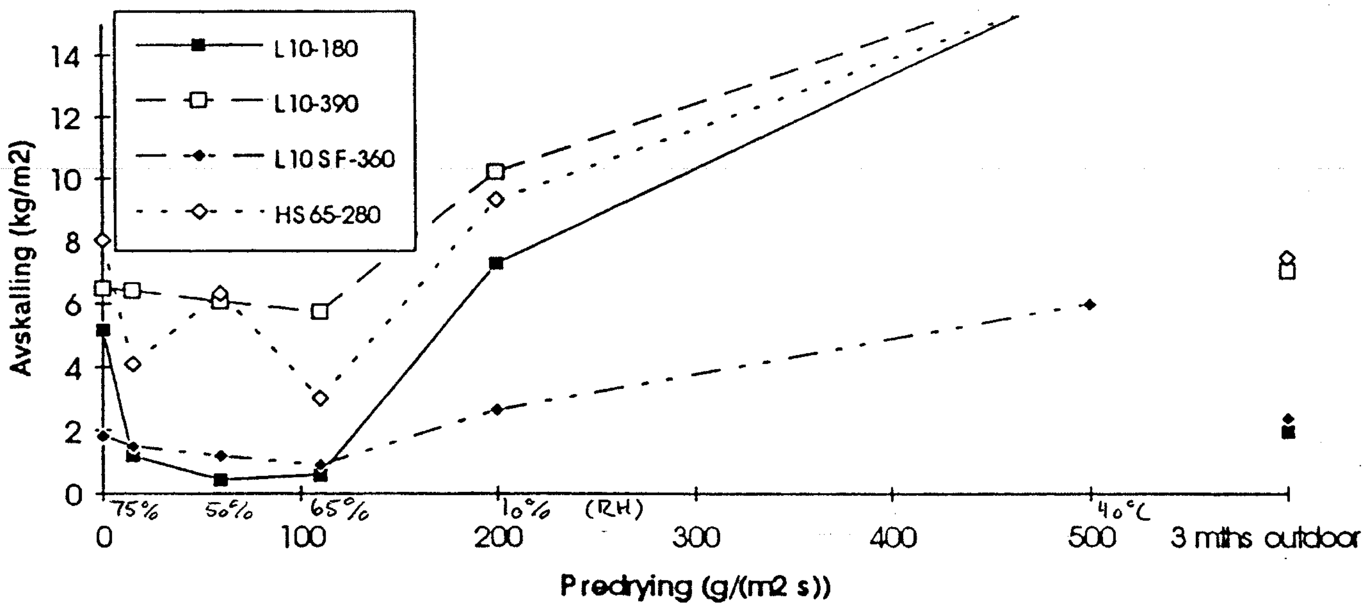


Figure 2.3.4 Weight loss of test specimens due to pre-drying at various RH-conditions. Note the correlation between predrying and RH. [Jacobsen et al.; 1995].

Selected Hypotheses and Observations to be Investigated

Freeze/thaw testing increases the rate of moisture ingress. The ingress is accelerated by exposure to salt [Jacobsen and Sellevold, 1994].

The moisture ingress after freeze/thaw testing is larger than the chloride ingress [Laugesen, 1996-b].

Severe drying causes less or similar moisture content after re-saturation e.g. [Rønning, 1996], [Guse and Hilsdorf, 1994].

Mild drying causes less scaling than no or severe drying, ref. e.g. [Jacobsen, 1995], who explained the observation by air voids may become water filled during water curing and drying increases the amount of freezable water.

Petersson [1994] compared the effect of drying before re-saturation. A larger and linear rate of scaling of specimens not dried was observed. This was explained by these specimens having no moisture gradient.

Concrete with microsilica is according to [Sellevold and Farstad, 1991] less susceptible to drying than ordinary concrete. Other than standard treatment according to the Borås method is too tough.

For high strength concrete (w/c+s=0.36, 10% microsilica) no pessimum value at 3% salt solution was observed by [Sellevold and Farstad, 1991]. That is, 3% salt solution was not found to cause maximal scaling. The effect of salt was not very large, but only as little as 0.5% was observed to have a significant effect.

Similar result have been found in tests performed at Lund Institute of Technology [Lindmark, 1995]. For concretes with w/c = 0,35 all concentrations in the range 1% - 5% NaCl gave almost excatly the same scaling which was considerably higher than the scaling in pure water.

Lindmark [Lindmark, 1996] is studying a hypothesis for the salt scaling mechanisms, according to which salt scaling depends on water being sucked from the solution to an ice lens formed about 1 mm from the surface (frost heave mechanism). Experiments seem to verify his hypothesis.

2.4 Test Programme

The effect on moisture movements of the following variations in testing parameters are investigated:

- Drying (temperature, relative humidity, duration)
- Re-saturation (duration)
- Testing regime (water, 0.5% and 3% NaCl-solution)
- Mix composition (w/c, +/- microsilica, +/- air)

Furthermore, the effect of conditioning and exposure during testing on frost resistance has been tested.

A method for measurement of moisture content and saturation levels at various places in the test specimen has been established (ref. Appendix 5).

Materials
Four different concrete types have been tested, ref. Table 2.4.1 and Appendix 1.

Table 2.4.1 Concrete mixes

	HUA-3, cylinders	HUA-5, cylinders	DBT-1 cylinders	DBT-2 cylinders
w/c	0.39	0.35	0.45	0.45
Microsilica fly ash	8% -	5% 12%	3.5% -	- -
Air	5.5%	5.5%	Natural air	Natural air
Cast	28 Sep 1994	21 Sep 1994	04 March 1996	04 March 1996
Water cured until	04 Oct 1995 12 months	04 Oct 1995 12 months	02 April 1996 1 month	02 April 1996 1 month
Packed in plastic	Until week 18 1996 7.5 months	Until week 18 1996 7.5 months	Until week 18 1996	Until week 18 1996

Specimens
Information on subdivision of cylinders and numbering of prisms is given in Appendix 2.

The specimens for measurement of moisture profiles were prepared by cutting prisms (35 mm x 35 mm x 70 mm) and sealing the prisms on all sides except one cut 35 mm x 35 mm face.

The material used for sealing is not permeable, but leakage (diffusion of water vapour) at the joint between bottom and sides may have happened, ref. profiles for drying according to method D2 i in Appendix 6. The ability of the sealer to remain unchanged during exposure has been documented (no deterioration, no weight change) [Laugesen, 1996-b].

Separate specimens (ø150/50 mm) were prepared for measurement of scaling (Borås).

Methods of Analysis and Testing
The method for determination of moisture and degree of saturation profiles is described in Appendix 4.

Chloride profiles were obtained by profile grinding and Volhard titration of the powder samples.

Thinsection (Petrographic) analysis were performed on thin sections placed perpendicular to surface and containing surface.

Testing

The test programme is given in Table 2.4.2.

HETEK, Task 3.3			MGE/22 April 1996, rev. 9 Aug.																	h:\28393\data\3.3\plan.xls											
			HUA-3																	HUA-5				DBT-1				DBT-2			
Initial	Density																														
analyses	Moisture		m																	m				m				m			
	Petrographic incl. air		p																	p								p			
Sample preparation, prism nos.			9	50-53	1-4	10-13	19-22, 27-31	37-40	46-49	55-58	100-103	30-31	104-106	107	5-8	14-17	23-26	32-35	41-44	72	64-47	73-76	68-71, 77-81	90	82-85	91-94	86-89, 95-99	117	109-112	118-121	113-116, 122-126
Drying	D0	None		x																											
	D1	20/65%, 21 days			x	x	x	x	x	x	x	x	x	x							x	x	x		x	x	x		x	x	x
	D2	50/low RH, 14 days													x	x	x	x	x												
Analyses	Weight change during drying				w	w	w	w	w	w	w	w	w	w	w	w	w	w	w		w	w	w		w	w	w		w	w	w
	Moisture				m										m						m				m				m		
Re-sat.	S1	3 days				x	x	x	x							x	x					x	x			x	x			x	x
	S2	14 days								x	x							x	x												
	S3	Vacuum saturation										x	(x)	(x)																	
Analyses	Weight change during re-sat.					w	w	w	w	w	w	w	w	w		w	w	w	w			w	w			w	w			w	w
	Moisture					m				m						m		m				m				m				m	
Frost	T1	3% NaCl		x			x				x		(x)				x		x				x				x				x
testing	T2	0.5% NaCl						x																							
(28 days)	T3	Water							x																						
Analyses	Weight change during testing																														
	Scaling during testing *			s			s	s											s				s				s				s
	Moisture			m			m	m	m		m		m				m		m				m				m				m
	Chloride						c						c										c								c
	Petrographic incl. air						p						p										p				p				p
Time Schedule																															
	Week	18																													
		19																													
		20																													
Signatures:		21																													
Preparation		22																													
Drying		23																													
Re-sat.		24																													
Testing		25																													
Reporting		26																													
Testing		27																													
Control		28																													
		29																													

* Separate specimens (2 each ø100 mm x 50 mm)

Table 2.4.2 Test Programme

2.5 Results and Discussion

Abbreviation are used for various drying (D), re-saturation (S), and testing (T) regimes, ref. Table 2.4.2.

The degree of saturation was determined by both capillary saturation and vacuum saturation. No significant different degree of saturation was found by these methods. Capillary degree of saturation data are given in the following.
It should be noted that none of the methods provide a filling of the airvoids, ref. [Fagerlund, 1996].

Effect of Conditioning - Drying and Re-saturation - on Moisture Content and Profiles

Desorption and Adsorption

Data on weight change of the prisms including figures are given in Appendix 5.

The Borås conditioning (D1/S1) is observed to have no significant influence on the moisture content of the HUA-concretes (w/c=0.35 and 0.39), ref. Ark 1, Chart 5. Prolonged re-saturation from 3 days (S1) to 14 days (S2) causes an increased moisture content, which is slightly higher than the initial. Ref. Ark 1, Chart 1. Vacuum saturation of HUA-3 (S3, no data in Appendix) resulted in 0.16 kg/m^2 absorption after 2 days, corresponding to half of the absorption after S2, and 0.33 kg/m^2 absorption after 12 days.

The DBT-concretes (w/c=0.45) are, as expected, more affected by the Borås conditioning, but does only absorb less than half of the moisture lost during drying, ref. Ark 1, Chart 5.

Drying of the HUA-3 concrete at 50°C for 14 days (D2) causes a significant moisture loss. Re-saturation for 3 days only causes an absorption of 1/4 of the moisture loss, and prolonged re-saturation (D2/S2) causes a further absorption at 1/8 of the moisture loss. Ref. Ark 1, Chart 4.

The mean rate of desorption and absorption during drying, re-saturation, and freeze-thaw exposure is illustrated in Ark 7, Chart 1 and 5. It can be observed that freeze-thaw exposure did not increase the rate of absorption of the HUA-samples. The DBT-samples did deteriorate during freeze-thaw exposure, and no weight change cold be measured.

Moisture Profiles

Data on moisture profiles and figures are given in Appendix 6.

In the figures of the profiles a few results have been omitted due to a very small specimen size (< 10 g). The illustrated data are single measurements (Chart 1-6) and mean of 2 (Chart 7, 8, 10, and 11).

Not all prisms were subdivided and profiles analyses performed immediately after exposure. The delay was between 0 and 6 (10) weeks ref. Appendix 4. The analysis of prisms exposed to D2 was delayed 5 weeks. Even after this long delay a significant difference in moisture content between surface and centre was observed, ref. Ark 3, Chart 1. Therefore, we assume - at least to some extent - the data to be valid.

In the following data for capillary degree of saturation (relative moisture content compared with the maximum moisture content at capillary suction) is used, as the measurements of moisture content will be affected by variations in paste content. A 5%-point difference in capillary degree of saturation between identically exposed samples is observed for several of the test series. Measurement of degree of saturation is destructive, i.e. the samples cannot be re-used. Thus, each profile is measured on a different prism.

The data referred to are degree of saturation based on saturation by capillary suction. Results based on saturation by evacuation are also illustrated in Appendix 6.

The HUA-3 samples dried according to the Borås method show no variation in the degree of saturation as a function of depth from the exposed surface, ref. Ark, Chart 1. Furthermore, re-saturation (S1 and S2) and freeze/thaw exposure (T1, T2 and T3) appear to cause no or only a slight increase in the degree of saturation of the outer layer, ref. Ark 3, Charts 2 and 4. The latter is maybe due to the high degree of saturation of the samples before re-saturation (90-95%).

The available data does not confirm information on salt (T1, T2) causing increased moisture ingress and the effect of the salt concentration in the exposure liquid.

The HUA-3 samples dried according to D2 show a large variation in degree of saturation as a function of depth from the exposed surface (50% in the outer layer (10 mm), 70-75% at further depth, 60-65% at the bottom), ref. Ark 3, Chart 1. (The low degree of saturation in the bottom layer is assumed to be caused by the joint between sealer at the bottom and sides not being intact.) Re-saturation causes an increase in degree of saturation (outer layer 75% (S1) to 80% (S2)), ref. Ark 3, Chart 3. And freeze-thaw exposure a further increase (85-95%), ref. Ark 3, Charts 5 and 6.

Drying and re-saturation according to Borås of the DBT-samples are observed to affect the outer layer only, where as the degree of saturation of the entire prisms is increased during testing, ref. Ark 3, Chart 7, 8, 10, and 11. The latter is explained by cracking opening up for moisture transport, ref. Section "Macro-and Microstructural Changes After Conditioning and Frost Exposure".

Effect of Conditioning and Frost Exposure on Frost Resistance

Results from testing of HUA-3, HUA-5, DBT-1, and DBT-2 according to the Borås method (D1/S1/T1) and of HUA-3 specimens varying in conditioning (D0, D1/S1, D2/S2) and freeze/ thaw exposure (3% salt (T1) and 0.5% salt (T2) are given in Appendix 7.

All HUA-specimens show very good to good performance. As expected, neither of the DBT-concretes are frost resistant (no air entrainment).

No effect of the conditioning and freeze-thaw exposure has been observed on the frost resistance of the tested concrete (HUA-3). This may be due to the good performance of this concrete.

Macro- and Microstructural Changes After Conditioning and Frost Exposure

Thin section petrography was performed on samples of HUA-3, HUA-5, DBT-1, and DBT-2 not tested (DO), (not DBT-1) as well as exposed to D1/S1/T1 (Borås, 28 days), ref. Appendix 9.

The sections were placed perpendicular to the exposed surfaces of the samples.

The effect of Borås freeze-testing is very pronounced in the non-air entrained DBT-samples. The surface is deteriorated to several mm depth and stone and sand particles are separated from the cement paste.

Coarse and fine cracks are found sub-parallel to the exposed surface further down in the samples. The cracks propagate mainly in the paste and around the aggregate particles. The latter phenomenon occurs to the full depth of the thin sections (5 cm).

The HUA-samples were air entrained and had lower w/c-ratio than the DBT-samples (0.35 and 0.39 against 0.45). There were no signs of interior cracking in the frost tested HUA-samples. Furthermore, surface cracking was only observed around one sand grain in one sample.

Moisture and Chloride Ingress during Frost Exposure

The chloride content in prisms exposed to Borås testing (28 days freeze-thaw exposure) is given in Appendix 8.

3 Improvements of Test Methods

3.1 Measurement of Absorption during Testing

This test method determines the content of absorbed water of samples of hardened concrete, during testing for freeze/thaw resistance according to SS 137244 and ASTM C666.

Referenced Documents

SS 137244: Concrete Testing - Hardened Concrete - Scaling at Freezing (Borås)
ASTM C666: Resistance of Concrete to Rapid Freezing and Thawing

Definitions

The test specimen weight is recorded during conditioning and testing as surface dry weight. Surface dry conditions are obtained by wiping the sample with a damp cloth. Conditioning comprises drying and resaturation as specified by the test methods.

Test Specimens

The test specimens are specified in the given test method.

Apparatus

Electronic scale, accuracy: +/- 0.02 g.
Heating closet, accuracy: +/- 2°C.
Sieves

Test Procedures

The samples received for testing are cut to the dimensions specified in the given method. Immediately after cutting, each test specimen is surface dried by use of a dry cloth, and placed in laboratory atmosphere for 30±5 min.

At this moment the initial weight (m_i) of the test specimen is recorded.

The weight of the conditioned test specimen (m_0) is recorded just before freeze/thaw testing is initiated.

At the intervals specified for freeze/thaw-measuring - scaling in SS 13 72 44 or change of E_{dyn}/length in ASTM C 666- the test specimen is surface dried and weighed (m_t). For each test specimen, the scaled material is collected, dried (105°C) and weighed (m_x). This is relevant for both freeze/thaw testing methods.

After testing, the test specimen is dried at 105°C to constant weight (m_{dry}).

Calculations

The relative water content of the test specimens, often referred to as the moisture ratio (expressed in weight-%) are computed according to (1).

$$W_t = \frac{(m_t + m_x) - m_{dry}}{m_{dry}} \tag{1}$$

The relative water uptake during conditioning is computed according to (2).

$$W_c = \frac{m_0 - m_i}{m_{dry}} \tag{2}$$

The relative water uptake during freeze/thaw testing is computed according to (3).

$$W_{ft} = \frac{(m_t + m_x) - m_0}{m_{dry}} \tag{3}$$

where

W_t: water content (at time t) relative to dry weight of test specimen.
W_c: water uptake during conditioning, relative to dry weight of test specimen.
W_{ft}: water uptake during freeze/thaw testing, relative to dry weight of test specimen.
m_i: weight of test specimen at beginning of curing.
m_0: weight of test specimen at beginning of freeze/thaw testing (corrected for weight of rubber cloth and glue for SS 137244; corrected for gauge studs for ASTM C 666)
m_t: weight of test specimen at the time t (corrected as above).
m_{dry}: weight of dry test specimen (corrected as above).
m_x: weight of scaled material, summarized from time: 0 to t.

The water absorption as function of time is computed according to (1), and shown graphically.

Reports

The test report comprises the following:
- general report information,
- sample no., test specimen dimensions, m_i and m_0,
- frost/thaw-time, frost/thaw-cycles, m_t, m_x, and m_{dry},
- computed results of W_t, W_c, and W_{ft},
- absorption curve.

Laboratory record for measurement of absorption during conditioning and testing.

Sample no.	Height (mm)	Diameter (mm)	m_i (g)	m_0 (g)	

Time (d) of freeze/thaw testing										
Freeze/Thaw cycles										
m_t (g)										
$m_{x(t)}$ (g)										
m_x (g) (summarized $m_{x(t)}$)										
m_{dry} (g)										

W_t: (%)	W_c (%)	W_{ft} (%)	

3.2 Conditioning of Test Specimens

The use of test methods for frost resistance of concrete can be divided in two situations:

* Pretesting of concrete as part of the documentation before casting
* Test on specimens taken from an existing structure or concrete product in use

Corresponding to this, different procedures for handling the concrete before testing may be used. It is necessary to distinguish between:

* Curing and conditioning related to curing of casted concrete specimens

* Conditioning related to the drilling or sawing of specimens from a structure

* Conditioning as part of the test procedure

Pretesting of Concrete

In this situation the concrete specimens are cylinders or cubes casted and cured following a standard procedure leading to a proper hardening before testing. Such procedures involves 1 day with the concrete in form followed by at least 6 days in water at 20 °C, for practical reasons this period often is prolonged until testing.

When dealing with HPC one can expect the concrete to be very dense towards water movement after proper hardening eg 7 days of hardening in water. Therefore only very high water moving thermodynamic potentials will be able to change the water content in the concrete. The water content will in this case for normal treatments stay at a rather constant level. Consequently the water content in the concrete will be high which is the most risky condition regarding to frost exposure and no difference in test results should be found between concrete water cured for 7 days followed by air curing and concrete water cured for a prolonged period. Figure 3.2.1 shows the results from tests using two different curing methods.

Curing of HPC in water until testing without air conditioning should lead to a judgement of the frost resistance of the concrete on the safe side. Conditioning would only reduce the water content slightly reducing the risk of frost damages.

Concrete Structure in Use

In this situation the concrete specimens are often cores taken from a structure with the water content somewhat in equilibrium with the surroundings depending of the age of the structure. Therefore it would be a natural procedure to test the concrete directly in this condition with due respect of the changes in the water content developing from the necessary preparations before testing.

Non-conforming air void structure may call for frost testing as bases of acceptance. In such cases the specimens should be conditioned to a water content corresponding to the expected level in use.

Conclusion

Based on results from the supplementary research on moisture movements in HPC and calculations the following conditioning procedures should be used

* Pretesting
  Directly testing with high water content at the time of the required maturity of the concrete

* Specimens from structure
  The specimens are - depending on actual exposure - conditioned to a specified degree of saturation.

Scaling as a Function of Conditioning - an Example

When using the test method SS 137244 as a pretesting method the specimens has to be cured until 28 days before testing. The swedish method for curing concrete specimens requires 1 day in form 6 days in water at 20 °C and 21 day in air 20 °C and 65 %RF. The danish method method for curing concrete specimens requires 1 day in form and 27 day in water at 20 °C.

Due to practical reasons test on frost resistance has been made on concrete cured in accordance with the danish standard. To investigating the influence of the different methods of curing before the conditioning related to the frost/thaw method two test series has been made.

Basically the same type of concrete is used in both series. The concrete should apply with the requirements for concrete in aggressive exposure class, exposed to frost attack with deicing salt. There is a difference in the water cement ratio. W/C = 0.37 and W/C= 0.42 are used. Specimens cured in water are 50 mm slices from D150 mm cylinders, while specimens cured in air are 50 mm slices from 150 mm cubes.

Figure 3.2.1 shows, that based on evaluations up to 56 cycles the test results are the same for the four series. Up to 112 cycles which sometimes are required there is some deviation; but it is not systematical, which leads to the conclusion that for the high performance concrete used these two curing regimes gives the same test results. The concretes tested are classified as "good".

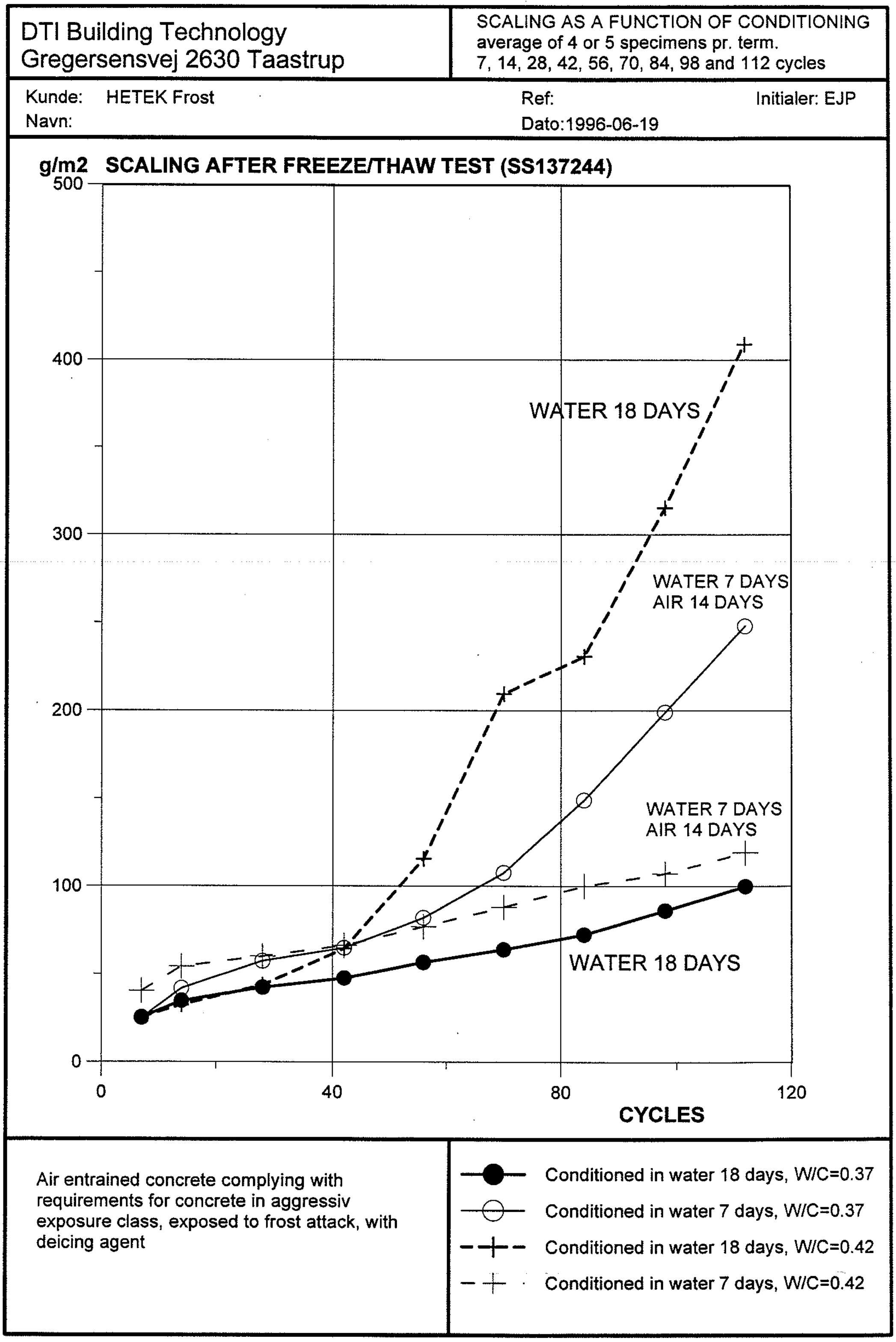


Figure 3.2.1: Scaling as a Function of Conditioning.

3.3 Temperature Control during Testing

All methods for determining the frost-resistance of concrete, concrete products and aggregates have a common feature - the temperature of the test specimens shall run through a pre-defined course a number of times. This temperature history must be documented, because it influences the result of the test. Inadequate temperature control is probably the main cause of the considerable scatter among the results of several Round Robin tests [Laugesen et al., 1996].

Various methods for controlling the temperature will first be described. An example of the influence of the temperature history on the test result will then be given.

Temperature Control and Documentation

There are several methods for controlling and documenting the temperature history.

The simplest form of temperature control requires only that the specimen be placed in a freezer set to a given temperature, and subsequently transferred to e.g. a water-bath. The documentation is usually confined to measuring and registering the temperatures in the freezer and the water. The advantage of this form of control is that it is easy to set up the apparatus.

The disadvantage is that a manual effort is needed for each temperature cycle and its documentation. It is the stationary state corresponding to the temperature of the surroundings that is documented, not the state of the test specimen. The result may be influenced by the total volume and weight of the test specimens.

Temperature control can be carried out with the aid of timers, by which it can be defined at e.g. 15-minute intervals, whether the heating or cooling system shall be switched on (or possibly on/off for heating and cooling independently). Temperature documentation is carried out by means of additional equipment, mounted separately. The advantage of this method is that any temperature history can be controlled with a modest amount of equipment.

The disadvantage is partly that it is extremely difficult and slow to get the timer correctly set, and partly that the setting must be adjusted if the number of specimens in the freezer is changed. This difficulty is overcome in practice by always having the same number of specimens in the freezer. Because of the need to cover peak loads, this involves having a very large number of specimens continuously in the freezer. And if water is used in the testing, it must be ensured that all the specimens are always immersed.

In direct temperature control the temperature is regulated automatically on the basis of the measured temperatures, the positions of the measuring-points corresponding to those required for the documentation.

Developments in personal computers and data registration have made automatic control of temperature and documentation technically feasible and economically acceptable.

The advantages of such a control method are:

- The system ensures that the temperature history of the specimens is correct.
- The control is independent of the number of specimens in the freezer, provided that the freezer capacity is sufficient.
- Data for documenting the temperature history can be collected and presented automatically.
- It is simple to adapt the freezers to different test methods.

System Components

A system for freezer control has the following components:

- Freezer with cooling and heating system
- Datalogger and/or personal computer capable of collecting data and controlling relays
- Two relays for on/off for heating/cooling, depending on the signal from the chosen control unit
- Thermo-sensors, e.g. copper/constantan
- Software for collection and presentation of data and for freezer control

The temperature for documenting that the temperature history corresponds to that required by the testing method shall be measured in accordance with the instructions; for example, the temperature in the Swedish method (salt scaling) is measured in salt water. In a modified "critical dilation test", with air-cooling, the temperature must be measured at the centre of the specimen or at the centre of a similar specimen. In accordance with these requirements, the measuring-point of the thermo-element is placed in the salt water and at the centre of the specimen respectively.

The freezer control must ensure that the required temperature history is achieved. It is therefore not necessary to control on the basis of the temperature that will subsequently be documented, but if it is possible to do so, this method is preferable. Both direct and indirect control will be described in the following, the above-mentioned test methods being used as examples.

Direct Control

In the Swedish method it will be an advantage to control directly on the basis of the temperature measured in the water either on the specimen or a similar specimen (a "dummy"). By controlling on the basis of the temperature to be documented, the system is very simple.

As there is a certain inertia in the system, the control should not be based on the absolute temperature, but on a temperature gradient.

If the heating and cooling is switched on and off depending on whether the temperature is above or below the temperature aimed at, the temperature curve can be kept within the permissible limits, but will oscillate around the target curve (see figure 3.3.1).

These oscillations can be largely avoided by controlling on the basis of temperature gradients (see figure 3.3.2); this means that switching on and off depends on whether the rate of change of temperature is higher or lower than that aimed at.

Figure 3.3.3 shows how control can be carried out in such a way that highly reliable curves are obtained.

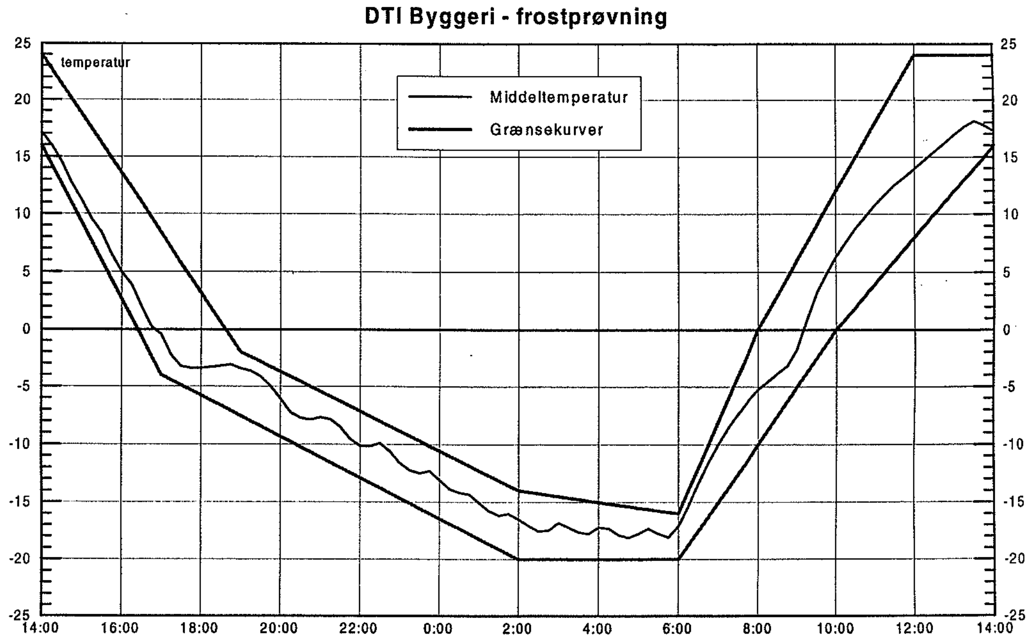


Figure 3.3.1.

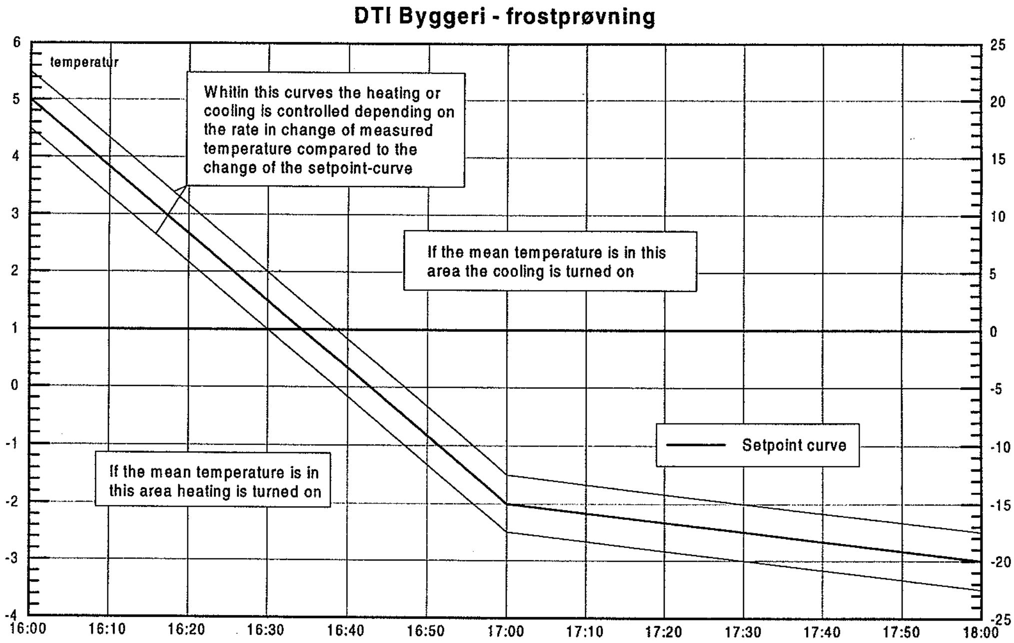


Figure 3.3.2

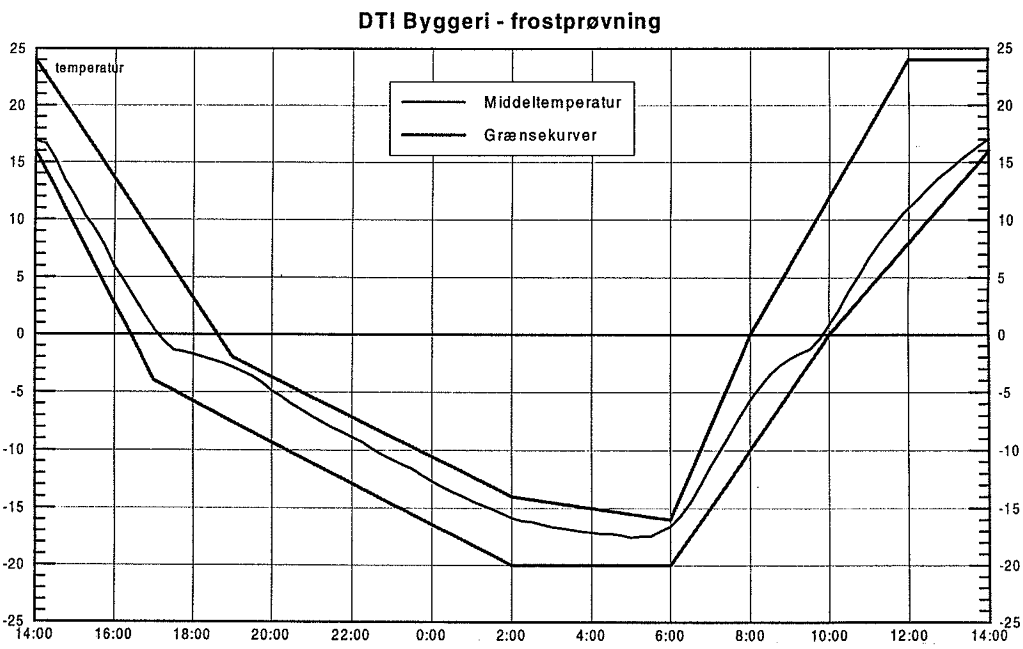


Figure 3.3.3

Indirect Control

In the modified "critical dilation test" and in the ASTM C 666 method, it is difficult if not impossible to base a control on the temperature to be documented, as this is the temperature at the centre of a specimen. This is due to the high inertia of the system. If the temperature of the specimen is too high, the cooling system will be activated, but it will take some time before the cooling affects the measured temperature. When the temperature has fallen below that aimed at, the cooling is switched off, and the heating may be switched on. But the temperature at the centre of the specimen will continue to fall for some time. The temperature will therefore oscillate, and the correct curve will be unattainable. With this type of control, the temperature in the freezer will vary considerably, and the subsequent temperature compensation for the measured changes in length will be subject to a wide margin of error.

In the case of "the Modified Critical Dilation Test" it is desirable to choose an indirect form of control, based on the temperature of the air, the temperature measured at the concrete surface or on the rods used for measuring the changes in length. This procedure involves experimenting a little with the temperature curve that is to be adopted as the target curve. In the frost-boxes of the DTI, the air temperature is maintained at approx. 4° C below the temperature required at the centre of the specimen during cooling, and 4° C above it during warming. This means that the air temperature changes by 8° C as rapidly as the capacity of the system permits when cooling is changed to warming. The control of the air temperature ensures a uniform cooling, both in air and concrete, and this in turn ensures that the temperature compensation can be effected with a high degree of reliability.

In the ASTM C 666 test method the temperature is measured on a dummy specimen, which creates problems similar to those mentioned above. In addition to this measurement, the temperature in the testing chamber and the specimen container is checked by using procedure A, in which the specimen is placed in water in a container, or directly on the specimen by using procedure B, in which the specimen is exposed to the air. These conditions give rise to additional requirements for the control and the extent of the documentation.

The Influence on the Test Result

Methods for testing the frost-resistance of concrete involve subjecting the test specimens to a temperature "loading" in the form of a number of temperature variations within a given time. To ensure uniformity, these temperature cycles must lie within certain limiting curves.

The significance of this requirement was investigated in a test programme based on the temperature requirements of test method SS 13 72 44. The effect of variations in the amplitude of the temperature cycle and the rates of cooling and warming were investigated by using three types of temperature cycle (see figure 3.3.4):

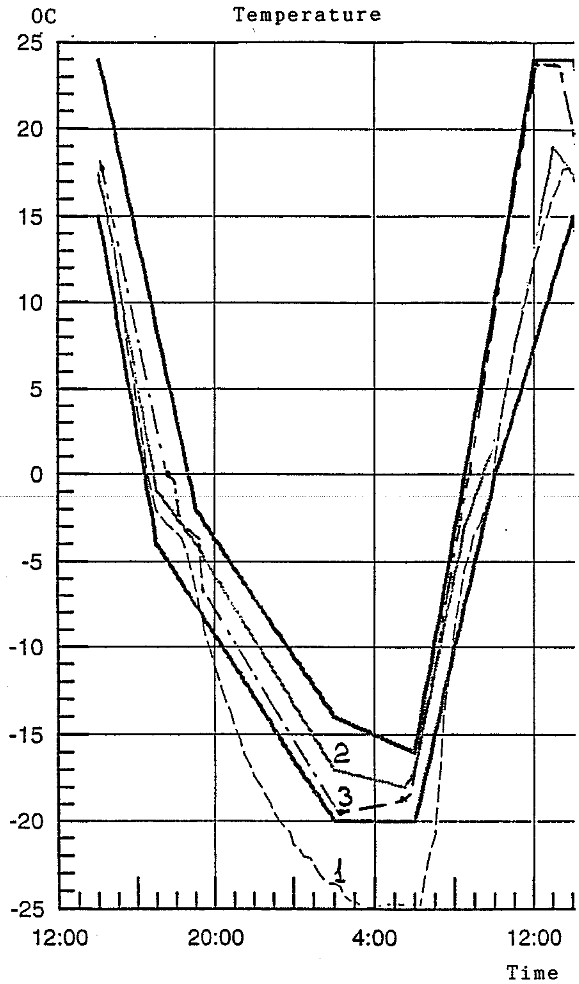


Figure 3.3.4

1: *In the frost phase the temperature is outside the limiting curve, so that the temperature falls very rapidly to a minimum of -25 °C and rises even more rapidly afterwards. The maximum gradient during cooling is -3.5 °C/h, during warming 7.7°C/h.*

2: *The temperature is at the middle of the permitted interval. The maximum gradient during cooling is -1.9°C/h, during warming 5.2°C/h.*

3: *The temperature is within the permitted interval, but the gradients are as high as possible. The maximum gradient during cooling is -2.0 ° C/h, during warming 6.0 °C/h.*

The scaling measurements were carried out on concrete that meets the requirements for an aggressive environment:

- Water Cement Ratio 0.41
- Air content in hardened concrete > 10% of the binder volume
- Specific Surface Area > 25 mm^2/mm^3
- Aggregates complying with the requirements for exposure class A

The results (see figure 3.3.5 and 3.3.6) after 56 cycles show:

* That temperature cycle No.1 results in an amount of scaled material 40% to 120% higher than cycle No.2, with a mean increase of 90%.

* That there is no significant difference in the amount of scaled material resulting from the cycles within the permitted temperature interval (Nos.2 and 3).

These results confirm those given in: Petersson, P.-E. and Lundgren, M., "Influence of minimum temperature on the scaling resistance of concrete", Swedish National Testing and Research Institute, Borås, Sweden. Table 2 (concretes A, C and S) shows that a change in the minimum temperature results in a 30 - 100% change in the amount of scaling, and that increasing the gradient from 2° C/h to 4° C/h increases it by 18 - 56%.

The results also (partly) confirm tests made at the Division of Building Materials, Lund Institute of Technology (Report TVBM 3060, 1995) according to which the effect of minimum freezing temperature can be described by:

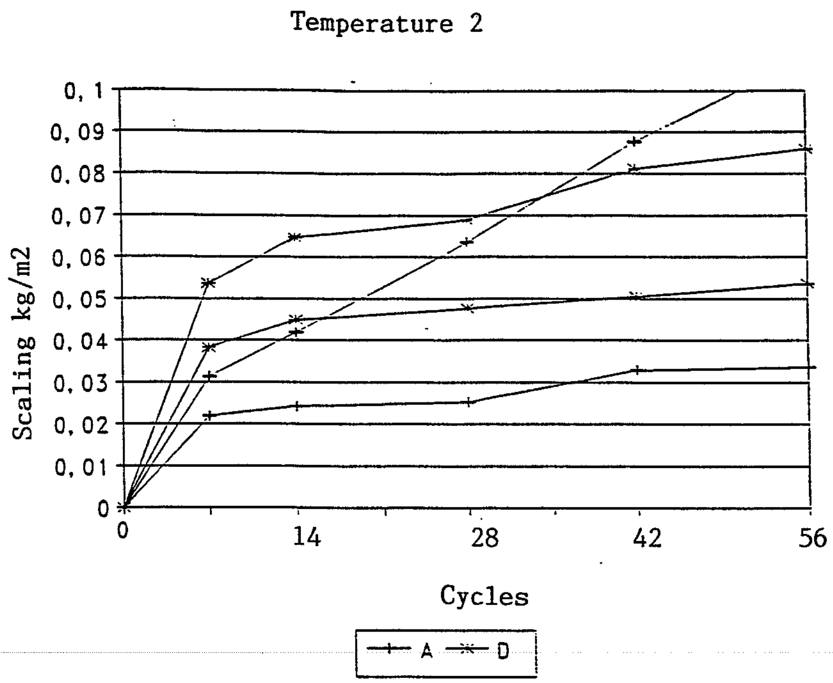
$$\text{Scaling} \approx \text{constant} \, |\theta_{min}|^2$$

[Fagerlund, 1996].

Conclusion

Temperature level and temperature gradient influence the results of the frost-resistance tests investigated.

It is important to keep the temperature cycle within the limiting curves if uniformity of effects and comparability of results are to be achieved.


Temperature 2
Scaling kg/m2
0, 1
0, 09
0, 08
0, 07
0, 06
0, 05
0, 04
0, 03
0, 02
0, 01
0
0
14
28
42
56
Cycles
A
D

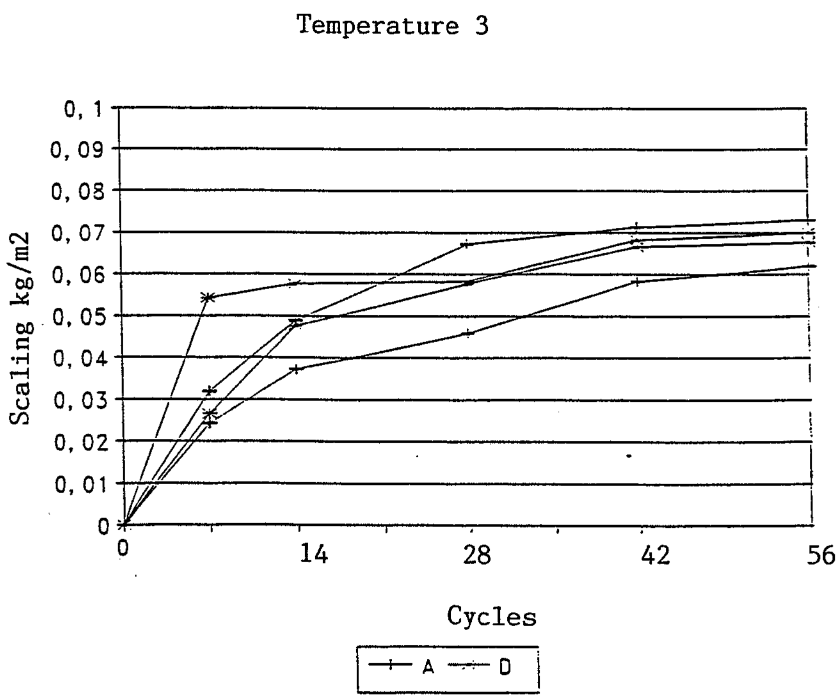

Temperature 3
Scaling kg/m2
0, 1
0, 09
0, 08
0, 07
0, 06
0, 05
0, 04
0, 03
0, 02
0, 01
0
0
14
28
42
56
Cycles
A
D

Figure 3.3.5

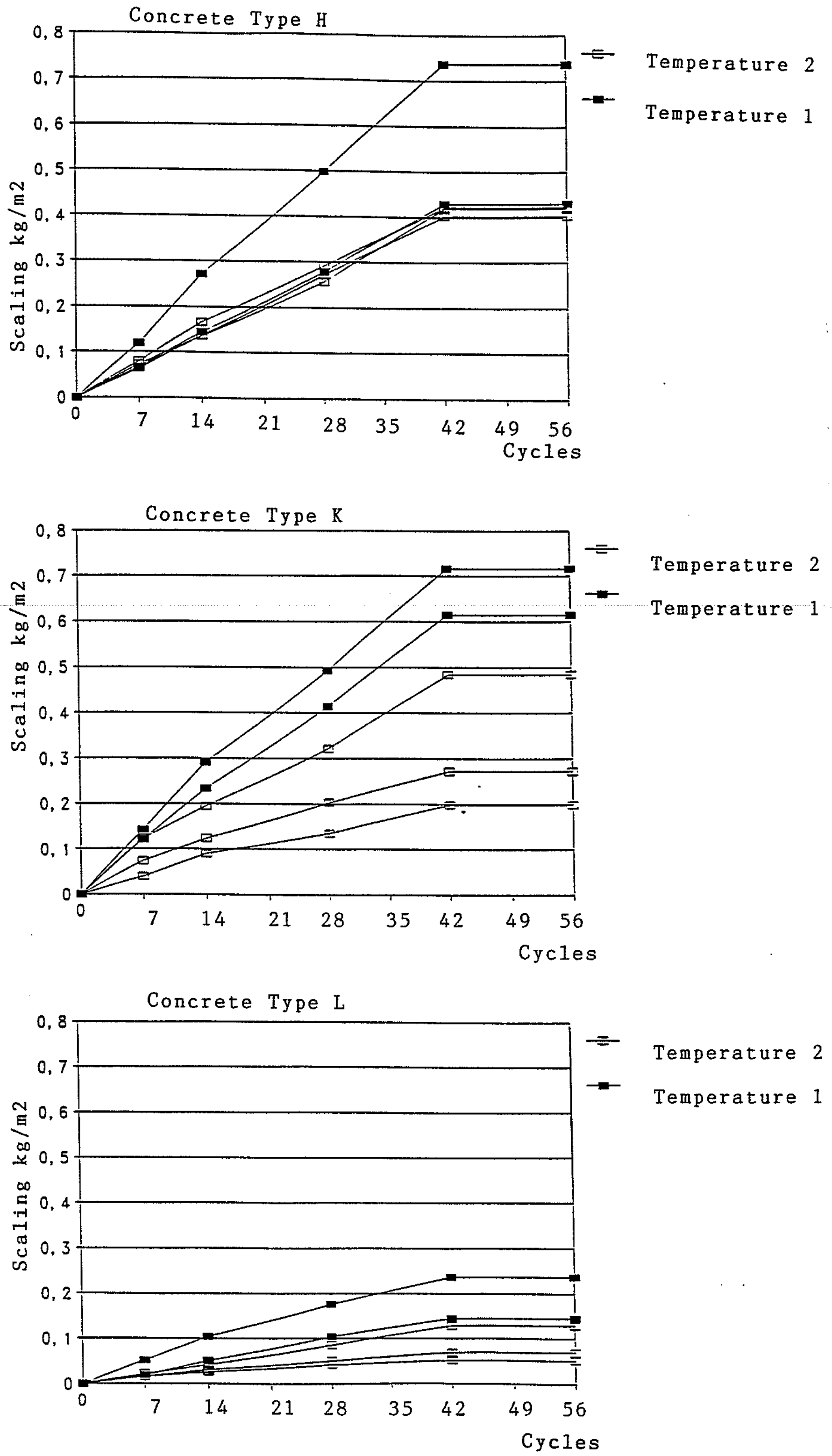

Concrete Type H
Scaling kg/m2
0,8
0,7
0,6
0,5
0,4
0,3
0,2
0,1
0
0
7
14
21
28
35
42
49
56
Cycles
Temperature 2
Temperature 1
Concrete Type K
Scaling kg/m2
Temperature 2
Temperature 1
Concrete Type L
Scaling kg/m2
Temperature 2
Temperature 1

Figure 3.3.6

3.4 Internal Damage Produced during Borås Testing

Concrete structures show two different types of freeze/thaw-damages:

* Surface scaling
* Internal cracking/delamination.

The existing freeze/thaw-test methods are generally regarded as representing only one of the two. Thus, the Borås method is applied for surface scaling, whereas ASTM C 666 is generally applied for internal cracking.

However, the present knowledge of freeze/thaw-testing may reflect how the samples were measured, rather than how they actually responded to the testing.

In order to investigate the possible formation of internal cracks in the concrete samples during Borås testing, 3 different methods were applied together with the normal scaling measurements:

* Length change measurements
* Dynamic E-modulus changes, by ultra sonic measurements
* Microstructural analysis, by thin section petrography

Two samples were tested from each of 4 different concrete compositions, two with expected high frost resistance (HUA 3 and 5) and two with expected low frost resistance (DBT 1 and 2).

The samples tested were concrete slices with 150 mm diameter, taken from the same concrete cylinders as applied in Chapter 2 of the present project - On the moisture movements in High Performance Concrete:

* HUA-3: Equiv. W/C 0.35 (C+S+F), air-entrained
* HUA-5: Equiv. W/C 0.39 (C+S+F), air-entrained
* DBT-1, W/C 0.45 (C), not air-entrained
* DBT-2, Equiv. W/C 0.45 (C+S), not air entrained

The mix compositions are given in Appendix 1.

The results of the scaling (see table 3.4.1) characterises the HUA samples as having very high frost resistance, and a (very) low frost resistance of the DBT samples, as intended.

Table 3.4.1 Scaling values for the tested cylinders.

Mix no. and Treatment[1]	Specimen no.	Scaled material [kg/m^2]				
		7 cycl.	14 cycl.	21 cycl.	28 cycl.	35 cycl.
DBT-1 (D1/S1/T1)	3-1 3-2	0.09 0,07	0,66 1,30	1,34 2,03	2,52 2,37	3,64 2,64
DBT-2 (D1/S1/T1)	6-1 6-2	0,14 0,10	1,00 1,02	2,20 2,19	3,31 3,49	4,75 4,72
HUA-5 (D1/S1/T1)	28-1 28-2	0,00 0,01	0,02 0,02	0,02 0,03	0,03 0,05	0,04 0,05
HUA-3 (D1/S1/T1)	47-1 47-2	0,00 0,00	0,01 0,00	0,01 0,00	0,01 0,00	0,01 0,00
HUA-3 (D1/S1/T2)	66-1 66-2	0,03 0,02	0,06 0,03	0,07 0,04	0,09 0,04	0,10 0,04
HUA-3 (D0/T1)	42-1 42-2	0,00 0,00	0,00 0,00	0,00 0,00	0,01 0,01	- -
HUA-3 (D2/S1/T1)	70-1 70-2	0,00 0,00	0,00 0,00	0,00 0,01	0,00 0,01	- -

Note 1: 'Treatment' refers to the treatment during conditioning and testing, D1/S1/T1 being standard Borås Drying/Saturation/Testing, see table 2.4.2

Length Change Measurements

The orientation of the internal cracking recorded in concrete structures are mainly surface parallel. To record a corresponding deterioration in Borås test samples, the length change should be measured perpendicular to the exposed surface.

However, practical problems of measuring perpendicular to the exposed surface restricted the possibilities to measuring parallel to the exposed surface.

A length change measuring frame with a precision dial gauge was produced for testing the samples. With the gauge studs applied, the lengths were recorded with an accuracy of ±2 microns.

The lengths parallel to the surface were recorded at depths of both 5 mm and 25 mm, see figure 3.4.1.

The lengths were measured at intervals corresponding to the scaling measurements: 0, 7, 14, 28, and 35 days. Due to the short time limits of the project, the samples were run for 35 days only.

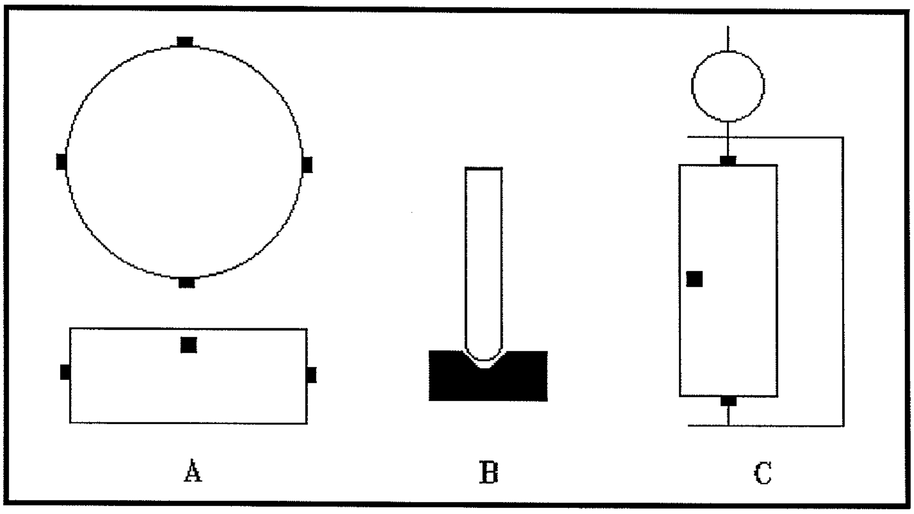


Figure 3.4.1 A: Placing of gauge studs on the test specimen.
B: Gauge stud and Dial gauge point.
C: The test specimen placed in the measuring frame.

The results of the length measurements are given in figure 3.4.2 and 3.4.3.

For the samples with low frost scaling resistance (DBT-1 and DBT-2), the length changes are distinct, most markedly developed in the surface of the samples. For the samples with very little frost scaling (HUA-3 and HUA-5) no or little length change can be noted.

The length changes appears to be gradual, but some irregularities are seen on the curves. This may be caused by:

* Inadequate bonding of the gauge studs, most prominent in the DBT-1 and DBT-2 samples having low frost resistance.
* Varied concrete temperature at measuring terms.

The lengths of the samples were measured at the scaling measurements, i.e. while the temperature of the test liquid was 23-24°C The temperature of the concrete samples were not measured. During the measurements, it was noted that the concrete samples were 'cold'. Hence, the concrete temperature may have varied from time to time, thus influencing the length. The maximum error due to this effect cooresponds to a temperature error of max. 10°C. This gives a length error of $\approx 10^{-2}$ % which is a low value compared to the measured expansions. It should be noted that this error in temperature indicates that the temperature of the concrete samples by far reaches 20°C during the heating of the Borås test.

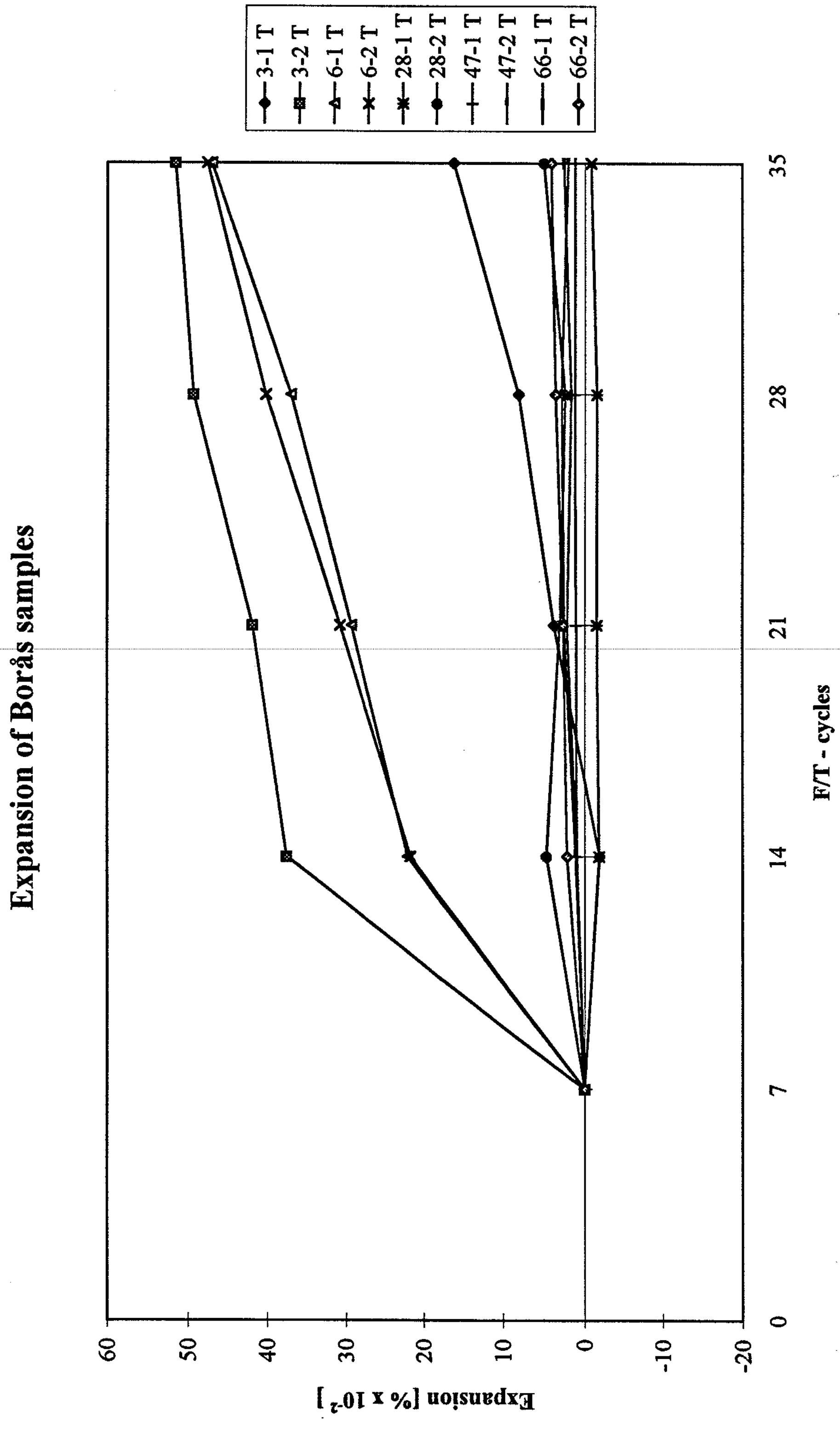


Figure 3.4.2 Horizontal length change during freeze/Thaw testing, measured 5 mm from the surface of the Borås samples.

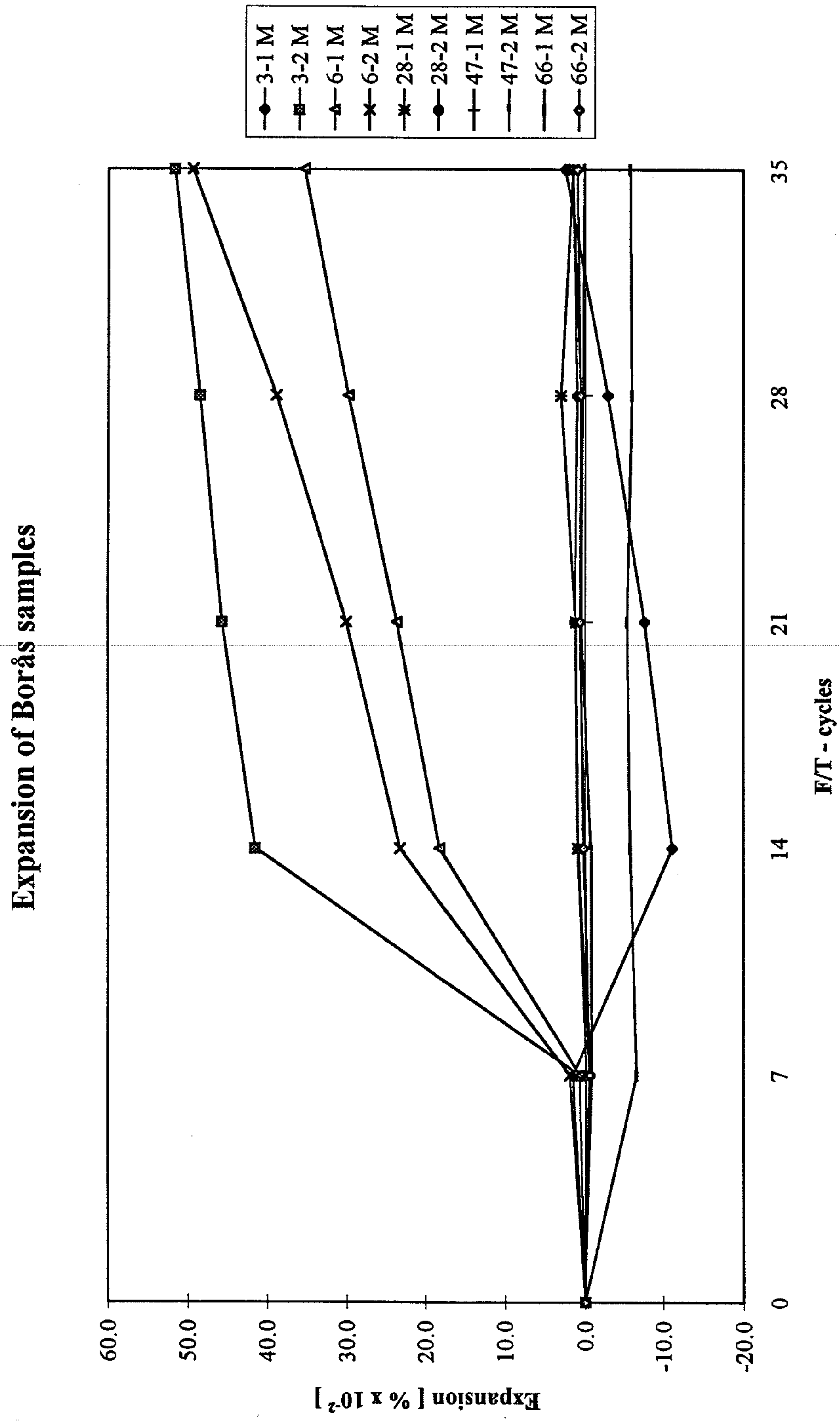


Figure 3.4.3.Horizontal length change during freeze/Thaw testing, measured 25 mm from the surface (~middle) of the Borås samples.

It appears that the frost scaling resistance correlates to the internal damage as measured by length change, see table 3.4.2. However, it is emphasized that the tested samples represent 'end members' in frost resistance. Hence, from a visual impression of the tested samples, the HUA samples largely show no effect of the Borås testing, whereas the DBT samples are completely disintegrated.

Table 3.4.2 Surface scaling versus horizontal expansion for the 4 concrete types

Mix no. and Treatment[1]	Specimen no.	Scaling after 35 freeze/thaw-cycles [kg/m^2]	Horizontal length change (expansion) 5 mm from top [% x 10^{-2}]	Horizontal length change (expansion) 25 mm from top [% x 10^{-2}]
DBT-1 (D1/S1/T1)	3-1 3-2	3,64 2,64	16.3 51,5	2,6 51,6
DBT-2 (D1/S1/T1)	6-1 6-2	4,75 4,72	47,0 47,5	35,4 49,4
HUA-5 (D1/S1/T1)	28-1 28-2	0,04 0,05	-0,8 5,1	1,5 1,6
HUA-3 (D1/S1/T1)	47-1 47-2	0,01 0,00	1,2 2,1	1,6 0,1
HUA-3 (D1/S1/T2)	66-1 66-2	0,10 0,04	2,6 4,1	-5,7 1,0

Dynamic E-Modulus Changes, determined by Ultra Sonic Measurements

The ultra sound equipment was tested on a concrete dummy with gauge studs, in order to test the accuracy of the measurements. Two types of ultra-sound heads were applied. The flat ultra-sound head gave values varying 5-10 % during one reading (over 5-10 seconds). Repeated measurements gave similar variations. In contrast it was found that the pointed ultrasound-heads gave a very low standard deviation, see table 3.4.3. Consequently, the pointed ultra-sound heads were chosen for the test series.

Table 3.4.3 Ultra sound measurements on concrete dummy. Standard deviations (in pct.) at repeated measurements.

	Flat US-head (~50 mm diameter)	Pointed US-head (~5 mm diameter)
Handheld	5.6 %	0.2 %
Supported	5.7 %	0.3 %

The measurements during the freeze/thaw-series gave unacceptably high variations, in the range of 10-30%. This was obviously caused by unwanted contact between ultra-sound heads and the isolation material covering the sides of the samples - a condition that could not at the time be changed. Hence it was decided not to carry out ultra sound testing during the freeze/thaw-testing, but only at the end of the period.

The results of the ultra-sound testing at the end of the freeze/thaw-testing is given in table 3.4.4.

Table 3.4.4 Ultra sound measurements of the Borås samples after testing.

Sample no.	3-1	3-2	6-1	6-2	28-1	28-2	47-1	47-2	66-1	66-2
Mix no.	DBT-1	DBT-1	DBT-2	DBT-2	HUA-5	HUA-5	HUA-3	HUA-3	HUA-3	HUA-3
Ultra sound signal (relative)	-	-	-	-	55.3	54.5	55.5	55.9	55.6	57.2

As can be seen from table 3.4.4, the values of the HUA-3 concrete and the HUA-5 concrete varies only a little.

No measurements of the DBT-1 and the DBT-2 samples were made, since the sides of the concrete surfaces were markedly damaged at upper 5-30 mm. Furthermore, the gauge studs of these samples fell off during the removal of the isolation.

The ultra-sound measurements were meant to be a support of the measurements of length change. The problems with the carrying out the ultra-sound measurements as described above, indicate that this may not be a fruitful method.

Microstructural Analysis, by Thin Section Petrography

The petrographic analysis of thin sections were carried out on concrete prisms exposed and treated identically to the Borås samples. The visual impression of the prisms and of the corresponding Borås samples, showed similar degree and type of damage.

The reporting of the analyses is given in Appendix 9.

Conclusion of the Measurements of Internal Damage in Borås Specimens

The internal damage, as seen by length change measurements and by thin section analysis, correlate well to the surface scaling, see figure 3.4.2. The expectedly highly frost resistant concretes showed little or no scaling, no length change and no change of microstructure. The expectedly low frost resistant concretes showed large scaling, marked length change both at depths of 5 mm and 25 mm from the exposed surface, and also showed marked change of microstructure.

It is, however, not known if the observed clear correlations of scaling and internal damage holds for concretes with intermediate frost resistance.

The results of this part of the research differs from observations of in-situ structures, often showing internal delamination without marked damage of the surface.

It may thus be questioned if the Borås testing can be applied for measurements of internal damage.

The coming work on calibration of the test methods should imply measurements of possible internal damage in Borås test specimens. The methods recommended are length change during the testing, and petrographic analysis of the samples after testing.

4 Temperature Induced Stresses during Heating of Frozen Test Specimens

4.1 Introduction

As discussed in the State-of-the-Art report [Laugesen et al., 1996] the existing frost damage models predict that the frost damage occurs when the water in the pores freezes. However, the observation in practice sometimes shows that the damage occurs during the thawing of the concrete or just before the thawing, when the temperature increases again. The damages are typically found after a winter with 10-15°C freezing or lower.

This chapter describes and develops an alternative model, which explains this phenomenon and eliminates the discrepancies in the existing models.

4.2 Discussion of Freeze/Thaw Damage Mechanisms

Freeze-thaw damages are due to a number of different damage mechanisms, of which a general description can be found in 13 Betonsygdomme (13 Concrete diseases). A more through literature review is presented in the State-of-the-Art report.

The damages are usually expected to be created during freezing due to the expansion of the water in the pores, cavities and cracks as water expands 9% in volume during freezing.

Frost damages are therefore to be expected in concrete, mortar or cement paste, where the pores are more than 90% filled with water. Actual tests with concrete samples shows that the critical water content is 60-90% of the saturated content [Beton-Bogen] and that this depends on the concrete type.

Frost resistance can be tested in the laboratory on mortar prisms, which are subjected to one or several freeze-thaw cycles. The mortar prism expansion is measured and considered as a result of crack formation. The crack formation and the expansion is usually found during the cooling of the prism, however, some results [Xiaofang and Mungshu, 1996] have shown that the expansion and crack formation can be found during the stage where the temperature of the prisms increases.

These observations cannot be explained by the existing freeze-thaw mechanisms so an alternative or supplementary damage mechanism is needed. Such a mechanism was presented by Thaulow and Geiker in Lund April 1996 [Thaulow and Geiker, 1996].

4.3 Alternative Model of Damage Mechanism

Basic Assumptions

Ice is in the following considered to be an elastic material

The temperature in a structure or in a prism will start to decrease as the temperature in the environment decreases, starting with the layers near the surface.

The water in the larger pores freezes and expands 9% during this freezing that causes stresses in the matrix if the water in the pores exceeds 90% of the pores' volume.

During lowering of the temperature below the freezing point, the ice in a pore will contract more than the volume of the pore, since the thermal expansion coefficient of ice is approximately 5 times that of the cement paste, mortar or concrete. The ice may therefore attract non-frozen water from the surrounding, smaller pores and fill out the pore completely at the time, where the lowest temperature is reached. This ice formation will not lead to any stresses at the lowest temperature, since the ice has only been formed in the space between the ice and the pore wall (in this argument possible ice-lens growth against a pressure is not considered).

The ice will, however, expand more than the surrounding paste/mortar/concrete when the temperature is increased and will act as a solid expanding particle until the melting temperature has been reached.

Critical Zone

The thawing front will propagate through the concrete, from the surface and towards the interior. The front will be more or less parallel to the surface, but will have some local variations, due the inhomogeneties in the material. The thawing front or some of the thermal contour curves will at a critical stage be in front on the aggregates and mainly pass through the paste.

Two types of damage types will need to be considered:

1. Creation of cracks through the cement paste as a result of the pressure of the ice in the pores

2. Creation of cracks between the aggregates and the cement paste

The stresses at this stage in the cement paste will depend on the properties of the ice and the cement paste and not on the properties of the concrete. This is due to the fact that the ice in the pore's presses on the pore walls in the cement paste and that the dimension of the pores is significantly different from the dimension of the aggregates.

The stresses in a material from a pressure in a pore decreases rapidly with increasing distance [Goltermann, 1994] and the pore ice pressure can therefore only act indirectly on the aggregates.

The frozen cement paste (cement paste plus ice in the pores) expands more than the aggregates when the temperature increases, which lead to contact stresses between the paste and the aggregate. These stresses are largest at the interface between the aggregate and the material and may lead to spalling of the concrete.

Composite Model

The behaviour of the ice and the cement paste can be described by a composite model, where the ice in the pores is treated as particles in a matrix (the cement paste). The behaviour of the ice in the pores, the cement paste and the aggregates can also be described by a composite model, where the aggregates are treated as particles in a matrix (cement paste and ice).

Ice and Cement Paste

The tensile stresses and strains in the paste can be estimated on the basis of a composite model, where the composite consists of a matrix (cement paste) and some inclusions or particles (ice):

$$\epsilon_{ct} = \Delta\epsilon \, / \, ((1.5 * E_c/E_i + 0.5)(1 - p) \, / \, 3p + (1 + 2p) \, / \, 3p)$$

where Poisson's ratio has been set to 0.2 and where:

$\Delta\epsilon$	is the difference between the free thermal expansions in the particle (ice) and in the matrix (cement paste)
E_c	is the modulus of elasticity of the matrix (cement paste)
E_i	is the modulus of elasticity of the particle (ice)
p	is the relative volume of the particle in the composite

The difference between the free thermal expansions in the particle and in the matrix is estimated as:

$$\Delta\epsilon = (\alpha_i - \alpha_c)^* T$$

where

α_i	is the thermal expansion coefficient of the particle (ice)
α_c	is the thermal expansion coefficient of the matrix (cement paste)
ΔT	is the increase in temperature of the system

The thermal expansion coefficient of the ice and cement paste together (α_{ip}) can be estimated for the composite by the above formulas as:

$$\alpha_{ip} = (\alpha_i - \alpha_c) / ((1.5^*E_c/E_i+0.5)(1 - p) / 3p + (1 + 2p) / 3p) + \alpha_c$$

The modulus of elasticity for the frozen cement paste (ice and cement paste) E_{ip} can also be estimated [Beton-Bogen] as:

$$E_{ic} = E_c^*(1 - p) + E_i^*p$$

since $E_c > E_i$.

Ice, Cement Paste and Aggregates

The tensile stresses and strains in the interface between the paste and the aggregates can be estimated on the basis of a composite model, where the composite consist of a matrix (ice plus cement paste) and some inclusions or particles (aggregates) in exactly the same way as in the above clause.

The tensile strain (ϵ_{it}) in the interface can be estimated [Xiaofang and Mungshu, 1996] as:

$$\epsilon_{it} = \Delta\epsilon / ((1.5^*E_{ic}/E_a+0.5)(1 - a) / 3a + (1 + 2a) / 3a)$$

where Poisson's ratio has been set to 0.2 and where:

$\Delta\epsilon$	is the difference between the free thermal expansions in the particle (ice) and in the matrix (cement paste)
E_{ic}	is the modulus of elasticity of the matrix (ice and cement paste)
E_a	is the modulus of elasticity of the particle (aggregates)
a	is the relative volume of the aggregates in the composite

The difference between the free thermal expansions in the particle and in the matrix is estimated as:

$$\Delta\epsilon = (\alpha_{ic} - \alpha_a) * \Delta T$$

where

α_a	is the thermal expansion coefficient of the particle (aggregate)
α_{ic}	is the thermal expansion coefficient of the matrix (ice plus cement paste)
ΔT	is the increase in temperature of the system

4.4 Material Parameters

The relevant material parameters for cement paste and for ice will be discussed in the following and the values to be used in the estimations will be determined.

Cement Paste Parameters

Pore Volumen

The freezeable water in the concrete can be situated in the capillary pores and in the air voids. The volume of this can thus not exceed the total porosity minus the gel porosity and plus the volume of natural air.

The total porosity (gel pores plus capillary pores) of hardened cement paste (p_p) can be estimated as [Nielsen, 1993]:

$$p_p = (W/C - 0.18q) / (W/C + 0.32) \quad \text{for } W/C > 0.38$$

where

W/C	is the water/cement ratio
q	is the degree of hydration

The volume of gel pores (p_g) is proportional to the amount of reacted cement [Beton-Bogen]:

$$p_g = 0.15*q / (W/C + 0.32)$$

The volume of natural air in a normal concrete, which have been fairly well vibrated, is in the range of 1-2%. This corresponds to a content of natural air in the cement paste in the range of

p_{an} = 6%

when the concrete contains 25-30% cement paste by volume.

Modulus of Elasticity

The modulus of elasticity of a cement paste with W/C=0.40 can be estimated as [Nielsen, 1993]:

E_c = 32.000*q Mpa

Tensile Strain Capacity

The paste will crack, when the tensile stresses exceeds the ultimate tensile strength, or when the tensile strain exceeds the ultimate tensile strain capacity (ϵ_{tu}).

The tensile strength is commonly used to describe the materials resistance against tension. This strength does, however, depend on a number of parameters and has a rather large variation and is therefore not quite suitable for our purpose. The ultimate strain capacity is as much better measure as this value is fairly constant and can be estimated as the ratio of the tensile strength and the modulus of elasticity.

A value of the ultimate tensile strain capacity of 0.01-0.02% is usual in concrete [Beton-Bogen]. A similar value can be expected for the cement paste as the modulus of elasticity of the cement paste corresponds fairly well with the modulus of the cement paste.

Tests on prisms with fibre reinforced cement paste [Dela, 1995] indicate a strain capacity of approximately 0.02%.

The value of the ultimate strain capacity of the cement paste is therefore expected to be in the range of:

ϵ_{tu} = 0.01% - 0.02%

Thermal Expansion Coefficient

The thermal coefficient of cement paste is approximately the same as for concrete [Teknisk Ståbi, 1986]:

$\alpha_c \quad = 10*10^{-6}/^{o}C$

Ice Parameters

Modulus of Elasticity

The modulus of elasticity of ice is somewhat complicated as the value depends on temperature and pressure [Eisenberg and Kauzmann, 1969]. The value usually measured is a furthermore a one-dimensional value and the equally important value of Poisson's ratio is often not measured. The best value will thus be the volumetric modulus of elasticity, corresponding to the stiffness at a hydrostatic loading. The effect of creep is ignored as no values can be found for creep of ice exposed to a hydrostatic loading.

The value used in our estimations is found in [Nielsen, 1993]:

$E_i \quad = 10000$ MPa

This value were in the SBI-publication [Nielsen 1993] used for prediction of eigenstresses due to ice-formation and should therefore be fairly correct.

Thermal Expansion Coefficient

The thermal coefficient of ice depends on the temperature [Eisenberg and Kauzmann, 1969] but will at approximately -10°C [Kirk-Othmer, 1984] be

$\alpha_i \quad = 52.7*10^{-6}/^{o}C$

Note that this value is about 5 times higher than the value for cement paste.

Aggregate Parameters

Modulus of Elasticity

The modulus of elasticity depends on the type of aggregate [Beton-Bogen] but will typically be

E_a = 40-50000 MPa

Thermal Expansion Coefficient

The thermal coefficient of the aggregate depends on the aggregate type [Beton-Bogen] but will for most granite be:

$\alpha_a \quad = 5*10^{-6}/°C$

4.5 Estimation of Tensile Strains

The tensile strains must be estimated for the two critical cases mentioned in clause 3.2; that is in the cement paste and in the interface between the paste and the aggregates.

Tensile Strains in the Cement Paste

The tensile strains can be calculated for a cement paste with a water/cement ratio of 0.42 and a degree of hydration of 80%. The cement paste is subjected to a freeze-thaw cycle, which included a lowering of the temperature to 10°C below the freezing point.

The porosity of the paste is estimated as:

$$p_p = (W/C - 0.18q) / (W/C + 0.32)$$

$$= (0.42 - 0.18*0.8) / (0.42+0.32) \quad = 37\%.$$

$$p_g = 0.15*q / (0.32 + W/C)$$

$$= 0.15*0.8/(0.42 + 0.32) \quad = 15\%$$

The volume with freezeable water (p) is then found as:

$$p = p_p - p_g + p_{an} = 37\% - 15\% + 6\% \quad = 28\%$$

The modulus of elasticity of the cement paste is estimated as:

$$E_c = 32.000*q \quad = 32000*0.8 \quad = 25600 \text{ Mpa}$$

The thermal strain difference is estimated as:

$$\Delta\epsilon = (\alpha_i - \alpha_c)* \Delta T = (52.7-10)*10^{-6}*10 \quad = 427*10^{-6}$$

after which the tensile strain is estimated as:

$$\epsilon_{ct} = \Delta\epsilon \,/\, ((1.5{*}E_c/E_i+0.5\,){*}(1 - p) \,/\, 3p + (1 + 2p) \,/\, 3p)$$

$$= 427/((1.5{*}25600/10000+0.5){*}(1\text{-}0.28)/3/0.28$$

$$+ (1+2{*}0.28)/3/0.28) \qquad = 77{*}10^{-6} \quad = 0.008\%$$

The expected ultimate strain capacity is 0.01% to 0.02%. This means that an increase of the temperature of 13 to 25°C can be sufficient to create frost damages in frozen concrete.

Tensile Strains in the Aggregate Interface

The thermal expansion coefficient of the frozen cement paste (ice+cement paste) is estimated as

$$\alpha_{ip} = (\alpha_i - \alpha_c) \,/\, ((1.5{*}E_c/E_i+0.5)(1 - p) \,/\, 3p$$

$$+ (1 + 2p) \,/\, 3p) + \alpha_c$$

$$= (52.7\text{-}10)/\,((1.5{*}25600/10000+0.5){*}(1\text{-}0.28)$$

$$/3/0.28 + (1+2{*}0.28)/3/0.28) + 10 \qquad = 17.7{*}10^{-6}/^{\circ}C$$

This leads for a concrete which consists of 30% paste and 70% granite aggregates by volume (a = 70%) to the thermal strain difference:

$$\Delta\epsilon = (\alpha_{ic} - \alpha_a){*}\,\Delta T \quad = (17.7 - 5){*}10^{-6}{*}10 \qquad = 127{*}10^{-6}$$

The modulus of elasticity for the frozen cement paste (ice+cement paste) is estimated as:

$$E_{ic} = E_c{*}(1 - p) + E_i{*}p$$

$$= 25600{*}(1 - 0.28) + 10000{*}0.28 \qquad = 21230\text{MPa}$$

and the tensile strain in the interface estimated as:

$$\epsilon_{it} = \Delta\epsilon \,/\, ((1.5{*}E_{ic}/E_a+0.5)(1 - a) \,/\, 3a + (1 + 2a) \,/\, 3a)$$

$$= 127{*}10^{-6}/((1.5{*}21230/45000 + 0.5)(1 - 0.70)/(3{*}0.70)$$

$$+ (1 + 2*0.70)/(3*0.70)) \quad = 97*10^{-6} \quad = 0.0097\%$$

The expected ultimate strain capacity of concrete /A/ is 0.01% to 0.02%. This means that an increase of the temperature of 10 to 20°C can be sufficient to create frost damages in frozen concrete.

4.6 Conclusions

The model's predictions of tensile strains/stresses correspond to the assumed tensile strain capacity in the cement paste. The model predicts potential frost damages at frost temperatures of the order 10-20°C.

The model explains - in contract to other models - why frost damages can occur at increasing temperatures, which correspond to observations in practice and the tests in a Swedish laboratory.

4.7 Implications on Test Methods

The analyses of the new supplementary damage mechanism indicate that the minimum freezing temperature during freeze-thaw testing should be in the range of -10 °C to -20 °C. This is satisfied for both the Borås method and the ASTM C666.

4.8 Symbols

E_a	is the modulus of elasticity of the aggregates
E_c	is the modulus of elasticity of the cement paste
E_i	is the modulus of elasticity of the particle (ice)
E_{ip}	is the modulus of elasticity of the ice and cement paste together
p	is the relative volume of the particle in the composite (pore with ice)
q	is the degree of hydration
ΔT	is the increase in temperature
W/C	is the water/cement ratio
α_a	is the thermal expansion coefficient of the aggregates
α_c	is the thermal expansion coefficient of the cement paste
α_i	is the thermal expansion coefficient of the ice
α_{ip}	is the thermal expansion coefficient of the ice and cement paste together
ϵ_{ct}	is the tensile strain in the cement paste
$\Delta\epsilon$	is the difference between the free thermal expansions in the ice and in the cement paste

5 Proposed Test Methods

5.1 Introduction

According to the contract with The Danish Road Directorate the task 2: "Test Methods for determining the Frost Resistance of High Performance Concrete" was initiated by the preparation of a State-of-the-Art Report [Laugesen et al, 1996]. Based on the outcome supplementary research was recommended within the limits of the contract [Laugesen, 1996]. The supplementary research is documented in the former chapters and is concluded in the present preliminary method description.

This method description is proposed as valid for the next part of the project: "Afprøvning og kalibrering" [testing and calibration) on existing structures!

The State-of-the-Art report concluded that two test methods are needed for describing freeze/thaw deterioration, one causing scaling and one causing internal cracking. This is in accordance with CEN TC 51/WD 12/TG 4.

The following methods are proposed used, taking into account the below mentioned proposed revisions:

* Scaling: SS 13 72 44
* Internal cracking: ASTM C 666

The proposed revisions are preliminary and is to be revised based on the outcome of the remaining activities of Task 2.

It should be noted that further documentation of the revised methods is anticipated necessary before applying them in general.

5.2 Proposed Revisions to SS 12 72 44, Borås Method to be Tested

Reference is given to edt. 3 of 1995-03-08.

Re: "3 Utrustning, Gummiduk"

Add: "The rubber cloth shall be dense or sealed, as to no condensated water is absorbed from the protection against evaporation."

Re: "5 Förbehandling"
Investigations on cement paste shows that drying may increase the amount of freezable water. It is not clear whether concrete is affected to the same extent, nor is the extent of drying in-situ known.

Assuming the amount of freezable water is not altered by drying, the moisture content is the determining factor for the occurence of freeze/thaw damage. Measurements of weight change during conditioning and testing has been performed as part of the supplementary research. The severely dried (50°C, low RH, 14 days) samples (w/c=0.39) did not absorbe as much water during re-saturation (14 days) and testing (28 days) as lost during drying. However, the rate of absorption indicates that the water content after 56 freeze/thaw cycles may be higher than the initial.

The following alternative method of conditioning will be tested as part of activity 3.4 of the project:

- After sawing, the specimens are placed in an oven at laboratory temperature. The temperature is increase by 10°C/h to 50°C, and the specimens are stored for 14 days in the oven at 50 +/- 2 °C and 10 +/- 5 %RH.

- After the 14 days the specimens are cooled to 20°C, and the rubber cloth is glued to all surfaces of the specimens except the surface to be tested. The edge of the rubber cloth is to reach 20 +/- 5 mm above the level of the test surface. The glue is added evenly to the surfaces of both the specimen and the rubber cloth. Glue is also placed at the circumference, over the joint between the specimen and the rubber cloth.

- Tap water (20 +/-2 °C) is poured on the test surface to a height of about 5 mm. 14 days +/- 2 hours after this, the frost test begins.

Re: "6.2 Metod A. Provning med Saltoplösning"
It may be that both scaling and internal cracking can be tested by the Borås method. The following is therefore to be performed as part of activity 3.4:

Add p. 7, before line 8 from bottom
"- Measure length change...". Reference is given to method proposed by Peter Laugesen.

Re: "7 Resultat"
Add: "Data on the coefficient of variation is given in Figure 8." (Figure 4 from Petersson, P.E. amd Lundgren, M. [1996]: Influence of the minimum temperature on the scaling resistance of concrete". Proceedings, 7th Int. Conf. on Durability of Building

Materials and Compenents, Stockholm. - Or, prefarably, the data given on a log (scaling) scale).

It is expected that European research project will provide more information on the accuracy of the method.

Re: "8 Rapport"

Add: "The actual temperature of testing is to documented."

It may be that the temperature of the exposure liquid on a specific sample is influenced by the amount of freezing taking place in the sample. Therefore, as part of activity 3.4 of the project the following will be measured:

* Temperature profile during testing
* Temperature in the exposure liquid on specimens differing in amount of freezable water.

5.3 Proposed Revisions to ASTM C 666 Procedure A to be Tested

Reference is given to edt. -92.

Re: "4.4"

"Optional Length Change Test Length Change Comperator"
is changed to
"Length Change Test Length Change Comperator"

Re: 7.2, line 4

Add: "The diameter of cores taken from structures is 100 +/- 5 mm, length approximately 200 mm. The cores are washed for sludge, dried with a cloth and wrapped as to secure against drying out. The cores are cut to a length of 150 +/- 20 mm."

Re: 8.2, line 6 and 7... fundamental transverse frequency and measure length change (optional) with the ..."
is changed to
"... fundamental transverse frequency and measure length change with the ..."

Re: 10.6.1, line 6

Add: "The durability factor is not reported for 150 mm length cores taken from structures."

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7 Appendices

1 Concrete mix compositions
2 Preparation and numbering of specimens
3 Time table
4 Determination of moisture and degree of saturation profiles, method
5 Desorption and adsorption during drying, re-saturation, and frost testing
6 Moisture and degree of saturation profiles
7 Scaling during frost testing
8 Chloride profile
9 Petrographic analysis

Appendix 1

Concrete Mix Composition

Receptoversigt

RECEPT NR.	BESKRIVELSE	V/C	SÆTMÅL mm	LUFT %	TILSÆTNINGSSTOFFER LEVERANDØR	LUFT	PLAST	SUPERPLAST	TRYKSTYRKE MPa	BEM.
1	GBC Type A (standard)	0,347	150	5,5	NBK	Amex SB	Additon P	Melment L10	45,0	Referencebeton
2	ASØ (Øresundsforbindelsen) Type SA	0,392	100	5,6	FOSROC	316 AEA	Plast 212	Peramin F	40,0	Referencebeton
2 M	ASØ Type SA (med reduceret vandindhold)	0,365	100	5,6	FOSROC	316 AEA	Plast 212	Peramin F	40,0	
3	VD Type A (med øget V/C-forhold)	0,448	100	5,5	FOSROC	316 AEA	Plast 212		35,0	Referencebeton
4	Svensk Anlægscement	0,392	150	5,5	NBK	Amex SB	Additon P	Melment L10	45,0	
4 A	Svensk Anlægscement og 3 % mikrosilika	0,391	150	5,5	NBK	Amex SB	Additon P	Melment L10	45,0	
6	Hollandsk Slaggecement	0,368	150	5,5	NBK	Amex SB	Additon P	Melment L10	45,0	
6 A	Hollandsk Slaggecement og 3 % mikrosilika	0,368	150	5,5	NBK	Amex SB	Additon P	Melment L10	45,0	
14	UNICON (moderat miljøklasse)	0,480	100	5,5	FOSROC	316 AEA	Plast 212		30,0	
16	Recept 18 med Heidelberg Kugler	0,392	150	5,5	NBK	Amex SB	Additon P	Melment L10	45,0	Heidlb. Kugler
18	GBC Type B uden vandrestriktioner	0,392	150	5,5	NBK	Amex SB	Additon P	Melment L10	45,0	
21	GBC Type B, uden flyveaske og vandrestriktioner	0,391	150	5,5	NBK	Amex SB	Additon P	Melment L10	45,0	8 % mikrosilika
22	GBC Type B m. SIKA-produkter	0,392	150	5,5	SIKA	FRO V5A		Sikament 110U	45,0	
23	GBC Type B m. SIKA-produkter (inkl. Sikapump)	0,392	150	5,5	SIKA	FRO V5A		Sikament 110U	45,0	Sikapump
28	Som recept 18, men med 8,5 % luft	0,392	150	8,5	NBK	Amex SB	Additon P	Melment L10	45,0	

Betonsammensætning (proportioneret)

RECEPT NR.	BESKRIVELSE	CEMENT TYPE	CEMENT INDHOLD kg/m³	FLYVEASKE kg/m³	OPSLEMMET MIKROSILIKA kg/m³	VAND (FRIT) kg/m³	LUFT-INDBLANDING kg/m³	PLAST kg/m³	SUPER-PLAST kg/m³	ANDET kg/m³
1	GBC Type A (standard)	PC (A/HS/EA/G)	320,0	47,0	40,0	133,0	1,40	1,60	7,60	
2	ASØ (Øresundsforbindelsen) Type SA	PC (A/HS/EA/G)	285,0	60,0	24,0	133,0	0,36	1,43	2,88	
2 M	ASØ Type SA (med reduceret vandindhold)	PC (A/HS/EA/G)	285,0	60,0	24,0	124,0	0,36	1,43	2,86	
3	VD Type A (med øget V/C-forhold)	PC (A/HS/EA/G)	265,0	30,0	48,0	146,8	0,90	2,00		
4	Svensk Anlægscement	PC (A/HS/EA/G)	383,0			150,0	0,50	1,00	4,00	
4 A	Svensk Anlægscement og 3 % mikrosilika	PC (A/HS/EA/G)	361,0		22,3	150,0	0,50	1,00	4,00	
6	Hollandsk Slaggecement	~ 77 % SLAGGE	408,0			150,0	0,40	1,60	4,00	
6 A	Hollandsk Slaggecement og 3 % mikrosilika	~ 77 % SLAGGE	384,0		23,8	150,0	0,40	1,60	4,00	
14	UNICON (moderat miljøklasse)	PC (A/HS/EA/G)	285,0	65,0		152,0	0,90	2,50		
16	Recept 18 med Heidelberg Kugler	PC (A/HS/EA/G)	320,0	60,0	33,0	150,0	0,60	1,00	4,00	3,34
18	GBC Type B uden vandrestriktioner	PC (A/HS/EA/G)	320,0	60,0	33,0	150,0	0,80	1,00	4,00	
21	GBC Type B, uden flyveaske og vandrestriktioner	PC (A/HS/EA/G)	327,0		57,0	150,0	0,60	1,00	4,00	
22	GBC Type B m. SIKA-produkter	PC (A/HS/EA/G)	320,0	60,0	33,0	150,0	0,60		4,50	
23	GBC Type B m. SIKA-produkter (inkl. Sikapump)	PC (A/HS/EA/G)	320,0	60,0	33,0	150,0	0,60		4,50	2,60
28	Som recept 18, men med 8,5 % luft	PC (A/HS/EA/G)	320,0	60,0	33,0	150,0	1,40	1,00	4,00	

Ovennævnte betonsammensætning blev for tilsætningsstoffernes vedkommende ændret under støbningerne for at opnå det tilstræbte sætmål og luftindhold.

OPGAVE 13 A — PRØVESTØBNINGER I KALUNDBORG

RECEPT NR. 01 — FORSØGSDATO: 1994.10.21

OPDATERET : 1994.10.27

FØLGE SEDDEL	LAB. NR.	LÆS TYPE	TIDS-PUNKT	PRØVE FØR/EFTER PUMPE	SÆTMÅL mm	UDBREDELSESMÅL 1 mm	UDBREDELSESMÅL 2 mm	BETON-TEMP. °C	LUFT %	DENSITET kg/m³	DBT AIR-VOID MÅLER < 2 mm %	DBT AIR-VOID MÅLER < 0,35 mm %	DBT AIR-VOID MÅLER SPEC. OVERFL. mm⁻¹	DBT AIR-VOID MÅLER AFST. FAKTOR mm	BEMÆRKNINGER
406107		B1	9:15												Blandetidspunkt
406107	1	B1	9:20	FØR	180	520	510	14	7,6	2299	7,2	6,5	32,1	0,11	10 stk cylindre 150/300 mm udstøbt (6 stk til trykstyrkebestemmelse)
406107	2	B1	10:00	EFTER	105	440	420	14	4,8	2387					
406107	3	B1	10:20	EFTER	90	380	380	14	6,6	2344	5,1	3,9	27,1	0,17	Prøve udtaget umiddelbart efter udstøbning af dækpladen
406107	3A	B1	10:49								5,9	3,6	23,3	0,19	Prøve udtaget fra dækplade efter udstøbning og vibrering
406107	4	B1	11:07	EFTER	70	350	360	13	5,8	2356					
406117		B2	11:47												Blandetidspunkt
406117	5	B2	11:55	FØR	125	400	450	14	6,9	2320	8,1	6,4	30,7	0,11	
406117	6	B2	12:15	EFTER	110	360	370	15	4,3	2393	4,5	3,9	31,2	0,16	
406117	8	B2	13:05	EFTER	80	380	380	15	6,6	2343					
406118		B3	13:29												Blandetidspunkt
406118	9	B3	13:37	FØR	180	420	430	15	7,3	2307	7,7	7,2	32,4	0,11	
406118	10	B3	14:09	EFTER	130	390	420	15	6,9	2332	7,2	6,5	30,6	0,12	
406118	11	B3	14:29	EFTER	120	360	380	14	6,2	2330					
406118	12	B3	15:08	EFTER	110	400	390	15	7,0	2319					
406119		B4	15:09												Blandetidspunkt
406119	13	B4	15:17	FØR	190	440	480	14	8,7	2257	9,7	9,1	37,4	0,07	
406119	14	B4	15:43	EFTER	100	370	360	12	4,2	2402					

EVENTUELLE BEMÆRKNINGER :

BETON TRYKSTYRKER

ALDER dage	CYLINDER VÆGT gr.	BETON-DENSITET kg/m3	TRYK-STYRKE MPa	BEMÆRKNINGER
4	12425	2344		
4	12390	2337		
7	12450	2348		
7	12480	2354		
28	12520	2362		
28	12490	2356		

OPGAVE 13 A — PRØVESTØBNINGER I KALUNDBORG

RECEPT NR. 21 — FORSØGSDATO: 1994.09.28

OPDATERET : 1994.10.27

FØLGE SEDDEL	LAB. NR.	LÆS TYPE	TIDS-PUNKT	PRØVE FØR/EFTER PUMPE	SÆTMÅL mm	UDBREDELSESMÅL 1 mm	UDBREDELSESMÅL 2 mm	BETON-TEMP. °C	LUFT %	DENSITET kg/m³	DBT AIR-VOID MÅLER < 2 mm %	DBT AIR-VOID MÅLER < 0,35 mm %	DBT AIR-VOID MÅLER SPEC. OVERFL. mm⁻¹	DBT AIR-VOID MÅLER AFST. FAKTOR mm	BEMÆRKNINGER
307792		B	9:33												Blandetidspunkt
307792	1	B	9:36	FØR	145	440	460	17	6,4	2316	5,7	5,5	44,8	0,10	10 stk cylindre 150/300 mm udstøbt (6 stk til trykstyrkebestemmelse)
307792	2	B	10:10	EFTER	105	390	400	18	6,5	2333	6,3	5,6	29,9	0,14	
307792	2A	B	11:11												Prøve udtaget fra dækplade efter udstøbning og vibrering
307792	3	B	10:24	FØR	120	430	450	16	7,4	2276					
307792	5	B	11:00	FØR	85	370	370	18	9,0	2248					
307792		B	11:13												Efterdocering med superplast (4,5 kg i ca. 3 m3 beton)
307792	6	B	11:17	FØR	125	450	430	17	11,0	2201	8,7	8,6	44,3	0,07	
307792	9	B	11:41	FØR	80	350	360	17	10,5	2194					
307792	10	B	12:02	EFTER	50	330	330	16	8,3	2238					
307793		A	10:36												Blandetidspunkt
307793	4	A	10:53		130	470	460	16	7,8	2296					
307793	7	A	11:52		100	400	390	18	8,6	2248					
307793		A	12:00												Efterdocering med superplast (6,75 kg i ca. 4,5 m3 beton)
307793	8	A	12:08		110	450	450	17	9,3	2252					
307793	11	A	13:00	EFTER	50	330	330	17	7,3	2301					
307793	12	A	13:20		85	350	350	18	10,0	2199					

EVENTUELLE BEMÆRKNINGER :

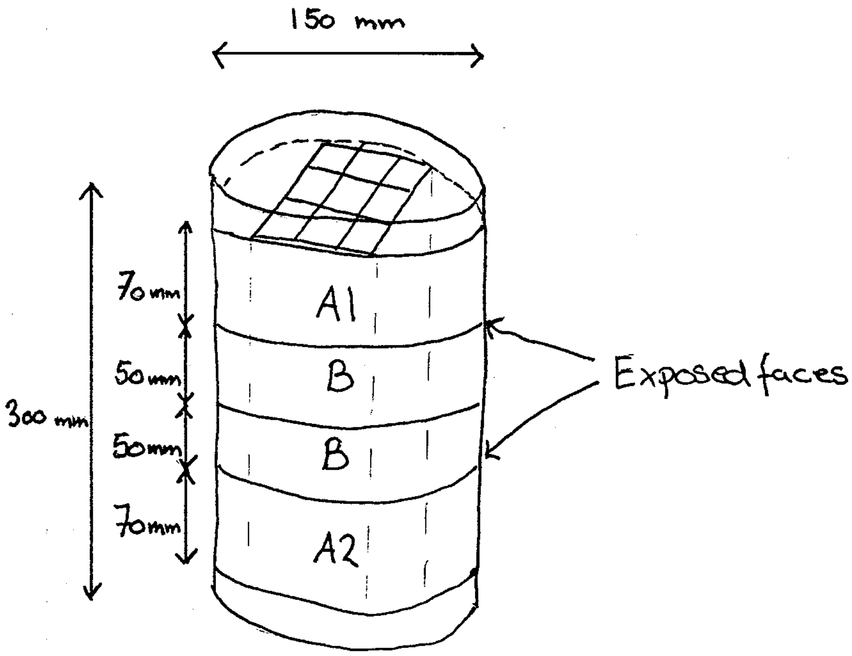
BETON TRYKSTYRKER

ALDER dage	CYLINDER VÆGT gr.	BETON-DENSITET kg/m3	TRYK-STYRKE MPa	BEMÆRKNINGER
3	12475	2353	27,0	
3	12365	2332	27,2	
7	12475	2353	36,7	
7	12390	2337	35,9	
28	12400	2339	49,7	
28	12420	2343	52,3	

Content (kg)	DBT I	DBT II
Low alkali Cement	330	330
Silica fume	0	16,5
Water	149	163
Plastifizicer	2	1
Airentraining	0	0
Sand	800	800
Coarse aggregate	1123	1066
Total	2404	2377
Equivalent W/C ratio	0,45	0,45

Appendix 2

Preparation and numbering of specimens



A Prisms

B Separate specimen for Borås testing

Numbering of prisms:

HUA-3

42

1	5	2
6	9	7
3	8	4

Corner 1 - 4
Side 5 - 8
Middle 9

10	14	11
15	18	16
12	17	13

10 - 13
14 - 17
18

47

19	23	20
24	27	25
21	26	22

19 - 22
23 - 26
27

28	32	29
33	36	34
30	35	31

28 - 31
32 - 35
36

66

37	41	38
42	45	43
39	44	40

Corner 37 - 40
Side 41 - 44
Middle 45

46	50	47
51	54	52
48	53	49

46 - 49
50 - 53
54

70

55	59	56
60	63	61
57	62	58

55 - 58
59 - 62
63

100	104	101
105	108	106
102	107	103

100 - 103
104 - 107
108

HUA-5

28

64	68	65
69	72	70
66	71	67

Corner 64 - 67
Side 68 - 71
Middle 72

73	77	74
78	81	79
75	80	76

73 - 76
77 - 80
81

DBT-1

3

82	86	83
87	90	88
84	89	85

Corner 82 - 85
Side 86 - 89
Middle 90

91	95	92
96	99	97
93	98	94

91 - 94
95 - 98
99

DBT-2

6

109	113	110
114	117	115
111	116	112

Corner 109 - 112
Side 113 - 116
Middle 117

118	122	119
123	126	124
120	125	121

118 - 121
122 - 125
126

Appendix 3

Time table

Series	Coated, week	Drying finished	Re-saturation finished	Frost testing finished	Analysis, week	Delay, weeks
D0	18	-	-	-	28	10
D1	18	28 May	-	-	28	6
D1/S1	18	28 May	31 May	-	28	6
D1/S2	18	28 May	11 Jun	-	28	6
D2	18	7 Jun	-	-	28	5
D2/S1	18	7 Jun	10 Jun	-	28	4
D2/S2	18	7 Jun	21 Jun	-	28	4
D0/T1	18	-	-	?	?	?
D1/S1/T1	18	28 May	31 May	28 Jun	29	3
D1/S1/T2	18	28 May	31 May	28 Jun	29	3
D1/S1/T3	18	28 May	31 May	28 Jun	29	3
D1/S2/T1	18	28 May	11 Jun	9 Jul	29	1
D2/S1/T1	18	7 Jun	10 Jun	8 Jul	29	1
D2/S2/T1	18	7 Jun	21 Jun	19 Jul	29	0

Appendix 4

Determination of moisture and degree of saturation profiles, method

1 The prism is weighed

2 The sealer is removed and the prism is divided in 5 pieces of 10 mm each and 1 of 20 mm. On each piece the following is performed

2.1 Weighing (w)

2.2 24 hours in water (boiled and cooled), weighing saturated, surface dry (w_{cap})

2.3 24 hours in water (boiled and cooled) under vacuum, weighing saturated, surface dry (w_{vac})

2.4 24 hours at 105°C, weighing (w_{105})

3 · Calculations

Moisture content:

$$u=w-w_{105}$$

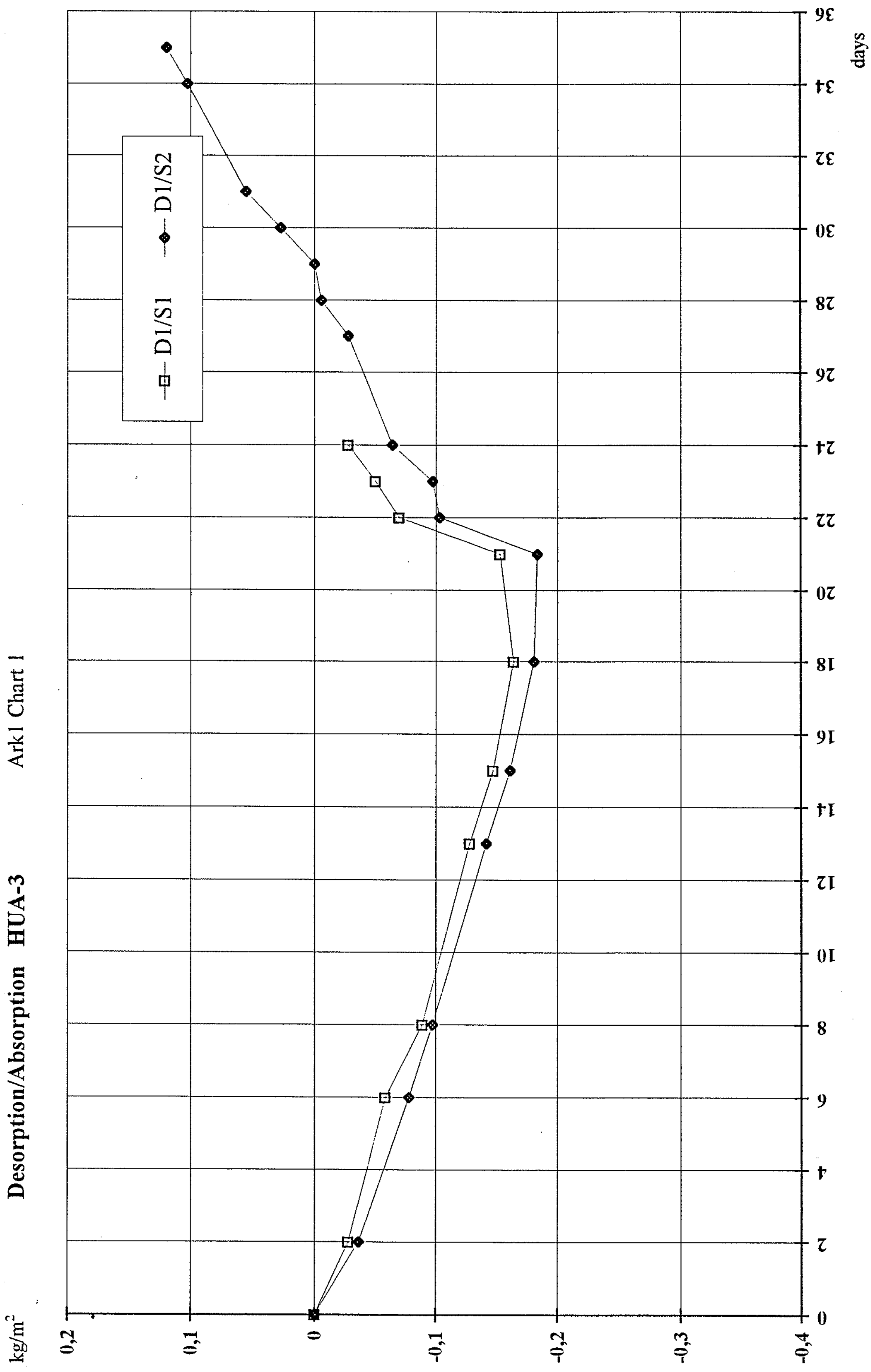
Degree of saturation:

$$S_{cap}=w-w_{105}/w_{cap}-w_{105}$$

$$S_{vac}=w-w_{105}/w_{vac}-w_{105}$$

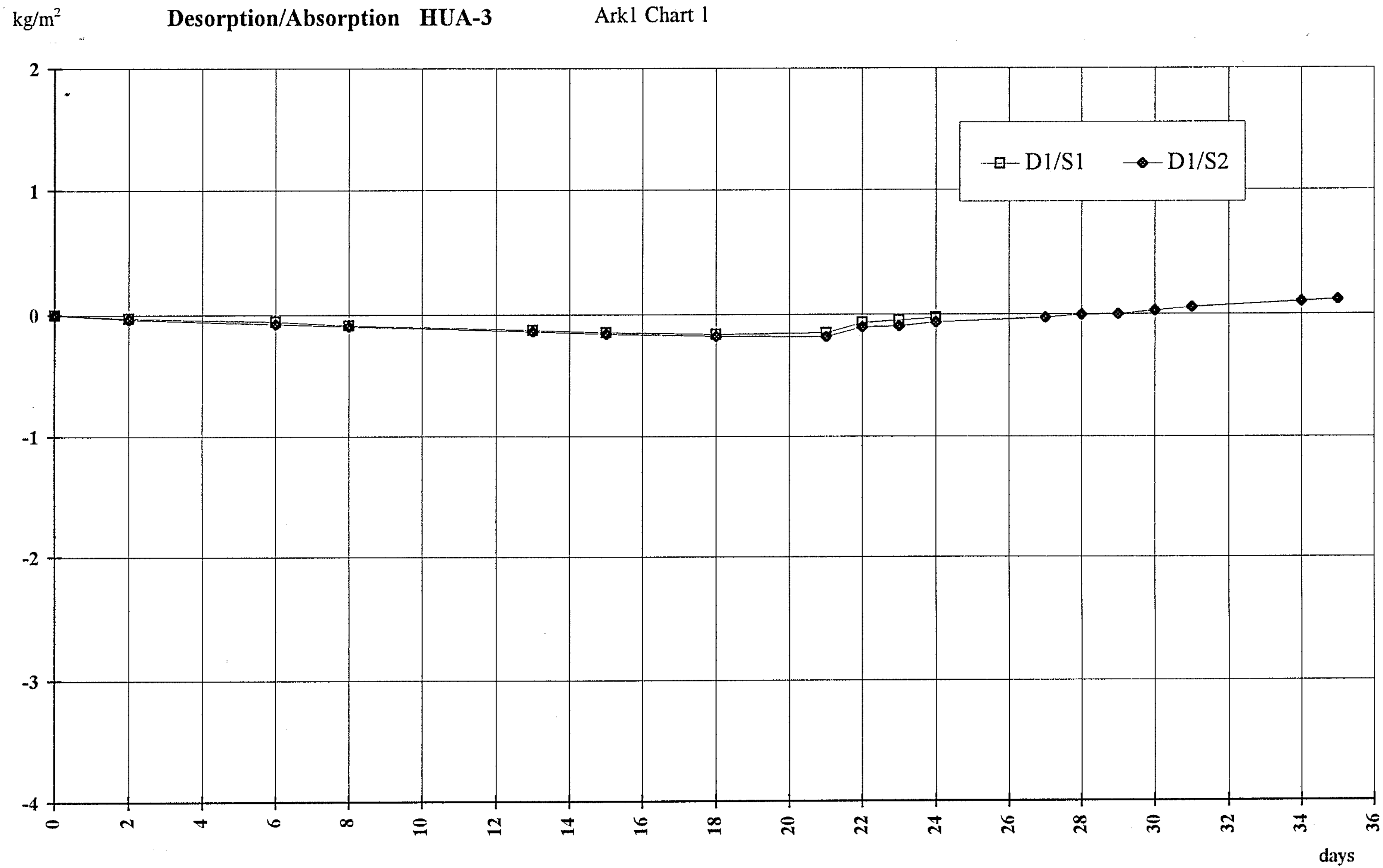
Appendix 5

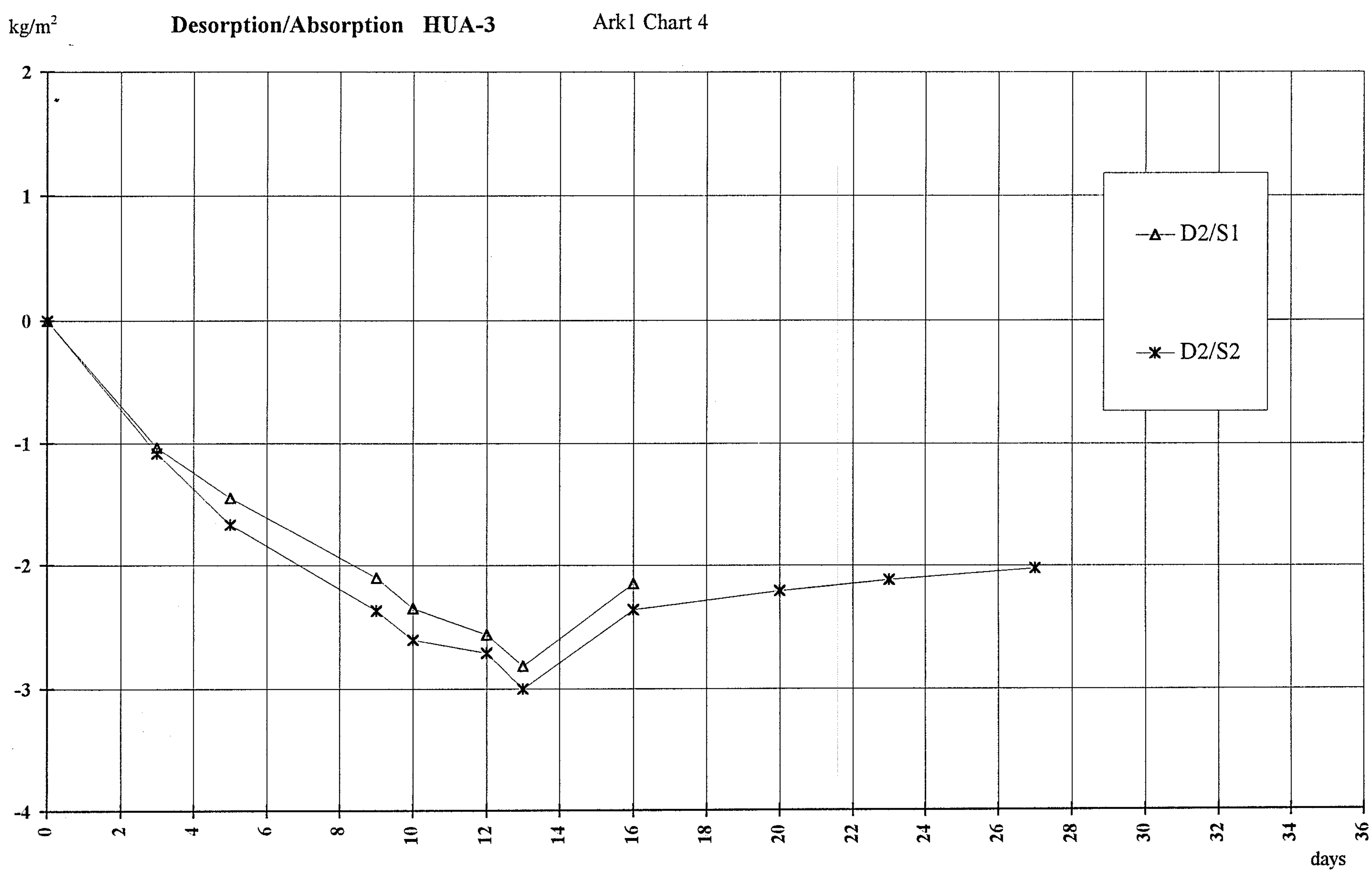
Desorption and absorption during drying, re-saturation and frost testing



Desorption/Absorption HUA-3

Ark1 Chart 1

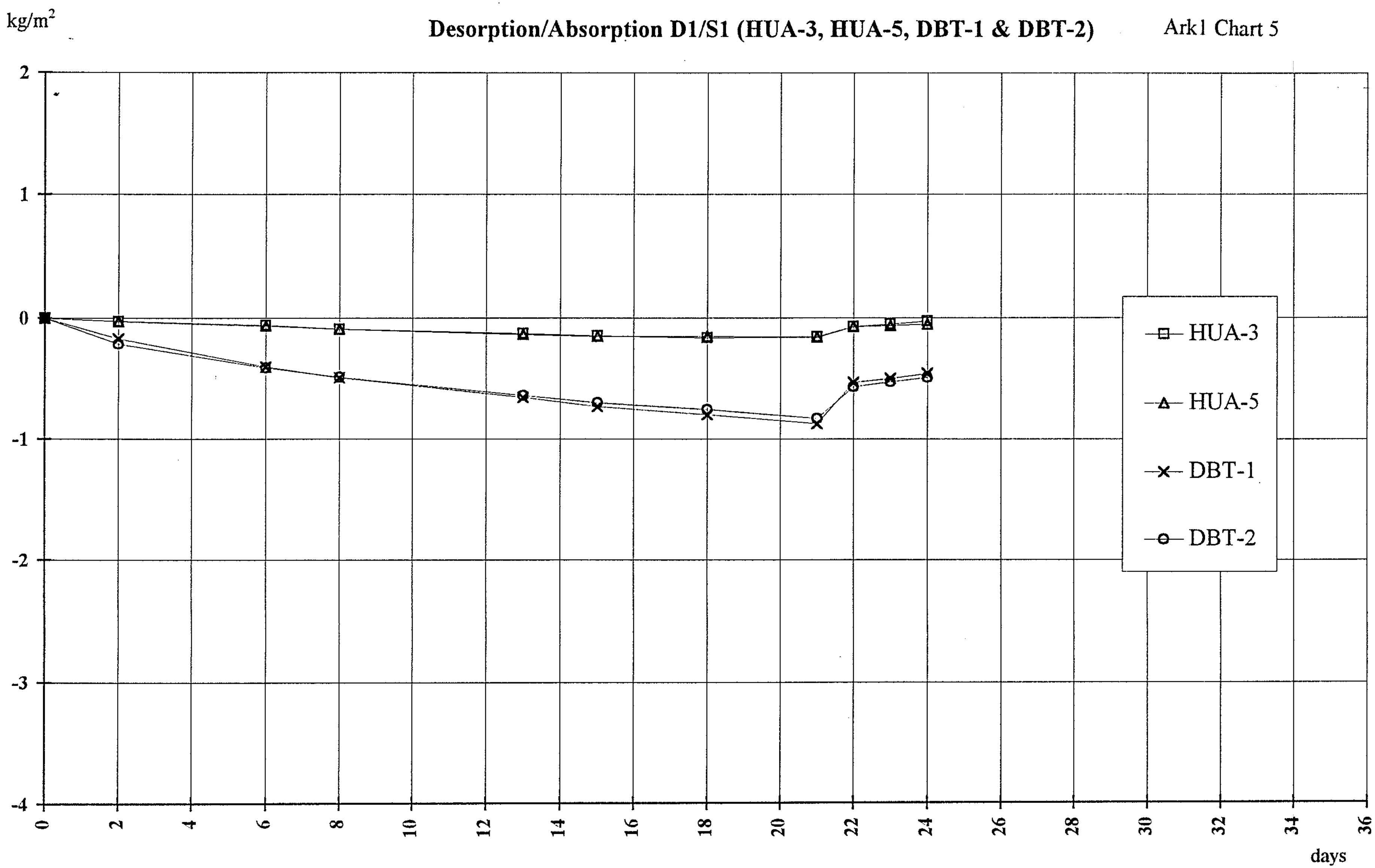


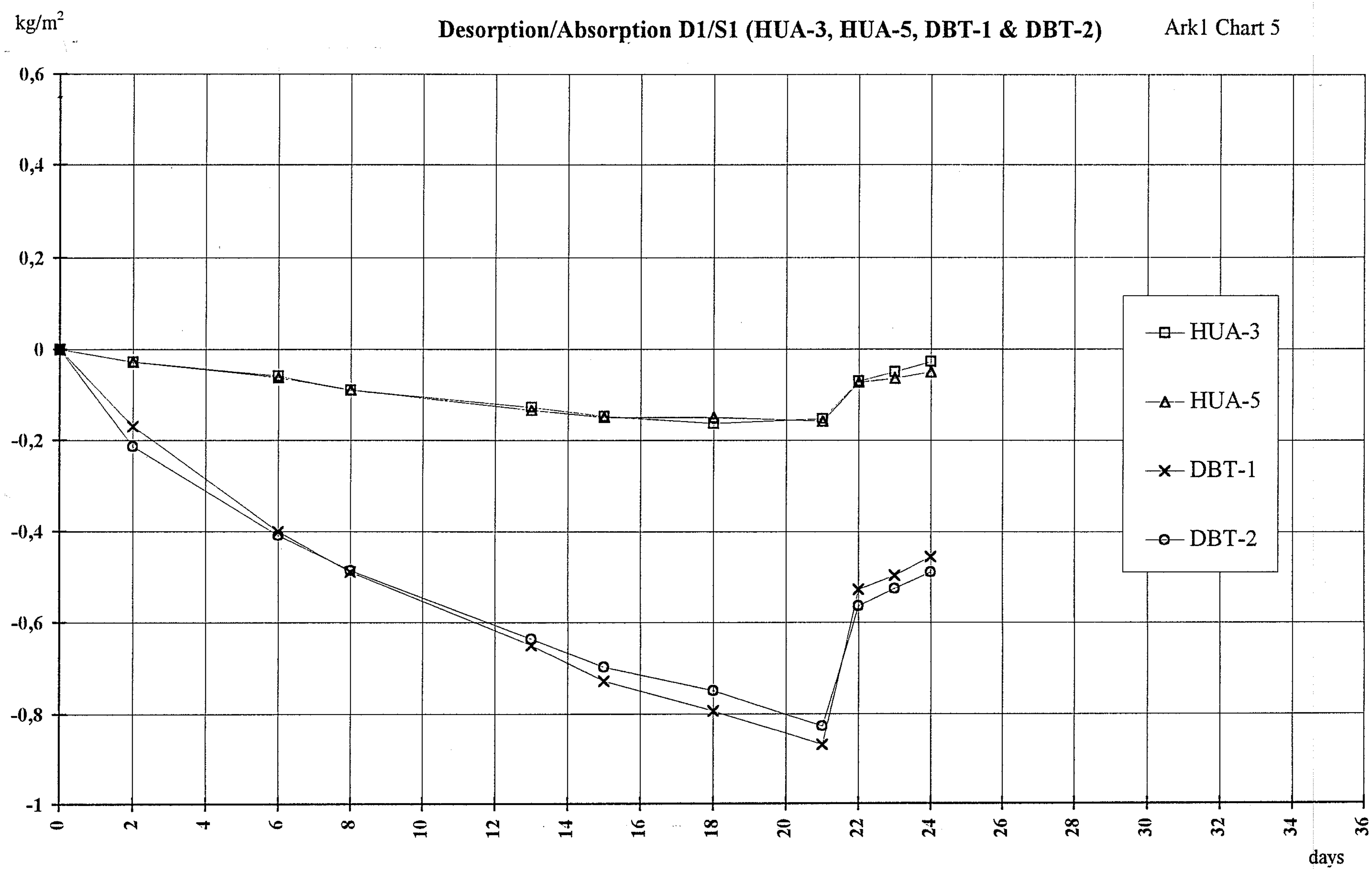


kg/m²
Desorption/Absorption HUA-3
Ark1 Chart 4
2
1
0
-1
-2
-3
-4
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
days
D2/S1
D2/S2

Desorption/Absorption D1/S1 (HUA-3, HUA-5, DBT-1 & DBT-2)

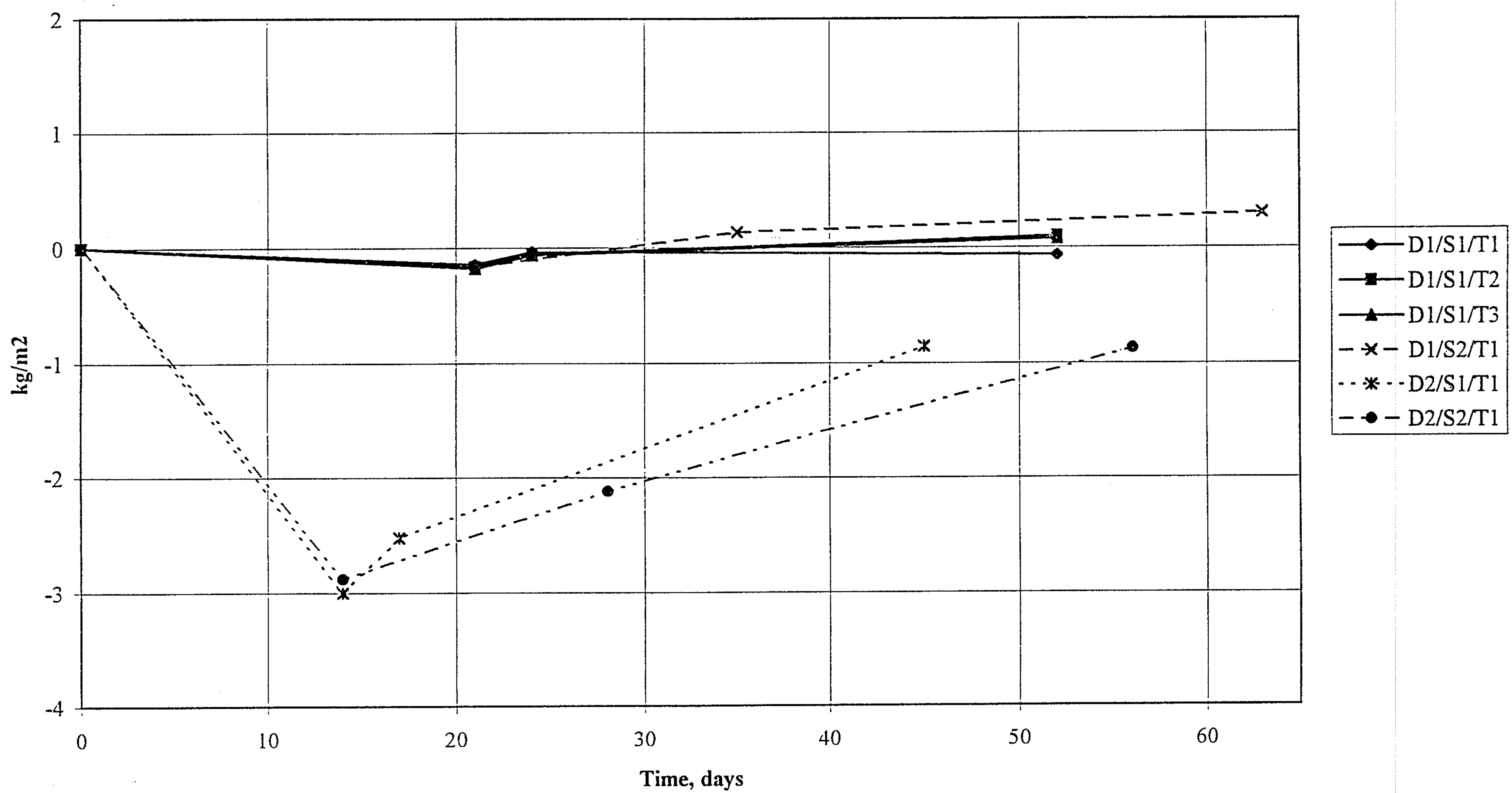
Ark1 Chart 5

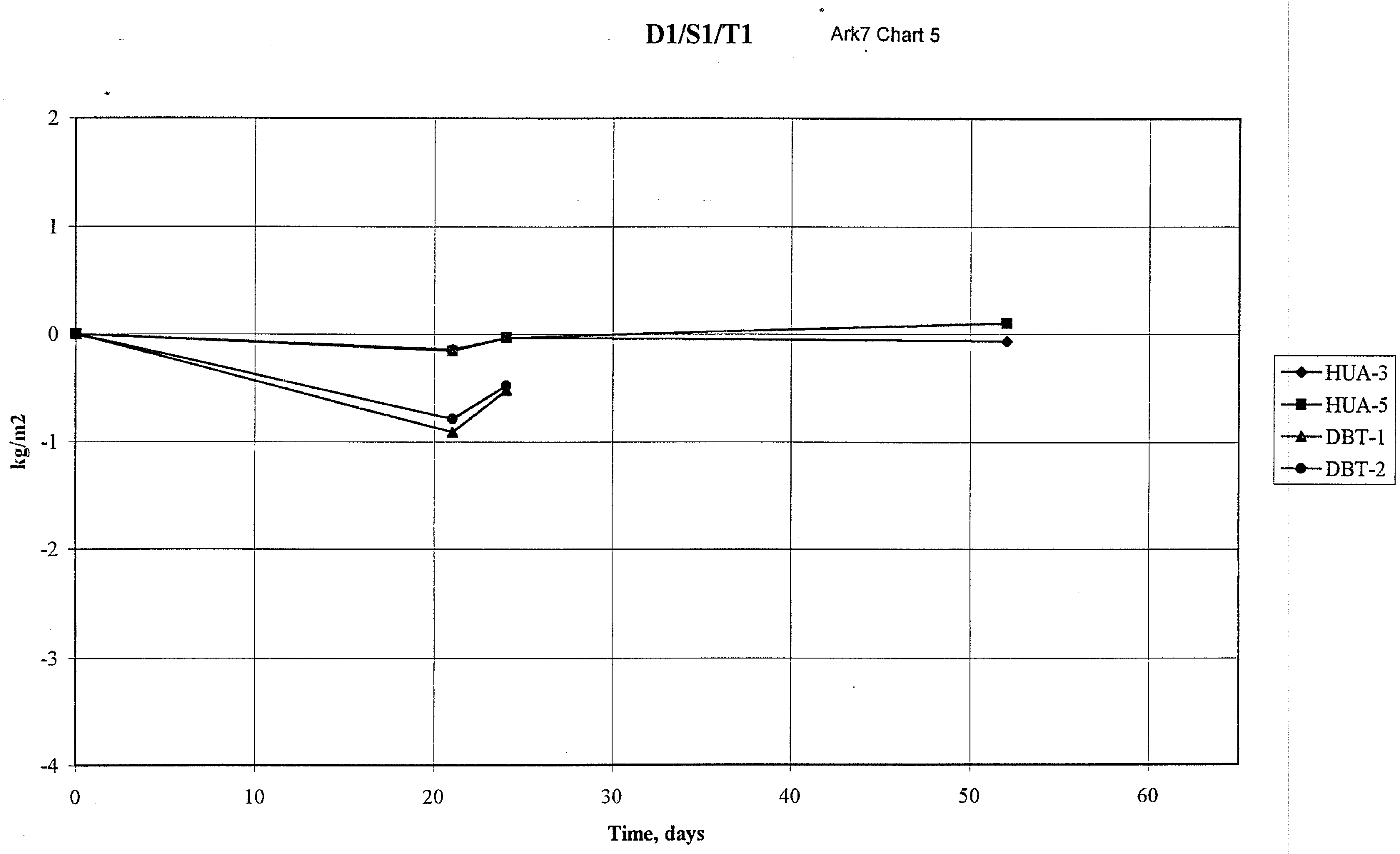




kg/m²
Desorption/Absorption D1/S1 (HUA-3, HUA-5, DBT-1 & DBT-2)
Ark1 Chart 5
0,6
0,4
0,2
0
-0,2
-0,4
-0,6
-0,8
-1
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
days
HUA-3
HUA-5
DBT-1
DBT-2

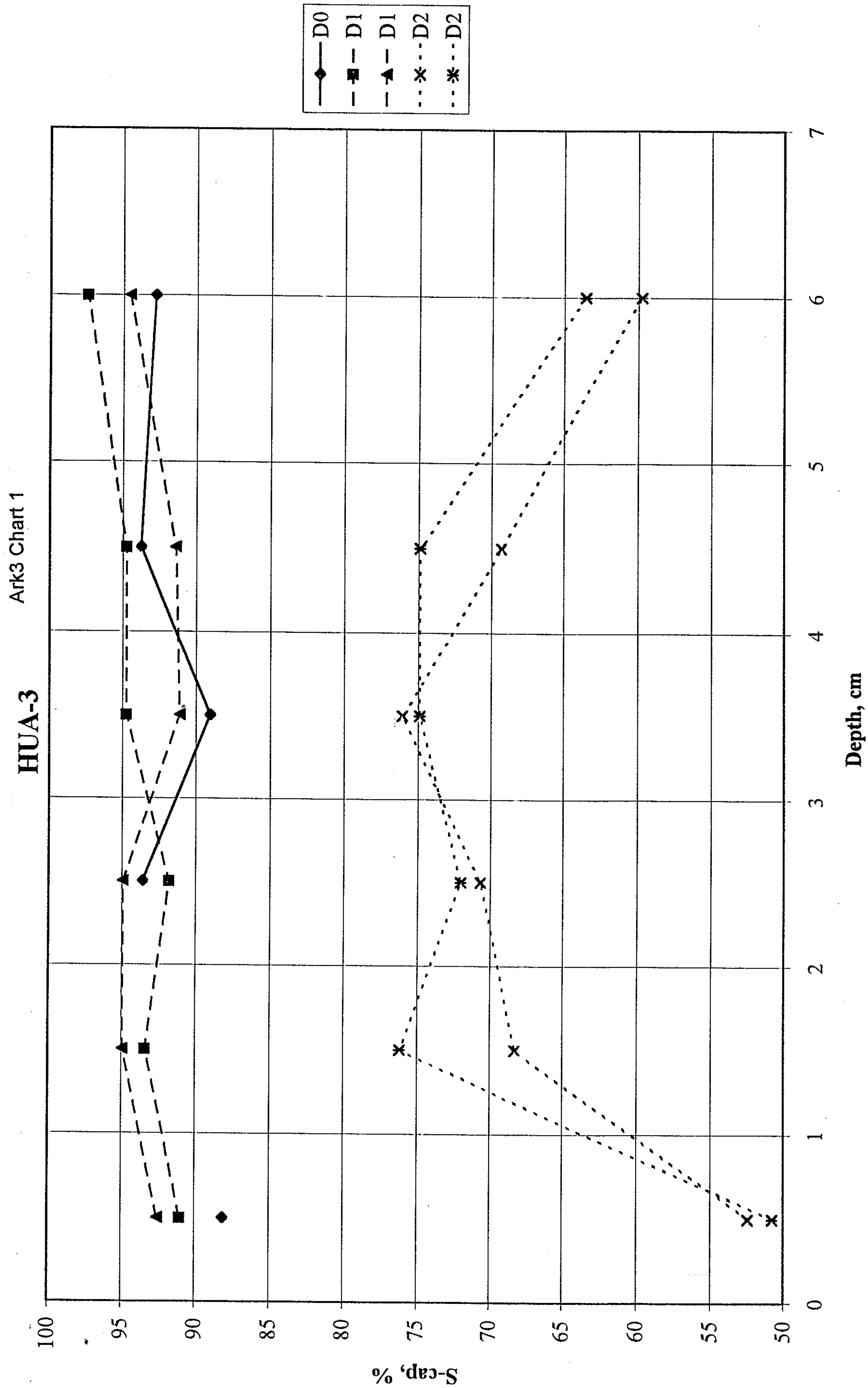
HUA-3 Ark7 Chart 1

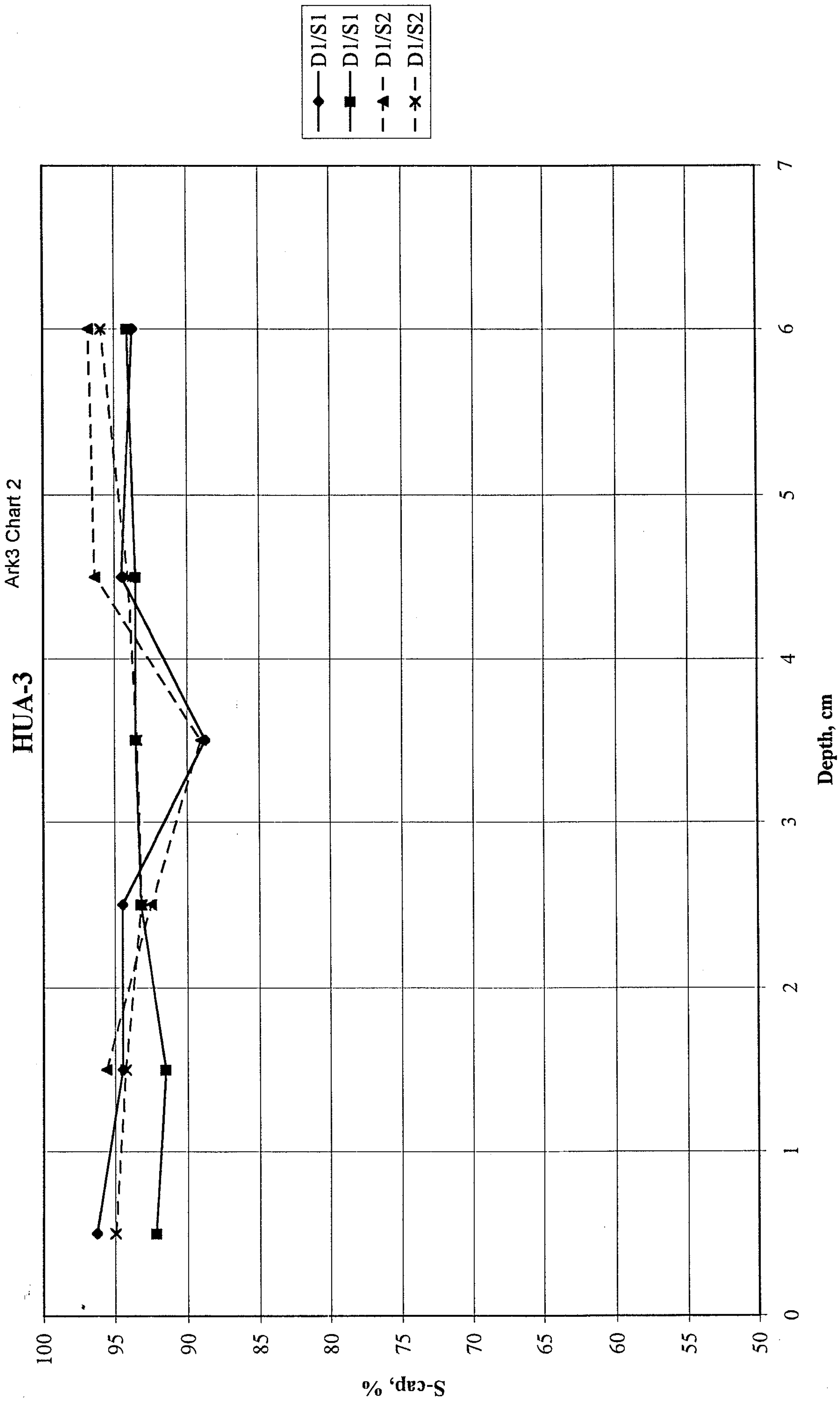



D1/S1/T1
Ark7 Chart 5
kg/m2
2
1
0
-1
-2
-3
-4
0
10
20
30
40
50
60
Time, days
HUA-3
HUA-5
DBT-1
DBT-2

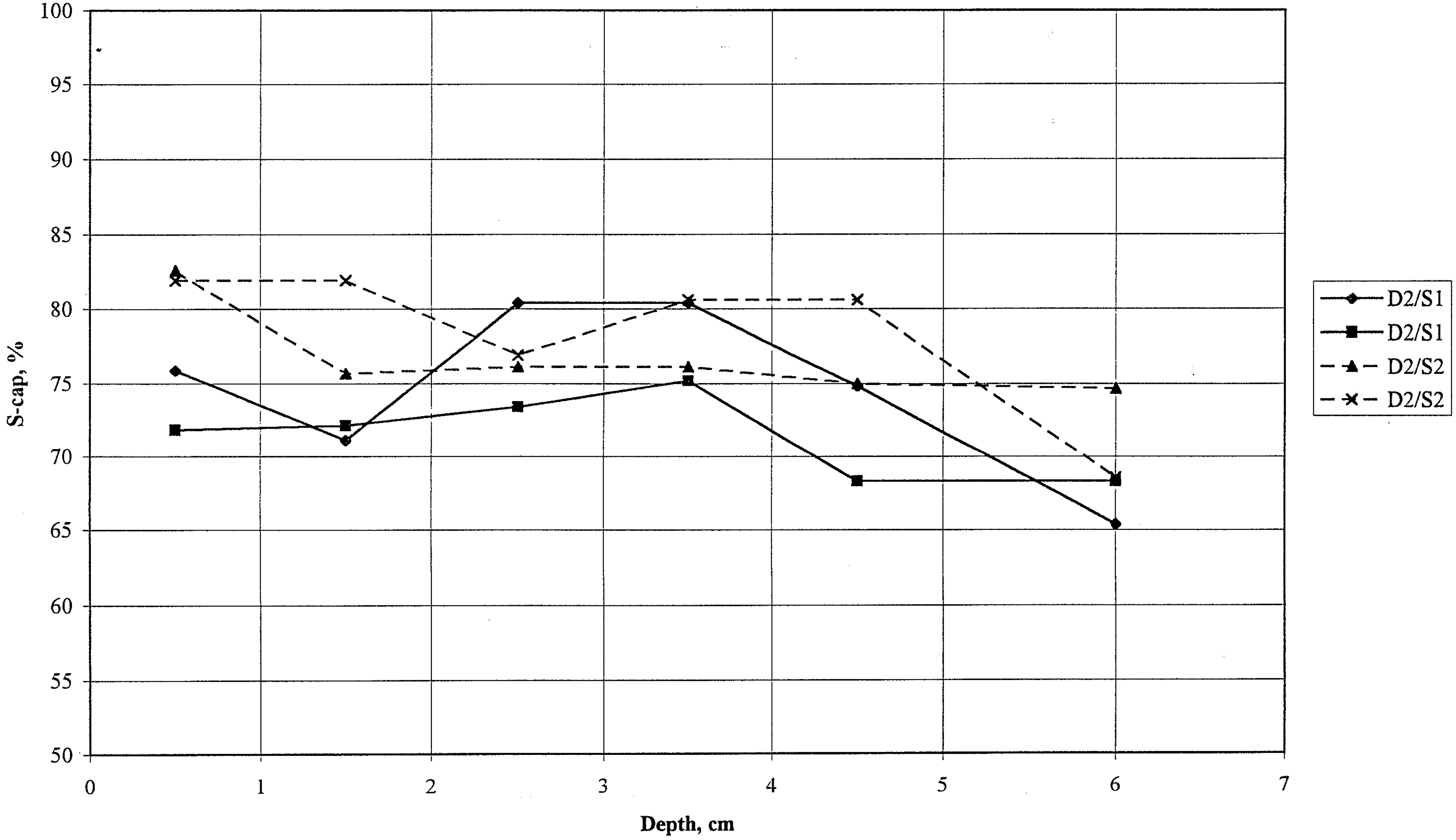
Appendix 6

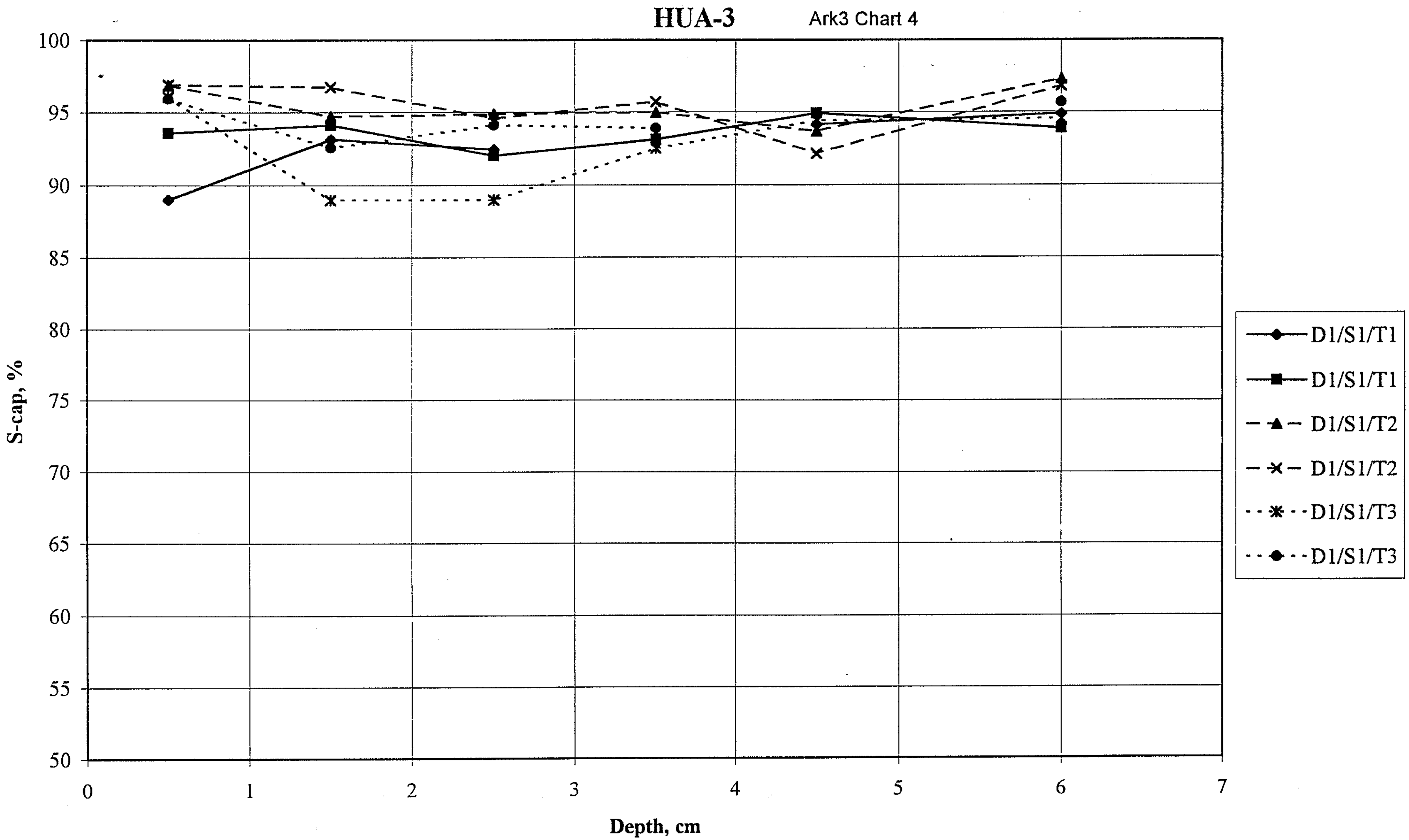
Moisture and degree of saturation



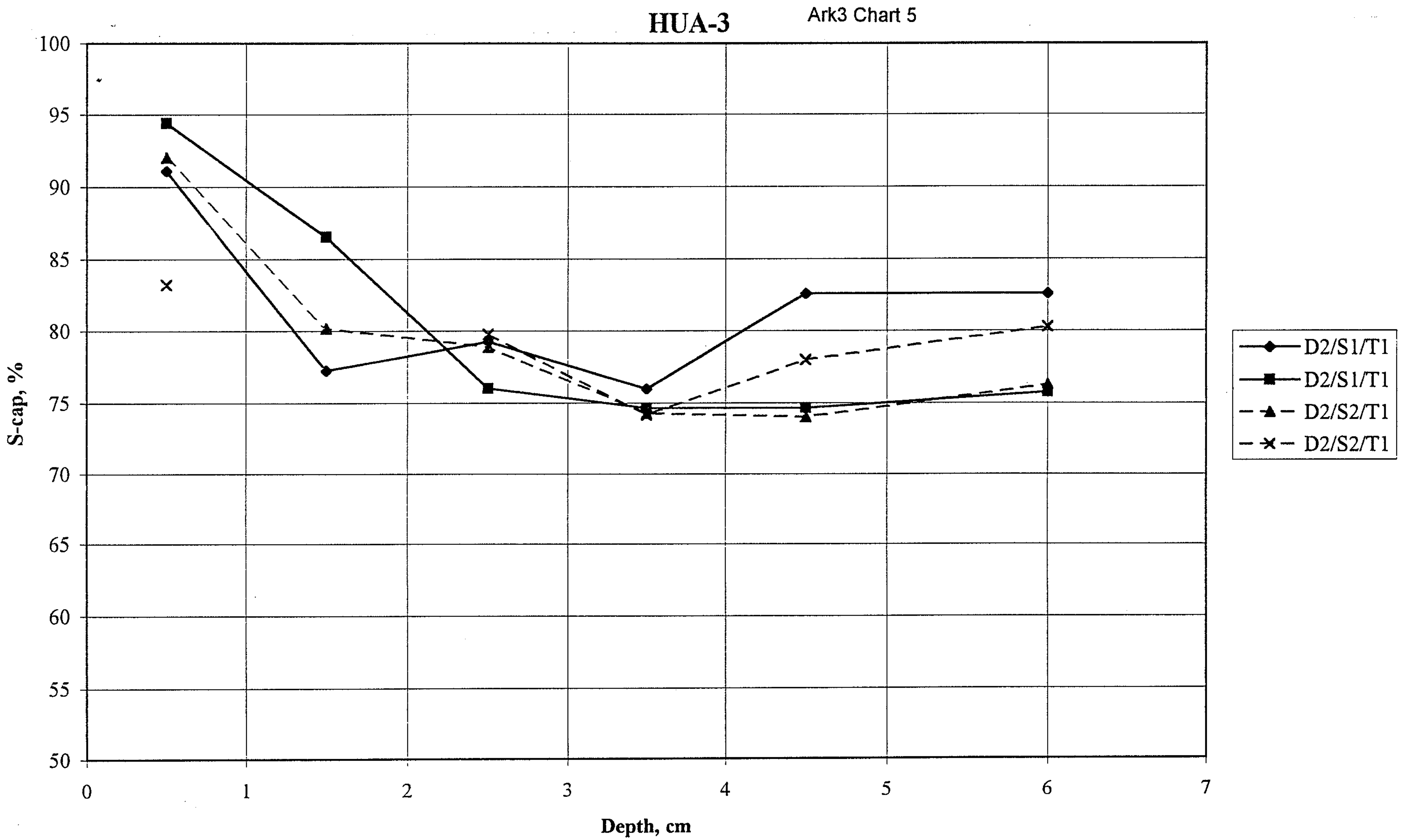

HUA-3
Ark3 Chart 2
D1/S1
D1/S1
D1/S2
D1/S2
S-cap, %
Depth, cm
100
95
90
85
80
75
70
65
60
55
50
0
1
2
3
4
5
6
7

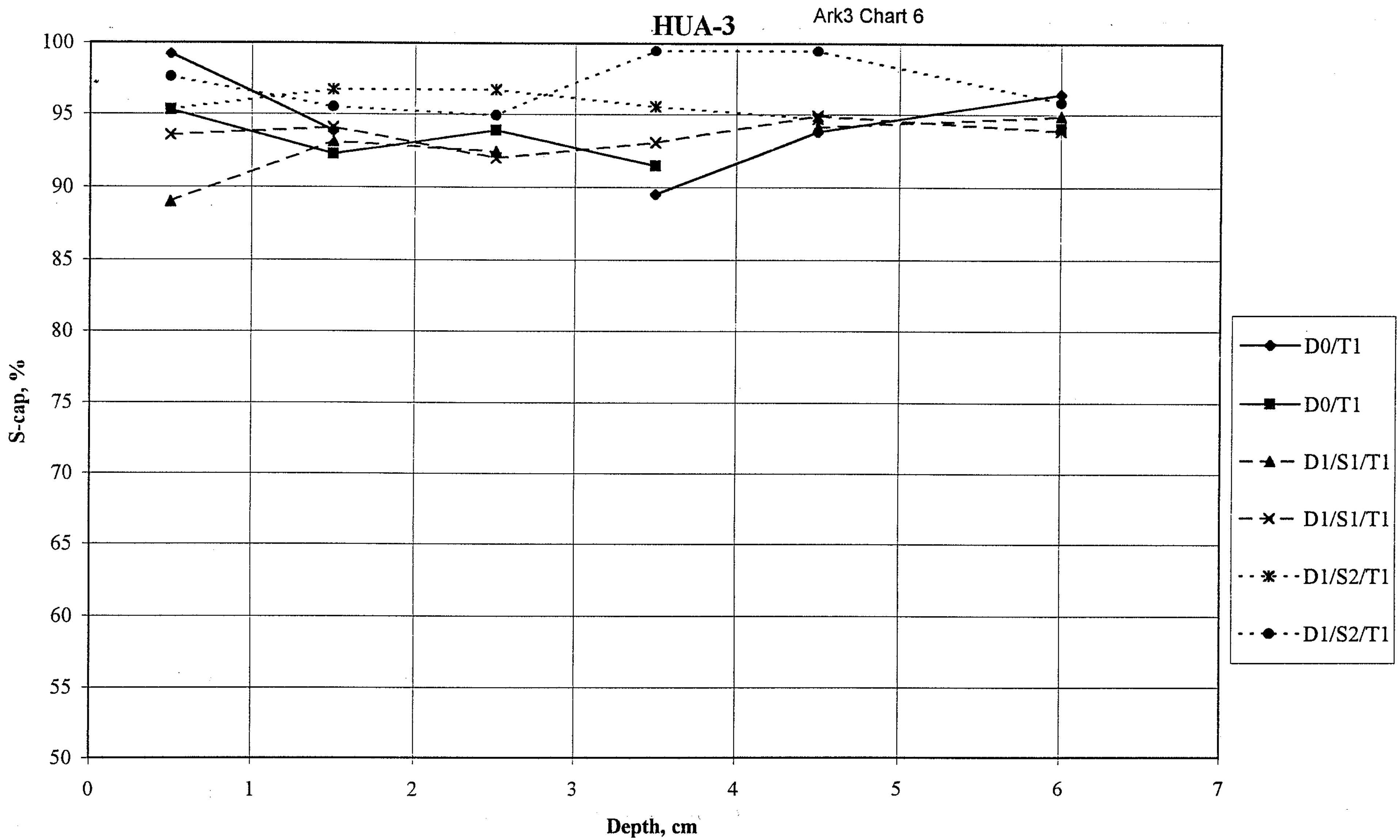
HUA-3 Ark3 Chart 3

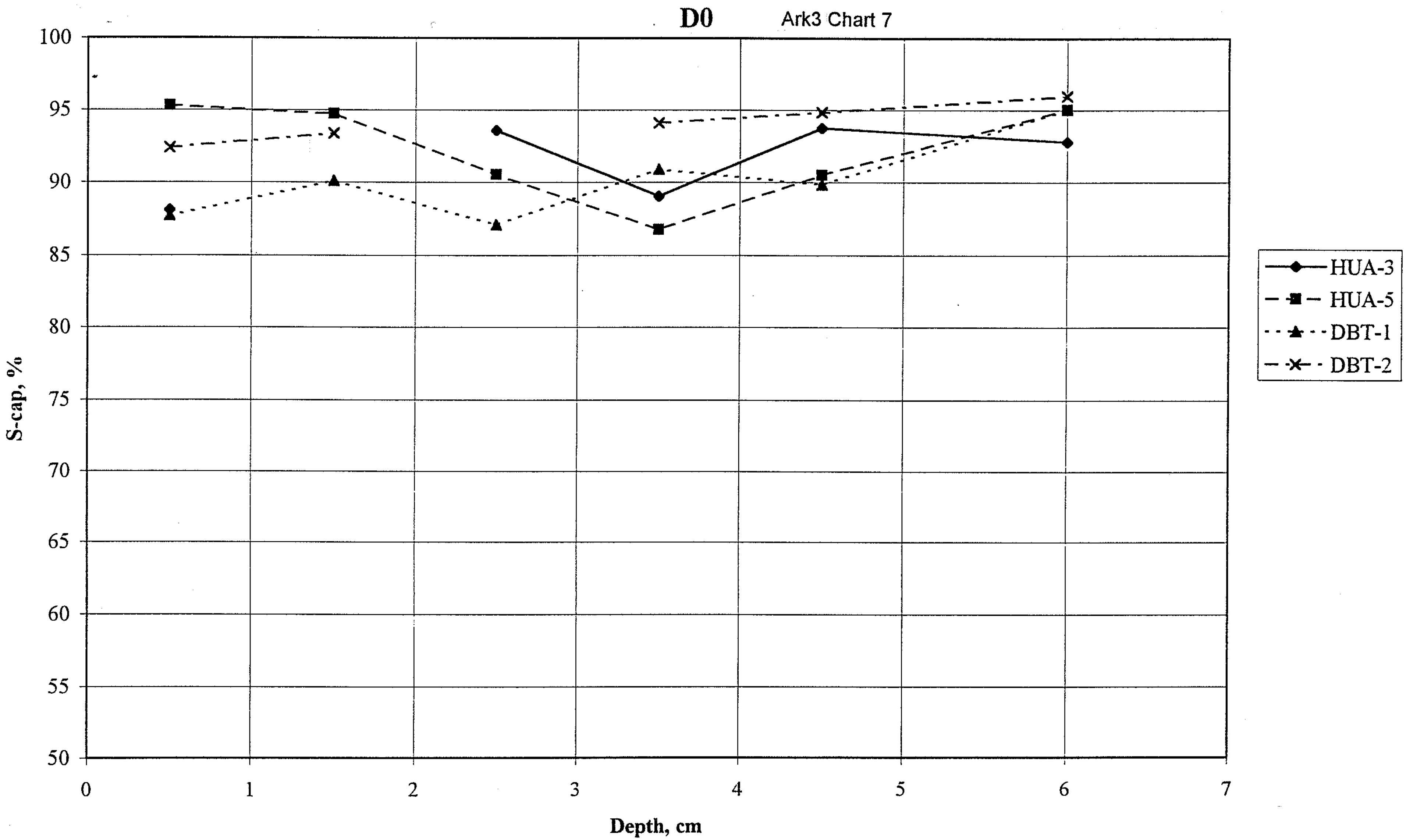


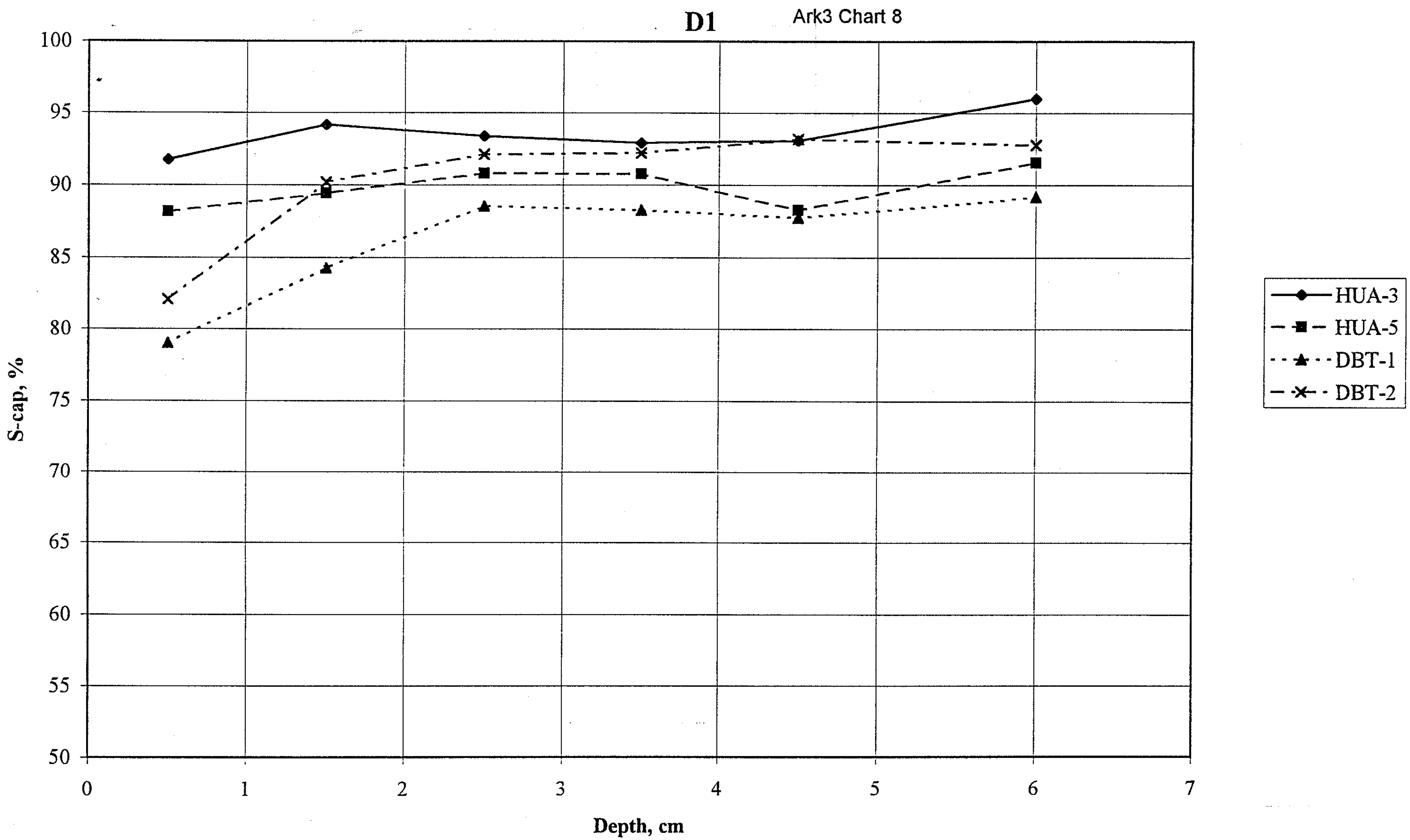


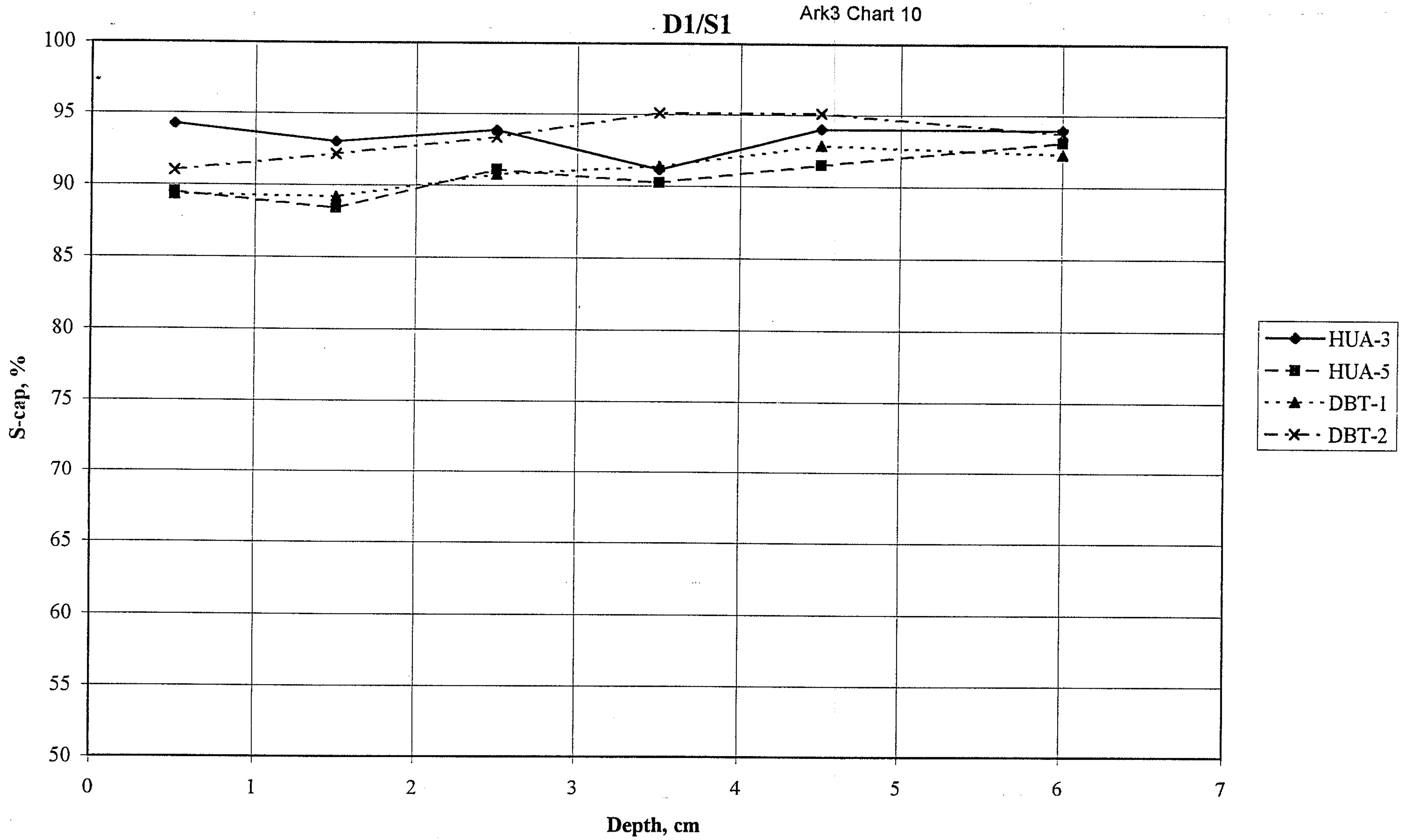
HUA-3
Ark3 Chart 4
S-cap, %
Depth, cm
100
95
90
85
80
75
70
65
60
55
50
0
1
2
3
4
5
6
7
D1/S1/T1
D1/S1/T1
D1/S1/T2
D1/S1/T2
D1/S1/T3
D1/S1/T3

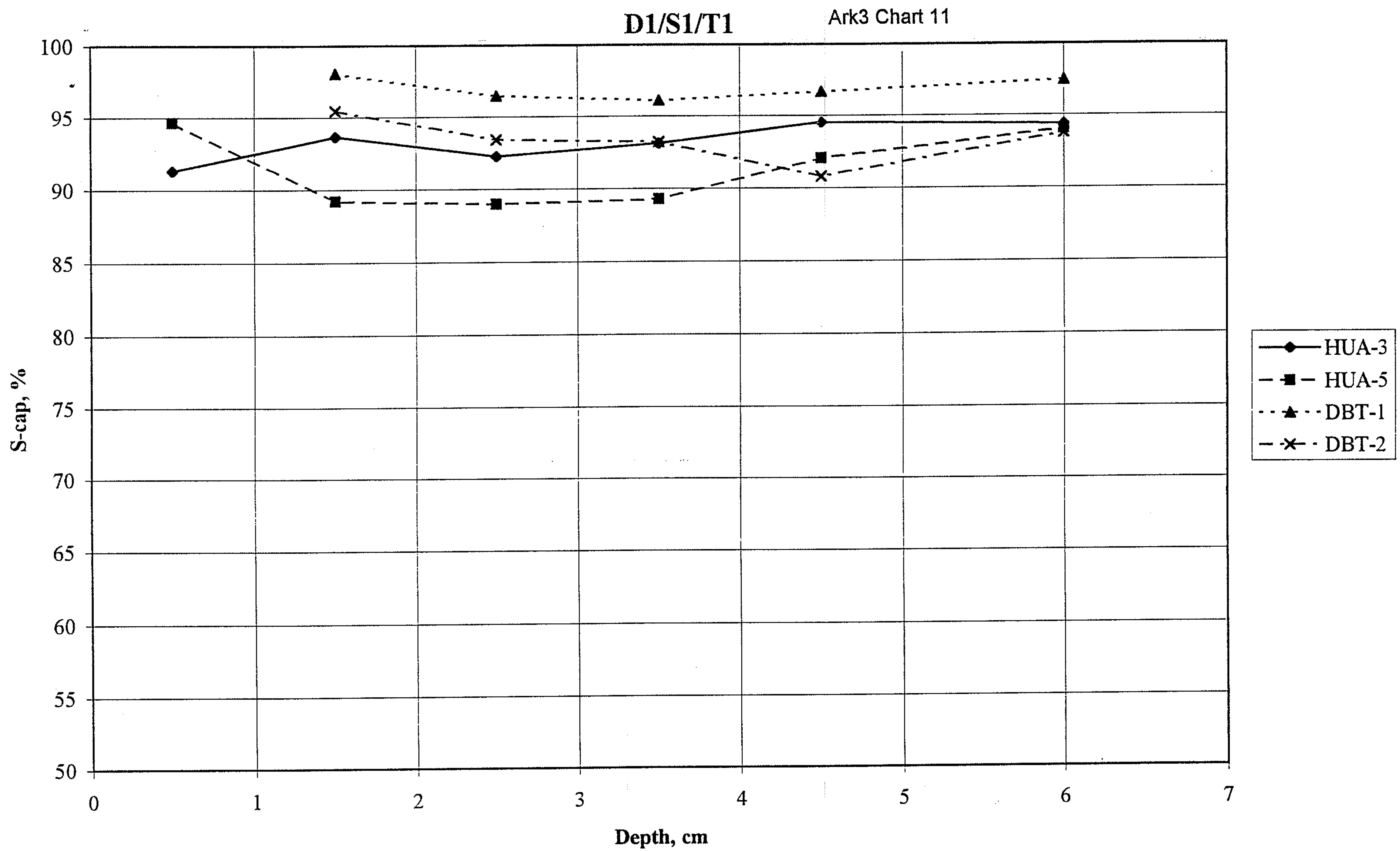

HUA-3
Ark3 Chart 5
100
95
90
85
80
75
70
65
60
55
50
S-cap, %
0
1
2
3
4
5
6
7
Depth, cm
D2/S1/T1
D2/S1/T1
D2/S2/T1
D2/S2/T1

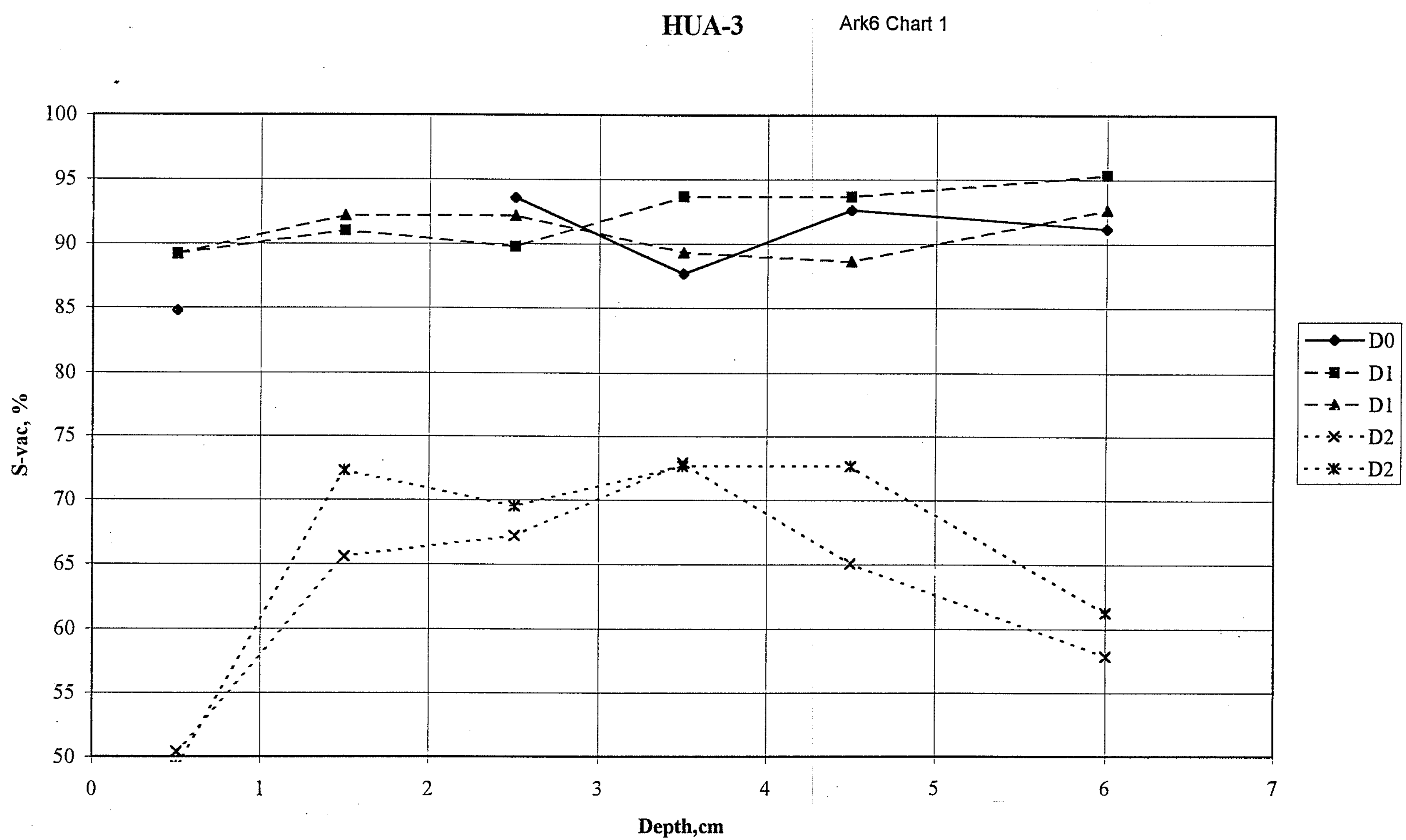

HUA-3
Ark3 Chart 6
S-cap, %
100
95
90
85
80
75
70
65
60
55
50
0
1
2
3
4
5
6
7
Depth, cm
D0/T1
D0/T1
D1/S1/T1
D1/S1/T1
D1/S2/T1
D1/S2/T1


D0
Ark3 Chart 7
100
95
90
85
80
75
70
65
60
55
50
S-cap, %
0
1
2
3
4
5
6
7
Depth, cm
HUA-3
HUA-5
DBT-1
DBT-2

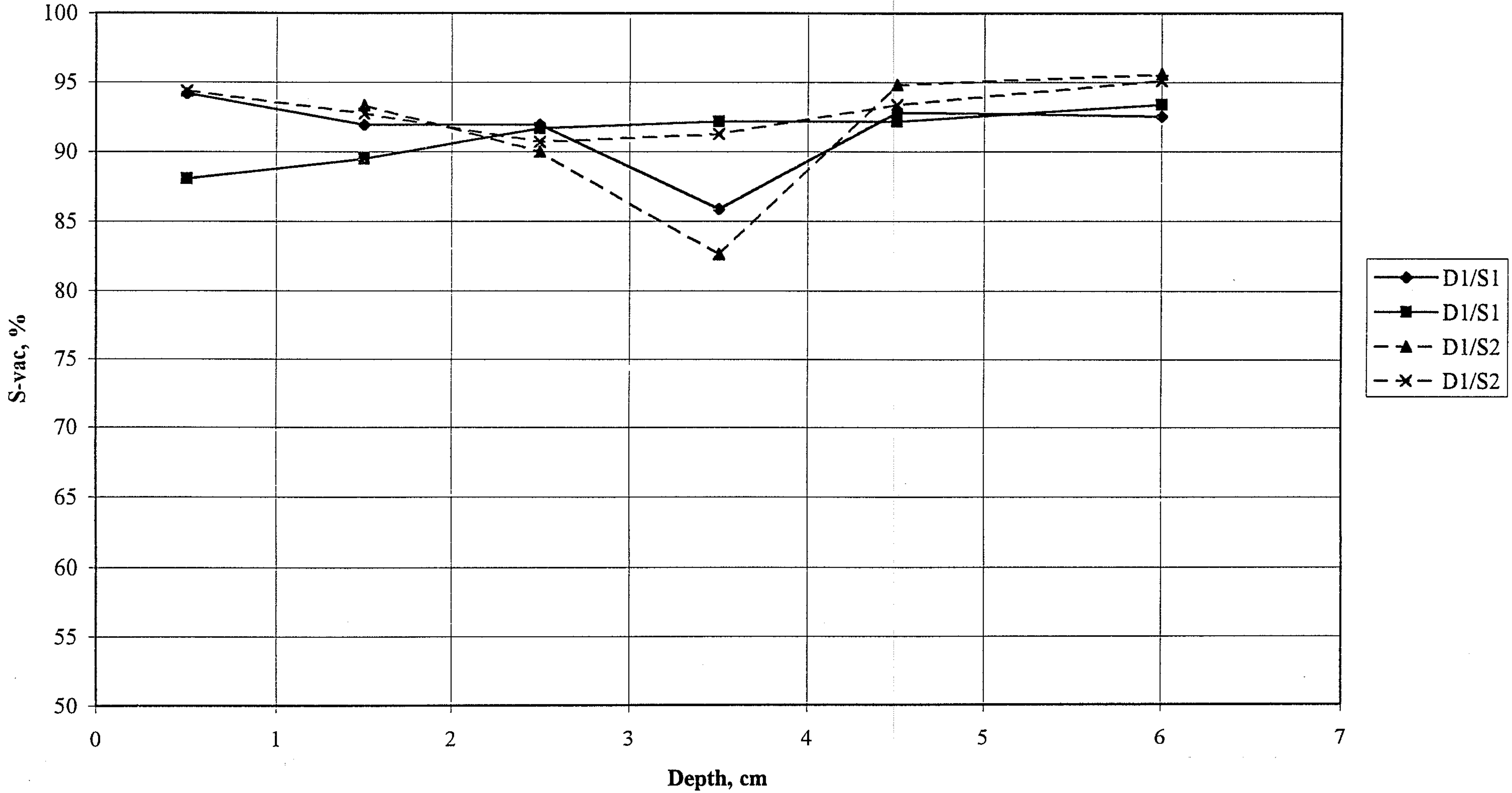

D1
Ark3 Chart 8
100
95
90
85
80
75
70
65
60
55
50
S-cap, %
0
1
2
3
4
5
6
7
Depth, cm
HUA-3
HUA-5
DBT-1
DBT-2


D1/S1
Ark3 Chart 10
100
95
90
85
80
75
70
65
60
55
50
S-cap, %
0
1
2
3
4
5
6
7
Depth, cm
HUA-3
HUA-5
DBT-1
DBT-2

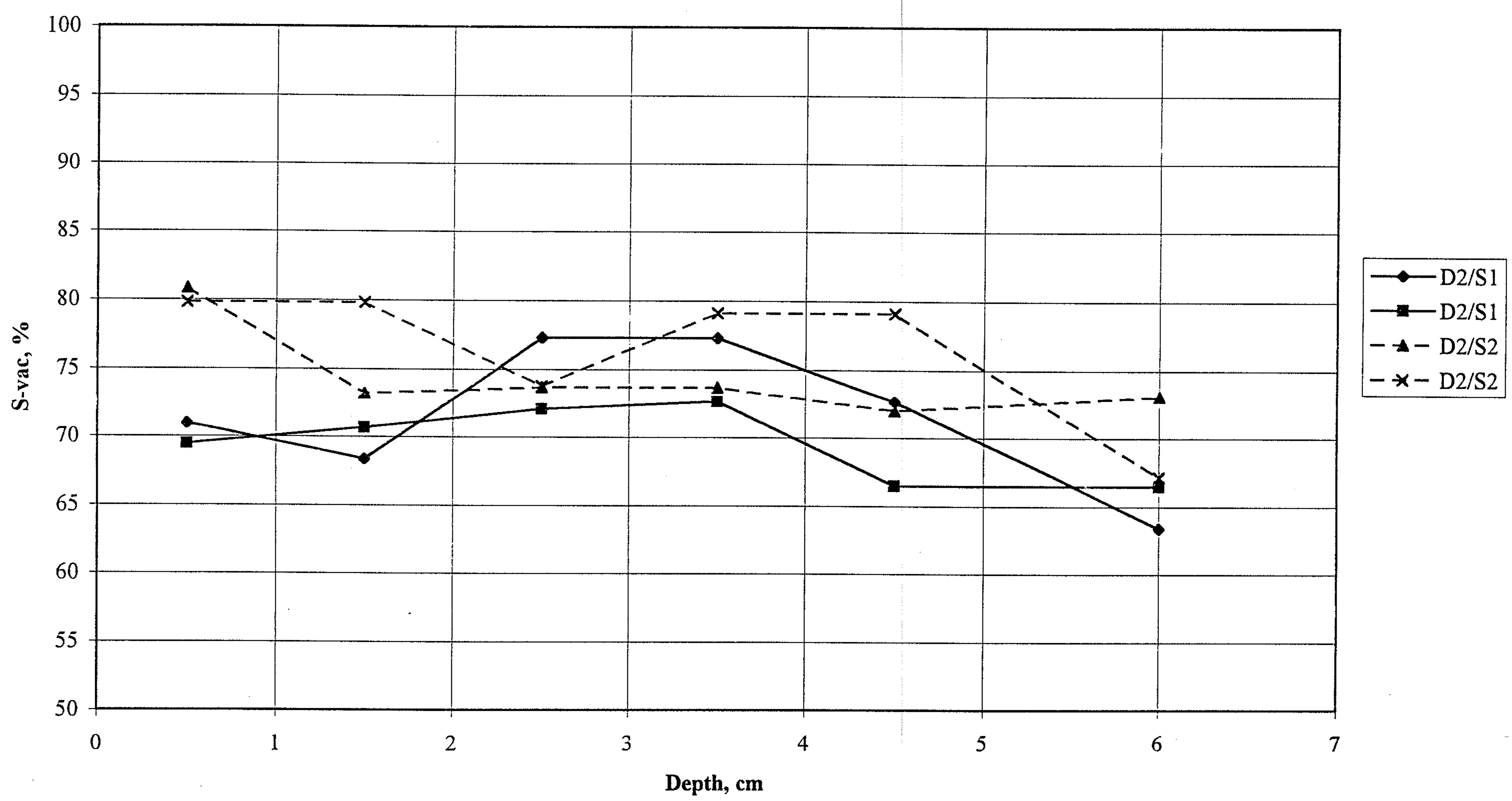

D1/S1/T1
Ark3 Chart 11
S-cap, %
100
95
90
85
80
75
70
65
60
55
50
0
1
2
3
4
5
6
7
Depth, cm
HUA-3
HUA-5
DBT-1
DBT-2


HUA-3
Ark6 Chart 1
S-vac, %
100
95
90
85
80
75
70
65
60
55
50
0
1
2
3
4
5
6
7
Depth,cm
D0
D1
D1
D2
D2

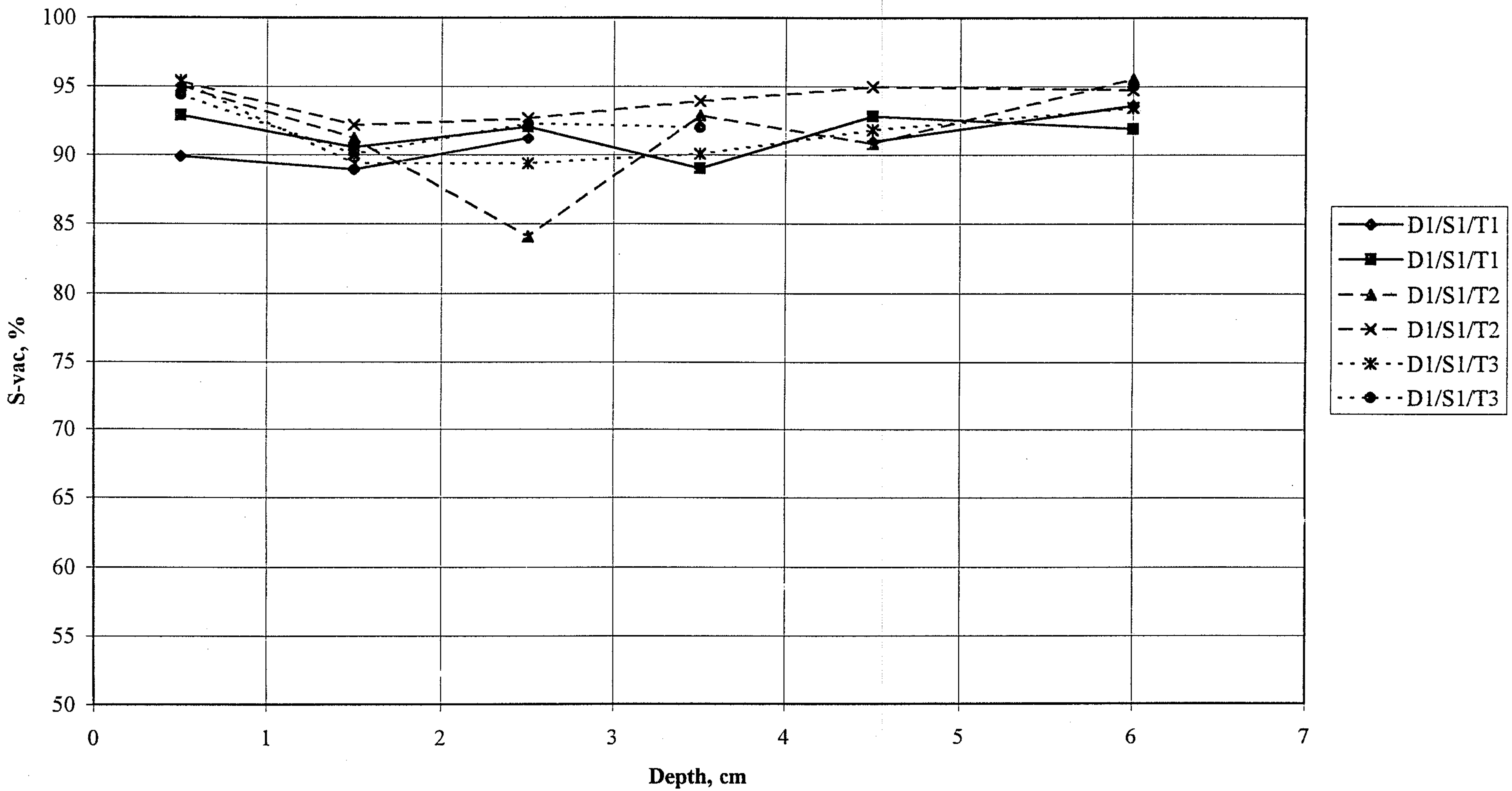
HUA-3 Ark6 Chart 4



HUA-3 Ark6 Chart 5

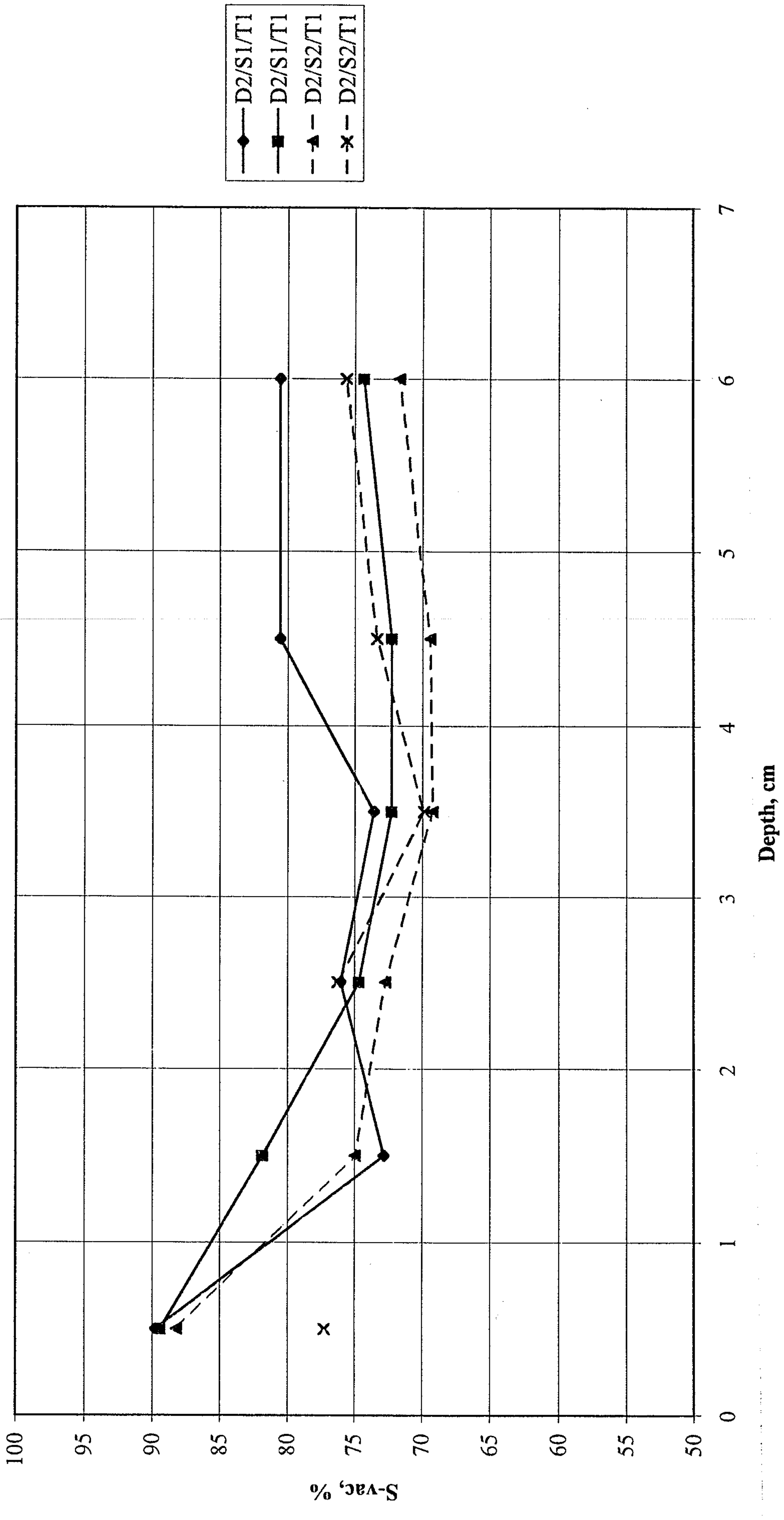


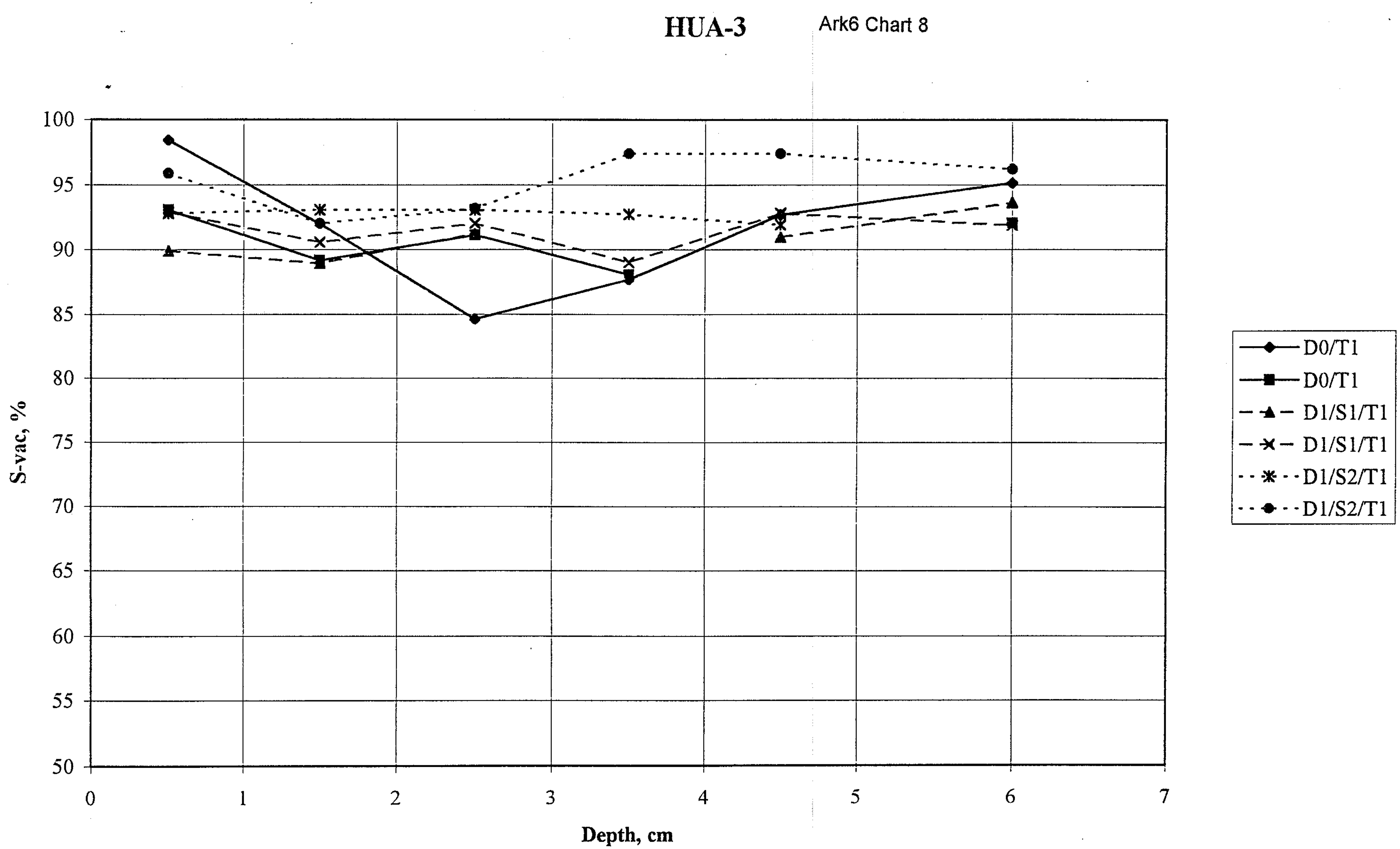
HUA-3 Ark6 Chart 6



HUA-3

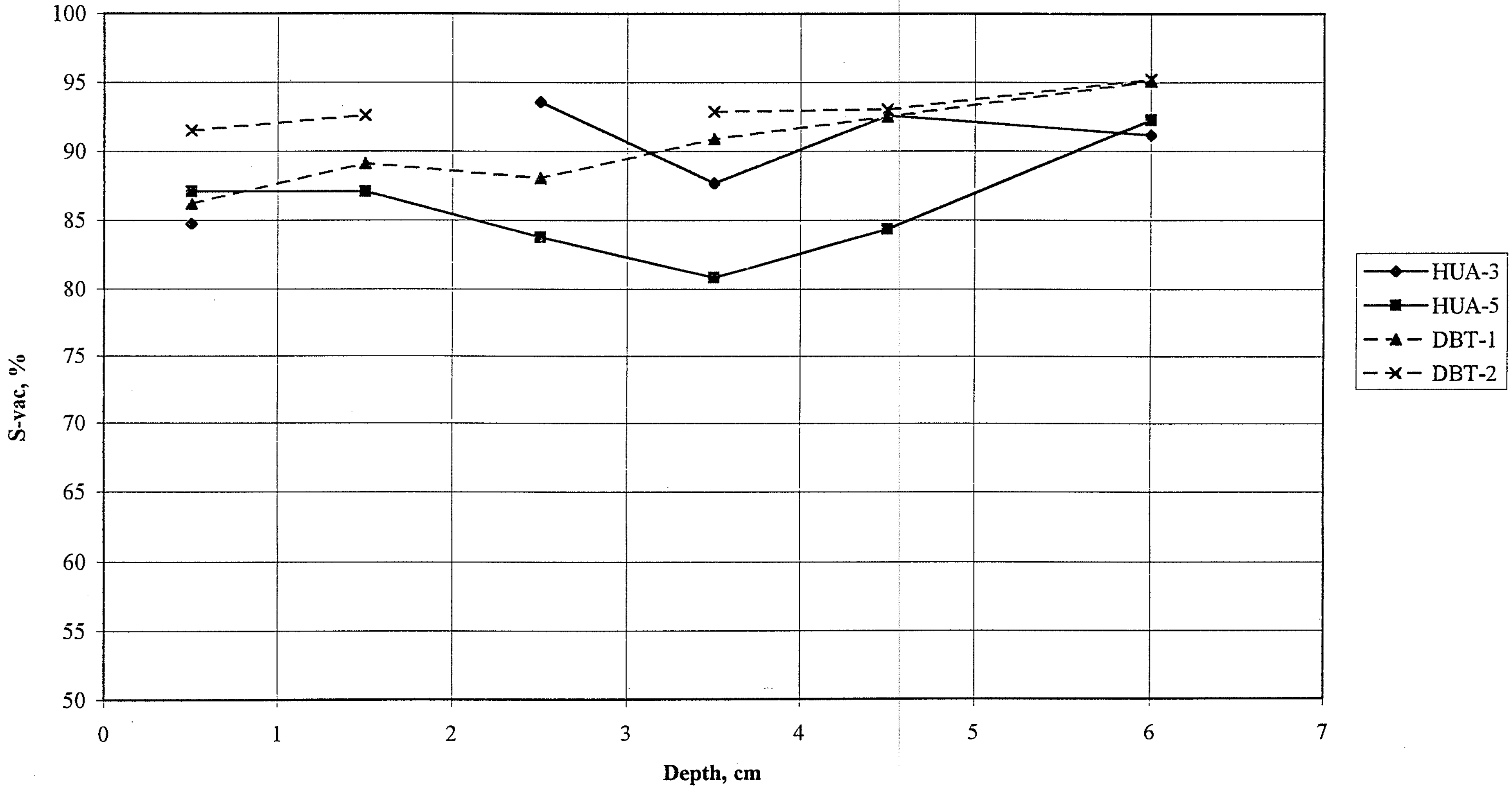
Ark6 Chart 7



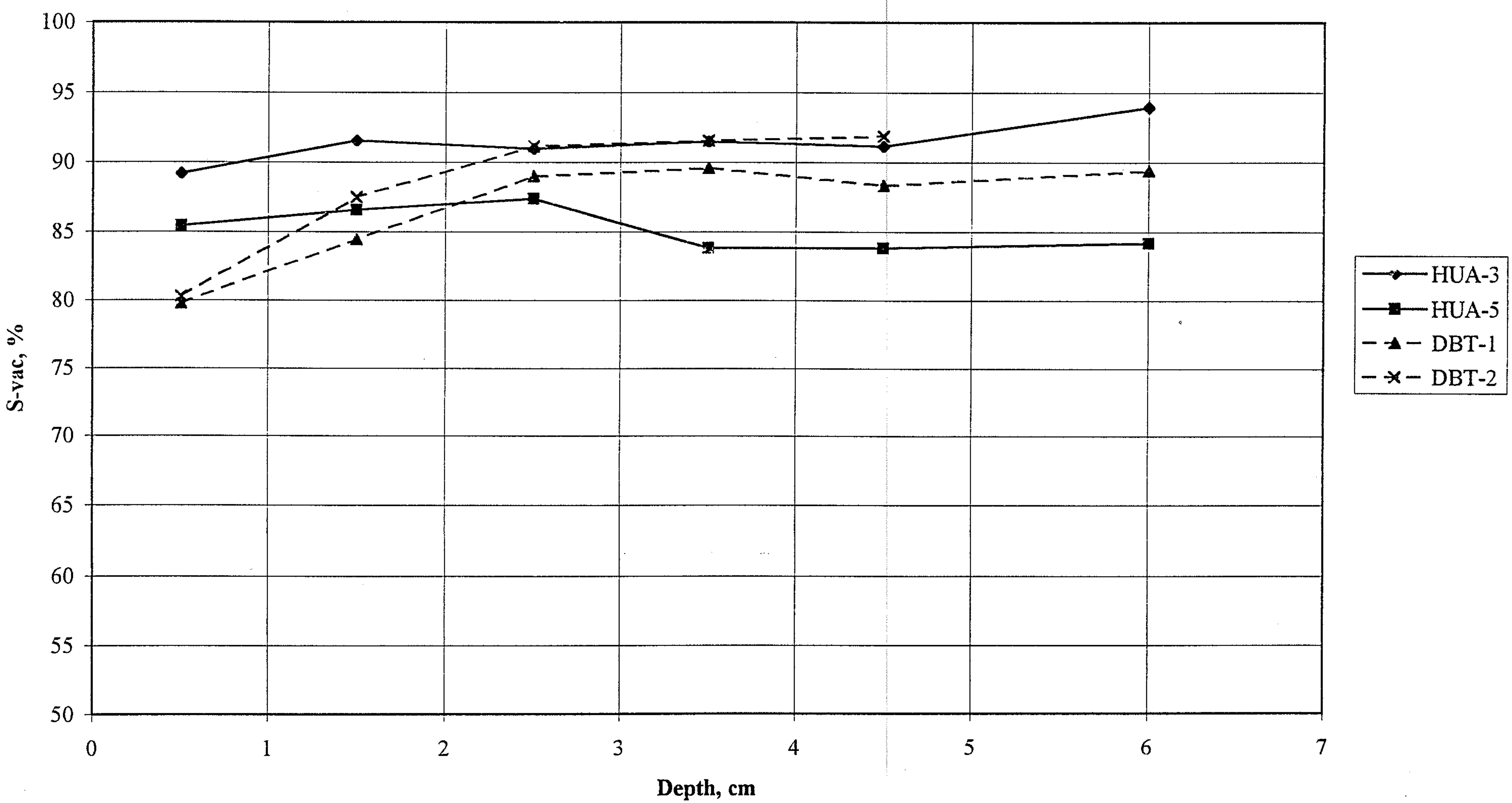


HUA-3
Ark6 Chart 8
100
95
90
85
80
75
70
65
60
55
50
S-vac, %
0
1
2
3
4
5
6
7
Depth, cm
D0/T1
D0/T1
D1/S1/T1
D1/S1/T1
D1/S2/T1
D1/S2/T1

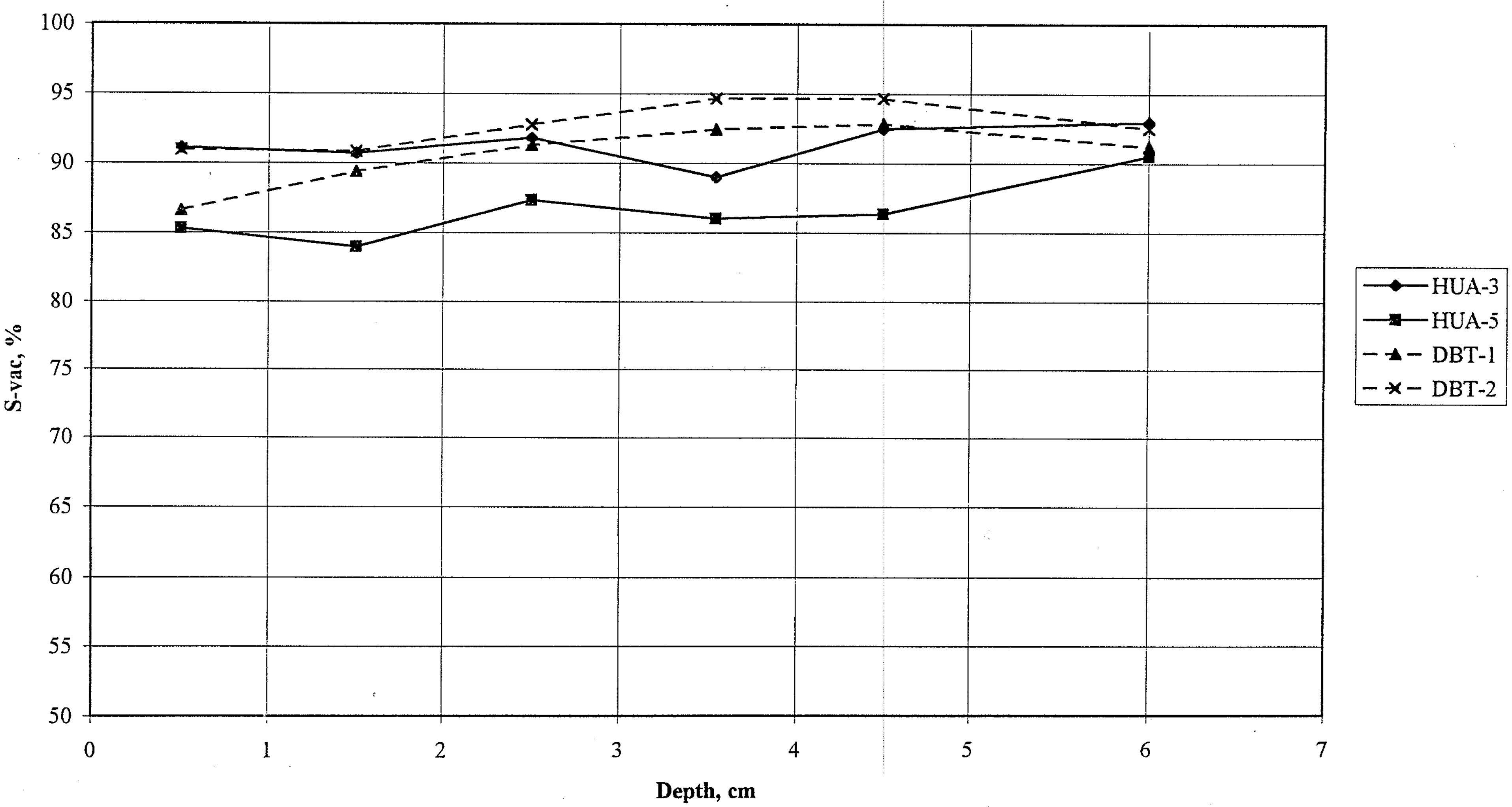
D0 Ark6 Chart 9



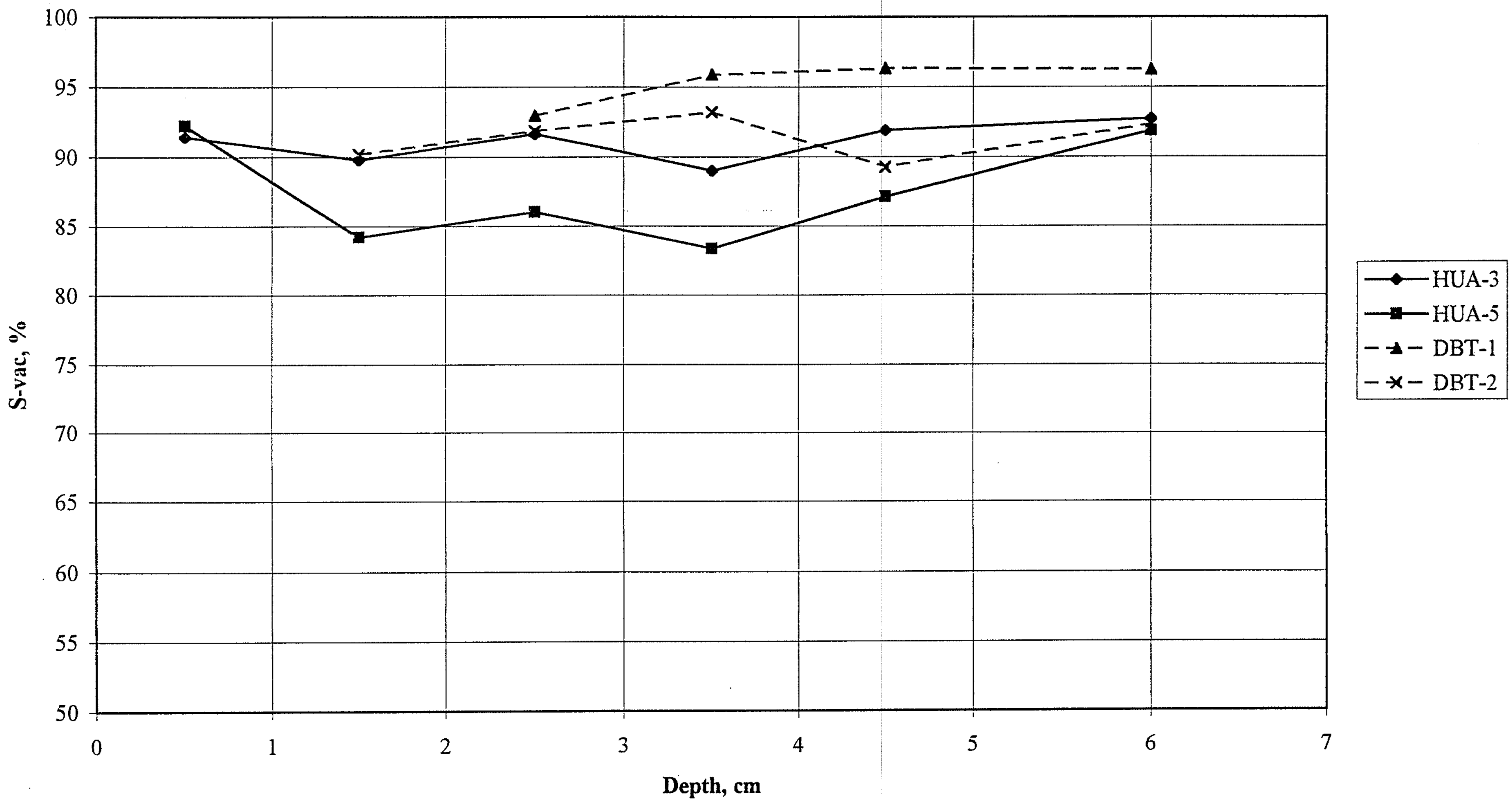
D1 Ark6 Chart 10



D1/S1 Ark6 Chart 12



D1/S1/T1 Ark6 Chart 13



Appendix 7

Scaling during frost testing

Table 1

Specimen no.	Scaled material [kg/m³]				
	7 cycl. (96.06.07)	14 cycl. (96.06.14)	21 cycl. (96.06.21)	28 cycl. (96.06.28)	35 cycl. (96.07.05)
DBT-1 3-1 D1/S1	0.09	0.66	1.34	2.52	3.64
3-2 T1	0.07	1.30	2.03	2.37	2.64
DBT-2 6-1 D1/S1	0.14	1.00	2.20	3.31	4.75
6-2 T1	0.10	1.02	2.19	3.49	4.72
HUA-5 28-1 D1/S1	0.00	0.02	0.02	0.03	0.04
28-2 T1	0.01	0.02	0.03	0.05	0.05
HUA-3 47-1 D1/S1	0.00	0.01	0.01	0.01	0.01
47-2 T1	0.00	0.00	0.00	0.00	0.00
HUA-3 66-1 D1/S1	0.03	0.06	0.07	0.09	0.10
66-2 T2	0.02	0.03	0.04	0.04	0.04

Table 2

Specimen no.	Scaled material [kg/m³]			
	7 cycl. (96.06.24)	14 cycl. (96.07.01)	21 cycl. (96.07.08)	28 cycl. (96.07.15)
HUA-3 42-1 D0/T1	0.00	0.00	0.00	0.01
42-2	0.00	0.00	0.00	0.01
HUA-3 70-1 D2/SI	0.00	0.00	0.00	0.00
70-2 T1	0.00	0.00	0.01	0.01

Appendix 8

Chloride profiles

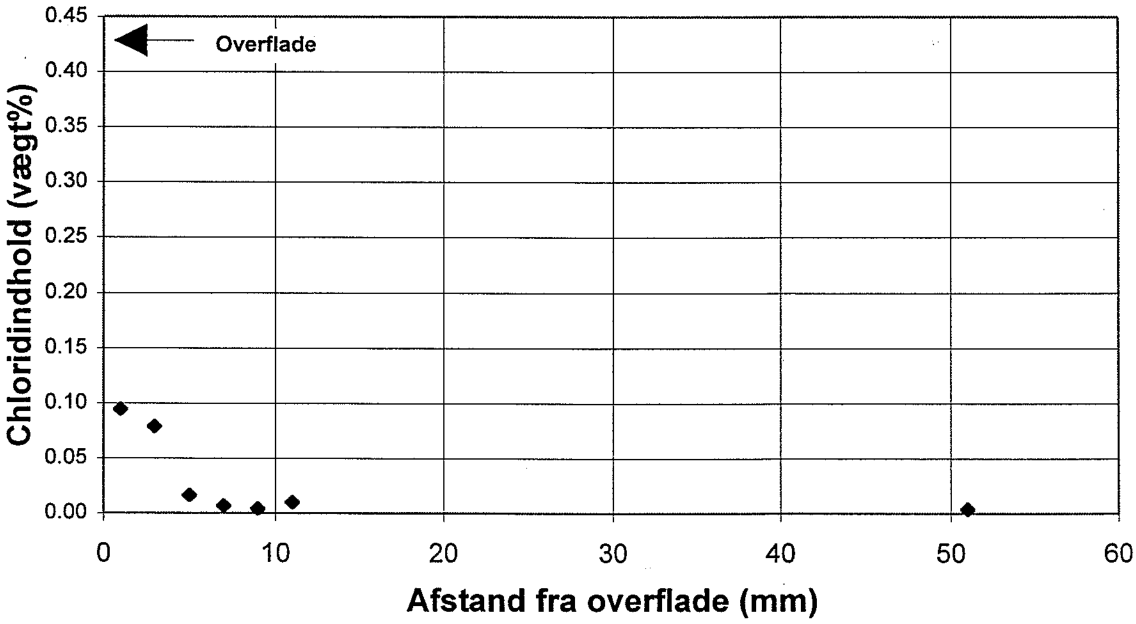
Chloridbestemmelse

G.M. Idorn Consult, RAMBØLL

Udført af:	CSP/MID	Sagsnr.:	951587
Dato:	1996.08.06	Labnr.:	11896
Kunde ref.:	HETEK		

Kolbe nr.	Prøve nr.	Lokalitet	Afstand fra overflade (mm	Tørvægt (gram)	AgNO3 0.1 N (ml)	NH4SCN 0.1N (ml)	Chlorid (vægt%)
1	11896	HUA3, D1/S1/T1	1	5.50	2.43	0.96	0.09
2	-	-	3	6.23	2.48	1.09	0.08
3	-	-	5	6.40	1.03	0.73	0.02
4	-	-	7	6.74	0.84	0.71	0.01
5	-	-	9	7.19	0.77	0.68	0.00
6	-	-	11	6.66	1.18	0.99	0.01
7	-	-	51	13.63	0.88	0.72	0.00

CHLORIDPROFIL GENNEM BETON

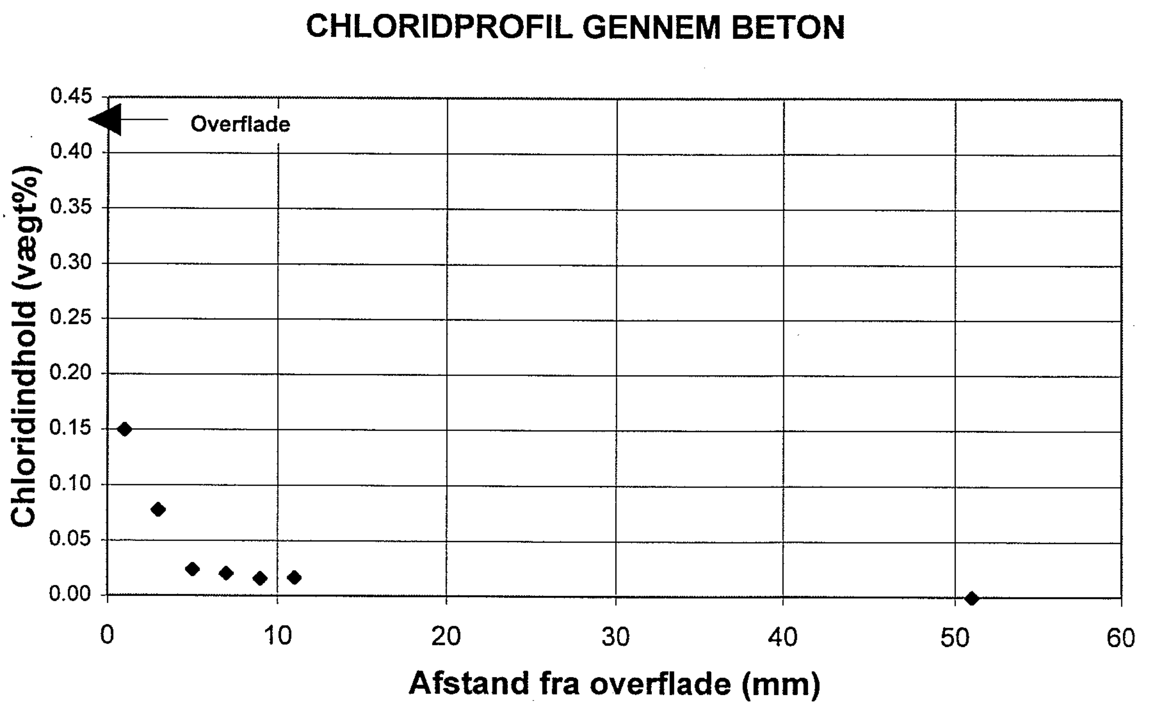


Chloridbestemmelse

G.M. Idorn Consult, RAMBØLL

Udført af:	CSP/MID	Sagsnr.:	951587
Dato:	1996.08.06	Labnr.:	11996
Kunde ref.:	HETEK		

Kolbe nr.	Prøve nr.	Lokalitet	Afstand fra overflade (mm	Tørvægt (gram)	AgNO3 0.1 N (ml)	NH4SCN 0.1N (ml)	Chlorid (vægt%)
1	11996	HUA5, D1/S1/T1	1	7.95	5.15	1.79	0.15
2	-	-	3	7.51	3.16	1.52	0.08
3	-	-	5	6.14	1.78	1.37	0.02
4	-	-	7	8.84	3.02	2.51	0.02
5	-	-	9	7.71	2.30	1.96	0.02
6	-	-	11	8.71	1.64	1.23	0.02
7	-	-	51	20.89	1.96	1.98	0.00

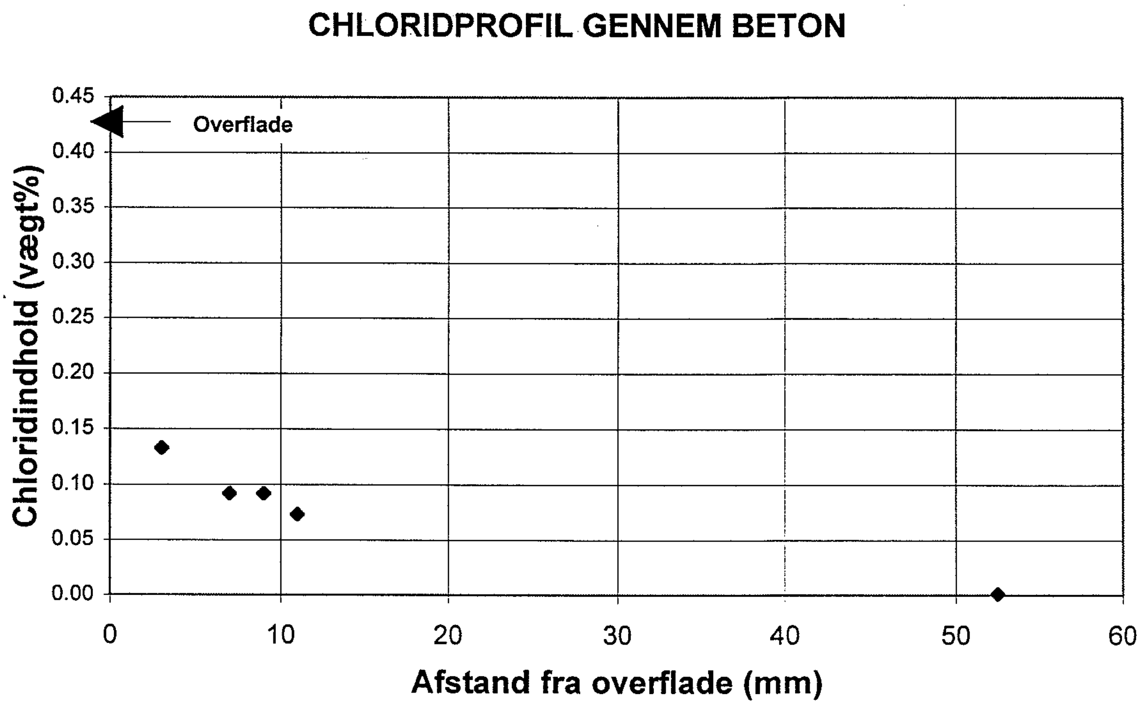


Chloridbestemmelse

G.M. Idorn Consult, RAMBØLL

Udført af:	CSP/MID	Sagsnr.:	951587
Dato:	1996.08.06	Labnr.:	12096
Kunde ref.:	HETEK		

Kolbe nr.	Prøve nr.	Lokalitet	Afstand fra overflade (mm	Tørvægt (gram)	AgNO3 0.1 N (ml)	NH4SCN 0.1N (ml)	Chlorid (vægt%)
1	12096	DBT2	3	9.82	6.23	2.55	0.13
2	-	-	7	5.55	3.23	1.79	0.09
3	-	-	9	6.86	4.69	2.91	0.09
4	-	-	11	6.06	3.06	1.81	0.07
5	-	-	53	20.11	1.31	1.23	0.00



Appendix 9

Thin section petrography

Thin section petrography was performed on the following samples:

Sample	GMIC Identification
No 10 HUA 3 Do/DI/SI	11196
No 19 HUA 3 DI/SI/TI	11296
No 73 Do/DI/SI	11396
No 81 HUA 5 DI/SI/TI	11496
No 118 DBT 2 Do/DI/SI	11596
No 116 DBT 2 DI/SI/TI	11696
No 99 DBT 1 DI/SI/TI	11796

The sections were placed perpendicular to the exposed surfaces of the samples.

Observations

There were no discernible difference in the microstructure of the sample with varying conditioning history.

Neither microcracking nor increased capillar porosity was observed. It should be emphasized that a possible coarsening of the capillary pores and the gel pores cannot be observed by fluorescence microscopy.

The effect of Borås freeze-testing is very pronounced in the non-air entrained DBT-samples.

The surface is completing to several mm depth and stone and sand particles are separated from the cement paste.

Coarse and fine cracks are found subparallel to the exposed surface further down in the samples. The cracks propagate mainly in the paste and around the aggregate particles.

Further down in the samples gaps completely surrounding the coarse aggregate particles are seen. This phenomenon occurs to the full depth of the thin sections (5cm).

The gaps are created by expansion of the cement paste that pulls away from the coarse aggregate particles. From this it can be deduced that the Borås freeze-thaw testing causes both surface scaling and internal cracking and expansion in non-frost resistant concrete.

The HUA-samples were air entrained and had lower w/c-ratio than the DBT-samples (~ 0.35 against ~0.45).

There were no signs of interior cracking in the frost tested HUA-samples.

Furthermore surface cracking was only observed around one sand grain in one sample. The Borås-testing had little or no effect on the HUA-samples.

PHOTO DOCUMENTATION

Case number: 951587 **GMIC lab. no.:** 11196 **Sample No.:** 10

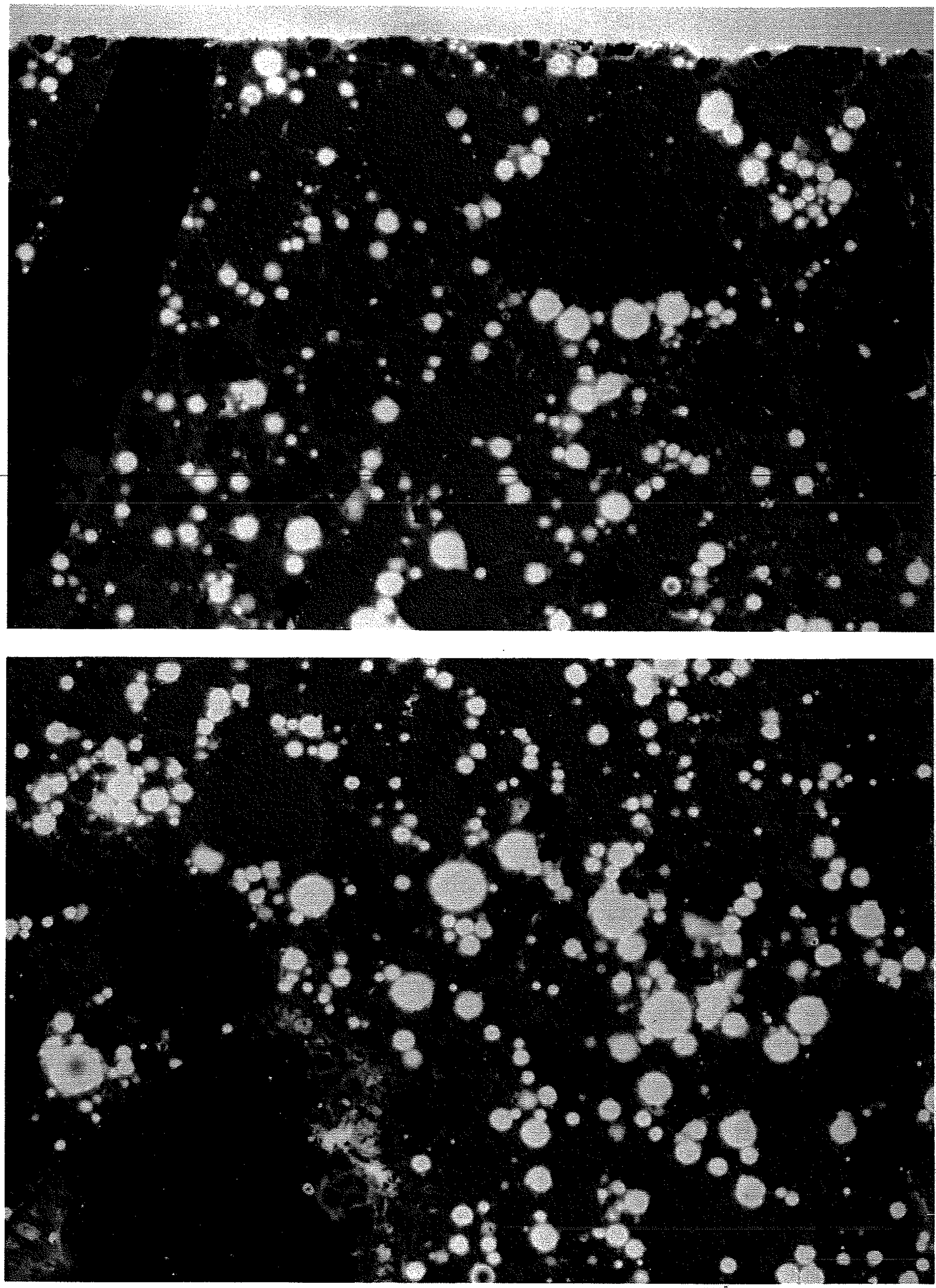


Figure 1 General appearance of the concrete surface and centre of thin section as viewed in fluorescent light. The sample is air entrained. No cracks observed. Field of view: 6 x 9 mm.

PHOTO DOCUMENTATION

Case number: 951587 **GMIC lab. no.:** 11296 **Sample No.:** 19

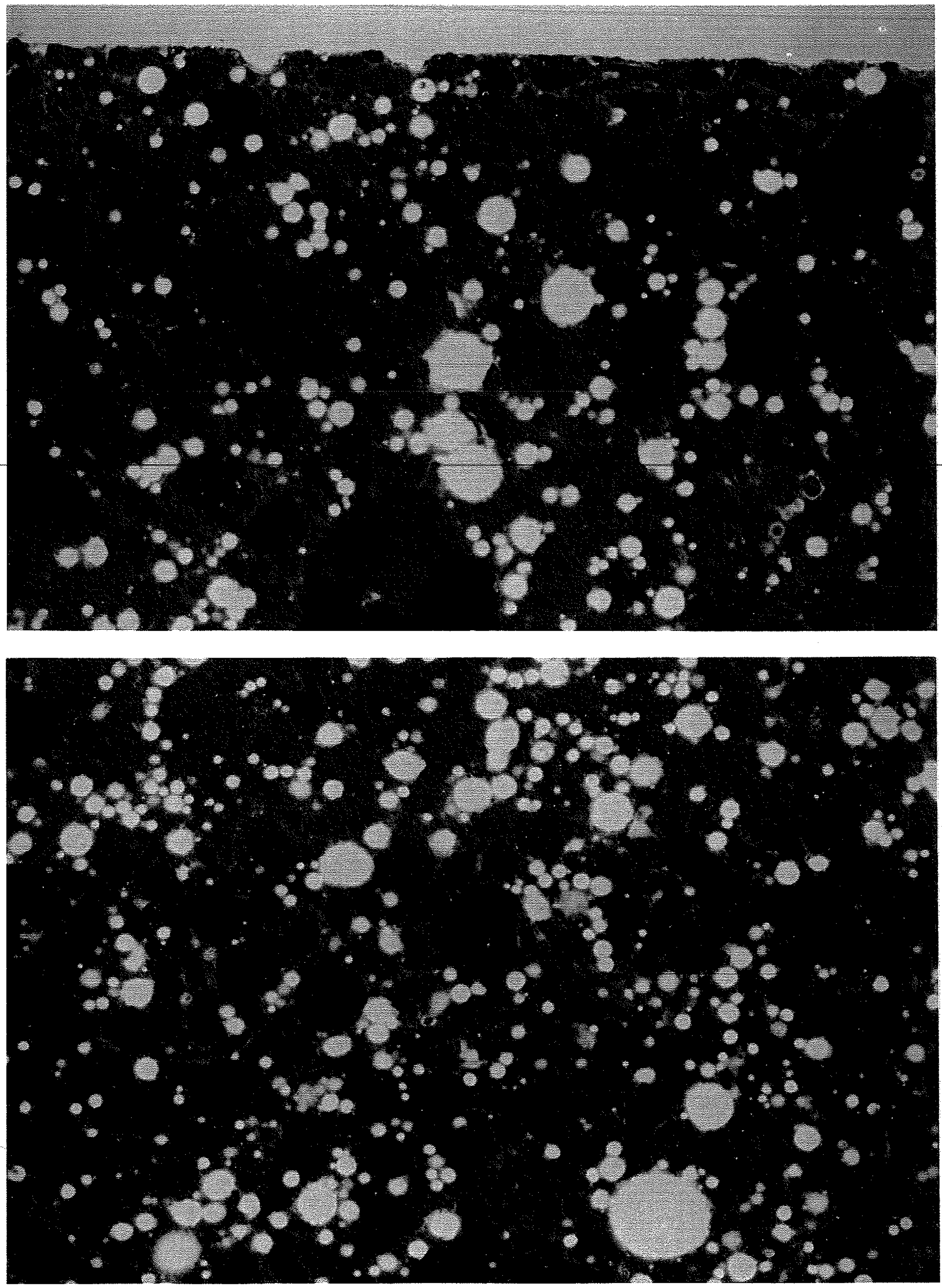


Figure 2 General appearance of the concrete surface and centre of thin section as viewed in fluorescent light. The sample is air entrained. No cracks observed. Field of view: 6 x 9 mm.

PHOTO DOCUMENTATION

Case number: 951587 **GMIC lab. no.:** 11396 **Sample No.:** 73

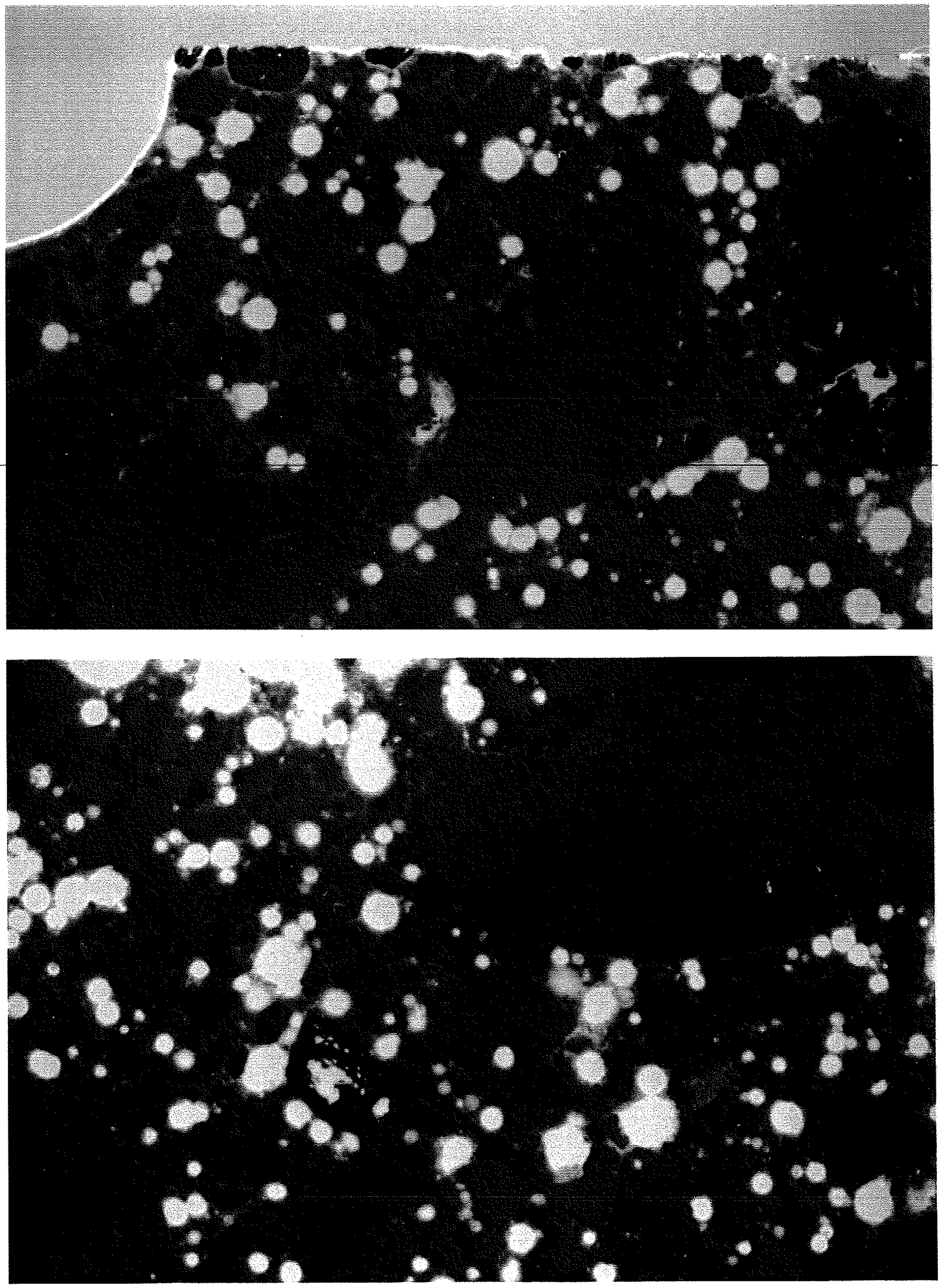


Figure 3 General appearance of the concrete surface and centre of thin section as viewed in fluorescent light. The sample is air entrained. No cracks observed. Field of view: 6 x 9 mm.

PHOTO DOCUMENTATION

Case number: 951587 **GMIC lab. no.:** 11496 **Sample No.:** 81

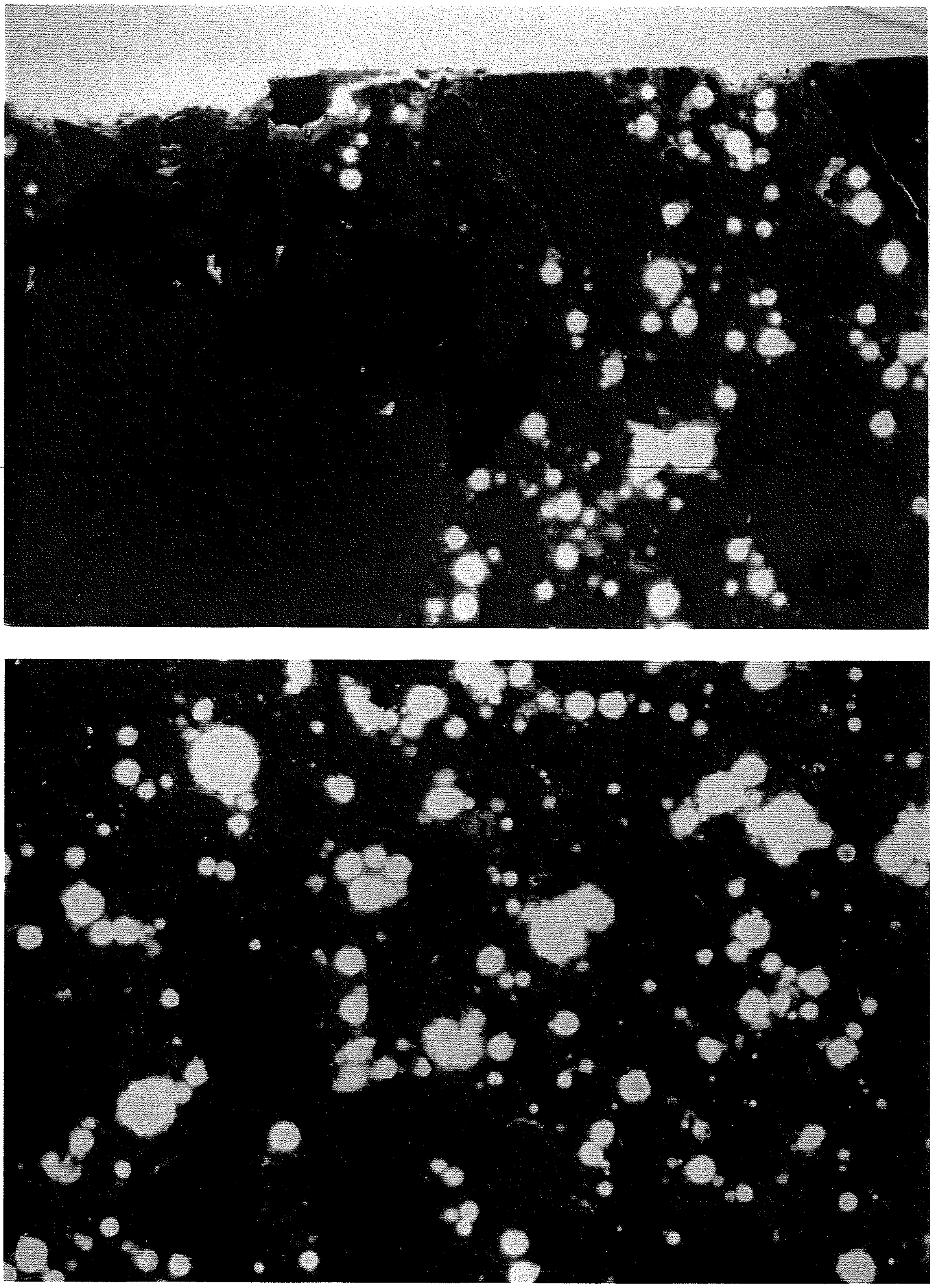


Figure 4 General appearance of the concrete surface and centre of thin section as viewed in fluorescent light. The sample is air entrained. One little crack is observed in the surface. Field of view: 6 x 9 mm.

PHOTO DOCUMENTATION

Case number: 951587 **GMIC lab. no.:** 11596 **Sample No.:** 118

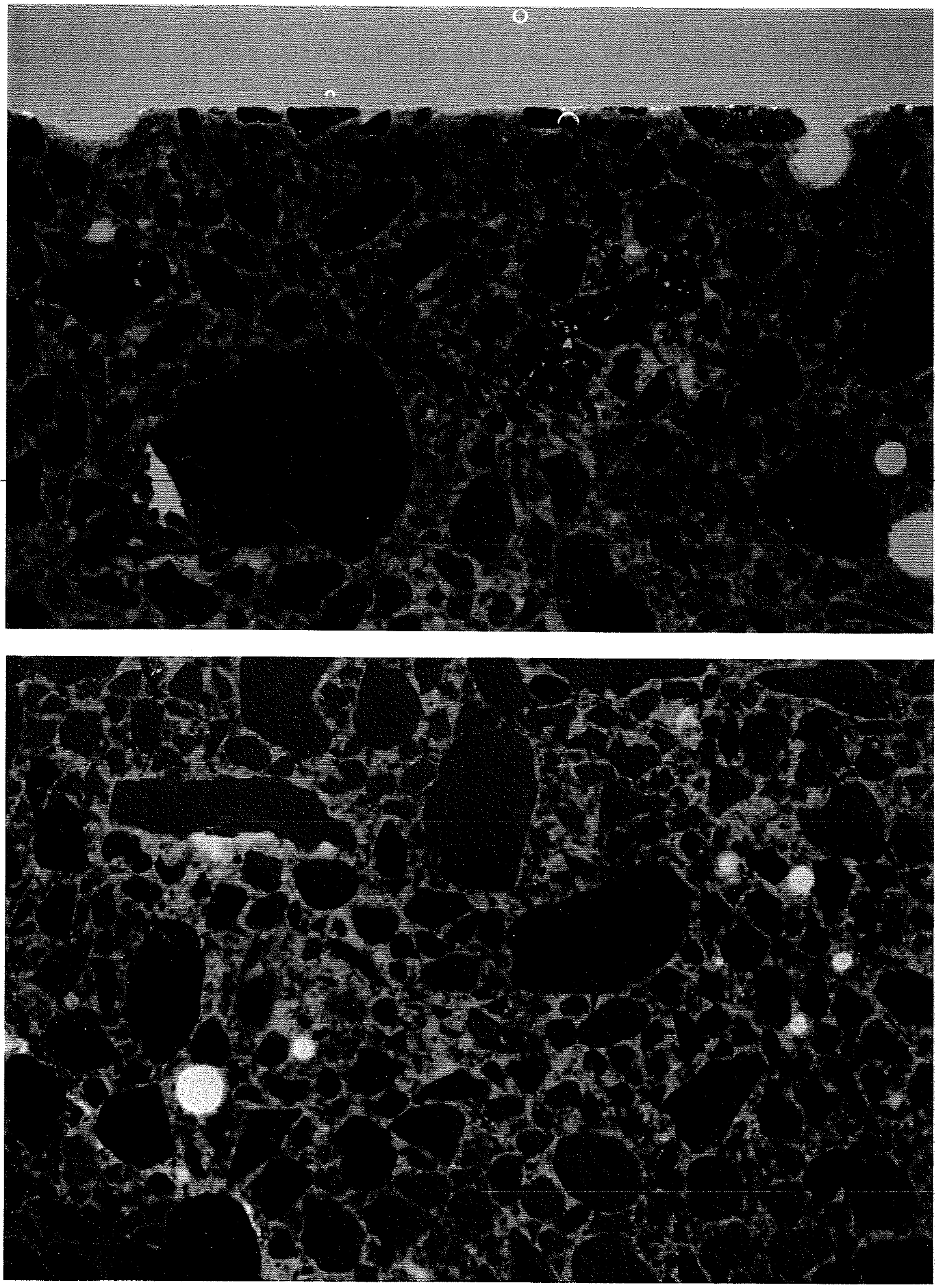


Figure 5 General appearance of the concrete surface and centre of thin section as viewed in fluorescent light. The sample contains little air. No cracks observed. Field of view: 6 x 9 mm.

PHOTO DOCUMENTATION

Case number: 951587 **GMIC lab. no.:** 11696 **Sample No.:** 116

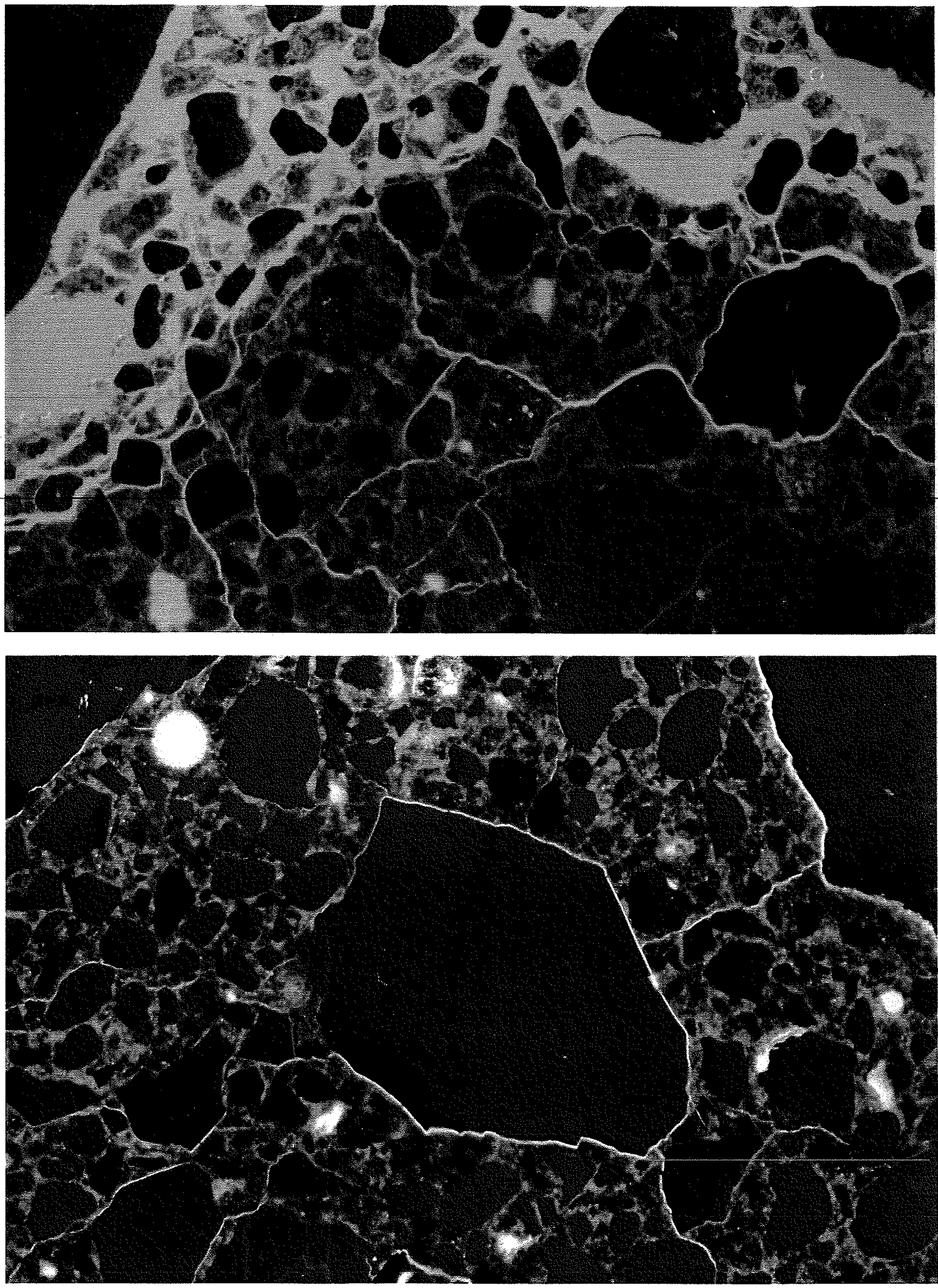
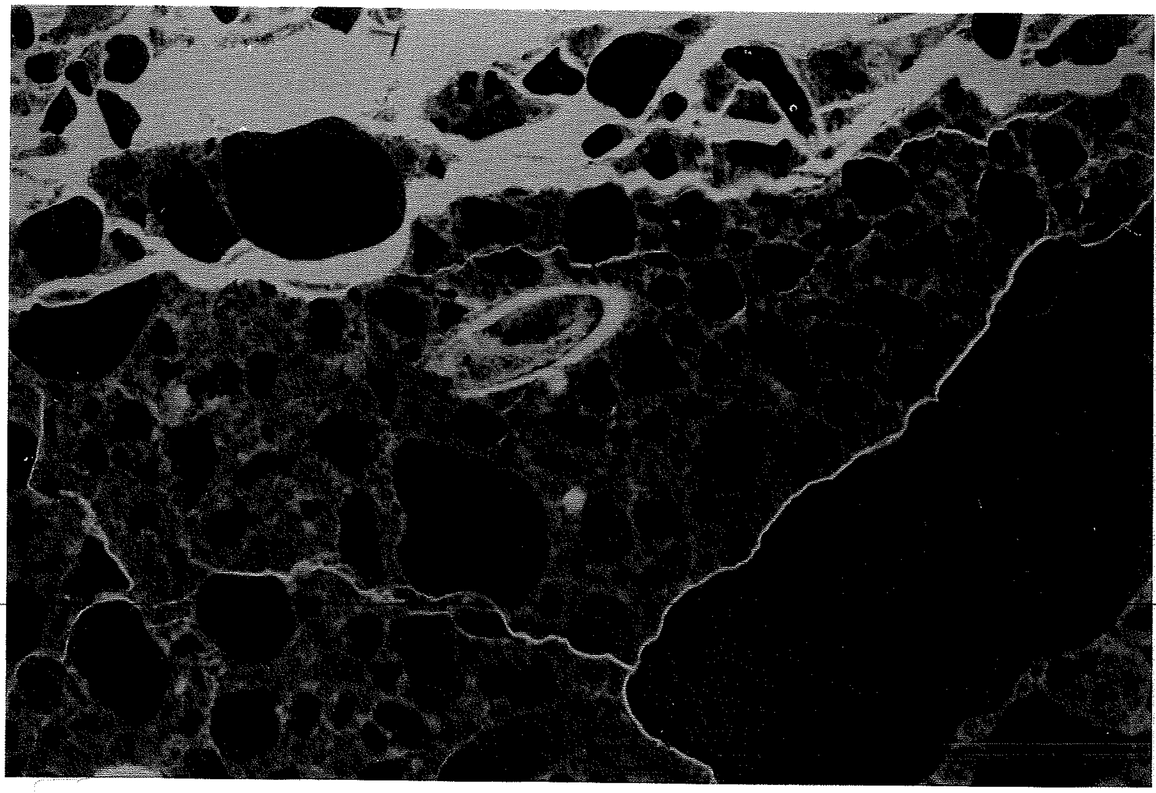


Figure 6 General appearance of the concrete surface and centre of thin section as viewed in fluorescent light. The sample contains little air. Cracks are observed in paste. Loose aggregate particles in surface. Gaps around aggregates. Field of view: 6 x 9 mm.

PHOTO DOCUMENTATION

Case number: 951587 **GMIC lab. no.:** 11796 **Sample No.:** 99



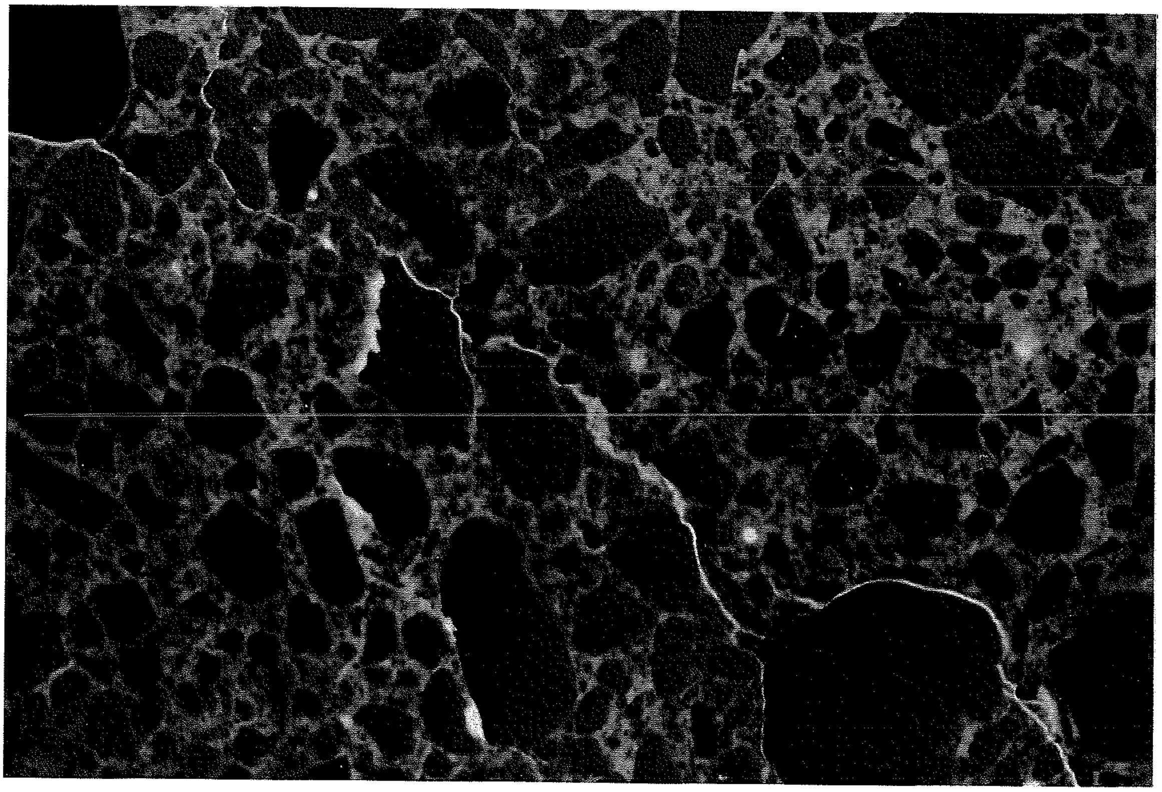


Figure 7 General appearance of the concrete surface and centre of thin section as viewed in fluorescent light. The sample contains little air. Cracks are observed in paste. Loose aggregate particles in surface. Gaps around aggregates. Field of view: 6 x 9 mm.