Study Project
Ice Forces against Offshore Structures

Report No. 1

Evaluation of Extreme Ice Forces on a Lighthouse in the Bothnian Bay

A study of the Björnklack overloading event in April 1985

VBB

National Administration of Shipping and Navigation, Sweden

University of Luleå

VBB Project M7334   Final Report Jan. 15th, 1987

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FOREWORD

The present report is the first in a series of technical reports presenting the essentials of premises, methodology and findings from the work carried out within the frame of the study project ICE FORCES AGAINST OFFSHORE STRUCTURES.

The project is sponsored by the following participating organizations:

- Arco Oil and Gas Company, USA
- Exxon Production Research Company, USA
- Mobil Research and Development Corporation, USA
- Norwegian Contractors, Norway
- Canadian Coast Guard, Canada
- Hamburgische Shiffbau-Versuchsanstalt, West Germany
- Mitsubishi Heavy Industries, Ltd, Japan
- The National Administration of Shipping and Navigation (NASAN), Sweden.

The project is also sponsored by the Swedish National Industrial Board.

The present study has been carried out by a working group with members from VBB, NASAN and Luleå University under the direction of Alf Engelbrektson, Ivar Berglund and Lennart Fransson from the respective organization. The responsibilities have been assigned essentially as follows:

- Planning, analyses and preparation of the main report: VBB

- Site inspection incl. divers inspection and preparation of inspection report: NASAN

- Field observations, collection of ice samples, laboratory investigations, preparation of the report on ice conditions and ice properties: Luleå University

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Ice Forces

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ABSTRACT

The analysis of an unexpected case of extreme ice forces overloading a small lighthouse in the Bothnian Bay has provided a very useful basis for future work within the study project ICE FORCES AGAINST OFFSHORE STRUCTURES.

The most remarkable conditions associated with the event were an extreme combination of ice thickness, ice strength and ice movement and the very course of events, which admitted detailed observations of the development of the ice situation, investigations of ice properties and evaluation of the failure mode of the lighthouse structure. From these findings it has been possible to determine, within a narrow range of confidence, the maximum ice load as well as the related ice thickness and ice sample strength etc. Thus the effective ice pressure exerted on the structure could be compared with the compressive strength determined in the laboratory.

The case study has also drawn attention to the importance of probabilistic and risk considerations in ice resistant design and it has given rise to some hypothetical assumptions to be checked in the future work.

1. Scope

The present report deals with observations made in connection with a case of extreme ice forces on a lighthouse in the Bothnian Bay and with conclusions, which could be drawn on the basis of these observations, partly against the background of previous experiences of severe ice action on structures in the same area.

The report has been produced within the frame of the study project ICE FORCES AGAINST OFFSHORE STRUCTURES and is intended to be one of two conclusive reports summarizing the results of works carried out within Task No. 1: Analysis of existing data.

As stated in the project description (Ref. 1), Task No. 1 was initially intended to be concentrated on the analysis of ice-induced vibrations on the basis of available records of accelerations measured on board the NORSTRÖMSGRUND lighthouse in the Bothnian Bay. Although the "existing data" are not only related to vibrational responses of lighthouses but also to more general actions of ice on such structures, the intention was to deal with "static" ice forces later, when more precise data are expected to be available as a result of Tasks Nos 2 and 4: "Improvement and extension of the instrumentation", notably for measuring static or guasi-static responses.

However, after the programme was written in 1984, an extraordinarily interesting case of overloading occurred during the severe winter of 1984-85, when the lighthouse BJÖRNKLACK, located not far from NORSTRÖMSGRUND, was displaced due to ice forces exerted by a large floe of ice under the stress of a strong wind. Among earlier cases of documented damage due to ice action, this case is unique in the following respects primarily:

- The winter was exceptionally cold and the ice cover correspondingly thick. By additional coincidence the ice thrust on the lighthouse became extreme.
- Due to the particular mode of structural failure, the maximum ice load can be estimated within a narrow range of confidence.
- The ice situation and ice properties at the overloading event could be exhaustively investigated thanks to a stationary ice situation after the event.

This unexpected opportunity has thus been taken and the present report has been written with two main purposes:

- To check current ice force prediction theories against a well documented case of severe ice action.
- To obtain a suitable basis for further studies of ice forces on structures, including the parametric dependence to the largest possible extent.

Thus, the present report may be regarded as the first presentation of experiences of "static" ice forces within the project. For more detailed information about previous experiences, reference is made to Ref. 2 and 3.

2. Description of the Björnklack lighthouse

A section through the BJÖRNKLACK lighthouse is shown in Figure 2.1.

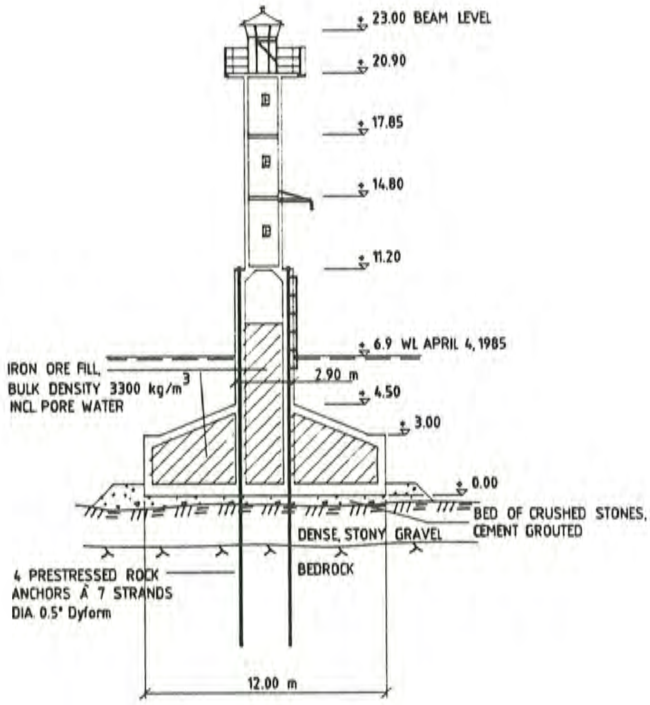

Figure 2.1 Section through the BJÖRNKLACK lighthouse

In a classification system of Swedish lighthouses with respect to sizes and equipment into large, medium-sized and small structures, the BJÖRNKLACK should be placed in the last class, which contains a great number of of similar structures located in different parts of the Baltic. Subgrouping the objects, however, roughly with respect to the degree of exposure to severe ice action, the number of objects in the same group as BJÖRN-KLACK is limited to less than 10. Of particular interest is the lighthouse BORUSSIAGRUND, which can be regarded as a twin of BJÖRNKLACK, having an identical structural design. The two light-houses were installed in 1969 to serve as lead marks along two alternative fairways to the Luleå harbour, the sites being located not more than 9 NM apart (see Figures 2.2 and 2.3).

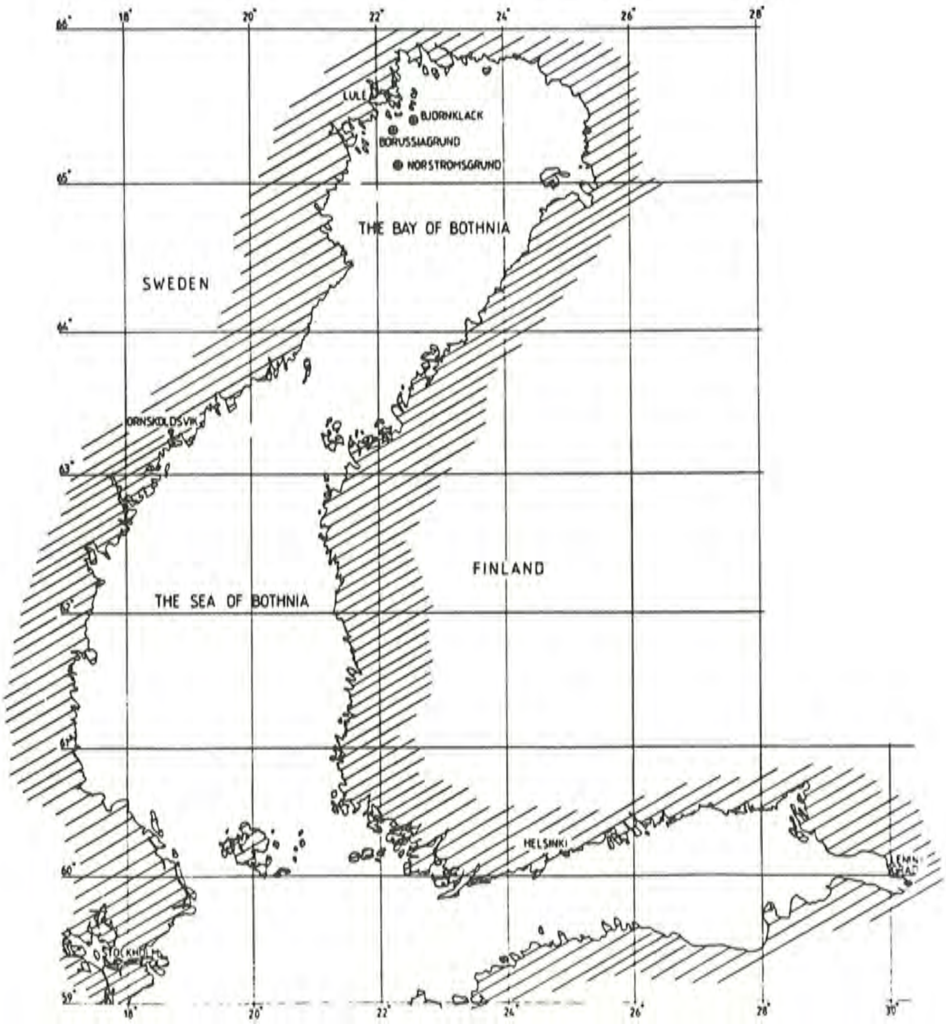

Figure 2.2 Map of the Bothnian Sea and Bay with lighthouses mentioned in the report

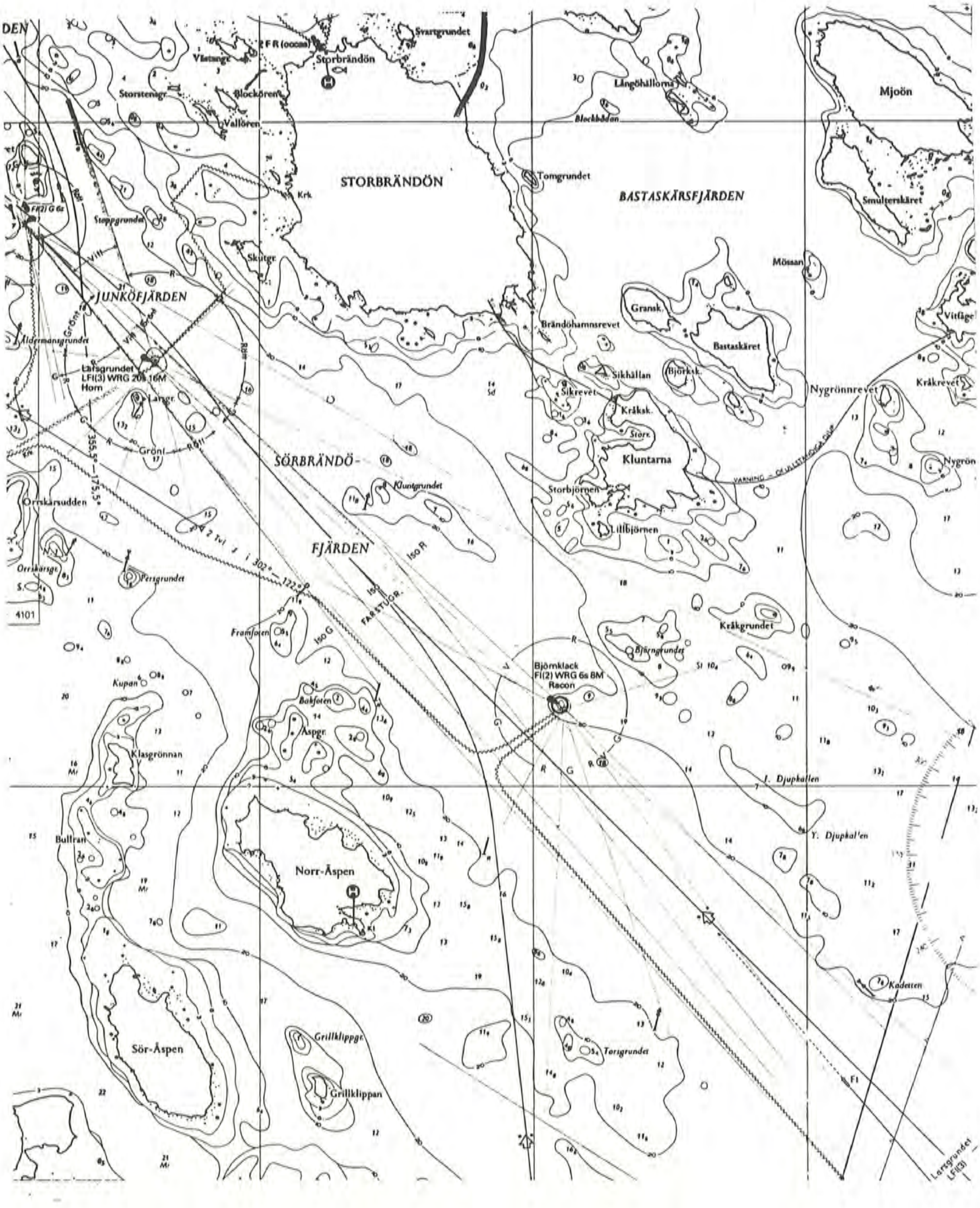

Figure 2.3 Fairways to Luleå harbour (chart extract)

Initially, the two lighthouses were designed as pure gravity structures, to be stabilized partly by iron fill, which happened to be available at an unusually low price at the time of construction. However, during the first winter of operation both structures were moved some inches by the ice, indicating clearly that the ice thrust was underestimated. Therefore the structures were provided with prestressed rock anchors, increasing the ultimate ice load capability with about 60 per cent. Since then the two structures resisted the ice successfully until the damage to the BJÖRNKLACK lighthouse in April 1985.

Four VSL tendons, each with seven 0,5" strands, were provided as rock anchors. The tendons were placed in bore holes, drilled vertically from the entrance platform, through the concrete structure and the underlying soil and into the rock (see Figure 2.1). Above the anchorage zone the greased strands were coated with plastic sheaths, preventing bond between the steel wires and the cement grout, which was injected into the bore holes immediately after the mounting of the tendons. The grouting could thus be performed in one single operation and the tendons could be subsequently tensioned.

The tendons were tensioned initially to about 70 per cent of the ultimate strength. Afterwards it has been checked periodically that the loss of prestress has been within intended limits, i.e. that there have not been any exceptional settlements of the soil below the caisson etc.

3. The overloading event

After 14 years without any damage the BJÖRNKLACK lighthouse became overloaded on the 4th of April 1985, when a large ice floe was pressed by wind drag and currents against the structure. After the ice floe, released from the surrounding continuous ice cover by a broken ice channel and two crossing cracks, had been somewhat indented by the lightpier, the ice thrust became too strong and the lighthouse was moved from its bed to a sloping part of the adjacent sea bed, where it came to rest in an inclined position as shown in Figure 3.1.

Figure 3.1 The frozen situation a few days after the overloading event

The course of events, from the initial ice movements to the stationary situation after the displacement, when the movement of the ice floe was restricted at the fixed boundary of the broken channel, was followed by several observers, as reported in ref. 4 (Appendix 1). On the basis of these observations and the mapping of the stationary situation a few days after the ice movement, it has been possible to obtain a rather detailed pictures of the course of events, both with respect to the ice conditions and the behaviour of the lighthouse structure. As described in the following text, it has been possible to trace the structural failure mode and to estimate the maximum ice force and its basic parametric composition. The observations have also provided a useful basis for introductory considerations of a probabilistic nature regarding the climatic and other conditions behind the development of an extreme ice load on a structure as in the actual case.

4. Local ice conditions

4.1 Field investigations and observations

A few days after the overloading event the lighthouse and the surrounding ice cover were inspected by personnel from the Administration of Shipping as well as from the University of Luleå. The inclination of the lighthouse as well as the thickness of the ice cover at various points were measured and photographs were taken, Figures 4.1-4.3. On the 10th of April the university staff went out and collected ice cores for laboratory investigations.

Figure 4.1 View from E, April 6th 1985

Figure 4.2 View from NW, April 6th 1985

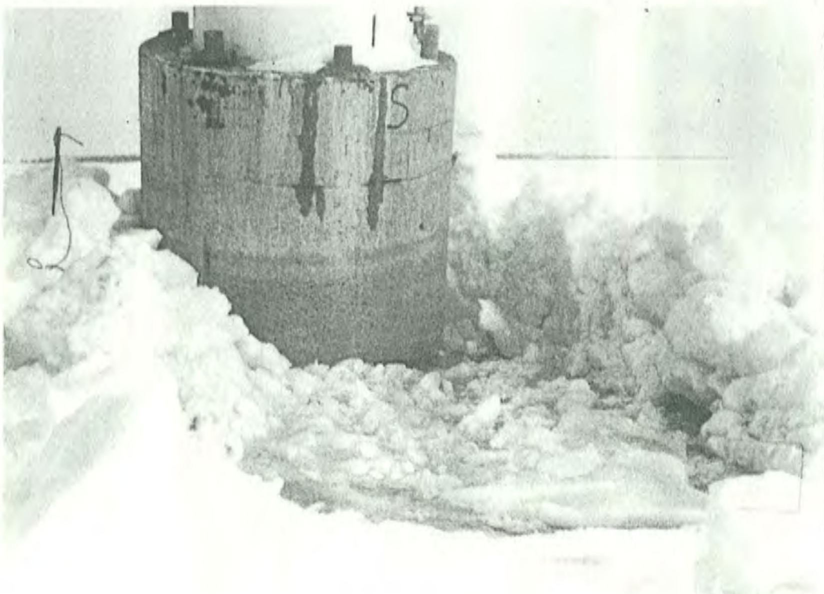

Figure 4.3 View from SW, April 6th 1985

The observations made by the university staff have been reported separately in Appendix 1. This report together with notations and verbal descriptions by Ivar Berglund of the Administration of Shipping, who visited the site on the 6th of April, have constituted the basis for the analysis.

Information regarding wind speed, air temperature and water levels have been provided by the Swedish Meteorological and Hydrological Institute (SMHI) and the Swedish Air Force.

The Administration of Shipping, notably the Ice Breaker Service, and SMHI have also provided information about the large scale ice situation, e.g. as illustrated in Figure 4.4.

4.2 Laboratory investigation of ice samples

Samples from the ice cores drilled out from the ice cover close to the lighthouse were tested in the ice laboratory of the University of Luleå. The main purpose was to study the structural composition of the ice and to determine the uniaxial compressive strength at ice temperatures representative for conditions on the lighthouse site on the 4th of April.

The results of the laboratory investigations have been reported in Appendix 1. The most important findings are commented in connection with the analytical considerations dealt with in the following sections.

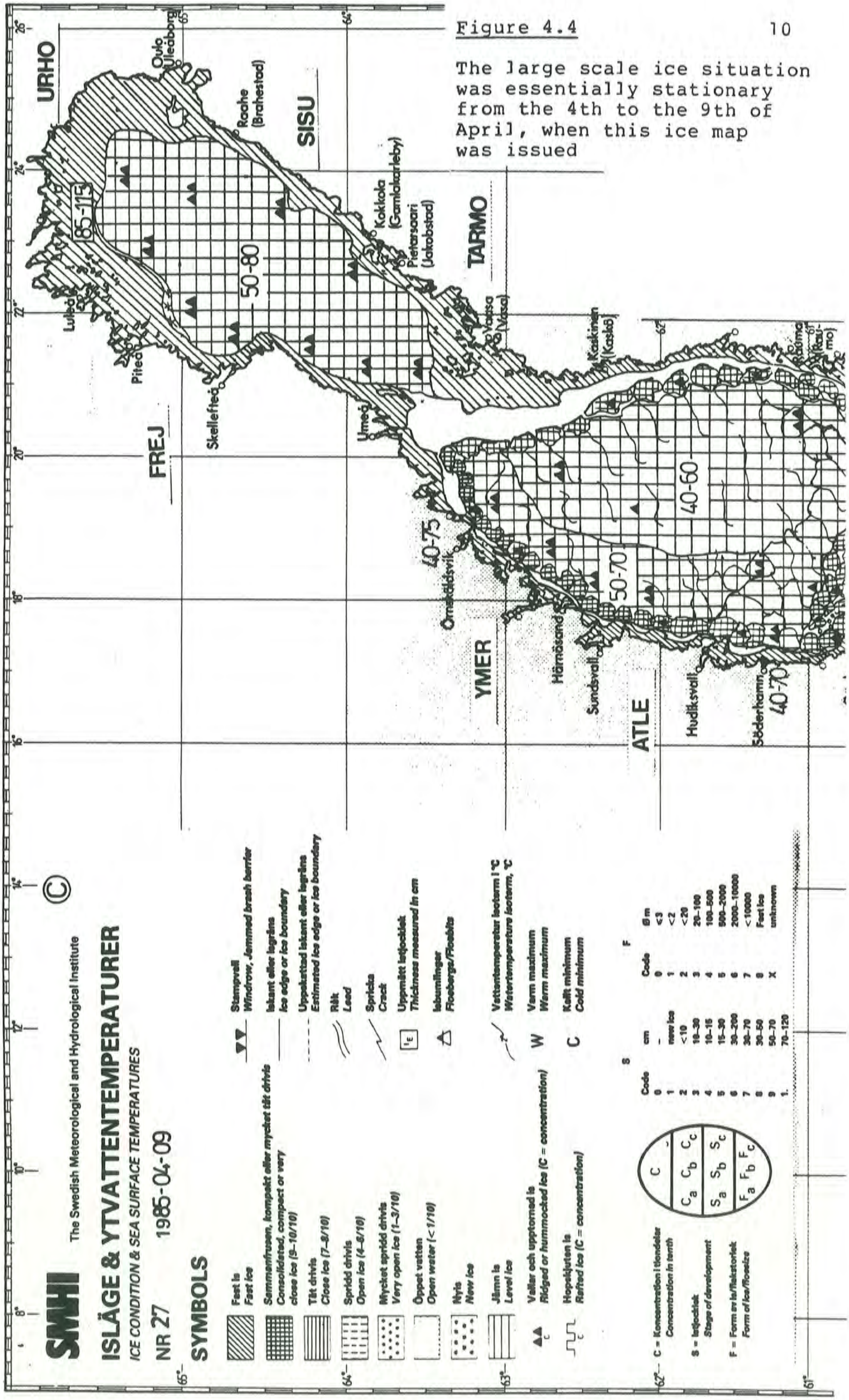

Figure 4.4

The large scale ice situation was essentially stationary from the 4th to the 9th of April, when this ice map was issued

5. Structural failure mode and ultimate ice load

5.1 Diver's inspection

The failure load can be determined with a fairly good accuracy on the basis of evidence gathered during the divers inspection of the sea bed after the overloading event.

The following observations are of primary interest:

- The lighthouse structure had been displaced about 17 m. All the rock anchors had ruptured at approximately the same level not far from the top of the cement grouted stone bed. No other damage was detected on the structure which had obviously slided southwards, first on the bed and then on the sloping sea bed, see Figure 5.1. The southernmost part of the stone bed had yielded. The structure had ploughed up some soil from the sea bed before it had come to rest with an inclination of 12° from vertical.

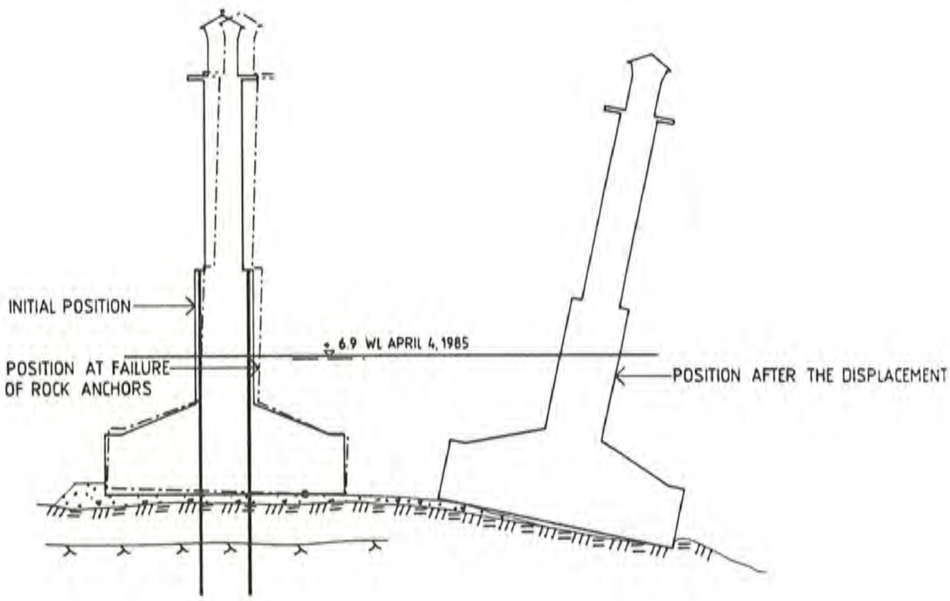

Figure 5.1 Reconstruction of the overloading event

-All the strands protrude between 10 and 15 cm from the top of the grouted stone bed (see Figure 5.2).

Figure 5.2 Part of grouted stone bed with protruding ends of a ruptured rock anchor tendon

- The strands have been bent southwards corresponding to a lateral displacement of about 10 cm. The bending radii below the surface of the bed are relatively large since the strands have indented the bed, but smaller bending radii were observed at the ruptured ends of the strands (see Figure 5.3).

- The strands were not affected by corrosion.

- The ruptured ends of plastic sheaths around the strands have been extended before the rupture and displaced southwards. (The elongated ends of the sheaths, which can be seen in Figure 5.2, should not be mistaken for strands.)

5.2 Conclusions regarding the structural behaviour before and after the failure of the rock anchors

The following conclusions can be drawn from the divers observations, regarded in the light of some numerical considerations:

- The lateral displacement of the lighthouse has been preceded by an overturning movement, which is the primary cause of the failure of the rock anchors. (Figure 5.1)
- Before the tendon failure and probably when the major part of the base slab had been lifted off from the bed, the bearing capacity of the bed below the highly compressed southernmost part of the structure was exceeded. The failure of the bed had the character of a minor settlement combined with a lateral displacement of the order of 0,1 m in the ice force direction. (Figure 5.1)
- As a consequence of this motion the anchor force increased corresponding to the reduction of the distance to the centroid of the vertical pressure on the bed. At the same time the lateral displacement introduced bending stresses in the strands due to the angular rotation at the lower face of the base slab. (Figure 5.3)

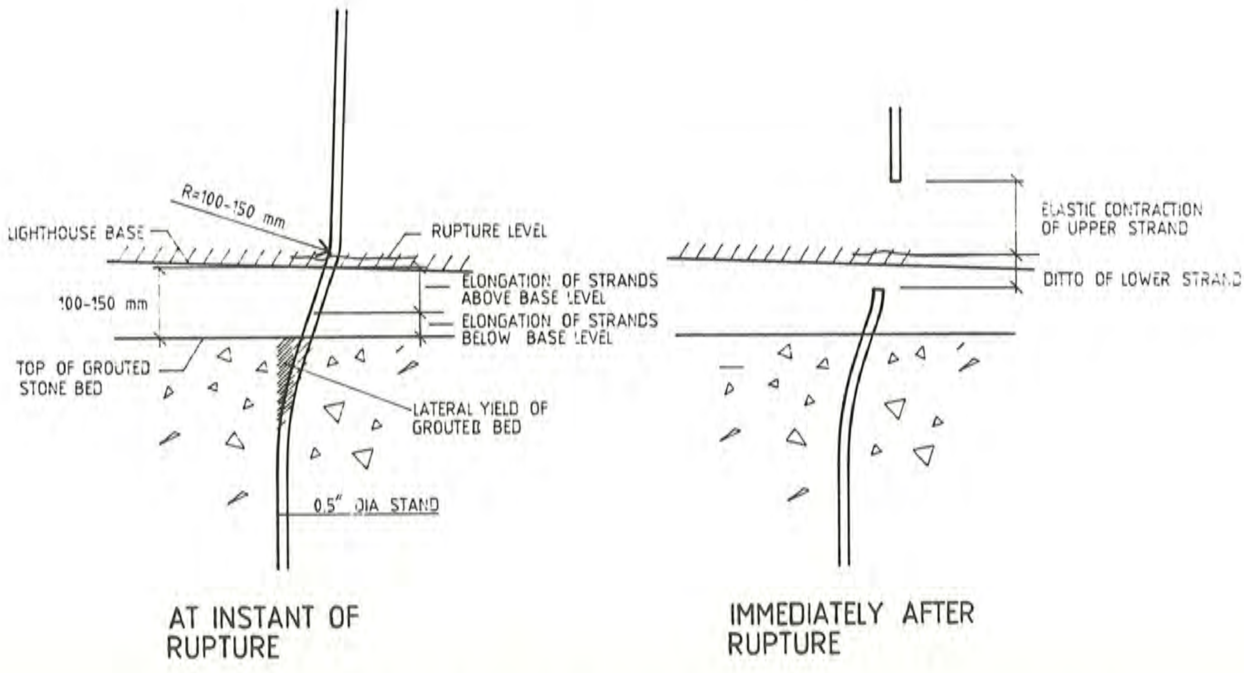

Figure 5.3 Failure mode of rock anchor strands

- After the first rupture due to the ultimate strain level being exceeded by the order of 6 per cent by combined tension and bending, the rest of the strands have failed one by one due to progressing overloading after taking over the load from previously ruptured strands.

The conclusions regarding the mechanism and sequence of failure are further supported by the following numerical considerations:

- Each one of the 4 rock anchors consisted of 7 Dyform dia. 0.5" strands, manufactured by Bridon Wire Ltd, Great Britain. A typical stress-strain relation diagram for such a strand is shown in Figure 5.4. The ultimate tensile strength of a single strand is 220 kN. Since the strands were affected by bending the ultimate tensile load was reduced.

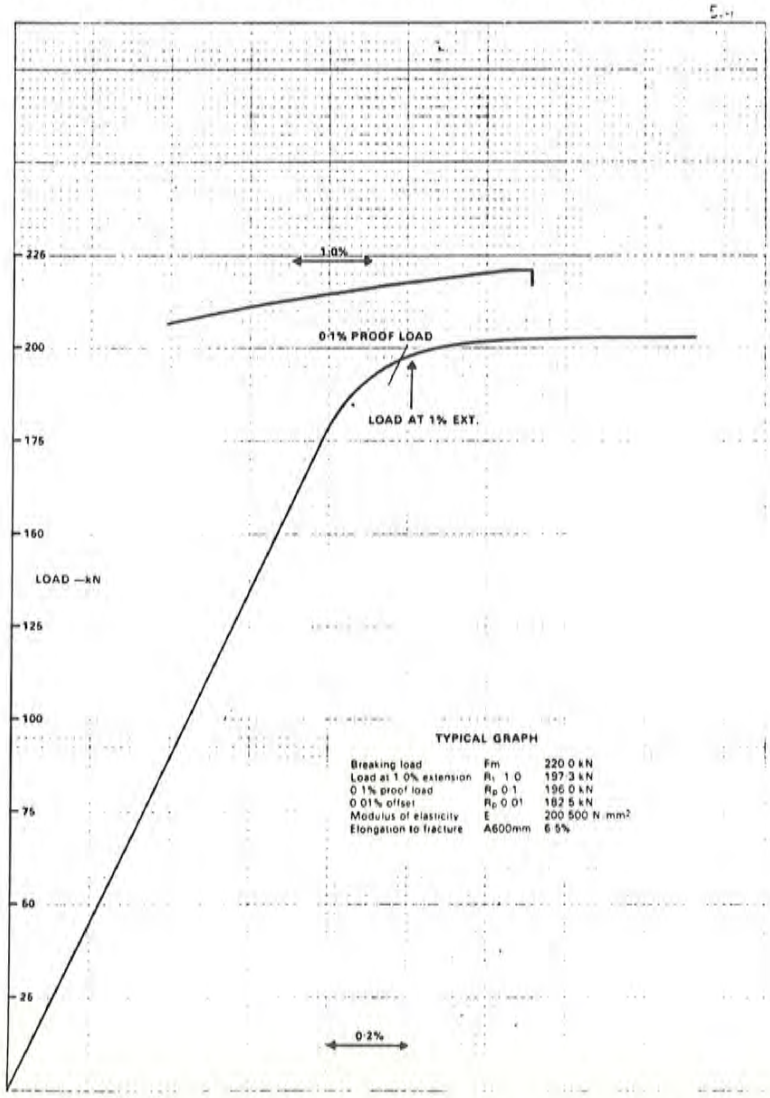

Figure 5.4 Typical stress-strain relation for a rock anchor strand

- The ends of the ruptured strands, protruding 0.10-0.15 m from the bed, indicate that the strands were stressed along their whole lengths somewhat over the elastic limit before failure (see Figure 5.3).

The protruding length is approximately equal to the elongation (caused by the overload) of the strands above the section of failure, i.e. above the base of the lighthouse. Thus the additional tensile strain was obviously between 0.10/11 = 0.009 and 0.15/11 = 0.014.

Since the prestrain was about 0.005 the corresponding load must have been about 200 kN (see Figure 5.4). The estimated actual ultimate force of all the rock anchors together is 4x7x0.20 = 5.6 MN ±3 per cent.

Since the ultimate tensile strain of the strands is expected to be at least 6 per cent, about 5 per cent was obviously consumed by bending, corresponding to a bending radius of

$$R = \frac{dia}{2x\epsilon_b} = \frac{12.7}{2x0.05} = 127 \text{ mm}$$

This bending radius is within the expected range, taking into account the crushing of cement grout due to high lateral contact pressure against the strands at the bend. The radius is also within the range reported by the diver who inspected the broken strands.

The level arm between the centre of the lighthouse and the centroid of the ground pressure is estimated to be 5.0 m ±5 per cent (see Figure 5.1).

The weight of the lighthouse minus the buoyancy force is 8.7 MN ±3 per cent.

The ultimate moment then becomes M_{max}=(8.7+5.6)x5.0= = 71 MNm ±8 per cent.

The ice force resultant acted 6.5 m ±4 per cent from the top of the bed.

Therefore, <u>the ultimate total ice force is estimated at 71/6.5 = 10.9 MN ±12 per cent</u>.

(Ice force peaks with very short duration may have reached above this "ultimate ice load" level, which is a reason for avoiding the notation "maximum" ice load.)

The calculated ice force value 10.9 MN may be regarded as the "most probable" value, realizing that there is a decreasing probability towards the "confidence limits" ±12 per cent.

6. Estimate of ice parameters

6.1 General approach

Once the magnitude of the global ice force has been determined, it is of major interest to study the parametric composition of the force.

As the first approach let us simply express the global force by the relation

$$F = k \cdot f_{um} \cdot h \cdot b = p_{eff} \cdot h \cdot b \qquad (1)$$

where

f_{um} is the mean uniaxial compressive strength in horizontal direction over the whole thickness of the ice sheet in contact with the structure

h is the mean ice thickness in contact

b is the width of the contact area (in the actual case b is considered to be equal to the diameter of the lightpier)

p_{eff} is the "mean effective ice pressure" over the contact area

k is a coefficient representing effects of structural shape, stress conditions in the ice, degree of contact and possibly also other factors governing the relation between the uniaxial strength and the effective ice pressure, i.e.

$$k = p_{eff}/f_{um} \qquad (2)$$

The k factor may, in accordance with current practice, be regarded as a product of various factors accounting for particular conditions as expressed by the relation

$$k = k_S \cdot k_I \cdot k_C \text{ where} \qquad (3)$$

k_S is the "shape factor" for factoring the load with respect to the shape of the obstacle

k_I is the "identation factor" accounting for multiaxial stress conditions in the ice

k_C is the "contact factor" expressing the degree of contact between the ice and the structure.

In the following subsections measured and estimated values of the ice force parameters thus defined are discussed one by one.

6.2 The uniaxial strength of the ice samples

As described in Appendix 1, section 3, test samples were taken from the ice on the 10th of April, i.e. 6 days after the overloading event. Due to essentially unaltered weather conditions the structure of the ice did not change during the week after the 4th of April. The test samples were stored at a temperature of -25°C until testing, which was carried out at temperatures between -0.2 and -4.5°C in accordance with the IAHR standard.

The samples were taken at distances of 10 m and more from the contact surface at overloading, but since the scatter of test values was moderate for samples with the same temperature and from the same depth, the samples are considered to represent the ice at the contact area fairly well.

The test data do not admit any rigorous statistical analysis. Therefore, a simple "characteristic value" approach has been applied.

Thus, the characteristic strength has been based on the mean compressive strength of the cores from samples Nos 2 and 3 after corrections for temperature influences.

The sample No. 2 except one defective core had a mean ice strength of 3.0 MPa at -3.5°C.

Tentatively, corrections for deviating ice temperature were made using the correction term

$$0.46\ (/T_2/^{0.78} - /T_1/^{0,78})$$

for additional strength due to a temperature decrease from T_1 to T_2°C.

This is in accordance with suggestions in ref. 5.

For example, the additional strength between -2.2 and -3.5°C becomes 0.4 MPa, which agrees well with the mean differences between the samples 2 and 3.

The mean ice temperature at the time of overloading can be estimated on the basis of the air temperature record from the 2nd to 4th April, taking into account a considerable time lag between air and ice temperature changes due to the heat insulating effect of the snow cover.

The mean ice temperature on the 4th of April is estimated at about -2°C. The sample No. 3, which had the mean temperature -2.2°C at testing, can be regarded as approximately representative in this respect, although the temperature distribution might have been somewhat different. In any case, since the mean strength 2.6 MPa of this sample equals the mean strength of sample No. 2, when corrected for the temperature deviation, this value is chosen as a best estimate of the uniaxial compressive strength of the ice at the time of overloading.

6.3 The contact area

The profile of the unbroken ice cover along the path of indentation and further to the north along the same line is sketched in Figure 6.1. The profile is based on data presented in Appendix 1. However, the profile has to be regarded as an approximation, since it has been necessary to fit part of it by "best estimate" between the points of measurements adjacent to the indentation channel and points several metres from the unbroken edges at the ends of the channel. The most important feature is of course the thickening of the ice at "the front edge", where the thickness is estimated at 1.4-1.5 m. The character of this part of the ice cover is decribed in Appendix 1.

It is obvious that the ice was crushed along the path of indentation. The almost linear increase of the ice thickness until the point of structural failure indicates that the structural failure load corresponds very closely to the limit force for crushing of the ice sheet.

The width of the contact area was obviously equal to the diameter of the lightpier, which is normal when the predominant mode of ice failure is crushing.

Thus, in the actual case, the gross contact area may simply be defined as h x d with a characteristic value of 1.45x2.90 = 4.2 m^2.

As indicated by the ice cores, the ice cover had a composite structure. The upper part consisted mainly of five or more layers with a nearly uniform thickness of about 60 mm. The layers seemed to be bonded tightly together. The lower part consisted of homogeneous columnar ice. Apparently the layering of the ice had been caused by repeated rafting at an early stage, maybe initiated by the lighthouse when obstructing the drifting ice.

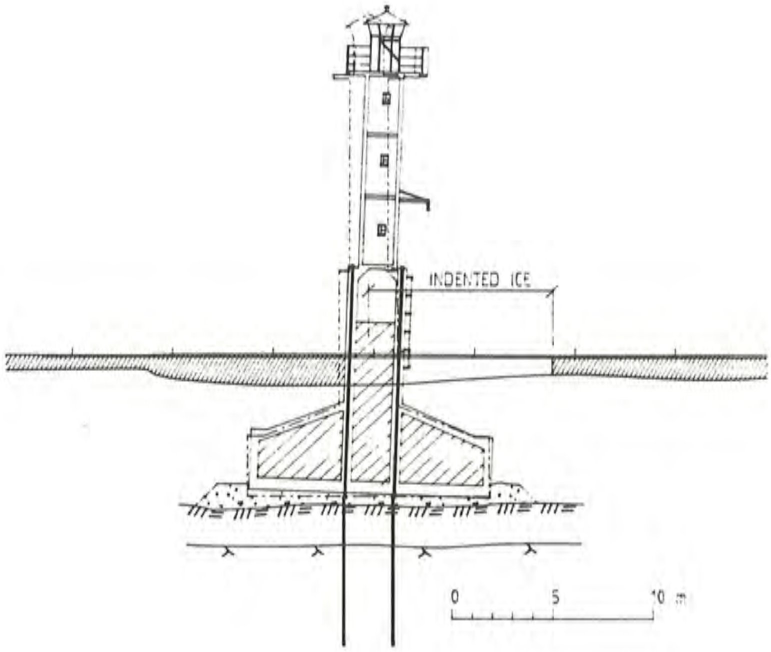

Figure 6.1 Ice profile along the centre line of indentation at the instant of anchor failure

6.4 The mean effective ice pressure

The mean ice load intensity or the mean effective ice pressure

$$p_{eff} = \frac{F}{bxh} = \frac{10.9}{4.2} = 2,6 \text{ MPa}$$

This happens to be the same value as estimated from the test samples for the mean uniaxial strength of the ice, so the factor $k = p_{eff}/f_{um}$ becomes unity.

In the actual case a simple integration of a pressure corresponding to the uniaxial strength of the ice apparently yields a correct value for the global ice load. This might be more or less coincidental, however, as discussed in the next section.

6.5 Considerations regarding the influence of structural shape, stress conditions in the ice, load transfer contact etc.

Tentatively, let us try to split the factor k into the product $k_S \cdot k_I \cdot k_C$ as suggested in subsection 6.1.

Generally, we must of course realize, that the data material does not offer any support for a distinct separation of these factors, so in this respect we must base the discussion mainly on information from the ice literature.

It is also realized that there are several reasons to suspect that the factors are generally interrelated more or less, at least through their dependence on the mode of ice failure.

However, it is of the greatest interest to find a formula (or a set of formulae) for the dependence of the effective ice pressure on the geometry of the indentor as well as the ice. Particularly, it is important to find out the influence of the aspect ratio b/h in order to be able to predict for instance the effective pressure on a larger structure on the basis of the experiences from smaller structures such as BJÖRNKLACK.

(Note that the width b of the contact area is used in order to generalize and include also cases of contact over a part of the width d of the indentor. Also the shape factor is expected to be somewhat influenced in such cases.)

Information from literature about the indentation factor and aspect ratio influences is mainly based on laboratory studies. Scale effects, different ice failure modes, vague definitions and undetected interrelations are probable explanations of the inconsistency of the information, particularly for aspect ratios of the order of 2 as in the actual case.

The two curves in Figure 6.2 have been chosen to represent the upper and lower boundaries of various suggested relations, when some extremes have been omitted. The two curves have been reproduced from ref. 7, where the background is also commented. The upper curve (from Korzhavin 1962) also approximately represents the equation

$$k_I = \sqrt{1 + 5 \cdot h/b} \qquad (4)$$

which has been applied in the design of Swedish lighthouses. For the actual case this expression would yield an indentation factor of roughly 1.9, since the aspect ratio at complete indentation is d/h = 2.90/1.45 =2.

The corresponding lower boundary value is 1.15.

Since the k factor is estimated at unity, the product $k_C \cdot k_S$ becomes 0.53 for the higher k_I value and 0.87 for the lower value. Accepting a k_S value of 0.9 the corresponding lower and upper limits of k_C would be about 0.6 and 1.0.

Since the lower k_C value is more in agreement with current recommendations for cases of brittle ice failure, it lies near at hand to believe, that the true combination is closer to

$$k = k_I \cdot k_S \cdot k_C = 1.9 \cdot 0.9 \cdot 0.6 = 1.0$$

than to

$$k = k_I \cdot k_S \cdot k_C = 1.15 \cdot 0.9 \cdot 1.0 = 1.0$$

The first combination is also in nearly exact conformity with the combination suggested in the Swedish design recommendations for lighthouses of the actual size in the Bothnian Bay. This might, however, be a coincidence and it must be borne in mind that the indentation factor chosen in the design recommendations was intended to be conservatively high, since it was used to factor experienced pressures on large structures to be used as conservative design values for smaller structures.

It is therefore probable that the real influence of the aspect ratio is less than that indicated by the factor $k_I = 1.9$ in the first combination. Another fact also points in that direction, namely that the ice failure mode in the actual case, as in other similar observed cases, was obviously of a type, which is reasonably well explained theoretically on the basis of the Tresca failure criterion as described in Ref. 5 and 7. This implies that the failure zone is located close to the indentor and the aspect ratio effect is minor except for very small values of b/h as shown in Figure 6.2. If that was the case the large scale multiaxial stress conditions did not increase the ice pressure very much and a contact factor of the order of unity seems to be explicable. It is quite possible that the strain rate varied during the sequence of overloading and that the ice at the contact surface became momentarily more ductile, particularly when the rock anchors began to yield and the structure moved in the direction of the ice drift.

In summary, we must be satisfied with the conclusion that the width of the indentor might have influenced the effective ice pressure corresponding to a range of k_I values from 1.15-1.9, which means that the corresponding ice pressure on a very large structure would have been between 2.6/1.9 = 1.4 and 2.6/1.15 = 2.3 MPa in case of complete indentation.

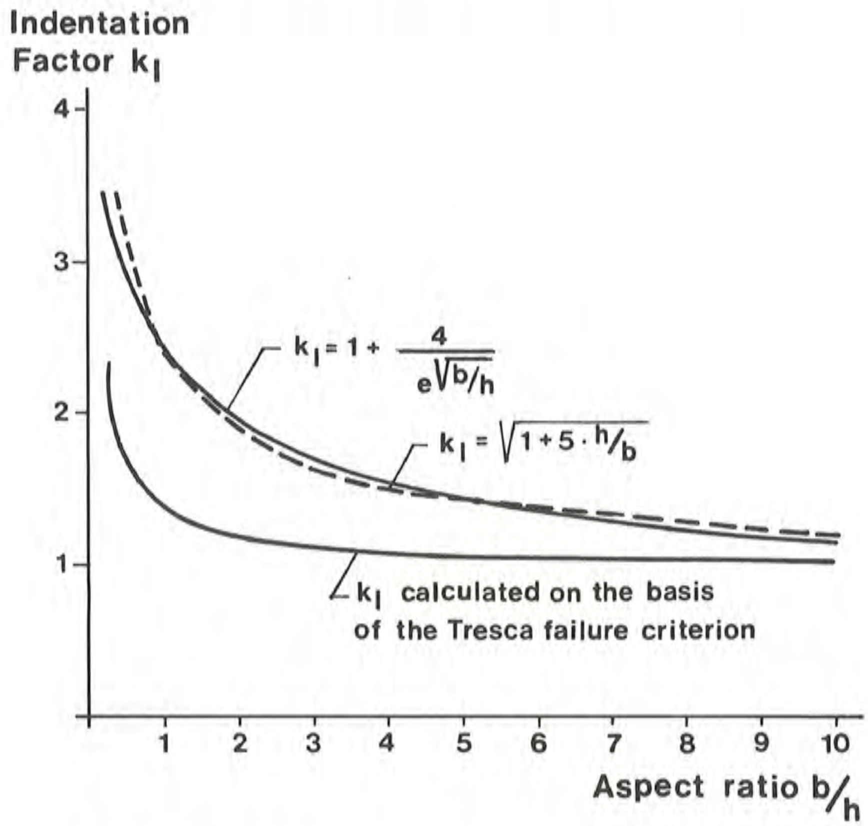

Figure 6.2 Graphs illustrating various relations between the indentation factor and the aspect ratio

7. Estimate of the driving force

The large triangular ice floe illustrated in Figure 7.1 and further described in Appendix 1 had a surface area of 20 km². According to observations crack No. 1 initially opened. After the development of crack No. 2 the strong wind forced the ice floe southwards. The movement was first restricted only by the lighthouse, inertia and possibly water drag. There was probably a rotation of the ice floe before its southeast corner was stopped at the southern edge of the ice channel. Since the lighthouse indented the ice about 6 m, it was probably free to rotate at least at the moment when the lighthouse was overloaded.

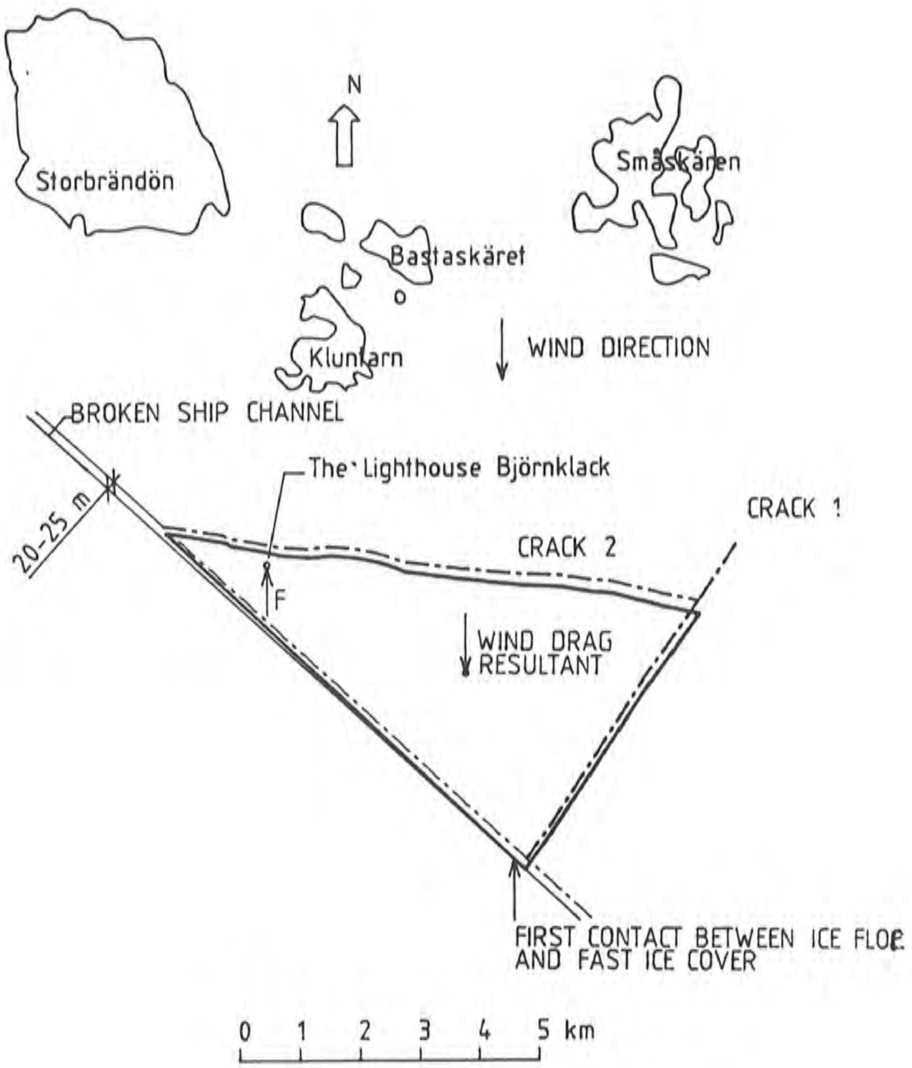

Figure 7.1 Ice situation at the BJÖRNKLACK site at the instant of overloading. (Channel width and ice floe displacements enlarged

The drag force may be expressed

$$P = \rho \cdot C_d \cdot \frac{U^2}{2} \cdot A = 1.3 \cdot C_d \cdot \frac{15^2}{2} \cdot 20 \cdot 10^6 \qquad (5)$$

The geometry of the ice floe and the support conditions are roughly sketched in Figure 7.1. If, tentatively, the drag coefficient C_d is derived from Eq. (5) on the basis of these geometrical assumptions, the value C_d=0,019 is obtained. This is at the least 3-4 times the expected value which might be explained partly by inaccurate geometrical assumptions and, most likely, by a drag contribution from currents, although the meteorological postcasts do not indicate any strong currents. Even the rather low current velocity of 0.1 m/s, however, would have been sufficient to cause drag of the same order as the wind drag.

8. Analysis of the Björnklack case in the light of previous experiences and current design recommendations

In the design philosophy applied to the Swedish lighthouses a certain taking of risks has been accepted, at least for the smaller lighthouses, since it has been realized that the costs for obtaining absolute safety against ice overloading would have reduced the number of feasible projects considerably.

It has been realized that the ice forces depend on a large quantity of environmental conditions of a more or less random character, resulting in a multitude of possible combinations and, to use probabilistic terms, in an extraordinarily wide frequency distribution.

When the design rules for the first Swedish open sea lighthouses were formulated, the complex nature of the ice force prediction problem was probably imagined, but since experiences were so few, a parametric dissolution was not possible.

The design requirements were expressed by some different expressions, depending among other things on varying design philosophies used in applied codes (serviceability or ultimate limit state design), but in principle the ice force was expressed as a function of the contact area and the effective mean ice pressure:

$$F = b \cdot h \cdot p_{eff}$$

where the product $h \cdot p_{eff}$ was initially expressed without disintegration of the factors h and p_{eff}. Different design values for this product were given for the west coast, the Baltic proper and the Gulf of Bothnia.

In the Bothnian Sea and the Bothnian Bay there are today about 20 Swedish lighthouses located in areas with what can be roughly described as open sea conditions. Along the Finnish coast there are some more lighthouses of similar type as well as several smaller lightpiers with water line diameters around 1 m.

In general, the larger structures have withstood the ice forces without significant damage except for some abrasion and minor vibrational effects. Several smaller structures have been damaged, however, and some of them even destroyed. Thus some Finnish lighthouses of the 1 m diameter class have been damaged as well as some Swedish structures with diameters between 2.5 and 3.0 m.

An overview of the situation with respect to the ice resistance of the Swedish lighthouses in the Bothnian Sea and Bay was published first in 1974 and later, with some modifications, in 1983 (Figure 8.1).

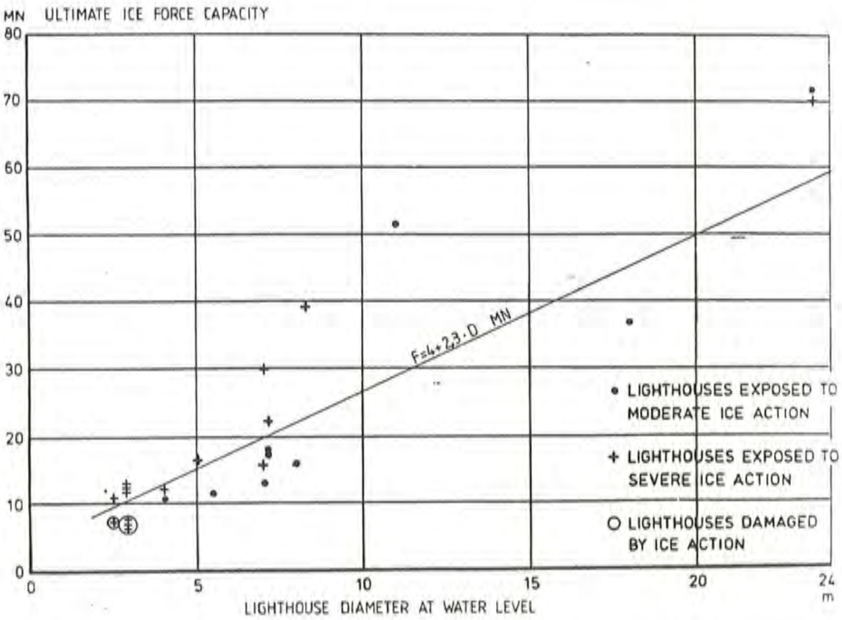

Figure 8.1 Overview of experiences with ice action on lighthouses in the Bothnian Sea and Bay

In the figure the design formula

$$F = 4 + 2.3 \cdot d \text{ MN} \qquad (6)$$

is illustrated grahpically together with symbols indicating sizes and ice force capabilities of existing lighthouses.

The formula is a simplification of

$$F = 1.6 \cdot \sqrt{1 + 5 \cdot \frac{h}{d}} \cdot h \cdot d \text{ MN} \qquad (7)$$

(the simplification valid for $2<d<23$ m and $h = 1.3$ m, approx.)

This formula has been derived from the expression

$$F = k_S \cdot k_C \cdot k_I \cdot f_{um} \cdot h \cdot d$$

with the product $k_S \cdot k_C \cdot f_{um} = 1.6$ MPa, which can thus be defined as the mean effective design pressure on a very wide structure, to be multiplied by the indentation factor accounting for a limited width.

Reverting now to the overloading of the Björnklack lighthouse we see that the design formula would yield.

$$F = 1.6 \cdot k_I \cdot h \cdot d = 1.6 \cdot 1.8 \cdot 1.3 \cdot 2.9 = 10.8 \text{ MN}$$

This is exactly the same value as estimated for the overloading case as the most probable failure load.

Let us once more express the ice force in the simple form

$$F = p_{eff} \cdot h \cdot d$$

Then in accordance with the design formulae

$$F = 2.9 \cdot 1.3 \cdot d = 3.77 \cdot d \text{ MN}$$

and for the overloading case

$$F = 2.6 \cdot 1.45 \cdot d = 3.77 \cdot d \text{ MN}$$

The immediate conclusions are:

1) The design load, intended to have a mean recurrence frequency of one event in at least 1 000 year at each site, was obviously reached.

   (There are reasons to believe that the magnitude of the 1000-yr load has been underestimated)

2) Compared with design assumptions the ice thickness was the parameter, which was really extreme.

However, as will be dealt with in the next section, we feel that it is necessary to consider the interrelation between the parameters, when comparing predictions with observations from a probabilistic viewpoint.

Therefore, we prefer to conclude that primarily the overload was a result of an extreme combination of ice thickness, ice strength and ice movement.

9. Probabilistic aspects

In order to be able to deal with the problem in probabilistic terms, the most important causal relations between the "ice load parameters" chosen in the preceeding section and the most important environmental conditions have to be clarified.

Some of these relations are clearly indicated by collected field data and observations made in connection with the overloading event, but other relations are supported by indications rather than evidence. In a simple form with less important parameters passed over, the most probable course of event is sketched in Figure 9.1.

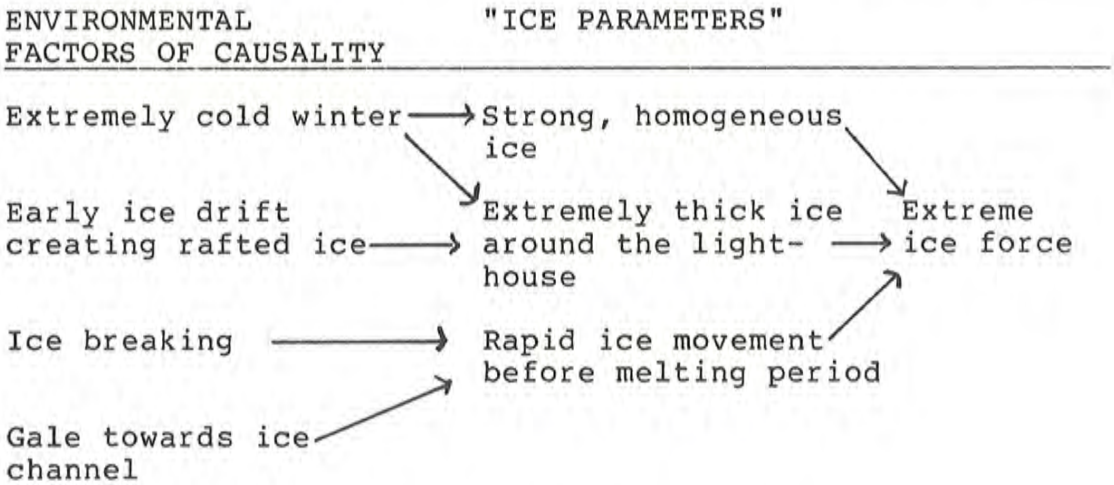

Figure 9.1 Course of main events resulting into the extreme ice load on the BJÖRNKLACK lighthouse

With respect to the "ice parameters" it has already been observed, that it is the combination rather than the individual values, which are remarkable. The strength of the ice was above the normal winter strength but not extremely so. Rafting of ice often occurs, although the ice sheets do not usually bond very well together. In the actual case the severe winter has probably been important in this respect.

Let us assume that the existence of rafted, about 1.5 m thick ice with a mean strength of 2.6 MPa within a certain area around the Björnklack lighthouse has an average recurrence period of about 100 years.

The problem is then to assess the probability that the thickened ice would move and reach the lighthouse before being softened due to increasing air temperature in the spring. It is obvious that this would depend on the size of the area considered, wind force, wind direction and currents, the crack pattern, the travel distance and velocity of the drifting ice etc. It is also obvious that the probability of such an ice movement taking place during a winter like the last is normally rather low, since the Bothnian Bay is then covered by a continuum of fast ice.

We can imagine that the joint probability of such an event would be several orders lower than 0.01, provided that the existence of a broken channel was disregarded.

Still disregarding the ice channel the most probable situations creating extreme loads would be indentation of softened ridges or rafted ice later in the spring, when ice drift is frequent, or ice thrust due to minor slow movements of a cover of cold and thick ice during mid-winter. In the latter case the ice pressure would most probably be mitigated by the low strain rate.

With regard to the statistic relation between ice strength and thickness and the movements of the ice cover, the Björnklack case also offers a valuable demonstration. The fact that the lighthouse remained undisplaced by the ice after the failure of the rock anchors, although its capability was reduced by about 50 per cent, shows that the large ice floe, after being stopped by the fast ice at the southern end of the broken channel, did not move very much before it was "softened" to the corresponding extent.

From the probability point of view the "environmental conditions" noted in Figure 9.1 may then be classified into at least two quite different groups: one with natural phenomena of more or less random character and the other with conditions established by human activites. Predominant in the latter group is of course the existence of the broken ice channel and its position in relation to the lighthouse, predominant wind directions, islands, shoals and coastlines. It is also obvious that there are modes of ice structure interaction, which contribute to increase the probability of thickened ice at the point of contact. Layering of relatively thin and large sheets of buckled ice is one mode. Development of thickened ice due to floading and refreezing has been observed around bridge piers etc. (ref. 8). Formation of rubble ridges is frequent, although such ridges are generally rather soft in the Gulf of Bothnia.

Our first conclusion is then, that our design rules as well as any ice load consideration with a probabilistic element should be selective in at least one more respect than the climatological and geographical and that is the consideration of consequences of breaking the ice in a way which can provoke movements of large and strong ice fields.

Considering indications from the Björnklack case a load factor of 1.5-2.0 does not seem exaggerated, to account for the presence of an ice channel. An increase of the current design loads for such conditions seems justified, as well as a reduction for conditions of clearly restricted ice movements.

10. A generalized interpretation of the field observations

The experiences from the Björnklack case together with previous observations from the Gulf of Bothnia emphasizes the urgency not only of supporting ice force predictions on field evidence but also of making clear distinctions with respect to different environmental conditions and related ice parameter values and combinations.

Particularly emphasized is the importance of considering the frequency of ice drift as related to different combinations of ice thickness and strength, also frequency distributed preferably both in time and space.

For wider application the following experiences from the Björnklack overloading event may provide useful quidance:

- The effective mean ice pressure was 2.6 MPa as was the mean uniaxial compressive ice strength.

Coincidentally or not, this is what would be predicted using current ice force prediction practice, when accounting for increased strength due to confined compression of the ice as well as an incomplete contact between the structure and the ice at brittle crushing.

The uniaxial strength 2.6 MPa is above average but not extreme for Baltic areas and is expected to be representative in order of magnitude for many Arctic and Subarctic areas.

- From the probabilistic point of view the conditions at Björnklack are probably more representative of Arctic areas with frequent ice drift than of the Bothnian Bay in general, where large movements of ice during severe winters are normally restricted.

11. Concluding remarks

Briefly the conclusions drawn from the BJÖRNKLACK case study can be summarized by the following points.

- The extreme ice force, which overloaded the lighthouse, developed basically as a sequel of an exceptionally severe winter and a partly random course of events, which caused an exceptionally early ice movement at the lighthouse site.

- Expressed in terms of ice parameters the ice force was a result of an extreme combination, where the ice thickness, the strength of the ice and primarily the large movement of ice with such properties are the most significant.

- The whole course of event and the results of the analysis of observations underline the necessity to use, at least partially, a probabilistic approach in the prediction of ice forces.

- Risk evaluation and corresponding risk classification of structures subjected to large ice forces then become logical elements in the design. Lighthouses, for example, should be classified with respect to operational, economical and environmental conditions.

- The mean effective ice pressure on the BJÖRNKLACK lighthouse was 2.6 MPa which is the same as the average value of the compressive strength of unconfined test samples. Multiaxial stress conditions in the ice cover due to the limited width of the contact area may have contributed with 10 to 50 per cent of the effectiv ice pressure.

- It is important to direct future field investigations within the actual project on finding out, among other factors, primarily the ratio $k = p_{eff}/f_{um}$ and its dependence on the aspect ratio b/h, the absolute area of contact etc. Secondly, it is of course of great interest to determine relations for the parameters k_I, k_S and k_C. It is expected, however, that a distinct separation of these factors might be obstructed by their reciprocal relations among other things.

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LIST OF SYMBOLS ETC.

Units

SI units are generally used in this study.

Symbols

h: ice thickness

b: instantaneous width of ice-structure contact area

d: diameter of a cylindrical indentor

f_u: uniaxial strength of the ice (compressive if not otherwise stated)

f_{um}: "mean uniaxial strength", averaged over the contact area (compressive if not otherwise stated)

p: contact pressure between the ice and the structure

p_{eff}: "mean effective ice pressure", the unidirectional ice pressure averaged over a defined gross contact area, normally hxb, i.e.

$$p_{eff} = \frac{F}{hxb}$$

F: "global ice force" i.e. the total unidirectional ice force on a structure, instantaneous or averaged over a certain period of time

F_{max}: the "peak global ice force" during a certain period of global ice force variations (in our studies the "peak force" is in fact also averaged, since it is related to the measured response of a structure, but it is normally averaged over a very small fraction of the period of the most significant mode of structural vibrations)

k_S: "shape factor", the ratio between global ice loads on e.g. cylindrical and plane indentors

k_I: "indentation factor", for factoring the ice load with respect to the aspect ration b/h

k_C: "contact factor", for factoring the ice load with respect to imperfect contact between the ice and the structure.

$k = \frac{p_{eff}}{f_{um}}$ a common factor, accounting for all the abovementioned and other factors for transformation of the uniaxial ice strength to effective ice pressure on a structure

STUDY PROJECT

ICE FORCES AGAINST OFFSHORE STRUCTURES

APPENDIX 1

This report, Document No. 85:13 from the Division of Structural Engineering, University of Luleå, contains results of ice investigations carried out by Lennart Fransson (planning and direction), Georg Danielsson (testing), Lars Åström och Håkan Johansson (core sampling, etc.) from the Div. of Struct. Eng. which is headed by Lennart Elfgren.

The report has been prepared by L. Fransson and G. Danielsson as a contribution to "existing data", providing a basis for Task No. 1 of the project ICE FORCES AGAINST OFFSHORE STRUCTURES.

CONTENTS

SUMMARY

The lightpier Björnklacken was overloaded by a moving ice sheet on April 4, 1985. The maximum wind speed at that time was 15 m/s from the north. The ice consisted of level ice and columnar ice with a total thickness of 1.4 m close to the lightpier. The mean yield strength, determined from 13 horizontal samples in a N-S direction, close to the lightpier, was 2.9 MPa. The salinity was less than 3 ppt.

1. INTRODUCTION

The ice in Luleå archipelago normally forms late in November and consists of a columnar rigid ice sheet at about 0.4-0.8 m. The winter of 1984/85 was extremely cold and the ice thickness exceeded 1 m.

A storm from the north occurred in early April. A large ice floe then moved south against an open track. The lightpier Björnklacken was embedded in the moving ice floe and became overloaded.

Shortly after the overloading event, the division of Structural Engineering conducted a field investigation of the ice situation close to the lightpier. Samples of the ice were taken, and the ice properties were examined at the ice laboratory in the University of Luleå.

The ice force acting on the structure can be estimated

1. as the force required to break the ice
2. as less than the driving force from the ice floe
3. as the force required to break the lightpier

This report serves as a foundation for calculations of the first two forces.

2. FIELD INVESTIGATION

2.1 Observations on the moving ice floe

Referring to observations made by Mr Tore Hjelte, Persön, Luleå the lightpier was intact on Thursday morning, 4th April. During the day the wind force from the north increased and a large crack (crack 1 in figure 2.2) developed. This crack isolated Mr Hjelte from the shore and he had to wait for rescue on the east side of crack no. 1. It was then he observed that the lightpier was tilted (Ref 1).

On the same day, 4th April, the Swedish Air Force flew over the area. Captain S-O Wiklund also observed that the lightpier Björnklacken had tilted at about noon, but he also said that the ice flow had not moved appreciably at that time. Further movement of the ice to the south occurred later (Ref 2).

Based on photos (figure 2.1) and on these two observations, the ice situation around the lightpier probably was as indicated by the sketch shown in figure 2.2.

Recordings of the weather situation on Thursday, 4th April was received from the Swedish Air Force at Kallax airport at the coast close to Björnklacken. The mean wind speed, direction and temperature are shown table 2.1.

Table 2.1 Mean wind speed, direction and temperature measured over a 10 minute interval

time	wind direction	wind speed (m/s)	temperature(C)
2 am	340	10	-2.4
8	340	9	-1.8
10	350	13	
11.20	north	15	
2 pm	350	11.5	+0.4
8	10	8	+1.2

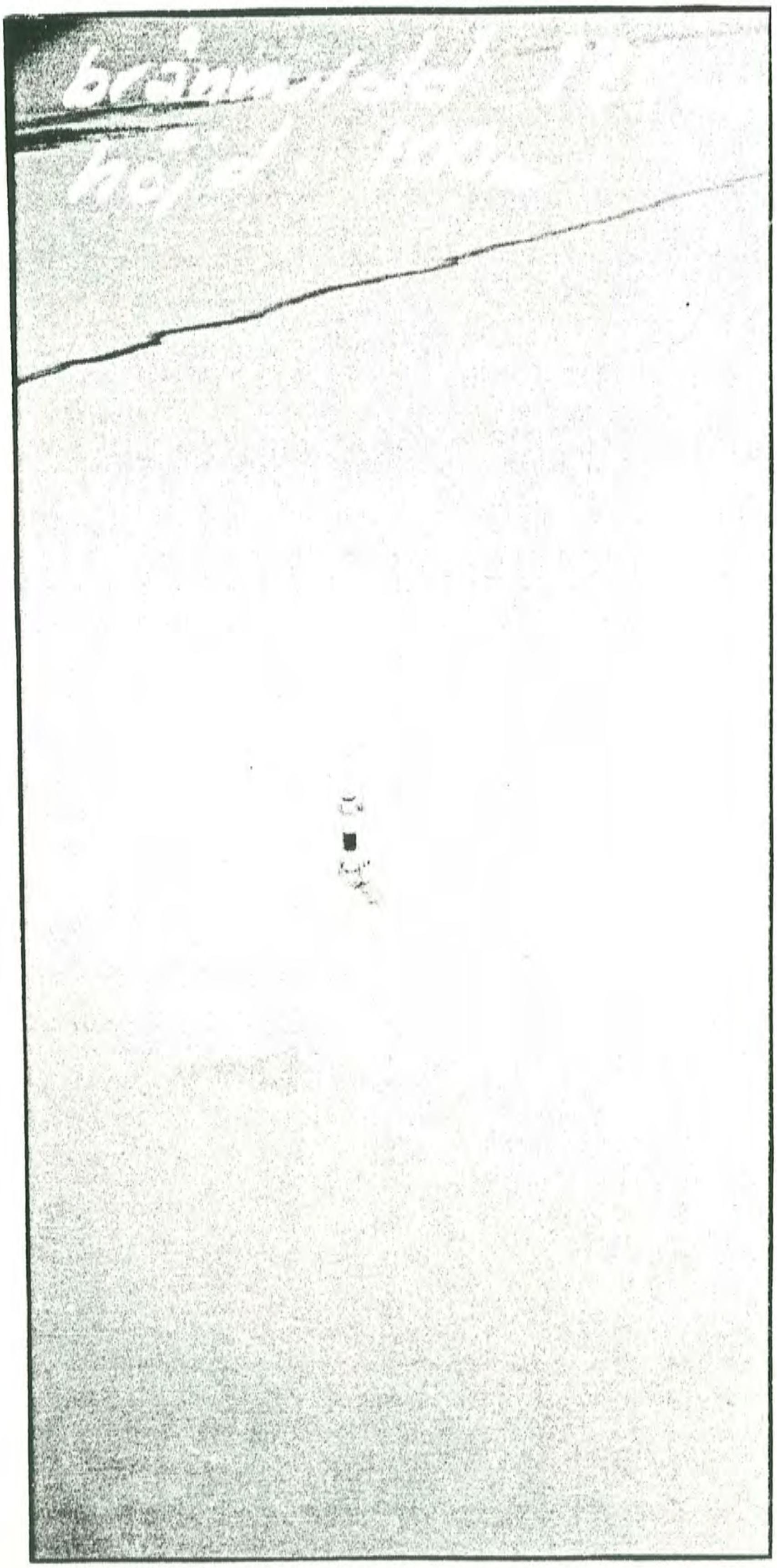

Fig 2.1 The icecrack north of the lightpier Björnklacken, April 15, 1985 (Photo: The Swedish Air Force)

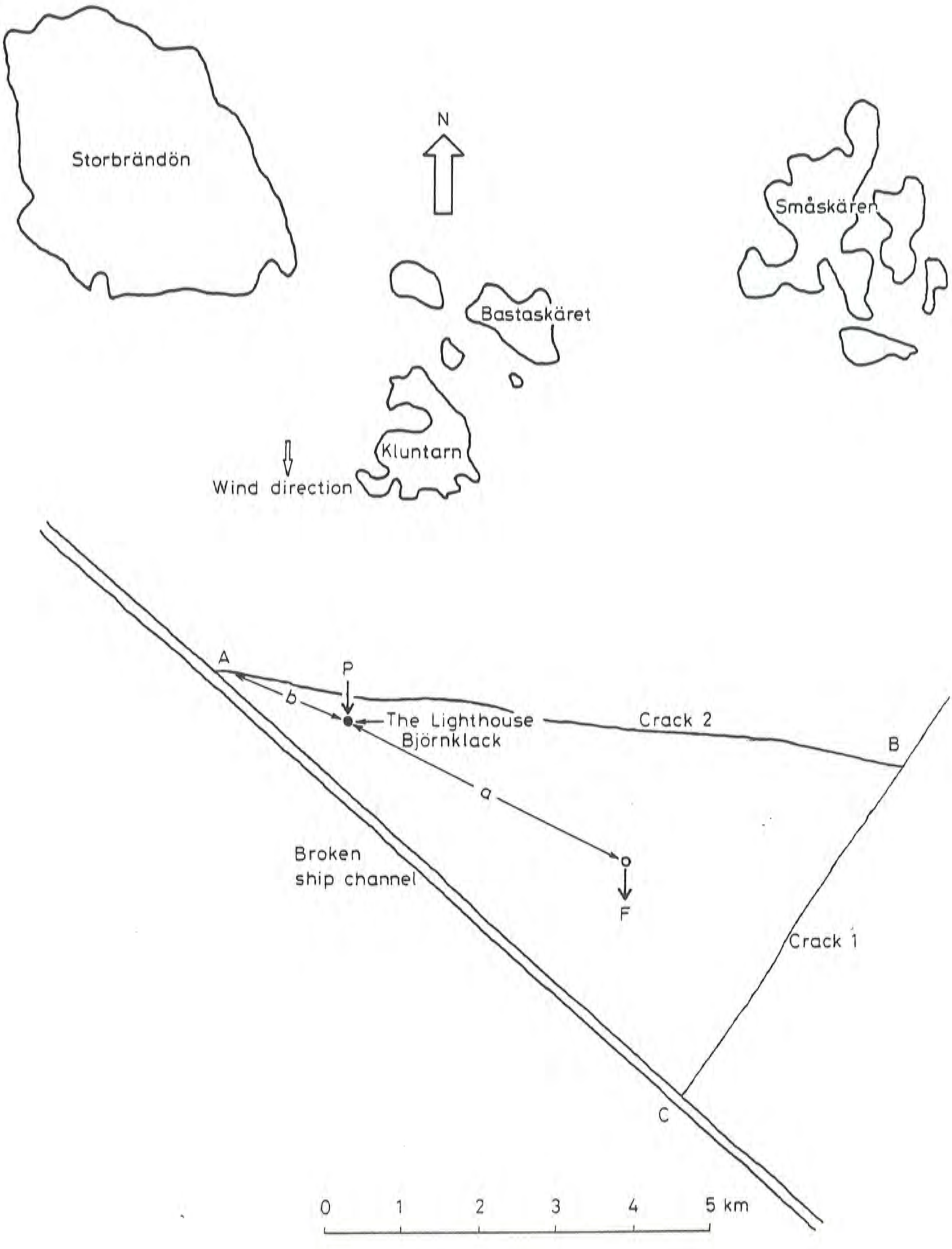

Fig 2.2 Probably ice situation around the lightpier based on observations (Ref 1 and 2)

Based on these facts we suppose that the lightpier Björnklacken was forced by the triangular ice floe ABC in figure 2.2. Initially the force (F) probably rotated the ice sheet. This resulted in an amplified force on the lightpier. If the location of the centroid of the driving load distribution was as shown in figure 2.2 and the effect of current is ignored, we can expect the magnitude of the ice load to be described by the equation

$$Pb < F(a+b)$$

$$P < (1 + a/b)\,F \qquad \ldots (1)$$

where P = ice load on the lightpier
F = total wind load on the ice floe ABC
a = distance from lightpier to load center
b = distance from lightpier to rotating center

2.2 Ice conditions close to the lightpier

South of the lightpier, a 6 m long ice track was observed. North of the lightpier a rubble field had formed. The ice was completely covered with packed snow. Small ice cracks were therefore not visible. Water had flooded some parts of the ice and during the drilling operation water was often discovered deep in the ice.

The ice formation close to the lightpier is shown in figure 2.3.

The ice thickness was measured in April in drilled holes along four profiles as shown in figures 2.4 to 2.6. The ice thickness was 1 m at a dis- tance from the pier where it was undisturbed. It was thicker close to the pier. East of the lightpier there was a ridge with a depth >2 m. The height of the rubble in front of the lightpier was about 1.4 m and the depth can be extrapolated from the profiles to 3 m below the water line.

Ice cores were taken from the three positions shown in figure 2.4. The north-south direction was marked on the top of each core before drilling so that the horizontal loading direction could be maintained in the laboratory tests. The core diameter was 200 mm and the maximum length of a continuous core was 0.7 m. Thicker ice was cored through by bending loose the first core.

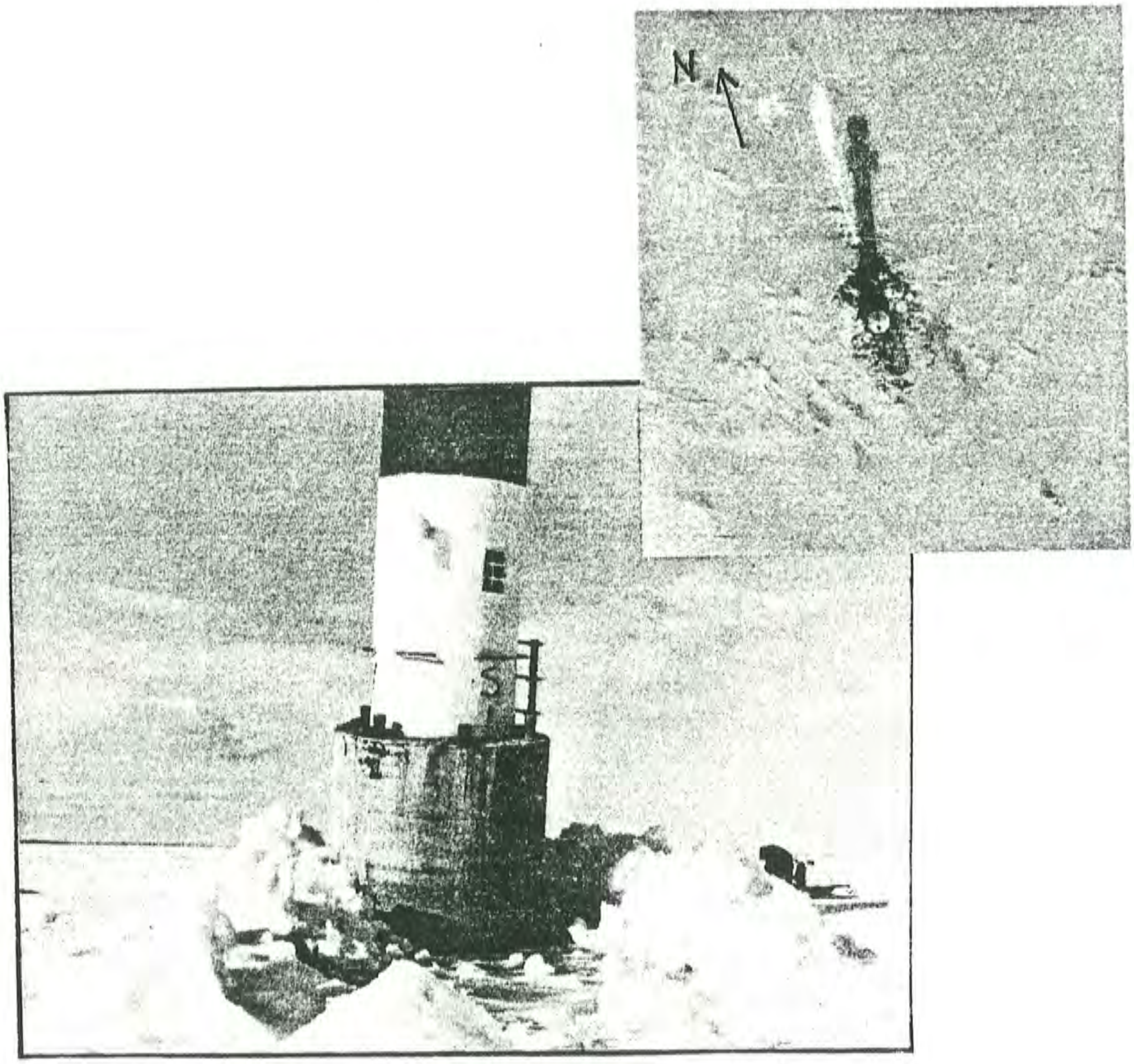

Fig 2.3 Ice situation close to the lightpier shortly after the overloading event
(Photo: Lars Åström and the Swedish Air Force)

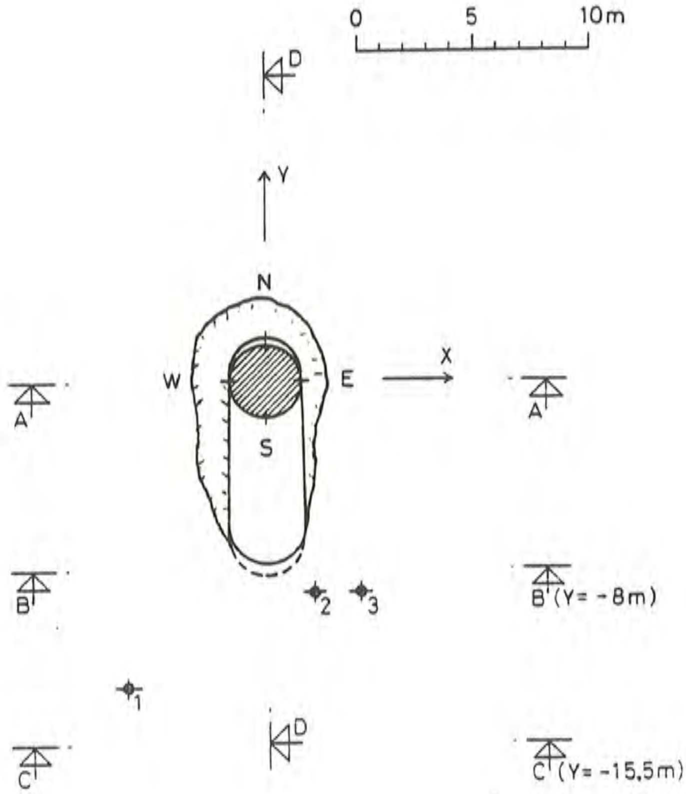

Fig 2.4 Site plan of the drilled profiles close to the lightpier Björnklacken, April 10, 1985

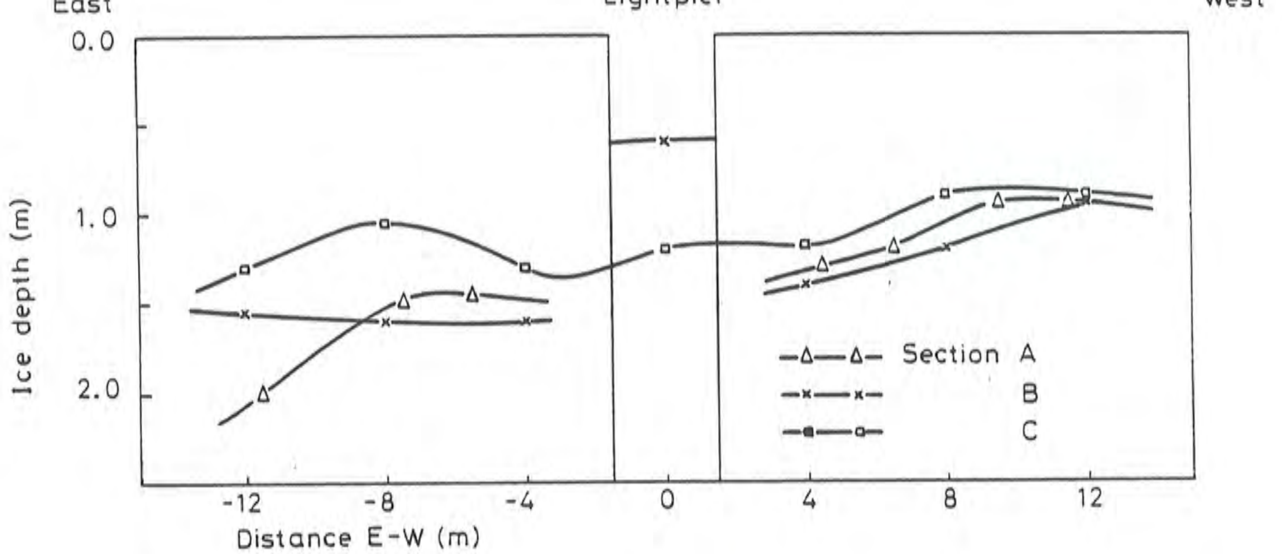

Fig 2.5 Ice thickness along profiles perpendicular to the ice moving direction. Björnklacken, April 10, 1985

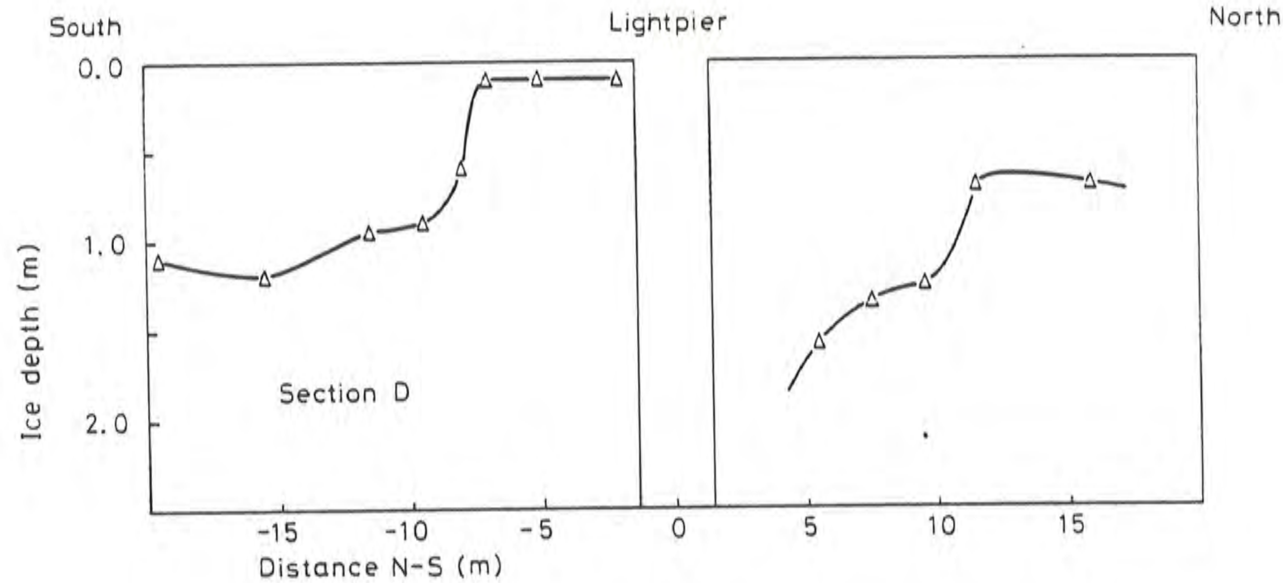

Fig 2.6 Ice thickness along the ice moving direction. Björnklacken, April 10, 1985

3. LABORATORY INVESTIGATION

3.1 Physical properties

The field samples were stored in a freezer in a cold room at the University at a temperature of -25°C. Each core sample was sealed in a plastic bag.

Horizontal cores in north-south direction with a diameter of 70 mm were taken at different levels from the larger vertical core. The distance between each horizontal core was about 0.25 m. After preparation, the small cores were also stored in tight plastic bags and at a temperature of -25°C.

To classify the different types of ice involved in the ice cover, vertical thin sections were examined between crossed polaroids. The thin sections were taken from the same core as the horizontal samples so that the ice type of each horizontal sample could be identified. In figure 3.1 the different ice types surrounding specimens 2a-e are shown.

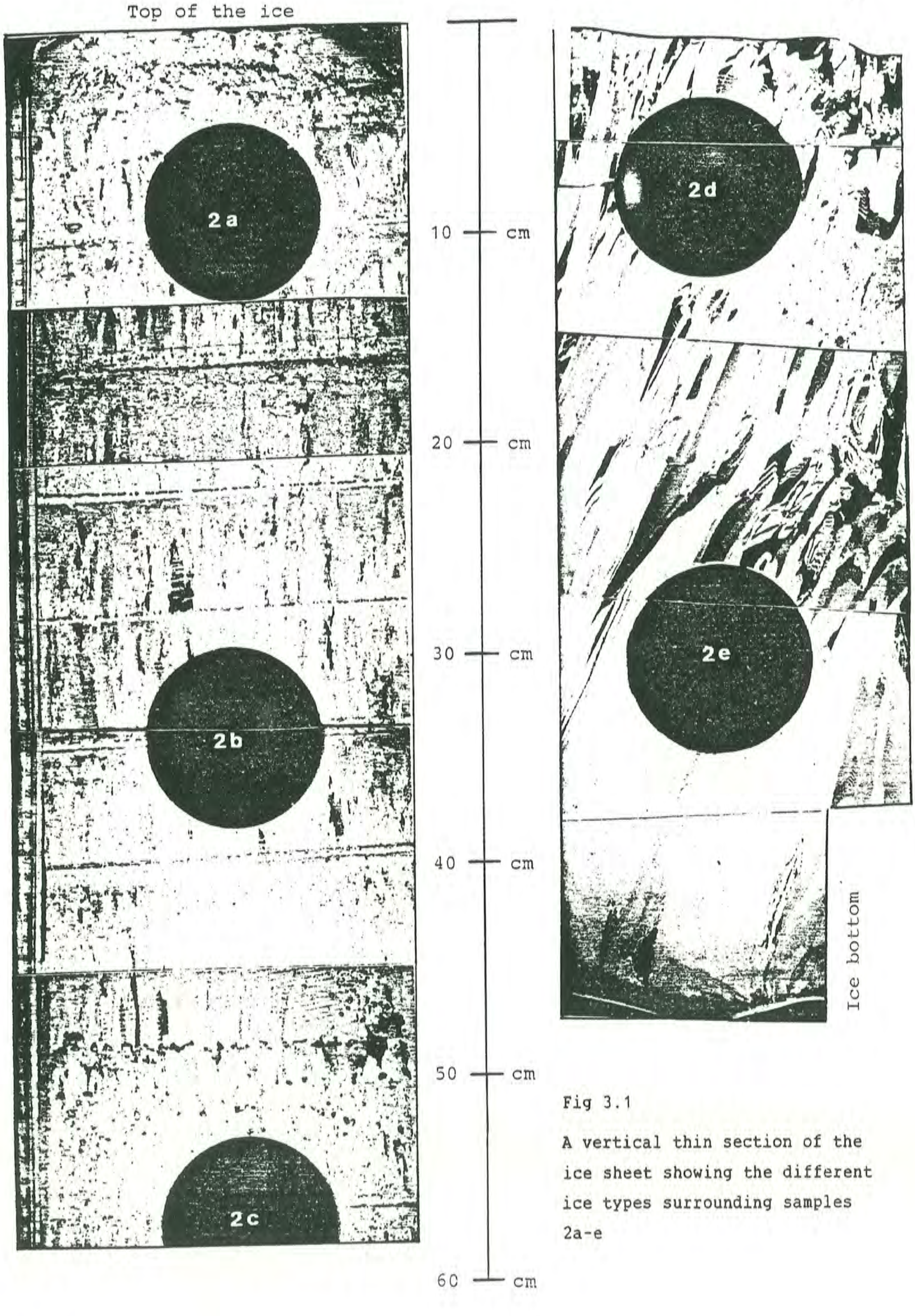

Fig 3.1

A vertical thin section of the ice sheet showing the different ice types surrounding samples 2a-e

In general, the upper half of the ice consisted of level ice (rafted or flooded) and the lower half consisted of columnar ice. In table 3.1, the ice types are divided into three groups, granular, columnar and level ice. The grain diameter of columnar ice and the level thickness of level ice are specified.

The density was calculated from the measured diameter, length and weight of the horizontal cores at a temperature of -5°C.

The salinity of a melted ice specimen was measured with a conductivity meter.

Table 3.1

Sample	Depth(mm)	Ice Type	Density(kg/m^3)	Salinity(O/OO)
1a	80	granular	888	1.9
1b	330	level/gran.	832	0.2
1c	580	granular	903	0.2
2a	90	granular	802	2.6
2b	340	levels 60mm	888	2.4
2c	580	granular	823	1.8
2d	750	col. 8mm	909	1.2
2e	990	col. 14mm	910	0.1
3a	90	levels 90,60mm	887	0.4
3b	340	levels 60mm	892	0.2
3c	585	levels 80,65mm	897	0.5
3d	745	granular	902	0.0
3e	990	col. 10mm	912	0.1

3.2 Mechanical properties

Uniaxial compression tests according to the IAHR standard (Ref 3) were performed on the horizontal cores in N-S direction. The diameter of the cores was 70 mm and the length was about twice the diameter. The load was applied with a servo-hydraulic loading machine (INSTRON T1) on a specimen placed inside a cold cabinet. The steel load surfaces were cooled before each test, and a fibreplaten was placed between the steel and ice. The test set up is shown in figure 3.2.

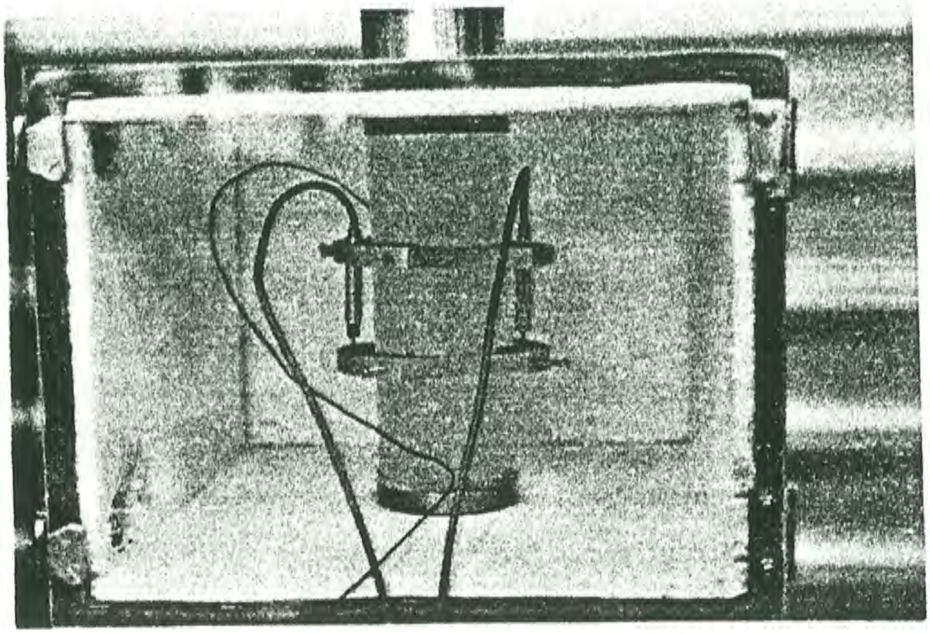

Fig 3.2 Uniaxial compression tests of cylindrical specimen (d=70 mm). The deformation of the ice is measured with two LVDT:s fixed at the middle part of the specimen

The ice temperature was held constant at about -3.5°C for the vertical section 2. For section 3 the test temperature was held at -4.2°C for sample 3a and for each sample the temperature was increased by 1°C. See table 3.2, below.

The load was measured with a high precision load cell (INTERFACE SSM-3000), and the deformation of the ice was measured with two LVDT:s on a length of 50 mm. All tests were performed at a constant rate of deformation, 0.01 mm/sec. (Strain rate = $2 \cdot 10^{-4}$ s^{-1}) in the middle part of the specimen.

The yield stress recorded in table 3.2 is defined as the maximum stress during a constant strain rate. The yield strain is defined as the strain measured at the yield stress. The stress-strain curves from all tests are shown in appendix 1.

Table 3.2

Sample	length(mm)	Temperature(^{o}C)	Yield strain(%)	Yield stress(MPa)
1a	160.0	-4.3	.11	3.48
1b	159.5	-4.5	.13	3.68
1c	158.0	-3.0	.12	3.89
2a	169.5	-3.5	.14	4.14
2b	160.0	-3.5	.10	3.37
2c	138.5	-3.5	.05	1.83
2d	144.0	-3.5	.05	2.69
2e	131.0	-3.5	.04	1.72
3a	160.5	-4.2	.14	3.18
3b	160.0	-3.2	.16	2.91
3c	159.5	-2.2	.12	3.06
3d	159.5	-1.2	.06	2.37
3e	148.5	-0.2	.09	1.36

The low stress maximum for sample 2c can be explained to be due to the presence of air pockets observed in the sample. This sample also had a low density (823 kg/m3) and it is possible that the core was crossed by a refrozen crack. The stress-strain curve (2c) in appendix 1 supports this claim. The curve illustrates a post failure behaviour typical for a cracked body.

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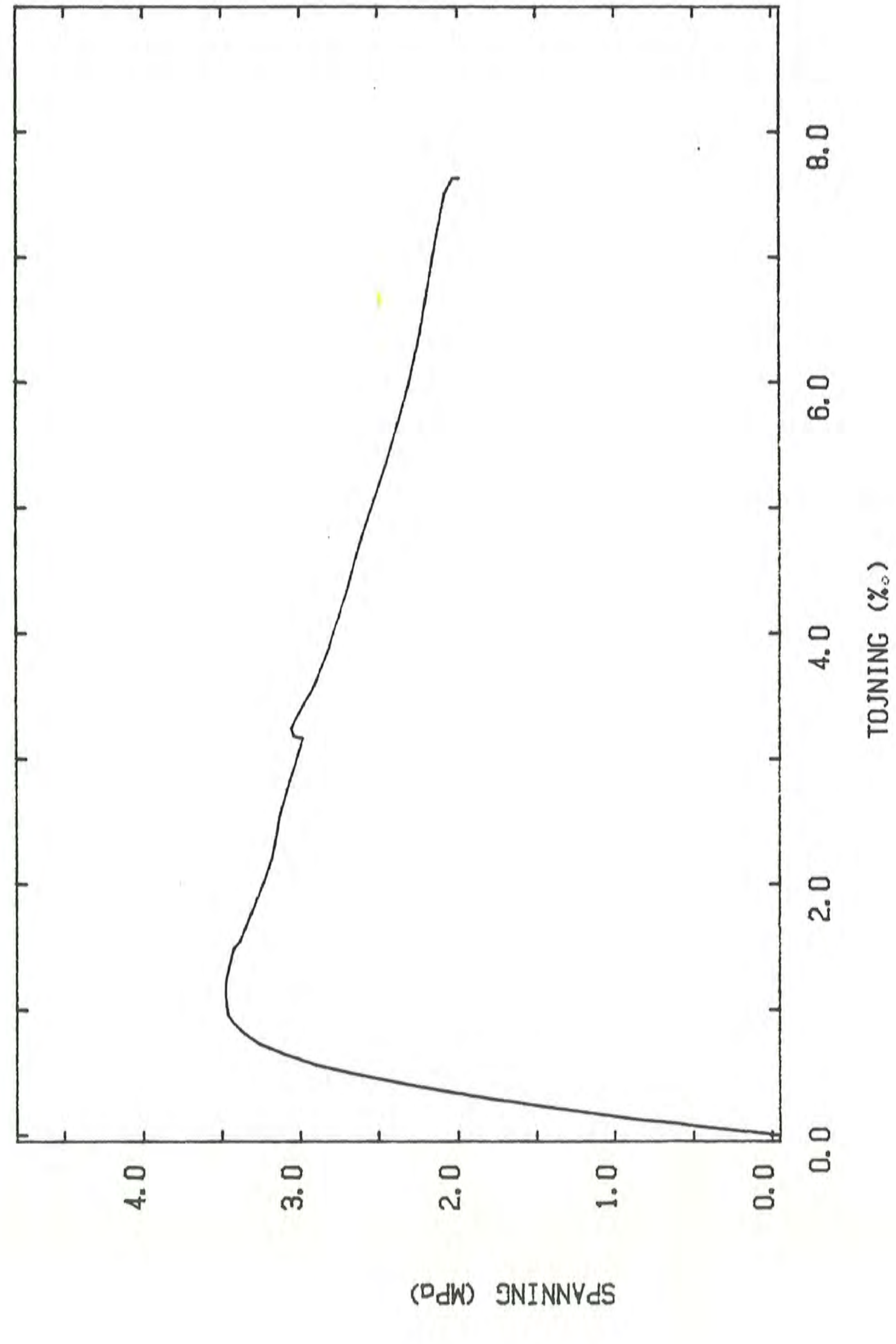
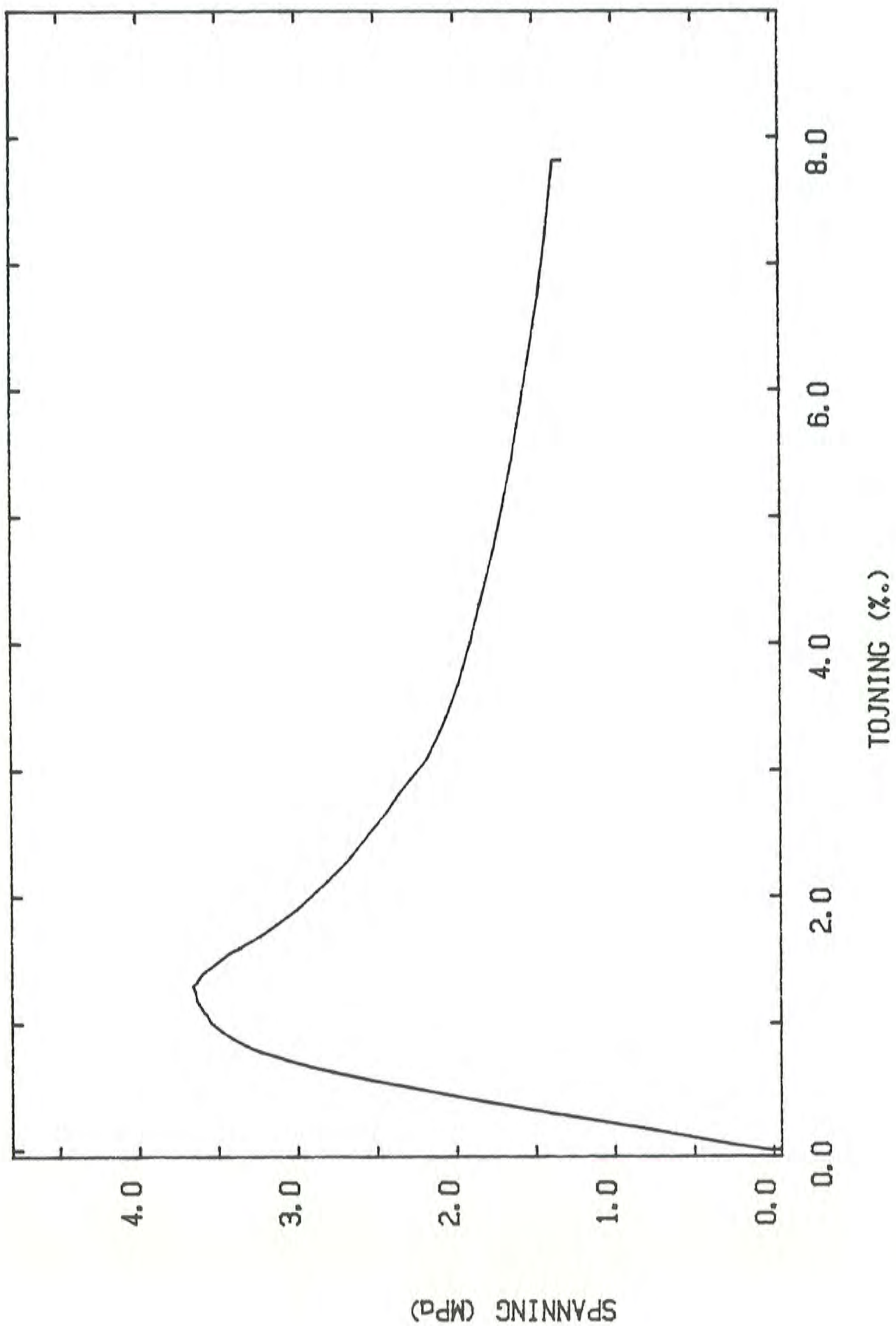
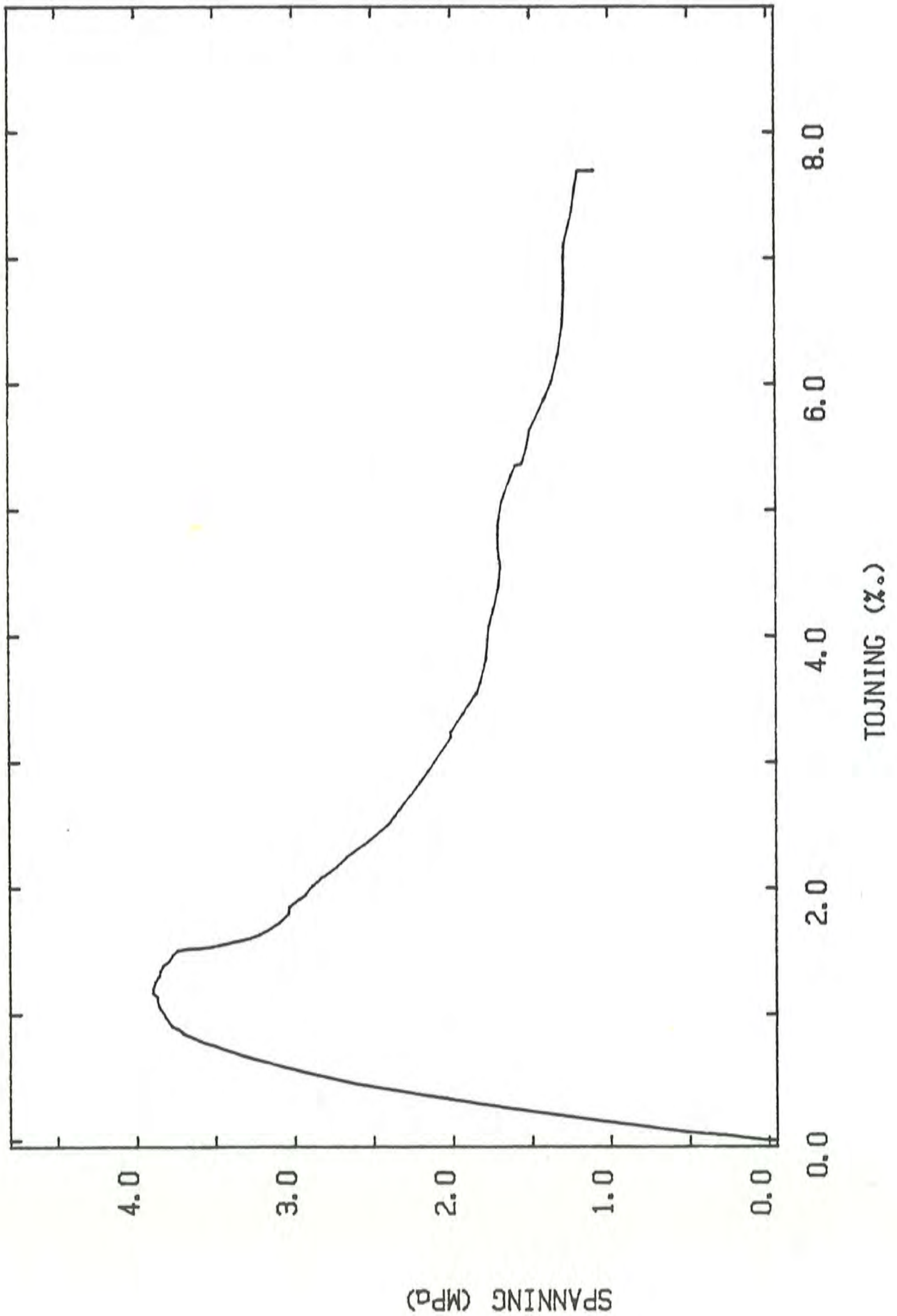
SAMPLE 1 a



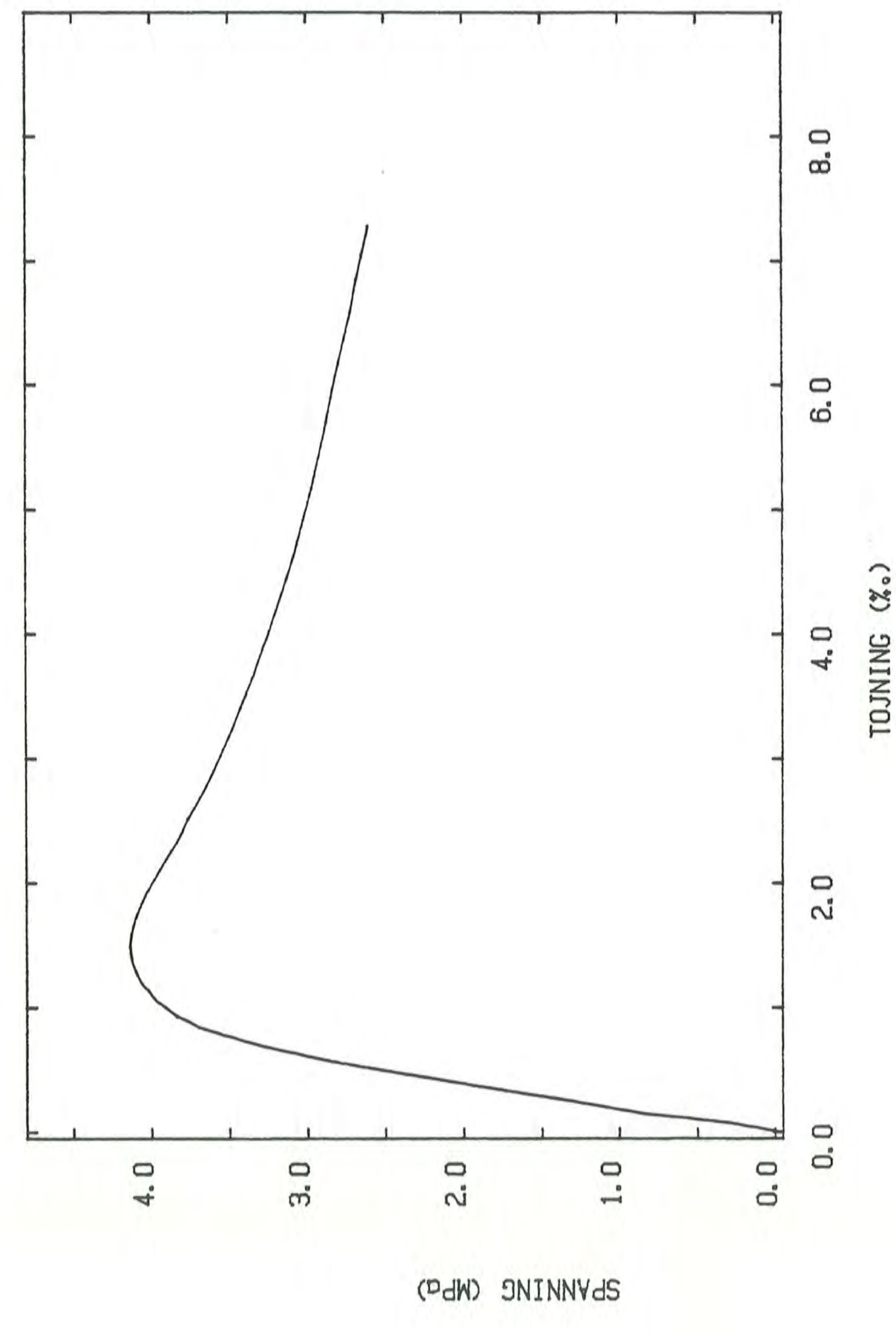
SAMPLE 1 b



SAMPLE 1 c



SAMPLE 2 a



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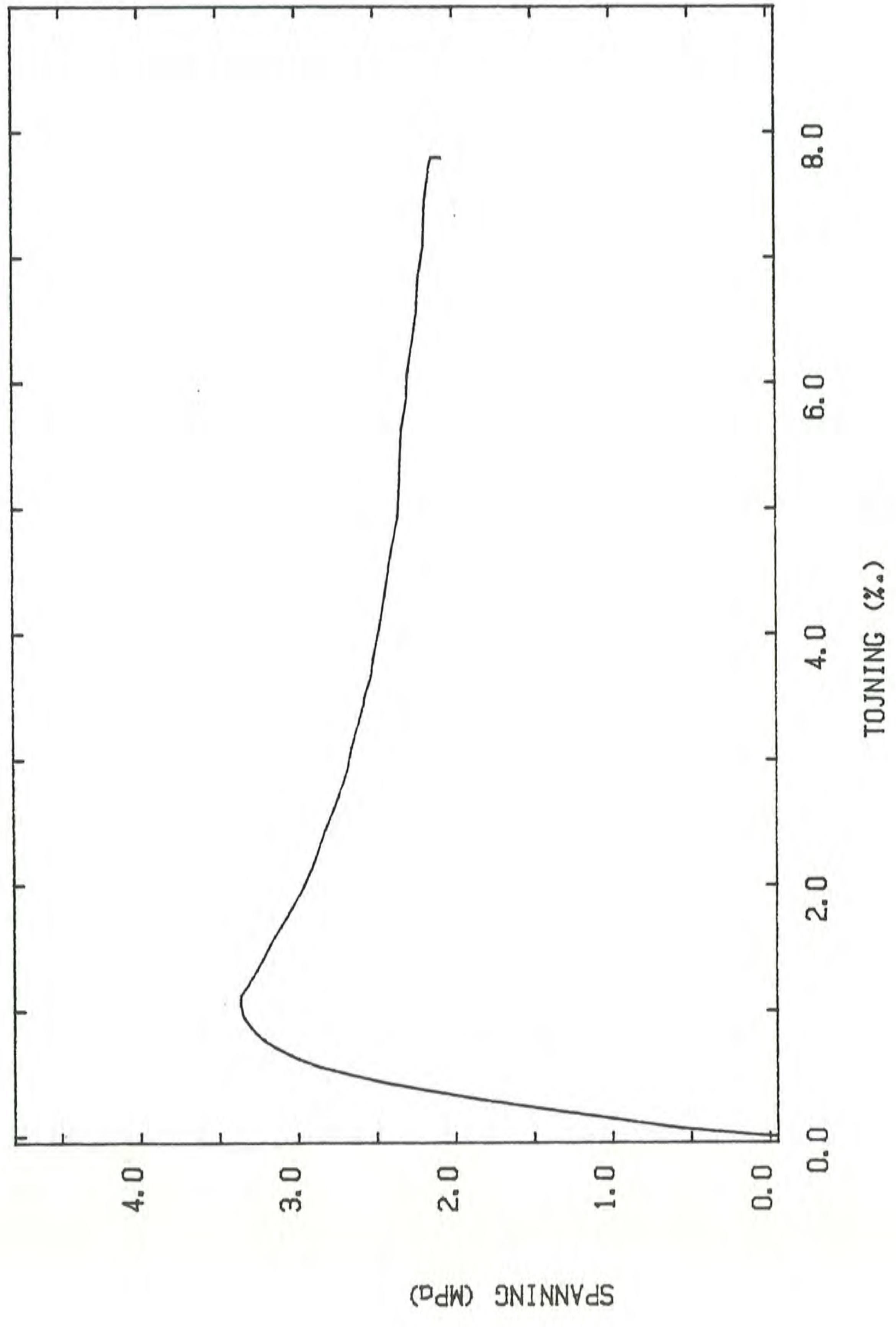
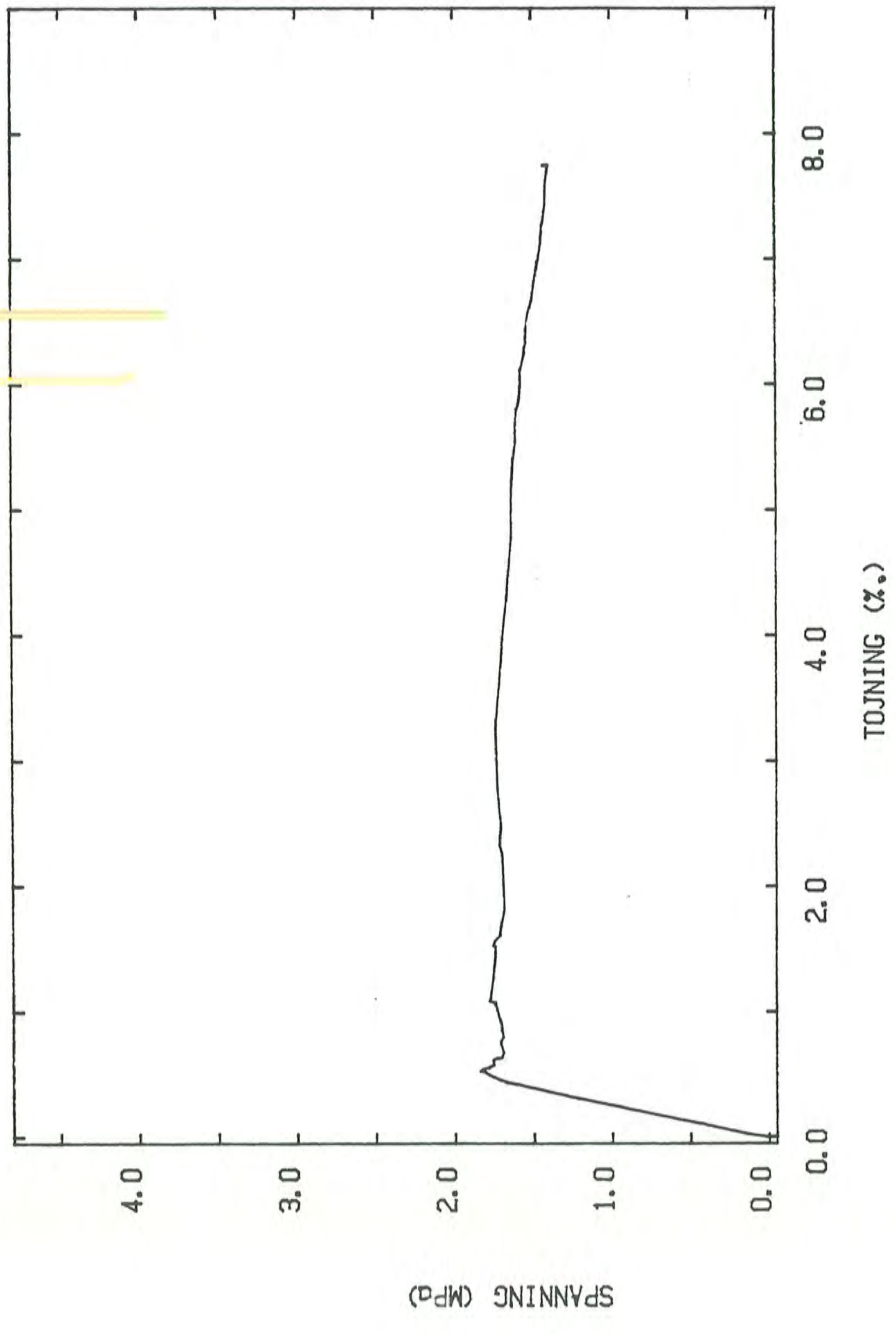
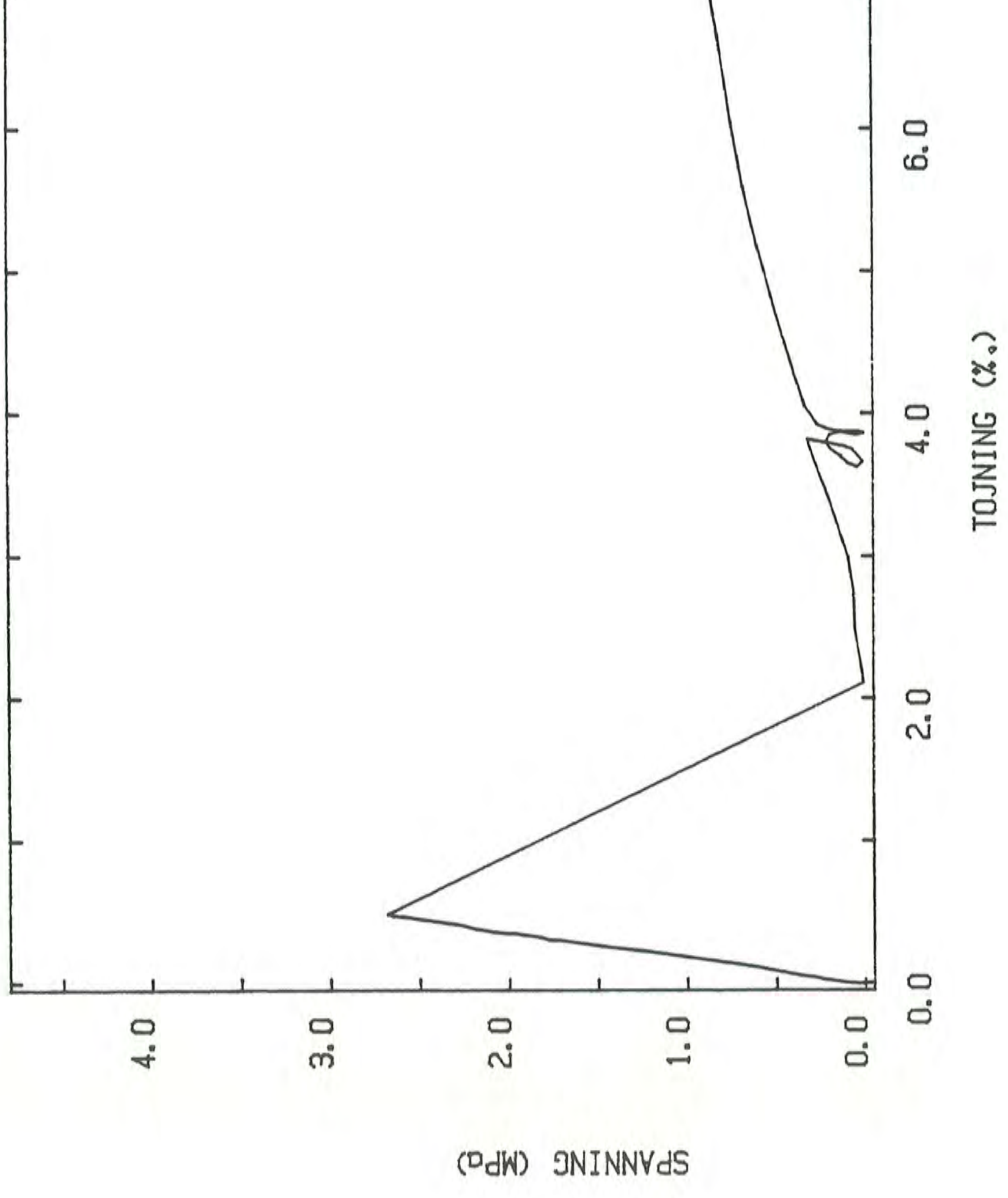


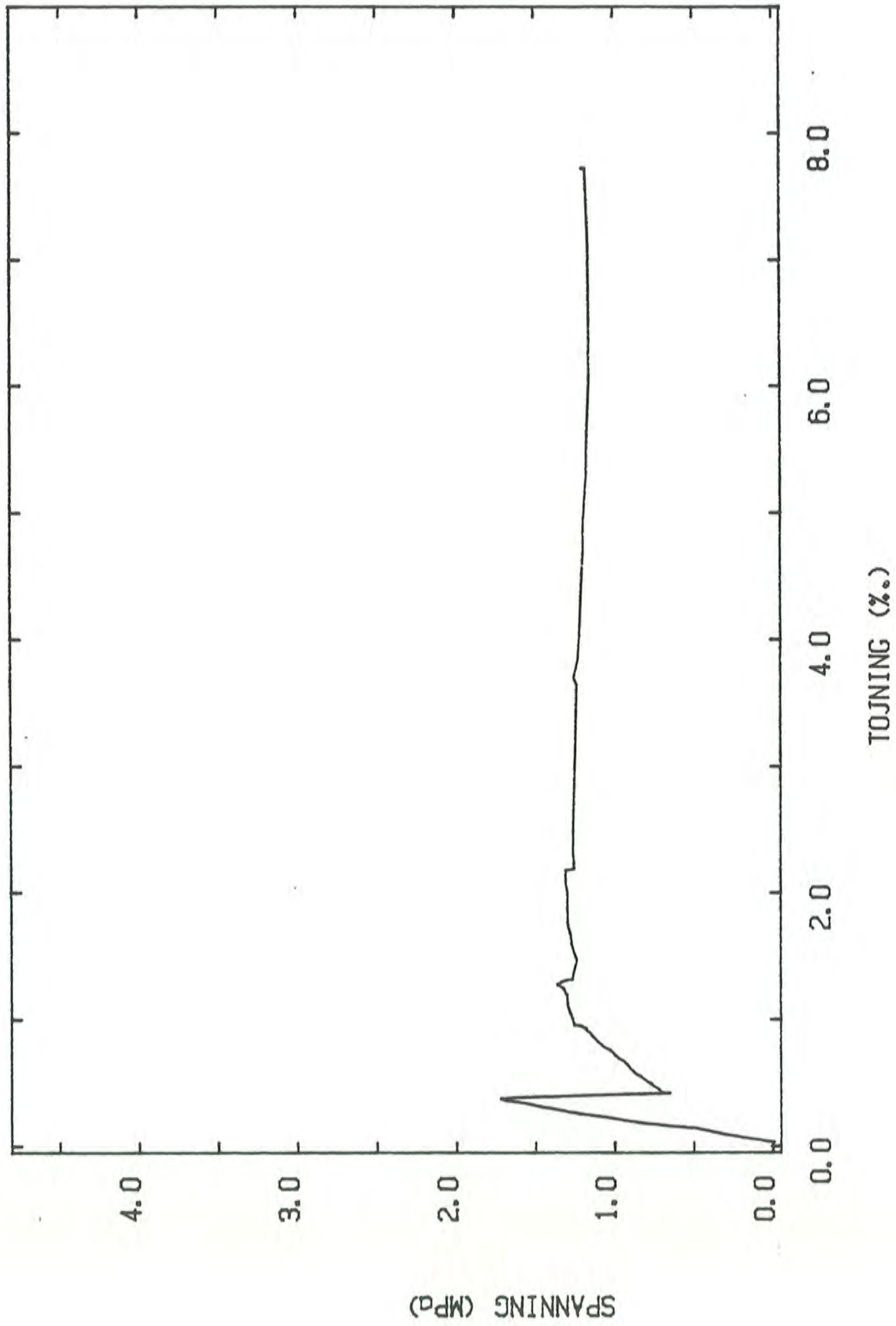
SAMPLE 2 c



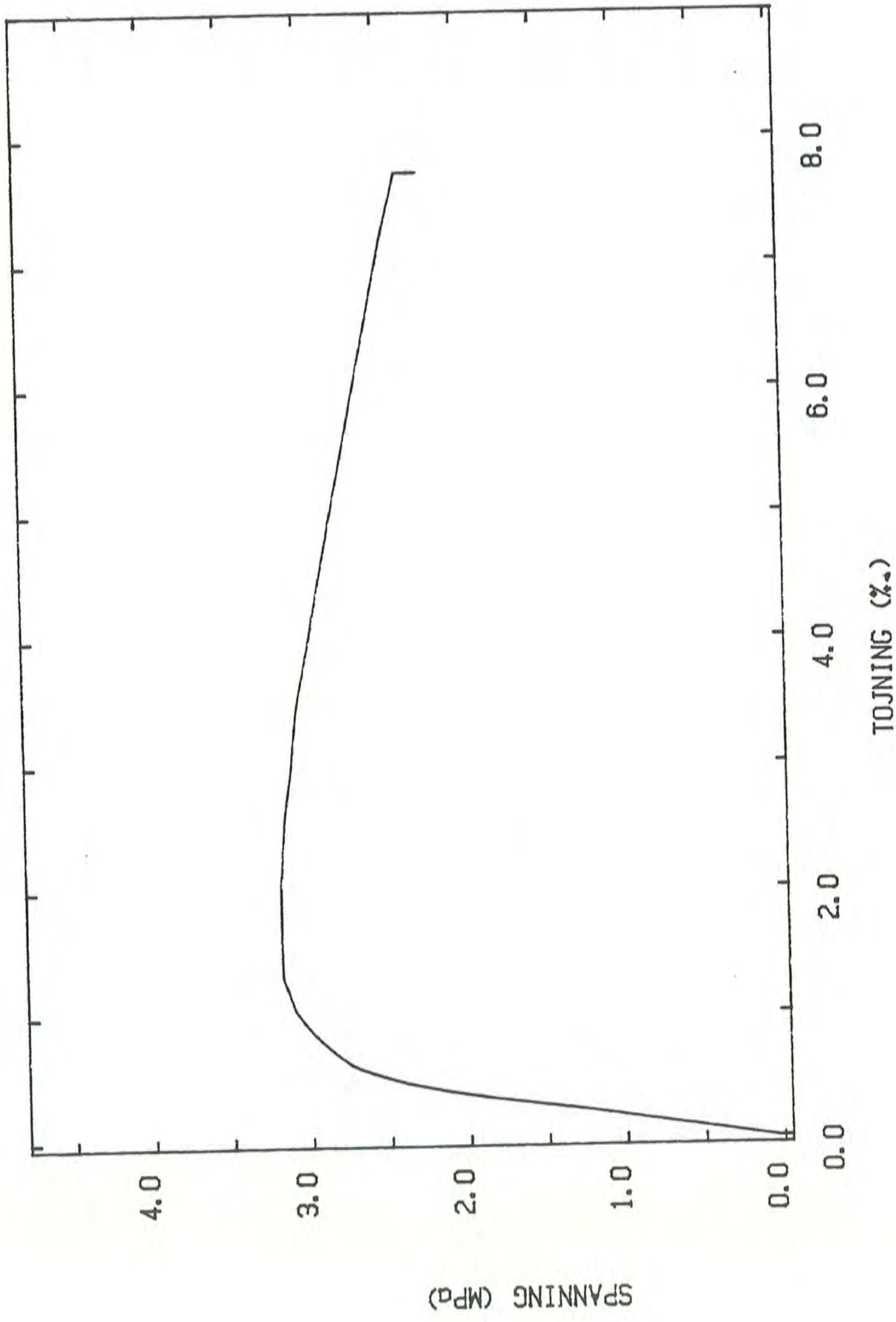
SAMPLE 2 d



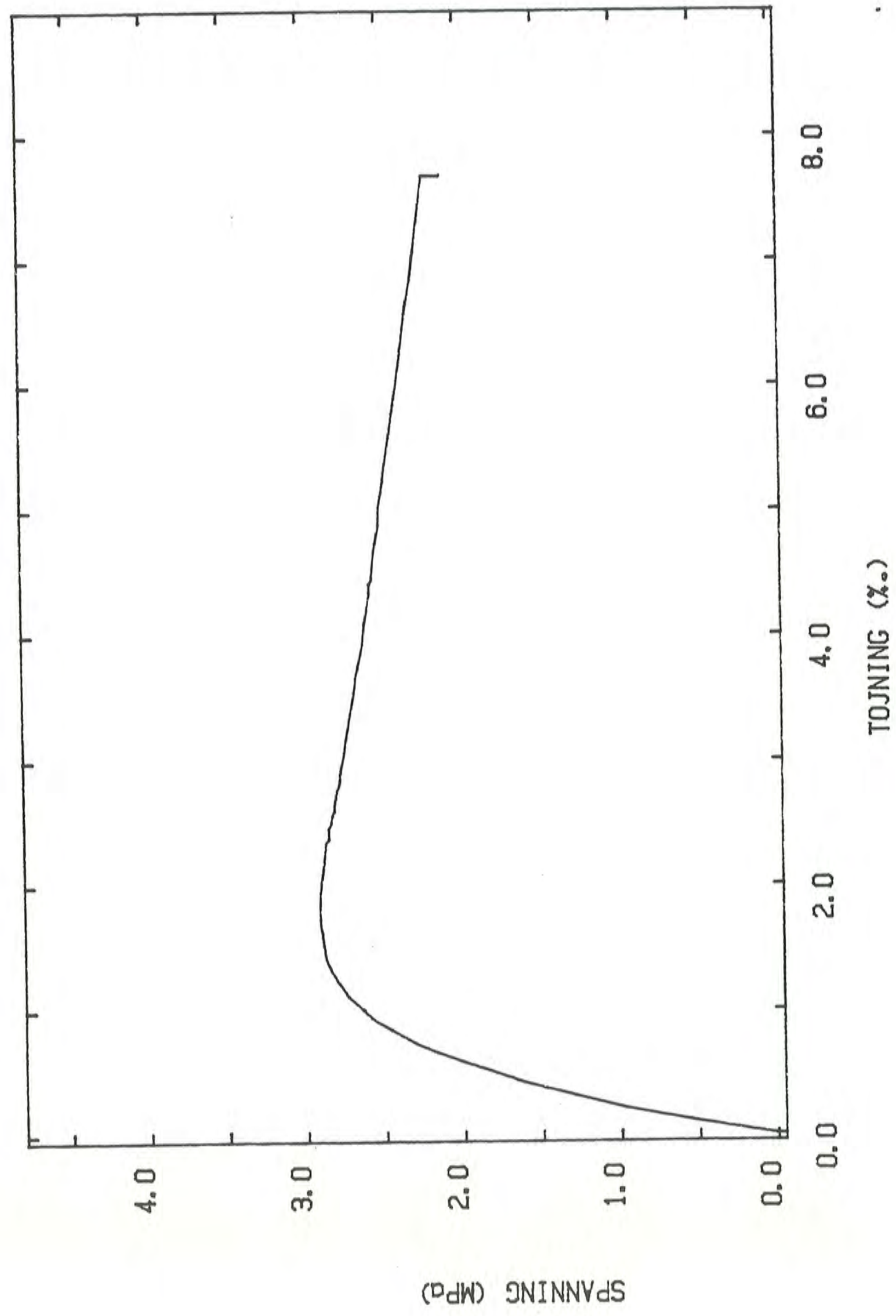
SAMPLE 2 e



SAMPLE 3 a



SAMPLE 3 b



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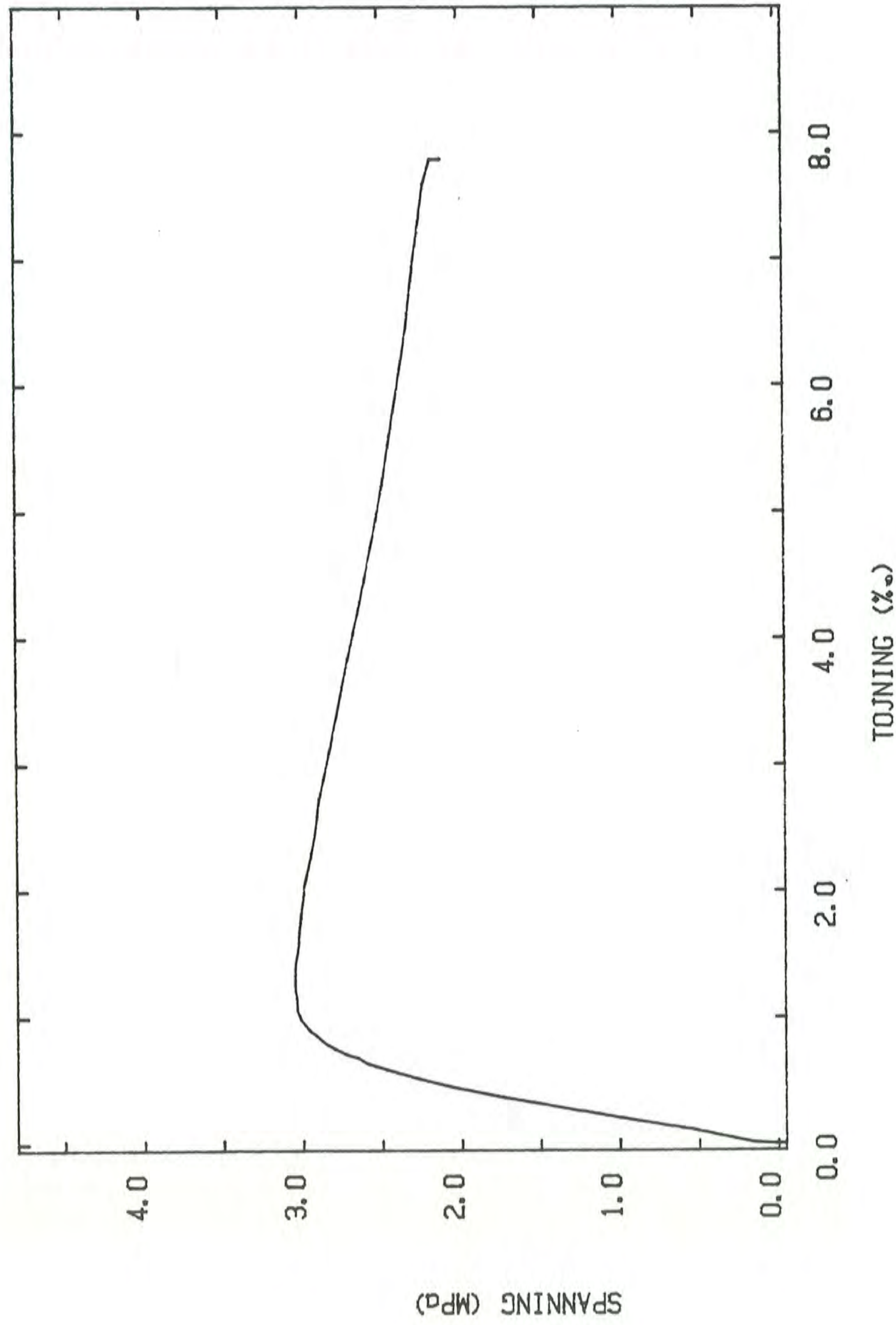

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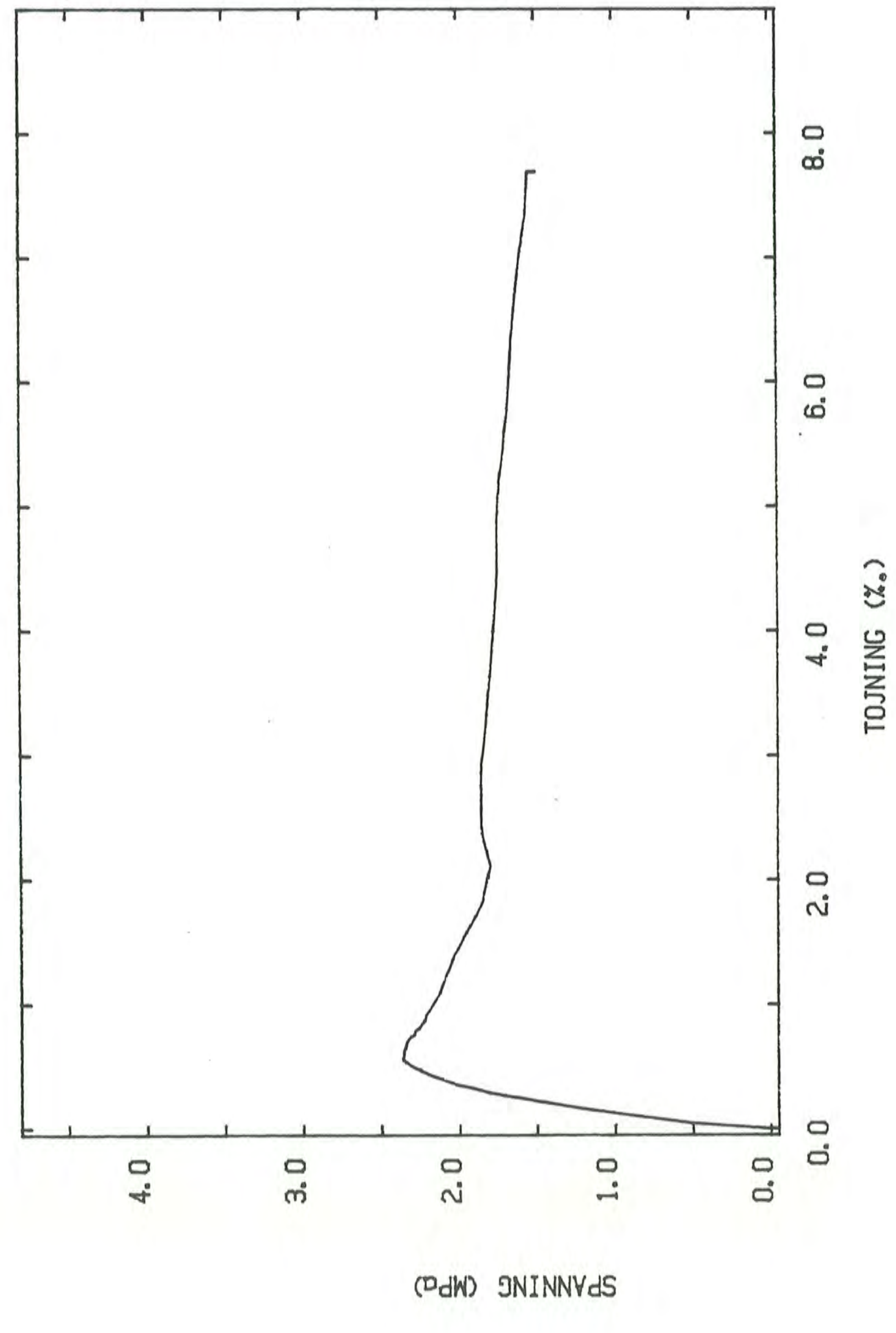

SAMPLE 3 e