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Winter Construction Committee: RILEM Recommendations for Winter Concreting

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WINTER CONSTRUCTION COMMITTEE

Rilem recommendations for winter concreting [1]

FOREWORD

In February 1956, an International Symposium on Winter Concreting was held in Copenhagen under the auspices of RILEM. The Symposium, which was attended by 250 specialists from 20 countries, demonstrated a considerable interest in the subject and promising possibilities for future international cooperation so that a special committee was set up during the concluding session to investigate the possibilities.

In connection with the completion of the Proceedings from the Symposium, the committee met in Copenhagen in September 1956 and prepared a recommendation to the Permanent Committee of RILEM, in which it suggested the setting up of a general RILEM Winter Construction Committee including members from interested countries.

The committee drew the attention to the possibility of establishing three subcommittees: 1) on Climatic Conditions, 2) on Concrete Technology and 3) on Practical Winter Construction Methods.

(1) Le texte français de ce rapport a été publié dans les Annales de l'I.T.B.T.P. nº 190, octobre 1963, Série béton-béton armé 72.

The Recommendations for the establishment of a general Winter Construction Committee including subcommittees, was accepted at the following meeting of the Permanent Committee and the Winter Construction Committee was set up including the following members:

S. G. Bergstrom, Tekn. dr. Cement- and Concrete Institute, Stockholm 70, Sweden.

C. J. Bernhardt, Civil Engineer. Norges tekniske Hojskole, Trondheim, Norway.

J. G. Buitink, Eng. Director. Storm van's-Gravesandeweg 23, Wassenaar, Holland.

C. R. Crocker, Chief of Construction Section. National Research Council, Ottawa 2, Ontario, Canada.

J. Grzymek, Professor. Akademia Gerniczo-Hutnicza, Krakow, Ul. Krzemionki 11, Poland.

Jorn Jessing, Civil Engineer. Danish National Institute of Building Research, Borgergade 20, Copenhagen K, Denmark (Secretary of the Committee).

R. Metzner, Civil Engineer. Deutscher Beton-Verein, Krohnskamp 1, Hamburg 39, Germany.

S. A. Mironov, Professor, D. Sc. USSR Academy of Building and Architecture, Moscow K-9, Puschinskaja 24, USSR.

A. Nykanen, Professor. State Institute for Technical Research, Lonnrotsgatan 37, Helsinki, Finland.

N. M. Plum, Civil Engineer, Director of Research. Danish National Institute of Building Research. Borgergade 20, Copenhagen K, Denmark.

T. C. Powers, Research Councelor, Portland Cement, Association, Research and Development Laborat tories, 5420 Old Orchard Road, Skokie, Ill. USA.

Franz Uhl, Civil Engineer Donaukraftwerke A.G. Wien I, Hohenstaufengasse 6, Austria (representing Hans Bohmer, Director).

L. Vironnaud, Chef de Service. Laboratoire du Bâtiment et des Travaux Publics, 12, rue Brancion, Paris-XVe, France.

A. Voellmy, Dr. Ing., EMPA, Leonhardstrasse 27, Zürich, Switzerland.

The committee met for the first time in September 1958, and Mr. Plum was elected President.

At this occasion, a working programme was established and the members of the subcommittees appointed.

Since then the Main Committee has met regularly each year and the subcommittees have held numerous meetings each year.

The subcommittee on Concrete Technology includes the following members:

P. Nerenst, Chief Engineer. H.H. Gasbeton A/S, Gl. Kongèvej 60, Copenhagen V, Denmark (Chairman of the subcommittee).

C. J. Bernhardt, Civil Engineer (from the Main Committee). (Secretary of the subcommittee).

U. Danielsson, Civil Engineer, Cement- and Concrete Institute, Stockholm 70, Sweden.

J. Grzymek, Professor (from the Main Committee).

R. Metzner, Civil Engineer (from the Main Committee).

A. Meyer, Dr. Ing., Direktor, Laboratorium der Westfalischen Zementindustrie, Beckum bez. Münster, Parallelweg 20, Germany.

S. A. Mironov, Professor (from the Main Committee).

Goran Moller, Civil Engineer, A.B., Gullhogens Bruk, Skovde, Sweden.

K. E. C. Nielsen, Civil Engineer, Technical Information Office of Danish Cement Works, Chr. Brygge 28, Copenhagen V, Denmark.

A. Nykanen, Professor (from the Main Committee).

T. C. Powers, Research Councelor (from the Main Committee).

J. M. L. Trouw, Civil Engineer, ENCI-CEMIJ, Herensgracht 507, Amsterdam-C, Holland.

Franz Uhl, Civil Engineer (from the Main Committee).

L. Vironnaud, Chef de Service (from the Main Committee).

In principle work completed the was in 1962 and at the meeting of the Main Committee in Stockholm 1962, the main text was accepted for publication. Later it was also accepted by the Bureau of RILEM.

In the meantime, the various appendices have also been completed to the effect that in the following pages, it is possible to present the complete material to the public.

The subcommittee has, during its activities also received valuable assistance from :

Jorn Jessing, Civil Engineer (from the Main Committee), S. Pihlajavaara, Lic. Sc. State Institute for Technical Research, Otaniemi, Finland, E. Rastrup, Civil Engineer, Carlsberg Breweries, Copenhagen, Denmark.

Though all basic problems, as will appear from the appendix, have not been finally cleared, it is our belief that the Recommendations are based on such solid knowledge that they can form a reliable basis for individual Recommendations in the member countries for some time to come.

Within a year or two, the Main Committee hopes that it will also be possible to publish the findings of the other subcommittees.

In my capacity as President of the Main Committee, it is on this occasion my privilege to thank all participants in the work for their valuable contributions to the result and their never failing interest in achieving a final report which could be accepted by all parties involved.

Niels Munk PLUM
President of the Committee,
Danish National Institute of Building Research.

INTRODUCTION

The present recommendations are dealing with concreting in cold weather, i.e. concreting at temperatures below + 5° C, and pertain to normal reinforced concrete construction. They treat mainly such problems, which are connected with winter concreting in particular, as it is assumed that the rules necessary for obtaining concrete of high quality at normal temperature are observed.

The recommendations are of a general character and should be regarded as a basis for the establishment of detailed specifications for winter concreting in different countries in accordance with national standards and other specifications.

CONTENTS

1. GENERAL PRINCIPLES FOR WINTER CONCRETING

The general principles for winter concreting are shortly stated in the following, which deals with the influences of low temperatures and the methods, which can be applied to diminish or avoid harmful effects. The more detailed technological considerations, which have lead to these statements, can be found in the Appendix.

1.1 Influences of low temperature

Cold weather may influence concreting in various ways, which are dealt with separately below. At the same time it is indicated how these effects can be counteracted.

1.11. Low temperatures cause delayed strength development.

When the temperature is falling to about + 5° C or below the development of concrete strength is essentially retarded compared with the development at normal temperatures. The hardening period necessary before removal of forms is increased, and experience from concreting at normal temperature can not be used directly.

Consequently it is necessary to maintain proper curing conditions and to take the prolonged curing time into consideration before forms and insulation can be removed.

1.12. Freezing of concrete at early ages.

At temperatures which involve that the concrete is freezing at an early age there is a risk that the concrete is suffering inrepairable losses of strength and other qualities, e.g. the permeability is increased and the durability is impaired.

Consequently it is necessary to prevent that the concrete is freezing before a sufficient degree of hardening has been obtained.

1.13. Repeated freezing and thawing of concrete.

If concrete is exposed to repeated freezing and thawing after final set and during the hardening period the final qualities of the concrete may also be reduced.

Consequently it is necessary to protect the concrete against cycles of freezing and thawing until the concrete may withstand them without suffering damage.

1.14. Stresses due to temperature differentials.

It is a general experience that large temperature differentials within the concrete member, may promote cracking and have a harmful effect on the durability. Such differentials are likely to occur in cold weather at the time of removal of forms and insulation.

Consequently it is necessary to protect the concrete until the temperature differences within the concrete and between the concrete and the surroundings become rather small.

1.2 Classification of winter concreting methods

Concreting in winter time calls for extra precautions and involves closer control than concreting at normal temperature, but if the necessary steps are taken the quality of the concrete placed in winter time will not be lower. The protection of concrete against freezing and maintenance of proper curing conditions to reach sufficient strength for removal of forms can be obtained in principally different ways, which are treated in separate sections below.

The freezing point of water may be depressed to — 10° C or below by adding large amounts of chlorides. This *cold concrete method* is only used in special cases for mass concrete without reinforcement and therefore not described here.

1.21. Change of mix design.

A more rapid development of strength and heat of hydration may be obtained by increase of cement content or by use of a cement of higher activity (cf. 2.1).

This method can be used in cold weather at temperatures above the freezing point.

1.22. Thermos method.

This method is based upon prevention of heat losses, i.e. that the concrete is placed at such initial temperature that the heat of hydration in combination with insulation is able to maintain the concrete at a sufficient high temperature level to obtain resistance to freezing and in some cases even to obtain sufficient strength for form removal.

The successful application depends on the thickness (volume-to-surface ratio) of the structural member, the concrete temperature and the activity of the cement.

This method can be used during moderate and short periods of frost.

1.23. Postheating method.

By this method artificial heating of the surrounding air is used to keep the temperature of the concrete at a certain level above the freezing point and thus maintain the development of strength. The same effect can be obtained by heating the concrete proper, e.g. by electricity.

This method can be applied under severe conditions also for thin structural members during prolonged frost.

1.3 Choice of procedure

Which of the three methods to choose in an actual case depends upon local conditions: i.e. the climate, the cost of different cement types, the massiveness of the structure, the cost of insulation against the cost of heating and enclosure material, the cost of labour etc., hence no general rules can be established. In some cases a combination of the methods might be advantageous.

These principles are further treated in later sections of the Recommendations and the corresponding parts of the Appendix.

2. CONCRETE CONSTITUENTS

Winter concreting requires a good quality of the concrete ingredients.

2.1. Cement

The activity of cements can normally be classified according to the heat of hydration developed during the first 72 hours at + 5° C. In Table 1 is shown a proposed classification of cements in accordance with this principle.

The amount of heat developed by hydration should be determined by the method described in the Appendix. The errors of the testing method combined with influence from the age of cement and variations in the quality during production makes a more detailed classification irrelevant.

The classification does not apply to aluminous cement. The application of this cement type is limited to special cases because its tendency to recrystallisation can reduce the strength. Its ability to protect the reinforcement against corrosion is lower than for Portland cement.

2.2 Aggregate

Aggregate should not contain minerals, which are vulnerable to freezing, e.g. large porous particles of lime and chert. This is especially important if there is risk of the concrete being exposed to several freezings.

Aggregate should be free from ingredients retarding the hydration, e.g. organic impurities (humic acids), as the presence of retarding substances are particularly dangerous during low temperature curing.

TABLE 1 Classification of cements.

Classification	Cement type (commonly termed)	Heat development at 5 °C during first 3 days [cal/g cement]	Notes
Q 55	Rapid/superrapid	> 50	—
Q 45	Standard	40 — 50	—
Q 35	Slow	30 — 40	Moderate heat
Q 25	Very slow	20 — 30	Low heat (only for mass concrete)

2.3 Water

Water should comply with normal requirements for good quality concreting, as e.g. impurities retarding hydration are dangerous.

2.4 Admixtures

Admixtures may only be applied, if their effect on the concrete properties is fully known. They should not delay the process of hydration at low temperature, nor increase the permeability of concrete to gases and water, and should not promote the corrosion of the reinforcing steel. It is required that the admixtures are tested with regard to their suitability in all cases at a temperature of + 5° C as well as at + 20° C, and by addition of the prescribed quantity as well as the double and the half amount of this quantity. If a combination of different admixtures is used the tests should be carried out with the actual combination.

During winter concreting mainly accelerators and air entraining agents are applied.

2.41. Accelerators.

Accelerators used in the proper amount decrease the time of set, increase the early strength and the development of heat of hydration at the beginning of the hardening period. The freezing point of the mixing water will not be lowered to any significant degree. The addition of an accelerator may decrease the final strength and increase shrinkage and electrical conductivity.

The risk of efflorescence on the concrete surfaces may also be increased.

Consequently a limitation in the use of accelerators is advisable.

Accelerators containing chloride increase the risk of corrosion of the reinforcing steel, depending upon type of cement, of steel and exposure.

Consequently such admixtures are not recommended for normal reinforced concrete. The use of admixtures containing chloride should not be permitted in prestressed concrete.

2.42. Air-entraining agents.

This type of admixture will in the proper amount increase the resistance of the hardened concrete to freezing and thawing, and normally at the same time improve the workability of the fresh concrete. The compressive strength of air-entrained concrete at 28 days should not be less than 85 % of a reference mix without air-entrainment. A small increase of the shrinkage may occur.

For prestressed concrete air-entraining agents should only be permitted, if they do not contain chlorides.

3. MIX PROPORTIONS

During winter concreting greater careis generally required in proportioning the mix than at other times of the year.

3.1 Water and cement content

The w/c-ratio should be kept as low as possible (see section 5.1) preferably by increasing the cement content rather than by reducing the water content.

3.2 Air content

In those cases where concrete is expected to be exposed to repeated freezing and thawing cycles in a condition of high degree of saturation, air-entraining agents should be used. The controlled air content should be from 4 % to 6 %. The greater air values should be used for concretes where the maximum aggregate size is small.

4. HEATING OF MATERIALS

4.1 Allowable heating of materials

High temperatures of freshly mixed concrete are always objectionable, as the quality of concrete may be impaired by false set, strength reduction, moisture loss and cracking due to thermal contraction and shrinkage.

To minimize such risks the following temperature limits for concrete constituents are recommended:

a) No part of the aggregate should be heated above 100° C.

b) The temperature of water should not be above 60° C, when coming in contact with cement.

The temperature of concrete as mixed should normally not exceed the values in Table 2:

TABLE 2

Maximum temperature of fresh concrete when using different cement types.

Cement type	Q 35	Q 45	Q 55
Temperature of the concrete mix, N_m °C	30	25	20

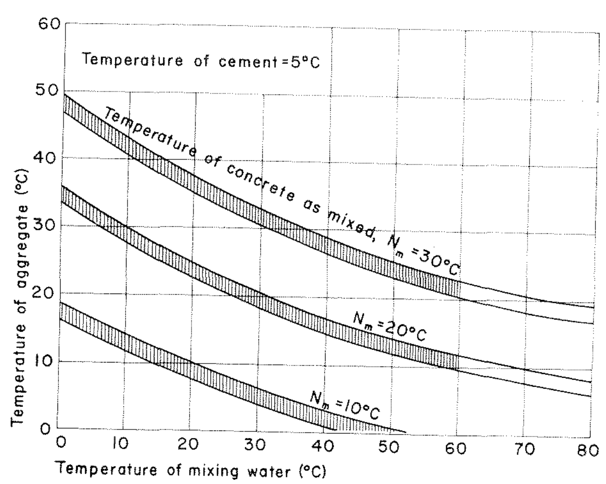


FIG. 1. — **Different temperatures of the concrete mix may be obtained by various combinations of the temperature of the mixing water and the temperature of the aggregate.**

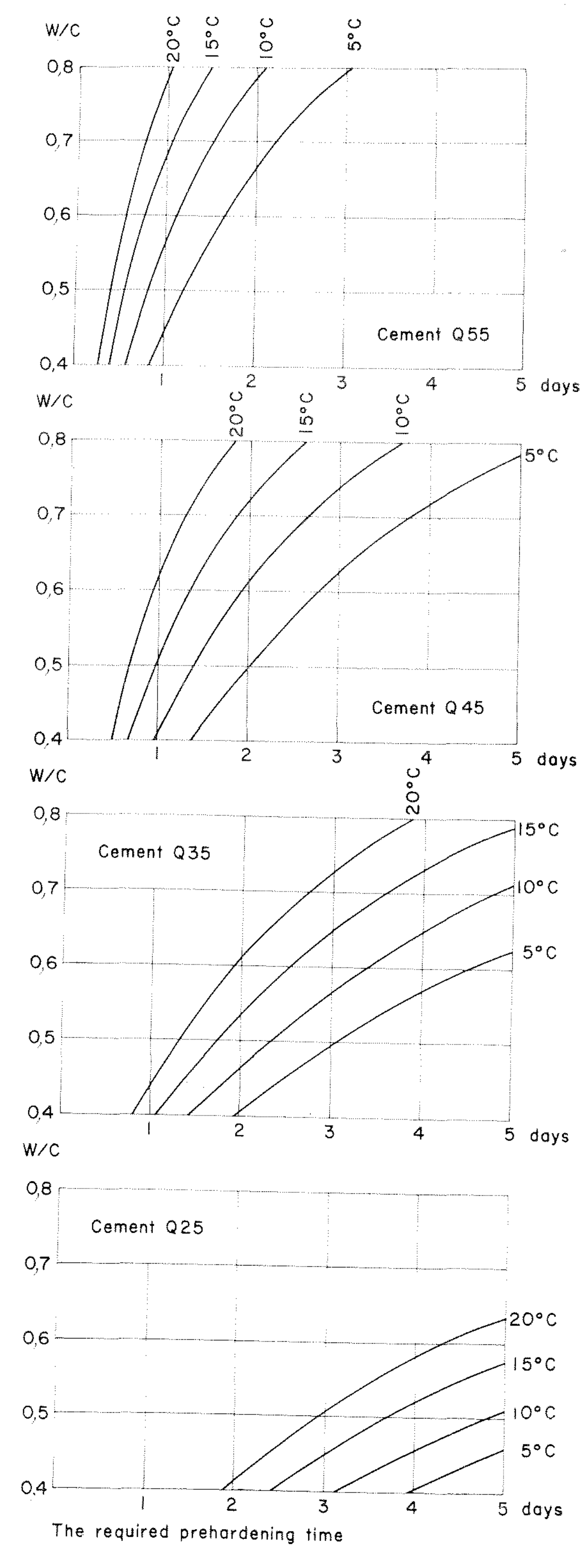


FIG. 2. — **The relation between the required pre-hardening time to obtain resistance to freezing and the W/C-ratio, for different cement types and at different (constant) hardening temperatures.**

4.2 Concrete temperatures

When the proportions of the concrete mix and the temperatures of the constituents are known, the temperature N_m of the concrete mass can be evaluated from figure 1. If the temperature of cement is different from + 5° C corrections can be made according to the rule that a 10° C higher temperature of cement means a 1° C higher temperature of the concrete.

5. OBTAINMENT OF FREEZING RESISTANCE

5.1 Required prehardening time

The resistance to early freezing is obtained, when the concrete has reached a certain degree of hydration, depending on the w/c-ratio. The relation between the w/c-ratio and required prehardening time at different concrete temperatures (20° C, 15° C, 10° C and 5° C) is shown in figure 2. The relationship is only valid for concrete that does not absorb water from the surroundings. After this prehardening time the compressive strength is approximately 50 kg/cm².

The curves shown in figure 2 have to be checked by experiments in each country. Whether the concrete member at varying temperature may obtain resistance to early freezing before it is cooled down to 0° C is treated in the following section.

TABLE 3 Classification of variables.

Changes possible at the planning stage	Changes possible before concreting (at the job site)	Changes possible after casting
Temperature of ambient air (by the postheating method)	Form type	Insulation
Shape (of construction member)	Cement type	Duration of protection
	Cement content	
	Water content (w/c-ratio)	
	Concrete temperature (at placing)	

5.2 Temperature history

The concrete temperature in relation to time in a structure is depending on several factors, viz:

Environment: Temperature of ambient air, insulation (type and thickness).

Construction: Shape (volume-to-surface ratio), form type.

Concrete: Cement type, cement content, concrete temperature (at placing), water content (w/c-ratio).

Duration of protection: Time.

At the design and planning stage it is possible to change all these factors in order to obtain either resistance to freezing or removal of forms within a reasonable time. They can be classified according to the extent to which it is possible to change them (Table 3).

Shape, form type and insulation can, however, be combined in a time constant, τ, which directly indicates the rate of cooling of a concrete member neglecting the heat of hydration. Methods and diagrams for determination of the time constant are given in the Appendix.

The number of variables is then reduced to seven, which are listed in Table 4 with indications of the possibility of changing them in dependence of the different winter concreting methods (cf. section 1.2).

When these factors have been determined for a specific winter concreting project it is possible to set up different equations to determine the temperature history with regard to cooling and the developed heat of hydration in the concrete. Such calculations give directly the time when the concrete has been cooled

TABLE 4

The possibilities of influencing the variables by the different winter concreting methods.

● Characteristic feature of the method.
○ Possible supplement to the method by combination with the other methods.

Variables	Change of mix design	Thermos method	Postheating method
Cement content	●	○	○
Water content (w/c-ratio)	●	○	○
Cement type	●	○	○
Time constant		●	○
Initial temperature of concrete		●	○
Duration of protection		●	○
Temperature of air			●

down to 0° C. In figure 3 is shown examples of the computed temperature history under the assumptions that the temperature of the ambient air is — 5° C and that the time constant has different values ranging from 100 hours to 10 hours.

Whether the concrete has obtained the necessary degree of hardening (the required prehardening time) before it is cooled down to the freezing point may be determined in different ways.

In the Appendix is shown how diagrams and tables have been worked out in Denmark, Germany and Sweden. The diagrams and tables are referring to special cement types obtainable in these countries, and the results can only be used in other countries for cements with similar characteristics with regard to rate of hydration.

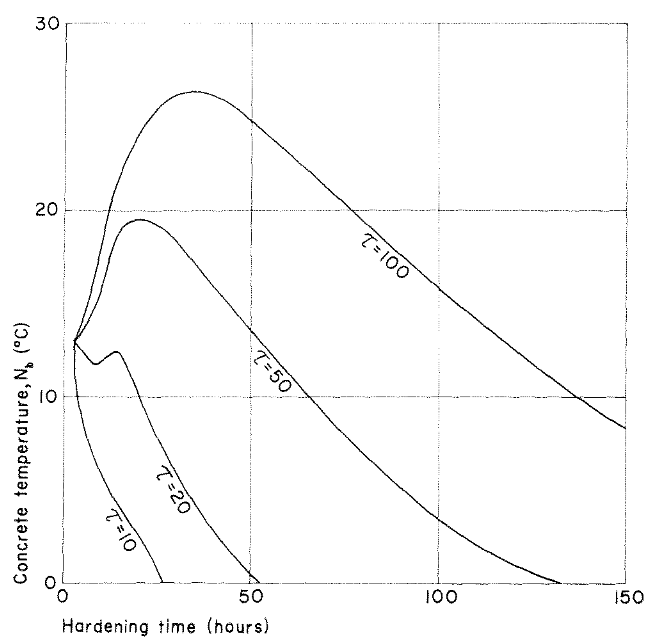


FIG. 3. — Temperature of concrete N_b as function of the hardening time for various values of the time constant, τ, and constant external temperature, N_u = + 5°C. With smaller time constant the concrete is cooled down to 0°C more rapidly and the degree of hardening obtained before freezing is less

τ = 100 hours corresponds to a 1 m wall in 1 1/4″ wooden form

τ = 50 hours corresponds to a 50 cm wall in 1 1/4″ wooden from

τ = 20 hours corresponds to a 20 cm wall in 1 1/4″ wooden form

τ = 10 hours corresponds to a 10 cm wall in 1 1/4″ wooden form

The concrete is made with 300 kg/m³ of a Q45 cement and w/c = 0.6.

6. CRITERIA FOR REMOVAL OF FORMS

The reduced rate of hardening in winter time makes it especially important to control that the concrete at every stage during the building period is capable to withstand the imposed loads with sufficient factors of safety.

Stripping of forms and shores will normally expose the concrete structures to the main part of the dead load.

Considerable stresses may also occur as a result of displacement and uneven settlements of the supports. Vertical as well as horizontal displacements may be caused by frost action in the ground.

In housing construction a special type of live load is caused by piling up of building materials on floors or by the form work being exposed to additional load from the floor above, which in many cases take place at early stages before stripping of forms.

Each of the following main problems are dealt with separately below.

a) The stresses caused by early loading must not exceed a certain percentage of the actual strength of the concrete at the time of application of the load.

b) It must also be ensured that the concrete has obtained sufficient rigidity so that excessive deflections are avoided.

c) The removal of forms will in many cases expose the concrete structure, not only to loading but also to thermal shocks, giving a risk of cracking due to temperature differentials.

6.1 Strength of concrete at stripping

The development of strength as influenced by the temperature may be estimated by the use of time-temperature functions. It should be remembered that these functions are valid for Portland cement only and cannot be applied for other cement types without certain corrections.

The development of strength may also be deduced from concrete specimens, cured at different temperature levels, viz. + 20° C and + 5° C.

The above mentioned principles are of great importance at a planning stage, and are combined with temperature measurements during the actual work a great help as a control of the hardening process. Actual stripping should not take place until the strength has been checked by testing of specimens, which have as far as possible been cured in the same way with regard to temperature and moisture as the most important part of the structure (hardening test).

Non-destructive testing methods may also be helpful in checking the strength especially of the most exposed parts of the structures, e.g. edges and corners — provided that the concrete is not frozen.

6.2 Deformations

The creep of concrete is depending on several factors, among which the relative strength and the moisture content of the concrete at the time of loading is very important. Excessive deformations may occur, if the delayed hardening due to low temperature has not been taken into account when fixing the stripping time. Excessive deflections may also develop if the concrete structure in state of loading is undergoing great temperature or moisture variations.

It is advisable to keep the forms and shores in place as long as economy and job schedule permit.

6.3 Risk of cracking

In order to avoid the formation of cracks the rate of evaporation from the surface should be kept as low as possible.

With increasing thickness of concrete members the risk of cracking is increased because the temperature differentials may become high in cold weather.

Whether temperature differentials will cause cracking depends upon their magnitude and the induced stresses, which are related to the modulus of elasticity and the actual stage of hydration.

7. CONTROL

Concreting in winter time requires that the ordinary concrete control is carried out with the utmost care. The obtained test results should be used for fixing the time of removal of insulation, forms etc or be the basis for taking further precautions at the building site.

In supplement to the ordinary concrete control special emphasis should be placed upon:

a) Determination of the suitability of concrete constituents for winter concreting and control of the fresh concrete.

b) Records of air temperature and measurements of concrete temperature at placing and during hardening.

c) Control of strength development in the structure by testing of similarly cured specimens (hardening test).

7.1 Concrete constituents and fresh concrete

The testing of the properties of the concrete constituents should be carried out in accordance with normal specifications. Of special importance is knowledge on the reactivity of the cement (cf. section 2) and control of the setting time of cement. During winter concreting it is important to control that the w/c-ratio does not exceed the design value (e.g. by measurement of the consistency). When air-entraining agents are used, the air content shall be controlled.

7.2 Measurements of temperature

Representative temperatures for the site should be read at fixed hours and stated in the daily record. For this purpose a thermometer must be placed in such a way that the influences of premises, heaps of material etc. can be neglected. The scale of the thermometer shall permit accurate reading.

To control the hardening process (the degree of hardening) it is necessary to measure the temperature of concrete at placing, at the time of applying the protection and 2-3 times each day until resistance to freezing has been obtained. If later temperature readings are only made once every day it should preferably be done at the same time each morning. From these measurements it is possible to predict the relative strength.

Thermometers should be inserted in those parts of the concrete, where maximum stresses will appear at removal of forms and if possible also in corners, edges etc., where the cooling is expected to be particularly rapid. In figure 4 is an example of where temperature readings could be considered.

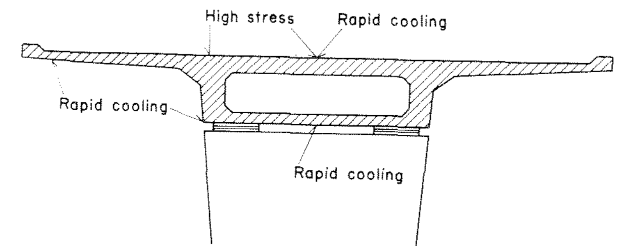


Fig. 4. — **Cross section of a bridge exemplefying locations of high stresses and of intense cooling, where measurements of temperature may be carried out.**

7.3 Control specimens

Under normal concreting conditions test specimens are cast and cured in a standardized way to indicate the "potential" strength properties of the concrete mix (design test). For winter concreting it is recommended that before the job is started tests are carried out of the selected concrete mix prepared in the laboratory at normal temperature and cured at + 5° C.

In addition to this it is under winter concreting conditions required to cast a number of specimens, the curing conditions of which are arranged in such a way that they are exposed to the same temperature and humidity conditions as the actual structure, and preferably similar to those of the most exposed part. These specimens are tested before stripping takes place to ensure that the indications of strength development obtained by temperature measurements are in fact obtained (hardening test).

In order to prevent drying-out of the specimens it is necessary that similarly cured specimens are kept in the moulds until testing takes place.

The specimens never should be tested in the frozen state.

Appendix

CONTENTS

1. GENERAL PRINCIPLES FOR WINTER CONCRETING

1.1. Influences of low temperature.

1.11. Low temperatures cause delayed strength development.

Saul (51 S 14) has in his experiments observed that the compressive strength of concrete was an univalent function of the product of hardening temperature, N, and time, *t*. Consequently concrete of a given mix at the same product has approximately the same strength whatever combination of temperature and time go to make up that product.

Saul, calling this product "maturity" found that the concrete after setting will continue to gain strength slowly in the interval between 0° C and — 10° C. Saul applied this function to concrete steamcured at normal pressure. Bergstrom (53 B 8) has plotted the compressive strength of concrete cured at normal temperature in accordance with Saul's theory, i.e. as a function of a product P, defined as

$$P = \Sigma (N + 10) \Delta t \ [^{\circ}C\ h] \qquad (A\ 1.1)$$

where

N = temperature of concrete during the time Δt [°C]

Δt = hardening time [hours].

Bergstrom found that this function was valid for the strength development within a test series but the curves took different shape for different mixes and types of Portland cement.

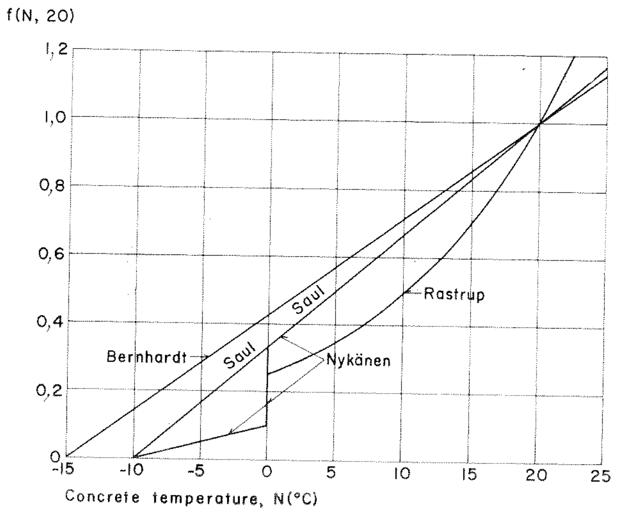


FIG. A.1.1. — **Comparison of different temperature functions (58 N 1).**

If the product P is divided by the constant temperature $N_a + 10$ the duration of the hardening process, t_a, corresponding to a hypothetical constant concrete temperature N_a can be computed.

$$t_a = \frac{P}{N_a + 10} = \sum_0^t \frac{N + 10}{N_a + 10}\,\Delta t = \sum_0^t f \Delta t \sim \int_0^t f\,dt. \quad \text{(A 1.2)}$$

The general temperature function f is defined as the factor which the time-element, Δt, for a certain part of the hardening process at the temperature, N, shall be multiplied by in order to convert it into the time-element, Δt_a, corresponding to the same part of the process at the temperature, N_a.

The function f has to be determined for each type of cement.

The Saul-Bergström maturity-concept (A 1.1.) has been found valid for strengths of concrete produced from Portland cement cured above 0° C.

Nykanen and Pihlajavaara (58 N 1) have found that the hardening of concrete slows down abruptly below 0° C and they suggest for $N_a = 20°$ C the formula:

$$f = 0.2\,\frac{N + 15}{20 + 10} \text{ for } -15° \text{C} < N < 0° \text{C}. \quad \text{(A 1.3)}$$

Bernhardt (56 B 14) has found the compressive strength of Portland cement concrete to be dependent on the function

$$f = \frac{N + 15}{N_a + 15}. \quad \text{(A 1.4)}$$

Rastrup (54 R 1, 56 R 7) has found that the heat of hydration of Portland cement is dependent on the function

$$\log f = k(N - N_a) \quad \text{for } 0 < N < 50° \text{C}. \quad \text{(A 1.5)}$$

The value of k varies by Brigg's logarithm between 0.02 and 0.03, the latter value corresponds to the wellknown observation that the velocity of chemical reactions are doubled by an increase of temperature of 10° C.

The different functions described above are shown in figure A. 1.1.

Various investigators — for instance Bernhardt (56 B 14), McIntosh (56 M 7), Brandt (56 B 15), Danielsson (56 D 7) — have indicated various reasons why the relation between strength and time-temperature product is not universally valid, partly due to the theoretical considerations, partly due to variations in the composition of clinker compounds in Portland cement. Research on development of similar functions for blastfurnace-slag cements and other mixed cement types is needed.

1.12. Freezing of concrete at early ages.

The freezing phenomenon is of significance during winter concreting even if no frost durability is required of the structure after completion. It is necessary to make a distinction between the concepts of *freezing resistance* and of *frost durability*.

The former is defined as the ability of the green concrete to endure the effects of one or a few numbers of freezing and thawing cycles.

The latter is defined as the ability of saturated concrete to resist the effects of freezing and thawing cycles for years without deterioration.

During the last decades the freezing phenomenon of concrete has been studied thoroughly by T. C. Powers and his co-workers (45 P 3) (48 P 22) (49 P 26) (53 P 5) (54 P 5) (56 P 8) (56 P 9) (58 P 10) (59 P 9). If their theories and experimental investigations are complemented with the earlier publications of Taber (30 T 2) and Beskow (35 B 6) on the frost action in soils, the investigations of Collins (44 C 3), Nerenst et al. (53 N 2), Danielsson (58 D 1), Nykanen (58 N 1), Möller (62 M 7), Warris (62 W 7) and numerous other works (55 S 5) (62 N 3) on the freezing of concrete, this problem can be considered fairly clarified qualitatively, and partly quantitatively.

1.121. The freezing of fresh concrete.

Immediately upon mixing the constituents, the concrete has very little strength, and the water is present in the mix mainly in the same state as in wet soils. The freezing phenomenon is similar to that of wet soils, in which the water may form so-called ice-lenses, when it freezes (theory of Taber-Collins).

Both in laboratory experiments and in practice the formation of ice lenses has been established by slow freezing of concrete. By more rapid freezing the ice will be distributed rather uniformly through the green concrete, often in form of needle-shaped ice crystals in the capillaries and on the boundary surfaces of the aggregate grains (56-15).

When the concrete has already gained some strength the formation of ice lenses and ice needles is normally prevented (55 P 3) (56 N 2) (58 N 4).

1.122. The freezing of hardened concrete.

When the strength of the cement paste of sufficiently hardened concrete prevents the formation of ice lenses, the expansions occuring at the freezing of water in saturated concrete forces water deeper into the concrete in accordance with the progress of freezing. If the permeability of concrete is low, the arising hydraulic pressure may damage the cement paste unless the concrete contains sufficiently closely spaced air pores into which the expanding water can escape without producing pressure.

If the hydraulic pressure exceeds the tensile strength of the cement paste the concrete will be damaged. In the following the variables effecting the hydraulic pressure are considered.

The degree of saturation of the cement paste has a great effect upon the hydraulic pressure. In practice, evaporation and self-desiccation frequently reduce the degree of saturation below the critical limit, and no frost damage will occur. These circumstances are thus of very great importance.

With increasing amount of water freezing per degree of temperature drop, the hydraulic pressure increases.

Low permeability increases the hydraulic pressure. Since the permeability diminishes with increasing age and strength of the concrete, the frost resistance of the concrete will thus not improve if the other factors remain unchanged.

The spacing of the air bubbles has a strong influence upon the magnitude of the hydraulic pressure.

Powers' hydraulic pressure theory concerning frost damage to concrete can be stated briefly as follows (55 S 5):

a) Ice first forms at the cold surface, sealing off the interior of the specimen.

b) Pressure exerted by expansion due to ice formation forces water inward to less saturated regions.

c) The relatively high resistance to flow of water in concrete sets up hydraulic gradients which exert pressure on the pore walls.

d) The hydraulic pressure increases with increasing rate of cooling, degree of saturation and spacing of pores, and with decreasing permeability and pore size.

e) When the hydraulic pressure surpasses the tensile strength of the concrete the pore walls are ruptured and the concrete is damaged.

The same process as described above may be applied to the freezing of the aggregate, but naturally the structural factors of the aggregate have great influence on the durability to freezing. The water absorption capacity of dense aggregates is so small that freezing can not damage them. In porous aggregates the size of the pores is usually much larger than in the cement paste and, consequently, the resistance to flow of water much smaller in the aggregate than in the cement paste.

The risk of freezing damage from the aggregate proper (pop-outs) is dependent on its porosity, sorption characteristics, permeability etc. and can not be predicted with any accuracy. For any type of porous aggregate the risk of pop-outs is increasing with the diameter of the stone. Further research on this problem is needed.

1.123. On the prevention of freezing damage.

a) When concrete freezes while in fresh state, it expands owing to the freezing of its water. As a consequence, the pore volume of the concrete increases and the adhesion between the different parts of mix is impaired. Concrete is therefore damaged when it freezes in the fresh state.

b) Water-saturated, hardened concrete without air-entrainment is not resistant to frost since it does not provide adequate space for the expansion of water at freezing. The air voids which are normally entrapped in the concrete are non-uniformly distributed, they are too few, and their spacing is too great. In standard concrete, evaporation and self-desiccation reduce the degree of saturation and at the same time the strength increases. It is thus possible already at a fairly early stage of hardening to obtain freezing resistance, so that the concrete is no more damaged by freezing (56-24). The investigations concerning the freezing resistance of concrete are still incomplete.

c) For winter concreting it is required that only frost resistant aggregate is used. No admixture will significantly increase the resistance to freezing of the aggregate proper. The compliance with this requirement may be proved by suitable experimental technique (61 T 5).

1.13. Repeated freezing and thawing of concrete.

If concrete is required to possess frost durability, this implies that it has to be able to endure the repeated effect of frost for many years. In usual building structures the concrete is comparatively dry and therefore frost resistant. If a concrete structure is subjected to the alternating influence of water and frost and if frost durability is required then it is necessary to provide for the entraining of air bubbles in the concrete with sufficiently uniform spacing (54-13) (54-28) (56-23) (56-24).

1.14. Stresses due to temperature differentials.

This problem will be treated in item 6.3, Risk of cracking.

2. CONCRETE CONSTITUENTS

2.1. Cement.

During winter concreting it is normally an advantage to use cements with high activity, i.e. rapid development of heat of hydration and strength.

The reactivity of cement is to a very high degree dependent upon the cement clinker composition and is increasing with increasing amounts of C_3S, tricalcium-silicate, and C_3A, tricalcium-aluminate.

The reactivity of cement is greatly influenced by the petrographic structure of the tricalcium-silicate as pointed out by Grzymek (59 G 1).

For a given composition a higher early strength is obtained the more finely the cement is ground.

For very slender structures exposed to cooling shortly after placing it is recommended to use cement ground to a specific surface of 4.500 cm²/g (Blaine). For winter concreting jobs where a slower strength development can be tolerated the grinding may be correspondingly coarser.

The reactivity of cement is decreased when it contains ground mineral additives which are nearly inert at normal temperature.

The above mentioned relations between reactivity and cement composition and fineness are known to apply to cement hardening at normal temperature but Russian experiments and experience presented by Mironov (58 M 7) indicates that the highly reactive cement will continue to develop strength at temperatures below 0° C down to — 10° C after an initial hardening period and do the same down to — 5° C even when exposed immediately after the placing. Cements of low reactivity either due to coarse grinding or due to the addition of inert mineral additives will cease to develop strength to any significant degree at temperatures below the freezing point.

It has also been found in the USSR that regrinding of the cement either by the wet or by the dry process just before the mixing has a beneficial effect on the activity of the cement. Further a considerable improvement of the quality of the concrete through a prehydration of the cement even of short duration and without re-grinding has been reported. Prehydration and re-grinding will only be applicable in special cases (61 M 10).

The behaviour of cement at low temperature (+ 5° C) can be characterized on the basis of the development of compressive strength, or heat of hydration, at an early age (3 days).

Classification of cement on the basis of the compressive strength attained after three days is possible, in principle. For simplification of the testing, test specimens are made and tested in the usual way at normal temperature (about 20° C). The curing of the specimens takes place at 5° C (±1° C) and is started 15 minutes after the mixing of the mortar and continues up to immediately before the testing. The classification of cement on the basis of the compressive strength after three days, attained under the described conditions of storage, is likely to afford certain advantages.

The committee has, however, decided to use the heat of hydration developed at an age of three days as criterion for the classification, because the testing procedure applied in the different countries for compressive tests are not giving comparable results.

When this is supplemented with the calculated value of the amount of heat of hydration at ultimate hydration a fairly good picture of the relationship between time and heat of hydration at 5° C is obtained, the relationship after three days being of the form (56 R 7) :

$$Q = q + (Q_\infty - q)\, e^{-\frac{\alpha}{\sqrt{t}}} \qquad (A\ 2.1)$$

where

Q is the heat of hydration

q is the heat of hydration after 30 min. (abt. 4 cal.)

Q_∞ is the ultimate heat of hydration and may be estimated by the following formula:

$$Q_\infty = 1.2\ (\%\ C_3S) + 0.6\ (\%\ C_2S) + 3.2\ (\%\ C_3A) + 1.0\ (\%\ C_4AF) \qquad (A\ 2.2)$$

t is the time from the moment of mixing

α is a constant dependent on the type of cement, estimated from Q at 3 days.

The formula A 2.2 is based on Verbeck's experiments (62 C 4) carried out with $w/c = 0.4$ at 21° C.

A detailed discussion of methods of measuring the heat of hydration is given in section 7.11.

3. MIX PROPORTIONS

3. 1. Water and cement content

As to water and cement content, the same rules as for for ordinary concreting should be observed. Excessive water contents in the concrete should be avoided.

3.2. Air content.

The optimum amount of entrained air, which gives the concrete frost durability is dependent on the cement content and the maximum size of aggregate. Such values (by per cent of the concrete volume) are shown in Table A 3.1 (54-13). (See page 17.)

TABLE A 3.1

Optimum air content for frost-resistant concrete (per cent of concrete volume).

Cement content C [kg/m³]	Max. aggregate size, mm				
	4	8	16	32	64
200	7.5	6.0	5.0	4.5	4.0
250	6.5	5.0	4.5	4.0	3.5
300	6.0	4.5	4.0	3.5	3.5
350	6.5	5.0	4.0	3.5	3.5
400	7.0	5.5	4.5	4.0	4.0
450	7.5	6.0	5.0	4.5	4.5

4. HEATING OF MATERIALS

4.1. Allowable heating of materials.

The temperature limits stated in the Recommendations are chosen with regard to the normal period for transportation, placing and compacting of the concrete. If the reduced setting time is taken into consideration higher concrete temperatures than those indicated may be justified.

4.2. Computation of concrete temperatures.

Fig. 1 in the Recommendations has been worked out for three different initial temperatures, N_b, of concrete, defined as the temperature after placing and immediately after the protecting cover has been placed. With a view to loss of heat during mixing, transportation and placing, the temperature of the

TABLE A 4.1

Example of calculation of the temperature of a concrete mix from the temperature of the components.

Materials		Specific heat	Water equivalent	Temperature	Amount of heat
Type	Quantity P_d [kg]	c_d [kcal/°C kg]	$P_d c_d$ [kcal/°C]	N_d [°C]	$N_d P_d c_d$ [kcal]
cement	300	0.2	60	0 °C	0
sand	735	0.2	147	2° C	294
gravel	1010	0.2	202	2° C	404
water in sand: 5 %	36	1.0	36	2° C	72
water in gravel: 1 %	10	1.0	10	2° C	20
gauging water	150	1.0	150	60 °C	9000
TOTAL	2241		605		9790

concrete during mixing, N_m, must be somewhat higher.

The loss of heat in the period from mixing to protection is estimated to 15 % per hour of the temperature difference between concrete and surrounding air, as stated in the formula below (53 N 4):

$$N_b = N_m - 0.15\,(N_m - N_u)t \qquad \text{(A 4.1.)}$$

where

N_m = mixing temperature [° C]

N_b = temperature immediately after placing of protection [° C]

N_u = temperature of surrounding air [° C]

t = time between mixing and protection [hours].

The formula A 4. 1 is a practical approximation. A more exact treatment is given by Pihlajavaara (60 P 7).

The amounts of heat contributed by materials to the heat contained in the concrete mix are determined as products of weight of material, specific heat and temperature. The temperature of the mix can be found then from.

$$N_m \,\Sigma\, P_d c_d = \Sigma\, N_d P_d c_d, \qquad \text{(A 4. 2)}$$

where

N_m = mixing temperature [° C]

P_d = weight of each component [kg]

c_d = specific heat of each component [kcal/° C kg]

N_d = initial temperature of each component [° C].

The product on the right side of the equation is computed separately for each component and then summed up.

Example

In this example heating of water is assumed, whereas aggregates are not heated. The amount of materials corresponds to 1 cub. m of concrete, and the values are taken from the mix design data. The air temperature is $N_u = -5°$ C. The calculation is shown in Table A 4.1.

From this the initial mixing temperature of the concrete is found: (A 4.2)

$$N_m = \frac{9790}{605} = 16° \text{C}.$$

Assuming that the protection of the concrete is placed 45 minutes after the mixing the temperature, N_b is found from equation A 4.1 :

$$N_b = 16 - 0.15\,(16 - (-5)) \cdot 3/4 = 13° \text{ C}.$$

5. OBTAINMENT OF FREEZING RESISTANCE

5.1. Required prehardening time.

T.C. Powers (62 P 3) has shown that the resistance of concrete to freezing is obtained when the formation of ice can take place without causing destructive forces. During the hydration process of green concrete an autogenous desiccation takes place and when the degree of saturation has reached below a critical value, *s*, no damage will occur (*s* is defined as the fraction of capillary space full of water — neglecting the air bubbles). The autogenous desiccation may be expressed as function of the *w/c*-ratio and degree of hydration combined with constants related to the specific type of cement. Powers has computed the critical degree of saturation to $s = 0.97$ by using one point of Göran Möller's curve (62 M 7), showing the empirical relationship between required prehardening time and *w/c*-ratio, and from this computed the whole curve, which is very close to the empirical curve (Fig. A 5.1). The relationship is only valid for concrete which has no access to imbibe water from the surroundings :

$$\log t_{20} = 1.18 \text{ w/c} + 0.53 \qquad \text{(A 5.1)}$$

where

t_{20} = the required preharding time at + 20° C

w/c = the water/cement-ratio.

This relationship is valid for the cements used in the tests treated by Göran Möller and in order to extend the results to cements with other rates of hydration, Rastrup has made the necessary computations (62 R 5). Rastrup is assuming a critical degree of saturation $s = 0.96$, which is on the safe side.

By expressing the degree of hydration as the ratio between the heat of hydration Q (at the time *t* and temperature N) and Q_∞ (at ultimate hydration) and utilizing the functions (53 N 2) (56 R 7) between Q and non-evaporable water, w_n, and the influence of temperature on the hydration process the necessary prehardening time has been computed for cements with varying reactivity as classified in the Recommendations. The results are shown in the table below and used in figure 2 in the Recommendations.

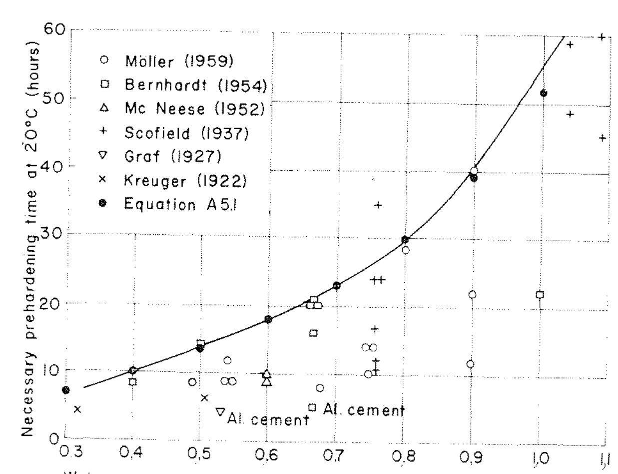


FIG. A.5.1. — **Data from Goran Moller compared with calculated results (Eq. A.5.1.).**

TABLE A 5.1

The required prehardening time in hours at + 20° C as function of w/c-ratio for cements with varying reactivity.

w/c =	0.4	0.5	0.6	0.7	0.8
Cement Q 55	7	10	14	19	26
Q 45	12	17	24	32	44
Q 35	20	31	47	65	95
Q 25	46	69	107	165	252

The results given in Table A 5.1 are based on theoretical considerations combined with empirical results. A. Meyer (61 M 11) has chosen a more direct approach by compiling a great number of strength test results for specimens with different German cement types and cured at different temperatures from which the following Table A 5.2 has been prepared.

TABLE A 5.2

Necessary prehardening time in days for obtainment of freezing resistance

Cement type	w/c	Necessary prehardening time in days at concrete temperatures		
		5° C	12° C	20° C
Q 60	0.4 0.6 0.8	1/2 3/4 1	1/4 1/2 3/4	1/4 1/2 3/4
Q 45	0.4 0.6 0.8	1 2 4	3/4 1 1/2 3	1/2 1 2
Q 35	0.4 0.6 0.8	2 5 7	1 1/2 3 1/2 5	1 2 3
Q 25	0.4 0.6 0.8	4 9 15	2 1/2 5 9	1 1/2 3 5

It will be observed that the main trend in the Tables A 5.1 and A 5.2 is similar, i.e. decrease of the required prehardening time with increase of reactivity of cement and curing temperature and with decrease of w/c-ratio.

5.2. Temperature history.

5.21. Method of computation.

Knowing the concrete mix and its thermal properties and assuming uniform temperature distribution within the concrete member the temperature of the concrete during the hardening process may be computed by the following equation (53 N 2) (54 R 1) (62 M 7) :

$$N = N_b + \frac{C}{c_b R_b}\, Q - \frac{1}{\tau} \sum_0^t (N - N_u)\,\Delta t \qquad \text{(A 5.2)}$$

where

N = Average temperature of concrete at time *t* [°C]

N_b = Initial temperature of concrete [° C]

N_u = Temperature of ambient air [°C].

t = Time from mixing [hours].

Q = Heat of hydration of cement [kcal/kg].

C = Cement content [kg/m³].

c_b = Specific heat of concrete [kcal/kg° C].

R_b = Density of concrete [kg/m³].

τ = Time constant [hours].

Time is divided in elements Δt, the size of which are corresponding to the rate of temperature changes. Q is normally given as test results at a constant temperature, N_a, as a function of the time, t_a. Therefore it is necessary to convert the time, *t*, to t_a by means of a temperature function (cf. section 1.11).

Hence, it is possible to calculate points of the temperature history in steps.

The temperature history will in principle be as shown in figure A. 5.2. The increase of temperature in the beginning is caused by the heat developed by the cement. Its effect is to a wide extent depending on the other factors and is, for instance, hardly discernible by very rapid cooling.

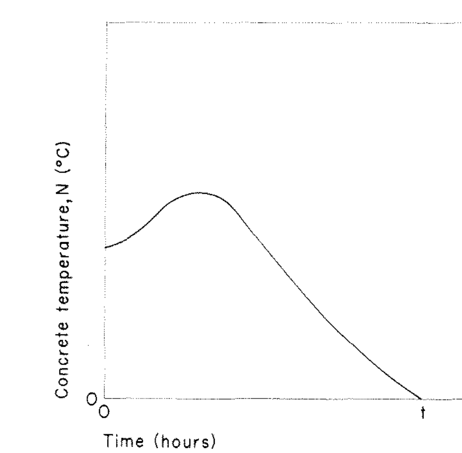


FIG. A.5.2. — **General shape of the temperature history of a new-cast concrete structure in cold weather (62 M 7).**

In order to know whether a concrete mix, for which the concrete history has been calculated as shown above, will be damaged, when the freezing point is reached, it is necessary to compare the converted hardening time, t_a, with the required prehardening time at the constant temperature, N_a.

If these calculations are carried out for different values for N_b with all other factors kept constant a

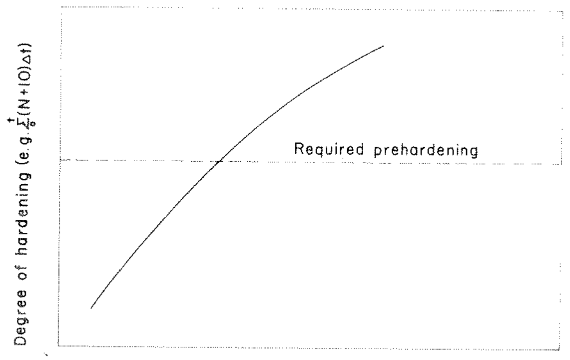


FIG. A.5.3. — **General shape of the curve representing the relation between the initial concrete temperature, N_b, and the degree of hardening at the time, when the concrete reaches the freezing point (62 M 7).**

relationship as shown in figure A 5.3 is obtained. The curve indicates the degree of hardening at freezing as a function of the initial temperature of concrete (62 M 7).

5.22. Time constant.

The time constant, τ, of a concrete member is dependent on the size, form and insulation of the member and on the specific heat and density of the concrete. It is defined by the equation:

$$\tau = \frac{V c_b R_b}{\Sigma kA} \text{ [hours]} \qquad \text{(A 5.3)}$$

where

V = Volume of concrete member [m³].

c_b = Specific heat of concrete [kcal/kg °C].

R_b = Density of concrete [kg/m³].

$k\,A$ = Coefficient of heat transmission multiplied by surface area [kcal/° C hour].

For normal cases the product $c_b R_b$ can be assumed to be 600 kcal/°C m³, and the τ-value can be calculated, when the insulation ($\Sigma\, k$) and the shape-factor $\left(\frac{V}{A}\right)$ of the concrete member are known.

Experimental k-values for uninsulated concrete for different types of insulating material and (under windy conditions) are given in the following table (53 N 1) (60 P 7) (Table A 5.3):

TABLE A 5.3

k-values of various materials.

Insulation	k [cal/°C m² hours]
No cover, wind	25.0
0.5 mm cardboard	17.0
one tarpaulin and air-space	4.0
4 cm straw-mat, wet	4.0
1 1/4″ wet form	3.0
5 cm wood-shaving-concrete slab	3.0
4 cm straw-mat. air-dried	3.0
4 cm straw-mat, air-dried on 1″ boards	2.5
4 cm straw-mat, on 0.5 mm cardboard on 1″ boards	2.0
6 cm straw-mat, air-dried + one tarpaulin	1.9
4 cm straw-mat, air-dried, sewed in sisal-kraft paper	1.6
5 cm wood-shaving concrete slab on 1″ boards	1.5
Three 4 cm straw-mats, air-dried + one tarpaulin	1.0
5 cm mineral wool	0.8

The shape-factor, $d\left(=\frac{V}{A}\right)$ and Σk for some common construction members are stated in Table A 5.4.

For these types of construction members the τ-values can be obtained from the diagram in figure A 5.4 using d and Σk as variables. For other types of constructions the τ-value may be calculated by means of the formula (A 5.3).

5.3. Determination of freezing resistance.

5.31. Danish example.

As one example of how suitable diagrams can be prepared for use in practice is referred below to a Danish booklet (58 N 4).

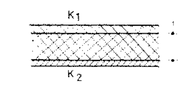
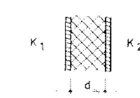
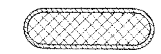
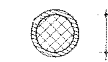
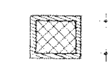
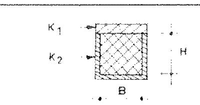
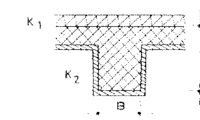
When the time constant of the concrete member has been determined and the mix composition (w/c-ratio, cement type and cement content) of the concrete is known the diagram in figure A 5.5 can be used to predict which degree of hydration the concrete has obtained at the moment it has been cooled down to 0° C.

In the diagram corresponding to the actual type, cement content and concrete temperature, a line parallel to the time axis and corresponding to the actual τ-value is brought to intersection with the curve for air temperature. The degree of hydration corresponding to the intersection point can be compared with that required to obtain resistance to freezing.

If freezing resistance is not obtained it is during planning possible to change some of the variables of concrete temperature history. If the insulation applied

TABLE A. 5. 4

" Shape factor " and $\Sigma\, k$ - values

Type of member		Shape-factor d =	Σk =
Slabs		Slab thickness	$k_1 + k_2$
Walls		Wall thickness	$k_1 + k_2$
Columns arbitrary section		F/0 = sectional area/circumference	k
circular section		1/4 . D	k
square section		1/4 . S	k
Beams rectangular		H	$k_1 + \left(1 + \frac{2H}{B}\right) k_2$
T-shaped		H	$k_1 + \left(1 + \frac{2H}{B}\right) k_2$*

* Approximate

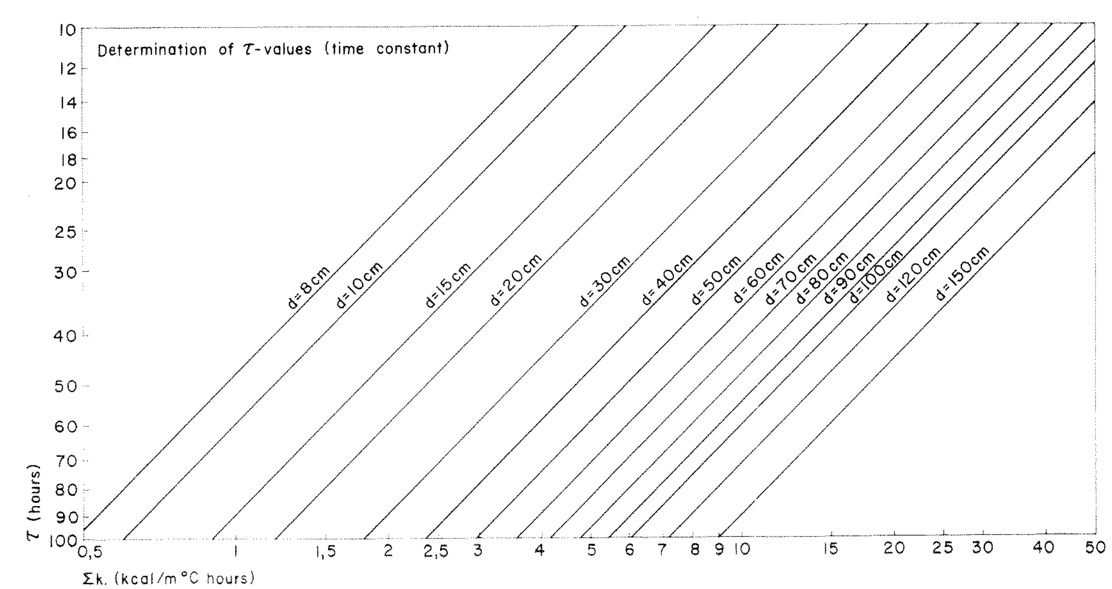


FIG. A.5.4. — **Determination of τ-value (time constant) from known values of d (shape factor) and Σk.**

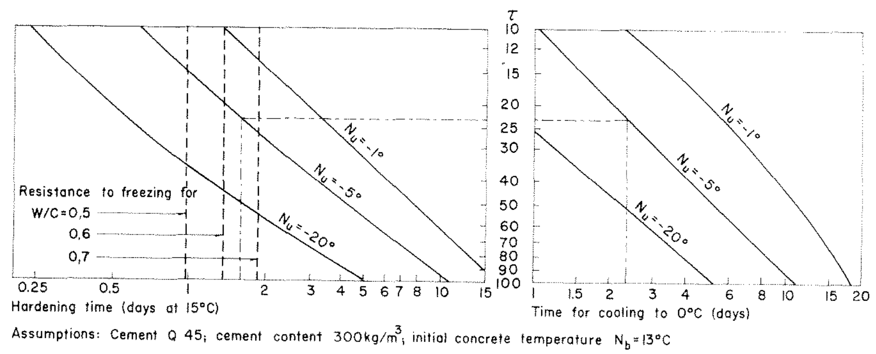


FIG. A.5.5. — **Diagrams showing the time during which a given concrete member cools down to 0°C at different temperatures of the ambient air (right), what degree of hardening has been obtained during that period and if resistance to freezing has been attained at that time (left).**

to the placed concrete is increased, the time constant must be re-calculated, but the original diagram can be used.

If a lower w/c-ratio is chosen the required prehardening time will be shorter. If a cement of higher reactivity is chosen the required prehardening time will be shorter, but another diagram shall be used because the temperature history will be different. If a higher cement content (which in turn might involve a lower w/c-ratio) is used or if installation of heating equipment suitable for producing concrete of a higher temperature is arranged for, it is also necessary to use a different diagram.

5.32. Swedish example.

Another example on how it is possible to work out diagrams is taken from a Swedish paper (62 M 7). The principles of this treatment is basically the same as in the first example, but more simple diagrams have been worked out for some standard constructions. The diagrams in figure 5.6 show allowable combinations of air temperature, concrete temperature and insulation for different concrete qualities.

When the cement content C and the concrete quality is known (e. g. K 300 means concrete with a minimum cube strength of 300 kg/cm² after 28 days normal curing) the vertically shaded area indicates the allowable combinations of air temperature and concrete temperature (at the time of covering) for a 16 cm floor slab made of concrete based on a Swedish cement (approximate Q 45), placed on a 1″ wooden form and covered with a tarpaulin. When the covering is improved by using for instance 5 cm mineral wool under the tarpaulin, the area of the allowable temperature combinations are extended to that indicated by the horizontal shading. If the actual temperature combination falls within the white area, it is necessary to heat the structure (post-heating method) if damage due to freezing shall be avoided.

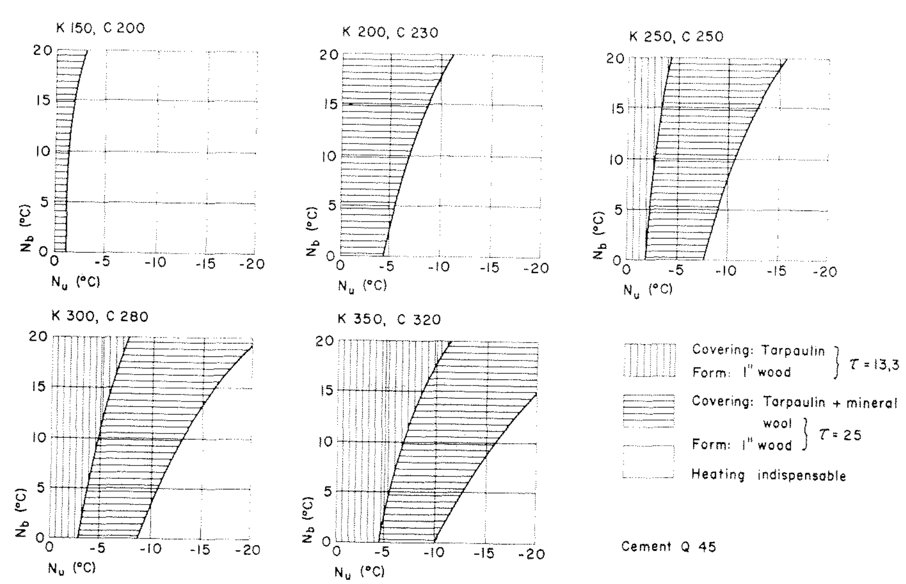


FIG. A.5.6. — **Graphs for a standard case: Floor slab, 16 cm in thickness. The concrete type is denoted by K, e.g. K 300 has minimum cube strength of 300 kg/cm² after 28 days normal hardening. C indicates the cement content, e.g. C 280 means 230 kg/cm².**

6. CRITERIA FOR REMOVAL OF FORMS

6.1. Concrete strength at stripping

Forms may be removed, when the concrete has obtained a certain percentage of its design strength, for instance the 28-days strength.

The following table contains proposed percentages in dependence of the load at the stripping time for different construction types:

When the final strength of the concrete is known, the required hardening time before stripping may be calculated from the strength development curve of the actual concrete mix at a known constant temperature by means of time-temperature functions applied to the actual temperature history.

6.11. Estimate of strength development by curves.

As proposed by A. Meyer (61-13) an estimate on the safe side for the time of form removal can be made using the relations shown in figure A. 6.1. By this procedure the type of cement, the required compressive strength as as percentage of the design strength and the average concrete temperature during the time between placing and stripping must be known. The use of the diagrams imply that the time during which the concrete temperature is below + 5° C is added to the period found in figure A. 6.1.

Example.

A cement of type Q 45 is used and the necessary strength at stripping in per cent of the 28-days strength shall be 70 %. The average concrete temperature during the time between placing and stripping is 8° C. From the diagram it appears directly that the time for removal of forms may be estimated to 14 days. If the concrete temperature has been below + 5° C in for instance 2 days, the time should be estimated to 16 days.

TABLE A 6.1

Proposed required concrete strength at removal of forms in percentage of the design strength (minimum strength at form removal = 120 kg/cm²).

Structural member	Actual locad (¹) in percentage of design load		
	100	75	50
Prestressed concrete	90	80	80
Beams (supporting form) Slabs (span > 4.5 m)	80	70	60
Slabs (span < 4.5 m)	70	60	50
Walls, columns, beams (non-supporting forms)	60	50	40

(¹) dead-load + live load during construction.

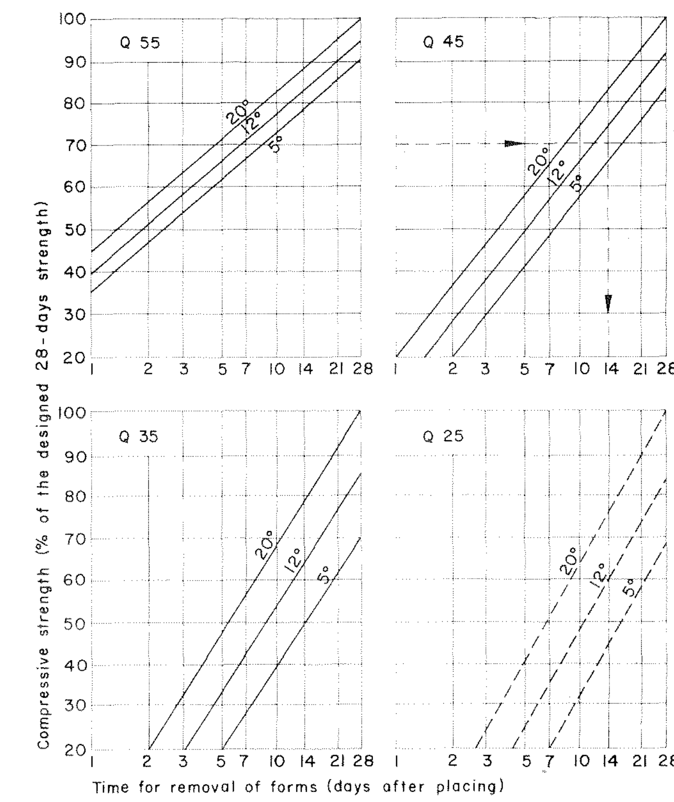


FIG. A.6.1. — **Estimation of stripping times for concrete made from different cement types.**

6.12. Time-temperature functions.

From the actual temperature history of the concrete (measured directly in the concrete structure) the corresponding hardening time at the known basis temperature is computed by using a time-temperature function.

Example.

In this example the Saul-Bergstrom function is used (see equation A. 1.2).

A beam with a span of 5 m is made of concrete with a w/c-ratio = 0.7 and cement type Q 45. The mixing water is heated and the temperature of concrete as placed and covered is measured to N_b = 13° C.

It is assumed that the actual load during the construction period is abt. 100 % of the design load. Table A 6.1 shows that non-supporting forms (side-forms) may be removed, when 60 % of the required strength has been obtained, whereas supporting forms only may be removed when 80 % has been obtained. Figure A 6.2 is only valid for a certain mix and a certain cement type. It shows that these percentages are obtained, when hardening periods of 6 days (144 hours) and 12 days (288 hours), respectively, at 20° C are reached. The table below of temperature measurements and computations of hardening time at 20° C shows that freezing resistance has been obtained the third day in the morning, that non-supporting forms may be removed the eleventh day, but that a longer hardening is still required before supporting forms may be removed.

It may be observed that the actual 235 hours of hardening correspond to only 149,8 hours at 20° C.

6.13. Strength development checked by field cured specimens.

When sampling and making these control specimens care should be taken to secure that the specimens at all periods have the same curing conditions as the concrete in the structure. This holds not only for temperature but also for moisture conditions.

TABLE A 6.2

Example of control of hardening in practice.

Time of temperature measurement	Concrete temperature [°C]	Mean concrete temperature [°C]	Δt [hours]	Hardening time at 20 °C Δt_a [hours]	Hardening time at 20 °C t_a [hours]
1st day at 12	13				
		15	4	3.3	
» 4 p.m.	17				3.3
		16	15	13.0	
2nd day at 7 a.m.	15				16.3
		14 ½	5	4.1	
» 12	14				20.4
		13 ½	4	3.1	
» 4 p.m.	13				23.5
		12	15	11.0	
3rd day at 7 a.m.	11				34.5 (1)
		11	5	3.5	
» 12	11				38.0
		11	4	2.8	
» 4 p.m.	11				40.2
		10 ½	15	10.2	
4th day at 7 a.m.	10				51.0
		9 ½	24	16.2	
5th day at 7 a.m.	9				67.2
		9	24	15.8	
6th day at 7 a.m.	9				83.0
		8 ½	24	15.4	
7th day at 7 a.m.	8				98.4
		7 ½	24	14.6	
8th day at 7 a.m.	7				113.0
		6	24	13.3	
9th day at 7 a.m.	5				125.3
		5	24	12.5	
10th day at 7 a.m.	5				137.8
		4 ½	24	12.0	
11th day at 7 a.m.	4				149.8 (2)
			235	149.8	

(1) Fig. 2 (in the Recommendations) shows that resistance to freezing for a concrete with w/c = 0.7 and made of Q 45 cement is obtained at 32 hours at 20° C, which is attained the 3rd day in the morning.

(2) Table A 6.1 and figure A 6.2 show that non-supporting forms may be removed on the 11th day in the morning.

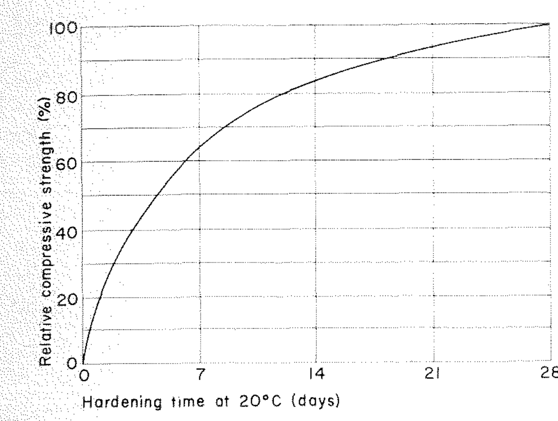


FIG. A.6.2. — **Strength development in concrete illustrated by an example (not valid in general).**

Sections of the concrete structure which will differ in rate of cooling (differing time constant, cf. App., section 5.22) should be represented individually by different sets of control specimens.

The "RILEM Method of Sampling, Making, Curing and Testing Concrete Specimens in the Field" should be followed as far as this method can be adapted to the special conditions.

The number of sets of specimens should depend on the type of structure, and on the specific importance of early form stripping.

6.2. Deformations.

Removal of formwork from a concrete structure is a critical phase of construction, because it entails, normally, the first exposure to appreciable load within the lifetime of the structure, and to direct action of climate.

Concrete construction in winter necessitates special attention in case of *any* change of the load on the structure, whether such change occurs in direct connection with the stripping of forms or *before* or *after* this operation.

6.21. Deformations due to temperature or moisture conditions.

Structural concrete members may—in addition to the deformation caused by loading —be subject to as well volume changes produced by changes in the average temperature or moisture conditions of these members as also to deformations developing as a consequence of temperature or moisture variations at different sides of the members. All these kinds of deformations, whether caused by temperature or moisture, require special attention in wintertime as many factors tend to increase them compared with ordinary concrete work.

6.22. Deformations produced by load.

The deformation produced by a load comprises of two components, viz., the immediate deformation and the deformation from sustained load, i.e. creep of concrete.

With the traditional types of concrete the deformations could be found under the following rather rough assumptions on the relation between deformation and properties of concrete :

a) The immediate deformation is inversely proportional to the modulus of elasticity of the concrete at the time of load application.

b) The rate of creep—i.e. the deformation from creep per unit of time—is inversely proportional to the modulus of elasticity within the unit of time considered and decreases after application of the load in a way which could be approximately described, as follows :

By *unchanged* modulus of elasticity during the creep the deformation from creep in the first week after the application of the load will be equal to the immediate deformation, in the second week 1/2 of the immediate deformation, in the third week 1/4 of the immediate deformation, etc.

c) The modulus of elasticity increases with increasing degree of hydration in approximately the same way as strength. This means that the same time-temperature function is used for calculation of the relative modulus of elasticity.

The assumption dealt with under (*b*) is of particular importance in connection with construction in wintertime. As an example could be mentioned that the total permanent deformation of a concrete structure 2-3 weeks after an early stripping, justified from the point of view of carrying capacity, will be considerably higher in the case of the hydration having been discontinued, wholly or partly, due to low temperature from the time of stripping, than in the case of the hydration developping in the normal way.

Consequently during winter concreting *either* normal development of hydration should be secured through heating *or* protection, *or* the time of application of load (stripping of forms) should be suitably postponed on the basis of calculation of the relevant deformations at the actual low temperatures.

In the latter case the project should clearly indicate the total deformations to be accepted—these deformations being specified for each part of the structure.

The deformations could be calculated from the formula below (63 N 1) which has been so arranged that a calculation for every full week after application of the load is possible.

$$\varepsilon_3 = \varepsilon_d \left(\frac{1}{E_{R0}} + \frac{1}{E_{R1}} + \frac{1/2}{E_{R2}} + \frac{1/4}{E_{R3}}\right). \quad \text{(A 6.1)}$$

In this formula

ε_3 is the total permanent deformation at the end of the third week after the application of the load ;

ε_d is the calculated immediate deformation, based on the modulus of elasticity for normal degree of hardening, as assumed in design;

E_{R0} is the relative modulus of elasticity at the time of application of the load;

E_{R1}, E_{R2} and E_{R3} are averages of the relative moduli of elasticity within the first, second and third week, respectively, after the application of the load. Relative moduli of elasticity are ratios of moduli of elasticity at the time in question and the design value of the modulus of elasticity (corresponding to 28 days normal hardening). As a consequence of the assumption under (c) above the relative modulus of elasticity could be derived by using time-temperature functions, once the temperature history is known.

The formula, as given above, will apply for the total deformation at the end of any week simply through cancellation or addition of terms.

In the case of application of other loads to the structure, deformations are to be calculated for each load individually, independent of the other loads, and the total deformation is found simply as the total of the individual deformations.

6.3. Risk of cracking.

Experience from practice indicates that concrete with 300 kg/m³ of cement type Q 45 does not call for special precaution when the structure is less than 1 m wide. When the structure is 2 m wide, forms of the insulation should not be removed during the first 3 weeks under severe winter conditions, and for structures up to 4 m width the period should be extended to 1 month (62 H 10). As the main point is to avoid fast cooling, forms should not be removed when it is windy and a convenient time may be

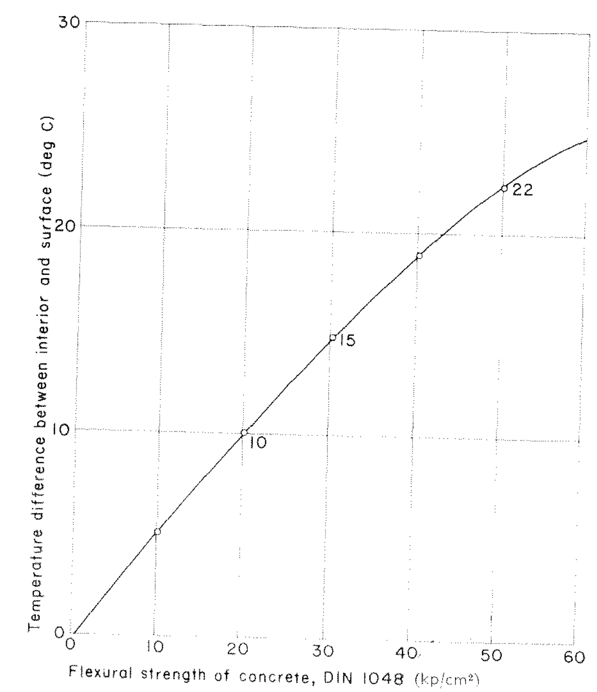


FIG. A.6.3. — **Proposal of permissible temperature differences between interior and surface in dependence of flexural strength of concrete.**

chosen from measurements of the air-temperature and the concrete temperature. In figure A 6.3 is shown which temperature differences between the interior and the surface of a concrete body may be considered permissible as function of the tensile strength in bending of the concrete.

7. CONTROL

7.1. Concrete constituents and fresh concrete.

7.11. Determination of heat of hydration.

The heat developed by the process of setting and hardening of cement is termed heat of hydration. The amount of heat, and the rate of heat development depend not only on composition and fineness of the cement, but also on temperature. The heat of hydration can be determined by direct or indirect methods. By the direct methods the variation of temperature of cement paste, mortar, or concrete is observed during the hydration. The test methods deviate in the way of measuring the heat of hydration. One appropriate determination is attained by application of an adiabatic calorimeter. This device prevents loss of heat from the specimen investigated, the calorimeter keeping the temperature of the immediate surroundings equal to the temperature of the test specimen. Basalla (62 B 4) has described a simple adiabatic calorimeter for investigation of specimens of mortar and concrete.

In the indirect method for determination of heat of hydration of cement, utilizing observation of heat of solution, Hesse's thesis is applied, stating that the total of heat of reaction, developed by a series of chemical reactions, is independent of number and kind of intermediate reactions. Hence, the heat of hydration could be found as the difference of heat of solution produced by non-hydrated cement and by hydrated cement, the difference being just the amount of heat released as heat of hydration.

Standard specifications for determination of heat of hydration by the Heat of Solution Method have already been adopted in a number of countries.

The methods for determination of the heat of hydration, described in (58-36) and (55-30) are applicable to Portland cement only.

Wittekind, in (61-14) has described a method applicable to all cement types, also for cements containing materials of low solubility, such as blast-furnace slag and pozzolans. There is only a slight deviation from the British and American specifications, mentioned above.

For classification of cement in regard to the activity at low temperature the criterion selected is the heat of hydration after 3 days. For testing, cement paste with a w/c-ratio of 0.4 is mixed at normal temperature (+ 20° C) and filled into small tubes which are sealed. The specimens are placed in water of 5° C (± 1° C) 15 minutes after the preparation and stored under these conditions until the time of testing. Determination of the heat of solution is done at normal temperature (+ 20° C).

For tests performed at an early age the standard deviation of the results is rather considerable. By 10 repetitive tests on a cement, whereby all conditions of testing were carefully considered, this method resulted in the variations shown in the following table:

TABLE A 7.1

Variations by the determination of heat of hydration in 3 days at different temperatures.

Storage temperature	Cœfficient of variation %	Average cal/g
+ 20 °C	2.6	58
+ 5 °C	4.0	42

Considering the errors to be anticipated from the testing procedure and from variations in the cement production, it would not be appropriate to specify a too narrow classification of cement on the basis of heat of hydration developed within 3 days at 5° C.

7.12. Air content of concrete.

Determination of contents of air in concrete can be effected by two methods:

a) *Pressure method.*

The fresh concrete is placed in a container and compacted to the same degree as on the site. Thereafter the container is airtight sealed and the remaining space filled with water. Pressure is applied by opening a valve connecting the container to an adjoining chamber, containing air at a certain definite pressure. The volume of the voids in the concrete can be derived from the reduction of pressure, caused by the reduction of the volume of the entrained air in the concrete. The air content can be read on a calibrated manometer (61-15).

b) *Unit weight test.*

Fresh concrete, without any addition of air entraining agents, is filled into a container of a definite content and compacted, whereafter the density is determined. At the same time, and in the same way, concrete with an air entraining agent is tested and the density of the fresh concrete determined.

From the difference of the two densities the additional voids volume produced by the air-entraining agent can be derived.

The amount of entrained air will depend, among other things, on the temperature, for which reason the air content should be determined at a temperature corresponding as close as possible to working temperature.

The two methods described above do not provide exact information in regard to the ratio of bubbles accessible to water and those not accessible to water. This ratio is important for frost durability and, as a consequence, the determination of the air contents will not alone afford an adequate basis for predicting the frost durability.

7.2. Measurements of temperatures.

7.21. Air temperature.

At low temperatures several constructions require accurate observation of the temperature on the site. Measurements for this purpose will be more or less extensive dependent on the sort of work, but the recording of a temperature which is representative for the site should take place every day at preset hours. The placing of a max-and-min thermometer for this purpose is very important. It is imperative that such a thermometer is nothung on a wall of a shed etc. This would result in a wrong reading because of the radiation of the wall and the entirely changed influence of the wind. The Royal Netherlands Meteorological Institute developed a thermometer stand which can be made by simple means and suits its purpose perfectly. It is recommended to use this observation stand on the site (see Fig. A 7.1).

7.22. Concrete temperature.

Readings of the temperature of concrete may be carried out in practice for instance by using a simple device as shown in figure A. 7.2; by different lengths of the pieces of reinforcement bars readings can be made in different depths. Temperature observations in connection with ascertainment of freezing resistance are to be made at a depth of 1/4 of the total depth of the structure and in the most exposed part. Observations with reference to determination of stripping time for members in bending are to be made at the place where maximum compressive stress will occur at the time at stripping. Temperature readings are recorded as shown in Table A 6. 2 above.

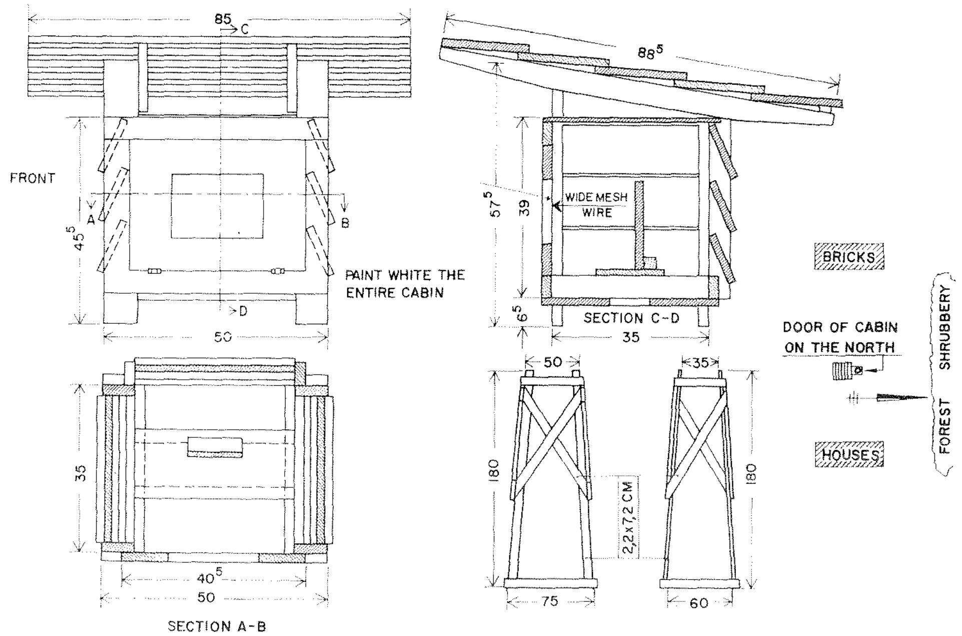


FIG. A.7.1. — **Thermometer cabin. The cabin should be kept as far as possible from houses, bricks, sand, gravel, sheds, forest etc., and if practicable be placed on a lawn or on earth as a brick or stone floor may cause excessive radiation. The location should further be so that wind may give a good ventilation.**

7.3. Control specimens.

Results of design tests at normal temperature (about 20° C) do not constitute adequate criterion in regard to winter concreting. By the recommended additional design tests at a temperature of + 5° C the specimens are made at normal temperature (20° C). The specimens are placed in a room where the temperature is 5° C (± 1°) as soon as possible after the preparation (max. 15 min.) and remain there until shortly before the testing.

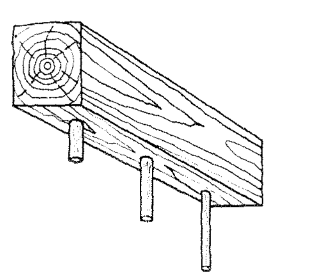


FIG. A.7.2. — **Temperature readings may be made in different depths of a concrete member by placing this simple device in the fresh concrete. The pieces of reinforcement should be oiled or greased before inserting the device.**

Results of a number of design tests on concrete specimens of ordinary proportioning, at + 20° C and + 5° C afforded the averages given in the table below, the compressive strength being given in per cent, whereby the compressive strength after 28 days, at 20° C, is taken as 100 %.

TABLE A 7.2

Results of design tests carried out at + 5° C (strength in percent of 28 days strength at 20° C)

Cement	Compressive strength in %					
	at + 5° C			at + 20° C		
	3 days	7 days	28 days	3 days	7 days	28 days
Q 55	50	70	95	65	85	100
Q 45	25	50	90	45	70	100
Q 35 (1)	10	30	75	35	60	100

(1) The figures are from a cement, which is not of the Portland type, and the Saul-Nurse (Bergström) relationship is not valid.

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