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Cold Weather Concreting

Reported by ACI Committee 306

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The general requirements for producing satisfactory concrete during cold weather are discussed, and methods for satisfying these requirements are described. One of the objectives of cold weather concreting practice is to provide protection of the concrete at early ages to prevent damage from freezing. For many structural concretes, protection considerably in excess of that required to prevent damage by early freezing is needed to assure development of adequate strength.

The following items are discussed in the report: recommended temperature of concrete, temperature records, temperature of materials, preparations prior to placement, duration of protection period, methods for determining in-place strength, form removal, protective insulating covers, heated enclosures, curing methods, and accelerating admixtures. References are included that provide supplementary data on the effects of curing temperature on concrete strength.

Keywords: accelerating admixtures; age; aggregates; calcium chloride; **cold weather;** compressive strength; **concrete construction;** concretes; curing; durability; form removal; formwork (construction); freeze-thaw durability; heating; **in-place testing; insulation;** materials handling; **protection;** subgrade preparation; **temperature.**

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This report supercedes ACI 306R-78 (Revised 1983).

CHAPTER 1 - INTRODUCTION

1.1 - Definition of cold weather

This report describes construction procedures which, if properly followed, can result in concrete placed in cold weather of sufficient strength and durability to satisfy intended service requirements. Concrete placed during cold weather will develop these qualities only if it is properly produced, placed, and protected. The necessary degree of protection increases as the ambient temperature decreases.

Cold weather is defined as a period when, for more than 3 consecutive days, the following conditions exist: 1) the average daily air temperature is less than 40 F (5 C) and 2) the air temperature is not greater than 50 F (10 C) for more than one-half of any 24-hr period.* The average daily air temperature is the average of the highest and the lowest temperatures occurring during the period from midnight to midnight. Cold weather, as defined in this report, usually starts during fall and usually continues until spring.

1.2 - Standard specification

If requirements for cold weather concreting are needed in specification form, ACI 306.1 should be referenced; if necessary, appropriate modifications should be added to the contract documents after consulting the specification checklist.

1.3 - Objectives

The objectives of cold weather concreting practices are to:

1.3.1 - prevent damage to concrete due to freezing at early ages. When no external water is available, the degree of saturation of newly placed concrete decreases as the concrete gains maturity and the mixing water combines with cement during hydration. Under such conditions, the degree of saturation falls below the critical level (the degree of water saturation where a single cycle of freezing would cause damage) at approximately the time that the concrete attains a compressive strength of 500 psi (3.5 MPa) (Powers 1962). At 50 F (10 C), most well-proportioned concrete mixtures reach this strength during the second day.

1.3.2 - assure that the concrete develops the required strength for safe removal of forms, for safe removal of shores and reshores, and for safe loading of the structure during and after construction.

1.3.3 - maintain curing conditions that foster normal strength development without using excessive heat and without causing critical saturation of the concrete at the end of the protection period.

1.3.4 - limit rapid temperature changes, particularly before the concrete has developed sufficient strength to withstand induced thermal stresses. Rapid cooling of concrete surfaces or large temperature differences between exterior and interior members of the structure can cause cracking, which can be detrimental to strength and durability. At the end of the required period, insulation or other means of protection should be removed gradually so that the surface temperature decreases gradually during the subsequent 24-hr period (see Section 5.5).

1.3.5 - provide protection consistent with the intended serviceability of the structure. Concrete structures are intended for a useful life of many years. The attainment of satisfactory strength for 28-day, standard-cured cylinders is irrelevant if the structure has corners damaged by freezing; dehydrated areas; and cracking from overheating because of inadequate protection, improper curing, or careless workmanship. Similarly, early concrete strength achieved by indiscriminate use of excessive calcium chloride is of no avail if the concrete becomes excessively cracked in later years because of the likelihood of disruptive internal expansion due to alkali-aggregate reaction or of possible corrosion of reinforcement (see Section 9.2). Short-term construction economy should not be obtained at the expense of long-term durability.

1.4 - Principles

This report presents recommendations to achieve the objectives listed in Section 1.3 (Schnarr and Young 1934a and 1934b). The practices and procedures described in this report stem from the following principles concerning cold weather concreting:

1.4.1 - Concrete that is protected from freezing until it has attained a compressive strength of at least 500 psi (3.5 MPa) will not be damaged by exposure to a single freezing cycle (Powers 1962).

1.4.2 - Concrete that is protected as in Section 1.4.1 will mature to its potential strength despite subsequent exposure to cold weather (Malhotra and Berwanger 1973). No further protection is necessary unless a certain strength must be attained in less time.

*The values in SI units are direct conversions of the in.-lb values. They do nor necessarily represent common metric ranges or sizes. For practical application. The user should adjust them to conform with local practice.

Table 3.1 - Recommended concrete temperatures

Line	Air temperature	Section size, minimum dimension, in. (mm) < 12 in. (300 mm)	12-36 in. (300-900 mm)	36-72 in. (900-1800 mm)	> 72 in. (1800 mm)
	Minimum concrete temperature as placed and maintained				
1	-	55 F (13 C)	50 F (10 C)	45 F (7 C)	40 F (5 C)
	Minimum concrete temperature as mixed for indicated air temperature*				
2	Above 30 F (- 1 C)	60 F (16 C)	55 F (13 C)	50 F (10 C)	45 F (7 C)
3	0 to 30 F (-18 to -1 C)	65 F (18 C)	60 F (16 C)	55 F (13 C)	50 F (10 C)
4	Below 0 F (- 18 C)	70 F (21 C)	65 F (18 C)	60 F (16 C)	55 F (13 C)
	Maximum allowable gradual temperature drop in first 24 hr after end of protection				
5	-	50 F (28 C)	40 F (22 C)	30 F (17 C)	20 F (11 C)

*For colder weather a greater margin in temperature is provided between concrete as mixed and required minimum temperature of fresh concrete in place.

1.4.3 - Where a specified concrete strength must be attained in a few days or weeks, protection at temperatures above 50 F (10 C) is required. See Chapters 5 and 6.

1.4.4 - Except within heated protective enclosures, little or no external supply of moisture is required for curing during cold weather. See Chapter 8.

1.4.5 - Under certain conditions, calcium chloride should not be used to accelerate setting and hardening because of the increased chances of corrosion of metals embedded in concrete or other adverse effects. See Chapter 9.

Times and temperatures given in this report are not exact values for all situations and they should not be used as such. The user should keep in mind the primary intent of these recommendations and should use discretion in deciding what is adequate for each particular circumstance.

1.5 - Economy

Experience has shown that the overall costs of adequate protection for cold weather concreting are not excessive, considering what is required and the resulting benefits. The owner must decide whether the extra costs involved in cold weather concreting operations are a profitable investment or if it is more cost effective to wait for mild weather. Neglect of protection against early freezing can cause immediate destruction or permanently weakened concrete. Therefore, if cold weather concreting is performed, adequate protection from low temperatures and proper curing are essential.

CHAPTER 2 - GENERAL REQUIREMENTS

2.1 - Planning

It is recommended that the concrete contractor, concrete supplier, and owner (or architect/engineer) meet in a preconstruction conference to define in clear terms how cold weather concreting methods will be used. This report provides a basis for the contractor to select specific methods to satisfy the minimum requirements during cold weather concreting,

Plans to protect fresh concrete from freezing and to maintain temperatures above the recommended minimum values should be made well before freezing temperatures are expected to occur. Necessary equipment and materials should be at the work site before cold weather is likely to occur, not after concrete has been placed and its temperature begins to approach the freezing point.

2.2 - Protection during fall and spring

During periods not defined as cold weather, such as in fall or spring, but when heavy frost or freezing is forecast at the job site,* all concrete surfaces should be protected from freezing for at least the first 24 hr after placement. Concrete protected in this manner will be safe from damage by freezing at an early age. If the concrete is air entrained and properly cured, the ultimate strength and durability of the concrete will be unimpaired. Protection from freezing during the first 24 hr does not assure a satisfactory rate of strength development, particularly when followed by considerably colder weather. Protection and curing should continue long enough - and at a temperature sufficiently above freezing - to produce the strength required for form removal or structural safety (see Chapters 5 and 6).

2.3 - Concrete temperature

During cold weather, the concrete temperature at the time of placement should not be lower than the values given in Chapter 3. In action, to prevent freezing at early ages, the concrete temperature should be maintained at not less than the recommended placement temperature for the length of time given in Chapter 5. This length of time depends on the type and amount of cement, whether an accelerating admixture is used, and the service category.

The recommended minimum placement temperatures given in Table 3.1 in Chapter 3 apply to normal weight

*Charts showing mean dates of freezing weather in the United States may be obtained from the National Climatic Center, Federal Building, Ashville, NC 28801 concrete. Experience indicates that freshly mixed lightweight concrete loses heat more slowly than freshly mixed normal-weight concrete. Lighter weight insulating concretes lose heat even more slowly. However, when exposed to freezing temperatures, such concretes are more susceptible to damage from surface freezing.

The temperature of concrete at the time of placement should always be near the minimum temperatures given in Chapter 3, Table 3.1. Placement temperatures should not be higher than these minimum values by more than 20 F (11 C). One should take advantage of the opportunity provided by cold weather to place low-temperature concrete. Concrete that is placed at low temperatures [40 to 55 F (5 to 13 C)] is protected against freezing and receives long-time curing, thus developing a higher ultimate strength (Klieger 1958) and greater durability. It is, therefore, less subject to thermal cracking than similar concrete placed at higher temperatures. Placement at higher temperatures may expedite finishing in cold weather, but it will impair long-term concrete properties.

2.4 - Temperature records

The actual temperature at the concrete surface determines the effectiveness of protection, regardless of air temperature. Therefore, it is desirable to monitor and record the concrete temperature. Temperature recording and monitoring must consider the following:

2.4.1 - The corners and edges of concrete are more vulnerable to freezing and usually are more difficult to maintain at the required temperature, therefore, their temperature should be monitored to evaluate and verify the effectiveness of the protection provided.

2.4.2 - Inspection personnel should keep a record of the date, time, outside air temperature, temperature of concrete as placed, and weather conditions (calm, windy, clear, cloudy, etc.). Temperatures of concrete and the outdoor air should be recorded at regular time intervals but not less than twice per 24-hr period. The record should include temperatures at several points within the enclosure and on the concrete surface, corners, and edges. There should be a sufficient number of temperature measurement locations to show the range of concrete temperatures. Temperature measuring devices embedded in the concrete surface are ideal, but satisfactory accuracy and greater flexibility of observation can be obtained by placing thermometers against the concrete under temporary covers of heavy insulating material until constant temperatures are indicated.

2.4.3 - Maximum and minimum temperature readings in each 24-hr period should be recorded. Data recorded should clearly show the temperature history of each section of concrete cast. A copy of the temperature readings should be included in the permanent job records. It is preferable to measure the temperature of concrete at more than one location in the section cast and use the lowest reading to represent the temperature of that section. Internal temperature of concrete should be monitored to insure that excessive heating does not occur (see Section 7.4). For this, expendable thermistors or thermocouples cast in the concrete may be used.

2.5 - Heated enclosures

Heated enclosures must be strong enough to be windproof and weatherproof. Otherwise, proper temperatures at corners, edges, and in thin sections may not be maintained despite high energy consumption. Combustion heaters should be vented and they should not be permitted to heat or to dry the concrete locally. Fresh concrete surfaces exposed to carbon dioxide, resulting from the use of salamanders or other combustion heaters that exhaust flue gases into an enclosed area, may be damaged by carbonation of the concrete. Carbonation may result in soft surfaces or surface crazing depending on the concentration of carbon dioxide, the concrete temperature, and the relative humidity (see Section 7.4). Carbon monoxide, which can result from partial combustion, and high levels of carbon dioxide are potential hazards to workers.

In addition, strict fire prevention measures should be enforced. Fire can destroy the protective enclosures as well as damage the concrete. Concrete can be damaged by fire at any age. However, at a very early age additional damage can occur by subsequent freezing of the concrete before new protective enclosures are provided.

2.6 - Exposure to freezing and thawing

If, during construction, it is likely that the concrete will be exposed to cycles of freezing and thawing while it is in a saturated condition, it should be properly air entrained even though it will not be exposed to freezing and thawing in service. The water-cement ratio should not exceed the limits recommended in ACI 201.2R, and the concrete should not be allowed to freeze and thaw in a saturated condition before developing a compressive strength of 3500 psi (24 MPa). Therefore, new sidewalks and other flatwork exposed to melting snow during daytime and freezing during nighttime should be air entrained and protected from freezing until a strength of at least 3500 psi (24 MPa) has been attained.

2.7 - Concrete slump

Concrete with a slump lower than normal [less than 4 in. (100 mm)] is particularly desirable in cold weather for flatwork; bleeding of water is minimized and setting occurs earlier. During cold weather, bleed water may remain on the surface for such a long period that it interferes with proper finishing. If the bleed water is mixed into the concrete during trowelling, the resulting surface will have a lower strength and may be prone to dusting and subsequent freeze-thaw damage if exposed. Thus, during cold weather, the concrete mixture should be proportioned so that bleeding is minimized as much as practicable. If bleedwater is present on flatwork, it should be skimmed off prior to trowelling by using a rope or hose.

CHAPTER 3 - TEMPERATURE OF CONCRETE AS MIXED AND PLACED AND HEATING OF MATERIALS

3.1 - Placement temperature

During cold weather, the concrete mixing temperature should be controlled as described in Section 3.2 so that when the concrete is placed its temperature is not below the values shown in Line 1 of Table 3.1. The placement temperature of concrete should be determined according to ASTM C 1064. The more massive the concrete section, the less rapidly it loses heat; therefore, lower minimum placement temperatures are recommended as concrete sections become larger. For massive structures, it is especially beneficial to have low placement temperatures (see ACI 207.1R). Concrete temperatures that are much higher than the values in Line 1 do not result in a proportionally longer protection against freezing because the rate of heat loss is greater for larger temperature differentials.

In addition, higher temperatures require more mixing water, increase the rate of slump loss, may cause quick setting, and increase thermal contraction. Rapid moisture loss from exposed surfaces of flatwork may cause plastic shrinkage cracks. Rapid moisture loss can occur from surfaces exposed to cold weather because the warm concrete heats the surrounding cold air and reduces its relative humidity (see AC1 302.1R). Therefore, the temperature of concrete as placed should be kept as close to the recommended minimum value as is practicable. Placement temperatures should not be higher than these minimum values by more than 20 F (11 C).

3.2 - Mixing temperature

The recommended minimum temperature of concrete at the time of mixing is shown in Lines 2, 3, and 4 of Table 3.1. As the ambient air temperature decreases, the concrete temperature during mixing should be increased to offset the heat lost in the interval between mixing and placing. The mixing temperature should not be more than 15 F (8 C) above the recommended values in Lines 2, 3, and 4. While it is difficult to heat aggregates uniformly to a predetermined temperature, the mixing water temperature can be adjusted easily by blending hot and cold water to obtain a concrete temperature within 10 F (5 C) of the required temperature.

3.3 - Heating mixing water

Mixing water should be available at a consistent, regulated temperature, and in sufficient quantity to avoid appreciable fluctuations in temperature of the concrete from batch to batch. Since the temperature of concrete affects the rate of slump loss and may affect the performance of admixtures, temperature fluctuations can result in variable behavior of individual batches.

Premature contact of very hot water and concentrated quantities of cement has been reported to cause flash set and cement balls in truck mixers. When water above 140 F (80 C) is used, it may be necessary to adjust the order in which ingredients are blended. It may be helpful to add the hot water and coarse aggregate ahead of the cement and to stop or slow down the addition of water while the cement and aggregate are loaded.

If the cement is batched separately from the aggregate, mixing may be more difficult. To facilitate mixing, about three-fourths of the added hot water should be placed in the drum either ahead of the aggregates or with them. To prevent packing at the end of the mixer, coarse aggregate should be added first. The cement should be added after the aggregates. As the final ingredient, the remaining one-fourth of the mixing water should be placed into the drum at a moderate rate.

Water with a temperature as high as the boiling point may be used provided that resulting concrete temperatures are within the limits discussed in Section 3.2 and no flash setting occurs. If loss of effectiveness of the air-entraining admixture is noted due to an initial contact with hot water, the admixture must be added to the batch after the water temperature has been reduced by contact with the cooler solid materials.

3.4 - Heating aggregates

When aggregates are free of ice and frozen lumps, the desired temperature of the concrete during mixing can usually be obtained by heating only the mixing water, but when air temperatures are consistently below 25 F (- 4 C), it is usually necessary to also heat the aggregates. Heating aggregates to temperatures higher than 60 F (15 C) is rarely necessary if the mixing water is heated to 140 F (60 C). If the coarse aggregate is dry and free of frost, ice, and frozen lumps, adequate temperatures of freshly mixed concrete can be obtained by increasing the temperature of only the sand, which seldom has to be above about 105 F (40 C), if mixing water is heated to 140 F (60 C). Seasonal variations must be considered, as average aggregate temperatures can be substantially higher than air temperature during autumn, while the reverse may occur during spring.

3.5 - Steam heating of aggregates

Circulating steam in pipes is recommended for heating aggregates. For small jobs, aggregates may be thawed by heating them carefully over culvert pipes in which fires are maintained. When aggregates are thawed or heated by circulating steam in pipes, exposed surfaces of aggregate should be covered with tarpaulins as much as is practicable to maintain a uniform distribution of heat and to prevent formation of ice crusts. Steam jets liberated in aggregate may cause troublesome moisture variation, but this method is the most thermally efficient procedure to heat aggregate. If steam is confined in a pipe-heating system, difficulties from variable moisture in aggregates are avoided, but the likelihood of localized hot, dry spots is increased. Wear and corrosion of steam pipes in aggregates will eventually cause leaks, which may lead to the same moisture variation problem caused by steam jets. Periodic inspection of the pipes and replacement as necessary are recommended.

When conditions require thawing of substantial quantities of extremely low temperature aggregates, steam jets may be the only practicable means of providing the necessary heat. In such a case, thawing must be done as far in advance of batching as is possible to achieve substantial equilibrium in both moisture content and temperature. After thawing is completed, the steam supply can be reduced to the minimum that will prevent further freezing, thereby reducing to some extent the problems arising from variable moisture content. Nevertheless, under such conditions, mixing water control must be largely on an individual batch adjustment basis. Dry hot air instead of steam has been used to keep aggregates ice free.

3.6 - Overheating of aggregates

Aggregates should be heated sufficiently to eliminate ice, snow, and frozen lumps of aggregate. Often 3-in. (76-mm) frozen lumps will survive mixing and remain in the concrete after placing. Overheating should be avoided so that spot temperatures do not exceed 212 F (100 C) and the average temperature does not exceed 150 F (65 C) when the aggregates are added to the batch. Either of these temperatures is considerably higher than is necessary for obtaining desirable temperatures of freshly mixed concrete. Materials should be heated uniformly since considerable variation in their temperature will significantly vary the water requirement, air entrainment, rate of setting, and slump of the concrete.

Extra care is required when batching the first few loads of concrete following a prolonged period of steaming the aggregates in storage bins. Many concrete producers recycle the first few tons of very hot aggregates. This material is normally discharged and recycled by placing it on top of the aggregates in the storage bins.

3.7 - Calculation of mixture temperature

If the weights and temperatures of all constituents and the moisture content of the aggregates are known, the final temperature of the concrete mixture may be estimated from the formula

$$T = \frac{[0.22(T_sW_s + T_aW_a + T_cW_c) + T_wW_w + T_sW_{ws} + T_aW_w]}{[0.22\ (W_s + W_a + W_c) + W_w + W_{wa} + W_{ws}]} \qquad (3\text{-}1)$$

where

T = final temperature of concrete mixture (deg F or C)
T_c = temperature of cement (deg F or C)
T_s = temperature of fine aggregate (deg F or C)
T_a = temperature of coarse aggregate (deg F or C)
T_w = temperature of added mixing water (deg F or C)
W_c = weight of cement (lb or kg)
W_s = saturated surface-dry weight of fine aggregate (lb or kg)
W_a = saturated surface-dry weight of coarse aggregate (lb or kg)
W_w = weight of mixing water (lb or kg)
W_{ws} = weight of free water on fine aggregate (lb or kg)
W_{wa} = weight of free water on coarse aggregate (lb or kg)

Eq. (3-l) is derived by considering the equilibrium heat balance of the materials before and after mixing and by assuming that the specific heats of the cement and aggregates are equal to 0.22 Btu/(lb F) [0.22 kcal/(kg C)].

If the temperature of one or both of the aggregates is below 32 F (0 C), the free water will be frozen, and Eq. (3-1) must be modified to take into account the heat required to raise the temperature of the ice to 32 F (0 C), to change the ice into water, and to raise the temperature of the free water to the final mixture temperature. The specific heat of ice is 0.5 Btu/(lb F) [0.5 kcal/(kg C)] and the heat of fusion of ice is 144 Btu/lb (80 kcal/kg). Thus Eq. (3-1) is modified by substituting the following expressions for T_sW_{ws} or T_aW_{wa}, or both, depending on whether the fine aggregate or coarse aggregate, or both, are below 32 F (0 C).

For in.-lb units

$$\text{for } T_s W_{ws} \text{ substitute } W_{ws}(0.50T_s - 128) \qquad (3\text{-}2)$$

$$\text{for } T_a W_{wa} \text{ substitute } W_{wa}(0.50T_a - 128) \qquad (3\text{-}3)$$

For SI units

$$\text{for } T_s W_{ws} \text{ substitute } W_{ws}(0.50T_s - 80) \qquad (3\text{-}4)$$

$$\text{for } T_a W_{wa} \text{ substitute } W_{wa}(0.50T_s - 80) \qquad (3\text{-}5)$$

In these equations, the numbers 128 and 80 are obtained from the heat of fusion needed to melt the ice, the specific heat of the ice, and the melting temperature of ice.

3.8 - Temperature loss during delivery

The Swedish Cement and Concrete Research Institute (Petersons 1966) performed tests to determine the expected decrease in concrete temperature during delivery in cold weather. Their studies included revolving drum mixers, covered-dump bodies, and open-dump bodies. Approximate temperature drop for a delivery time of 1 hr can be computed using Eq. (3-6)-(3-8). For revolving drum mixers

$$T = 0.25\ (t_r - t_a) \qquad (3\text{-}6)$$

For covered-dump body

$$T = 0.10\ (t_r - t_a) \qquad (3\text{-}7)$$

For open-dump body

$$T = 0.20\ (t_r - t_a) \qquad (3\text{-}8)$$

where

T = temperature drop to be expected during a 1-hr delivery time, deg F or C. (This value must be added to t_r to determine the required temperature of concrete at the plant.)

t_r = concrete temperature required at the job, deg F or C.

t_a = ambient air temperature, deg F or C.

The values from these equations are proportionally adjusted for delivery times greater than or less than one hour.

3.9.1 - The following examples illustrate the application of these approximate equations:

1. Concrete is to be continuously agitated in a revolving drum mixer during a 1-hr delivery period. The air temperature is 20 F and the concrete at delivery must be at least 50 F. From Eq. (3-6)

$$T = 0.25\ (50 - 20) = 7.5\ \text{F}$$

Therefore, allowance must be made for a 7.5-deg temperature drop, and the concrete at the plant must have a temperature of at least (50 + 7.5 F), or about 58 F.

2. For the same temperature conditions given in Example 1, the concrete will be delivered within 1 hr and the drum will not be revolved except for initial mixing and again briefly at the time of discharge. Assuming that Eq. (3-7) represents this situation best, the temperature drop is

$$T = 0.10\ (50 - 20) = 3\ \text{F}$$

Thus provisions must be made for a concrete temperature of (50 + 3 F), or 53 F, at the plant.

The advantage of covered dump bodies over revolving drums suggests that temperature losses can be minimized by not revolving the drum more than is absolutely necessary during delivery.

CHAPTER 4 - PREPARATION BEFORE CONCRETING

4.1 - Temperature of surfaces in contact with fresh concrete

Preparation for concreting, other than mentioned in Section 2.1, consists primarily of insuring that all surfaces that will be in contact with newly placed concrete are at temperatures that cannot cause early freezing or seriously prolong setting of the concrete. Ordinarily, the temperatures of these contact surfaces, including subgrade materials, need not be higher than a few degrees above freezing, say 35 F (2 C), and preferably not more than 10 F (5 C) higher than the minimum placement temperatures given in Line 1 of Table 3.1.

4.2 - Metallic embedments

The placement of concrete around massive metallic embedments that are at temperatures below the freezing point of the water in concrete may result in local freezing of the concrete at the interface. If the interface remains frozen beyond the time of final vibration, there will be a permanent decrease in the interfacial bond strength. Whether freezing will occur, the volume of frozen water, and the duration of the frozen period depend primarily upon the placement temperature of concrete, the relative volumes of the concrete and the embedment, and the temperature of the embedment. An analytical study, using the finite element method to solve the heat flow problem, has been reported (Suprenant and Basham 1985). Two cases were investigated: a No. 9 bar in a slab and a square steel tube filled with concrete. Based on that limited study, it was suggested that steel embedments having a cross-sectional area greater than 1 in.^{2} (650 mm^2) should have a temperature of at least 10 F (- 12 C) immediately before being surrounded by fresh concrete at a temperature of at least 55 F (13 C). Additional study is required before definitive recommendations can be formulated. The engineer/architect should determine whether the structure contains large embedments that pose potential problems. If heating is required, the heating process should not alter the mechanical or metallurgical properties of the metal. The contractor should submit the plan for heating to the engineer for approval.

4.3 - Removal of snow and ice

All snow, ice, and frost must be removed so that it does not occupy space intended to be filled with concrete. Hot-air jets can be used to remove frost, snow, and ice from forms, reinforcement, and other embedments. Unless the work area is housed, this work should be done immediately prior to concrete placement to prevent refreezing.

4.4 - Condition of subgrade

Concrete should not be placed on frozen subgrade material. The subgrade sometimes can be thawed acceptably by covering it with insulating material for a few days before the concrete placement, but in most cases external heat must be applied. Experimenting at the site will show what combinations of insulation and time causes subsurface heat to thaw the subgrade material. If necessary, the thawed material should be recompacted.

CHAPTER 5 - PROTECTION AGAINST FREEZING AND PROTECTION FOR CONCRETE NOT REQUIRING CONSTRUCTION SUPPORTS

5.1 - Protection to prevent early-age freezing

To prevent early-age freezing, protection must be provided immediately after concrete placement. Arrangements for covering, insulating, housing, or heating newly placed concrete should be made before placement. The protection that is provided should be adequate to achieve, in all sections of the concrete cast, the temperature and moisture conditions recommended in this report. In cold weather, the temperature of the newly placed concrete should be kept close to the val-

Table 5.1 - Length of protection period required to prevent damage from early-age freezing of air-entrained concrete

Line	Exposure	Protection period at temperature indicated in Line 1 of Table 3.1, days*	
		Type I or II cement	Type III cement, or accelerating admixture, or 100 lb/yd³ (60 kg/m³) of additional cement
1	Not exposed	2	1
2	Exposed	3	2

*A day is a 24-hr period.

ues shown in Line 1 of Table 3.1 for the lengths of time indicated in Table 5.1 for protection against early-age freezing. The length of the protection period may be reduced by: (1) using Type III cement; (2) using an accelerating admixture; or (3) using 100 lb/yd³ (60 kg/m³) of cement in excess of the design cement content. Line 1 of Table 5.1 refers to concrete that will be exposed to little or no freezing and thawing in service or during construction, such as in foundations and substructures. Line 2 refers to concrete that will be exposed to the weather in service or during construction.

It has been shown that when there is no external source of curing water, concrete that has attained a strength of 500 psi will not be damaged by one cycle of freezing and thawing (Powers 1962; Hoff and Buck 1983). The protection periods given in Table 5.1 may be reduced if it is verified that the concrete, including corners and edges, has attained in in-place compressive strength of at least 500 psi (3.5 MPa), and will not be expected to be exposed to more than one cycle of freezing and thawing before being buried or backfilled. Techniques for estimating the in-place strength are discussed in Chapter 6. To protect massive concrete against thermal cracking, a longer protection period than given in Table 5.1 is required. For concrete with a low cement content, a longer protection period may be needed to reach a strength of 500 psi (3.5 MPa).

5.2 - Need for additional protection

The comparatively short periods of protection shown in Table 5.1 are for air-entrained concrete having the air content recommended in ACI 211.1. These are the minimum protection requirements to prevent damage from one early cycle of freezing and thawing* and thereby assure that there is no impairment to the ultimate durability of the concrete. These short periods are permissible only when: (1) there is sufficient subsequent curing (see Chapter 8) and protection to develop the required safe strength for the specific service category (see Section 5.3); and (2) the concrete is not subject to freezing in a critically saturated condition. When there are early-age strength requirements, it is necessary to extend the protection period beyond the minimum duration given in Table 5.1.

*Since non-air-entrained concrete should not be used where freezing and thawing occur, this concrete is not covered in the recommendations. However, the limited durability potential of non-air-entrained concrete is best achieved by using a protection period that is at least twice that indicated in Table 5.1.

Table 5.3 - Length of protection period for concrete placed during cold weather

Line	Service category	Protection period at temperature indicated in Line 1 of Table 3.1, days*	
		Type I or II cement	Type III cement, or accelerating admixture, or 100 lb/yd³ (60 kg/m³) of additional cement
1	1 - no load, not exposed	2	1
2	2 - no load, exposed	3	2
3	3 - partial load, exposed	6	4
4	4 - full load	See Chapter 6	

*A day is a 24-hr period.

5.3 - Length of protection period

The length of the required protection period depends on the type and amount of cement, whether an accelerating admixture is used, and the service category (Sturrup and Clendening 1962). Table 5.3 gives the minimum length of the protection period at the temperatures given in Line 1 of Table 3.1. These minimum protection periods are recommended unless the in-place strength of the concrete has attained a previously established value. The service categories are as follows:

5.3.1 *Category 1: No load, not exposed* - This category includes foundations and substructures that are not subject to early load, and, because they are buried deep within the ground or are backfilled, will undergo little or no freezing and thawing in service. For concrete in this service category, conditions are favorable for continued natural curing. This concrete requires only the protection time recommended for Category 1 (Line 1) in Table 5.3. It is seen that for Category 1, the length of the protection period in Table 5.3 is the same as the requirement for protection against early-age freezing given in Line 1 of Table 5.1. Thus, for this service category, only protection against early-age freezing is necessary.

5.3.2 *Category 2: No load exposed* - This category includes massive piers and dams that have surfaces exposed to freezing and weathering in service but have no early strength requirements. Interior portions of these structures are self-curing. Exterior surfaces will continue to cure when natural conditions are favorable. To provide initial curing and insure durability of surfaces and edges, the concrete should receive at least the length of protection recommended for Category 2 (Line 2) in Table 5.3. It is seen that for Category 2, the length of the protection period in Table 5.3 is the same as the requirement for protection against early-age freezing given in Line 2 of Table 5.1. Thus, for this service category, only protection against early-age freezing is necessary.

5.3.3 *Category 3: Partial load, exposed* - The third category includes structures exposed to the weather that may be subjected to small, early-age loads compared

Table 5.5 - Maximum allowable temperature drop during first 24 hr after end of protection period

Section size, minimum dimensions, in. (mm)			
< 12 in. (< 300 mm)	12 to 36 in. (300 to 900 mm)	36 to 72 in. (900 to 1800 mm)	> 72 in. (> 1800 mm)
50 F (28 C)	40 F (22 C)	30 F (17 C)	20 F (11 C)

with their design strengths and will have an opportunity for additional strength development prior to the application of design loads. In such cases, the concrete should have at least the length of protection recommended for Category 3 in Table 5.3.

5.3.4 *Category 4: Full load* - This category includes structural concrete requiring temporary construction supports to safely resist construction loads. Protection requirements for this category are discussed in Chapter 6.

5.4 - Stripping of forms

During cold weather, protection afforded by forms, except those made of steel, is often of great significance. In heated enclosures, forms serve to evenly distribute the heat. In many cases, if suitable insulation or insulated forms are used, the forms, including those made of steel, would provide adequate protection without supplemental heating. Thus it is often advantageous to keep forms in place for at least the required minimum period of protection. However, an economical construction schedule often dictates their removal at the earliest practicable time. In such cases, forms can be removed at the earliest age that will not cause damage or danger to the concrete. Refer to Chapter 6 and ACI 347 for additional information on form removal.

If wedges are used to separate forms from young concrete, they should be made of wood. Usually, if the concrete is sufficiently strong, corners and edges will not be damaged during stripping. The minimum time before stripping can best be determined by experience, since it is influenced by several job factors, including type and amount of cement and other aspects of the concrete mixture, curing temperature, type of structure, design of forms, and skill of workers. After removal of forms, concrete should be covered with insulating-blankets or protected by heated enclosures for the time recommended in Table 5.3. If internal heating by embedded electrical coils is used, concrete should be covered with an impervious sheet and heating continued for the recommended time.

In the case of retaining walls, basement walls, or other structures where one side could be subjected to hydrostatic pressure, hasty removal of forms while the concrete is still relatively young may dislodge the form ties and create channels through which water can flow.

5.5 - Temperature drop after removal of protection

At the end of the protection period, concrete should be cooled gradually to reduce crack-inducing differential strains between the interior and exterior of the structure. The temperature drop of concrete surfaces should not exceed the rates indicated in Table 5.5. This can be accomplished by slowly reducing sources of heat, or by allowing insulation to remain until the concrete has essentially reached equilibrium with the mean ambient temperatures. Insulated forms, however, can present some difficulties in lowering the surface temperatures. Initial loosening of forms away from the concrete and covering with polyethylene sheets to allow some air circulation can alleviate the problem. As shown in Table 5.5, the maximum allowable cooling rates for surfaces of mass concrete are lower than for thinner members.

5.6 - Allowable temperature differential

Although concrete should be cooled to ambient temperatures to avoid thermal cracking, a temperature differential may be permitted when protection is discontinued. For example, Fig. 5.6 can be used to determine the maximum allowable difference between the concrete temperature in a wall and the ambient air temperature (winds not exceeding 15 mph [24 km/h]). These curves compensate for the thickness of the wall and its shape restraint factor, which is governed by the ratio of wall length to wall height.

CHAPTER 6 - PROTECTION FOR STRUCTURAL CONCRETE REQUIRING CONSTRUCTION SUPPORTS

6.1 - Introduction

For structural concrete, where a considerable level of design strength must be attained before safe removal of forms and shores is permitted, additional protection time must be provided beyond the minimums given in Table 5.1, since these minimum times are not sufficient to allow adequate strength gain. The criteria for removal of forms and shores from structural concrete should be based on the in-place strength of the concrete rather than on an arbitrary time duration. The recommendations in this chapter are based on job conditions meeting the requirements listed in Section 6.10.

6.2 - Tests of field-cured specimens

One method used to verify attainment of sufficient in-place strength before support is reduced, changed, or removed, and before curing and protection are discontinued, is to cast at least six field-cured test specimens from the last 100 yd^3 (75 m^3) of concrete. However, at least three specimens should be cast for each 2 hr of the entire placing time, or for each 100 yd^3 (75 m^3) of concrete, whichever provides the greater number of specimens. The specimens should be made in accordance with ASTM C 31, following the procedures given for "Curing Cylinders for Determining Form Removal Time or When a Structure May be Put into Service." The specimens should be protected immediately from the cold weather until they can be placed under the same protection provided for the parts of the structure they represent. After demolding, the cylinders should

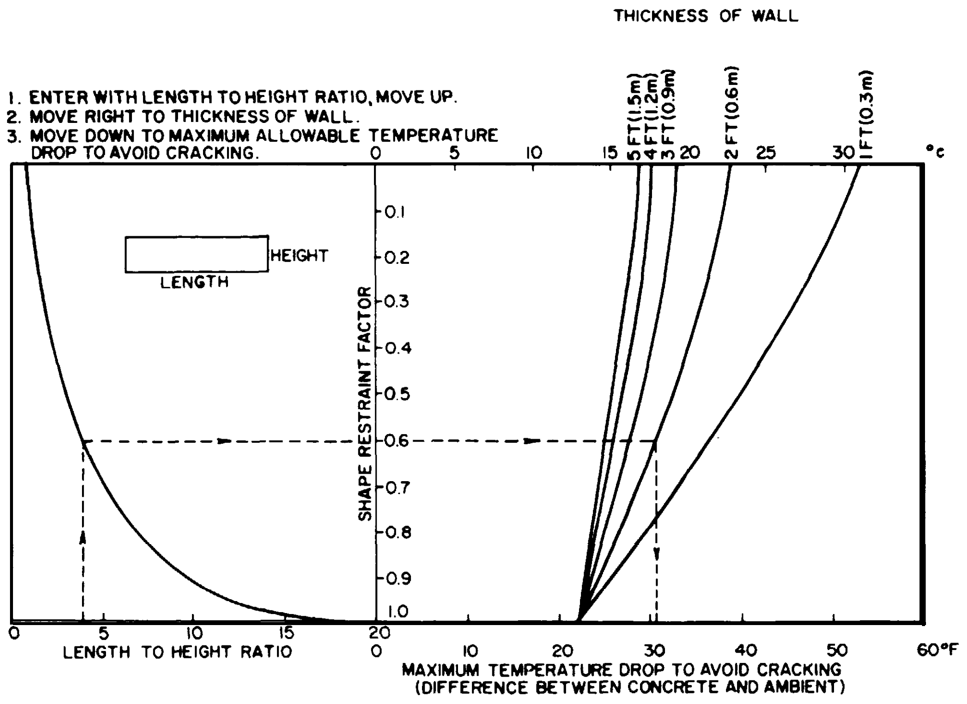

Fig. 5.6 - Graphical determination of safe differential temperature for walls (Mustard and Ghosh 1979)

be capped and tested in accordance with the applicable sections of ASTM C 31 and ASTM C 39.

For flatwork, field-cured test specimens can be obtained by using special cylindrical molds that are positioned in the formwork and filled during the placement of concrete in the structure (ASTM C 873). Since the test specimens are cured in the structure, they experience the same temperature history as the structure. When a strength determination is required, the molds are extracted from the structure and the cylinder is prepared for testing according to ASTM C 39. The holes remaining in the structure would be filled with concrete.

6.3 - In-place testing

In-place and nondestructive concrete strength testing (Malhotra 1976), when correlated with field-cured and standard-cured (ASTM C 192) cylinder test results, is another method that can be used to verify attainment of strength. These tests are performed on the concrete in the structure using portable, hand-held instruments, and they offer advantages compared with testing field-cured specimens. For example, in-place testing eliminates the difficulty of trying to prepare test specimens that truly experience the same temperature history as the concrete in the structure. Hence, they are usually preferable to testing field-cured specimens prepared according to ASTM C 31. Applicable in-place test methods include the probe penetration method (ASTM C 803) and the pullout test method (ASTM C 900). The architect/engineer should review and accept the proposed method, including appropriate correlation data, for estimating in-place strength.

6.4 - Maturity method

Since strength gain of concrete is a function of time and temperature, estimation of strength development of concrete in a structure also can be made by relating the time-temperature history of field concrete to the strength of cylinders of the same concrete mixture cured under standard conditions in a laboratory. This relationship has been established (Bergstrom 1953) by use of a maturity factor M expressed as

$$M = \Sigma\ (T - T_o)\ \Delta t \qquad (6\text{-}1)$$

where
M = maturity factor, deg-hr
T = temperature of concrete, deg F (C)
T_o = datum temperature, deg F (C)
Δt = duration of curing period at temperature T, hr

When concrete temperature is constant, as in laboratory curing methods, the summation sign in Eq. (6-l) is not necessary. The appropriate value for the datum temperature T_o depends on the type of cement, the type and quantity of admixture, and the range of the curing temperature. A value of 23 F (-5 C) is suggested (Carino 1984) for concrete made with Type I cement and cured within the range of 32 to 70 F (0 to 20 C). This value may not be applicable to other types of cements or to Type I cement in combination with liquid or mineral admixtures. A procedure for experimental determination of the datum temperature is given in ASTM C 1074.

6.4.1 - The principle of the maturity method is that the strength of a given concrete mixture can be related

Table 6.4.4 Calculation of maturity factor and estimated in-place strength

1	2	3	4	5	6	7	8	9	10
Date	Elapsed time *h*, hr	Temperature in structure, F	Average temperature in structure F	C	Col. 5 + 5 C, C	Time interval *h*, hr	Col. 6x Col. 7 C hr	Maturity factor Σ Col. 8, C hr	Corresponding compressive strength psi
Sept. 1	0	50							
	12	50	50	10	15	12	180	180	-
2	24	50	50	10	15	12	180	360	-
	30	46	48	8.9	13.9	6	83	443	400
3	48	48	47	8.3	13.3	18	240	683	1080
	60	46	47	8.3	13.3	12	160	843	1400
4	72	44	45	7.2	12.2	12	146	989	1600
8	168	42	43	6.1	11.1	96	1065	2054	2600
11	240	42	42	5.6	10.6	72	763	2817	3100
14	312	42	42	5.6	10.6	72	763	3580	3400

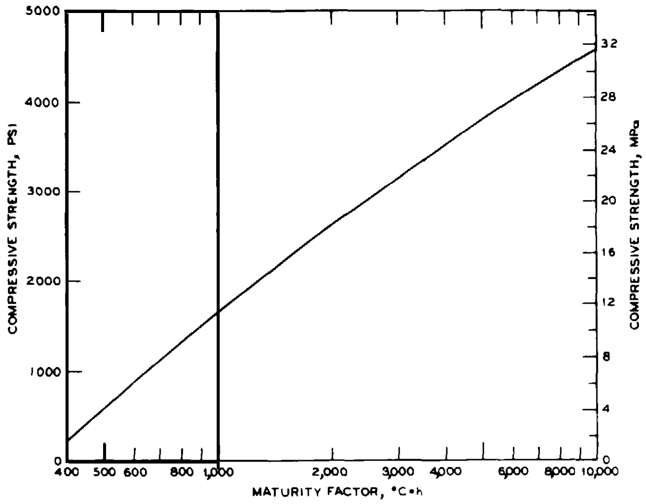

Fig. 6.4.4 - Example of a strength-maturity factor relationship for laboratory-cured cylinders (22.8 C)

to the value of the maturity factor. To use this technique, a strength versus maturity factor curve is established by performing compressive strength tests at various ages on a series of cylinders made with concrete similar to that which will be used in construction. The specimens are usually cured at room temperature and the temperature history of the concrete is recorded to compute the maturity factor at the time of testing. The average cylinder strengths and corresponding maturity factors at each test age are plotted, and a smooth curve is fitted to the data.

6.4.2 - The in-place strength of properly cured concrete at a particular location and at a particular time is predicted by determining the maturity factor at that time and reading the corresponding strength on the strength-maturity factor curve. The in-place maturity factor at a particular location is determined by measuring the temperature of the concrete at closely spaced time intervals and using Eq. (6-l) to sum the successive products of the time intervals and the corresponding average concrete temperature above the datum temperature.

Temperatures can be measured with expendable thermistors or thermocouples cast in the concrete. The temperature sensors should be embedded in the structure at critical locations in terms of severity of exposure and loading conditions. Electronic instruments known as maturity meters are available that permit direct and continuous determination of the maturity factor at a particular location in the structure. These instruments use a probe inserted into a tube embedded in the concrete to measure the temperature, and they automatically compute and display the maturity factor in degree-hours.

6.4.3 - Strength prediction based on the maturity factor assumes that the in-place concrete has the same strength potential as the concrete used to develop the strength-maturity factor curve. Prior to removing forms or shores, it is necessary to determine whether the in-place concrete has the assumed strength potential. This may be done by additional testing of the concrete in question, such as by testing standard-cured cylinders at early ages, by using accelerated strength tests as described in ASTM C 684, by testing field-cured cylinders for which the maturity factor has been monitored, or by using one of the in-place tests listed in 6.3.

6.4.4 *Example* - The following example illustrates the use of the maturity factor method:

In anticipation of cold weather, a contractor installed thermocouples at critical locations in a concrete wall placed at 9:00 A.M. on Sept. 1. A history of the strength gain for the particular concrete mixture to be used in the wall had been developed under laboratory conditions, and the strength-maturity factor curve, which is shown in Fig. 6.4.4, had been established. A record of the in-place concrete temperature was maintained as indicated in Columns 2 and 3 of Table 6.4.4.

After 3 days (72 hr), the contractor needed to know the in-place strength of the concrete in the wall. Using the temperature record, the contractor calculated the average temperature (Column 5) during the various time intervals and the cumulative maturity factors at different ages (Column 9). Based on the strength-maturity factor curve (Fig. 6.4.4), the predicted in-place

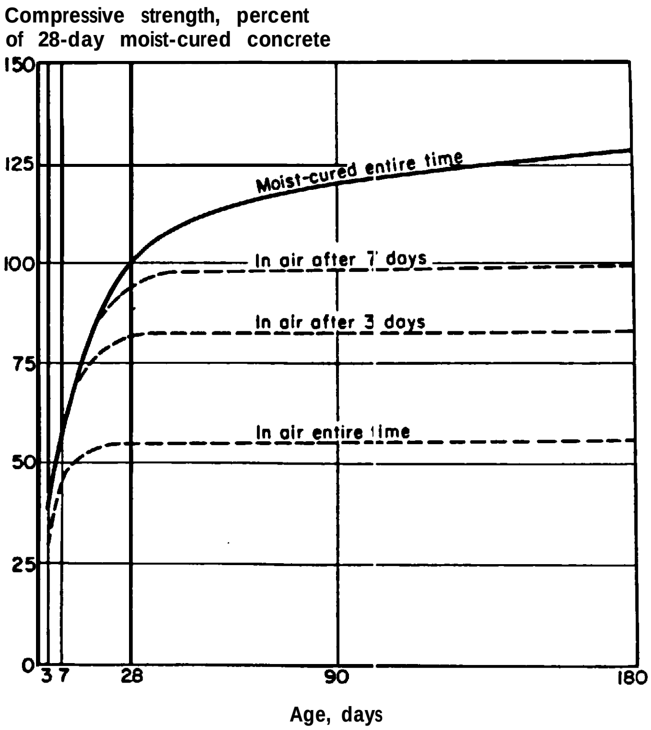

Fig. 6.5 - Compressive strength of concrete dried in laboratory air after preliminary moist curing (Price 1951)

strength (Column 10) at 72 hr is 1600 psi (11.0 MPa). By continuing the procedure, strength at later ages can be predicted.

6.5 - Attainment of design strength

Generally there is little opportunity for further curing of structural concrete beyond that provided initially. Fig. 6.5 illustrates the strength development of concrete specimens removed from moist curing at various ages and subsequently exposed to laboratory air. It is seen that as the specimens dry out, strength gain ceases. For this reason, early strengths high enough to assure later attainment of design strength must be attained before temporarily supported structural concretes can be safely released from cold weather protection and exposed to freezing weather.

6.6 - Increasing early strength

The time needed for concrete to attain the strength required for safe removal of shores is influenced by many factors. Most important among them are those that affect the rate and level of strength development, such as the initial temperature of the concrete when placed, the temperature at which the concrete is maintained after placing, the type of cement, the type and amount of accelerating admixture or other admixtures used, and the conditions of protection and curing. Economic considerations may dictate an accelerated construction schedule even though the resulting concrete may be of lesser quality in terms of reduced long-term ultimate strength or increased thermal cracking. In

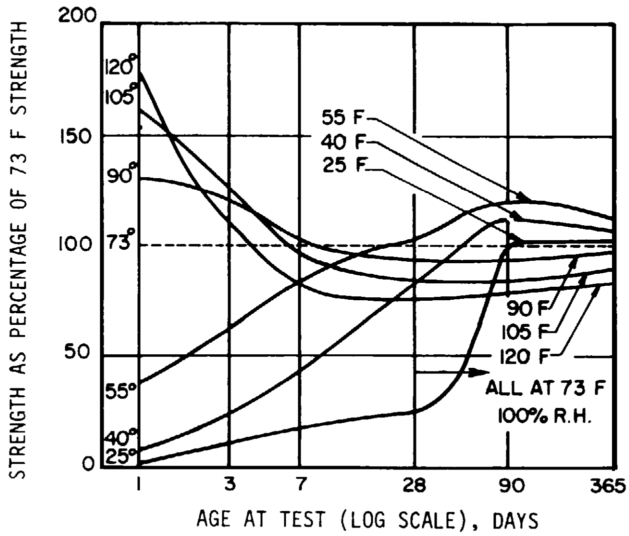

Fig. 6.6.1 - Effect of temperature conditions on the strength development of concrete (Type I cement) (Kleiger 1958)

such cases, the duration of protection may be substantially reduced by:

6.6.1 - holding the temperature during protection and curing at a higher level than that indicated in Line 1 of Table 3.1. Fig. 6.6.1 illustrates the effects of curing temperature on strength development, where strength is expressed as a percentage of the strength at the same age for curing at 73 F (23 C). Type I and III cements give somewhat higher strengths than Type II at early ages. Because of variations in the performance of any one given cement, the data in Fig. 6.6.1 should be used only as a guide.

6.6.2 - using types and compositions of cement that exhibit higher earlier strength development and by using a higher cement content with a lower water-cement ratio (see Section 9.1).

6.6.3 - using an accelerating admixture conforming to ASTM C 494, Type C (accelerating), or Type E (water-reducing and accelerating). The several items for concern mentioned in Chapter 9 should be reviewed before using calcium chloride or Type C or Type E admixtures containing calcium chloride. Due to variation in performance with different brands and types of cement, tests should be performed in advance at the anticipated curing temperature using the cement, aggregates, and admixtures proposed for use.

6.7 - Cooling of concrete

To lower the likelihood of cracking due to thermal stresses, precautions should be taken to assure gradual cooling of concrete surfaces at the termination of the protection period (see Section 5.5).

6.8 - Estimating strength development

There may be times when conservative estimates of concrete strength must be made when adequate curing and protection was provided but no actions were taken to determine the level of strength development. In such

Table 6.8 - Duration of recommended protection for percentage of standard-cured 28-day strength*

Percentage of standard-cured 28-day strength	At 50 F (10 C), days			At 70 F (21 C), days		
	Type of cement			Type of cement		
	I	II	III	I	II	III
50	6	9	3	4	6	3
65	11	14	5	8	10	14
85	21	28	16	16	18	12
95	29	35	26	23	24	20

*The data in this table were derived from concretes with strengths from 3000 to 5000 psi (20.7 to 34.4 MPa) after 28 days of curing at 70 ± 3 F (21 ± 1.7 C). The 28-day strength for each type of cement was considered as 100 percent in determining the times to reach various percentages of this strength for curing at 50 F (10 C) and 70 F (21 C). These times are only approximate, and specific values should be obtained for he concrete used on the job.

cases, Table 6.8 may be used as a guide to determine the recommended duration of curing and protection at 50 or 70 F (10 or 21 C) to achieve different percentages of the standard-cured 28-day strength.

6.9 - Removal of forms and supports

The removal of forms and supports and the placing and removal of reshores should be in accordance with the recommendations of ACI 347:

6.9.1 - The in-place strength of concrete that is required to permit removal of forms and shores should be specified by the engineer/architect.

6.9.2 - Suitable tests of field-cured concrete specimens or of concrete in place should be made (see Sections 6.2 and 6.3).

6.9.3 - Methods for evaluating the results of concrete strength tests and the minimum strength required for form and shore removal should be completely prescribed in the specifications.

6.9.4 - The results of all tests, as well as records of weather conditions and other pertinent information, should be recorded and used by the engineer/architect in deciding when to remove forms and shores.

6.9.5 - The reshoring procedure, which is one of the most critical operations in formwork, should be planned in advance and reviewed by the engineer. This operation should be performed so that large areas of new construction are not subjected to combined dead and construction loads in excess of their capacity, as determined by the in-place concrete strength at the time of form removal and reshoring.

6.10 - Requirements

The recommendations made in this chapter and in Table 6.8 are based on job conditions meeting the following requirements:

6.10.1 - The internal concrete temperature is at least 50 F (10 C) after the concrete is in place. To reduce subsequent thermal contractions, this temperature should be exceeded as little as is practicable.

6.10.2 - Facilities are available to maintain the concrete temperature throughout the structure at 50 F (10 C) or above until protection can be safely discontinued. Such facilities should incorporate, as required, the following:

a. suitable protection from wind and loss of heat

b. effective and sufficient heating equipment and personnel to maintain all parts of the concrete at the required temperature

c. necessary fire protection equipment

d. protection and heating to include the top surface of newly placed slabs or floors

e. venting and circulation to maintain an even temperature at the top and the bottom of vertical units such as walls, piers, and columns

6.10.3 - Reshores are left in place as long as is necessary to safeguard all members of the structure. The number of tiers reshored below the tier being placed and how long such reshores remain in place should be based on reliable evidence that sufficient strength exists to safely carry the applied loads.

6.10.4 - The concrete is made with Type I, II, or III portland cement.

6.10.5 - Proper curing is used, particularly to avoid drying of the concrete in heated enclosures.

6.10.6 - Inspections are performed to check compliance with the construction requirements of ACI 301, ACI 318, and other ACI standards for construction practices.

CHAPTER 7 - MATERIALS AND METHODS OF PROTECTION

7.1 - Introduction

Concrete placed during cold weather should be maintained as close as possible to the recommended temperatures in Line 1 of Table 3.1 and for the length of times recommended in Table 5.3 unless the in-place strength has reached a previously established target value. The specific protection system required to maintain the recommended temperatures depends on such factors as the ambient weather conditions, the geometry of the structure, and the concrete mixture proportions. In some cases, it may only be necessary to cover the concrete with insulating materials and use the natural heat of hydration to maintain the temperature at the recommended levels. In more extreme cases, it may be necessary to build enclosures and use heating units to maintain the desired temperatures.

7.2 - Insulating materials

Since most of the heat of hydration of the cement is generated during the first 3 days, heating from external sources may not be required to prevent freezing of the concrete if the generated heat is retained. Heat of hydration may be retained by using insulating blankets on unformed surfaces and by using insulating forms (Tuthill, et al. 1951; Wallace 1954; Mustard and Ghosh 1979). Insulation must be kept in close contact with the concrete or the form surface to be effective. Some commonly used insulating materials include:

7.2.1 *Polystyrene foam sheets* - These may be cut to shape and wedged between the studs of the forms or glued into place.

7.2.2 *Urethane foam* - This foam may be sprayed onto the surface of forms making a continuous insulating layer. A good weather-resistant enamel should be sprayed over urethane foam to reduce water absorption and protect it from the deteriorating effect of ultraviolet rays. Urethane foam should be used with caution because it generates highly noxious fumes when exposed to fire.

7.2.3 *Foamed vinyl blankets* - These materials are pliable blankets of foamed vinyl with an extruded vinyl backing. They also may have electric wires embedded in the foam to provide additional heat. Blankets without embedded wires are available in rolls of standard widths. Heated blankets must be custom ordered.

7.2.4 *Mineral wool or cellulose fibers* - Generally, mineral wool or cellulose fibers are encased in heavy polyethylene liners and formed into large mats or rolls. The plastic liners are sometimes given a rough surface finish to reduce the risk of slipping. They may be laid flat to cover slabs or they may be draped over other structural elements.

7.2.5 *Straw* - Straw is still popular although it is not as effective as blankets or mats Disadvantages of straw are its bulk, its flammability, and the need to protect it from moisture. Tarpaulins, polyethylene sheets, or waterproof paper must be used as a protective cover to inhibit wind convection and to keep the straw dry and in place.

7.2.6 *Blanket or batt insulation* - Commercial blanket or batt insulation must be adequately protected from wind, rain, snow, or other moisture by means of tough, moistureproof cover material because wetting will impair its insulating value. Closed-cell material is particularly advantageous because of its resistance to wetting.

7.3 - Selection of insulation

Concrete temperature records reveal the effectiveness of different amounts or kinds of insulation and of other methods of protection for various types of concrete work under different weather conditions. Using these temperature records, appropriate modifications can be made to the protection method or selected materials. Additional methods for estimating the temperatures that can be maintained by various insulation arrangements under given weather conditions have been published (Tuthill et al. 1951; Mustard and Ghosh 1979).

As was mentioned in Section 7.2, the heat of hydration is high during the first few days after placement and it then gradually decreases with age. Thus, to maintain a specified temperature throughout the protection period, the amount of required insulation is greater for a long protection period than for a shorter period. Conversely, for a giver insulation system, concrete that is to be protected for a short period, such as 3 days, can be exposed to a lower ambient temperature than concrete that is to be protected for a longer period, such as 7 days.

Based on the requirements of this report, Tables 7.3.1, 7.3.2, 7.3.3, and 7.3.4 and Fig. 7.3.1, 7.3.2, 7.3.3, and 7.3.4 indicate the minimum ambient air temperatures to which concrete walls or slabs of different thicknesses may be exposed for different values of thermal resistance *R*, for different cement contents, and for protection periods of 3 or 7 days. For protection periods less than 3 days, the tables or figures for 3 days should be used. For protection periods between 3 and 7 days, interpolation between the values in the tables or figures should be used. For these figures and tables it is assumed that the concrete temperature as placed is 50 F (10 C).

These tables and figures can also be used to determine the required thermal resistance *R* under different conditions. The thickness of the chosen insulating material that is needed to obtain the required thermal resistance can be calculated using the insulation values in Table 7.3.5. The thermal resistance of the various insulating materials have been calculated under the assumption that the insulation is applied to the exterior surfaces of steel forms. When 3/4-in. (20-mm) plywood forms are used, an *R* value of 0.94 (0.17) for the plywood form should be added to the *R*-value of the added insulation. The values shown in the tables and figures are based on the assumption that wind speeds are not greater than 15 mph (24 km/h). With higher wind speeds, the effectiveness of a given thickness of insulation diminishes. However, at a wind speed of 30 mph (48 km/h), the decrease in the effective thermal resistance amounts to less than an *R* - value of 0.1. Thus, for all practical purposes, the effects of wind speed may be neglected in determining the required thickness of added insulation.

Corners and edges are particularly vulnerable during cold weather. Therefore, the thickness of insulation for these parts should be about three times the thickness that is required for walls or slabs. In addition, the tables and figures are for cement having a heat of hydration similar to Type I portland cement. For Type II cement and blended hydraulic cements with moderate heat of hydration, the insulation requirements given in the tables and figures should be increased by about 30 percent. Where other types of cements or blends of cement and other cementitious materials are used,* similar proportional adjustments should be made to the amount of insulation (Tuthill et al. 1951; Mustard and Ghosh 1979). Insulation beyond the required amount should not be used because it may raise the internal temperature of the concrete above recommended levels, which will lengthen the gradual cooling period, increase thermal shrinkage, and increase the risk of cracking due to thermal shock.

7.3.1 *Example* - The following example illustrates how to determine the required thickness of insulation for a given condition.

*Typical heat of hydration curves for various cements can be found in "Concrete for Massive Structures," *Bulletin No. IS128T*, Portland Cement Association, Skokie, IL.

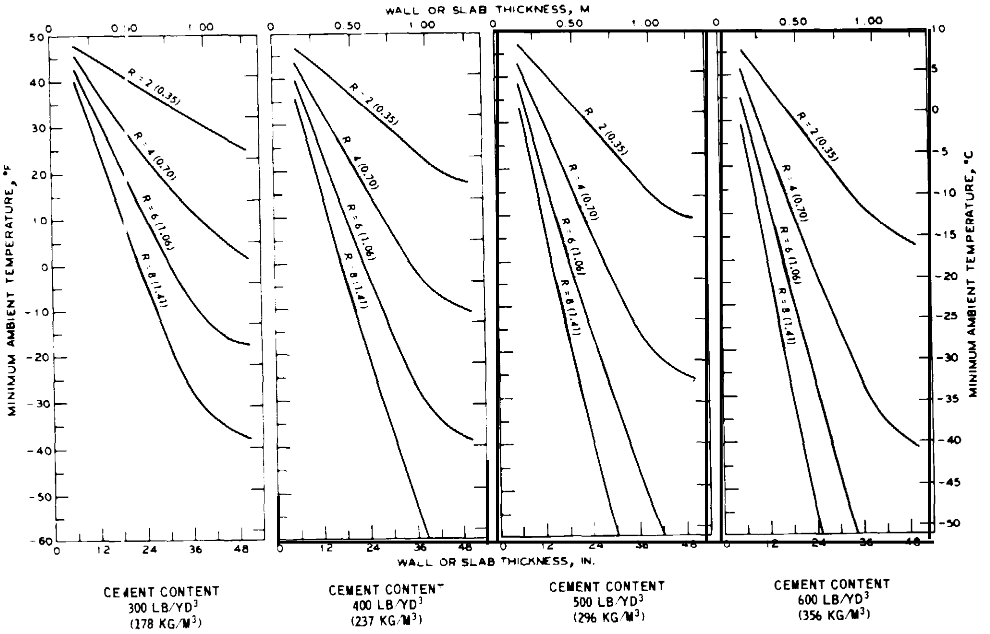

Fig. 7.3.1- Minimum exposure temperatures for concrete slabs above ground and walls as a function of member thickness, R*-value, and cement content. Concrete placed and surface temperature maintained at 50 F (10 C) for 7 days*

Table 7.3.1 - Minimum exposure temperatures for concrete slabs above ground and walls for concrete placed and surface temperature maintained at 50 F (10 C) for 7 days

Wall or slab thickness, in. (mm)	Minimum ambient air temperature, deg F (C) allowable when insulation having these values of thermal resistance R, hr•ft^2•F/Btu (m^2•K/W), is used			
	R = 2 (0.35)	R = 4 (0.70)	R = 6 (1.06)	R = 8 (1.41)
Cement content = 300 lb/yd^3 (178 kg/m^3)				
6 (0.15)	48 (9)	46 (8)	43 (6)	40 (4)
12 (0.30)	45 (7)	39 (4)	32 (0)	25 (-4)
18 (0.46)	41 (5)	31 (-1)	21 (-6)	11 (-12)
24 (0.61)	38 (3)	24 (-4)	10 (-12)	-2 (-19)
36 (0.91)	32 (0)	12 (-11)	-8 (-22)	-28 (-33)
48 (1.2)	26 (-3)	3 (-16)	-17 (-27)	-37 (-38)
60 (1.5)	26 (-3)	3 (-16)	-17 (-27)	-37 (-38)
Cement content = 400 lb/yd^3 (237 kg/m^3)				
6 (0.15)	47 (8)	44 (7)	40 (4)	36 (2)
12 (0.30)	43 (6)	35 (2)	26 (-3)	17 (-8)
18 (0.46)	39 (4)	25 (-4)	11 (-12)	-2 (-19)
24 (0.61)	34 (1)	16 (-8)	-2 (-19)	-20 (-29)
36 (0.91)	25 (-4)	-1 (-18)	-27 (-31)	-53 (-47)
48 (1.2)	18 (-8)	-10 (-23)	-38 (-39)	*
60 (1.5)	18 (-8)	-10 (-23)	-38 (-39)	*
Cement content = 500 lb/yd^3 (296 kg/m^3)				
6 (0.15)	47 (8)	43 (6)	38 (3)	33 (1)
12 (0.30)	42 (6)	31 (-1)	20 (-7)	9 (-13)
18 (0.46)	36 (2)	19 (-7)	2 (-17)	-15 (-26)
24 (0.61)	30 (-1)	7 (-14)	-16 (-27)	-39 (-39)
36 (0.91)	18 (-8)	-15 (-26)	-46 (-43)	-79 (-62)
48 (1.2)	10 (-12)	-25 (-32)	-60 (-51)	*
60 (1.5)	10 (-12)	-25 (-32)	*	*
Cement content = 600 lb/yd^3 (356 kg/m^3)				
6 (0.15)	46 (8)	41 (5)	35 (2)	29 (-2)
12 (0.30)	40 (4)	28 (-2)	14 (-10)	0 (-18)
18 (0.46)	33 (1)	13 (-11)	-7 (-22)	-29 (-34)
24 (0.61)	26 (-3)	-1 (-18)	-28 (-33)	-55 (-48)
36 (0.91)	12 (-11)	-27 (-31)	-66 (-54)	*
48 (1.2)	4 (-16)	-40 (-40)	*	*
60 (1.5)	4 (-16)	-40 (-40)	*	*

*<< -60 F (-51 C).

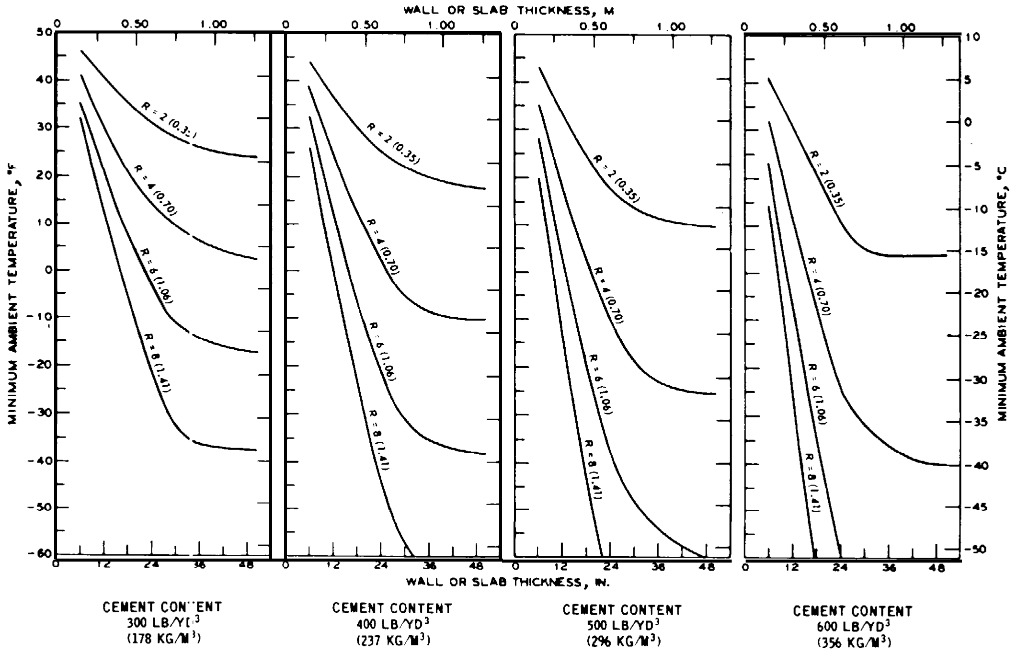

Fig. 7.3.2 - Minimum exposure temperatures for concrete slabs above ground and walls as a function of member thickness, R*-value, and cement content. Concrete placed and surface temperature maintained at 50 F (10 C) for 3 days*

Table 7.3.2 - Minimum exposure temperatures for concrete slabs above ground and walls for concrete placed and surface temperature maintained at 50 F (10 C) for 3 days

Wall or slab thickness, in. (mm)	Minimum ambient air temperature, deg F (C) allowable when insulation having these values of thermal resistance *R*, hr•ft²•F/Btu (m²•K/W), is used			
	R = 2 (0.35)	R = 4 (0.70)	R = 6 (1.06)	R = 8 (1.41)
	Cement content = 300 lb/yd³ (178 kg/m³)			
6 (0.15)	46 (8)	41 (5)	36 (2)	32 (0)
12 (0.30)	41 (5)	31 (-1)	21 (-6)	11 (-12)
18 (0.46)	36 (2)	21 (-6)	8 (-13)	-5 (-21)
24 (0.61)	31 (-1)	14 (-10)	-3 (-19)	-21 (-29)
36 (0.91)	26 (-3)	8 (-13)	-14 (-26)	-36 (-38)
48 (1.2)	26 (-3)	3 (-16)	-17 (-27)	-37 (-38)
60 (1.5)	26 (-3)	3 (-16)	-17 (-27)	-37 (-38)
	Cement content = 400 lb/yd³ (237 kg/m³)			
6 (0.15)	44 (7)	38 (3)	32 (0)	26 (-3)
12 (0.30)	37 (3)	24 (-4)	12 (-11)	0 (-18)
18 (0.46)	30 (-1)	12 (-11)	-6 (-21)	-24 (-31)
24 (0.61)	25 (-4)	2 (-17)	-21 (-29)	-44 (-42)
36 (0.91)	20 (-7)	-9 (-23)	-36 (-38)	-63 (-53)
48 (1.2)	18 (-8)	-10 (-23)	-38 (-39)	*
60 (1.5)	18 (-8)	-10 (-23)	-38 (-39)	*
	Cement content = 500 lb/yd³ (296 kg/m³)			
6 (0.15)	43 (6)	35 (2)	28 (-2)	20 (-7)
12 (0.30)	34 (1)	18 (-8)	3 (-16)	-12 (-24)
18 (0.46)	25 (-4)	2 (-16)	-21 (-29)	-44 (-42)
24 (0.61)	18 (-8)	-10 (-23)	-38 (-39)	-68 (-56)
36 (0.91)	12 (-11)	-23 (-31)	-60 (-51)	*
48 (1.2)	13 (-12)	-25 (-32)	*	*
60 (1.5)	13 (-12)	-25 (-32)	*	*
	Cement content = 600 lb/yd³ (356 kg/m³)			
6 (0.15)	41 (5)	32 (0)	23 (-5)	14 (-10)
12 (0.30)	31 (-1)	12 (-11)	-7 (-22)	-26 (-32)
18 (0.46)	21 (-6)	-7 (-22)	-35 (-37)	-63 (-53)
24 (0.61)	11 (-12)	-24 (-31)	-59 (-51)	*
36 (0.91)	1 (-16)	-36 (-38)	*	*
48 (1.2)	1 (-16)	-40 (-40)	*	*
60 (1.5)	1 (-16)	-40 (-40)	*	*

* << -60 F (-51 C).

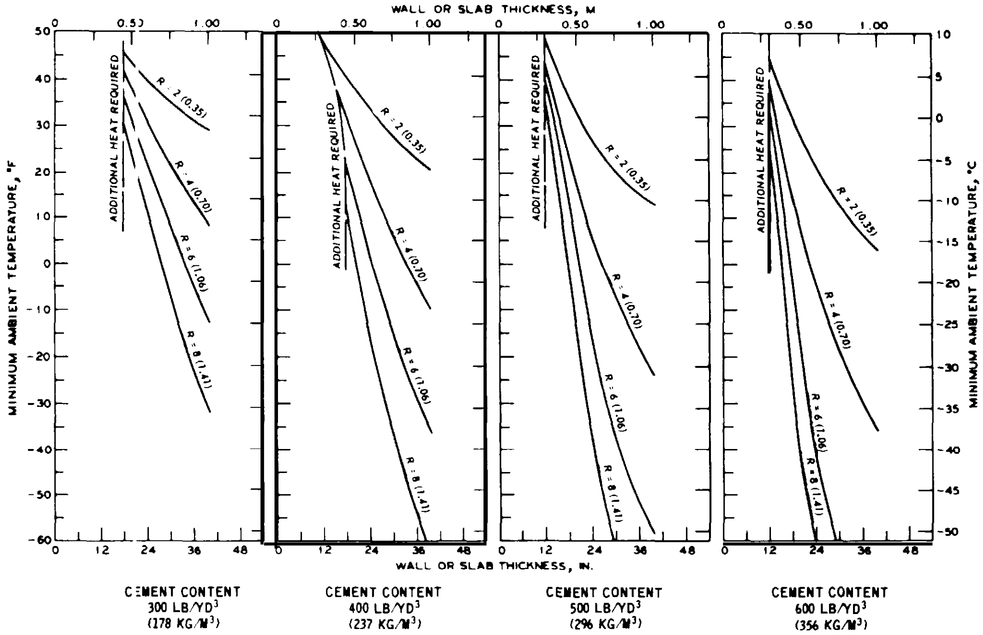

Fig. 7.3.3. - Minimum exposure temperatures for concrete flatwork placed on the ground as a function of member thickness, R-*value, and cement content. Concrete placed and surface temperature maintained at 50 F (10 C) for 7 days on ground at 35 F (2 C)*

Table 7.3.3 - Minimum exposure temperatures for concrete flatwork placed on the ground for concrete placed and surface temperature maintained at 50 F (10 C) for 7 days on ground at 35 F (2 C)

Wall or slab thickness, in. (mm)	Minimum ambient air temperature, deg F (C) allowable when insulation having these values of thermal resistance *R*, hr•ft^2•F/Btu (m^2•K/W), is used			
	R = 2 (0.35)	R = 4 (0.70)	R = 6 (1.06)	R = 8 (1.41)
	Cement content = 300 lb/yd^3 (178 kg/m^3)			
4 (0.10)	*	*	*	*
8 (0.20)	*	*	*	*
12 (0.31)	*	*	*	*
18 (0.46)	46 (8)	42 (6)	36 (2)	30 (-1)
24 (0.61)	40 (4)	31 (-1)	22 (-6)	11 (-12)
30 (0.76)	35 (2)	22 (-6)	7 (-14)	-8 (-22)
36 (0.91)	31 (-1)	13 (-11)	-5 (-21)	-23 (-31)
	Cement content = 400 lb/yd^3 (237 kg/m^3)			
4 (0.10)	*	*	*	*
8 (0.20)	*	*	*	*
12 (0.31)	*	*	*	50 (10)
18 (0.46)	41 (5)	32 (0)	22 (-6)	12 (-11)
24 (0.61)	35 (2)	19 (-7)	-1 (-17)	-15 (-26)
30 (0.76)	28 (-2)	8 (-13)	-14 (-26)	-36 (-38)
36 (0.91)	23 (-5)	-4 (-20)	-29 (-34)	-54 (-48)
	Cement content = 500 lb/yd^3 (296 kg/m^3)			
4 (0.10)	*	*	*	
8 (0.20)	*	*	*	*
12 (0.31)	48 (9)	44 (7)	40 (4)	36 (2)
18 (0.46)	36 (2)	22 (-6)	8 (-13)	-6 (-21)
24 (0.61)	28 (-2)	6 (-14)	-16 (-27)	-38 (-39)
30 (0.76)	22 (-6)	-7 (-22)	-36 (-38)	-64 (-53)
36 (0.91)	16 (-9)	-18 (-28)	-50 (-46)	†
	Cement content = 600 lb/yd^3 (356 kg/m^3)			
4 (0.10)	*	*	*	*
8 (0.20)	*	*	*	*
12 (0.31)	44 (7)	38 (3)	32 (0)	26 (-3)
18 (0.46)	31 (-1)	14 (-10)	-5 (-21)	-24 (-31)
24 (0.61)	22 (-6)	-5 (-21)	-32 (-36)	-61 (-52)
30 (0.76)	14 (-10)	-19 (-28)	-67 (-55)	†
36 (0.91)	7 (-14)	-30 (-34)	†	†

* > 50 F (10 C): additional heat required.
† << -60 F (-51C).

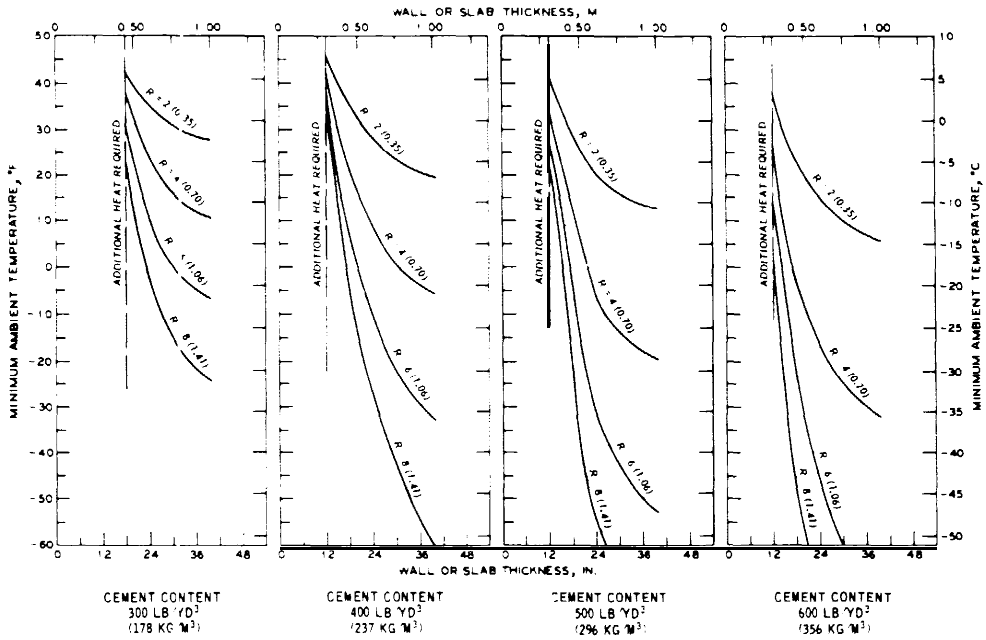

Fig. 7.3.4 - Minimum exposure temperatures for concrete flatwork placed on the ground as a function of member thickness, R-*value, and cement content. Concrete placed and surface temperature maintained at 50 F (10 C) for 3 days on ground at 35 F (2 C)*

Table 7.3.4 - Minimum exposure temperatures for concrete flatwork placed on the ground for concrete placed and surface temperature maintained at 50 F (10 C) for 3 days on ground at 35 F (2 C)

Slab thickness, in. (m)	Minimum ambient air temperature, deg F (C) allowable when insulation having these values of thermal resistance *R*, hr•ft^2•F/Bru (m^2•K/W), is used			
	R = 2 (0.35)	*R* = 4 (0.70)	*R* = 6 (1.06)	*R* = 8 (1.41)
Cement content = 300 lb/yd^3 (178 kg/m^3)				
4 (0.10)	*	*	*	*
8 (0.20)	*	*	*	*
12 (0.31)	*	*	*	*
18 (0.46)	42 (6)	38 (3)	32 (0)	26 (-3)
24 (0.61)	37 (3)	25 (-4)	11 (-12)	-3 (-19)
30 (0.76)	31 (-1)	15 (-9)	-1 (-18)	-17 (-27)
36 (0.91)	31 (-1)	12 (-11)	-5 (-21)	-22 (-30)
Cement content = 400 lb/yd^3 (237 kg/m^3)				
4 (0.10)	*	*	*	*
8 (0.20)	*	*	*	*
12 (0.31)	46 (8)	44 (7)	42 (6)	40 (4)
18 (0.46)	36 (2)	22 (-6)	8 (-13)	-6 (-21)
24 (0.61)	28 (-2)	9 (-13)	-10 (-23)	-29 (-34)
30 (0.76)	21 (-6)	0 (-18)	-21 (-29)	-42 (-41)
36 (0.91)	21 (-6)	-4 (-20)	-29 (-34)	-50 (-46)
Cement content = 500 lb/yd^3 (296 kg/m^3)				
4 (0.10)	*	*	*	*
8 (0.20)	*	*	*	*
12 (0.31)	42 (6)	36 (2)	30 (-1)	24 (-4)
18 (0.46)	30 (-1)	12 (-11)	-6 (-21)	-22 (-30)
23 (0.61)	11 (-6)	-5 (-21)	-31 (-35)	-50 (-46)
30 (0.76)	16 (-9)	-10 (-23)	-42 (-41)	-74 (-59)
36 (0.91)	16 (-9)	-18 (-28)	-50 (-46)	
Cement content = 600 lb/yd^3 (356 kg/m3)				
4 (0.10)	*	*	*	*
8 (0.20)	*	*	*	*
12 (0.31)	38 (3)	26 (-3)	14 (- 10)	2 (-17)
18 (0.46)	24 (-1)	0 (-18)	- 24 (-31)	-48 (-44)
24 (0.61)	14 (-10)	-16 (-27)	-46 (-33)	-82 (-63)
30 (0.76)	10 (-12)	-20 (-29)	-62 (-52)	†
36 (0.91)	7 (-14)	-30 (-34)	†	†

* > 50 F (10 C): additional heat required
† < < -60 F (-51 C)

Table 7.3.5 - Thermal resistance of various insulating materials

Insulating material	Thermal resistance *R* for these thicknesses of material*: 1 in., hr•ft²•F/Btu	10 mm, m²•K/W
Boards and slabs		
Expanded polyurethane (*R* - 11 exp.)	6.25	0.438
Expanded polystyrene extruded (*R* - 12 exp.)	5.00	0.347
Expanded polystyrene extruded, plain	4.00	0.277
Glass fiber, organic bonded	4.00	0.277
Expanded polystyrene, molded beads	3.57	0.247
Mineral fiber with resin binder	3.45	0.239
Mineral fiber board, wet felted	2.94	0.204
Sheathing, regular density	2.63	0.182
Cellular glass	2.63	0.182
Laminated paperboard	2.00	0.139
Particle board (low density)	1.85	0.128
Plywood	1.25	0.087
Blanket		
Mineral fiber, fibrous form processed from rock, slag, cr glass	3.23	0.224
Loose fill		
Wood fiber, soft woods	3.33	0.231
Mineral fiber (rock, slag, or glass)	2.50	0.173
Perlite (expanded)	2.70	0.187
Vermiculite (exfoliated)	2.20	0.152
Sawdust or shavings	2.22	0.154

*Values from *ASHRAE Handbook of Fundamentals,* 1977, American Society of Heating, Refrigerating, and Air-Conditioning Engineers, New York.

Problem - A contractor anticipates having to place an 18 in. (0.46 m) thick concrete wall when the ambient temperature will be about 0 F (- 18 C). The concrete will have a cement content of 500 lb/yd^3 (296 kg/m^3). There will be no early-age strength requirements for the concrete; therefore, a 3-day protection period will be used. The forms are made of 3/4-in. plywood. The contractor would like to use plain expanded polystyrene boards for insulation. What should be the thickness of the polystyrene boards?

Solution - Table 7.3.2 or Fig. 7.3.2 may be used to solve the problem. According to Table 7.3.2, an *R*-value of 4 (0.7) is sufficient for an ambient temperature of 2 F (- 16 C), which will be assumed to be close enough to the expected ambient temperature. Since the plywood forms will provide some of this insulation, the required added insulation must have an *R-value* of 4 - 0.94 = 3.06 (0.70 - 0.17 = 0.53). Referring to Table 7.3.5, it is seen that a 1 in. (25 mm) plain polystyrene board has an R-value of 4 (0.7). Therefore, 3/4-in. (19 mm) thick polystyrene boards will provide the needed additional insulation.

7.4 - Enclosures

Enclosures are probably the most effective means of protection, but they are also the most expensive. The need for enclosures is dependent on the nature of the structure and the weather conditions (wind and snow). Experience has shown that they are generally required for placing operations when the air temperature is less than - 5 F (- 20 C). Builders attempting to place concrete at such low temperatures without enclosures will encounter manpower and equipment difficulties that will result in inferior constructions.

Enclosures block the wind, keep out the cold air, and conserve heat. They can be made with any suitable material such as wood, canvas, building board, or plastic sheet. Enclosures made with flexible materials are less expensive and easier to build and remove. Enclosures built with rigid materials are more effective in blocking wind and maintaining perimeter temperatures. Enclosures should be capable of withstanding wind and snow loads and be reasonably airtight. Sufficient space should be provided between the concrete and the enclosure to permit free circulation of the warmed air. Sufficient headroom should be provided, so that workers can work efficiently.

Heat can be supplied to the enclosures by live steam, forced hot air, or combustion heaters of various types. Although live steam heating provides an ideal curing environment, it offers less than ideal working conditions and can cause icing problems around the perimeter of the enclosure. Heaters and ducts should be positioned so as not to cause areas of overheating or drying of the concrete surface. During the protection period, concrete surfaces should not be exposed to air at more than 20 F (11 C) above the minimum placement temperatures given in Line 1 of Table 3.1 unless higher values are required by an accepted curing method.

Combustion heaters should be vented to prevent reaction of the carbon dioxide in the exhaust gases with the exposed surfaces of newly placed concrete. Carbon dioxide reacts with calcium hydroxide in the concrete and forms calcium carbonate (Kauer and Freeman 1955) resulting in a weak dusty surface.

Combustion heaters called vented heaters have a heat exchanger in the firebox so that none of the products of combustion are blown into the heated area. The exhaust gases are vented from the firebox to the outside atmosphere. Even though they use energy inefficiently compared to direct firebox heaters, vented heaters must be used to maintain concrete floor finish quality and for safety. The fact that a conventional combustion heater is located outside the enclosed concreting area and hot air is blown into the enclosure through ducts does not mean that the heater is vented, since the hot air may contain carbon dioxide.

Operation of combustion heaters should be supervised continuously, and adequate fire fighting equipment should be available at the job site at all times.

7.5 - Internal electric heating

Concrete can be heated internally by using embedded coiled and insulated electrical resistors. Low voltage current is passed through the coils, which are embedded near the surfaces of the section in a predetermined pattern. Internal concrete temperature may be raised to any required level by selecting the appropriate spacing or pitch of the coils. Gradual cooling may be controlled by intermittently interrupting the current passing through the coils. Heating usually begins after a presetting period of 4 to 5 hr depending on the setting characteristics of the concrete. Therefore, insulated forms are required only to prevent freezing of concrete during the presetting period and to minimize heat dissipation from surfaces where coils are not used. Moisture loss due to evaporation from unformed surfaces should be prevented by covering the surfaces with plastic sheets. Concrete temperatures must be monitored so that the recommended values are not exceeded.

7.6 - Covering after placement

Tarpaulins or other easily movable coverings, supported on sawhorses or on other framework, should follow closely the finishing of the concrete. The tarpaulins should be arranged so that heated air can circulate freely on both the top and, where exposed, the bottom of the slab. Layers of insulating material placed directly on the concrete are also effective in preventing freezing. This protection is pa:-titularly important in the case of lightweight structural concrete. The lower rate of heat diffusion of 1ightweight concrete allows more rapid freezing of surfaces to occur compared to normal weight concrete.

7.7 - Temporary removal of protection

Housing and enclosures should be left in place for the entire recommended protection period. Sections may be temporarily removed to permit placing additional forms or concrete, but scheduling of this work must insure that the previously placed concrete does not freeze. Sections that are removed should be replaced as soon as forms or concrete are in their final position. The time during which protection is temporarily removed should not be considered as part of the protection period. Time lost from the required period of protection should be made up with twice the number of lost degree-hours before protection is discontinued. For example, if protection was temporarily removed for 6 hr and the surface temperature dropped 15 F (8 C) below the minimum value given in Table 3.1, the deficiency in protection would be 90 F-hr (48 C-hr). Therefore, the protection period should be extended for 180 F-hr (96 C-hr).

7.8 - Insulated forms

When insulated forms are used in addition to heated enclosures, the interior temperature as well as the surface temperature of the concrete should be monitored to insure that the concrete is not heated more than is necessary. This recommendation applies particularly to mass concrete.

CHAPTER 8 - CURING REQUIREMENTS AND METHODS

8.1 - Introduction

Newly placed concrete must be protected from drying so that adequate hydration can occur. Normally, measures must be taken to prevent evaporation of moisture from concrete. During cold weather, when the air temperature is below 50 F (10 C), atmospheric conditions in most areas will not cause excessive drying. However, new concrete is vulnerable to freezing when it is in a critically saturated condition. Therefore, if concrete has been saturated during the protection period, it should be allowed to undergo some drying before being exposed to freezing temperatures.

8.2 - Curing during the protection period

Concrete exposed to cold weather is not likely to dry at an undesirable rate; however, this may not be true for concrete that is being protected from cold weather. As long as forms remain in place, concrete surfaces adjacent to the forms will retain adequate moisture. On the other hand, exposed horizontal surfaces, particularly finished floors, are prone to rapid drying in a heated enclosure.

When concrete that is warmer than 60 F (16 C) is exposed to air at 50 F (10 C) or higher, it is essential that measures be taken to prevent drying. The preferred technique is to use steam for both heating and preventing excessive evaporation. When dry heating is used, the concrete should be covered with an approved impervious material or a curing compound meeting the requirements of ASTM C 309, or it may be water cured. However, water curing is not recommended and is the least desirable method, since during periods of subfreezing temperatures it produces icing problems where water runs out of the enclosure or where there is a poor seal. It also increases the likelihood of the concrete freezing in a nearly saturated condition when protection is removed. If water or steam curing are used, they should be terminated 12 hr before the end of the temperature protection period, and the concrete should be permitted to dry prior to and during the period of gradual adjustment to ambient cold weather conditions, as discussed in Section 5.5.

When the air temperature within the enclosure has fallen to 50 F (10 C), the concrete can be exposed to the air provided the relative humidity is not less than 40 percent. During very cold weather, it is always necessary to add moisture to heated air to maintain this humidity. For example, if the outside temperature is 10 F (-12 C), the relative humidity of the air within the heated enclosure will be less than 20 percent if no moisture is added.

8.3 - Curing following the protection period

Following the removal of the temperature protection, it is usually not necessary to provide measures to prevent excessive drying as long as the air temperature remains below 50 F (10 C), except when concrete is placed in extremely arid regions. If a curing compound is applied during the first period of above-freezing temperature after protection is removed, the need to conduct further curing operations if the temperature should rise above 50 F (10 C) is eliminated.

The severity of drying is dependent on four factors: (1) the temperature of the concrete, (2) the temperature of the air, (3) the wind speed, and (4) the relative humidity of the air. For example, drying will be excessive if concrete at 50 F (20 C) is exposed to air having a temperature of 50 F (10 C) and a relative humidity less than 40 percent. As the air temperature decreases, the air requires a higher relative humidity to prevent excessive drying. For example, if concrete at 50 F (10 C) is exposed to air having a temperature of 40 F (5 C), the air must have a relative humidity greater than 60 percent to prevent excessive drying. However, when the concrete temperature has dropped to 40 F (5 C), an ambient air temperature of 40 F (5 C) with a relative humidity of 11 percent can be tolerated. For air temperatures above 50 F (10 C), the rate of drying increases rapidly.

If excessive drying is anticipated, concrete may be water cured when no freezing is expected. Otherwise, the use of curing compounds or an impervious cover is the preferred procedure. During cold weather periods when freezing occurs, occasional peak temperatures above 50 F (10 C) should not be a cause of concern. However, when temperatures above 50 F (10 C) occur during more than half of any 24-hr period for three consecutive days, the concrete should no longer be regarded as cold weather concrete and normal curing practice should apply (see ACI 308).

CHAPTER 9 - ACCELERATION OF SETTING AND STRENGTH DEVELOPMENT

9.1 - Introduction

If proper precautions are taken, accelerating admixtures, Type III cement (high early strength), or additional cement can be used to shorten the times needed to achieve setting and required strength. The reduction in setting time and the acceleration of strength gain often result in savings due to a shorter protection period, faster reuse of forms, earlier removal of shores, or less labor in finishing flatwork (see ACI 302). The strength development of concrete in massive structures, in which internal temperature rise is critical, should not be accelerated.

Methods used to obtain high early strength concrete increase the heat of hydration, which can be favorable in some instances. When resistance to sulfate attack is not a concern, cement having higher percentages of tricalcium silicate and tricalcium aluminate may be advantageous in cold weather because these compounds contribute to earlier strength development and higher heat of hydration. Cements of the same type and even of the same brand name, however, can have wide variations in their setting times and rates of strength development. Tests on concrete made with the cements to be used for a particular job are recommended to show which of the alternatives will produce the desired properties. Additional information on the subject of accelerating setting and strength development may be found in ACI 212.

9.2 - Calcium chloride as an accelerating admixture

9.2.1 - Calcium chloride is a popular and widely used accelerating admixture that reduces the setting time and increases the rate of early-age strength development. The use and effects of calcium chloride are discussed in ACI 212, ACI 201, and Shideler (1952). The maximum chloride ion content of concrete is prescribed in ACI 318, and these limits govern where the code is in use.

Calcium chloride should not be used under certain conditions; these restrictions are as follows:

a. Calcium chloride should not be used in prestressed concrete because of its potential for accelerating the rate of stress corrosion (Monfore and Verbeck 1960).

b. The presence of chlorides has been associated with galvanic corrosion between galvanized and plain steel when galvanizing is used on permanent forms for roof decks or embedded parts. Calcium chloride should not be used in such construction (Wright and Jencks 1963).

c. Galvanic corrosion of metals embedded in concrete is intensified by addition of calcium chloride to the concrete. Examples of this occurrence are as follows (Wright and Jencks 1963):

1. When aluminum and steel sheets embedded in concrete were connected externally, the galvanic corrosion of the aluminum was proportional to the concentration of calcium chloride.

2. Where large slabs of reinforced concrete were cast with aluminum conduit electrically connected to embedded steel, the results showed that calcium chloride promoted galvanic corrosion.

d. Calcium chloride should not be used where sulfate resisting concrete is required.

e. Calcium chloride has been found to increase the expansion caused by cement alkali-silica reaction (Shideler 1952).

f. Varying the calcium chloride content of concrete in a single structural unit because of changes in weather conditions may create concentration cells and, in the presence of moisture, may promote corrosion of reinforcing steel in the areas of higher chloride ion concentration. This practice is dangerous and should not be permitted in structures where continuity of reinforcement exists.

g. Even where a uniform dosage of calcium chloride has been used throughout a structural element, galvanic cells can develop within the reinforcing network due solely to differences in moisture contents. The relatively dry portions can act as cathodes, due to greater amounts of available oxygen, and the relatively damp portions can act as anodes where corrosion occurs. The degree of dampness required to support anodic activity has not been established definitively; it is believed that relative humidities above 70 percent may be sufficient (Tuutti 1982).

9.2.2 - Calcium chloride, and most other chemical admixtures, added to the concrete mixture in permissible quantities will not lower the freezing point of water in concrete to any significant degree. To avoid misplaced confidence in such a practice, and to avoid use of harmful materials, any attempt to lower the freezing point of the water in concrete by the use of calcium chloride should not be permitted.

9.3 - Other accelerating admixtures

9.3.1 - Some water-reducing accelerating admixtures, conforming to Type E in ASTM C 494, have been found to accelerate setting and strength gain at ambient temperatures of 50 F (10 C) and below, and also to reduce the required water content of the mixture. Some Type E admixtures contain small percentages of calcium chloride, usually amounting to less than 0.2 percent by weight of the cement when used at recommended dosage rates (see Section 9.3.2). This amount by itself has a limited effect on early strength development of the concrete. Tests indicate, however, that other Type E water-reducing accelerating admixtures, meeting the requirements of ASTM C 494, will substantially improve the 24-hr strength of concrete maintained at 50 F (10 C). The strength of concrete containing these admixtures approaches the strength obtained by using 2 percent calcium chloride and will be appreciably greater than for some, but not all, concretes made with Type III cements. The data also indicate that water-reducing accelerating admixtures may reduce setting time as well as produce substantial strength increases at all later ages.

9.3.2 - If adequate information or past performance records are not available, tests should be made to evaluate the effect of a pa-titular admixture or admixtures on the concrete for the job. These tests should be carried out at the expected job temperatures using the materials approved for the job. The manufacturers of many proprietary admixtures claim accelerated setting and strength developmen properties. It should be determined whether the admixture under consideration contains chloride. If chloride is present, the percentage of chloride, by weight of cement, that would be introduced into the concrete if the admixture were to be used should be determined and compared with the permissible limits given in ACI 201, or ACI 318. The admixture should not be used if the limits are exceeded.

Not all accelerating admixtures contain added chlorides. If job requirements prohibit the use of admixtures containing added chloride, noncorrosive accelerating admixtures are available

CHAPTER 10 - REFERENCES

10.1 - Recommended references

The documents of the various standards-producing organizations referenced in this report are listed with their serial designation, including year of adoption or revision. The documents listed were the latest effort at the time this report was revised. Since some of these documents are revised frequently, usually in minor detail only, the user of this report should check directly with the sponsoring group if it is desired to refer to the latest revision.

American Concrete Institute

201.2R-77	Guide to Durable Concrete
207.1R-70 (Reapproved 1980)	Mass Concrete for Dams and Other Massive Structures
211.1-81	Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
212.2R-81	Guide for Use of Admixtures in Concrete
301-84	Specifications for Structural Concrete for Buildings
302.1R-80	Guide for Concrete Floor and Slab Construction
306.1-87	Standard Specification for Cold Weather Concreting
308-81	Standard Practice for Curing Concrete
318-83	Building Code Requirements for Reinforced Concrete
347-78 (Reapproved 1984)	Recommended Practice for Concrete Formwork

ASTM

C 31-87a	Standard Practice for Making and Curing Concrete Test Specimens in the Field
C 39-86	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 192-81	Standard Method of Making Curing Concrete Test Specimens in the Laboratory
C 309-81	Standard Specification for Liquid Membrane-Forming Compounds for Curing Concrete
C 494-82	Standard Specification for Chemical Admixtures for Concrete
C 684-81	Standard Method of Making, Accelerated Curing, and Testing of Concrete Compression Test Specimens
C 803-82	Standard Test Method for Penetration Resistance of Hardened Concrete
C 873-85	Standard Test Method for Compressive Strength of Concrete Cylinders Cast in Place in Cylindrical Molds
C 900-87	Standard Test Method for Pullout Strength of Hardened Concrete
C 1064-86	Standard Test Method for Temperature of Freshly Mixed Portland-cement Concrete
C 1074-87	Standard Practice for Estimating Concrete Strength by the Maturity Method

These publications are available from the following organizations:

American Concrete Institute
P.O. Box 19150
Detroit, MI 48219-0150

ASTM
1916 Race Street
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This report was submitted to letter ballot of the committee and was approved in accordance with ACI balloting procedures.