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ABSTRACT

To illustrate some of the current practices being followed for winter concreting in Canada, the authors have studied three projects located in different areas of the country and involving various sectors of the industry. Methods used by contractors to protect and cure concrete under winter conditions have been described. The paper also discusses some of the techniques employed to ensure quality, such as the heating of the water and aggregates used in the mix, the on-site testing of the concrete mix and certain methods used in monitoring the concrete strengths. The basic principle used on all sites consisted of controlling the temperature of the concrete at the time of placing and for a certain period of time thereafter to ensure the safety of the structure. This practice followed fairly closely the requirements outlined in CSA Standards. The authors conclude that there is a need for a reliable, nondestructive method of on-site testing to monitor strength development to determine when the minimum required strength has been reached and ensure that the concrete is not cured for a period shorter than required or longer than necessary. This would eliminate the present uncertainty and possibly reduce construction time.

LE BÉTONNAGE PENDANT L'HIVER AU CANADA

par R.G. Turenne et P.J. Sereda

RÉSUMÉ

Afin de montrer quelques-unes des méthodes utilisées au Canada pour le bétonnage pendant l'hiver, les auteurs ont visité trois chantiers, dans différentes régions du pays, mettant en jeu divers secteurs de l'industrie. Les méthodes employées par les entrepreneurs pour la protection et la prise du béton dans des conditions hivernales sont décrites. Le présent article examine également quelques-unes des techniques assurant la qualité, par exemple le chauffage de l'eau et des agrégats utilisés dans le mélange, les essais in situ du mélange de béton et certaines méthodes d'observation continue de la résistance du béton. Le principe de base pour tous les chantiers est la régulation de la température du béton lors de la mise en place et quelque temps après en vue d'assurer la sécurité de la construction. Cette méthode suit d'assez près les exigences des normes CSA. Les auteurs concluent qu'il faudrait un procédé fiable et non destructif de mise à l'essai in situ, permettant de surveiller l'augmentation de la résistance, de déterminer à quel moment la résistance minimale souhaiteé est atteinte et d'assurer que la période de prise du béton n'est ni trop courte ni trop longue. Cela éliminerait l'incertitude qui existe actuellement et réduirait peut-être la durée des travaux.

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by

R.G. Turenne and P.J. Sereda

The degree of severity of winter climate influences the techniques used for winter concreting. Although reported in 1956, the climatic information presented by Swenson[1] is still valid for this present paper. A diagrammatic representation of the severity of the Canadian winter is presented in Fig. 1, which shows isolines of the freezing index in degree-days. (The freezing index is defined as the cumulative total of the monthly net freezing and thawing degree-days during winter; a degree-day is the unit of measurement of the difference between the daily mean temperature and the freezing point of water.[2])

The social, economic and technical reasons for maintaining construction in year-round activity are well known and hence winter concreting is a common necessity. At present, winter concreting is carried on in Canada as a regular practice whenever required by the circumstances of a project.[3] The problems of winter concreting alone are not considered reason enough to delay a construction project even when the requirements of the finished concrete are as demanding as they were in the case of the CN Communication Tower (one of the cases reported herein).

The methods used by Canadian builders to protect and cure concrete placed in freezing weather differ for different types of jobs and for different locations. The three Canadian projects selected as examples are considered representative of types of construction and climatic conditions. Their locations are shown in Fig. 1.

Three general contractors agreed to provide details of their winter concreting practices on projects just completed or in process of construction. The co-operation received not only from contractors, but also from designers, owners, and testing laboratories, is gratefully acknowledged.

FERMONT PROJECT

The decision of the Quebec Cartier Mining Company to work the iron ore deposits near Mount Wright in Northern Quebec also entailed the construction of a townsite for the firm's employees. The future population of the new town of Fermont was estimated at 5,000; accommodation would consist of single-family dwellings, duplexes, townhouses and 3 1/2- and 5-storey apartment buildings. The designers joined all the apartment buildings together to form a windscreen, 600 m long, along the northeast edge of the town (Fig. 2).

The region has a freezing index of 2500 degree-days and its mean January temperature is -20°C. Because of the severe climate, the construction schedule, as originally drawn up, eliminated winter concreting. The early stages of the project were not suited to winterwork and it was thought that the cost of hoarding and heating concrete over such a large project would be extremely high. Delays caused the schedule to be modified, however, with the result that some winter concreting became necessary. It was restricted to the construction of the apartment buildings and confined to two principal activities:

(a) placing the 6-cm-thick elevated concrete slabs within the 5-storey section;

(b) construction of the foundations for the 3 1/2-storey wood-frame portion.

Oil was selected as the source of heat because it was readily available locally. (Gas would have to be brought in by rail from Seven Islands, some 400 km to the south, and deliveries had already proved unreliable; electric heating was impractical as the distribution system was still under construction.) Accordingly, some 40 heating units in sizes ranging from 300 to 500 MJ were used.

The exterior walls of the 5-storey apartment buildings were not to be erected until all the interior floors had been concreted as they would transmit any horizontal wind pressures to the columns. Permission was given by the designers, however, to erect the walls after temporary cross-bracings were installed at all floor levels. The exterior walls thus provided an enclosure to work in. As it would be expensive to heat the entire building for the sole purpose of concreting and finishing relatively small floor areas, heating was limited to the required areas by suspending tarpaulins around the perimeter of the floor to be concreted and placing heaters on the floor below (Fig. 3). The heat was thus concentrated below the slab; the light-gauge metal pans supporting the fresh concrete offered no resistance to the flow of heat through the slab. The concrete floors were finished, then cured for seven days at a minimum temperature of 20°C. To reduce labour costs, the heaters were all connected to central oil reservoirs.

The construction of the foundations for the 3 1/2-storey wood-frame apartments posed a different problem. A complete temporary enclosure was required if proper conditions were to be maintained for all the various operations including footings, basement walls, columns and structural floor slabs and beams. A hoarding was therefore designed to take advantage of the materials on site which would eventually be incorporated in the superstructure of the building (Fig. 4). The wood trusses that supported the roof provided a clear span with lean-to's added on either side to give the required width. Beams and posts were laminated from 4- by 14-cm lumber; floor joists were used for the lean-to roofs. The trusses were assembled into panels and covered with polyethylene prior to being hoisted into place. The enclosure performed satisfactorily; it even withstood, without damage, winds gusting to 100 km/hr (Fig. 5).

All materials for concreting operations were proportioned in a central batching plant and transported to the construction site in truck-mixers. The water, sand and aggregate were heated. All concrete was placed by means of a truck-mounted concrete pump. The truck was equipped with a boom which facilitated the operation. The hose through which the concrete was pumped was completely wrapped in insulation to protect the fresh concrete from cold temperatures. Although the contractor had certain reservations about using a concrete pump under severe winter conditions, this experience proved to him that pumping is in fact an ideal method of placing concrete during cold weather.

In the winter concreting mix, 320 kg of cement per cubic metre of concrete were used with 1% calcium chloride. Between 4 and 6% air entrainment was used depending on the exposure. All concrete was cured for seven days at a temperature of 16°C or higher. Quality control of the concreting operation was assured by a group of qualified technicians. Concrete strengths were determined from site- and laboratory-cured cylinders.

EDMONTON CENTRE

The City of Edmonton, located in Western Canada, is the most northerly of all major Canadian cities; it is also one of the coldest. Edmonton has a freezing index of 1425 degree-days and a mean January temperature of -15°C. Until recently, outside construction work practically stopped during the winter months in Western Canada. However, as projects became larger and more complex and completion dates less flexible, techniques were refined and most builders now face the prospect of winter concreting with confidence, both from the point of view of cost and of quality. A good example of this attitude is found in the construction of the second office tower of the Edmonton Centre complex.

This office tower, 29 storeys high, measures 45.36 m by 30.35 m. The structure consists of a steel frame with interior support provided by a concrete core which contains all the services, e.g., elevator shafts, stairwells, washrooms and duct spaces. The concrete core rises 124 m above the basement floor and measures 26.26 m by 9.5 m at its base. The wall thickness varies from 81.28 cm at the base to 20.32 cm at the top and the specified concrete strength from 34.5 to 27.6 MPa. The schedule called for the steel to be erected in three stages; the construction of the core naturally had to precede these stages.

Work started on the core in October 1974 and its walls were concreted at the rate of one storey per week. The schedule called for the placing of concrete every Thursday, thus providing close to four days of heating and curing as the exterior forms were not stripped until the following Monday. The concrete was then exposed to the ambient temperature without protection.

The forms for the core walls were of metal. Because of the decreasing wall thickness, slipforming was impractical so the flying form method was adopted. For winter concreting operations, the exterior form was sprayed with polyurethane foam insulation to a thickness of 4 cm to minimize heat losses. The 3.66-m-high forms complete with scaffolds at various levels were supported by bolts cast into the previously placed concrete. The workmen were protected from cold weather and winds by tarpaulins that enclosed all the outside forms (Fig. 6).

A platform which completely covered the area bounded by the walls was supported by the interior forms. When the exterior forms were raised to their new position, this platform acted as a floor for the workmen, e.g., when placing the reinforcing steel (Fig. 7). When the interior forms were raised in readiness to place a new lift, the platform served both as a floor while placing concrete and as the top of the hoarding for the purpose of heating and curing the concrete walls. Heaters placed on small panels suspended from the main platform burned natural gas. These heaters, rated at 380 MJ, were hooked onto a steel pipe, installed inside the stairwell, which carried the natural gas to the top of the core. This pipe was extended in 3.66-m increments as each new storey was completed. A special cap fitted with six connections and valves was attached to the top end of the pipe. Rubber hoses were used to connect the heaters to the supply line. The temperature within the enclosure was maintained at 18°C for the four days of heating and curing.

Floors for the three lower levels, which housed retail stores, were of reinforced concrete. The slab thickness varied between 15 and 20 cm. Plywood was used to form the slab, columns and beams. Heaters were placed below the slab to be cured and the area enclosed with polyethylene and canvas. These heaters, which also burned natural gas, were larger than those used for the core; they were rated at 1950 MJ. The temperature within the enclosure was maintained at about 18°C. The slabs were not required to be trowel finished, they were merely screeded to the correct elevation and given a light wood float finish. As soon as the concrete had hardened sufficiently, the floors were covered with a 5-cm-thick insulating blanket with the polyethylene cover acting as a curing membrane (Fig. 8). Although these structural slabs were heated for seven days, stripping of the formwork generally started on the fourth day. The specified concrete strength for the floors was 20 MPa. Columns were not heated but merely wrapped with insulation.

Concrete was supplied by a firm whose ready-mix plant was located a short distance away (a 10- to 15-minute drive). All concrete delivered during the late autumn and winter was heated. The temperature of the water, sand and aggregate was controlled to give a mixing temperature of 27°C. The water and coarse aggregate were heated to about 24°C; the temperature of the sand varied between 27 and 38°C, depending on the ambient temperature. As a result, the temperature of the concrete delivered at the site was generally about 18°C.

The concrete mix was designed to give a 28-day strength some 15% greater than the specified strength. This practice ensured that the 28-day strength would not fall below the design strength even allowing for a 10 to 12% deviation. Usually, two additives were used: an air-entraining agent and a water-reducing admixture. Calcium chloride was also added to the mix occasionally at the request of the project superintendent when a high early strength was required. The amount varied between 1/2 and 2% in increments of 1/2%. Adding water to the concrete at the site was discouraged both by the concrete supplier and the contractor. When water had to be added to ensure adequate workability, the plant was notified immediately by two-way radio and the delivery slip signed by the superintendent.

As a rule, the plant monitored the strength of the various mixes daily by taking samples in order to make a complete set of tests (slump and compression using three cylinders) for every different mix supplied on any given day. One cylinder was tested at 7 days and two at 28 days.

All concrete testing at the job site was done by personnel of an independent testing laboratory. A complete set of tests included slump, air content, concrete temperature, and unit weight, as well as the making of three cylinders. The three cylinders were left at the site for 24 h before being taken to the laboratory for curing. One cylinder was broken after 7 days and two after 28 days.

Table I shows the data obtained from the concrete test reports submitted to the general contractor by the testing laboratory. These results are for eight different concrete batches placed during the months of November and December 1974. The specified concrete strength was 34.5 MPa in all cases.

The average 7-day strength of all eight cylinders is 30.12 MPa or 87.3% of the specified 28-day strength and the deviation is only 3%. The average 28-day strength on the other hand is 39.76 MPa, which is very close to the mix design strength of 39.67 MPa. The maximum deviation between cylinder strength and mix design strength is approximately 6%. These results indicate good quality control on the part of the concrete supplier.

In accord with usual Canadian concreting practice, no attempt was made to determine the in-place strength of the concrete by testing cored samples or by non-destructive test methods. It can be determined from laboratory-cured cylinders if the right concrete mix was used in the construction. If this proves to be the case, it is assumed that, providing the fresh concrete is given adequate protection for a certain period of time, it will eventually develop approximately the same strength as the test cylinders. The experience of builders in Canada bears this out; most failures involving reinforced concrete structures experienced in this country have resulted from inadequate curing immediately following the placing of the concrete. It is obvious that, from a practical point of view, the first three or four days of curing are critical if the contractor is to strip and re-use the forms according to a predetermined schedule. The only way this can be done with confidence is by providing an environment conducive to rapid curing.

It should also be noted that very little calcium chloride was added to the concrete. The practice on the project was to add calcium chloride only when high early strengths were required (when stripping of forms had to proceed within 24 h, for example, or where anchors were cast in the concrete which might have to be loaded very early). Otherwise, it was believed that the 4 to 7 days of curing were sufficient for the concrete to develop adequate strength without the use of calcium chloride.

CN COMMUNICATION TOWER, TORONTO

Because of its geographical location, the City of Toronto enjoys a relatively mild winter. The mean January temperature is -5°C. The temperature occasionally falls to -20°C or climbs to 10°C during the month. Winds are normally light, averaging 17 km/h. They seldom exceed 65 km/h with gusts of 100 km/h.

The CN Tower in downtown Toronto (Fig. 9) consists of a hexagonal core 11 m in diameter with 61-cm-thick concrete walls rising 446 m above the foundation.

The core is supported by three buttresses projecting radially 120° apart. These buttresses measure 27.4 m by 7 m at the base; the 1.5-m-thick end walls taper inwards until they intersect the main core at the 342-m level. Side and intermediate walls of the wings are 46 cm thick. Sleeves to receive the tensioning cables were cast in the core walls and in the intermediate and end walls of the wings. The tower was post-tensioned at the 55-, 170-, 342-, 417- and 451-m levels.

The tower, including the wing walls, was constructed by the slipform method. Casting began in late June 1973, and was completed in February 1974. Initially, it had been planned to complete the placing of the concrete by the end of November, thus avoiding winter concreting. Progress was slowed down by various factors, however, including the use of slower-setting Type-40 low-heat cement which was used exclusively for the first 122 m. Thereafter, the low-heat cement was replaced by normal portland cement Type 10 in 10% increments as the weather got progressively colder. The last 12% of the volume of concrete used only normal portland cement.

When it was decided to continue with the construction of the tower during the months of December, January and February, modifications had to be made to the slipform to provide heat for curing the concrete as well as for the comfort of the workmen. The specific requirements of the protection system were:

(a) to allow the concrete to cure at an elevated temperature for a period of about 4 days in order to attain a strength of about 60% of final when concreting was proceeding at normal rate; and

(b) to prevent an extreme change in temperature in the concrete when it was exposed to outdoor conditions.

If slipforming were to proceed at the rate of 3.65 m per day, the second requirement would be satisfied by suspending an 11-m-high insulated skirt from the slipform. This skirt, shown in Fig. 10, was constructed as follows: top section, 6 m high, of 2.5-cm-thick rigid polystyrene sandwiched between 2 sheets of plywood; middle section, 2.5 m high, of 1.25-cm-thick sheet of polystyrene between two sheets of plywood; bottom section, 2.5 m high, of plywood. This provided adequate protection for initial strength gain. It also allowed the concrete to cool sufficiently before being exposed to the atmosphere to avoid temperature gradients severe enough to cause cracking (Fig. 11). Some heat was provided to the interior of the core by a 7,000 MJ heater installed at the base of the tower.

The specifications called for a minimum concrete strength of 34.5 MPa at 28 days. The following concrete mix proportions were selected:

	kg/m^3	
Cement	368	
Coarse aggregate	1127	(19.5 mm crushed dolomitic limestone)
Fine sand	712	(natural sand)
Water	154	

No additives were used except for an air-entraining agent and a water-reducing admixture. The water/cement ratio was 0.45 and the specified slump 100 mm ±12 mm. Air content was 5% ±1.

A batching plant was assembled on the site and all concrete used for the foundations (7,646 m^3) and the tower (30,584 m^3) was mixed at the job site. As a result, the engineers had excellent control over all the various phases of the concreting operation, including proportioning of the ingredients, mixing time, and the time required for placing the concrete.

As the ambient temperature decreased, the water used in the concrete was heated, and eventually the sand and aggregate had to be heated as well. Maximum water temperature was set at 57°C; that of the sand was set at 55°C. Although it was relatively easy to maintain a fairly uniform temperature in the sand, steam heating the larger aggregate produced hot cone-shaped patches surrounded by cold aggregate. The heated aggregate therefore had to be moved to another pile where it was thoroughly mixed and left to reach an even temperature. An in-place concrete temperature of about 32°C was aimed for as observations had shown that the concrete gained strength faster when placed at that temperature than at, say, 20°C. To prevent excessive loss of heat during the placing of concrete, all the concrete was hoisted inside the core so that the bucket was not exposed to the cold air and wind on its way to the top. The conveyor belt which transported the concrete from the trucks to the bucket was heated by infrared lamps.

A fully equipped testing laboratory set up on the site was staffed 24 h a day during slipforming operations. Four qualified technicians assured quality control. Concrete strengths were determined from cylinders measuring 15 cm by 30 cm. Five cylinders were cast for every 57 m^3 of concrete. Two were subjected to accelerated testing after 48 h using the autogenous method developed by Smith and Tiede.[4] The other three cylinders were cured by standard laboratory fog room methods. One was tested at 7 days, two at 28 days and one at 90 days. The coefficient of variation of all cylinders tested at 28 days based on a mean strength of 47.6 MPa was 6.1.

With a planned rate of rise of 3.65 m per day, the 2-day accelerated tests did not enable the resident engineer to monitor concrete strengths as they developed. It was therefore decided to adopt the maturity testing concept along with the 2-day accelerated tests. A strength maturity curve for the concrete mix in use was established. Thermocouples were installed in the concrete during each shift (1.2 m vertical spacing). The thermocouple wires were coiled on free-turning reels attached to the slipform framework so that the wires unwound as the form rose and readings could be taken at regular intervals within the enclosure. Individual thermocouple readings were plotted against the previously established strength-maturity curve as a function of time and temperature so that the in-place strength of the concrete could be predicted as it developed (Fig. 12). Verification of the in-place strength and of the maturity concept was obtained from the testing of concrete cylinders cored out of the tower walls.

These two testing methods, accelerated curing and strength maturity measurement, demonstrated the adequacy of the concrete and thus the safety of the structure. The concreting crew, however, could not wait for these results to determine the safe rate of concrete placing at any given time. It was necessary to determine quickly and accurately if the 8-h-old concrete was hard and strong enough to be slipped out of the forms; neither testing method could provide this information. As a result, a very simple test was devised which proved reliable. It consisted in having a man push a smooth steel bar 1 cm in diameter into the fresh concrete and measure the depth of penetration to the hardened concrete. The bar bore a mark 1.37 m up from the bottom end corresponding to the over-all depth of the slipform. The depth of penetration was obtained by measuring the vertical distance between the top of the form and the mark on the bar. When the measured length was less than 60 cm, the rate of pour was accelerated; when the length was greater than 60 cm, the rate was slowed down.

The rate of hardening of the concrete was affected by the temperature within the enclosure. Although the contractor tried to maintain the temperature at about 20°C, excessive air leakage during high wind conditions sometimes caused the temperature to drop considerably. Operations usually had to be suspended when the inside temperature approached the freezing point as the concrete was no longer setting properly. The winds that caused such drops in temperature were usually so strong as to make the use of the tower crane hazardous. As no more than a one-hour supply of reinforcing steel could be stock-piled in the working area atop the tower at any time, concreting operations usually had to stop.

The core was topped on 22 February 1974; still to be added was the 4.88-m-thick slab to receive the steel antenna. Although the extra cost in terms of protection, heating and reduced productivity because of a slower rate of pour was high, this winter concreting operation was very successful, and prevented the undesirable effects that a three- or four-month shutdown might have had on the project's ultimate completion.

GENERAL DISCUSSION AND CONCLUSIONS

The three projects described in this paper present evidence that, in Canada, winter concreting is carried out successfully on all types of construction from housing units to large structures such as the CN Tower.

The survey showed that the methods to protect concreting in cold weather differ on different types of construction; however, all the methods follow one basic principle, i.e., that the temperature of the concrete at the time of placing and for a period thereafter is controlled to allow the development of the minimum strength required to ensure the safety of the structure.

Canadian concreting practice is outlined in standards of the Canadian Standards Association.[5] Particular emphasis is placed on the temperature of the mix and the protection requirements. Tables II and III are based on this standard. Canadian Standards [6] also provide for the addition of accelerating agents such as calcium chloride by up to 2% by weight of cement in the concrete mix. This is normally at the discretion of the engineer in charge. The use of calcium chloride is not allowed in prestressed concrete nor in sulfate-resisting concrete. Only two of the three cases discussed in this paper made use of an accelerating agent and in amounts less than the limit set by the standard.

Other useful sources of information are the publications of the American Concrete Institute, e.g., refs. 7 and 8.

As there have been only few structural failures resulting from winter concreting in the past 25 years, it can be concluded that present practice is adequate to ensure good results. Nevertheless, it will not be possible to assess accurately the level of safety attained or whether these practices are excessive until such time as non-destructive methods of testing of strength development have been adopted for field inspection. Current testing of concrete cylinders ensures only that the concrete has the <u>potential</u> to develop the design strength and not that it <u>will</u> in a particular situation. Experience of contractors, however, gives confidence to the belief that concrete which is given the minimum required protection in the first few days will eventually develop the strength indicated by these tests.

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REFERENCES

1 Swenson, E.G. Weather in relation to winter concreting. Proceedings of the RILEM Symposium on Winter Concreting, Theory and Practice, Copenhagen, 1956.

2 Boyd, D.W. Normal freezing and thawing degree-days for Canada 1931-1960. Canada, Dept of the Atmospheric Environment Service, CLI 4-73.

3 Crocker, C.R. Winter construction in Canada. Build International, Vol. 4, No. 6, Nov-Dec 1971, p. 326-330.

4 Smith, P., and Tiede, H. Earlier determination of concrete strength potential. Highway Research Board, HRB Record No. 210, Washington, D.C., 1967, p. 29-66.

5 Canadian Standards Association, CSA Standard A23.1-1973. Concrete materials and methods of concrete construction. (Tables 12 and 13).

6 Canadian Standards Association, CSA Standard A266.4, Guidelines for the use of admixtures in concrete.

7 American Concrete Institute, ACI Standard 306-66. Recommended practice for cold weather concreting (reaffirmed 1972).

8 American Concrete Institute. Behavior of concrete under temperature extremes. ACI Publication SP-39, 1973.

TABLE I

RESULTS OF CONCRETE TESTS

Slump, cm	Air entrained, %	Unit wt., kg/m^3	Temperature, °C		Concrete strength, MPa			Calcium chloride, % (by wt. of cement)
			Mix	Air	7 days	28 days	28 days	
7.6	2.8	2335	19.5	-0.5	30.77	not available		1/2
6.3	2.5	2345	20.0	1.0	29.19	39.81	38.78	1/2
7.0	2.6	2313	21.0	-4.5	30.15	40.50	40.36	-
10.2	3.0	2307	21.0	0	30.02	40.64	41.74	1/2
8.9	4.5	2307	18.0	0.5	29.88	39.05	37.33	-
7.6	2.8	2307	18.0	-6.0	30.84	41.88	40.85	-
8.9	3.2	2326	18.0	1.0	29.81	39.54	40.50	-
10.2	3.7	2323	12.0	0	29.39	38.09	37.60	-

TABLE II

CONCRETE TEMPERATURES
FOR COLD WEATHER CONSTRUCTION

	Temperatures	
Least Dimension of Section	Concrete as Placed, Minimum/Maximum,* Degrees C	Maximum Allowable Gradual Drop of Concrete Temperature For Each 24-hr period at Termination of Protection Degrees C
Less than 0.30 m	13/32	28
0.30 to 0.90 m	10/32	22
0.90 to 1.80 m	7/27	17
Greater than 1.80 m	4.5/21	11

*Concrete temperature at time of mixing shall not exceed the maximum shown in Table. Concrete temperature at time of placing should be kept as close as possible to the minimum.

TABLE III

PROTECTION AND CURING REQUIREMENTS
FOR COLD WEATHER CONSTRUCTION

Outside Air Temperature During Protection Period	Least Dimension of Section	
	Less than 0.90 m	Greater than 0.90 m
Below -12°C	see Note 1	see Note 2
From -12°C to -1°C	see Note 2	see Note 3
From -1°C to 10°C	see Note 3	see Note 4

Notes

1. Complete housing plus supplementary heat or adequate insulation.

2. Enclose with suitable covering plus supplementary heat or adequate insulation.

3. Enclose with suitable covering with supplementary heat in readiness or adequate insulation

4. Protection by suitable covering or adequate insulation.

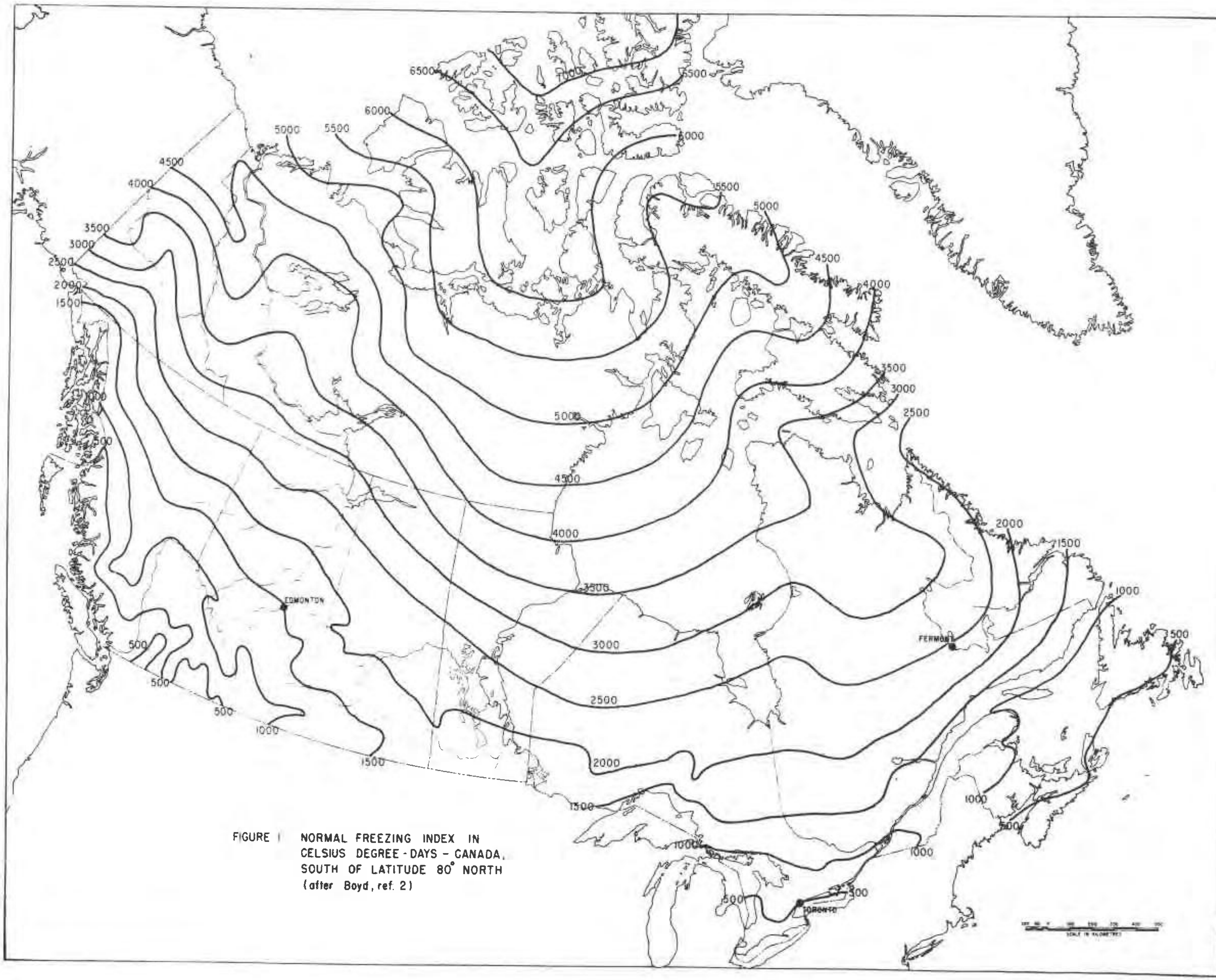

FIGURE 1 NORMAL FREEZING INDEX IN CELSIUS DEGREE - DAYS - CANADA, SOUTH OF LATITUDE 80° NORTH (after Boyd, ref. 2)

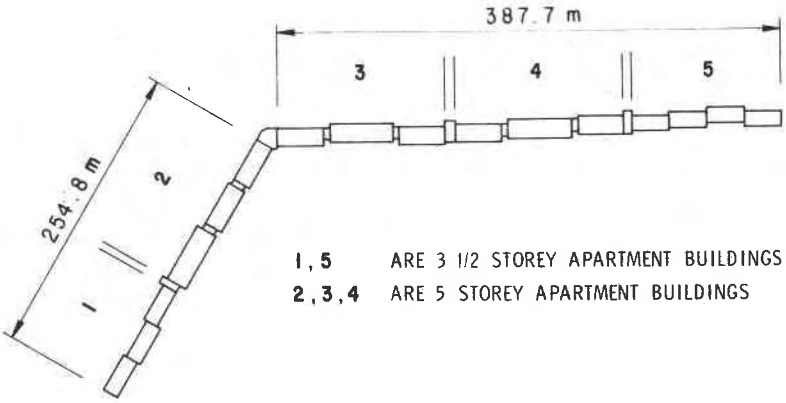

FIGURE 2

WINDSCREEN LAYOUT, FERMONT PROJECT

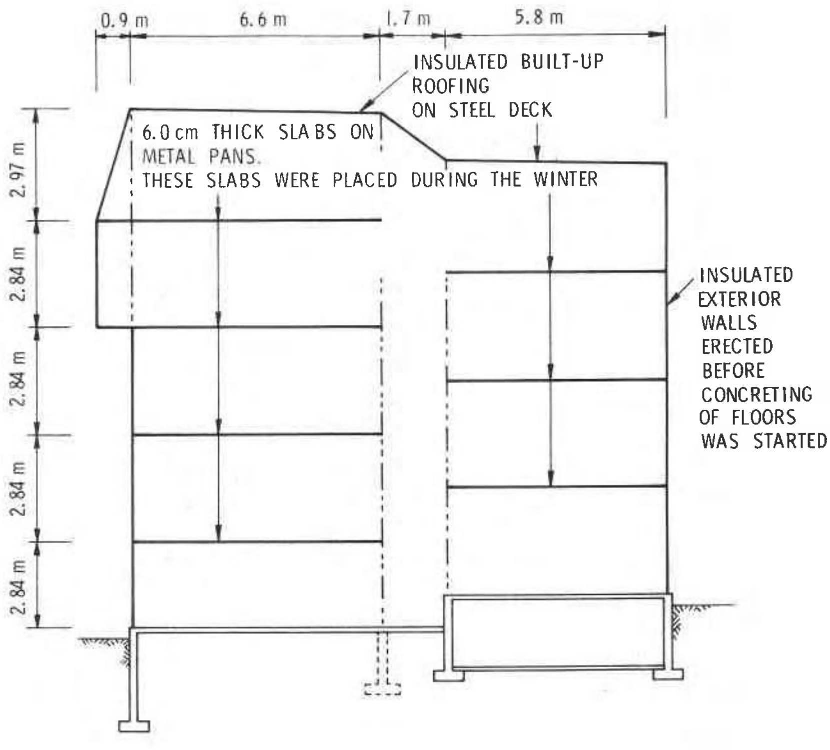

FIGURE 3

SECTION THROUGH 5 STOREY WINDSCREEN BUILDING, FERMONT PROJECT

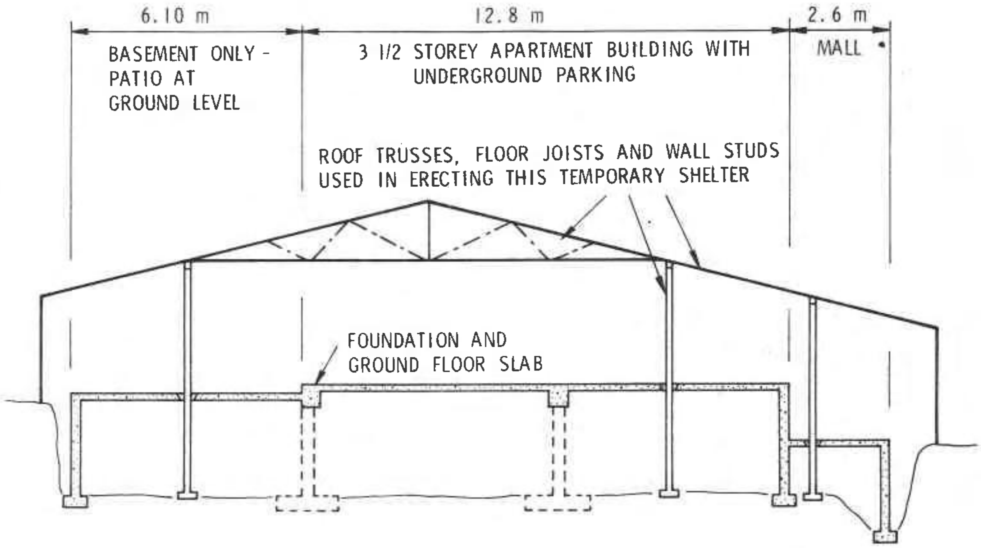

FIGURE 4

CROSS-SECTION THROUGH FOUNDATION OF 3 1/2 STOREY WINDSCREEN BUILDING SHOWING OUTLINE OF TEMPORARY SHELTER, FERMONT PROJECT

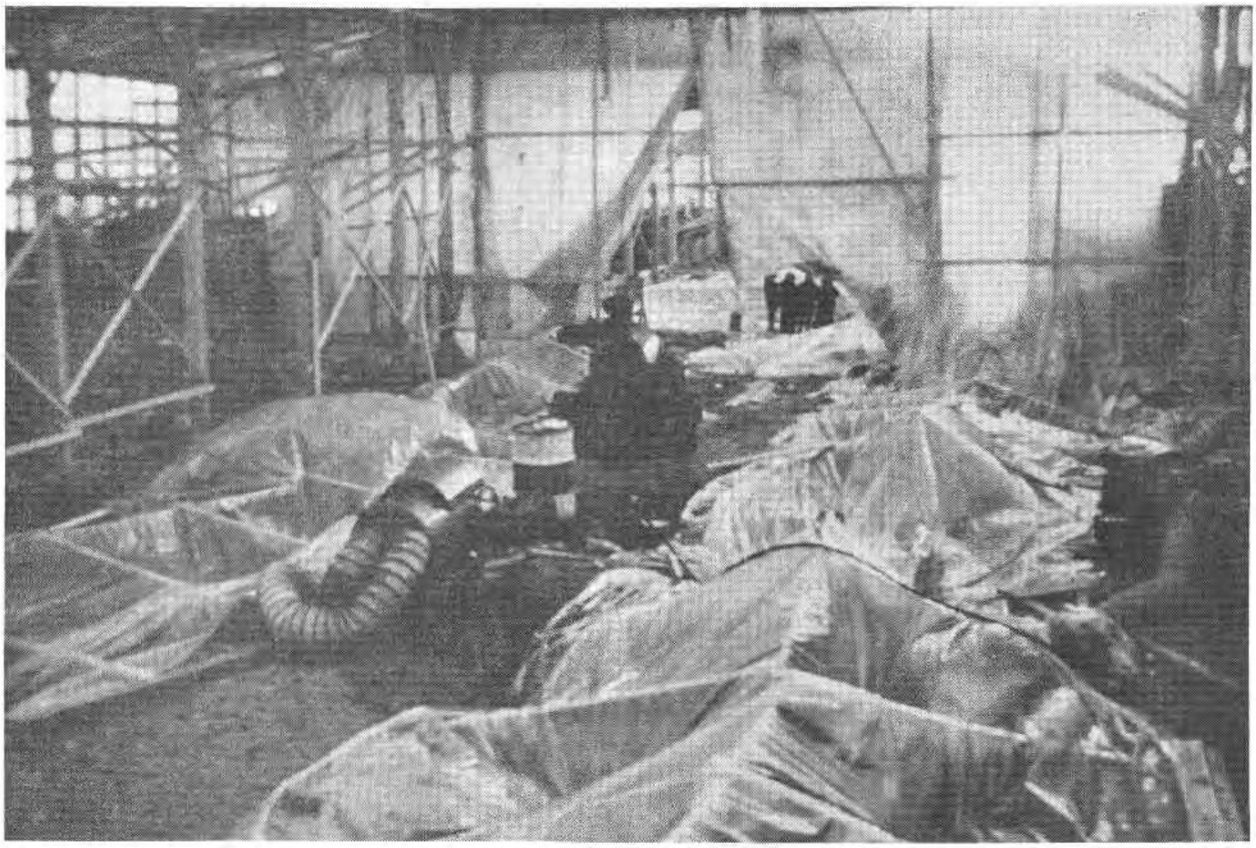

FIG. 5 Temporary enclosure, Fermont project (courtesy Pentagon Construction (1969) Co. Ltd.)

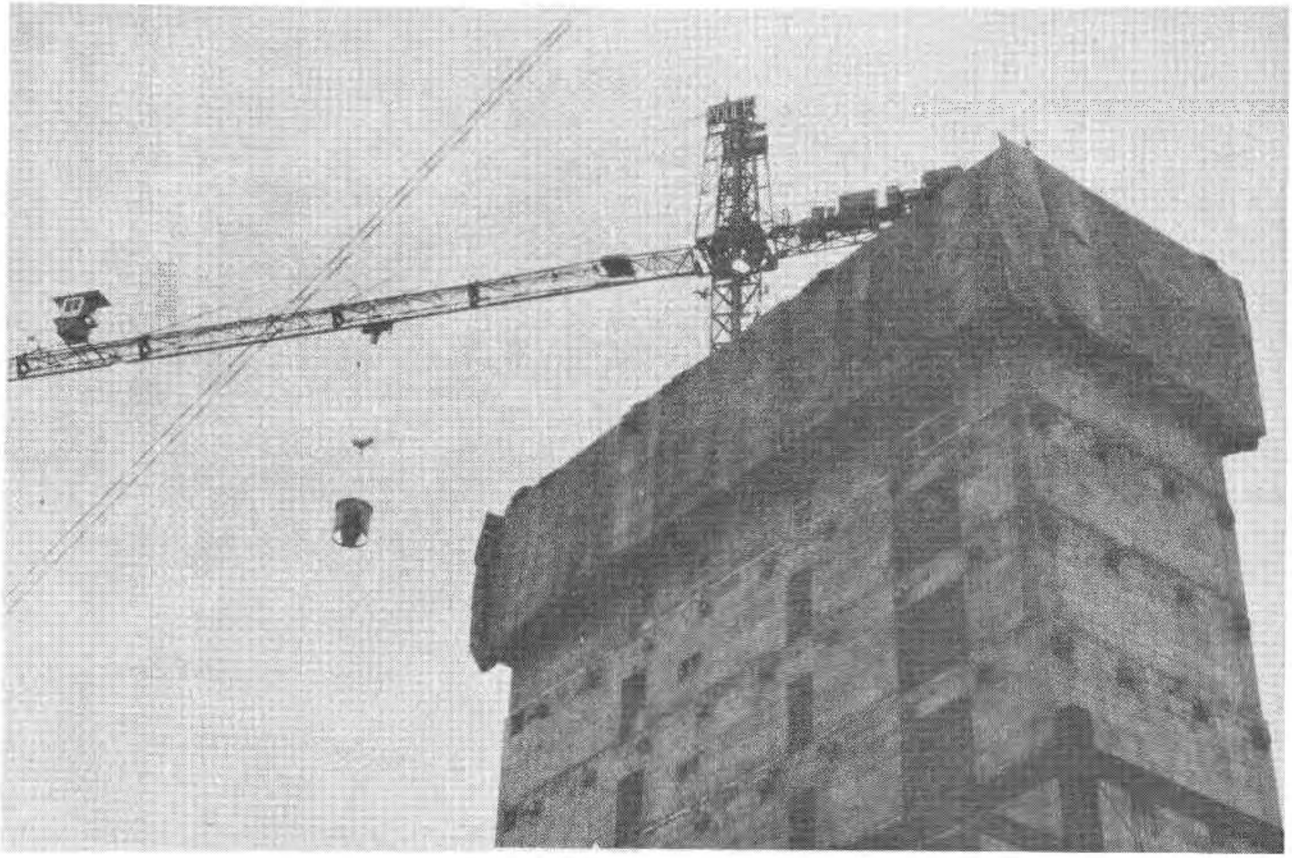

FIG. 6 Photograph of concrete core showing protection around formwork, Edmonton Center

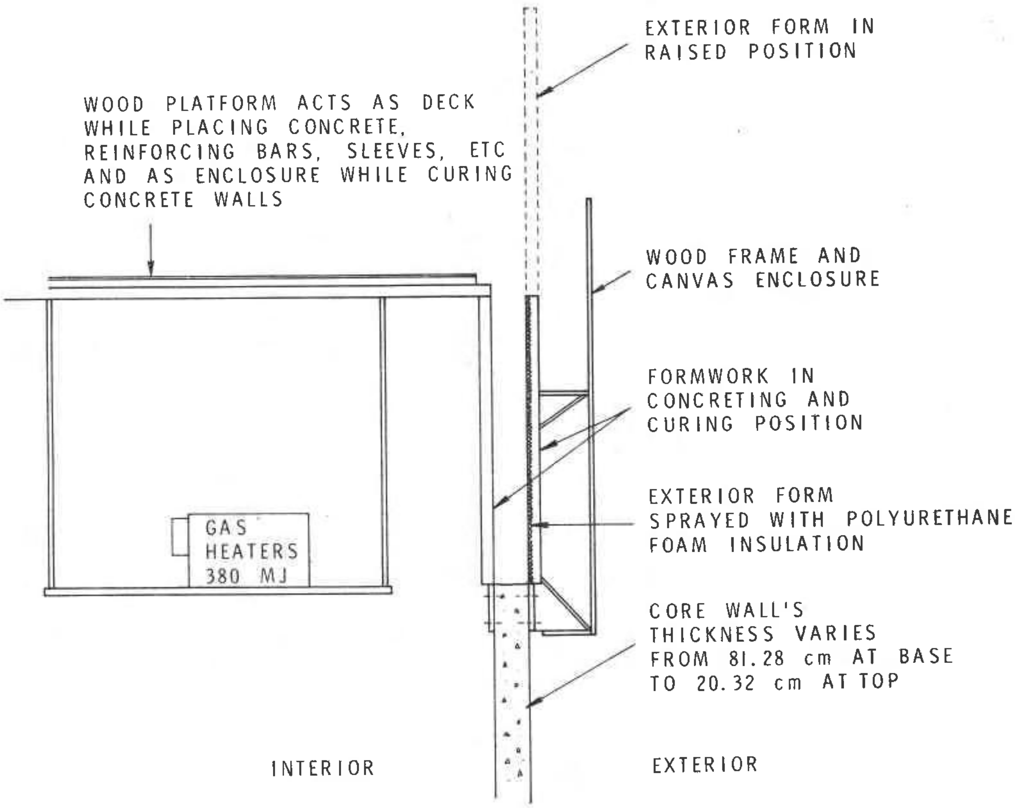

FIGURE 7

TYPICAL SECTION THROUGH EXTERIOR WALL AND FORMWORK OF CONCRETE CORE. EDMONTON CENTER

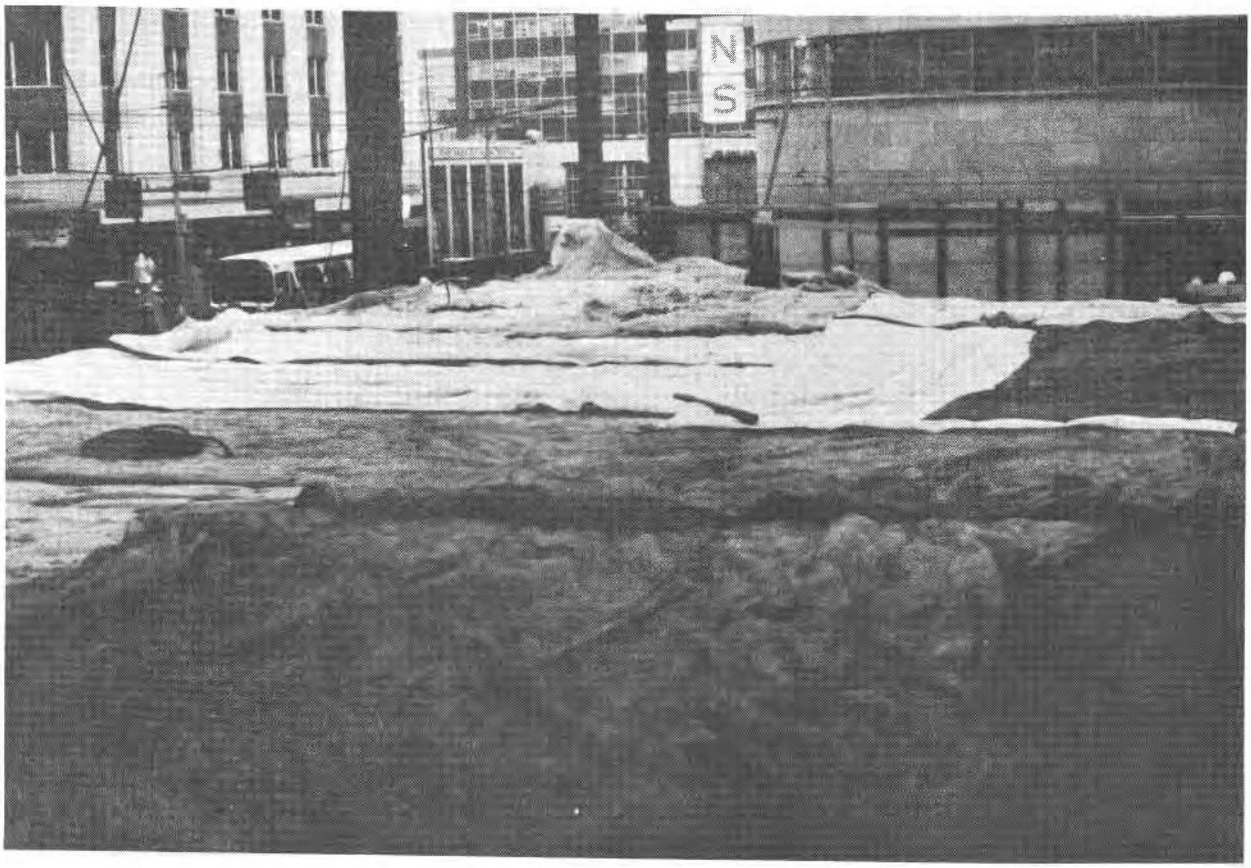

FIG. 8 Insulating blankets protecting newly placed concrete floor, Edmonton Centre

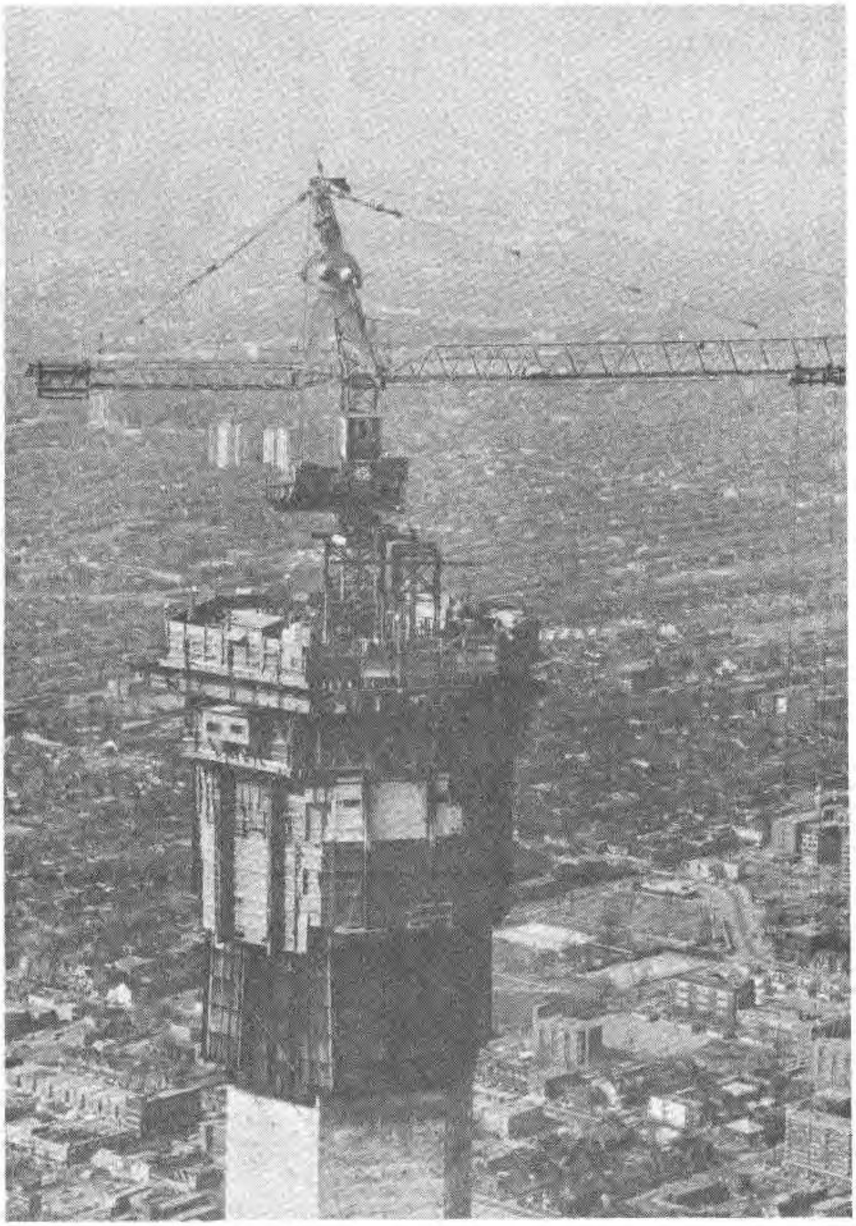

FIG. 9 View of tower showing winter protection and working area at the top of the slipform, CN Communication Tower (Courtesy CN Tower)

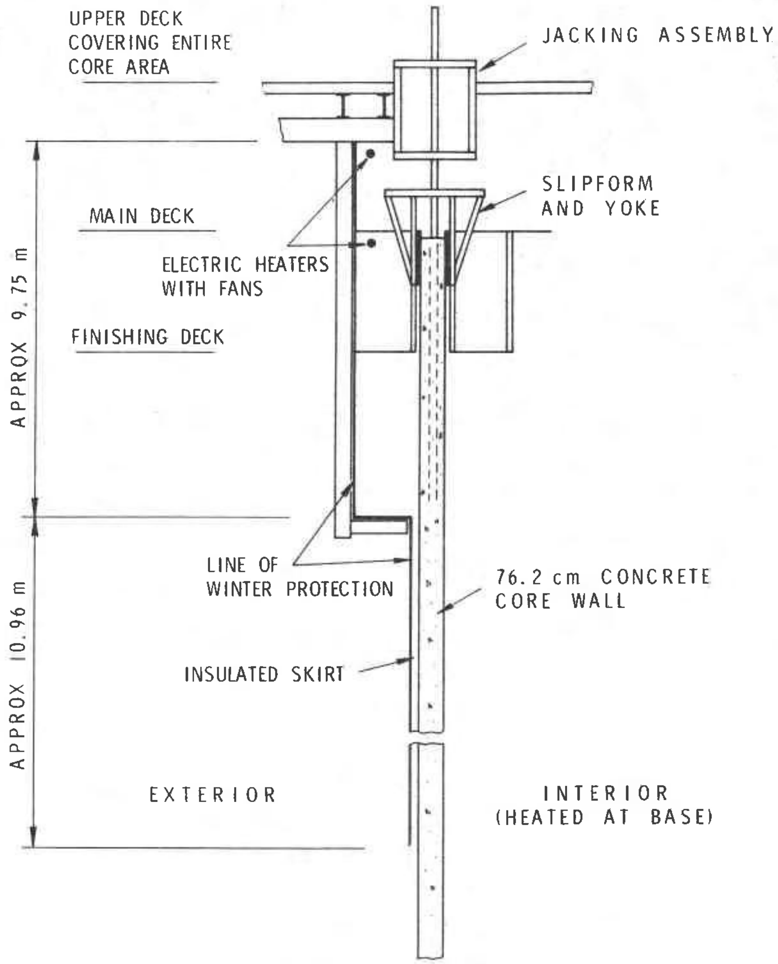

FIGURE 10

SECTION THROUGH SLIPFORM ASSEMBLY SHOWING LINE OF WINTER PROTECTION AND LOCATION OF ELECTRIC HEATERS, CN TOWER

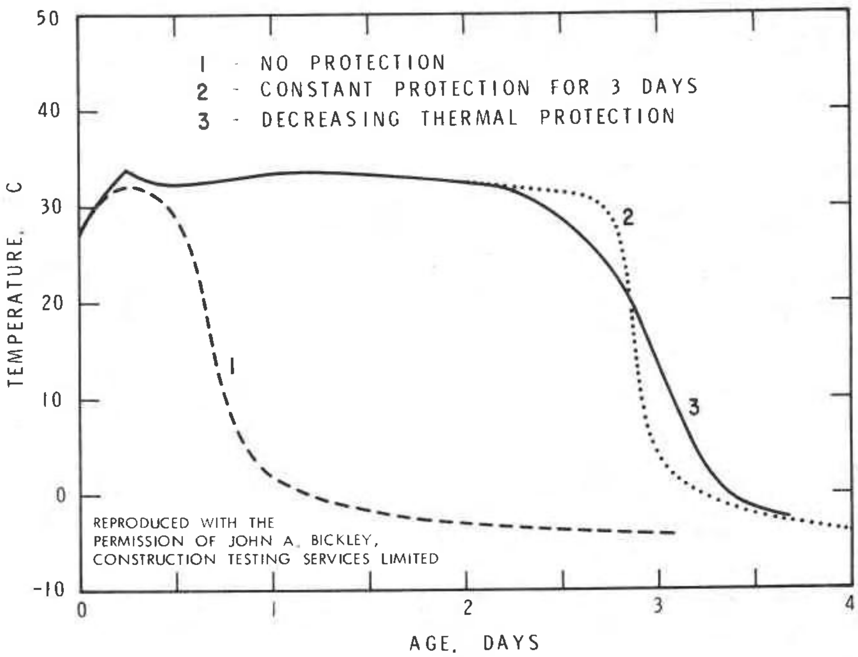

FIGURE 11

GRAPH SHOWING CALCULATED SURFACE TEMPERATURES OF CONCRETE WALL, CN TOWER

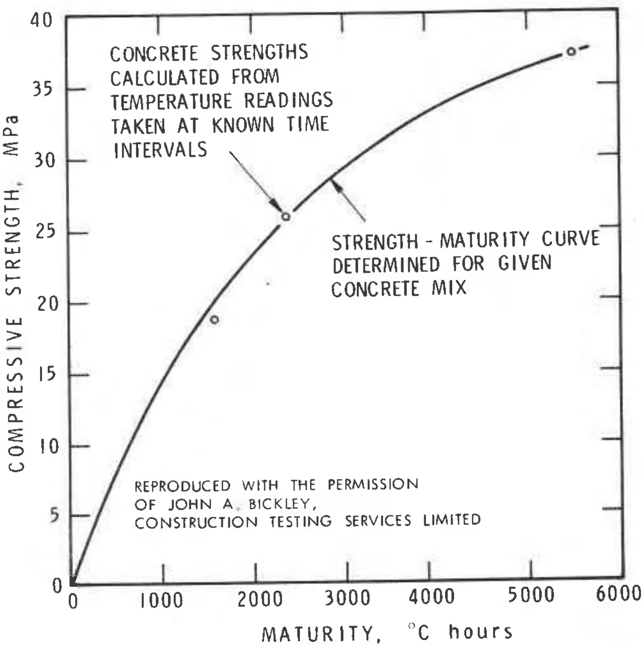

FIGURE 12

GRAPH SHOWING CORRELATION OF STRENGTH - MATURITY CONCEPT, CN TOWER