

**DETERMINE POTENTIAL OF CARBONATION
OF CONCRETE IN CANADA**

STATE-OF-THE-ART LITERATURE REVIEW

**Prepared for the
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Caution

The analyses, interpretations and recommendations are those of the consultants and do not necessarily reflect the view of Canada Mortgage and Housing Corporation, which assisted in the study and its publication. The results of the study are published for information purposes and the reader is cautioned that the scientific data generated herein are derived from a study and interpretation of selected international references.

Extrapolation of this data to most structures in Canada may be feasible, provided that the vagaries of climates and the behaviour of structure systems and building materials in those local areas are understood.

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EXECUTIVE SUMMARY

Introduction

Deterioration of steel-reinforced concrete is occurring at higher rates than anticipated. Many components which form part of any nation's infrastructure, roads, bridges, dams, water and sewage treatment plants, airports, docks and the structural elements and facade assemblies of public and private high rise buildings, are all prone to various decay processes. Most of these time-dependent decay processes are known to result from freeze-thaw reactions in concrete, alkali-aggregate reactivity damage, roadway de-icing salt damage or sulphate reaction.

Another time-dependent phenomena, affecting rate of decay of concrete, is carbonation. Carbonation occurs when concrete reacts with naturally occurring carbon dioxide. This event changes the passivity of concrete, over steel, allowing the onset of corrosion of the reinforcing steel. Carbonation is not a problem in very dry concrete or in water saturated concrete.

Carbonation of concrete has attracted considerable, and recent, interest in Europe and Australia, where it is known to be the cause of current problems. As the Canadian building stock is younger than the troubled European structures it was considered prudent to investigate the literature to assess the likely occurrence of future carbonation-induced decay of reinforced concrete in Canada.

Methodology

A literature survey was carried out in the general subject area of carbonation of concrete. Literature having relevance to Canadian construction materials and practice was reviewed. An analysis from this review is presented in this report.

Since a potential carbonation problem is thought to exist in Canada, a research design has been provided which will include techniques for:

- . identification of the reported problem in Canada;
- . development of survey techniques;
- . pilot testing of survey techniques;
- . technical analysis of pilot test results;
- . evaluation of the significance of the results and, if required, and as a third phase in this study area;
- . a survey to determine the extent and severity of the problem in Canada.

Findings

Carbonation rates are largely a function of the climate and the environment to which the concrete is exposed. Relative humidities in the range of 50-75% produce the highest rates of carbonation. Corrosion of the steel proceeds at rates increasing with temperature. Corrosion is very slow below 10°C; the threshold for active corrosion considered in this report.

Environment Canada data has been used to identify principal Canadian cities in which carbonation induced corrosion might occur. This data may be modified for effect of solar impact on building facades. An arbitrary division has already been made, separating cities likely to have high and low susceptibility to carbonation.

Conclusion

The literature review has confirmed that all concrete is subject to carbonation. The rate and extent to which it occurs are primarily functions of concrete quality and climate. Incorporation of supplementary cementitious materials may have little affect on rates of carbonation in Canada.

Adequate total cementitious content, low water-cement ratio, good compaction, adequate cover over reinforcing steel, and competent curing practices are important for producing carbonation-resistant concrete.

This review of the literature illustrates the causes through research and field study of the occurrence of carbonation in several countries. As Canadian concrete technology practices by and large parallel those practices, it is reasonable to assume some incidence of carbonation in Canada. The influence of the Canadian climate has to be considered.

Research Study

A research design for a pilot field and laboratory study has been provided. It covers the areas listed above under "Methodology".

Where carbonation is found to occur, it is anticipated that existing materials and technology can be used to protect concrete from carbonation.

1. INTRODUCTION

1.1 Background

In recent years widespread and often premature deterioration of concrete structures has caused professional attention to be focussed on all aspects of the corrosion of concrete. The deleterious effects of de-icing salts, freeze-thaw cycles and alkali-aggregate reactions are the dominant factors which have preoccupied cement and concrete technologists internationally. National Standards Committees in Canada and elsewhere are devoting much of their time to re-drafting durability requirements for concrete.

In Europe an additional and widely identified cause of concrete deterioration is carbonation. Since carbonation is a time-dependent phenomenon, it is probable that its identification in Europe as a major cause of deterioration is related to the greater average age of the concrete infrastructure there.

This literature review is a first step in assessing the potential for the deterioration of Canadian structures due to the carbonation of concrete. This review seeks to assess if, in time, carbonation induced damage may become significant in Canadian structures.

1.2 Objectives

The objectives of this project are to:

Carry out a literature review.

Provide an analysis of the findings from the review, and
should further work be considered necessary,
Provide a reasearch design.

1.3 Scope

The project contract required that:

- a) A literature search be carried out. The search would cover all aspects of the carbonation problem as it is likely to occur in Canada. The literature search would highlight work already completed in relation to basic chemistry, impact of this chemistry on standard concrete and how the chemistry would be further affected by the inclusion of different types of cements, cementitious materials, pozzolans and admixtures.

The literature review included an assessment of this basic chemistry in relation to past and current Canadian building practices and choices of building materials.

- b) Based on the literature review and the researcher's experience, and after concurrence with Corporation, a research design was prepared which proposed a further program of work to:

- . highlight technical unknowns
- . identify reported problems in Canada
- . develop survey techniques
- . pilot test survey techniques
- . technically analyze pilot test results.
- . determine significance of the pilot results to the federal treasury and,

as a third phase in the study area, propose a survey to determine the extent and the severity of any carbonation problem in Canada.

1.4 Intended Audience

It is expected that the main beneficiaries of this project will be agencies owning or financing major building inventories. These agencies would include CMHC, Public Works Canada, Provincial Governments and those representing the private sector such as the Canadian Institute of Public Real Estate Companies.

1.5 Organization

To develop the literature survey an initial computer search was made through the Canadian Housing Information Centre with the assistance of Ms. Hera Arevian. The CAN/SDI and CAN/OLE services of CISTI were used. With the assistance of Mr. B. Scott Mellon, the NRC library was also searched for suitable references.

The most comprehensive source of published data on concrete technology in North America is the American Concrete Institute (ACI). Their 55 Year Index 1905-1959 (3), Supplemental Index 1959-1968 (4), and their most recent 10 Year Index 1969-1978 (5) were searched for relevant titles. All ACI data published since 1978 are in the hands of the author of this report. Other major journals searched were Concrete (U.K.), Concrete Construction (USA) and The Journal Of The Prestressed Concrete Institute (USA)

In September 1986 a visit was made to the Building Research Establishment in Garston, England. The subject of carbonation as it affects steel reinforced structures was discussed with Messrs. Osborne, Treadaway and Currie, three leading authorities on the subject. A

significant volume of up-to-date literature was obtained as a result of this visit together with some unpublished data. Also contacted in the U.K., in September, was Mr. M.B. Leeming of Arup Research and Development who provided some up-to-date references, particularly from continental Europe.

62 references were compiled for this literature review and these are listed in the annotated bibliography shown as Appendix A. Recent material was preferred, particularly with regard to the modifying effects of supplementary cementitious materials. All references considered relevant were used in assessing the Canadian situation.

2. CARBONATION

2.1 General

The reaction of carbon dioxide with cementitious products has been attributed to controlled or uncontrolled building practices and to uncontrollable natural phenomena..

2.1.1 Carbon dioxide from unvented heaters, used to protect newly placed concrete floors during winter, combines with bleed water on the surface to form carbonic acid. This acid destroys the immature surface of the concrete leaving it soft and friable, Kauer et al (34).

2.1.2 Carbon dioxide is applied to pre-cast concrete blocks during the initial curing state, generally under pressure. This reduces most long-term shrinkage and moisture movement effects and results in dimensionally stable blocks, NCMA (44).

2.1.3 Atmospheric carbon dioxide causes surface crazing and cracking due to shrinkage but also produces hardening of the surface of dense mature concrete. This can be beneficial in its resistance to aggressive chemicals, Biczok (12). In a Roman aquaduct in Caesarea, carbonation of the polished mortar surface of the open canal was seen as beneficial, Malinowski (40).

2.1.4 Atmospheric carbon dioxide penetrates concrete lowering its pH and ultimately, on reaching embedded metals, creates an environment in which corrosion can occur. It is with this last phenomena only that this report is concerned.

2.2 Mechanism

There is no disagreement about the mechanism of carbonation as it affects the corrosion of metals in concrete. Carbon dioxide reacts with the hydroxides in concrete pore solutions to form carbonates plus water. This reaction reduces the pH to about 8.3, Locke et al (39), B.R.E. (13), Lea (35), Roberts (54). This effect impairs the protective layer to steel formed by the calcium hydroxide solution in the concrete. In this modified environment, where sufficient oxygen and water are available, reinforcing steel will corrode. The resulting rust products occupy a greater volume than their constituents. The great pressures created by these reactions fracture the concrete and start the process of deterioration.

Arup (7) summarizes in a flow diagram all the factors affecting the ability of concrete to protect embedded steel from corrosion. Data on rates of corrosion after carbonation are reported by Gonzalez et al (30). A more detailed analysis of the chemical process, Jungerman (33), separates the reactions caused by carbon dioxide into three stages, viz

1. Ingress by diffusion.
2. Reaction of CO₂ with water molecules.
3. Reaction of resultant carbonic acid with the alkaline constituents in the concrete.

2.3 Presence and Effect of Carbonation

From the literature we are able to conclude that generalized surveys to determine the extent and severity of building populations can be made with simple indicator tests, Currie (24). For the determination of exact degrees of carbonation and the resulting changes in the chemical and physical characteristics of carbonated concrete, or to determine by the measurement of altered characteristics the degree to which carbonation has occurred, more sophisticated testing techniques are appropriate, Litvan et al (38), Lin et al (37).

Most authorities agree that a measurement of the presence and extent of carbonation can be obtained by spraying a freshly fractured face of concrete with a dilute solution, usually in the range of 1/2 to 2%, of phenolphthalein in alcohol, BRE (13), Lea (35), Roberts (54). This procedure is standardized in the U.K. by the British Standards Institute (16). The accuracy of this method is satisfactory for most surveys to determine the potential for carbonation induced corrosion. The procedure is simple, cheap and safe, and interpretation of results is easy.

Other carbonation indicators can be used, Lea (35), Roberts (54) but phenolphthalein provides the best colour contrast between zones of high and low alkalinity. Such tests, however, only distinguish between concrete that is still highly alkaline and that in which the alkalinity has been reduced below about a pH of 10, a pH likely to permit the onset of reinforcing steel corrosion.

For more exact analysis of concrete samples, X-ray diffraction or chemical analyses, Roberts (54) and thermogravimetry, pore size analyses, surface tensile tests and compressive tests may be appropriate, Litvan et al (38), Lin et al (37) additionally used scanning electron microscopy, and nitrogen gas absorption.

2.4 Carbonation Depth Predictions

Carbonation is a progressive process proceeding from the concrete surfaces exposed to atmosphere towards the remote interior of a concrete component. Many factors affect the rate of progress, in concrete, of this carbonated front. The principal factors are the porosity and permeability of the concrete, and the presence or absence of cracks. The many factors which modify these characteristics will be considered, in more detail, later in this report.

The carbonated front creates conditions conducive to corrosion when it reaches the reinforcing steel. The amount of concrete cover to this reinforcement is therefore a significant factor in the time to corrosion.

A number of methods have been proposed for the prediction of rates of carbonation. Allen et al (6), after Pihlajavaara, proposed a function of the square root of time, $10 B \sqrt{t}$, where t is time in years and B is a constant related to concrete quality and environmental conditions. Use of this formula results in the following typical carbonation depths:

Carbonation Time (Years)	Quality of Concrete and Storage Conditions					
	Low Strength		Medium Strength		High Strength	
	Outdoors	Indoors	Outdoors	Indoors	Outdoors	Indoors
	(moist)	(moist)	(moist)	(moist)	(moist)	(moist)
Thickness of Carbonation Layer (mm)						
	<u>B=0.6</u>	<u>B=1.0</u>	<u>B=0.2</u>	<u>B=0.5</u>	<u>B=0.1</u>	<u>B=0.2</u>
1	6	10	2	5	1	2
2	9	14	3	7	1.5	3
5	13	22	4	11	2	4
10	19	32	6	16	3	6
25	30	50	10	25	5	10

The relationship between carbonation depth and concrete quality is very clear in the above table. Browne et al (15) propose the use of a function developed by Klopfer where depth of penetration of CO₂ in mm can be calculated as $\sqrt{2 D t}$, where t is time in years and D is a diffusion* coefficient in mm²/year. Using this function Browne et al (15) derive a graph for various qualities of concrete. Values so derived are summarized below:

Age (Years)	Carbonation Diffusion* Coefficient (mm ² /year)	Depth of Carbonation (mm)
10	15	17
	5	8
30	15	30
	5	15
100	15	55
	5	12

Note: While the term diffusion may not be appropriate, it is the term used in the referenced document.

As will be seen from the above, a high rate of diffusion would double the depth of penetration which would result from a moderate rate. Weber (61) proposes a more complex calculation for carbonation depth y in mm:

$$y = \sqrt{2 D \cdot \frac{C_1}{C_2} \cdot t}$$

Where D is a diffusion coefficient in mm^2 per year, C_1 is the CO_2 concentration in atmosphere (about 0.6 gm/m^3), C_2 is the quantity of CO_2 required for the carbonation of 1 m^3 of concrete ($10,000$ to $50,000 \text{ gm/m}^3$) and t is carbonation time in years. Not only can this formula be used to determine carbonation time of unprotected concrete but it can be modified to predict the beneficial effects of protective surface treatments. Diffusion factors are given for various types and grades of concrete, and also for protective coatings described as carbonation retarders. Hobbs (32) states that the depth of carbonation in a normal portland cement concrete is:

$$d = A_t^{0.5} \left[\frac{w}{c} - B \right]$$

Where t is the age in years, $\frac{w}{c}$ is water-cement ratio and A and B are factors. From a series of laboratory tests on concretes, with and without fly-ash, and with 7 and 91 days curing, Nagataki et al (43) derives the following formula:

$$x = A B (\alpha (w/c) - \beta) t \quad \text{cm. Where}$$

x = depth of carbonation
 A = correcting factor
 B = factor for initial curing in water
 α, β = factors for fly-ash
 w/c = water-cement ratio
 t = age in years
 γ = factor for environmental conditions.

The laboratory test data from which the formula was derived by Nagataki et al (43) resulted in the following values for the various coefficients:

Exposure Condition	Fly Ash Content (%)	A	B	α	β	γ
Outdoors	0	0.55	1	4.211	1.831	0.25
	30	0.77		3.311	1.777	
Indoors	0	1.50	1 for 7 days	1.656	0.622	0.5
	30	1.61	0.78 for 91 days	1.445	0.619	

There are some potential pitfalls in using the above formulae. Determination of the constants or coefficients is difficult. Suggested values given in the referenced documents can change the order of the result. In the method suggested by Weber (61) a worked example for the effect of a carbonation retarder predicts a life of 1000 years, much beyond the actual life of the protective material!

Perhaps the most useful aspect of the papers referred to in this section are the Tables of examples which give some idea of what order of carbonation depths can be expected with time and confirm the effect of concrete quality.

2.5 Carbonation Depths In Structures

While the formulae given above may be used for prediction, many data are available of measurements of carbonation depths in existing structures. These data make clear the effects of high quality and strength and differences between prestressed and non-prestressed components.

Higgins (31) quotes typical values for external concrete as follows:

Age (Years)	Depth of Carbonation (mm) 28-day Cube Strength (MPa)	
	20	40
1	5	0.5
5	8	1
10	12	2
50	25	4

For structures in the Middle East, Treadaway et al (58) quoted the following data for cast-in-place concrete:

Age (Years)	Carbonation Depth (mm)
1.75	50
5 - 6	10 - 22 (Average 14.8)
6 - 7	5 - 25
7	0 - 25
12	70

The above values, which are for concrete structures built in the small state of Bahrein in the 70's, clearly reflect a wide range of concrete quality. Data given for pre-cast units in the same area reflect a very slow rate of carbonation. Maximum values at 4 years reach 9 mm but a mean 4 year value of 8 mm would be typical. Presumably the marked difference between cast-in-place and pre-cast concrete again reflect quality.

Risager (53) tested 74 year old pre-cast concrete beams and found carbonation depths less than 1.5 mm. Maage (46) reported mean corrected depths of 8.8 mm for normal portland cement concrete at 5 years.

Perhaps the most comprehensive summary of data from structures to date, Currie (24) shows carbonation depths from many structures in the U.K. and Germany, and includes data from cast-in-place, normal pre-cast and prestressed pre-cast structures. This paper includes data from Matthews (42) and relevant depths are summarized below:

1. German data from bridges and buildings:

<u>Age (Years)</u>	<u>Average Cube Strength (MPa)</u>	<u>Average Carbonation Depth (mm)</u>
11	49	5 #
31	28.4	23 - 35
41	53	Negligible
46	26.5	25 - 50

Here high strength results in little carbonation and low strength in significant penetration.

Pre-cast unit

2. Normally reinforced cast-in-place and pre-cast components.

<u>Age (Years)</u>	<u>Cement Content (Kg/m³)</u>	<u>Average Cube Strength (MPa)</u>	<u>Average Carbonation Depth (mm)</u>	
			<u>Exposed to Rain</u>	<u>Sheltered from Rain</u>
9	250	28	1 - 2	10 - 15
7	255	27.3	3	10 - 20
30	300	-	3	18 - 30
12	350	23	1 - 3	15 - 20
17	300	-	<3	60 *
9	300	40.8	8 - 14	25 - 30

Here the increase in carbonation depth of sheltered components compared to those exposed to further curing is shown.

* Honeycombed concrete

3. Prestressed concrete components.

Age (Years)	Average Cube Strength (MPa)	Average Carbonation Depth (mm)	
		Exposed to Rain	Sheltered from Rain
10	66.4	<1	<1
10	59.5	0.5	2.7
7	51.3	0.2	0.2
8	49.3	0.5	0.25 - 0.5
9	58.7	0.5	0.5 - 2.0
13	48.0	5 - 8	5 - 17
11	56.0	-	<0.2
11	58.1	-	0.25 - 1.5
9	61.5	-	0.2
6	-	-	0.1
10	48.5	-	<0.5 - 3.0
8	66.6	-	<0.2

Here high quality and high strength components generally show little carbonation.

Extensive data is available for pre-cast units used in various U.K. prefabricated housing systems. The average carbonation depths at 35 years ranged from 7.4 to 14.6 mm with an average for all systems of 10.4 mm. Maxima were up to 45 mm with a significant proportion of the results greater than 20 mm.

In a recent survey of 42 structures in the Middle East, Rasheeduzzafar (51) observes that the deleterious effects of carbonation both of itself and also in enhancing effects due to chlorides have been discounted.

2.6 Environmental Parameters

It is generally recognized that the rate of carbonation ingress into concrete is greater in the range of 50 - 75% relative humidity, BRE (13), Wierig (62), although a range of 40 - 70% is suggested by Weber (61).

While the rate of penetration will be influenced by humidity, all other factors being constant, the resulting depassivation of the steel will only result in corrosion if moisture and oxygen are available to the steel. Thus, although the concrete inside a building may normally carbonate about three times as fast as external concrete, Higgins (31) corrosion is unlikely because of the usually dry environment. Additionally, even in the presence of moisture and oxygen, the rate of corrosion will be much influenced by the temperature. A minimum of 10° C is usually accepted as the minimum to sustain a significant rate of corrosion.

There are no data in the literature identifying the effects of energy conservation nor such factors as solar gain, building orientation, or moisture and air ingress or egress.

3. MODIFYING FACTORS

3.1 Cover

For any given quality of concrete, it has been long been accepted that time to corrosion is a function of cover, Clear et al (19). While chloride ions from de-icing salts penetrate to reinforcing steel relatively quickly even in good quality concrete, the rate of advance of a carbonation front is relatively slow in good quality concrete. If, therefore, a cover is provided in excess of the likely carbonation penetration within the service life of the structure, then carbonation can be ignored as a potential cause of corrosion and a potential source of ultimate failure. Such a cover has to take into account the variation inherent in reinforcement placement. The actual minimum cover achieved will be the governing factor.

The potentially disastrous effect of reduced cover due to placing tolerances was demonstrated by Currie (24) who showed that a 25% reduction in cover in a high quality pre-cast concrete component from 20 mm to 15 mm nearly halved the service life of the component. Data published recently

shows that a minimum 10 mm standard deviation is difficult to achieve, Bickley (10). Specified cover must therefore take into account a realistic placing tolerance and must be set with due regard to the minimum actual cover likely to be achieved subsequently..

In the United Kingdom investigation of the Ronan Point building and other structures made from pre-cast concrete BRE (14) found carbonation depths up to 20 mm. Cover down to 20 mm was found although it was generally 35 - 40 mm, confirming other data on the variability of cover in practice, Bickley (10). On the basis of their data, BRE predicted increasing carbonation induced corrosion damage in the following 10 years. Protective coatings were recommended as a means of reducing carbonation rates and thus increasing the life of the structural components of this building.

3.2 Cracking

There is little data in the literature on the effect of cracking on corrosion in the absence of chloride ions. It is however acknowledged that the depassivating effect of carbonation will reach the reinforcement relatively quickly through a crack compared to the surrounding uncracked concrete, Beeby (8). The concept of a depassivating front moving towards the centre of a concrete component at variable rates is illustrated graphically. Where a crack occurs, this front is shown reaching the steel where it intersects the crack long before it does so in the uncracked concrete. Nevertheless in this study and one by Schiessel (56) the necessity for limitations on crack width for durability is supported.

While the corrosion thus initiated will be local to the crack, it can propagate along the steel in both directions from the crack and cause extensive deterioration with time. Cracks are thus seen as important triggers of corrosion sites. Discussing cracking caused by the carbonation shrinkage as opposed to reinforcement corrosion, Currie (25) points out that except where honeycombing or structural cracking provide easy access CO₂ carbonation

cracking is fine_and parallel to reinforcement. Cracking of cover therefore occurs prior to significant loss of structural strength. Remedial action can therefore be taken before a structural hazard occurs.

3.3 Aggregate Type And Quality

Dense natural aggregates can be used to make high quality carbonation resistant concretes, Roberts (54). Studies by Collins (20, 21) show potentially higher risks with porous sandstones and jurassic limestones. Such aggregates are characterised by high to moderately high water absorptions. Concrete strengths are much lower relative to cement content compared to proven aggregates of high quality.

It has been shown by Osborne (47) that lightweight concrete made with pelletized slag was durable in a marine environment. The tests on laboratory-cast specimens were made at a marine exposure site and showed no carbonation after 2 years.

3.4 Cement Quality

Normal portland cement has changed in composition over the years, Concrete Society (22). From the point of view of durability, the change means that the same concrete strength can be achieved at a lower cement content and higher water-cement ratio than previously. For instance a concrete meeting grade 32.5 MPa requirements needed 350 Kg/m³ of cement at a water-cement ratio of 0.50 in 1970. In 1984 only 310 Kg/m³ of cement at a water-cement ratio of 0.57 meets the same requirement. This change together with a slight decrease in C₃A content would make concrete of the same grade more susceptible to carbonation than previously.

Cement type is considered to be of minimal importance in providing protection from corrosion according to ACI Committee 225 (2). The quality of concrete is stressed and given high quality, good protection is assumed to be achieved regardless of cement type.

3.5 Curing

In the series of tests on Middle East concrete reported above (58), curing was shown to reduce carbonation depths very significantly compared to those for non-cured concretes. Differences in carbonation depths were of the order of 3:1. Tests by Fattuhi (27) confirmed the beneficial trend of low water-cement ratio and good curing. Curing is also stressed as a major factor by Weirig (62).

3.6 Concrete Quality

In the context of this review, high quality is taken to mean concrete made with materials meeting all CSA 23.1 requirements for quality, an adequate cement content, a low water-cement ratio and fully compacted.

In North American literature it is generally accepted that good quality concrete is, in a corrosion sense, entirely resistant to carbonation damage. Committee 222 of ACI (1) gives only passing acknowledgement to carbonation and Manning (41) states that in good quality concrete, carbonation probably never penetrates more than 30 to 40 mm within the lifetime of the structure. In view of data from other sources, Newman et al (45), this conclusion may be too optimistic.

In a field study of existing structures and laboratory cast specimens, a relationship was found by Treadaway et al (58) between carbonation depths for Middle East concretes. In some structures it was found that penetration of carbonation to the reinforcing steel had occurred well within the planned service life of these structures. In the laboratory cast specimens, quality was shown to relate well to carbonation depth. Higher cement contents and strengths and lower water-cement ratios reduced carbonation depths as would be expected.

3.7 Supplementary Cementing Materials

3.7.1 General

Supplementary cementing materials as defined in Canadian Standard CAN A23.5-M86 includes ground granulated slag, fly ash and silica fume. Widespread recognition of technical and economic benefits resulting from their use has led to much research activity in recent years. Beneficial effects include reductions in deleterious heat of hydration effects, alkali-aggregate reactivity, sulphate attack and improvements in strength and impermeability. Included as a probable improvement has been a reduction in carbonation rates.

3.7.2 Ground Granulated Blast Furnace Slag

Carbonation depths were measured by Osborne (48) in three sets of 100 mm cubes after curing in air at 20° C and 65° R.H. from the time of demoulding for periods of one to nine years. Depth of carbonation correlated well with strength and was found to be approximately proportional to the square root of the time of exposure. Where good curing was used carbonation depths were shallow after one year. This confirmed Smoleyk's[#] findings that with good compaction, low water-cement ratio, adequate curing and relatively high strengths (35 MPa or higher) carbonation depths can be limited to a maximum of about 10 mm. Carbonation depths increased with increasing slag contents and for the normal portland cement samples with decreasing C₃A contents. Maximum typical values for the two kinds of concrete tested were:

	<u>Concrete Age at Test</u>	
	<u>1 Year</u>	<u>9 Years</u>
Normal Portland Cement	5 mm	10 mm
Slag	18 mm	68 mm

[#] Smolczk, H.G. State of Knowledge On Chloride Diffusion In Concrete, Beton work Fertigteil - Technik, 1984, v 50 (12) pp 837-843.

The above values for slag concrete were however for very high slag contents. Approximate interpolated values for the range of slag contents used or considered for use in Canada would be as follows:

Slag Content %	1 Year	9 Years
30	5	15
50	7	18
65	10	31

Bearing in mind however that the above results are for poor curing it seems that it is the curing and not the slag content which is the more important factor.

In laboratory tests on small mortar specimens subjected to controlled applications of carbon dioxide, Paillere et al (50) found that carbonation increased as slag content increased. It was similar to that of portland cement concrete at about 20% slag content. Carbonation increased with increasing freeze-thaw cycling and the use of air entrainment was shown to have no beneficial effect on carbonation rate.

A good summary review of the carbonation resistance of concretes containing slag was made by Reeves (52). It recognized the potentially negative aspect of reduced calcium hydroxide and the positive effect of decreased permeability. It confirmed the good performance in service of slag concrete. No evidence was found that there is more carbonation induced corrosion in slag concrete than in normal portland cement concrete. The paper re-affirms the good performance of all good quality concrete.

3.7.3 Ground Gasifier Slag

This is a solid by-product of a gasification scheme to produce natural gas from coal. Osborne (49) showed that again when slag was used the most important factor in reducing carbonation depths was curing. Strength and carbonation depths were related linearly. Carbonation depths the concretes were particularly susceptible to the effects of curing.

3.7.4 Fly Ash

Paillere et al (50) showed in tests on small mortar specimens that at contents of less than 20% fly-ash, concrete performed similarly to normal portland cement concrete. As the fly-ash content increased so did carbonation depths. The most important factor in limiting carbonation depths was shown to be curing. Freezing and thawing increased carbonation and air entraining did not of itself reduce carbonation.

In attempting to improve the performance of their low calcium (5% CaO) fly ashes the Chinese produced data showing the effects of calcium enrichment. Cuijuan et al (23) made the tests on laboratory prisms with carbonation being achieved by controlled laboratory application. Typical carbonation results before calcium enrichment of the fly ash were:

	Carbonation Depths At 25 Days (mm)	
	<u>Control</u>	<u>15% Fly-Ash</u>
Slag Concrete	14.6	21.3
Normal Portland Cement Concrete	4.9	12.4

After calcium enrichment, carbonation depths in the slag concrete were similar to the normal portland cement concrete depths. For specimens given normal curing up to 1 year, the low calcium

fly-ash has more than 3 mm of carbonation while for the calcium enriched specimens, carbonation depths were less than 1 mm.

In an extensive and elegant series of tests made by Nagataki et al (43) laboratory cast specimens were given a range of initial curing periods and both indoor and outdoor exposure. The effects of curing were well defined showing no deleterious effects for outdoor storage provided an initial 7 days of water curing was given before exposure. Thereafter, water replenishment by rain was beneficial. Outdoors carbonation only increased slightly after five years but continued significantly indoors. For indoor exposure, initial curing was a significant factor. Interestingly, corrosion was found in specimens stored indoors. As noted in 2.4 a formula was derived to calculate depth of carbonation based on age and factors for initial curing, fly-ash, water-cement ratio and environmental conditions.

The effects of elevated temperatures on the carbonation induced corrosion properties of fly ash concrete up to one year were studied in the laboratory by Ehm et al (26). Tests were made at 95° C on both sealed and unsealed specimens. Elevated curing temperatures led to the significant consumption of calcium hydroxide but pH values at the level of the reinforcing steel did not fall below 9.5 and no corrosion was seen.

Because the use of fly-ash in structural concrete has been widespread for a long time, there is good data available on its performance in service. Core samples were recovered by Newman et al (45) from internal and external power station structures in England between 10 and 30 years old. For the external structures, carbonation depths were 1 mm for normal portland cement concrete and 1 to 12 mm for fly ash concrete with an average of 6 mm. For the internal structures the

carbonation depths for normal portland cement concrete was 16 mm and for fly-ash concrete 22 to 26 mm. In the normal portland cement concrete cover to reinforcement was 22 mm and the reinforcement was corroding, suggesting that the carbonation front may have penetrated deeper than the phenolphthalein test indicated. Interestingly, the data tends to support the thesis of Higgins (31) that interior concrete carbonates faster than exterior concrete. Presumably because moisture is available, periodical curing of external structures can continue.

In tests on twenty-five year old foundations using X-ray diffraction tests on core samples virtually no carbonation was found in the fly ash concrete, Cabrera et al (17). While the reason for this would appear to be very low porosity, it is assumed that this property resulted from ideal curing in a massive foundation structure which could not dry out.

In Australia, Roper et al (55) surveyed over 200 structures with ages up to more than 20 years. The overall conclusion was that the structures containing fly ash exhibited at least equal durability with portland cement concretes in service. However, 85% of all buildings showed undesirable deterioration. Carbonation rates were sometimes more rapid for fly-ash concretes. From the cracking and deflection data it was deduced that a major factor was inadequate curing.

In considering code provisions for Australia, it was reported by Walsh (60) that corrosion was considered more likely to occur in fly ash concrete when moist curing was inadequate. Additional cement is required when fly ash is used, the amount depending on the degree of moist curing applied. For exposed building components, adequate moist curing is assumed. The requirements are the same for slag concrete. When reliable test data on durability for the proposed concrete is available, the requirements for extra cement may be reduced.

In a literature study by Berry et al (9) it was concluded that good quality fly ash is comparable to plain concrete in its resistance to carbonation. Where a low cement content is used or curing is inadequate, durability is poor.

3.7.5 Silica Fume

Eighteen structures in Norway with ages up to almost eight years were tested by Skjoldovold (57) to determine carbonation depths. Silica fume contents of between nil and 13.7% were involved although in all but one structure they were below 10%. A correction factor was applied to compare all concretes tested at a "standard" age of 5 years. While decreased protection might be expected from silica fume because of a reduction in calcium hydroxide content, proper curing reduced permeability to the point where susceptibility to corrosion was much reduced. Carbonation depth was found to be proportional to water-cement ratio and not related to strength. However, almost all concretes tested showed unacceptable carbonation depths after 5 years ranging from 2.2 to 21.0 mm with an average close to 10 mm. Laboratory tests confirmed even greater depths even where good curing was used except for low-water cement ratios. Why test results should be relatively poor for normal portland cement concrete as well as for that containing silica fume for the tests on structures was clearly because of high water-cement ratios, which ranged from 0.57 to 0.80 and averaged a surprisingly high 0.71. In the laboratory tests a wide range of water-cement ratios was used from low to high. The structures reported on were presumably built during the early period of the use of silica fume in concrete. Similar results for tests from structures containing concrete with and without silica fume were reported by Maage et al (46). North American practice invariably uses low water-cement ratios in silica fume concretes.

4. THE CANADIAN SCENE

4.1 General

This literature study makes it clear that there has been little concern with the phenomenon of carbonation in North America to date. American Concrete Institute indices (3, 4, 5) and all journals since their publication have few references to carbonation. Most of them refer to the aspects of carbonation other than the subject of this report. Recent North American publications, Bickley et al (11), ACI Committee 222 (1) and ACI Committee 225 (2), make only passing reference to carbonation. It is not seen as a major concern with respect to corrosion. Two of these documents (11, 1) are however mainly concerned with highway structures and no consideration is given to buildings in the public or private sector.

4.2 Environment and Climate

Carbonation rates are largely a function of the climate to which the concrete is exposed. The atmospheric environment may have additional influence but this is discounted here. Relative humidities in the range of 50 - 75% R.H. produce the highest rates of carbonation while corrosion proceeds at increasing rates with increasing temperature. Corrosion is very slow below 10° C, which will be taken as the threshold for active corrosion in this report.

A simple way of initially judging the degree to which carbonation and corrosion might occur in Canada has been provided in this report. R.H. and temperature diagrams have been produced for 21 cities from coast to coast to illustrate climatic conditions where most significant structures are located.

For each city a graph has been drawn showing monthly mean maximum and minimum values for relative humidity and temperature for each month of the year. The data was abstracted from Environment Canada records (18) and are attached as Appendix B to this report.

An arbitrary division has been made into cities in which concrete structures may have high or low susceptibility to carbonation.

As stated earlier, the effects of energy conservation, solar gain, building orientation and air and moisture ingress and egress are not addressed in the literature. These factors will therefore be considered in subsequent research designs.

Susceptibility To Carbonation

<u>High</u>	<u>High</u>	<u>Low</u>
Edmonton	Hamilton	Vancouver
Calgary	Windsor	Victoria
Yellowknife	Ottawa	London
Saskatoon	Toronto	Saint John
Regina	Montreal	Halifax
Winnipeg	Quebec City	St. John's
Thunder Bay	Fredericton	
Sudbury		

Temperatures at which corrosion would occur exist in most cities for a significant proportion of the year.

4.2 Building Stock

The overall Canadian steel reinforced structural concrete stock is younger than the equivalent European stock, which forms the infrastructure of roads, bridges, dams, water and sewage treatment plants, airports and docks. These more massive units are relatively immune from the effects of carbonation. They tend to suffer from failure induced by de-icing salt, freeze-thaw, sulphate or alkali-aggregate reactivity damage to an extent which makes any carbonation-induced effects inconsequential.

It is considered that the structures most susceptible to carbonation damage are buildings such as offices and apartment blocks, and components such as pre-cast concrete cladding, vertical exposed columns and balcony extensions of floor slabs. The majority of these buildings have been constructed since the early 1960's and thus include many approaching an age at which carbonation damage might be expected.

4.4 Current Building Practice

The building industry is efficient, cost conscious and extremely competent in using state-of-the-art technology to deliver completed buildings quickly. This it has done through successive building booms since the 1960's. The quality of concrete and concrete workmanship have often left much to be desired. Fling (28) has commented recently on the striking difference between concrete quality in Europe and North America.

It has been common practice for several decades to use concrete mixes designed by concrete suppliers and sold to meet a guaranteed strength specification. In the face of severe market competition and by the incorporation in these mixes of chemical admixtures and supplementary cementitious materials, it has been possible to produce very economical concretes. Economical that is as defined by the cost of supply. Unfortunately, as is now being realized, such concretes are not durable when subjected to some of the aggressive environments common to Canada.

Typical Canadian practice for high-rise construction of apartments has been the use of mixes to meet a 28-day strength requirement of 20.7 MPa. This strength has been met with cement contents as low as 230 kg/m³. Even at the specified workability, the intended water-cement ratio would typically be 0.65. With water often added at site beyond that permitted by National Standards, water-cement ratios of 0.70 would not be uncommon.

Curing practice has always been deficient and continues to be so. Many building components receive little if any moist curing. As the literature shows, this must increase susceptibility to carbonation.

Reinforcement cover standards are now seen to be inadequate for some widespread exposure conditions. The accuracy of reinforcement placing has been shown by Bickley et al (10) to be variable and results in extensive deficiencies in cover.

The quality of concrete in pre-cast concrete facades is generally better than cast-in-place concrete. However, many architectural finishes have been produced with exotic facing mixes. To meet complex architectural profiles cover to reinforcement has of necessity often been lower than desirable for protection from carbonation.

It is therefore logical to hypothesize that some Canadian buildings because of concrete age and quality must be considered vulnerable to carbonation induced corrosion.

4.5 Data On Carbonation

Very little data is available on the carbonation of concrete in Canada. In the 60's and early 70's, a proprietary type of foamed structural lightweight concrete was found to be very susceptible to carbonation, Trow (59). Its manufacture has however long been discontinued, and in many structures the material has been replaced.

Some measurements made on 11 year old floor slabs in a parking structure in Ottawa in 1983 showed a range of carbonation depths of 3 - 13 mm with an average of about 6 mm, Trow (59).

In the investigation of a pre-cast facade in Toronto in 1983, the 15-year old pre-cast units were found to have negligible carbonation (about 1 mm). The concrete was found to be of exceptionally high quality, Trow (59).

No other Canadian data on carbonation is known to the author.

4.6 Possible Extent Of Future Problems

This literature review has confirmed that all concrete is subject to carbonation. The rate and extent to which it develops is primarily a function of its quality. This is true whether the concrete is made with or without the incorporation of supplementary cementitious materials. Adequate total cementitious materials content, low water-cement ratio, good compaction, adequate cover and, perhaps above all, adequate curing are the important criteria for producing carbonation resistant concrete.

It has been shown that humidity and temperature conditions which are conducive to corrosion exist in many Canadian cities at least part of the year.

The ageing of the Canadian building population and the potential susceptibility to carbonation of much of the concrete used are factors which predicate the appearance of carbonation damage in the near future. In fact some may already be occurring. Prime candidates for damage are pre-cast concrete facades with low cover and exposed vertical and horizontal components of buildings.

In view of the size of the Canadian high-rise and major building inventory carbonation induced corrosion damage could lead to significant repair costs. The majority of the buildings of potential concern were built by private enterprise including many built for and financed by public authorities.

As has been demonstrated elsewhere, existing materials and technology can be used to protect concrete from carbonation. It would therefore seem prudent to make at least a pilot study to determine the extent to which carbonation is or will be a problem in Canada.

This study would set out to determine if carbonation induced corrosion is or will be a problem and the optimum time for the initiation of preventative measures. Such measures would likely cost only a fraction of the cost of delayed repairs if carbonation damage is allowed to occur.

5. RESEARCH DESIGN

5.1 Rationale

Findings from the literature survey suggest that carbonation of concrete is occurring in balconies and pre-cast concrete facades of Canadian buildings, to the point where corrosion damage may soon result. The rate of penetration of the carbonation front and its proximity to the reinforcing steel is the key factor to be determined now.

While climate has been identified as the major influencing factor, some technical unknowns are the potential affects of solar gain, building orientation and modifications to building envelopes for energy conservation purposes. The latter may affect air and moisture ingress and egress and have a modifying effect on the environment in which the building component is located.

Problems of carbonation induced balcony deterioration have been reported in Canada, but to date in this country, no carbonation induced corrosion of pre-cast concrete building facades has been reported.

It is proposed that initially a pilot programme be carried out to develop and test survey techniques. After these data have been obtained and analysed, and their significance to the Federal and other treasuries determined, a more detailed survey of the effects and consequences of carbonation of concrete in Canada may be justified, and if it is, it will be proposed.

A simple testing programme is proposed, initially. This programme will provide a large number of specimens, and will determine, in one city with a climate potentially conducive to carbonation, if carbonation has reached a phase at which significant investigative or remedial action is justified. The programme will be designed to allow the assessment of some aspects of the environment such as solar gain and building orientation. If buildings which showing impact, on the envelope, due to energy conservation retrofits can be identified by the study sponsors and access is provided to the study team, then test specimens from affected building components will be obtained.

The testing carried out in this study will be restricted to the identification of the depth of carbonation in each sample. The samples obtained will, however, be retained and will be available should more sophisticated and extensive testing be decided upon in later phases of the work. Toronto is proposed as the pilot study location. This city is identified in this report, as having a climate in which concrete is potentially susceptible to carbonation damage, and contains a large number of the total population of buildings which might be damaged by carbonation. Toronto will be an economic location in which to carry out the pilot study, so that the maximum volume of data can be obtained within the budget limits of the study.

5.2 Pilot Study Design

The phases of the pilot study can be summarized as follows:

5.2.1 Identification of Building Sample

Suitable buildings in the Federal and Provincial stock will be identified with the assistance of Central Mortgage and Housing Corporation and the Ontario Housing Corporation.

Suitable buildings in the private sector will be identified with the assistance of the Canadian Institute of Public Real Estate Companies and by the project contractor.

A building history and data sheet will be compiled of details relevant to the study such as the building age and where possible details of the quality of concrete used.

5.2.2 Access

Agreement to take core test samples from the buildings will be negotiated by the contractor with the assistance of the agencies listed in 5.2.1.

5.2.3 Field Sampling

Field sampling will consist of obtaining nominal 25 mm diameter concrete cores a minimum of 60 mm long. These will be obtained using a diamond drill. Immediately after drilling, the cores will be sealed to exclude the atmosphere and will be marked for identification. Core holes will be filled with a durable high strength material where necessary.

5.2.4 Sampling Locations

Sampling locations will be chosen so that the extreme and opposite orientations of the building from the point of view of solar effects will be included. Otherwise, sampling locations will be chosen so as to be representative of the range of quality of concrete in the structure.

5.2.5 Number of Test Samples

Each test sample will consist of three concrete core samples. From each building about six test samples will be taken. Assuming a building sample of at least fifty buildings, a total of 900 core samples or more will result.

5.2.6 Testing

Each core test sample will be examined visually in the laboratory and split across a diameter along a major axis using a splitting tensile testing technique. One of the freshly exposed surfaces will be sprayed with a dilute phenolphthalein solution to determine if carbonation exists and to establish the depth of any carbonation. This depth will be measured. The splitting tensile test value will be recorded. The quality of the concrete will

be assessed visually. After testing the samples will be resealed and put into storage in case additional testing is required in the future.

5.2.7 Analysis and Reporting

The data will be analysed primarily to show carbonation depth and its relationship to visual quality and strength of the concrete and the proximity of the carbonation front to the reinforcing steel. The effect of influencing factors such as orientation, solar gain and any other building science aspects which may be relevant will be considered.

The report will show whether carbonation is now or is imminently a problem requiring attention. If it is not now a problem, a prediction will be made of time for the carbonation front to reach the steel and to induce corrosion. If the data shows the need for a further study, then an outline will be designed for such a study.

5.2.8 Benefits of Pilot Research Programme

If, in this study, carbonation is shown to be at or approaching depths at which it could result in corrosion damage to structures, then timely remedial action can be taken. If the damage problem is shown to be some way in the future, then the timing of further investigative work and appropriate protection measures will be proposed. Either result will avoid the major repair bills for carbonation damage that have already occurred in other countries.

APPENDIX A

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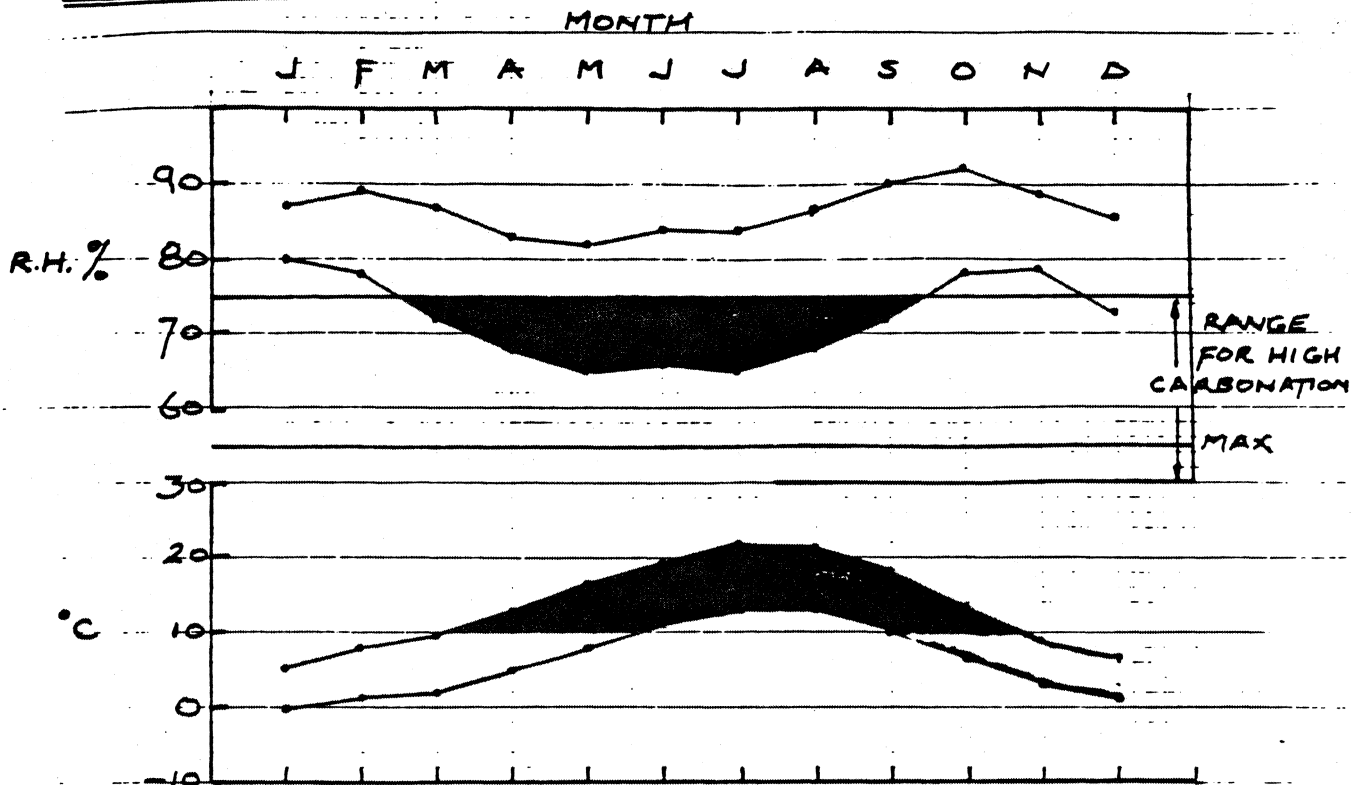
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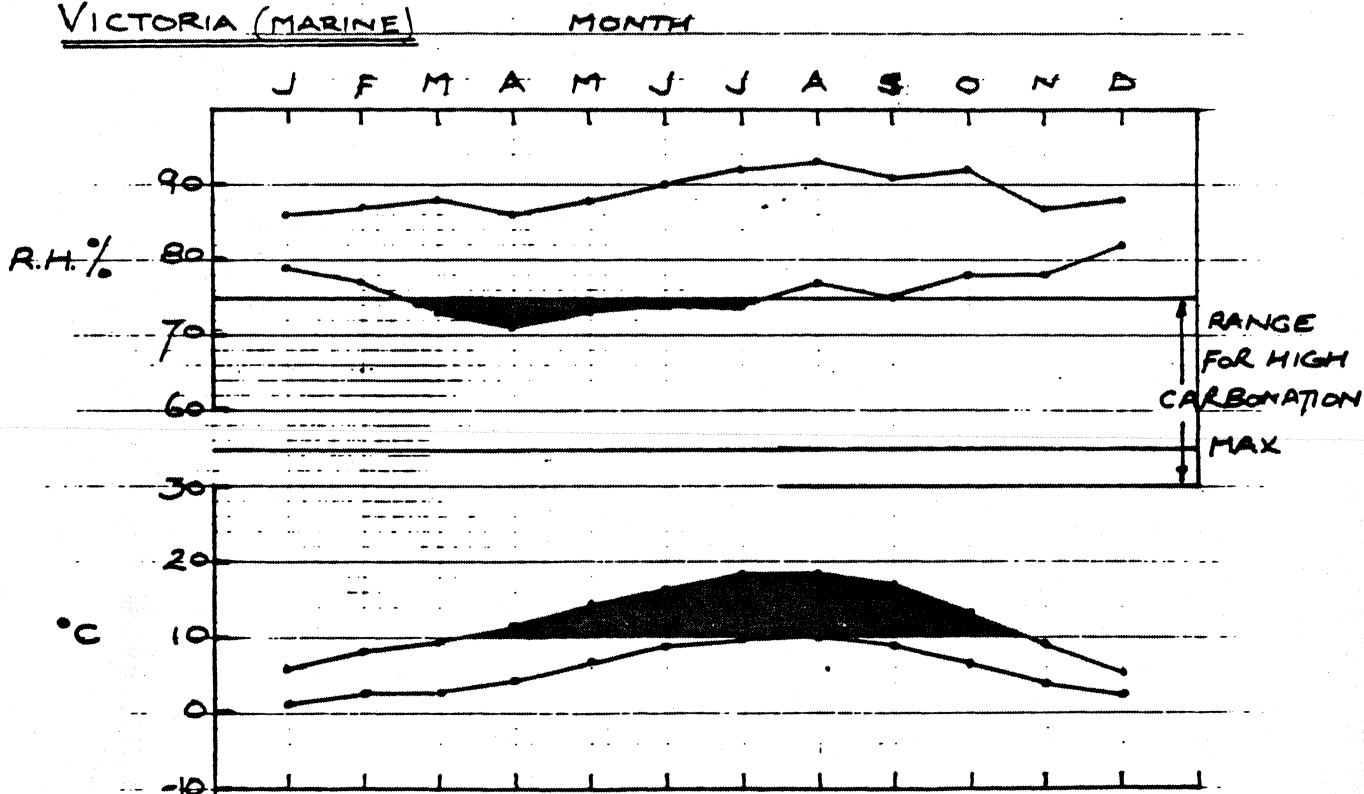
APPENDIX B

**Data Extracted from
Environment Canada Records**

VANCOUVER



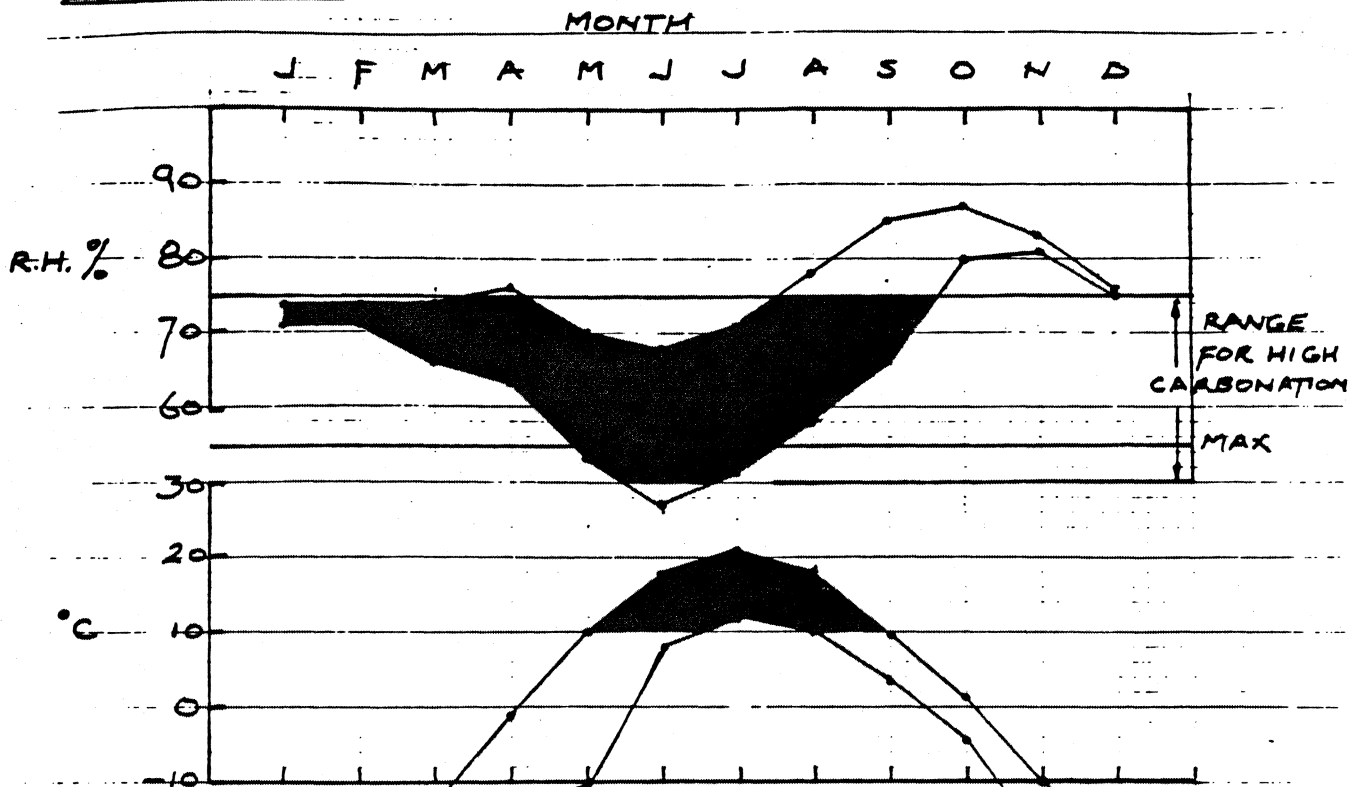
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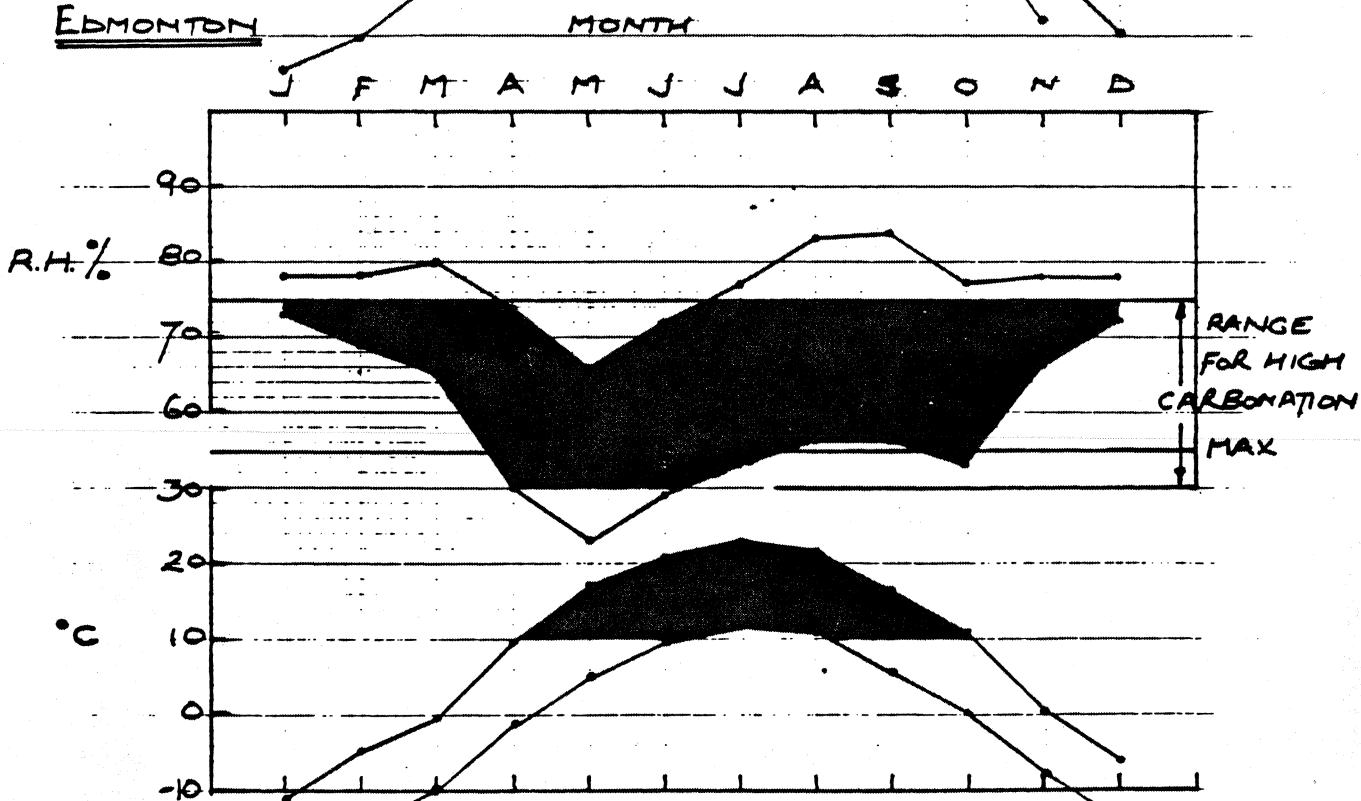
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PROJECT 86/018.

YELLOWKNIFE



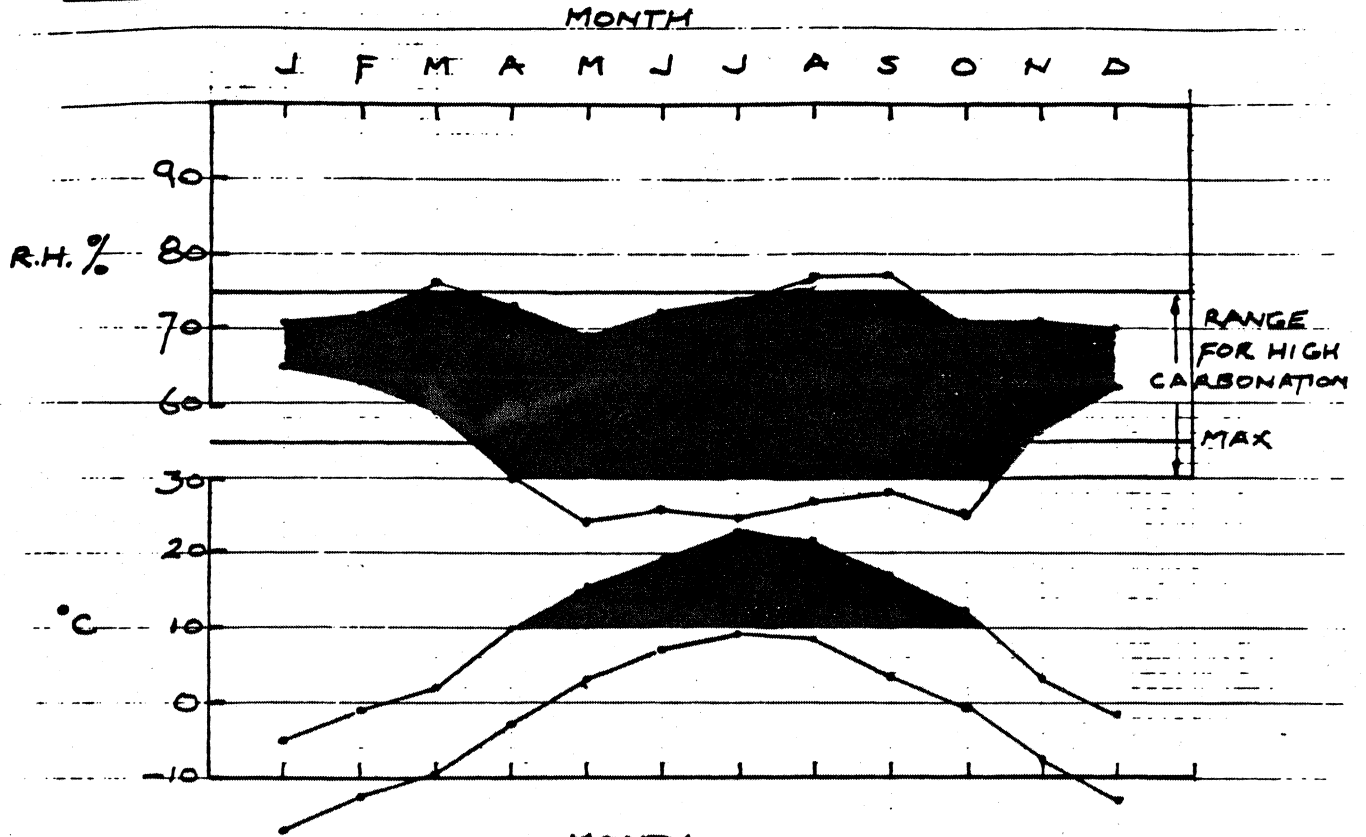
EDMONTON



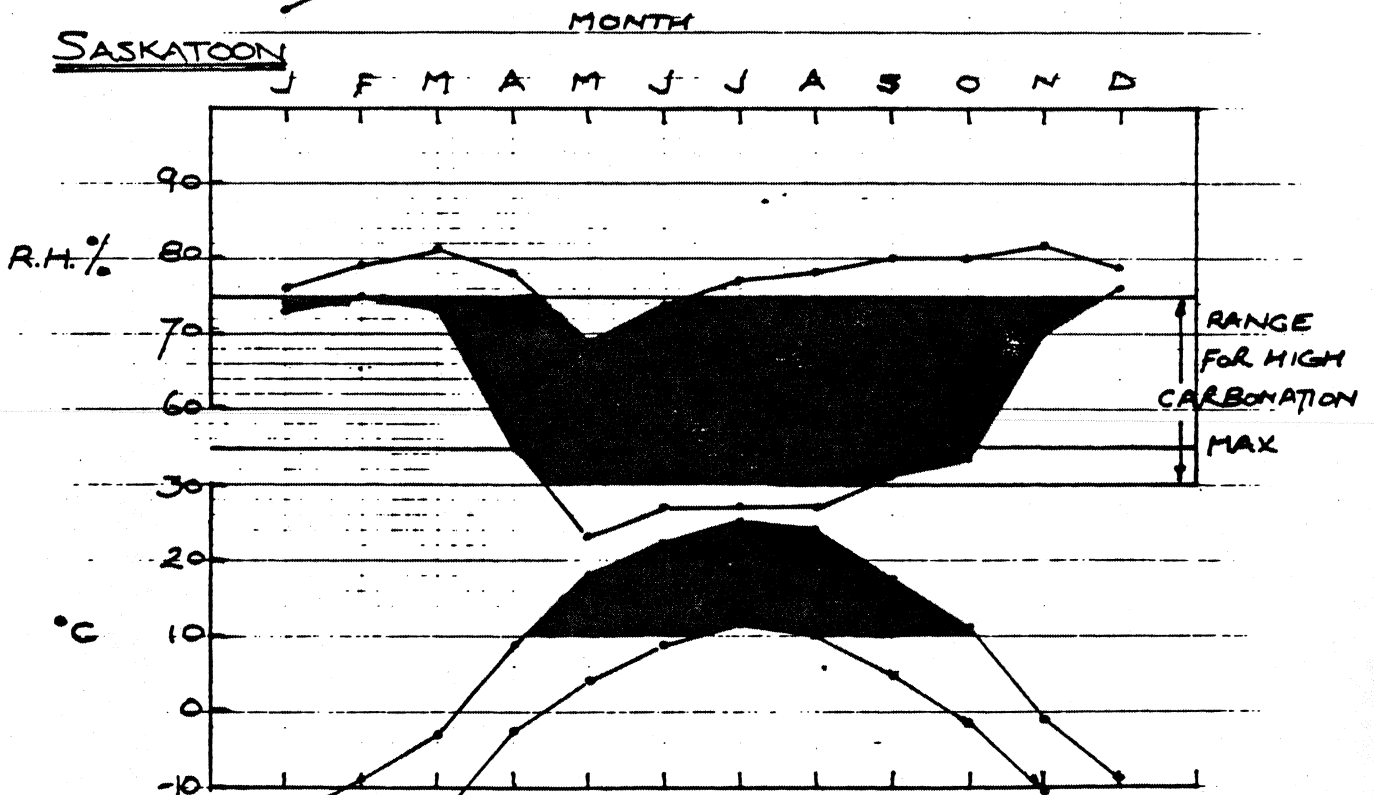
MEAN DAILY MINIMUM AND MAXIMUM R.H. AND
TEMPERATURE.

PROJECT 86/018.

CALGARY



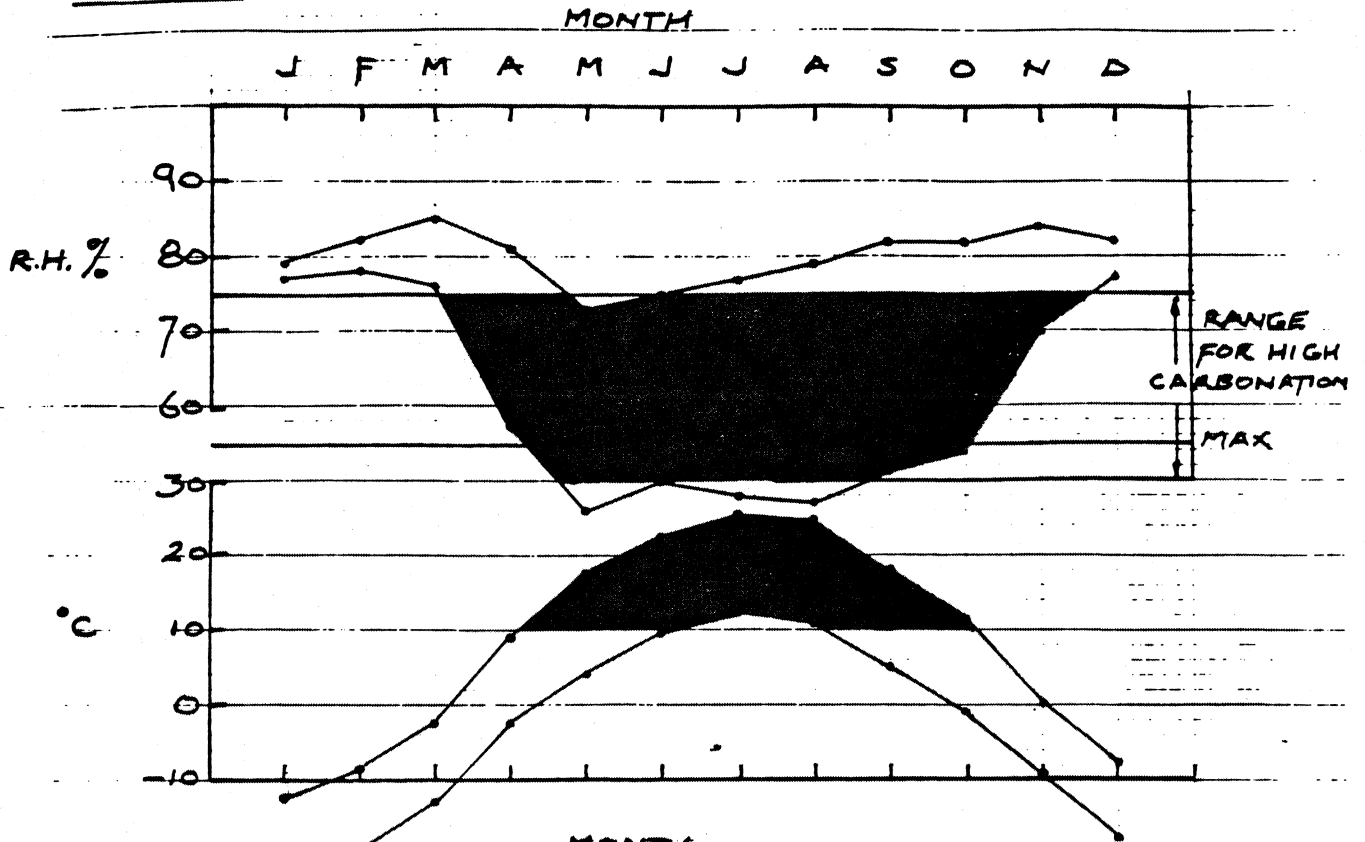
SASKATOON



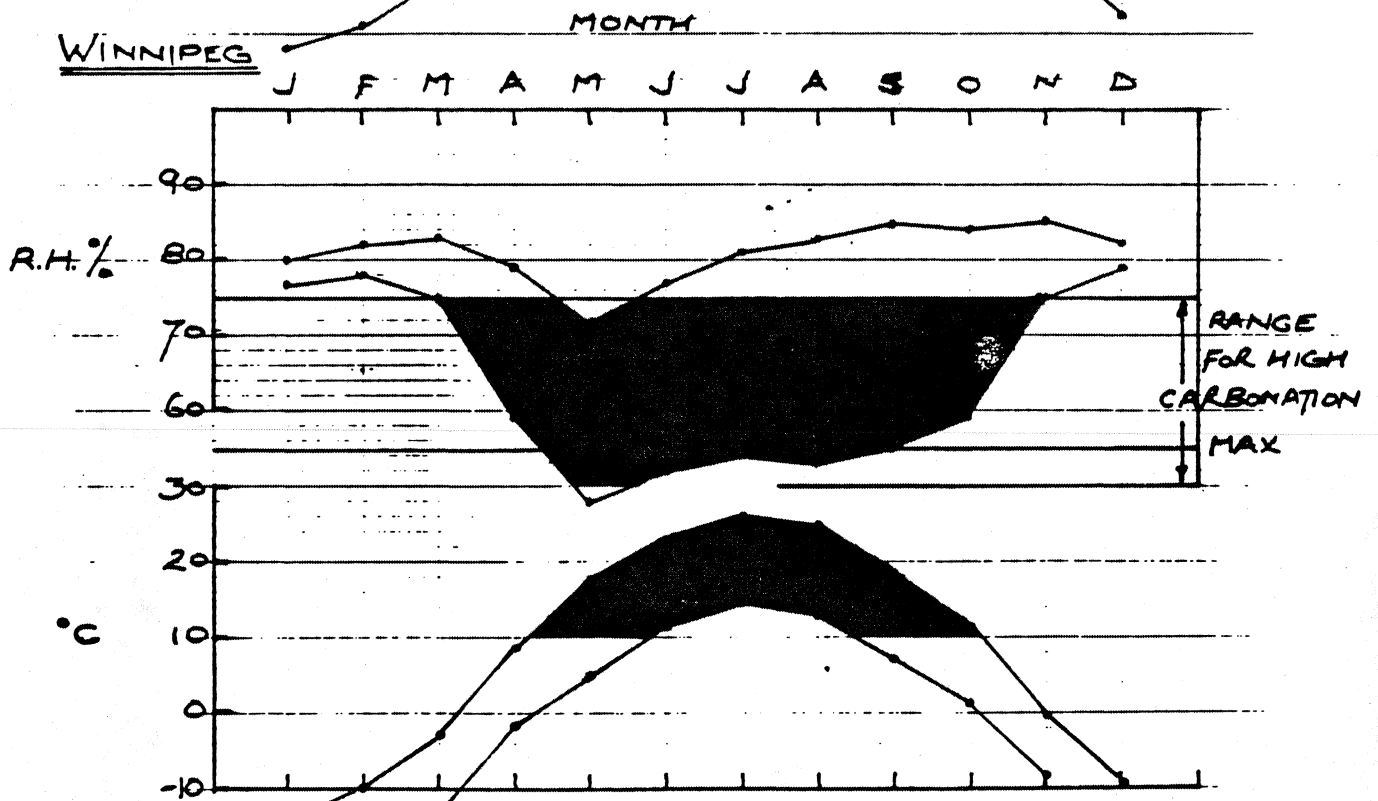
MEAN DAILY MINIMUM AND MAXIMUM R.H. AND TEMPERATURE.

PROJECT 86/018.

REGINA



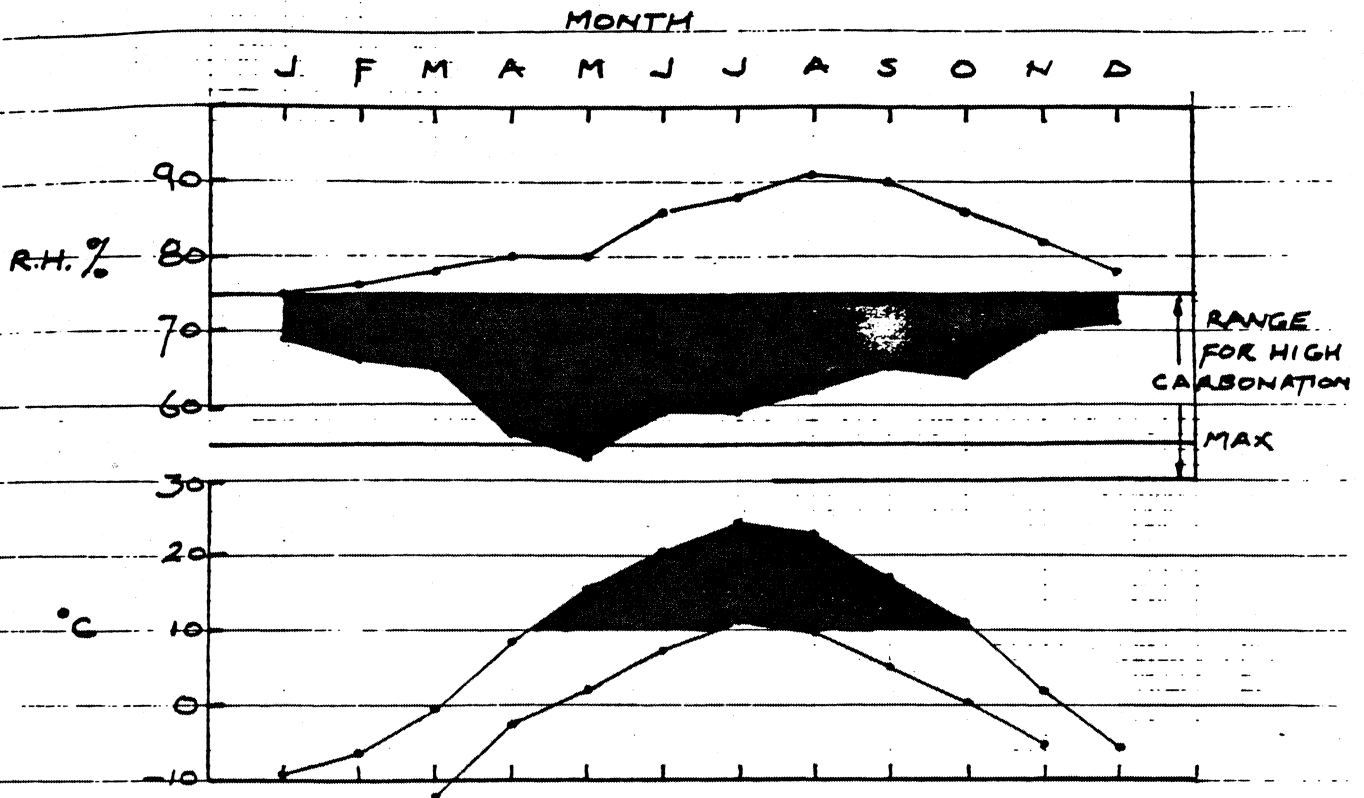
WINNIPEG



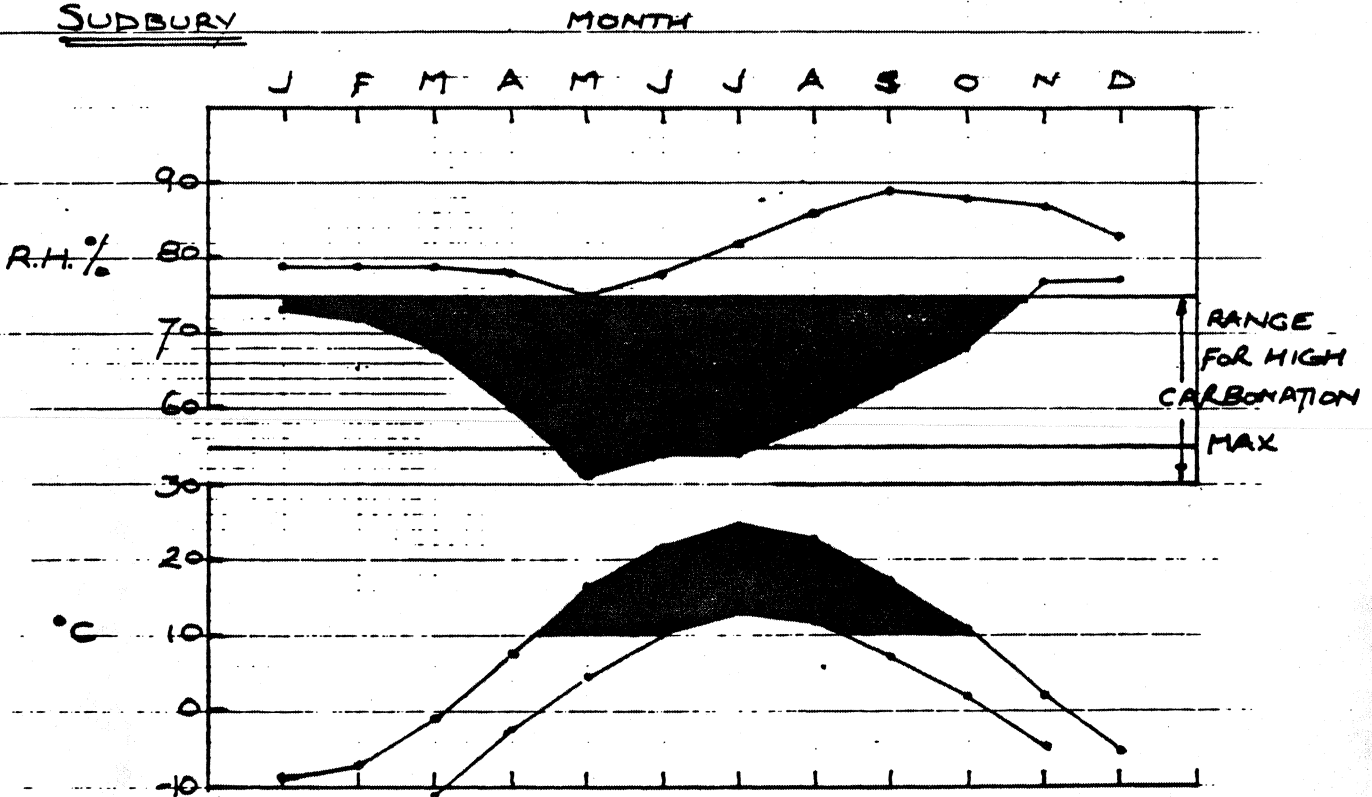
MEAN DAILY MINIMUM AND MAXIMUM R.H. AND TEMPERATURE.

PROJECT 86/018.

THUNDER BAY



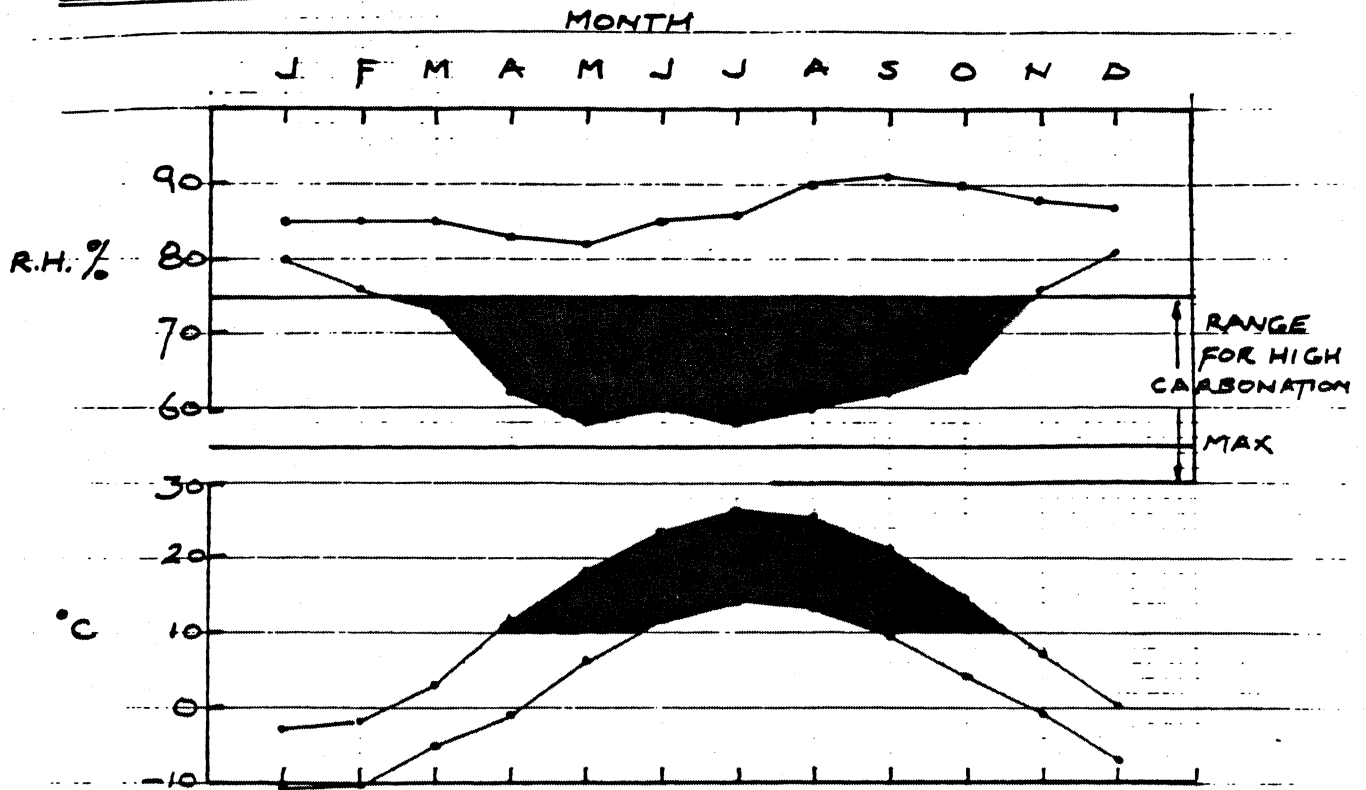
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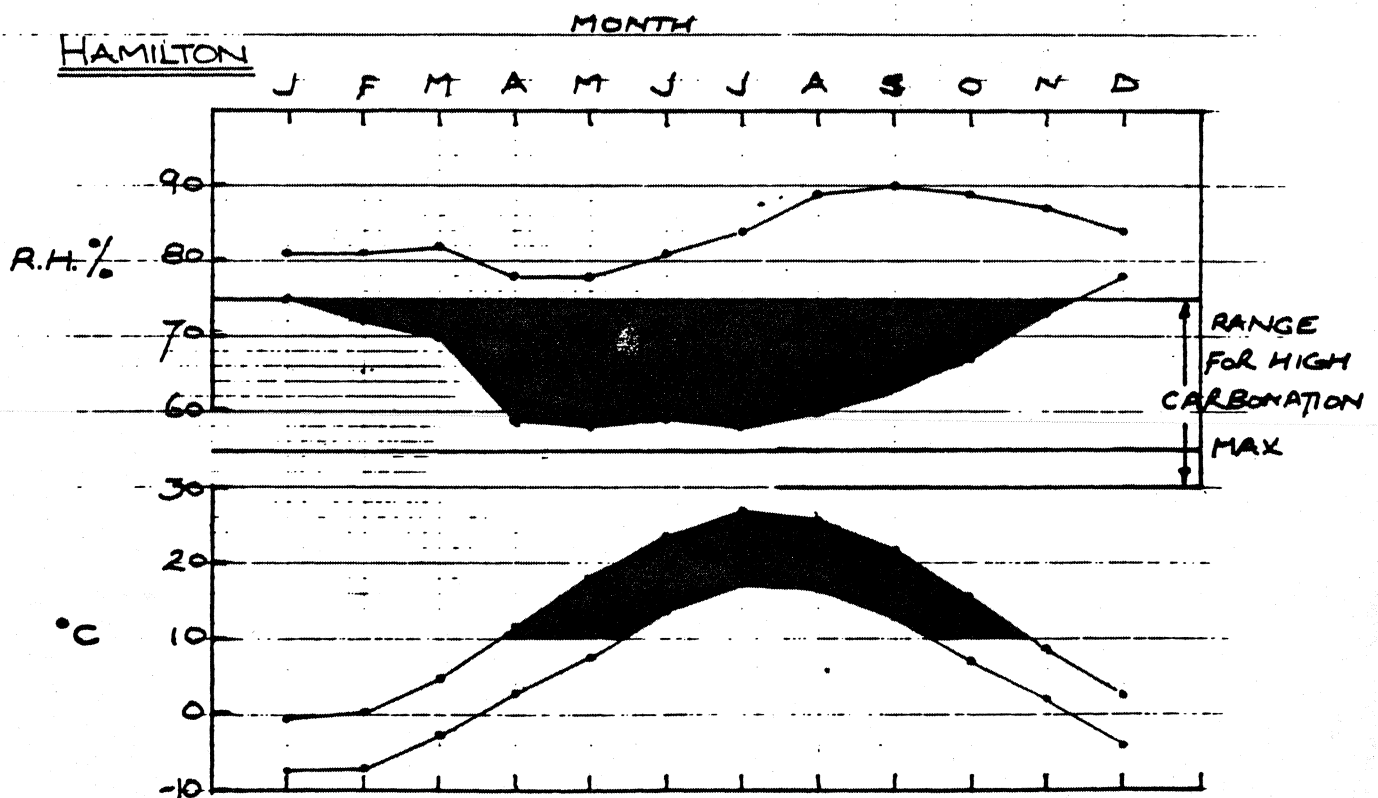
MEAN DAILY MINIMUM AND MAXIMUM R.H. AND TEMPERATURE.

PROJECT 86/018.

LONDON



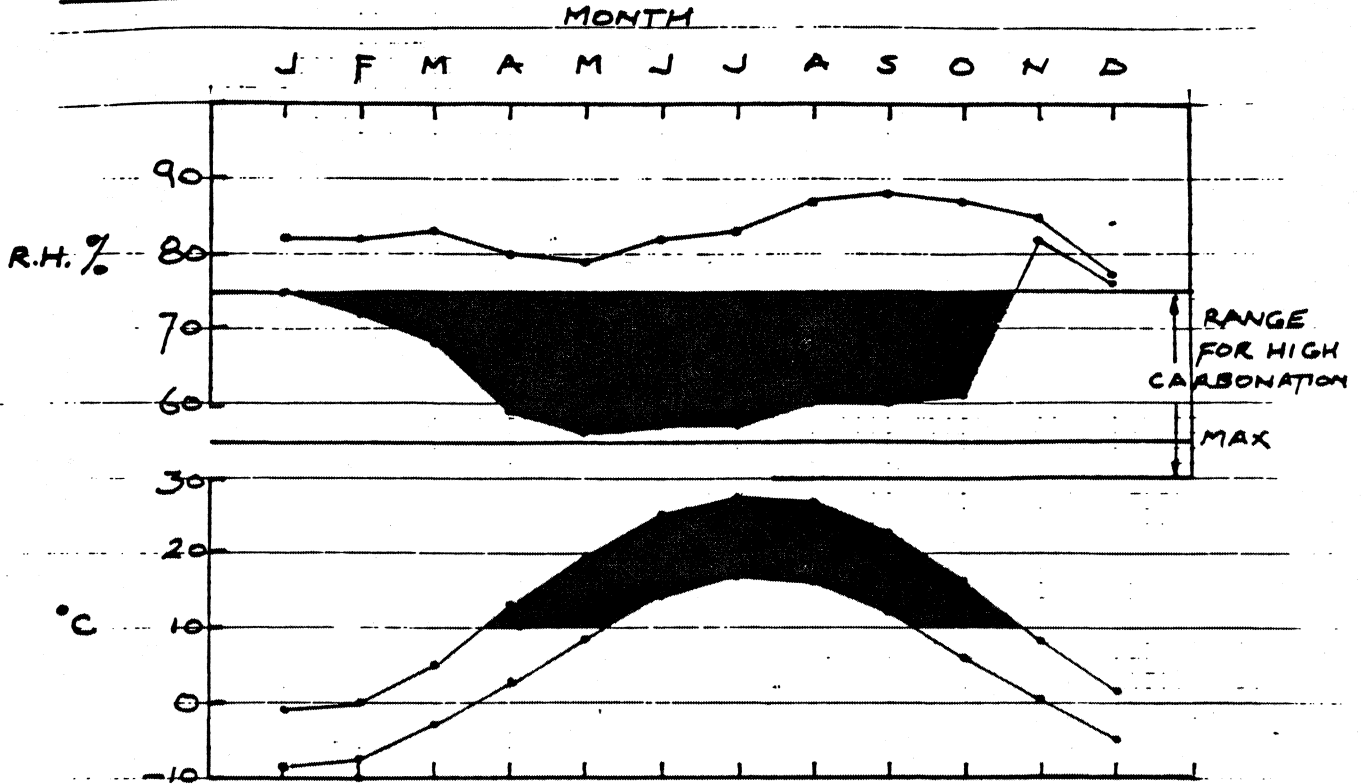
HAMILTON



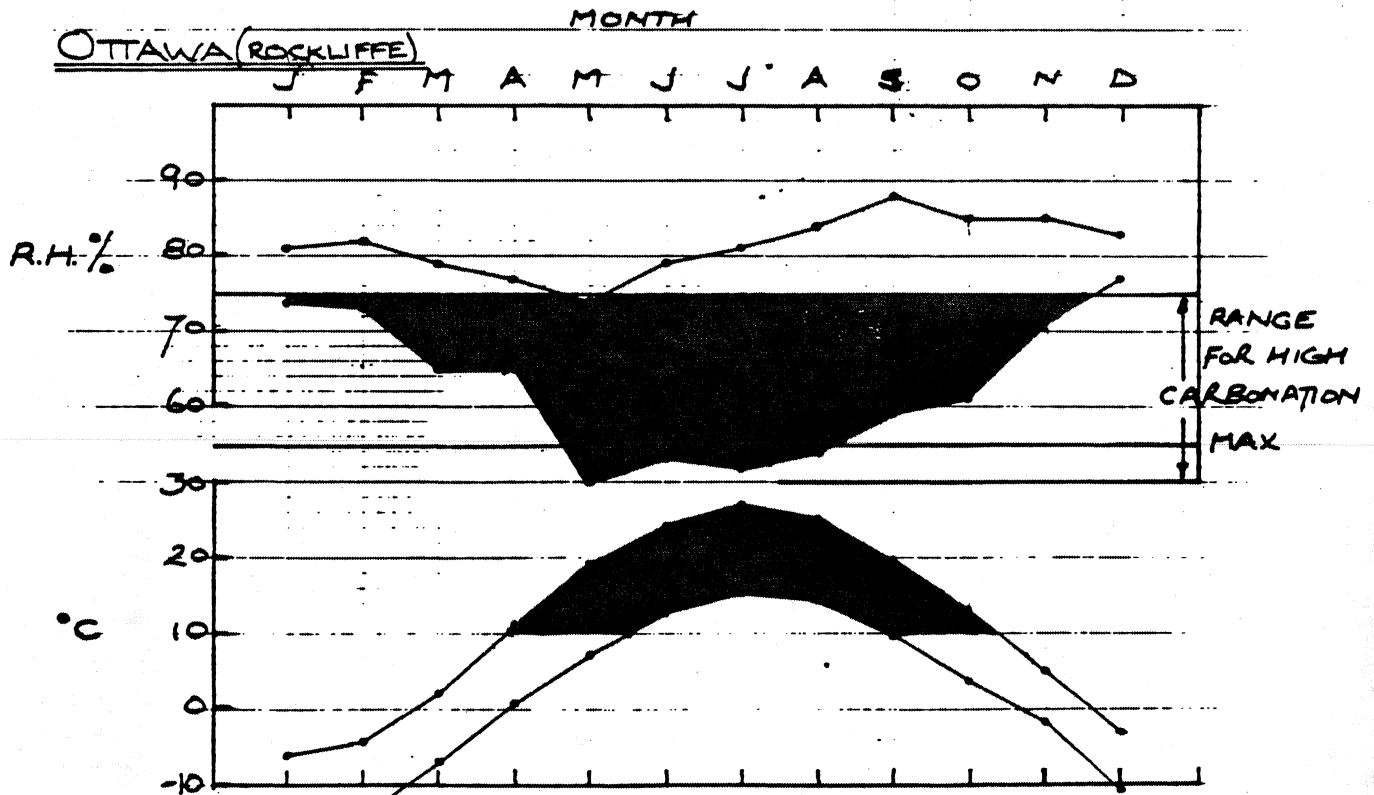
MEAN DAILY MINIMUM AND MAXIMUM R.H. AND
TEMPERATURE.

PROJECT 86/018.

WINDSOR.



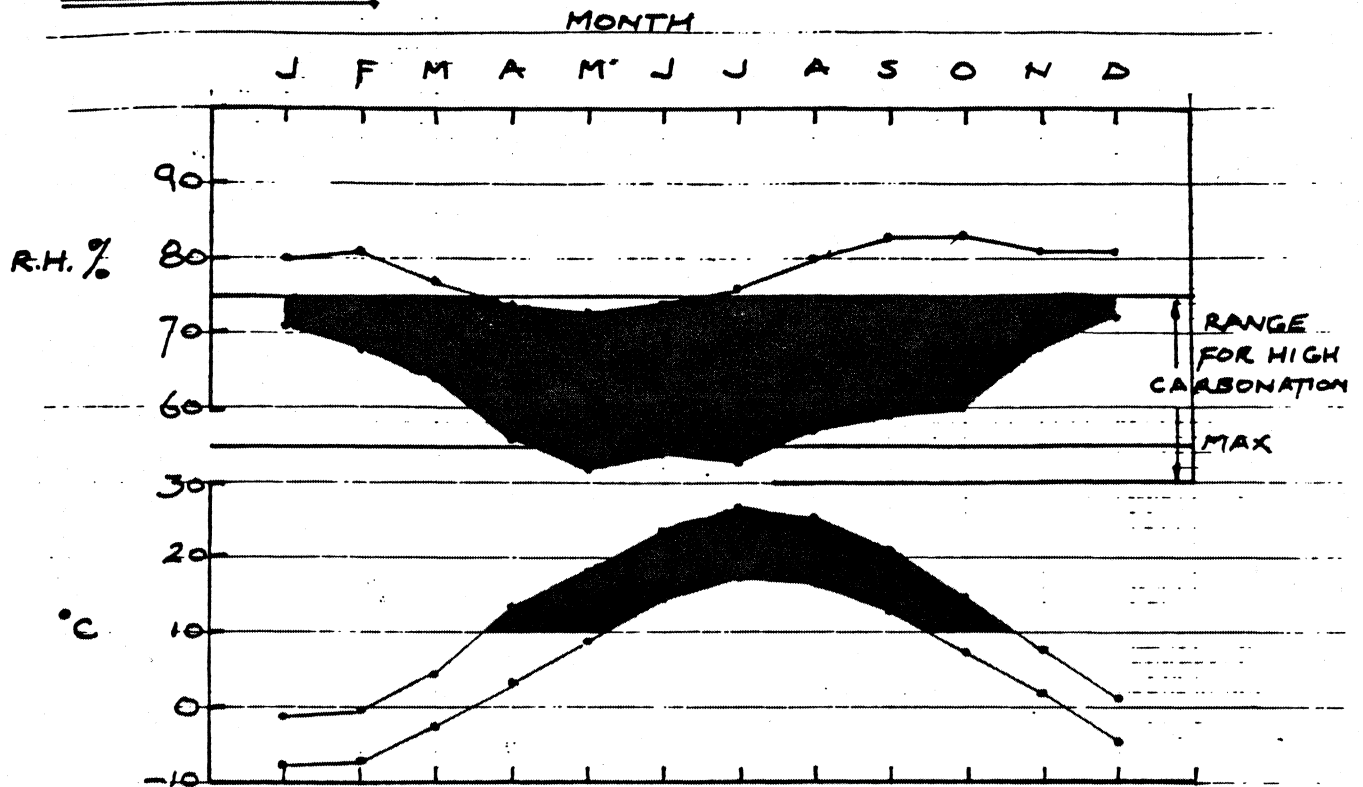
OTTAWA (ROCKLIFFE)



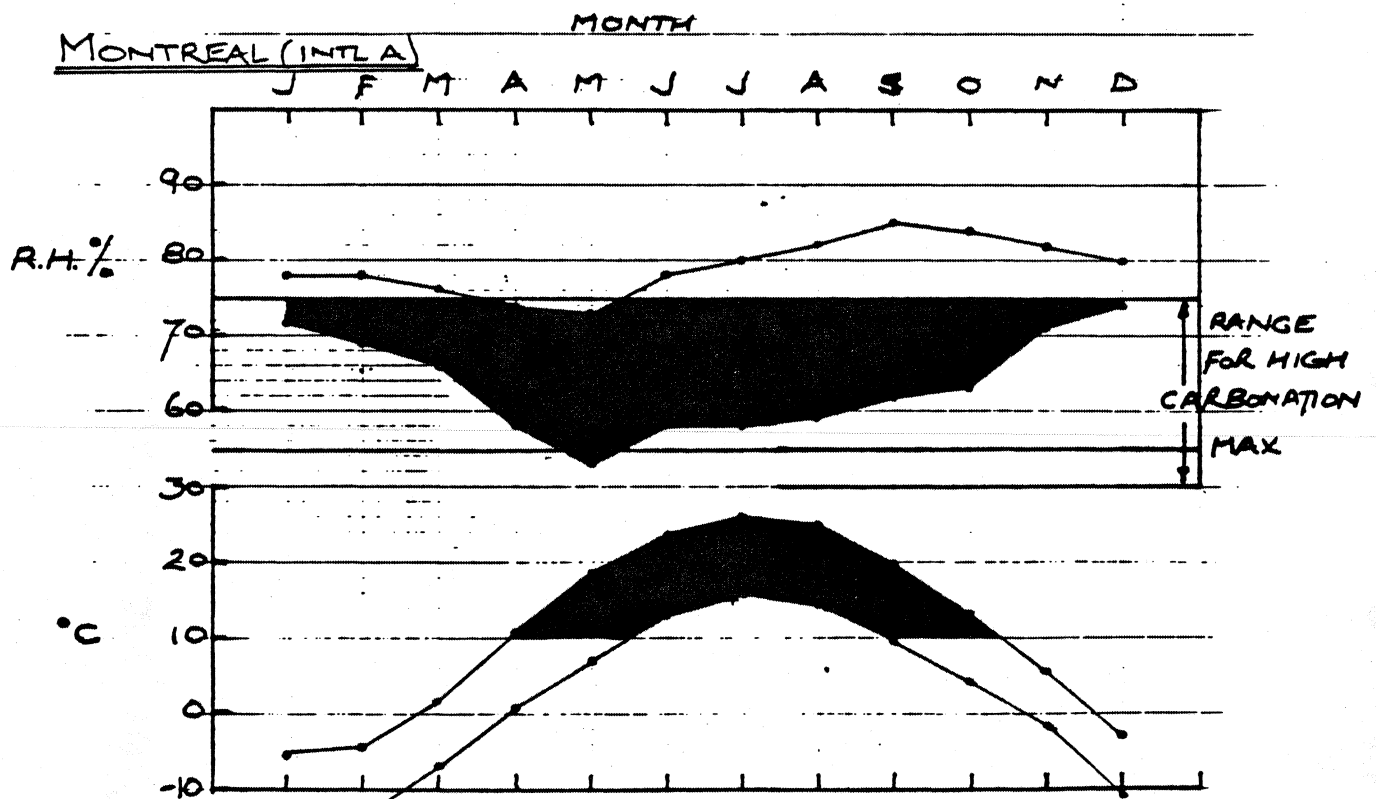
MEAN DAILY MINIMUM AND MAXIMUM R.H. AND TEMPERATURE.

PROJECT 86/018.

TORONTO (INT)



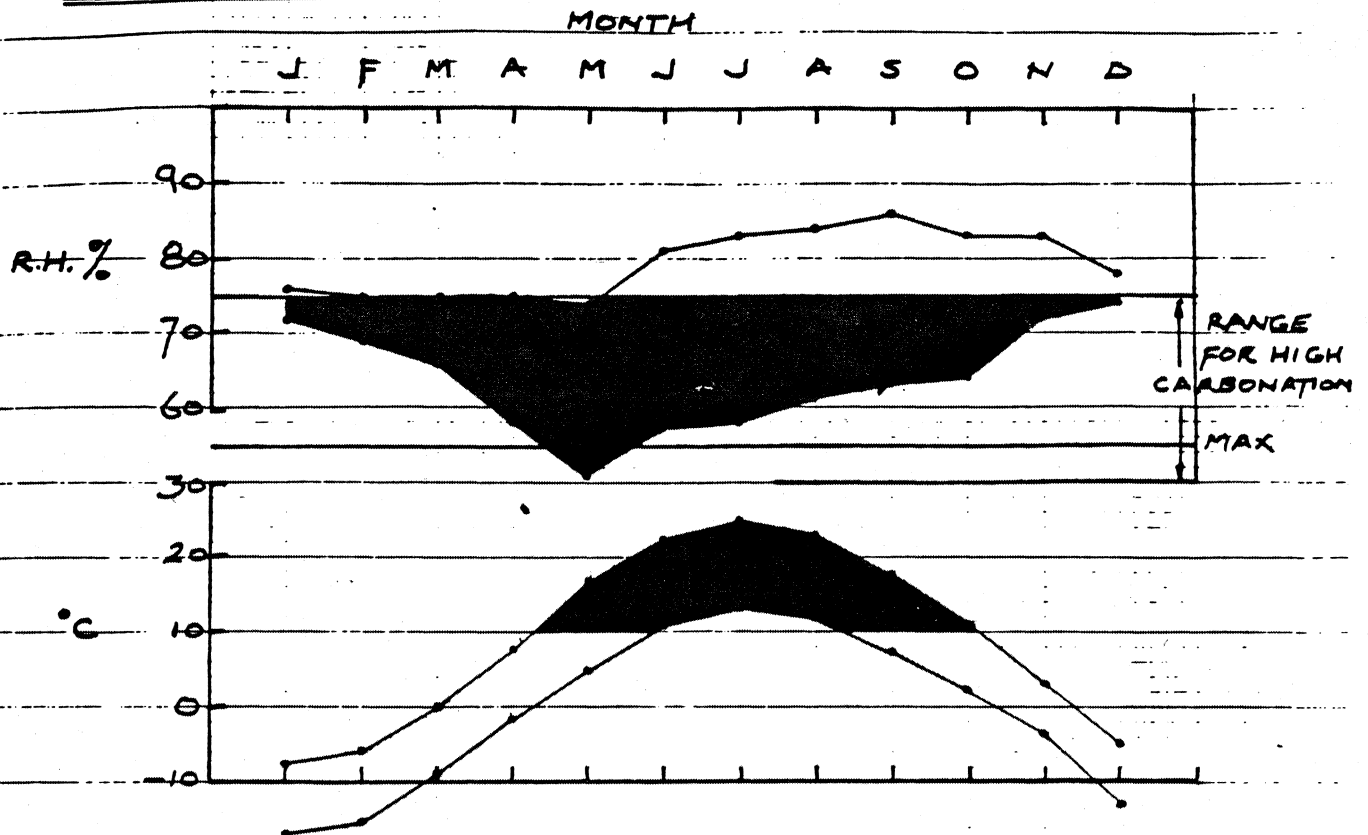
MONTREAL (INTLA)



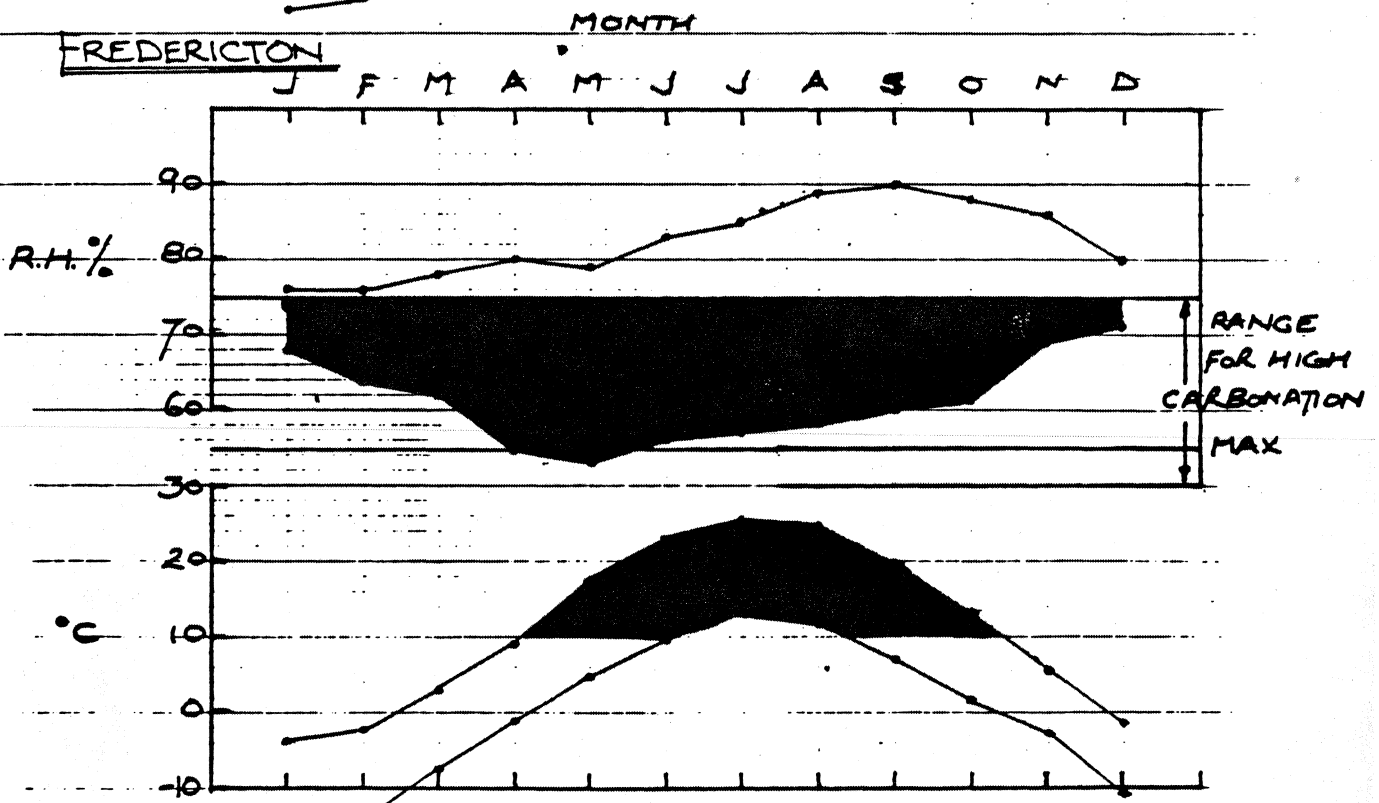
MEAN DAILY MINIMUM AND MAXIMUM R.H. AND TEMPERATURE.

PROJECT 86/018.

QUEBEC CITY



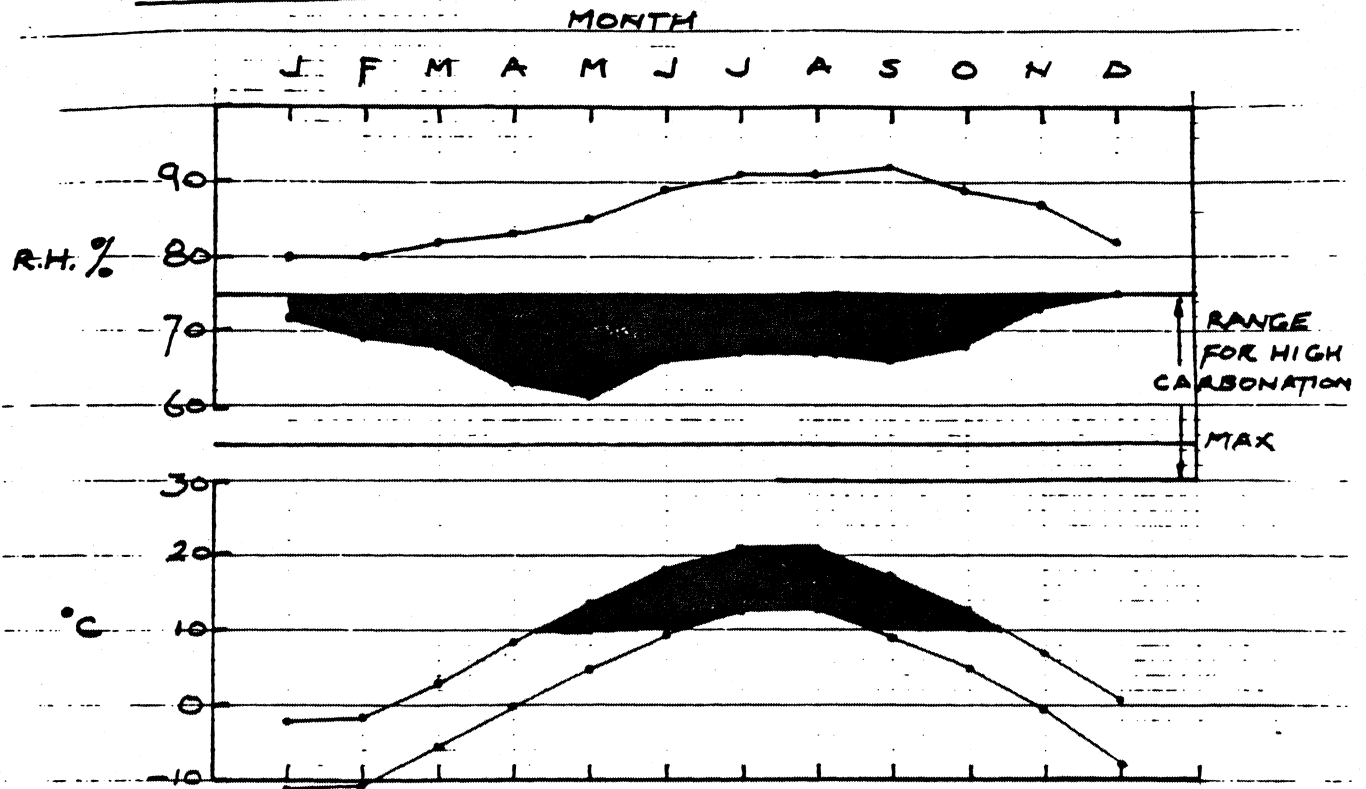
FREDERICTON



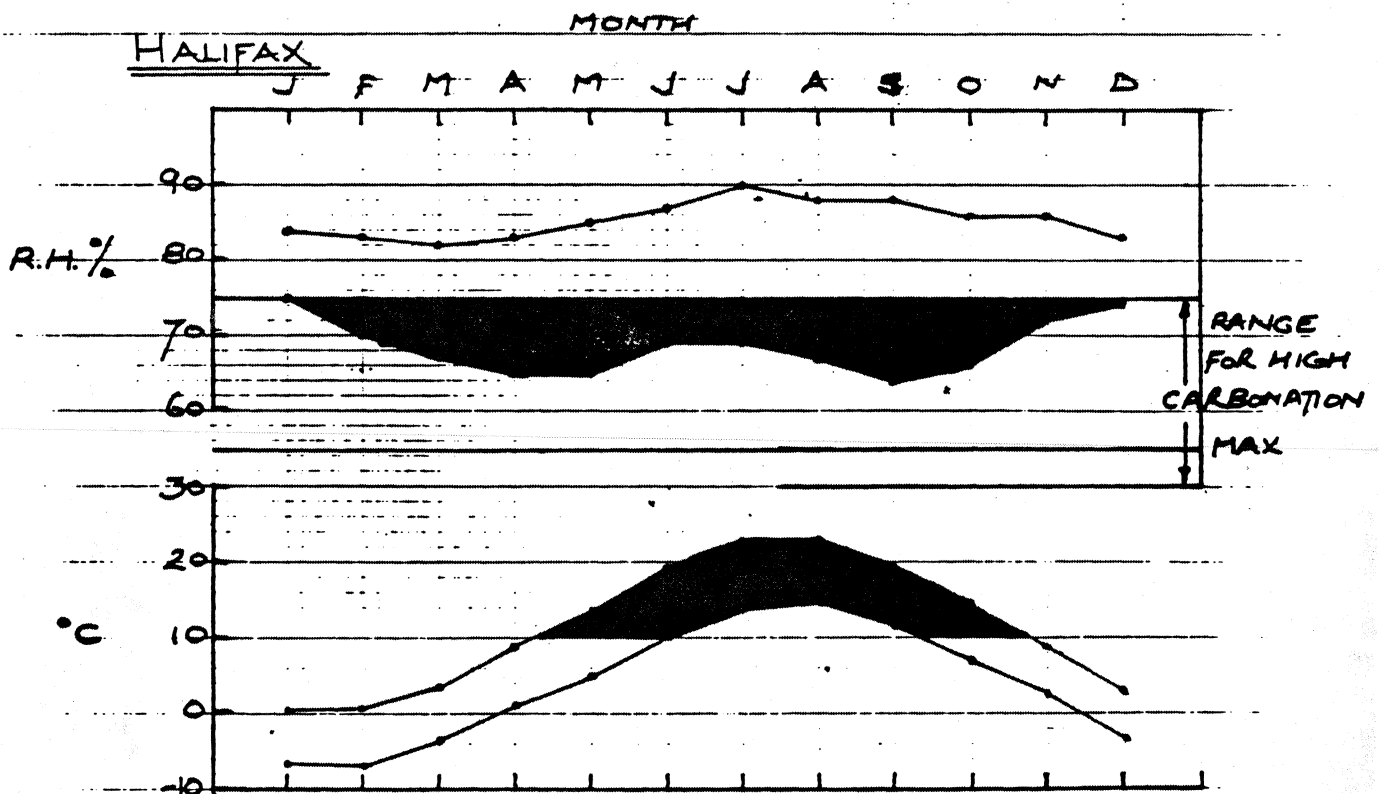
MEAN DAILY MINIMUM AND MAXIMUM R.H. AND TEMPERATURE

PROJECT 86/018.

SAINT JOHN



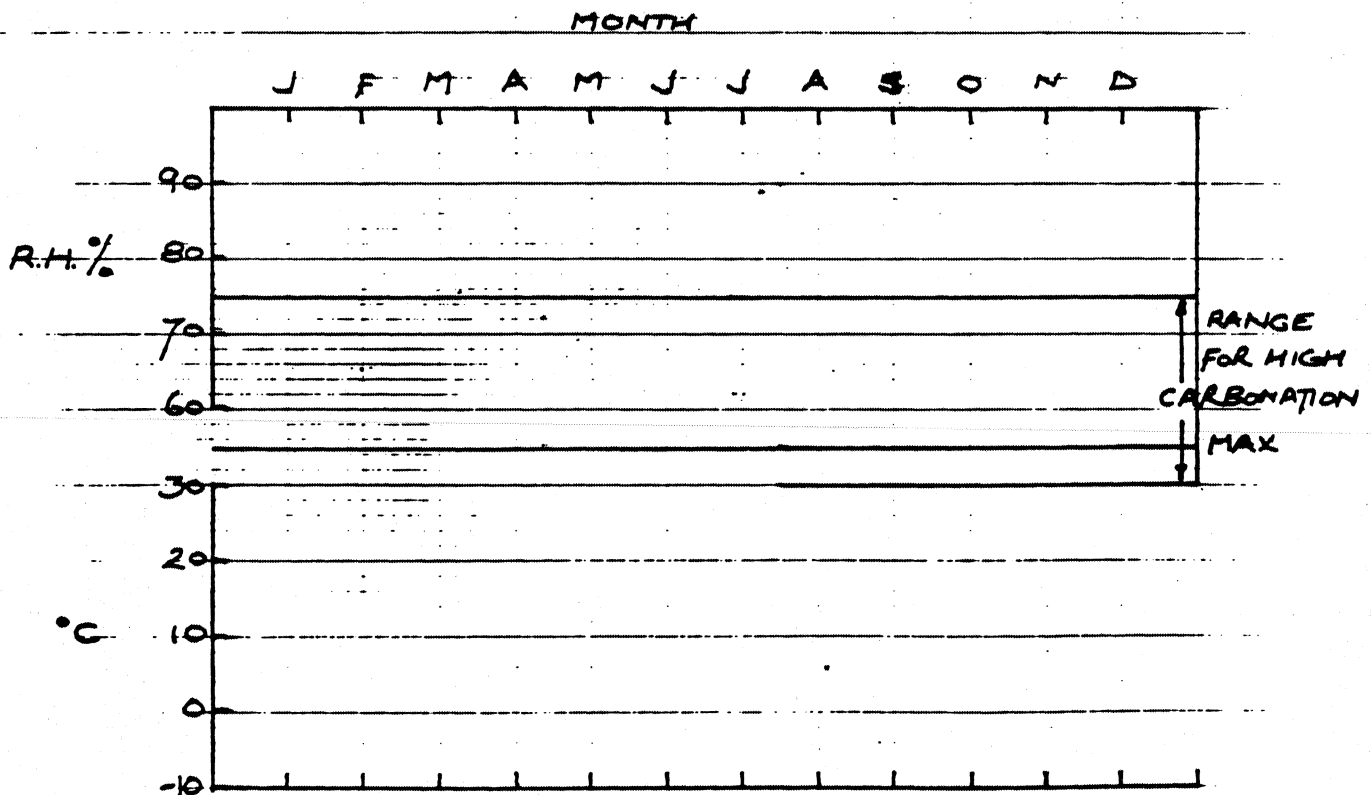
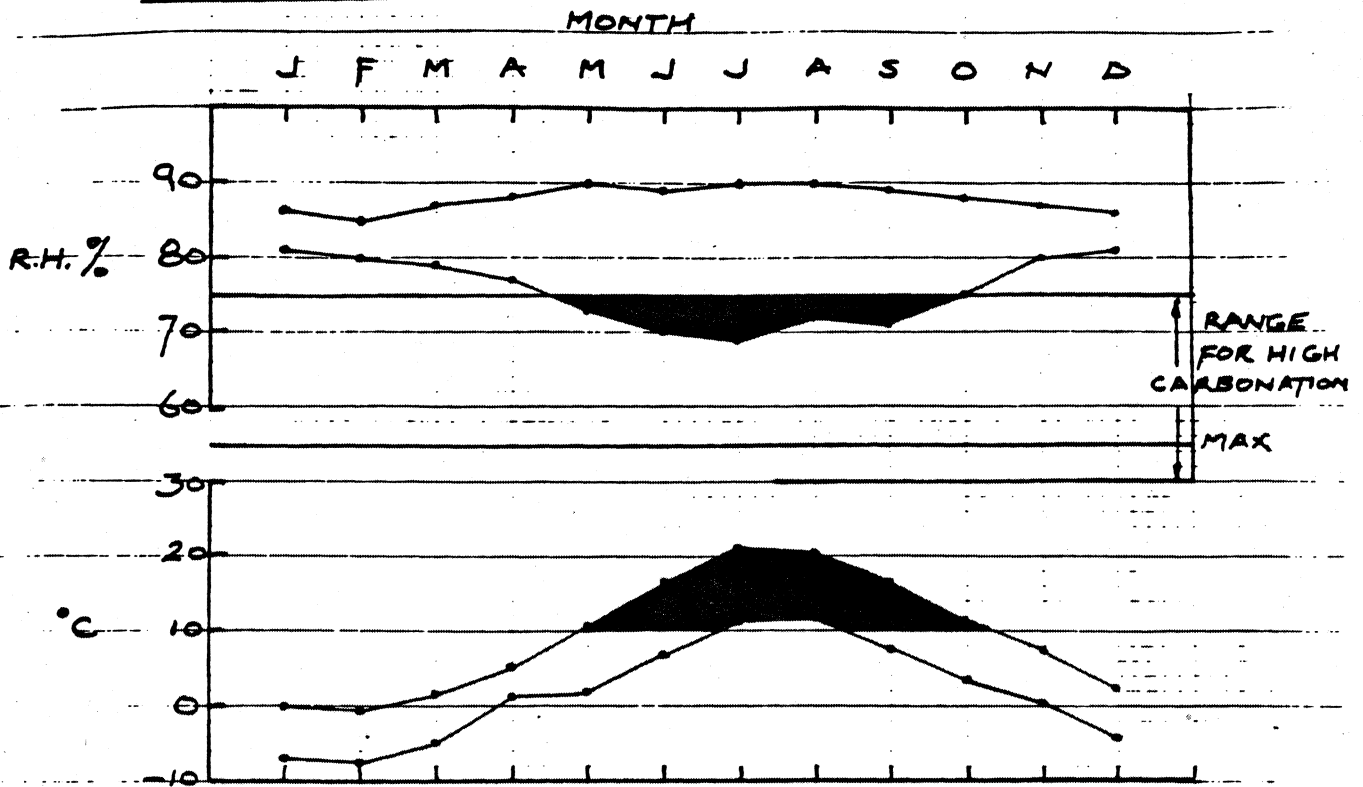
HALIFAX



MEAN DAILY MINIMUM AND MAXIMUM R.H. AND
TEMPERATURE.

PROJECT 86/018.

ST JOHNS



MEAN DAILY MINIMUM AND MAXIMUM R.H. AND TEMPERATURE

PROJECT 86/OIB.