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FIRE RESISTANCE OF NEW ZEALAND CONCRETES - PART 1

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PREFACE

The Building Research Association of New Zealand (BRANZ) undertook the work reported here primarily to establish the validity of current data for the Fire Resistance Ratings of structural concrete. Such data have been derived from overseas where aggregate types may differ significantly from those used in New Zealand. This report covers the fire testing of 130 mm thick unloaded concrete slabs using different types of coarse aggregate. Further work is currently underway primarily investigating the effect of slab thickness. This further work will be reported separately as Part 2.

ACKNOWLEDGEMENT

The authors wish to acknowledge the considerable contribution to this work of Dr Graham Rowe and his staff (Central Laboratories, WORKS, Consultancy services).

This report is intended for researchers in fire engineering, code writers and as a background document for designers and approving authorities.

FIRE RESISTANCE OF NEW ZEALAND CONCRETES - PART 1

BRANZ Study Report SR 34

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REFERENCE

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KEYWORDS

From Construction Industry Thesaurus, BRANZ edition:
Aggregate; BRANZ; Building Research Association of New Zealand; Compressive Strength; Concrete; Damage; Fire; Fire Resistance; Mathematical Models; New Zealand; Non-Destructive Testing; Non-Loadbearing; Rating; Reinforced Concrete; Repairing; Spalling; Strain; Stress; Ultrasonic Pulse Testing; Walls.

ABSTRACT

The thicknesses of concrete elements required to achieve various Fire Resistance Ratings (FRR) in New Zealand are notional and based on overseas building codes and test results. Because of variation in the properties of the constituent aggregates, a test program was undertaken to determine the reliability of the notional ratings when applied to concretes containing locally produced aggregates. The results showed that the current Fire Resistance Ratings used in New Zealand are adequate for the thickness tested. Indicative information has been collected on damage to concrete after firing and a technique for the measurement of residual compressive strength has been evaluated.

Recommendations are made for further work in the extrapolation of data to other concrete thicknesses for both walls and floors; the assessment of the influence of loading, and performance of high strength concrete; and the use of techniques for assessing post-fire loadbearing capacity.

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INTRODUCTION

Fire Resistance Ratings (FRR) for reinforced concrete elements are published in New Zealand by the Standards Association of New Zealand (SANZ, 1989) in MP 9, Fire Properties of Building Materials and Elements of Structure. Such ratings are notional and based upon overseas standards and codes of practice. The current values are largely derived from the Federation International De La Precontrainte/ Comite Euro-International Du Beton publication (FIP/CEB, 1975) and are dependent on overseas knowledge and fire tests of overseas concretes.

Aggregate occupies approximately 80 percent by weight of concrete. It is recognised that, in terms of thermal transmission, the properties of the aggregate have the most influence in determining the performance of the concrete as a whole. Such properties include specific heat, thermal conductivity, crystallinity, moisture content and porosity.

New Zealand is significantly different in geological terms from the European continent, where the FIP/CEB findings were derived, and from North America (where for example Galbreath (1966), refers to "Traprock" and "Gravel" in descriptions of aggregates used in research work. These terms are either not used or have different meanings outside of their country of origin). It was therefore considered necessary to confirm the validity of overseas data as they are applied to local materials. Initial, unreported work by BRANZ determined the experimental parameters for a sensitivity study of the fire resistance performance of a series of reinforced concrete slabs made from local aggregates. It was decided that fire resistance testing of two replicate slabs of 130 mm thickness made from twelve different aggregate sources would provide a representative picture of the relative fire performance of New Zealand concretes.

The testing programme was established to give confidence in the Fire Resistance Ratings being used in New Zealand and whether the FRR of New Zealand concretes were in reasonable agreement with the data, derived from overseas testing, and currently used in the SANZ MP 9 (1989) publication.

Once a relationship had been established between the fire performance of local and overseas concretes, it was anticipated that the local test data could be used to validate numerical heat transfer models in need of verification for New Zealand conditions. An example of a thermal model and its evolution is the TASEF-2 program (Wickstrom, 1979). This model consists of a computer program for temperature analysis of structures exposed to fire. The model was used by O'Meagher and Bennetts (1987) to look specifically at the structural fire performance of load-bearing concrete walls. Munukutla (1989) developed a simpler, one dimensional thermal model and also further developed O'Meagher and Bennetts' structural model to include alternative end conditions. This enabled the model to be more versatile and suitable for New Zealand wall construction methods. Some of the data from the testing programme reported here were used to partially validate Munukutla's thermal model.

Other research projects running concurrently in BRANZ that had influence on the content and direction on this project were; investigating the repairability of structures following fire attack, and the use of fire engineering design methods for determining the fire resistance of concrete elements.

It was also intended that any deficiencies in the knowledge of the fire performance of New Zealand concretes could be identified and recommendations made on the need for further work.

METHOD

The twelve aggregates chosen for this sensitivity study of the fire resistance of New Zealand concretes are typical of those commonly used in New Zealand although neither limestone nor pumice are routinely used in structural concretes. Details of their lithology and source are given in Appendix 1. A full description of the mix design, placing and curing of each concrete slab has been published by Rowe et al (1988). Details of the mix design are reproduced in Appendix 2.

Two slabs, each 1 metre by 1 metre by 130 mm thick, were cast for each concrete type. Each slab incorporated a steel reinforcing grid made of grade 275, 10 mm diameter deformed steel, (Figure 1). 1.4 mm diameter high tensile wires were strung across the mould to support 1.0 mm diameter, sheathed, type K thermocouples at a pre-determined depth within each slab, see Figures 2 and 3.

Concrete was placed into the mould after positioning the reinforcing grid at mid-depth and fastening the sheathed thermocouples to their support wires. Relative Humidity (RH) sensor wells were cast using formers that could be removed after casting. A fully instrumented mould prior to casting is shown in Figure 4.

Lifting lugs were cast into each slab for handling purposes. The lifting lugs and the RH sensor wells, were positioned as far away as possible from each other and any thermocouples, both internal and external, to minimise any effects they could have on heat transmission through the concrete.

After casting and floating, the concrete was left in the mould and covered with plastic for 6 days. The slabs were then demoulded and placed in a water filled bag with their top surfaces covered with wet sand. Twenty eight days after casting, the slabs were unbagged, and allowed to dry in ambient conditions. Curing temperatures ranged from 6°C to 25°C. Cylinders were cast from each batch for measurement of their compressive strength after 28 days of curing at 20°C ± 2°C and 100% Relative Humidity.

Drying and Relative Humidity Measurement

The slabs were stored vertically, spaced from each other and dried in ambient conditions, see Figure 5. Relative humidity in the atmosphere of the storage area ranged from 50 to 90 percent and temperature from 4°C to 25°C.

The R.H. sensor was a stretched membrane coated on both sides with gold; the moisture sensitive membrane forming a dielectric and the gold forming the electrodes of a capacitor, see Figure 6.

Each slab had two R.H. sensors installed in wells reaching to mid depth. The sensors were sealed into each well with silicone rubber sealant, as shown in Figure 7.

Each R.H. sensor was connected to a 50 channel terminal box which converted the capacitance into a d.c. voltage to give a direct reading of R.H. The system has an inherent reading error within ± 2.5 per cent.

Fire Test Method

The concretes were tested in the 1.0 m by 2.2 m diesel-fired pilot furnace at BRANZ Laboratories, Judgeford, (Figure 8). Testing to ISO 834 (1975); Fire Resistance Tests - Elements of Building Construction, was carried out between December 1988 and July 1989.

The specimens were tested unloaded in a vertical orientation and fastened to a frame using four bolts, two on each side at approximately one quarter and three quarters height. These bolts were inserted into pre-drilled holes in the side of each specimen and through the steel frame of the specimen holder. They were thus partially restrained against thermal expansion but not to the extent where such restraint would significantly influence their fire performance.

Thirteen tests were run in total, one for each of the twelve aggregate types and a repeat firing of the pumice aggregate concrete to evaluate the effect of firing a concrete specimen a second time.

Temperature Measurement

The unexposed surface temperature was measured to determine the fire resistance according to the procedure outlined in ISO 834 (1975). This gives a 140°C rise in average temperature or a 180°C rise in maximum temperature of the unexposed face as the criteria for an insulation failure. The other criteria for failure are integrity and structural adequacy which do not usually occur before insulation failure for unloaded concrete elements.

Five type K chromel/alumel disc thermocouples were fixed to the unexposed face of each slab in accordance with ISO 834. Each specimen also had five sheathed "internal" thermocouples placed at various depths for measuring temperatures within the concrete slab (as shown in Figures 2 and 3). These internal thermocouples were fixed to high tensile wire strung parallel to the face and located to within ± 1 mm of the specified depth. The reinforcing bars were instrumented with type K thermocouple wire joined with a "quick tip" at the hot junction and fastened to the steel with epoxy resin as shown in Figure 3.

A thermocouple made from a "quick-tip" connection was attached to the fire-exposed face in a slight indentation. This was held in place by a washer pressing on to the ceramic insulator carrying the thermocouple wires. The washer was screwed to the hot face using a 6 mm "Dynabolt".

Deflection Measurement

Three wires were strung from the specimen holder across the unexposed face of the specimens at mid-height and at each quarter. The distance of each specimen from each wire was measured at 15 minute intervals during each test. This gave an indication of how much each specimen deflected in the vertical plane as a result of thermal bowing during the firing.

Datalogging

The thermocouples were connected to a Hewlett-Packard HP 3497 data acquisition unit (DAU) capable of monitoring the 40 thermocouple channels used every 30 seconds. The DAU was controlled by a Hewlett-Packard series 9000 computer.

Elastic Modulus of Fire Damaged Concrete

A full description is given by Rowe et al (1989) of the preparation of one of the fire tested slabs (Quarried Greywacke(2) Appendix 2) and the subsequent evaluation of its static and dynamic modulus of elasticity.

Briefly, the slab was cut into pieces (see Appendix 3) and the dynamic modulus determined with depth by measuring the ultrasonic pulse velocity, using the commercially available "Pundit" instrument with 50 kHz transducers. The static modulus was measured using a standard strain-gauging technique with concrete cores loaded in compression at a constant rate of 15 MPa per minute until compressive failure occurred.

RESULTS

Drying Rates

Figure 9 shows the average drying rate of each concrete slab as measured prior to fire testing, expressed as the percentage decrease in mid-depth relative humidity per week.

Temperature Measurements

Figures 10 - 48 show temperatures measured on and within the concrete slabs of each aggregate type, with Figures 19, 20 and 21 representing the repeat test on the pumice aggregate concrete. All values for Unexposed Face Temperatures are the average of the ten thermocouples (five on each slab) of each concrete type tested. The Exposed Face temperatures are the average of two thermocouples, one on each slab, fastened to the hot face. The Rebar Temperatures are the average of twelve thermocouples, six on each reinforcing grid, in each slab. The internal thermocouple temperatures are the average of the two thermocouples, one at each depth in each slab. The furnace average is the average of the four furnace thermocouples.

Figure 49 shows the mean time for the thermocouples on the unexposed face of each aggregate type to reach a 140°C temperature rise. The 95% confidence band derived from the readings of the (up to ten) thermocouples is shown as horizontal lines on this graph. All results were corrected to a standard condition of 75% mid depth relative humidity as discussed later. In no case did any individual thermocouple reach the failure temperature of 180°C before the average rise of 140°C was reached.

Deflection Due to Thermal Expansion

Table 1 lists the average maximum deflection measured for the slabs of each concrete type during fire testing.

Elastic Modulus

Table 2 shows the results of both static and dynamic modulus measurements on cores taken from the fire-tested Quarried Greywacke (2) slab.

Table 3 shows results of ultrasonic pulse velocity measurements on the same slab, (locations of measurement shown in Appendix 3).

Note: Tables 2 and 3 and Appendix 3 have been extracted from Rowe et al (1989).

DISCUSSION

Fire Resistance Ratings Recommended by Codes and Standards

FRR for concrete elements have traditionally been listed in approval documents according to results achieved by test or calculation or through being accorded a generic or notional value. Examples are shown in Figures 50 - 52 of non-loadbearing data used in selected countries, (Canada, USA, UK, Europe and New Zealand) from NRCC (1985), ICBO (1988), CEB (1987) and BSI (1985). Such countries, and others, vary in their specification of the concrete, for example, the British Standard;- BS 8110: Part 2; 1985 Table 4.6, refers to "dense concrete" as being made from aggregate of up to 2400 kg/m^3 and "lightweight concrete" as being up to 1200 kg/m^3. The Canadian national building code refers to, "density of up to 2400 kg/m^3"; and also to "Type S concrete -(granite, quartzite, siliceous)". Although some other countries give separate data on load-bearing walls and non-loadbearing walls and slabs and in some cases include limits on slenderness ratio, the New Zealand standard makes no distinction between load-bearing and non-loadbearing elements. For this reason, only those data which relate to non-loadbearing, concrete walls made from siliceous, limestone and pumice aggregate concretes (as per current descriptions in MP9 (SANZ 1989)) were compared using what was considered to be the equivalent or most appropriate concretes included in the overseas documents. The New Zealand aggregates used, were measured for density at Central Laboratories and the results are included in Appendix 2.

Mixes were nominally identical except for the coarse aggregates. The sand used in all specimens was composed of "greywacke" materials. The thermal transmission effect of the fine sand used in the concrete specimens would be nominally constant in each different slab tested. The main path for thermal transmission is through the coarse aggregate as the fine aggregate is expected to provide a more insulated path with air surrounding each particle.

Fire Test Data for New Zealand Concretes

Overseas data of FRR, published in National Standards, together with that recommended by MP9 (SANZ, 1989) for non-loadbearing, dense aggregate reinforced concrete walls, was interpolated and plotted for 130 mm thickness in Figures 53 - 55. These are plotted with the fire resistance times measured by test in this work to show a comparison. The values used for the time to reach a 140°C temperature rise determined in this study have been corrected to a standard 75 percent mid depth RH according to the method of ASTM E 119, developed principally from the work of Harmathy (1965). In practice (see Table 4) this correction made negligible difference to the measured fire resistance at "non-standard" moisture contents. The correction has been made to the BRANZ data to give the most accurate correlation with data from overseas.

It can be seen, that for the thickness tested, the New Zealand concretes in all cases performed better than the ratings for equivalent overseas concretes and better also than the current notional approvals listed in MP9 (SANZ,1989).

Significance of New Zealand Results

To give a statistical validation, each aggregate tested was used in two separate slabs. Each slab had 5 "key" thermocouples as required by the test standard.

To calculate a 95% confidence level in the time to reach a 140°C temperature rise, the individual thermocouples (up to a maximum of ten) were used to calculate a mean time for a 140°C rise and the standard error in the mean calculated. Then using 'T' value tables a 95% level of confidence in the mean was calculated. Figure 49 shows the mean time to reach a 140°C rise for a 130 mm thick concrete slab made up with each aggregate type. The bars extend to the 95% confidence limits to which these results are likely to be repeatable.

Table 5 details the mean fire resistance for each aggregate, the standard error of this mean, the 95% confidence level for the fire resistance time and this fire resistance time adjusted for the variation in relative humidity at testing.

Extrapolation of Results to Different Slab Thickness

Abrams and Gustaferro (1968) established good correlation between experimental data and "fire endurance" (equivalent to fire resistance for the purposes of this comparison) as calculated by the expression:

$$R = Ct^{n}$$

where:
R = fire resistance
t = slab thickness
C,n = constants determined from least squares analysis of their test data.

The constants were assigned values depending upon concrete aggregate type, viz., Carbonate, Siliceous or Sanded Expanded Shale and are given in their method for calculating the Fire Resistance. When further experimental studies on concrete of different thicknesses have been done it will be possible to determine more accurately constants applicable to New Zealand aggregates.

The concrete slabs containing the ten siliceous aggregate types used in the present work, yielded on average, (with Abrams and Gustaferro's (1968) values in parenthesis after) concrete of; water-to-cement ratio of 0.57 (0.55), 37 MPa 28-day compressive strength (28 MPa) and 76 percent mid-depth RH (75%) at test time.

In the present study the mean (corrected) fire resistance for the ten siliceous aggregate concretes tested at 130 mm thickness was 181 minutes with a standard deviation of 18 minutes. The results were consistently better than data used by selected overseas countries. MP9 recommends 120 mm thickness of concrete for a 2 hour FRR and 150 mm thickness for a 3 hour FRR. Interpolation on a graph of FRR versus concrete thickness from MP9 (SANZ, 1989) gives a FRR of about 138 minutes for a 130 mm thickness as was tested at BRANZ. Table 5 gives the range, at 95% confidence level, that the mean fire resistance falls within for each aggregate. The worst performing aggregate is Alluvial Quartz, which achieved a mean fire resistance time (corrected to 75% mid depth R.H.) of 154 minutes, and a 151 minutes minimum fire resistance time at 95% confidence level. This is 9% longer than MP9 would indicate.

In order for current thicknesses in MP9 to be reduced it is likely that the "normal weight" or "Siliceous" aggregate concretes would need to be divided into subgroups. An analysis of variance procedure using Duncan's multiple range test, to compare the uncorrected mean values of times to 140°C rises, groups the aggregates as not having significantly different mean values as follows:

Group A: Pumice

Group B: Alluvial Andesite, Limestone

Group C: Rhyolite, Quarried Andesite, Phonolite

Group D: Quarried Dacite, Quarried Basalt

Group E: Quarried Basalt, Quarried Greywacke(2), Quarried Greywacke(1)

Group F: Quarried Greywacke(1), Alluvial Greywacke

Group G: Alluvial Quartz

Quarried Basalt is included in both groups D and E and Quarried Greywacke(1) is in groups E and F. Although statistically their means are separable into the seven narrow bands that these different groups give, a more practical approach would be to combine groups D, E and F and groups B and C making four groups so that any recommendations made on the Fire Resistance of New Zealand concretes would not be divided into more different groups than is likely to be useful. Tables for Fire Resistance requirements would become unnecessarily complex if there are too many sub-groups.

The worst performing of the aggregates tested, alluvial quartz, was compared with data from Abrams and Gustaferro (1968). By coincidence, the alluvial quartz is also the nearest in lithology to what Abrams and Gustaferro (1968) describe as siliceous aggregate. The results of a comparison of temperature distributions within the concrete specimens after 2.0 hours of the test are shown in Figure 56.

As part of the recommendations for further work (see below) it is suggested that fire-testing of 60mm and 175mm thick slabs be undertaken. Initially a representative of siliceous aggregate type, probably Alluvial Quartz, would be most useful as it gave the lowest fire resistance time in the current study.

There is good correlation between the New Zealand data and that of Abrams and Gustaferro (1968) and this corroborates the validity of the BRANZ results, given the inevitable differences in mix design, drying rate, and the repeatability of the test method and laboratory conditions. The close correlation of the results of the vertical tests done by BRANZ and the horizontal tests of Abrams and Gustaferro (1968) suggests that there may be only a small difference in heat transmission rates for vertical non-loadbearing walls and horizontal floor slabs.

Figure 56 also plots data from the CEB (1982) work on the fire performance of European concretes. Although nothing is known about testing conditions or concrete type, the correlation of these results with data from both the USA (Abrams and Gustaferro, 1968) and this work is very close.

Drying Rates and Their Influence on Fire Resistance

As reported above, drying of the concretes prior to fire testing was carried out under ambient storage conditions. The slabs were stacked vertically, allowing for free air circulation, (Figure 5). Analysis of measured mid-depth RH during drying yielded the following mean values for drying rate, expressed as percent fall in mid-depth RH per week (from casting to fire testing). See Table 6.

The effect upon fire resistance of changes in mid-depth RH from the "standard" level of 75 per cent is minimal. Abrams and Gustaferro (1968) quote a value of 1 per cent as the change in "fire endurance" for each 5 percent change in mid depth RH.

Abrams and Gustaferro (1968) proposed another empirical method for this adjustment especially if the concrete had been conditioned in a non-standard environment. This method was applied to the data collected in the present work and was found to coincide closely with results obtained by the ASTM E 119 (1982) method (based on Harmathy, 1965) with the exception of the pumice concrete. This is not unexpected as the constant used in the calculation for "structural lightweight concrete" may very well not be appropriate for New Zealand pumice.

The data on drying times from this work are being used in another BRANZ research project on the drying rate of concrete floors which will more rigorously analyse them, in order to, among other things, identify whether aggregate type significantly affects the drying rate.

One observable effect of the moisture condition of concrete at test is the extent of the time/temperature "plateau" which occurs when heat energy is expended in driving moisture through the slab at the expense of a rise in temperature. This effect can readily be seen by comparing the plateaux in Figures 16 and 19, being the first and second fire tests on the pumice concrete specimens. Eighteen weeks under ambient storage elapsed between the two tests. Mid-depth RH at the time of testing was 80.5 and 56.3 percent respectively. The non-moisture adjusted fire resistance times were 307 and 254 minutes respectively.

These results confirm the observations of Abrams and Orals (1965) that "a significant characteristic of the (sorption) cycle is that moisture which is lost during the first drying phase cannot be completely replaced through adsorption". This leads to a reduced fire endurance in a repeat fire test, consequent upon less energy (and time) being necessary to drive off water within the slab, observed as a reduced plateau in the time/temperature curve.

Deflection Due to Thermal Expansion

In the case of concrete, expansion of the fire-exposed surface results in bowing towards the heated area relative to the ends of the element and depends upon the degree of restraint of the element. Cooke and Morgan (1988) derived the following expression to calculate the thermal bowing in an unrestrained element, due to this expansion:

$$m = \frac{\alpha L T}{8d}$$

where,
- m = deflection, mm.
- α = coefficient of linear thermal expansion, $°C^{-1}$
- L = length/height of specimen, mm.
- d = thickness of specimen, mm.
- T = temperature gradient across specimen, °C.

This equation was applied to data from the BRANZ study for three different types of aggregate. See Table 7 for results.

The agreement between measured and calculated deflection appears to be very good.

Spalling

Spalling is a term used to describe the breaking off of layers or pieces of concrete from the fire exposed surface when it is subject to high and rapidly rising temperatures. It may be quite insignificant such as the breaking off of a small piece from a corner, or it may be so extensive as to compromise the structural integrity of a member.

Malhotra (1984) listed six factors as having an influence on the occurrence and degree of spalling:

- aggregate type.
- free moisture content.
- restraint against expansion.
- aggregate size.
- concrete density.
- concrete age.

Malhotra (1984) considers the first three of these to be the most important.

Recent work, (Richardson, personal communication) by the National Research Council in Canada has indicated that spalling may be more likely in high strength concrete than in normal strength concretes, and this topic area requires further investigation.

Definitions of spalling type vary, but most authors would agree with the general categories described by the American Concrete Institute (1980) as:

- Surface spalling, which includes pitting, blistering and local removal of surface material.
- Aggregate splitting, which is the failure of aggregate near the surface and is often accompanied by surface spalling.
- Corner separation, which is the removal (often violent) of external corners from beams and columns.

Harmathy (1965) makes the further distinction between thermal spalling (as described in the types above) and "moisture clog spalling" which is due to the movement of a moisture front through the concrete as one surface is heated. Depending upon the porosity of the concrete there can be a build up of vapour pressure which leads to the explosive spalling or delamination of large areas of the surface.

In the present work, several of the concretes showed the effects of minor surface spalling, being no more than blistering of the surface or breakout of some of the larger aggregate pieces. The pumice concrete (see Figures 57 and 58) showed this effect more than any other.

The only other form of structural damage to be suffered by this series of concretes was that of surface sloughing in the limestone specimen. This was the result of rehydration on storage in the open air following calcination during firing and was observed as a large scale delamination of the fire-exposed surface (see Figure 59). This obviously has potentially serious implications for the post-fire structural integrity of such concretes but in practice, limestone aggregate is rarely used in structural concrete.

Elastic Modulus and Residual Compressive Strength

Central Laboratories, WORKS Consultancy services, was given a brief to see if a field technique could be developed for measuring the correlation between the structural integrity of fire-damaged concrete and ultra-sonic pulse velocity measurements. The modulus of elasticity expresses the ratio of the deforming stress to the deformation of the material. It is thus a measure of the ability of the material to resist deformation and, in general, the modulus decreases gradually with increasing temperature. This aspect of the work has been reported by Rowe et al (1989).

In summary, a fired slab of Quarried Greywacke(2) was used by Central Laboratories for comparing the elastic modulus with the temperatures that the concrete had been subjected to at various depths.

A practical difficulty with obtaining good quantitative data using the ultrasonic technique is that all fire-damaged concretes will show cracking due to thermal shrinkage. These cracks, their position and density will tend to attenuate the signal and lead to a wide variation in pulse velocities, as was the case in this work. These factors make the results of the ultrasonic pulse velocities doubtful although in broad terms the results do follow expected patterns.

As Rowe et al (1989) say, perhaps a more useful application of the technique is in the comparison of ultrasonic velocities between undamaged and damaged concrete in a structure. In this way the worst damaged areas can be identified and the loadbearing capacity of the concrete, as measured by residual compressive strength, can be assessed by taking core samples oriented appropriately according to observed cracking patterns. Such data could then be broadly compared with those presented by Lie et al (1986) relating ultrasonic pulse velocity to compressive strength.

Recommendations for Further Work

1. To confirm the validity of FRR for non-loadbearing walls, further limited experimental work is necessary. This should involve the fire-testing of slabs of 60 mm and 175 mm thickness of selected concrete types. Initially a siliceous (dense) aggregate would be chosen as this is the most commonly used type of aggregate for structural concrete in New Zealand.

2. It will be necessary to assess the behaviour of loadbearing walls and concrete floors and propose FRR's for tabulation in MP 9. Given the relationship between the New Zealand and overseas data for non-loadbearing walls and that the load on a concrete element does not affect heat transfer it should be possible to propose FRR for load cases without further testing.

3. Limited experimental work may be necessary, if answers are not forthcoming from a literature search, to establish confidence in the fire performance of high-strength concrete. Instances of severe spalling of high-strength concrete columns have been observed experimentally overseas on loaded elements. Initially some tests will be done on unloaded high strength concrete elements at BRANZ.

4. Further evaluation is needed of non-destructive test methods, other than the ultrasonic pulse velocity technique, for the field assessment of fire-damaged concrete.

CONCLUSIONS

1. Current New Zealand requirements for FRR of non-loadbearing reinforced concrete walls have been shown to be adequate for the single thickness tested. This reinforces confidence in the New Zealand codes in use and that the differences existing in the aggregates available in New Zealand will not cause earlier failure of concrete elements than expected from the current design codes.

2. Data on insulation failure times compare favourably with those from overseas. In general the performance of concrete made up with volcanic sourced aggregate was better (greater Fire Resistance) than both quartz and greywacke aggregate concretes.

3. Concrete made from pumice aggregate performed significantly better, in terms of its insulation properties, than any other concrete and pumice concrete should be treated separately in any future revision of FRR tables.

4. After further limited experiments, it could be possible to propose a reduction in concrete thickness from values currently used in New Zealand for a given fire resistance. To achieve this it will probably be necessary to divide the existing "siliceous" category into further subgroups as indicated in the discussion.

5. No spalling occurred, except for surface spalling in pumice concrete.

6. Significant structural damage was evident in only one concrete type. The limestone aggregate concrete showed severe deterioration following chemical changes during firing and reabsorption of water from the atmosphere later.

7. Information gathered relating residual compressive strength of concrete to ultra sonic pulse velocity measurements to assess the damage to concrete after firing was inconclusive due to unpredictable variations in the cracking of fired concrete, but can be used to identify the most damaged areas.

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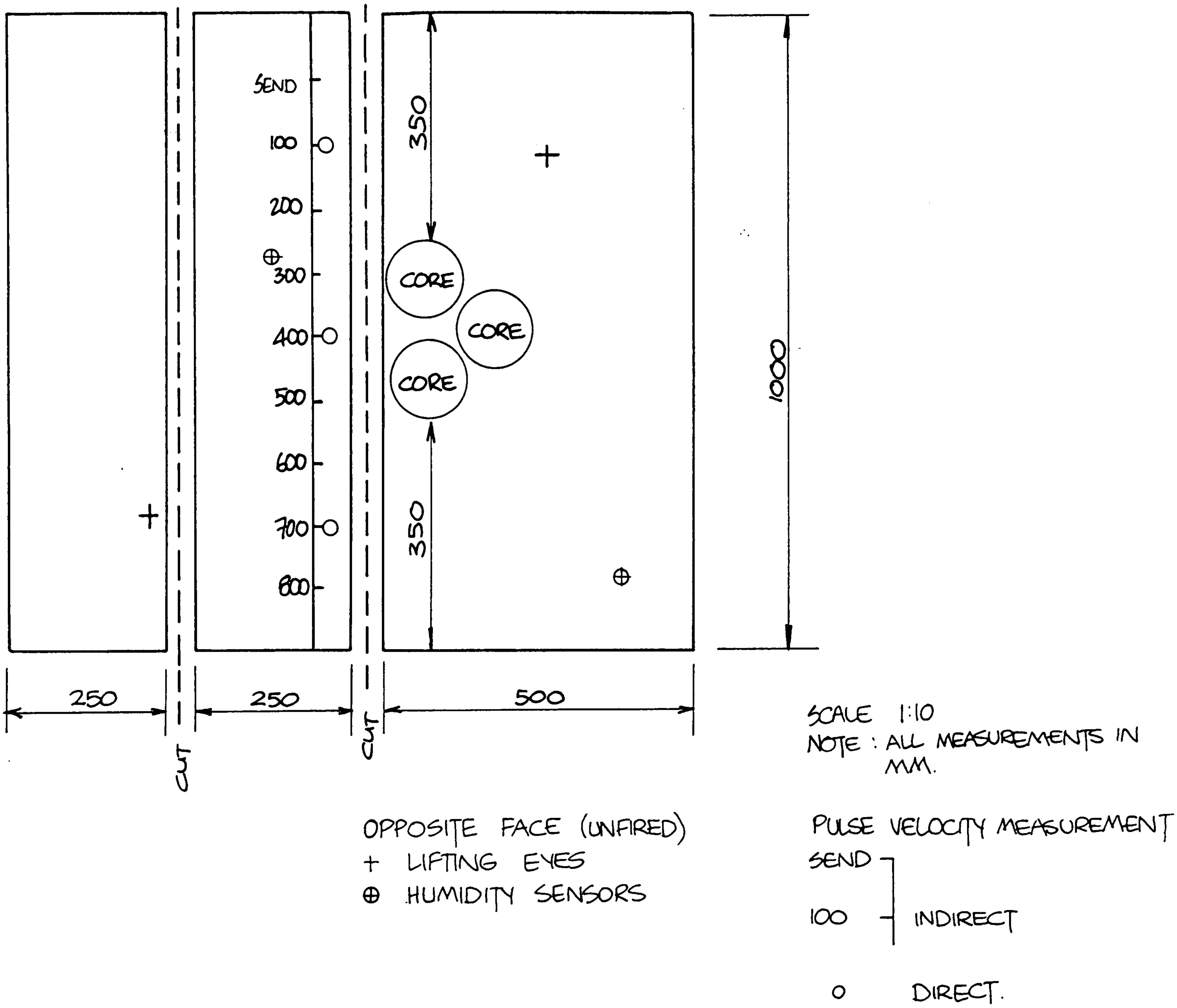
Appendix 1: Aggregate Sources

Aggregate		Local Name of Quarry or Source	Owner	Location	Production T (Year)	Notes on Usefulness for Fire Testing Concretes
Acid Volcanic	Dacite (Quarried)	Tauhara Quarry	J T Burrows	Taupo, Taupo County	17,000 (84)	Priority A; large production
	Rhyolite	Henderson's Quarry	Matt Henderson & Sons	Ngongotaha	3,600 (84)	Priority B; could provide reasonable performance limits with material from Tauhara Quarry for acid volcanics
	Pumice	Okareka Quarry	Firth Industries (leased)	Okareka Loop Rd, Rotorua	10,000 (85)	Priority B; not widely use for general concrete but pumice concrete is recognised as a competent thermal insulator
Intermediate Volcanic	Andesite (Alluvial)	Bell Block Quarry	Bell Block Quarries	Bell Block, Taranaki	107,020 (84)	Priority B
	Andesite (Quarried)	Okauia (Bartons Road) Quarry	Matamata Metal Supplies	Okauia, Matamata	88,010 (84)	Priority A; shape characteristics same as other volcanic materials
Basic Volcanic	Basalt (Quarried)	Lunn Ave Quarry	Winstone Limited	Mount Wellington, Auckland	295,550 (83)	Priority A; very large production
	Phonolite	Logan Point Quarry	Palmer & Sons	Dunedin	60,200 (84)	Priority B; useful for performance limits
	Greywacke (Alluvial)	Waimakariri River (Coutts Island)	Farrier-Waimak Limited	Coutts Island, Eyre County	146,860 (84)	Priority A; large production, rounded
	Greywacke (Quarried 1)	Hongoeka Bay	Carson Contracting	Plimmerton, Porirua	15,130 (84)	Priority A; close to BRANZ; uncontaminated, massive bedding
	Greywacke (Quarried 2)	Stevensons' Quarry	W Stevenson & Sons Ltd	Drury Quarry, Auckland	160,401 (86)	
	Quartz (alluvial) 19 mm		Walton Park Sand Co	Balclutha		
	Quartz (alluvial) 10 mm					
	Limestone (Coquina)	Pakipaki (Amners) Quarry	Firth Industries	Pakipaki, Hastings	23,630 (84)	Priority A; essential comparative material
Sand	Greywacke standard concrete sand	Puketapu Quarry		Kakariki, Rangitikei		
	Greywacke dune sand		Firth Industries	Otaihanga		

Appendix 2: Mix Design and Test Data for Concrete Used in Slabs

CONCRETE	UNIT	MATERIAL QUANTITIES PER m^3							CONCRETE PROPERTIES							
		C.A.		SAND		CEMENT	WATER	BATCH VOLUME	MIX	W/C	SLUMP mm	DENSITY kg/m^3	AIR %	YIELD m^3	TEMP °C	f'c MPa 28 d
		19.0	9.5	CONCRETE	DUNE											
Quarried Dacite 21/7/88	kg	692	282	811	87	295	168.5		1		80	2240	2.0	1.04	12	33.5
	m^3	0.2792	0.1196	0.3060	0.0339	0.0928	0.1685	0.15	2	0.57	100	2240	2.0	1.04	13	33.2
Rhyolite 15/12/88	kg	445	449	811	87	295	168.5		1		10	2150	3.9	1.05	19	29.7
	m^3	0.1994	0.1994	0.3060	0.0339	0.0928	0.1688	0.15	2	0.57	10	2150	3.8	1.05	20	29.0
Pumice 18/8/88	kg	341.4	85.3	811	87	295	163.6		1		150	1790	(8.5)	1.00	15	17.5
	m^3		0.3988	0.3060	0.0339	0.0928	0.1636	0.15	2	0.55	200	1840		0.97	15	18.3
Alluvial Andesite 14/7/88	kg	591	389	811	87	295	168.5		1		100	2250	3.6	1.04	15	34.2
	m^3	0.2392	0.1595	0.3060	0.0339	0.0928	0.1685	0.15	2	0.57	90	2270	3.7	1.03	16	34.7
Quarried Andesite 16/11/88	kg	545	543	81	87	295	168.5		1		50	2350	1.7	1.04	20	36.1
	m^3	0.1994	0.1994	0.3060	0.0339	0.0928	0.1685	0.15	2	0.57	60	2370	1.7	1.03	22	34.2
Quarried Basalt 7/7/88	kg	838	354	811	87	295	168.5		1		0	2440	2.9	1.05	-	46.2
	m^3	0.2792	0.1196	0.3060	0.0339	0.0928	0.1685	0.15	2	0.57	10	2440	2.5	1.05	-	46.4
Phonolite 28/7/88	kg	726	312	811	87	295	168.5		1		40	2310	1.9	1.04	14	38.2
	m^3	0.2792	0.1196	0.3060	0.0339	0.0928	0.1685	0.15	2	0.57	50	2300	1.7	1.04	14	39.0
Alluvial Greywacke 20/6/88	kg	741	318	811	87	295	168.5		1		80	2340	2.1	1.03	-	35.5
	m^3	0.2792	0.1196	0.3060	0.0339	0.0928	0.1685	0.15	2	0.57	140	2310	2.2	1.05	15	36.5
Quarried Greywacke (1) 23/6/88	kg	537	541	811	87	295	171.8		1		60	2340	1.9	1.040	15	39.3
	m^3	0.1996	0.1996	0.3060	0.0339	0.0928	0.1718	0.15	2	0.58	50	2340	1.9	1.040	-	41.0
Quarried Greywacke (2) 8/12/88	kg	540	536	811	87	295	168.5	0.15	1		50	2380	1.8	1.03	19	34.0
	m^3	0.1994	0.1994	0.3060	0.0339	0.0928	0.1685		2	0.57	70	2350	1.9	1.04	19	33.7
Alluvial Quartz 4/8/88	kg	526	514	811	87	295	168.5	0.15	1		20	2320	2.6	1.04	14	39.8
	m^3	0.1994	0.1994	0.3060	0.0339	0.0928	0.1685		2	0.57	20	2310	3.1	1.04	15	41.0
Coquina Limestone 11/8/88	kg	905		811	87	295	168.5	0.15	1		10	2180	3.7	1.04	13	29.4
	m^3	0.3988		0.3060	0.0339	0.0928	0.1685		2	0.57	10	2180	3.9	1.04	13	30.0

C.A = Coarse Aggregate W/C = Water/Cement ratio



Appendix 3: Positions of Coring and Ultrasonic Pulse Velocity Measurements on Slab. Plan View of Fired Face.

Concrete Type	Maximum Deflection, mm
Quarried Dacite	11.0
Rhyolite	13.5
Pumice	11.0
Alluvial Andesite	14.0
Quarried Andesite	13.5
Quarried Basalt	12.0
Phonolite	13.5
Alluvial Greywacke	14.5
Quarried Greywacke (1)	14.5
Quarried Greywacke (2)	15.0
Alluvial Quartz	19.5
Coquina Limestone	11.5

Table 1: Horizontal Out-of-plane Deflection at Mid-height of Concrete Specimens as a Result of Exposure to Fire.

Notes: The deflections listed are the average of those measured for the two slabs of each concrete type tested.

(1) Carson's Quarry, Plimmerton.
(2) Stevenson's Quarry, Drury.

Core	A	B	C
Elastic modulus (E_{static}) (GPa)			
17 mm below fired surface	17.1	-	7.1
27 mm below fired surface	19.5	-	10.3
37 mm below fired surface	21.6	-	12.4
47 mm below fired surface	23.6	-	8.7
57 mm below fired surface	22.0	-	14.9
f'c, failure load (MPa)	45.5	39.5	32.5
Design Elastic Modulus (GPa) (= $4.7\sqrt{f'c}$)	31.7	29.5	26.8
Ultrasonic pulse velocity (m/s)	3257	2979	3054
Elastic modulus, E_{dyn} (GPa) from ultrasonic pulse velocity measurements	26.7	22.4	23.5

Table 2: Elastic Modulus and Ultrasonic Pulse Velocity Measurements on Cores of Fire Tested Quarried Greywacke (2)

TABLE 3: ULTRASONIC PULSE VELOCITY MEASUREMENTS ON QUARRIED GREYWACKE (2) AGGREGATE CONCRETE AFTER FIRING (m/s)

SURFACE	INDIRECT DISTANCE FROM "SEND" TRANSDUCER (MM)								AVERAGED DIRECT
	100	200	300	400	500	600	700	800	
Fired Surface	2500	2439	2703	2857	2604	2326	1772	1684	3260 m/s
12 mm below fired surface	2778	2469	2344	2286	2392	1583	1549	1471	3200 m/s
20 mm below fired surface	2381	2299	2000	2174	2222	1504	1414	1327	3105 m/s
33 mm below fire surface	1961	1818	1923	1455	1381	1333	1062	843	3016 m/s
45 mm below fire surface	2000	2000	1523	1423	1362	1095	1084	1088	3126 m/s
57 mm below fired surface	1613	2020	1546	1299	1247	1147	1056	1050	3276 m/s
129 mm below fired surface (opp face)	2778	2273	2055	2151	2183	1695	1643	1553	

AGGREGATE TYPE	AGE AT TEST (Weeks)	MID-DEPTH R.H. AT TEST (Percent)	MEAN TIME TO REACH A 140 °C RISE (min)	ADJUSTED TIME TO 75% R.H. MID-DEPTH, (min)
QUARRIED DACITE	38	73.0	182	183
RHYOLITE	27	82.9	196	192
PUMICE	27	80.5	306	302
ALLUVIAL ANDESITE	34	79.9	220	218
QUARRIED ANDESITE	26	78.2	192	191
QUARRIED BASALT	34	72.9	176	177
PHONOLITE	38	71.5	190	192
ALLUVIAL GREYWACKE	33	74.5	163	164
QUARRIED GREYWACKE (1)	23	84.9	171	167
QUARRIED GREYWACKE (2)	25	73.1	173	174
ALLUVIAL QUARTZ	38	69.6	152	154
COQUINA LIMESTONE	26	86.1	217	211

Table 4: Age of slab, Relative Humidity (R.H.) and Mean time to 140° C Temperature Rise Adjusted According to ASTM E 119 .

Notes: - (1) Carson's Quarry, Plimmerton
- (2) Stevenson's Quarry, Drury
- Age is given to the nearest week and is measured from the casting date.
- R.H. data are the average values for the two slabs tested for each aggregate.
- Times to 140 °C rise are the mean values from up to ten thermocouples on the two slabs for each aggregate type.

AGGREGATE TYPE	AVERAGE FIRE RESISTANCE TIME (Minutes to 140 C rise)	DUNCAN LETTERS	STD ERROR OF MEAN	95% CONFIDENCE INTERVAL	95% CONFIDENCE ADJUSTED TO 75% MID DEPTH R.H.
QUARRIED DACITE	182	d	2.6	176 - 188	177 - 189
RHYOLITE	196	c	3.0	189 - 203	186 - 199
PUMICE	306	a	5.2	295 - 318	290 - 314
ALLUVIAL ANDESITE	220	b	2.3	214 - 225	212 - 223
QUARRIED ANDESITE	192	c	3.1	185 - 199	184 - 198
QUARRIED BASALT	176	de	4.1	167 - 185	167 - 186
PHONOLITE	190	c	2.5	185 - 196	186 - 198
ALLUVIAL GREYWACKE	163	f	1.9	159 - 168	159 - 168
QUARRIED GREYWACKE (1)	171	ef	2.1	166 - 175	162 - 171
QUARRIED GREYWACKE (2)	173	e	2.0	169 - 178	169 - 179
ALLUVIAL QUARTZ	152	g	1.4	149 - 155	151 - 157
COQUINA LIMESTONE	217	b	2.8	211 - 223	205 - 217

Table 5: Average Fire Resistance Time and 95% Confidence Interval for Fire Resistance Time According to Aggregate Type.

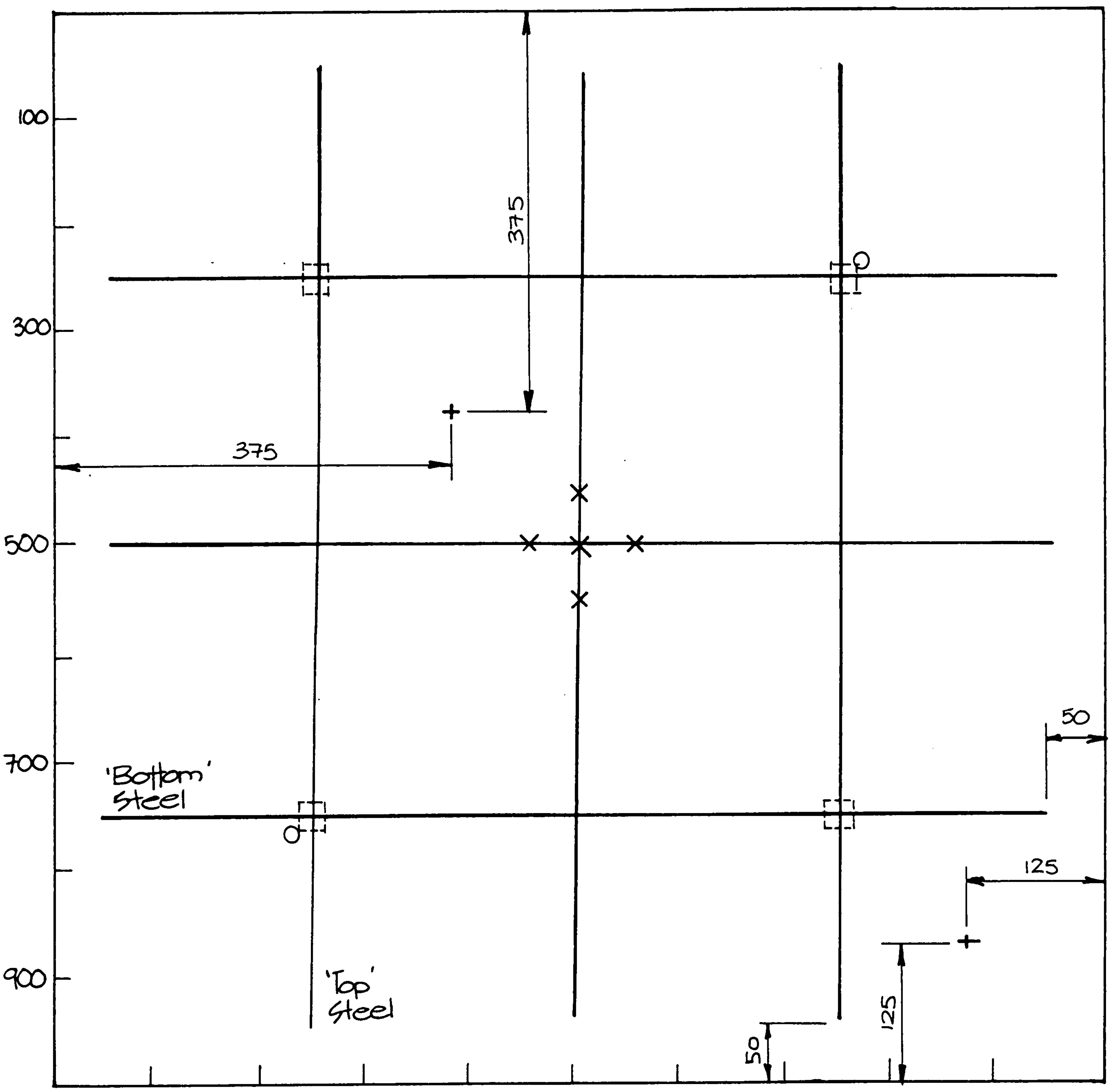
Notes: - (1) Carson's Quarry, Plimmerton.
- (2) Stevenson's Quarry, Drury.
- Times to 140 ^{o}C rise are the average values for the up to ten thermocouples on two slabs for each aggregate type.
- 95% confidence at 75% R.H. adjusted times are rounded to the nearest whole minute.
- This table is plotted in graph form in Figure 49.
- Mean times not significantly different have the same Duncan letters

	Mean % fall R.H./week	Std. Deviation
All concretes	0.74	0.14
Siliceous/Dense	0.76	0.14
Pumice	0.72	-
Limestone	0.53	-

Table 6: Drying Rates of Concrete Slabs

Concrete Type	Calculated Deflection, mm.	Measured Deflection, mm.
Quarried Greywacke (1)	10	14.5
Limestone	11	11.5
Pumice	9	11

Table 7: Measured and Calculated Values for Deflection of Selected Concretes



Legend

× Thermocouples on steel
+ Humidity probe formers
O Lifting lugs
⬚ Reinforcing supports (mortar) - reinforcing supported from top as cast
— D10 reinforcing steel.

Note
Dimensions in mm.

Fig 1: Location Plan of Hardware and Instrumentation

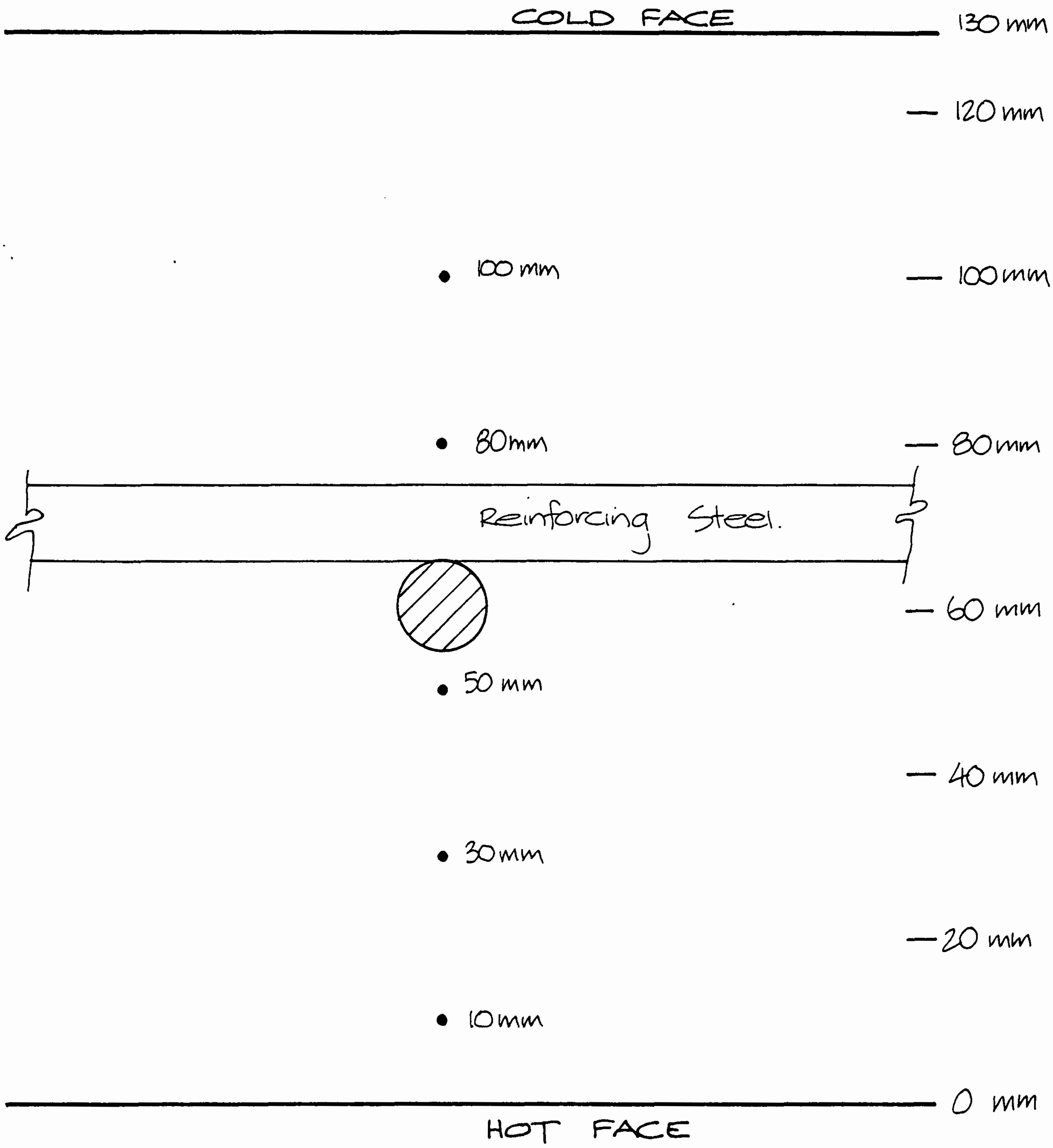


Fig 2: Sheathed Thermocouple Placement

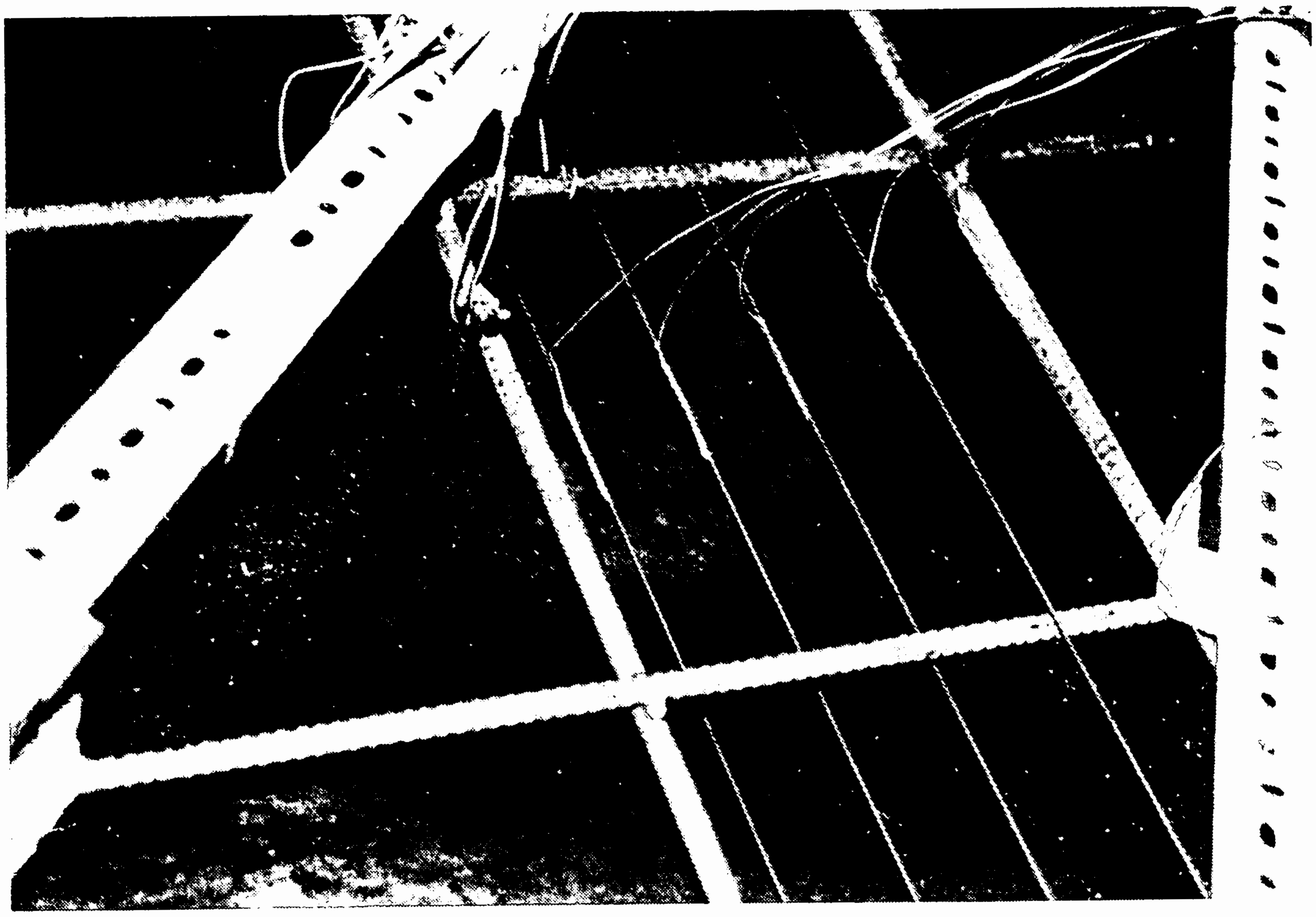


Fig 3: Detail of Thermocouple Placement in Mould

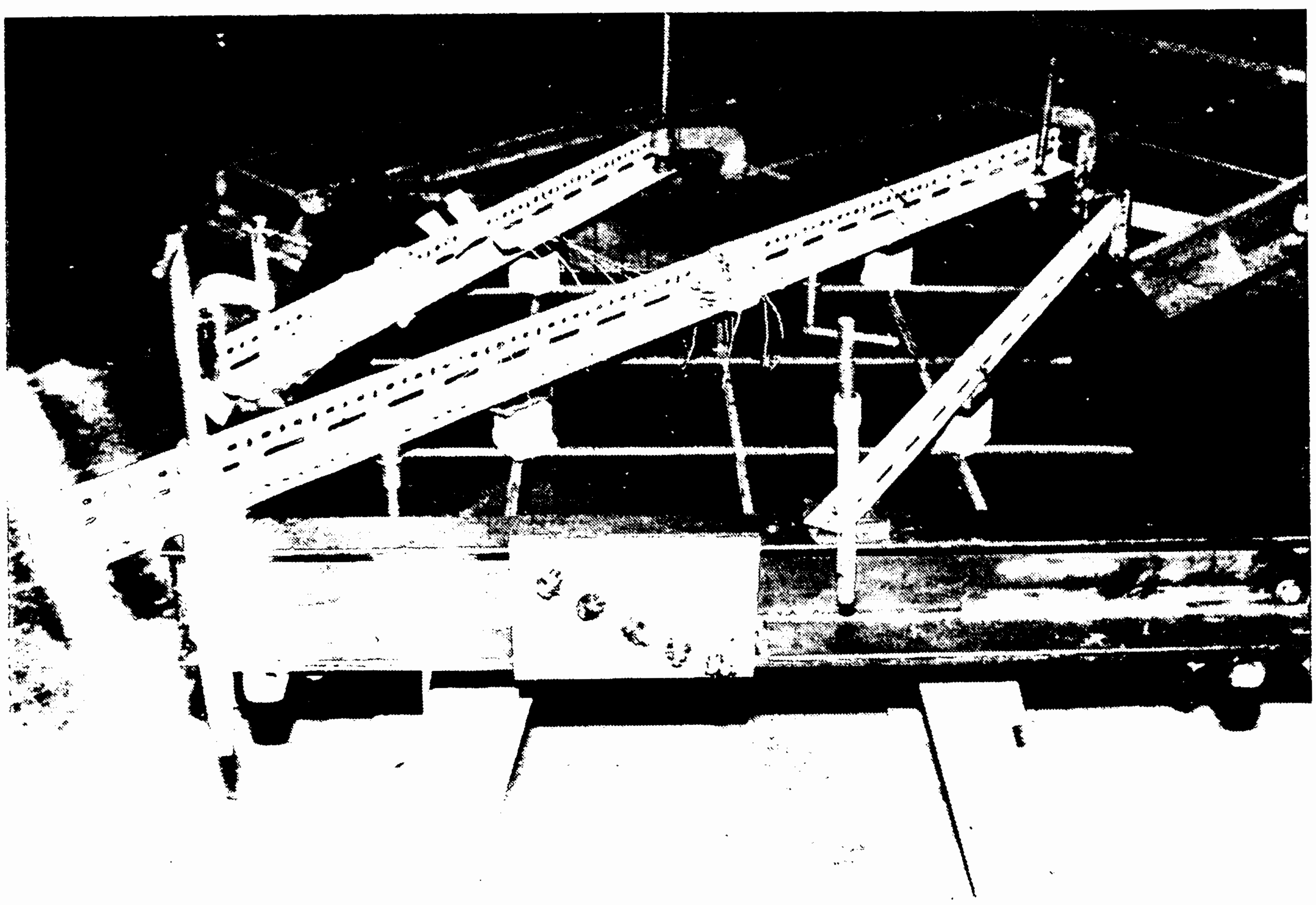


Fig 4: Instrumentation of Mould Prior to Casting

Fig 5: Storage Conditions of Slabs

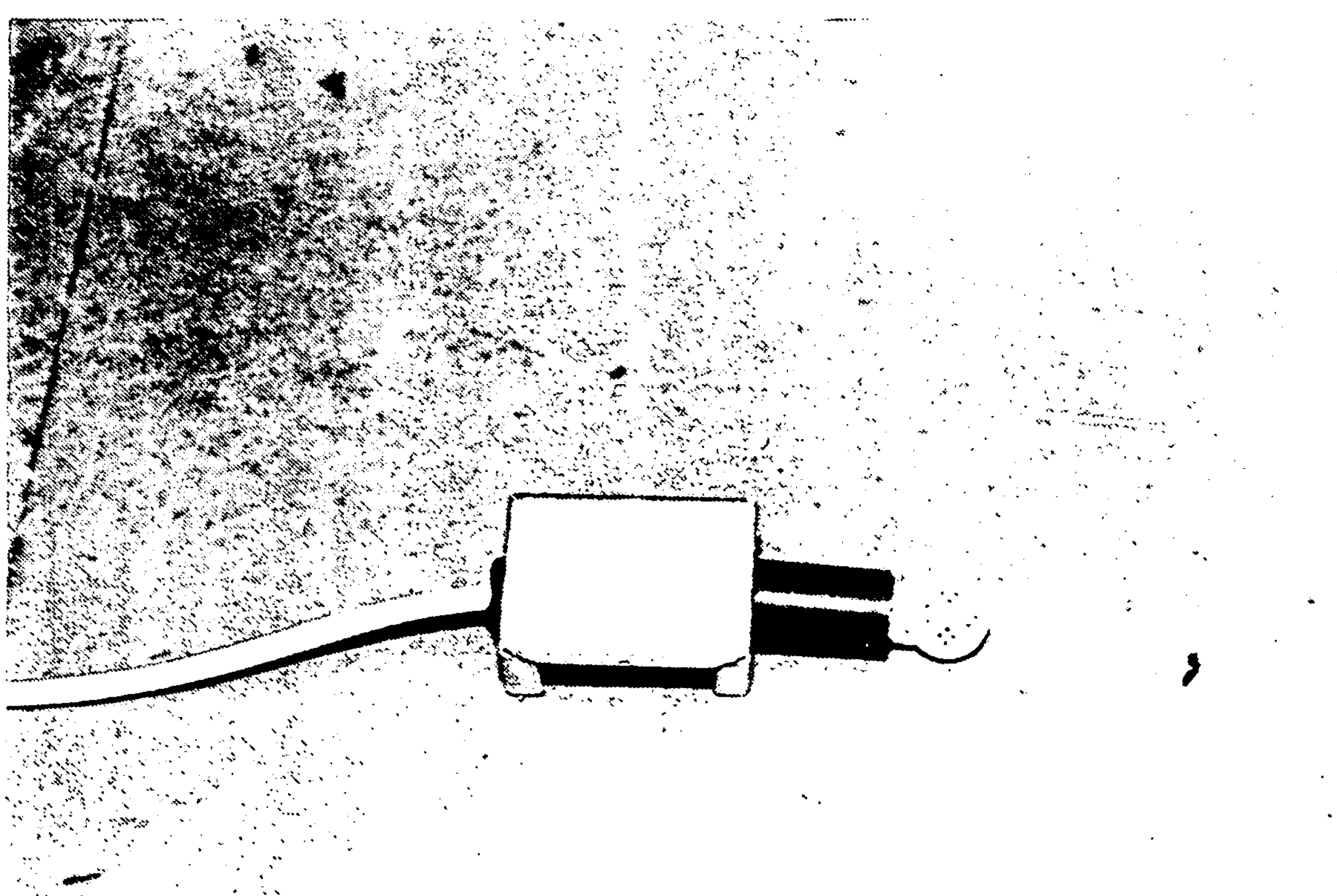


Fig 6: Probe used for Measuring Mid Depth RH

Fig 7: Relative Humidity Probe in Place on a Concrete Slab

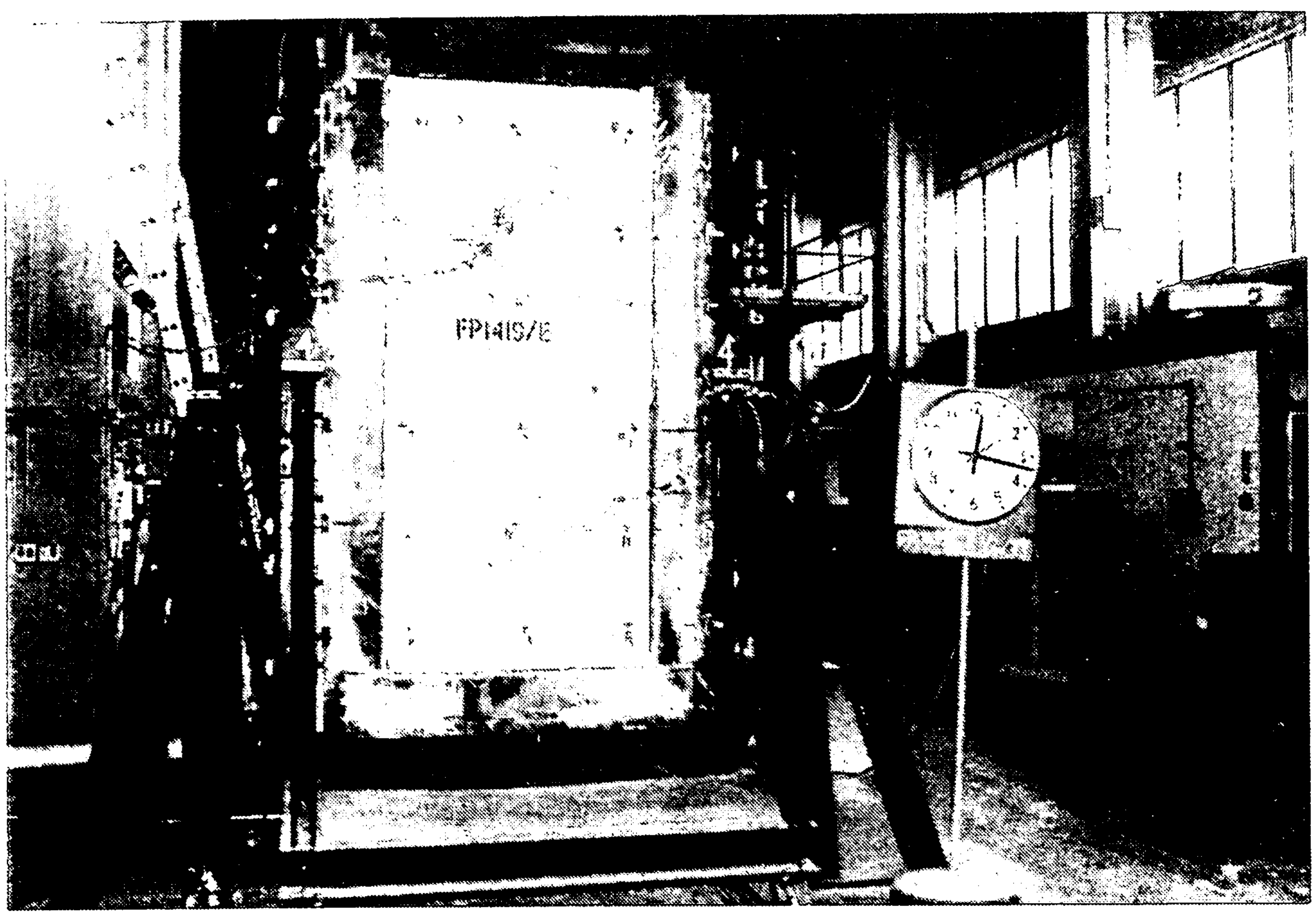


Fig 8: Pilot Furnace and Instrumented Slabs

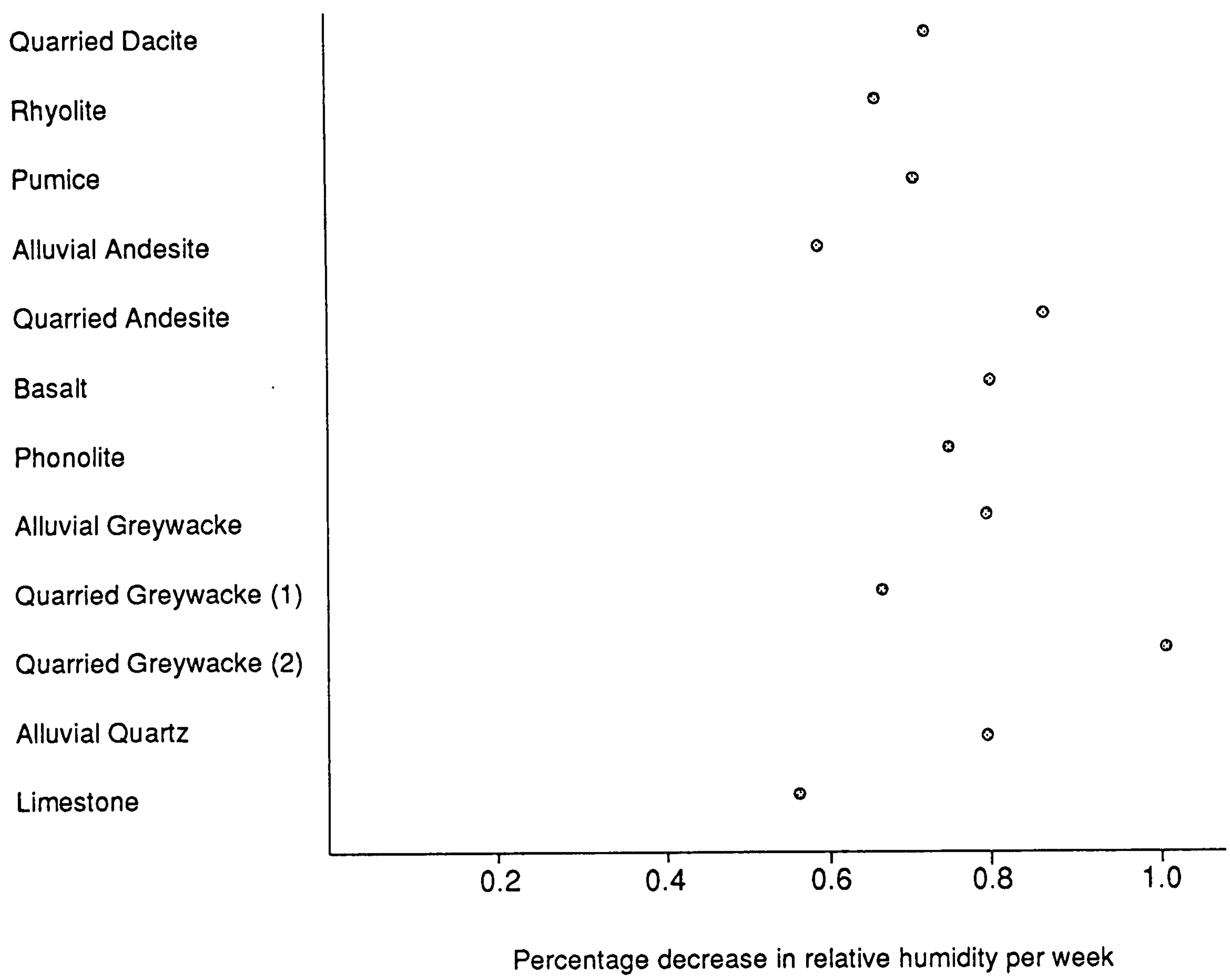


Figure 9 : Average concrete drying rates from casting to testing

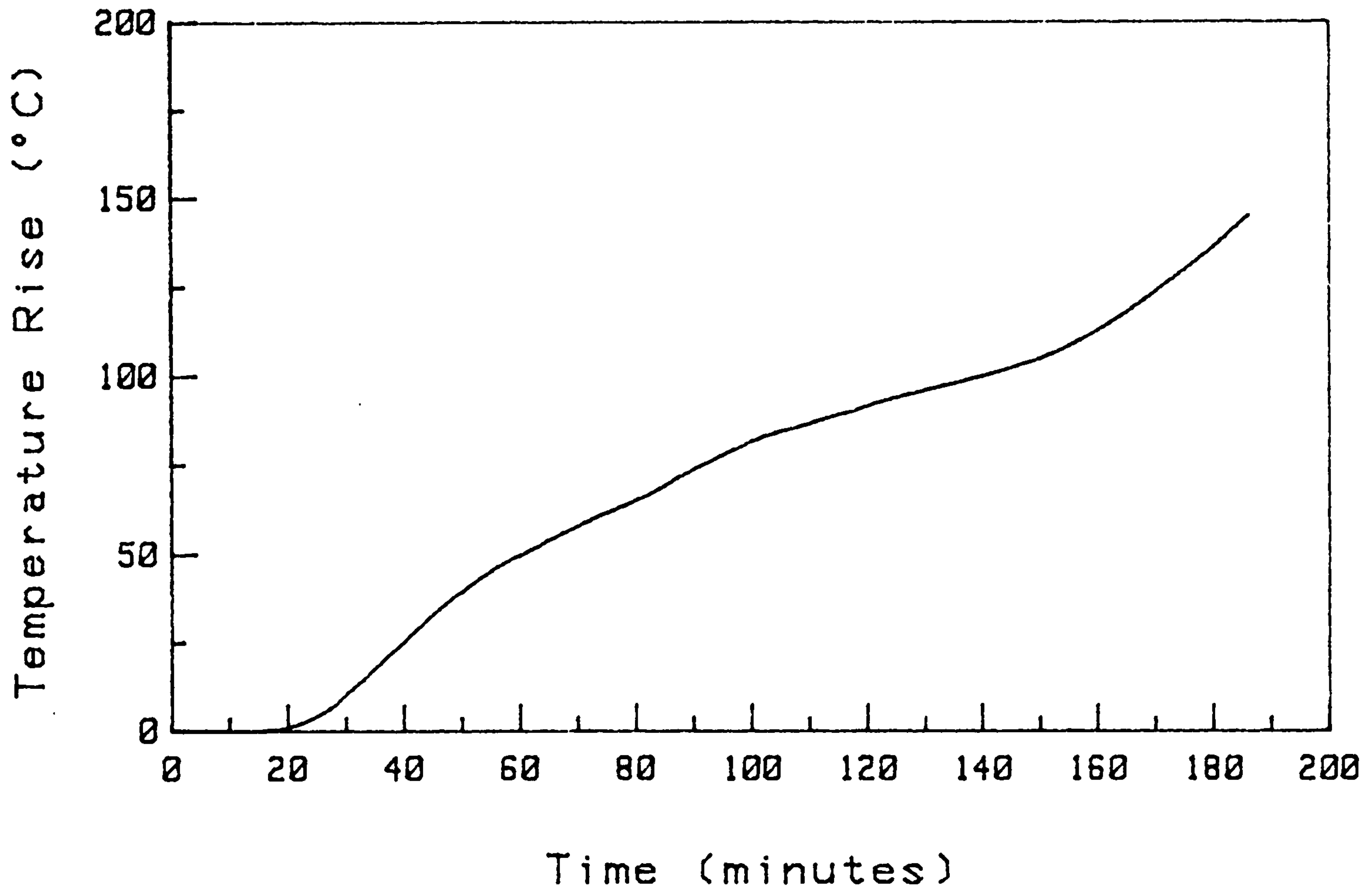


Fig 10: Quarried Dacite. Unexposed Face

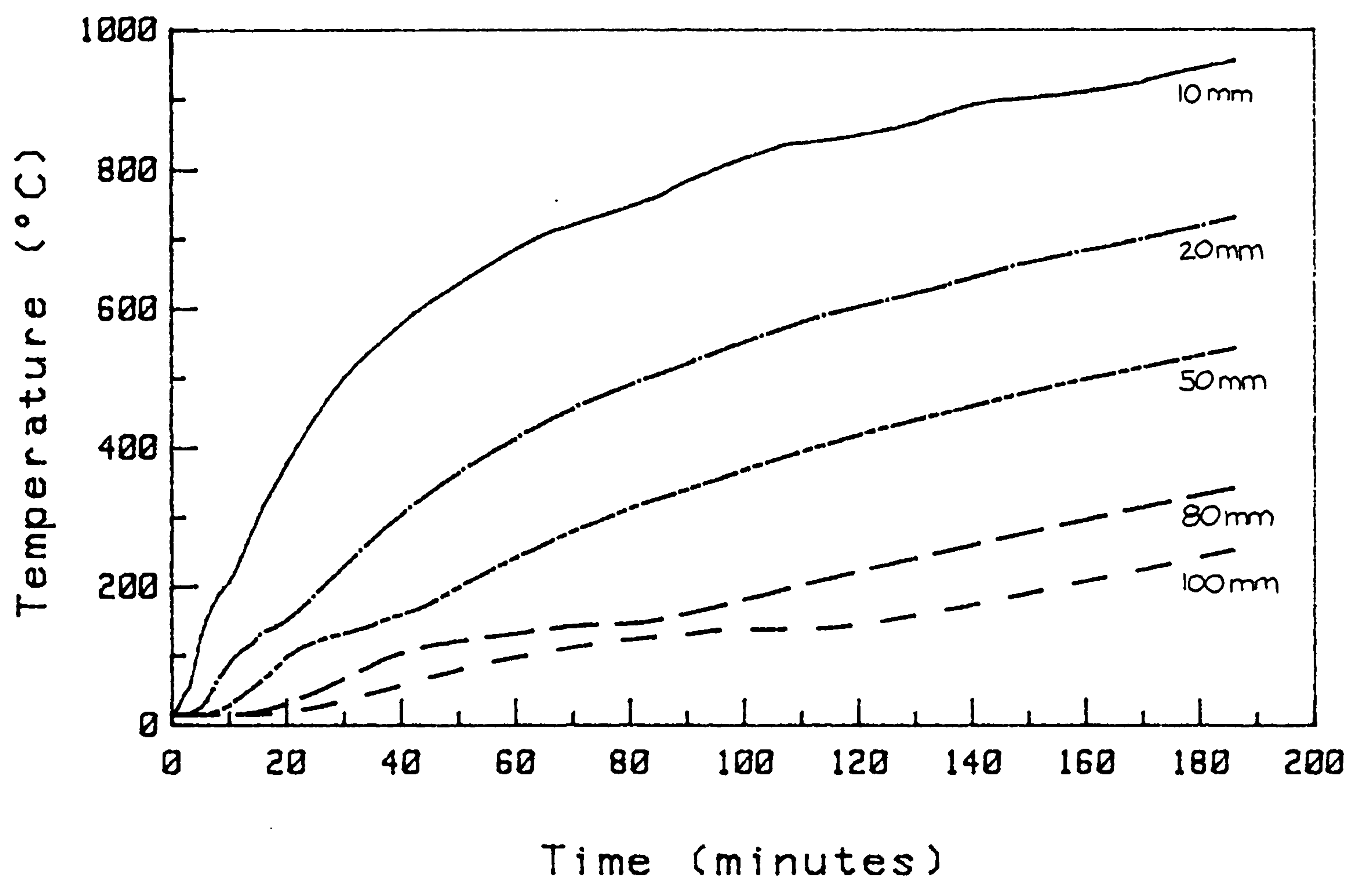


Fig 11: Quarried Dacite. Internal Temperatures

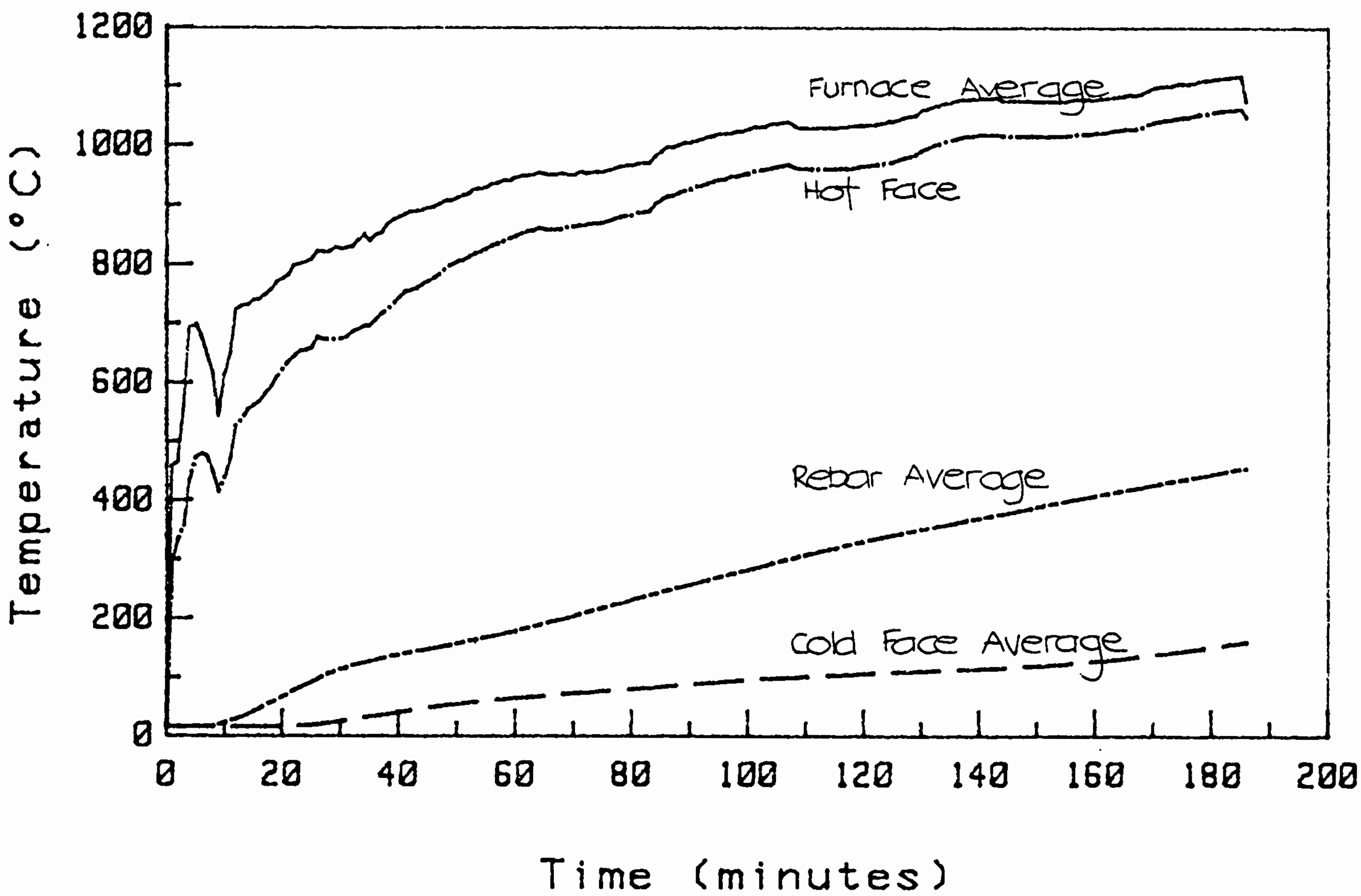


Fig 12: Quarried Dacite. External and Rebar Temperatures

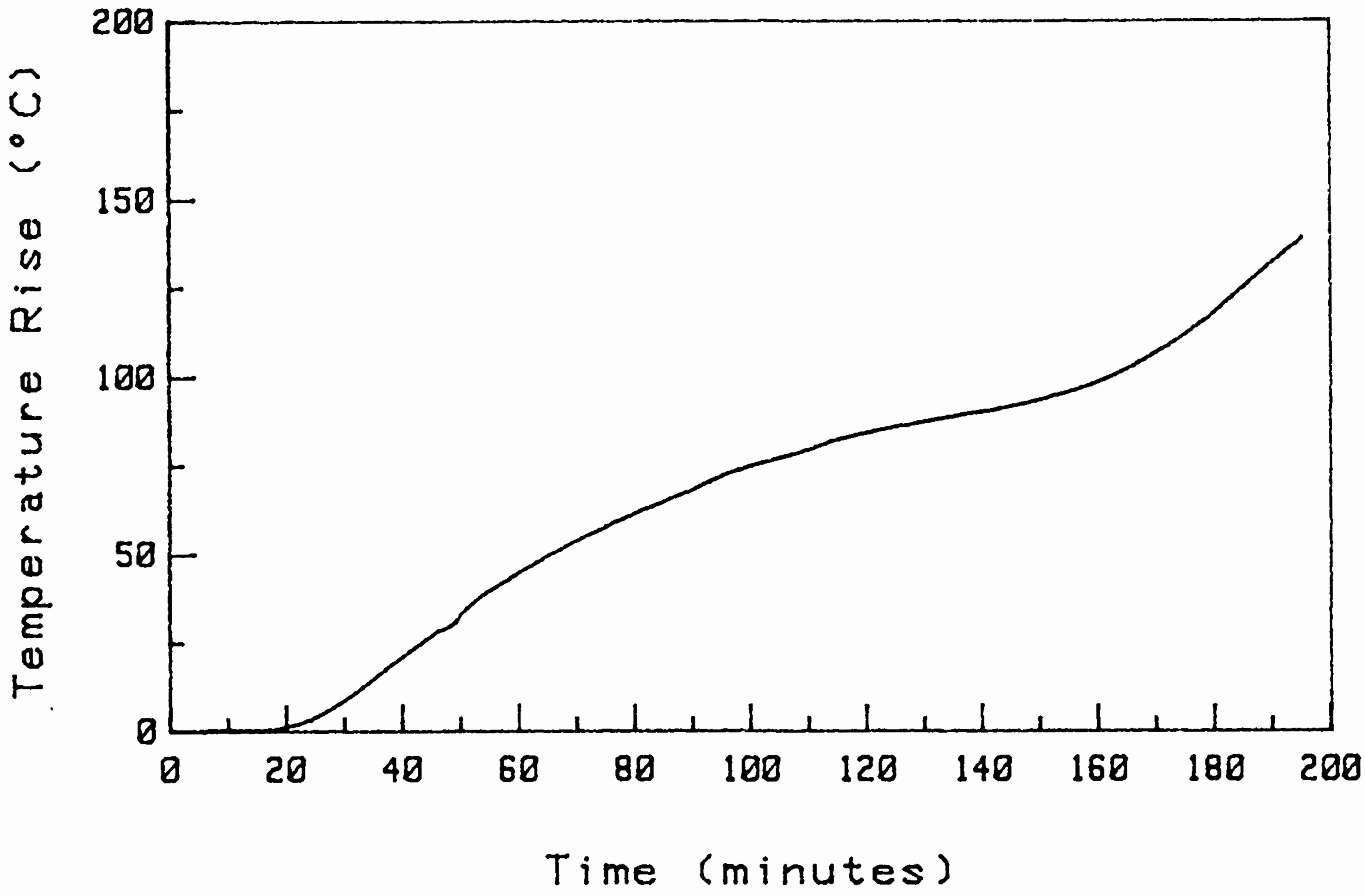


Fig 13: Rhyolite. Unexposed Face

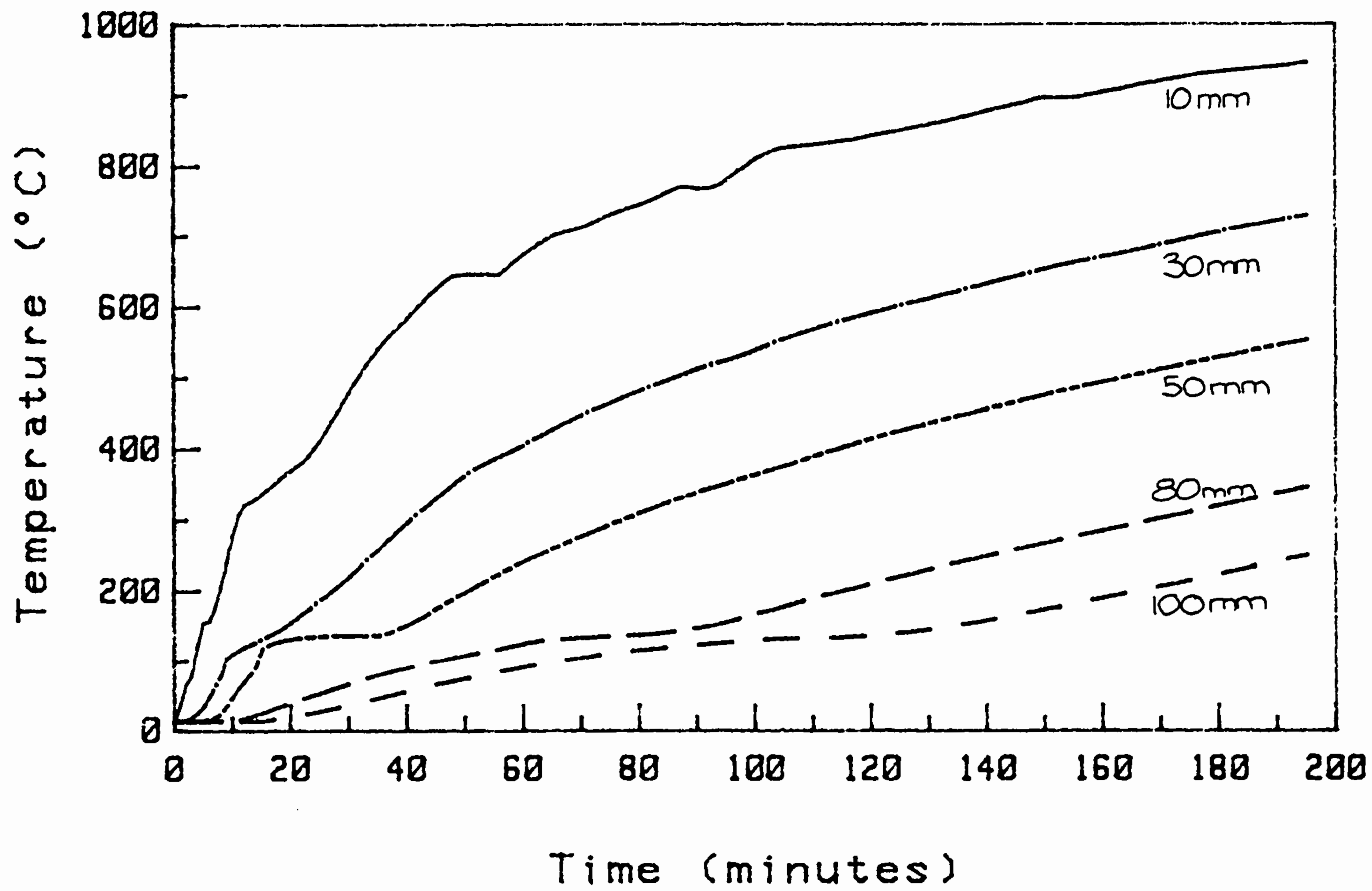


Fig 14: Rhyolite. Internal Temperatures

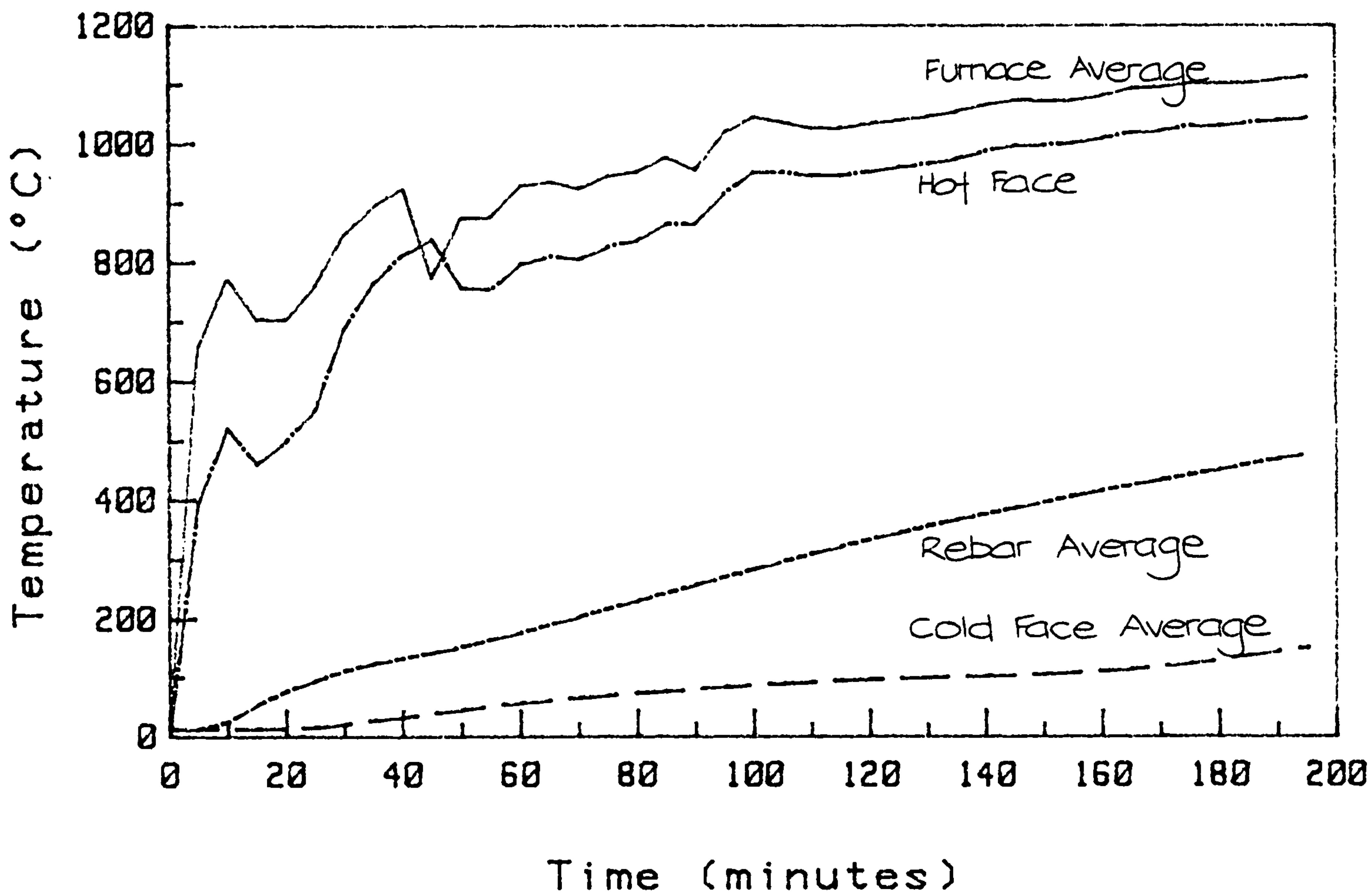


Fig 15: Rhyolite External and Rebar Temperature

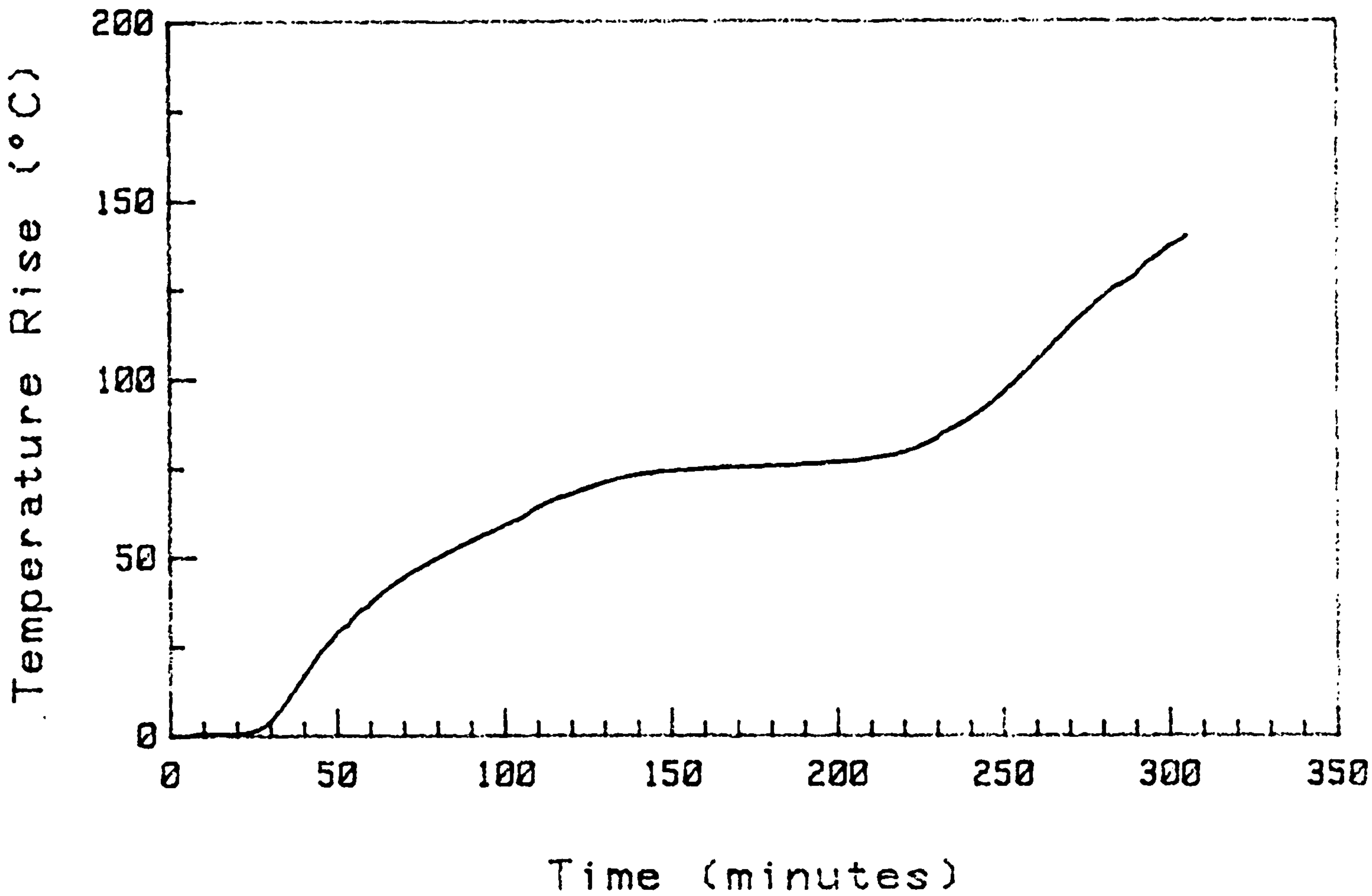


Fig 16: Pumice. Unexposed Face Temperature

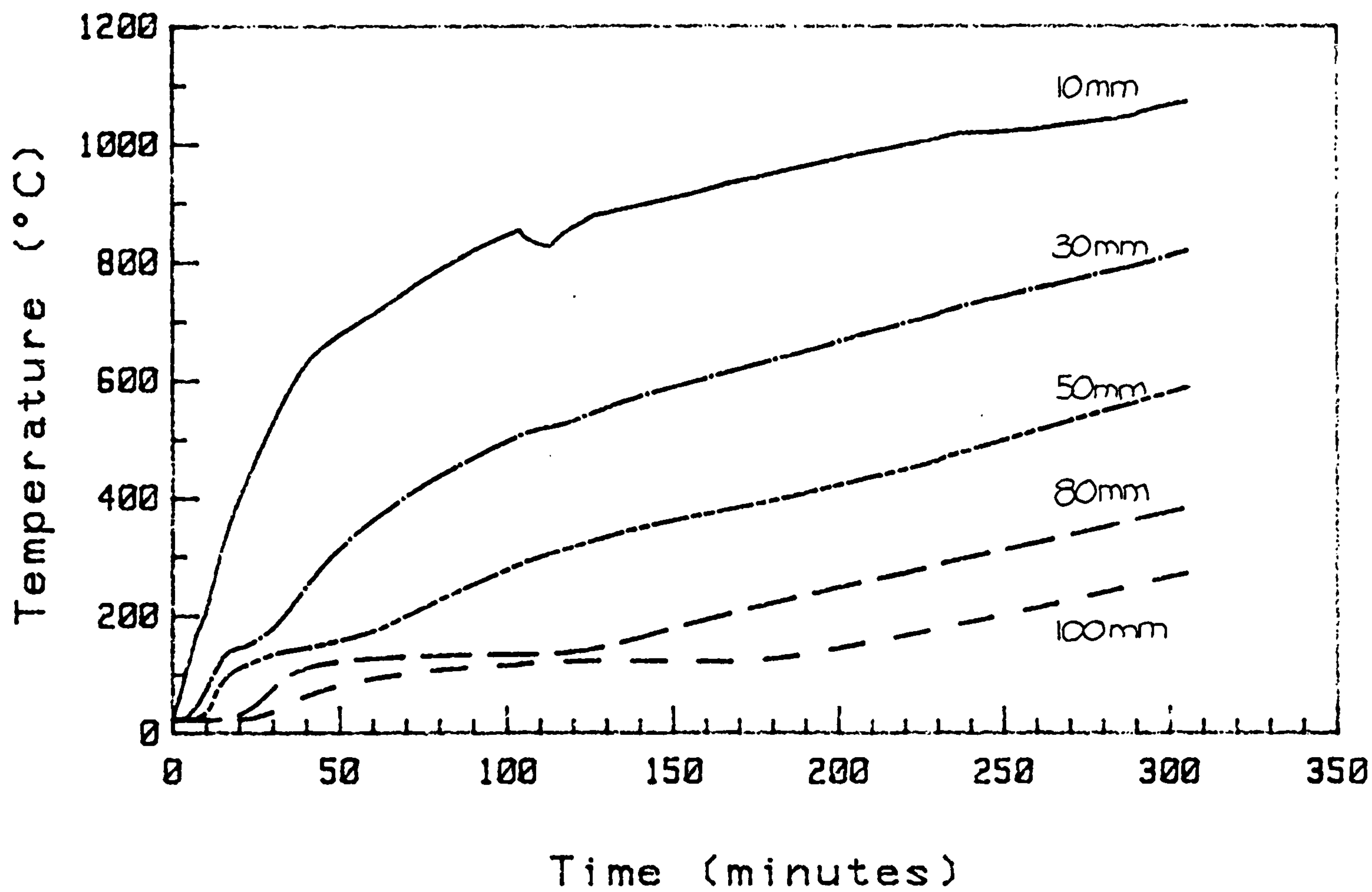


Fig 17: Pumice. Internal Temperatures

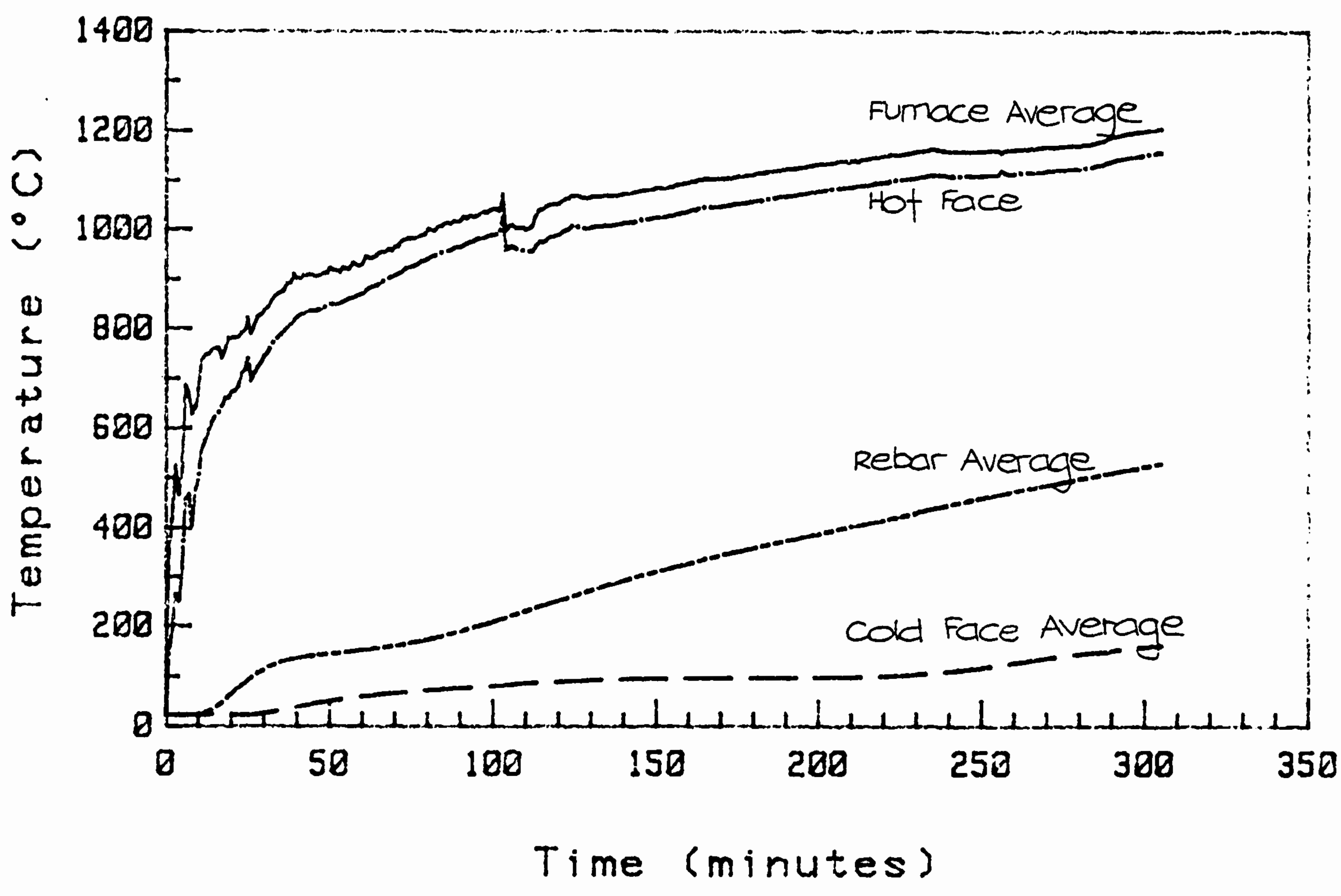


Fig 18: Pumice External and Rebar Temperatures

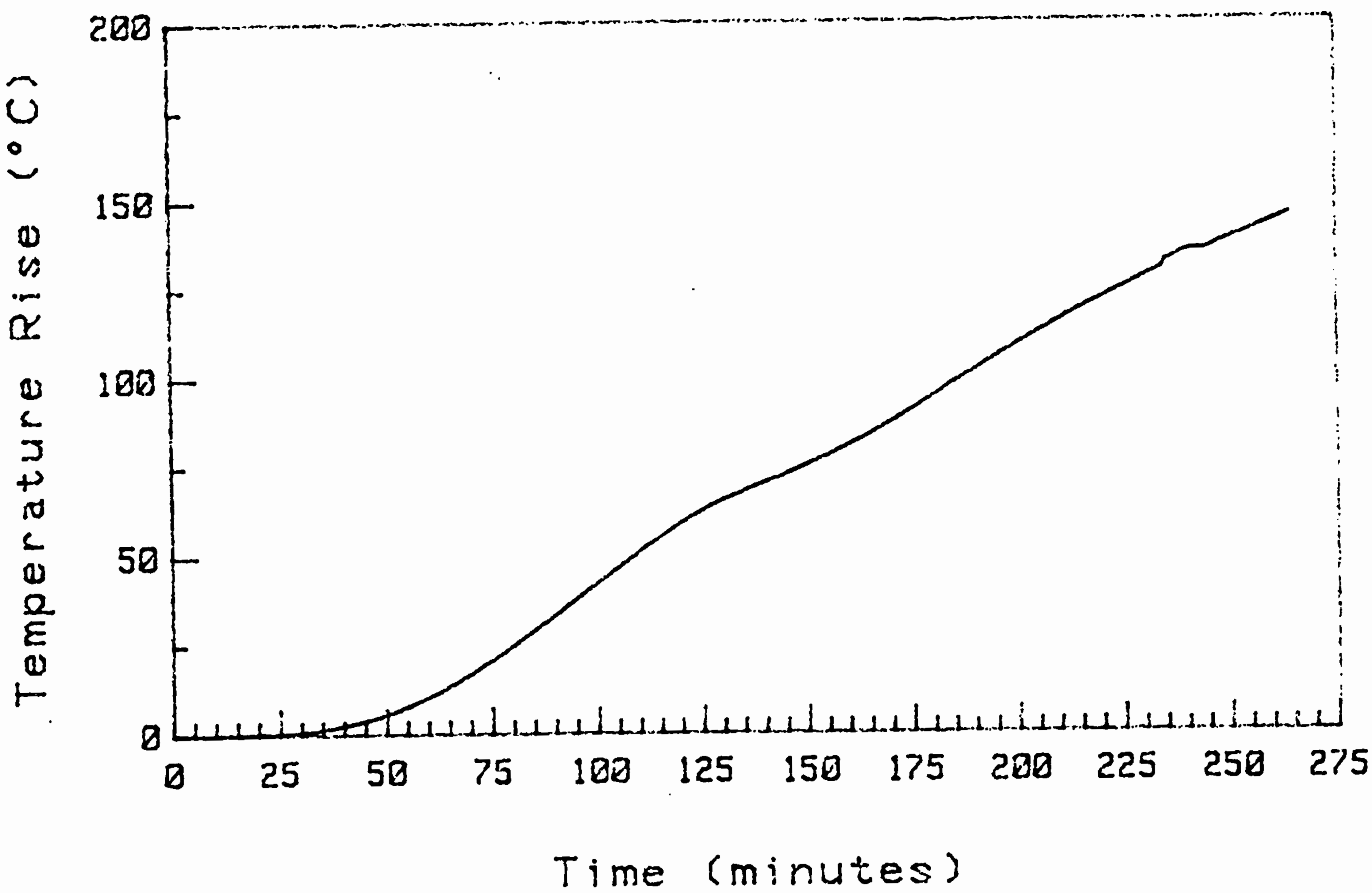


Fig 19: Pumice Reburn. Unexposed Face Temperature

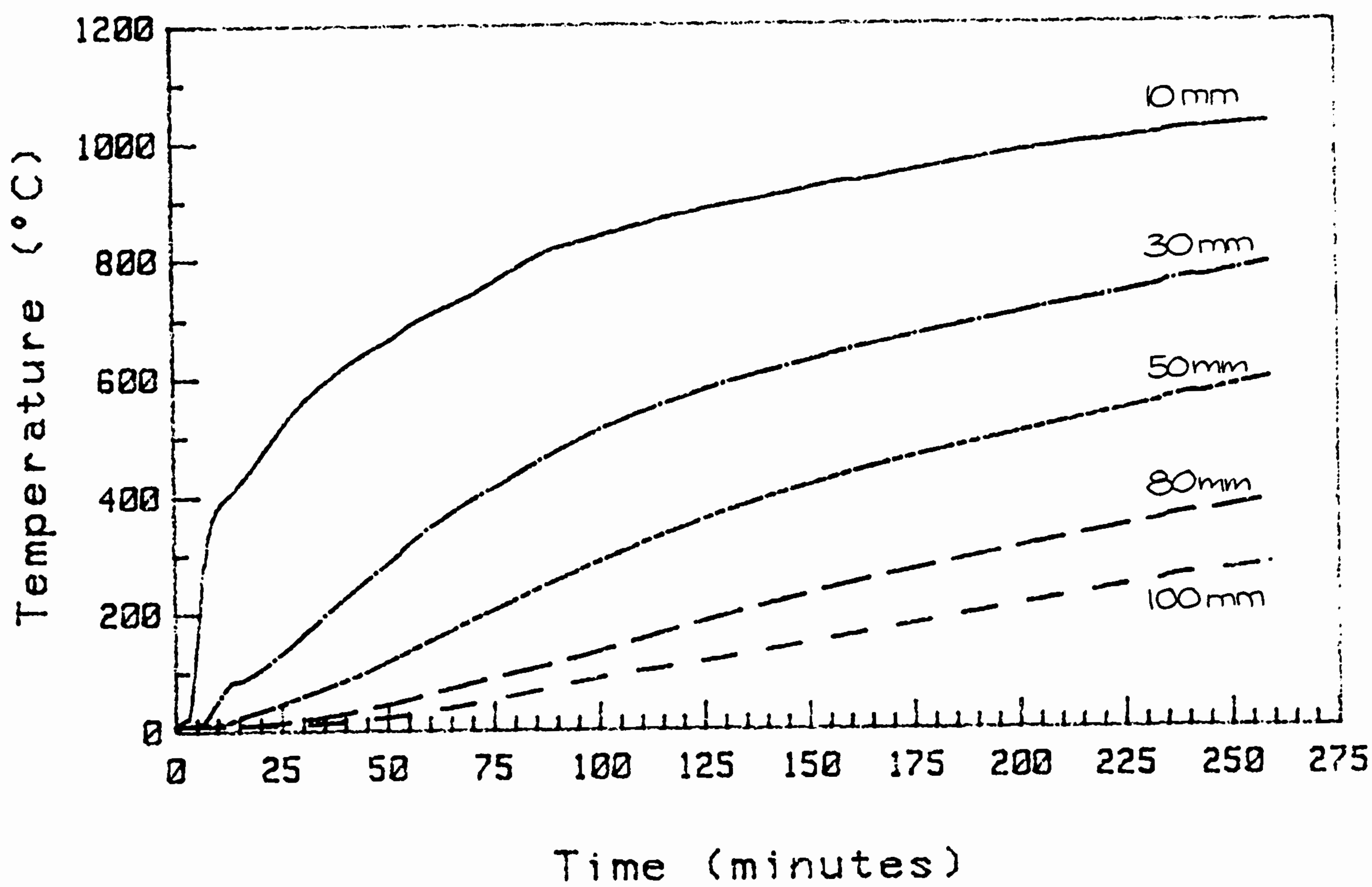


Fig 20: Pumice Reburn. Internal Temperatures

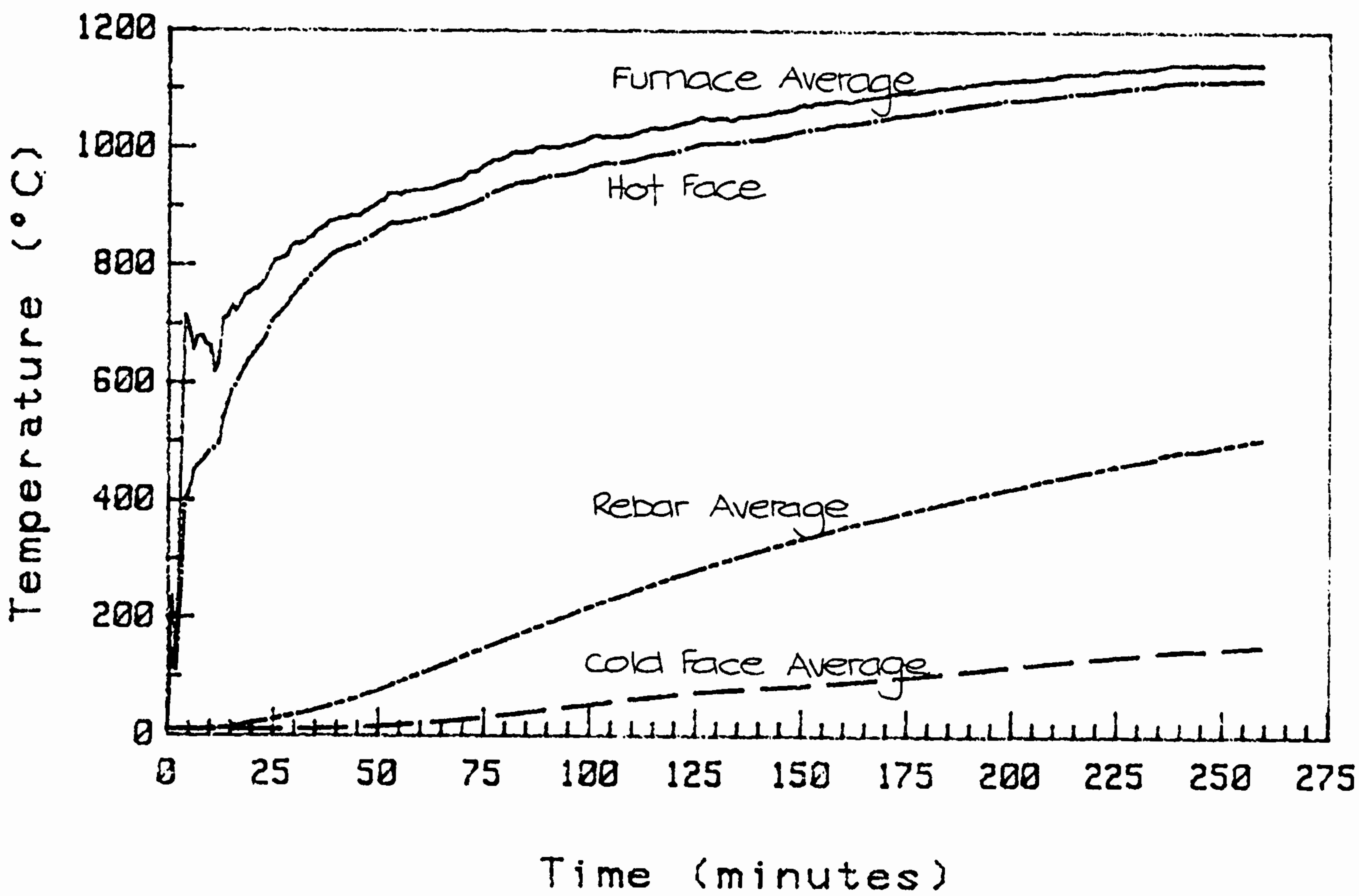


Fig 21: Pumice Reburn. External and Rebar Temperatures

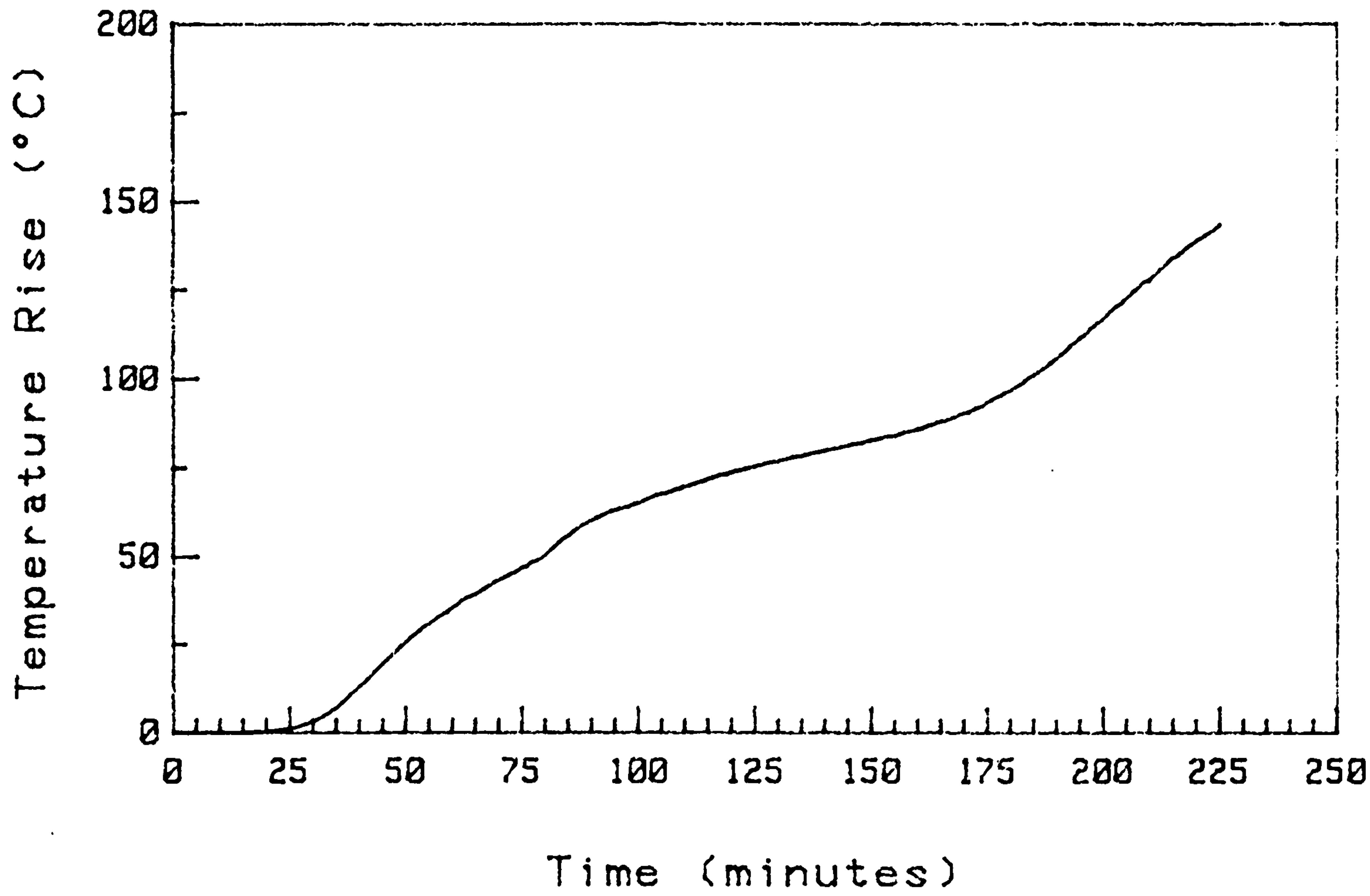


Fig 22: Alluvial Andesite. Unexposed Face Temperature

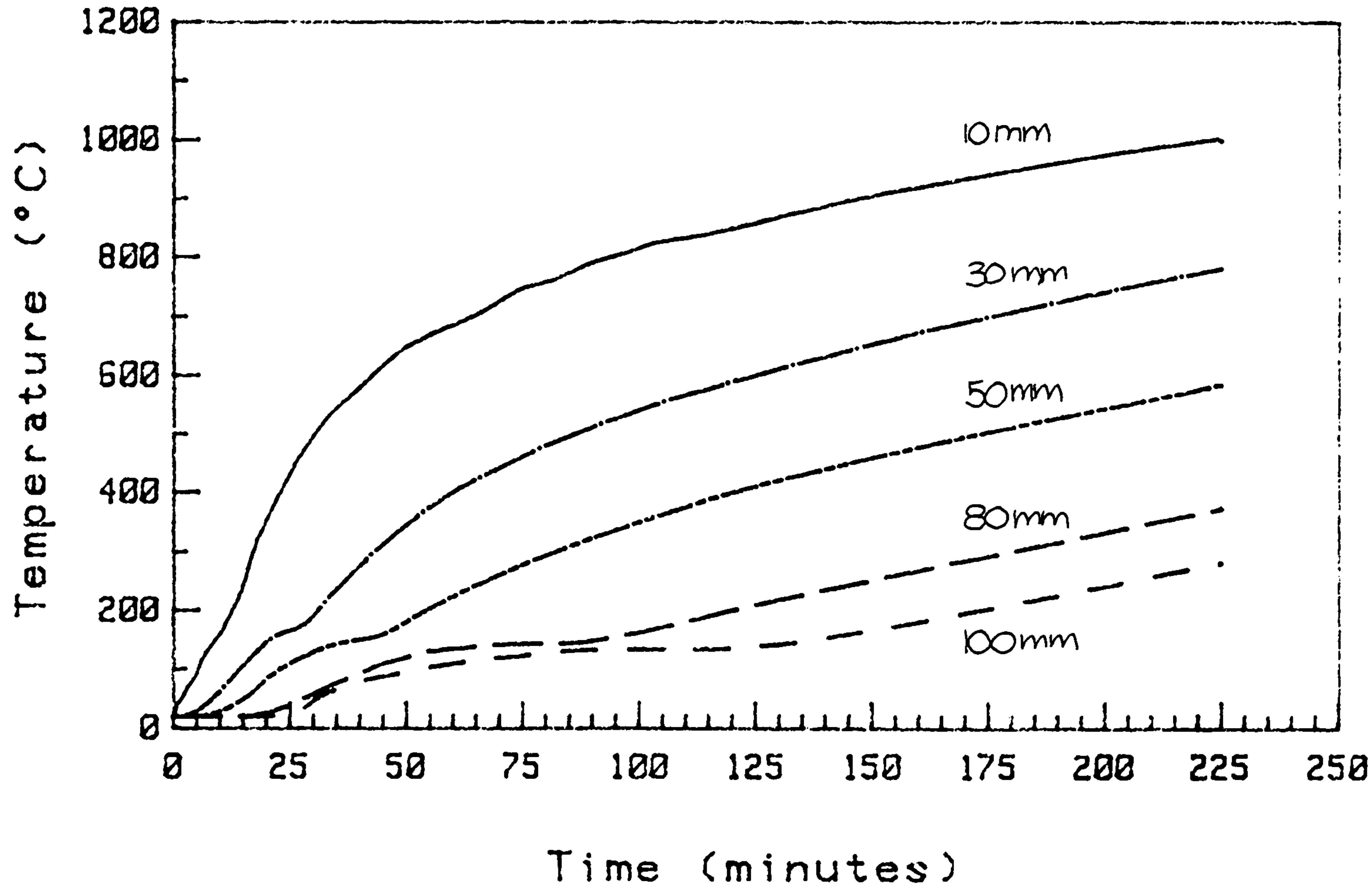


Fig 23: Alluvial Andesite. Internal Temperatures

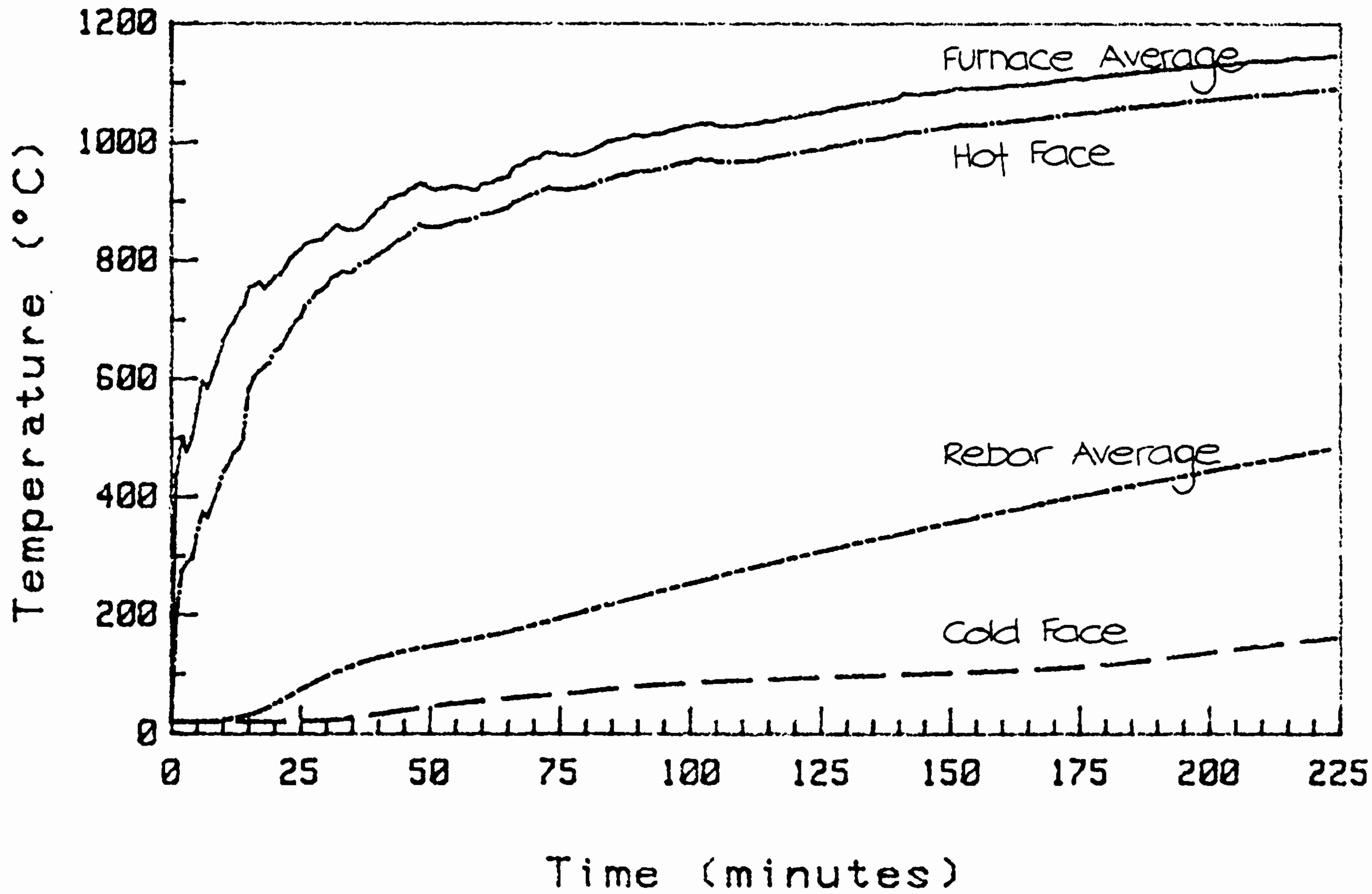


Fig 24: Alluvial Andesite. External and Rebar Temperatures

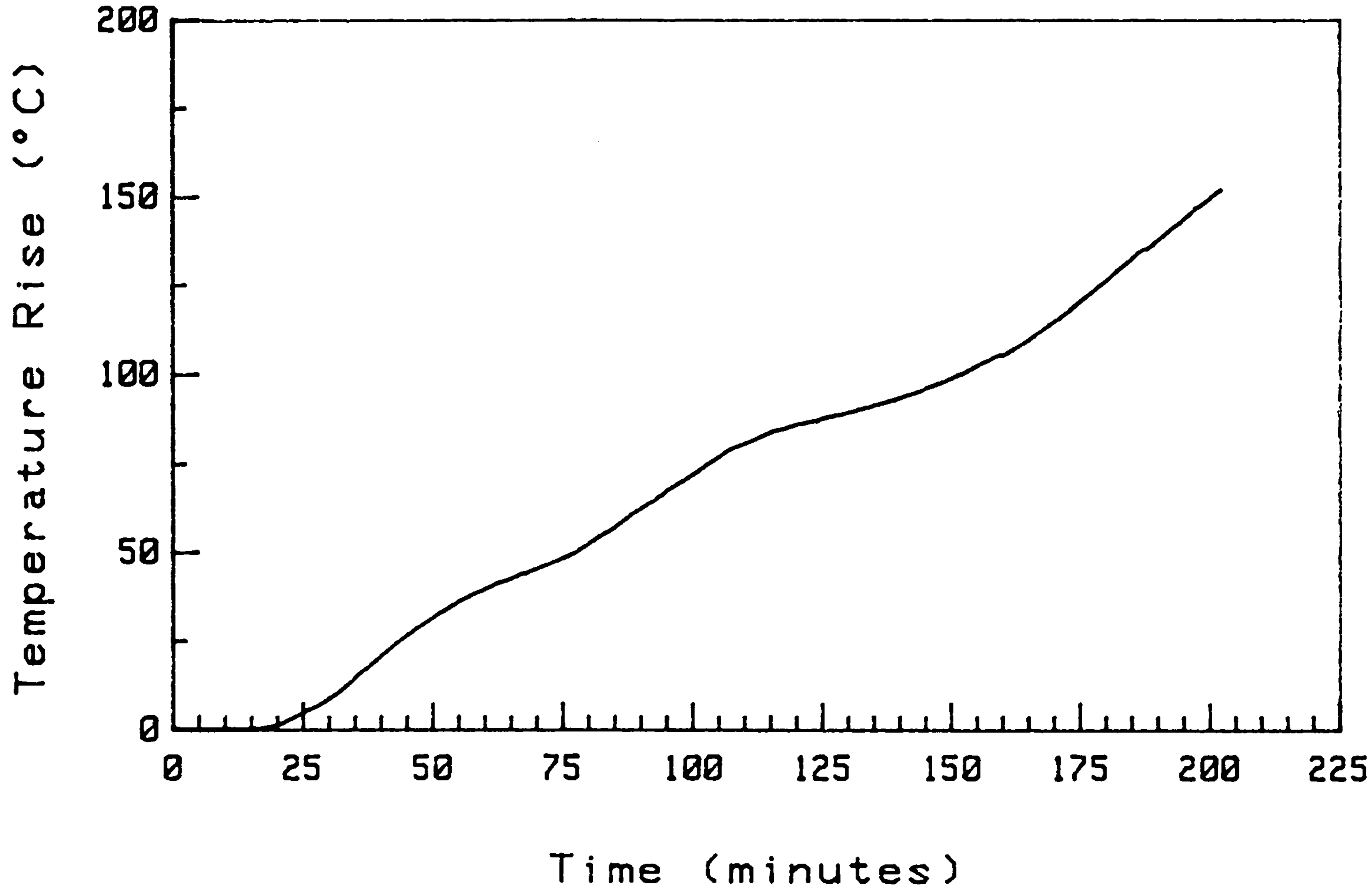


Fig 25: Quarried Andesite. Unexposed Face Temperature

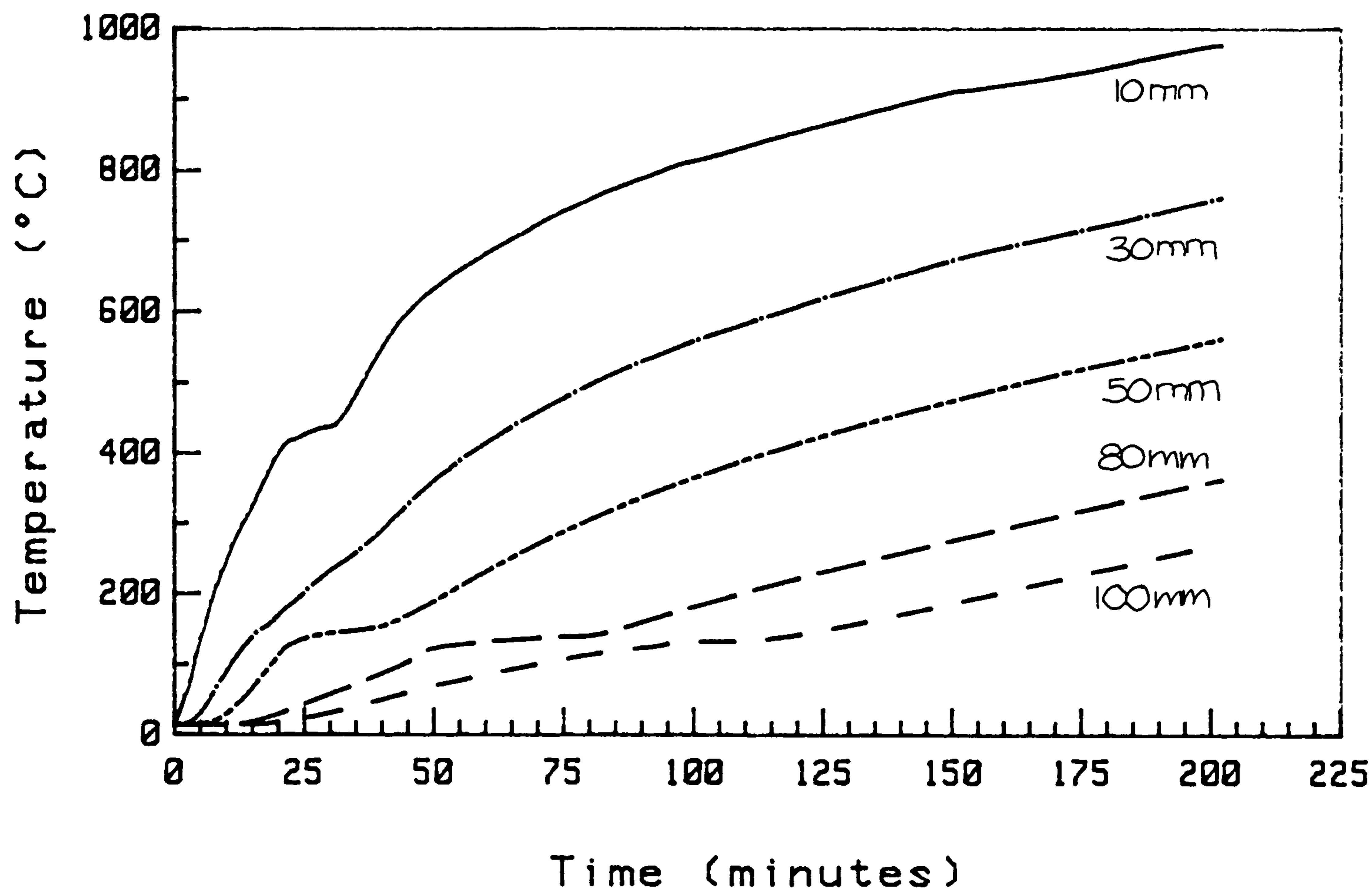


Fig 26: Quarried Andesite. Internal Temperatures

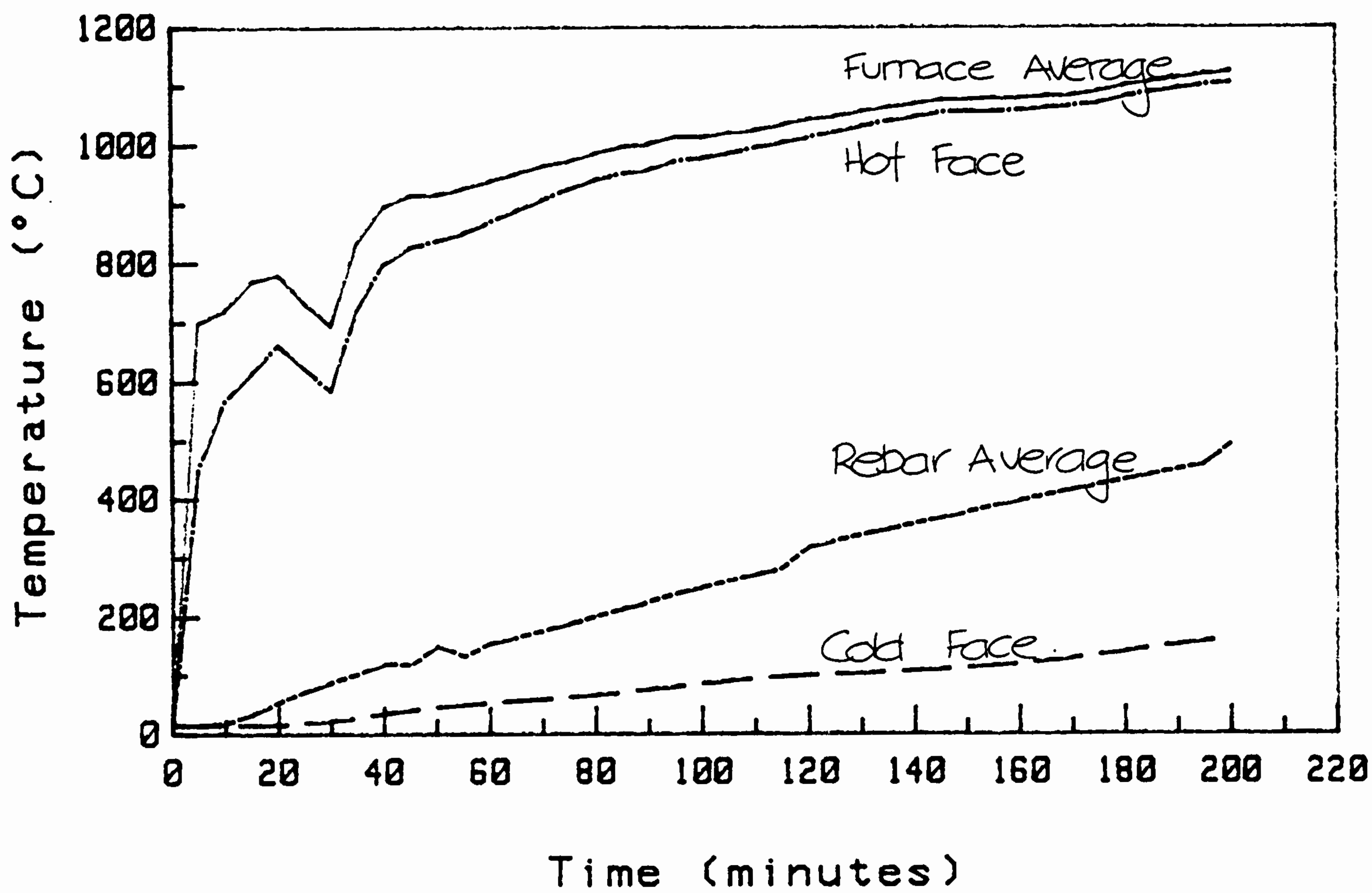


Fig 27: Quarried Andesite. External and Rebar Temperatures

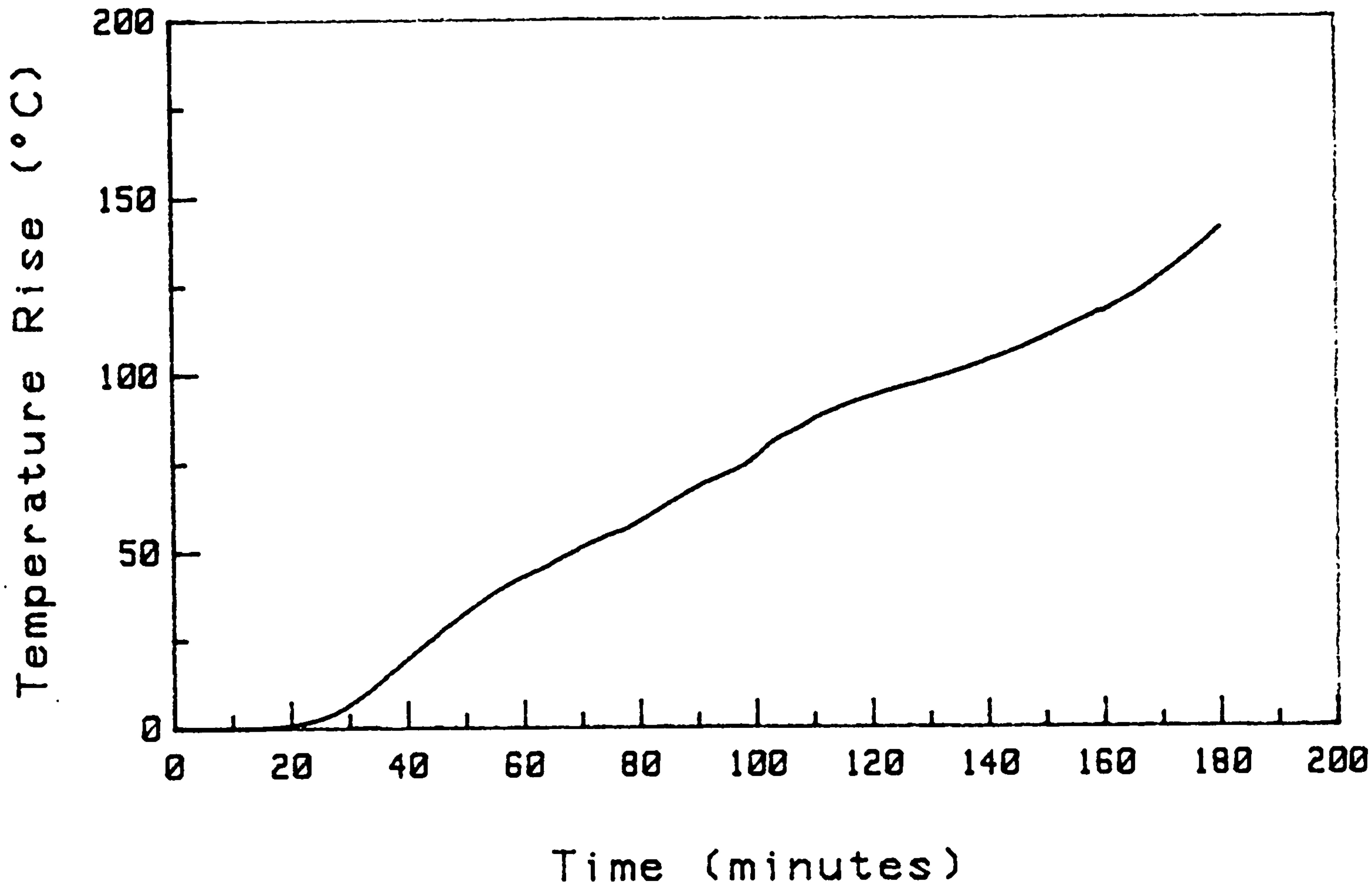


Fig 28: Quarried Basalt. Unexposed Face Temperature

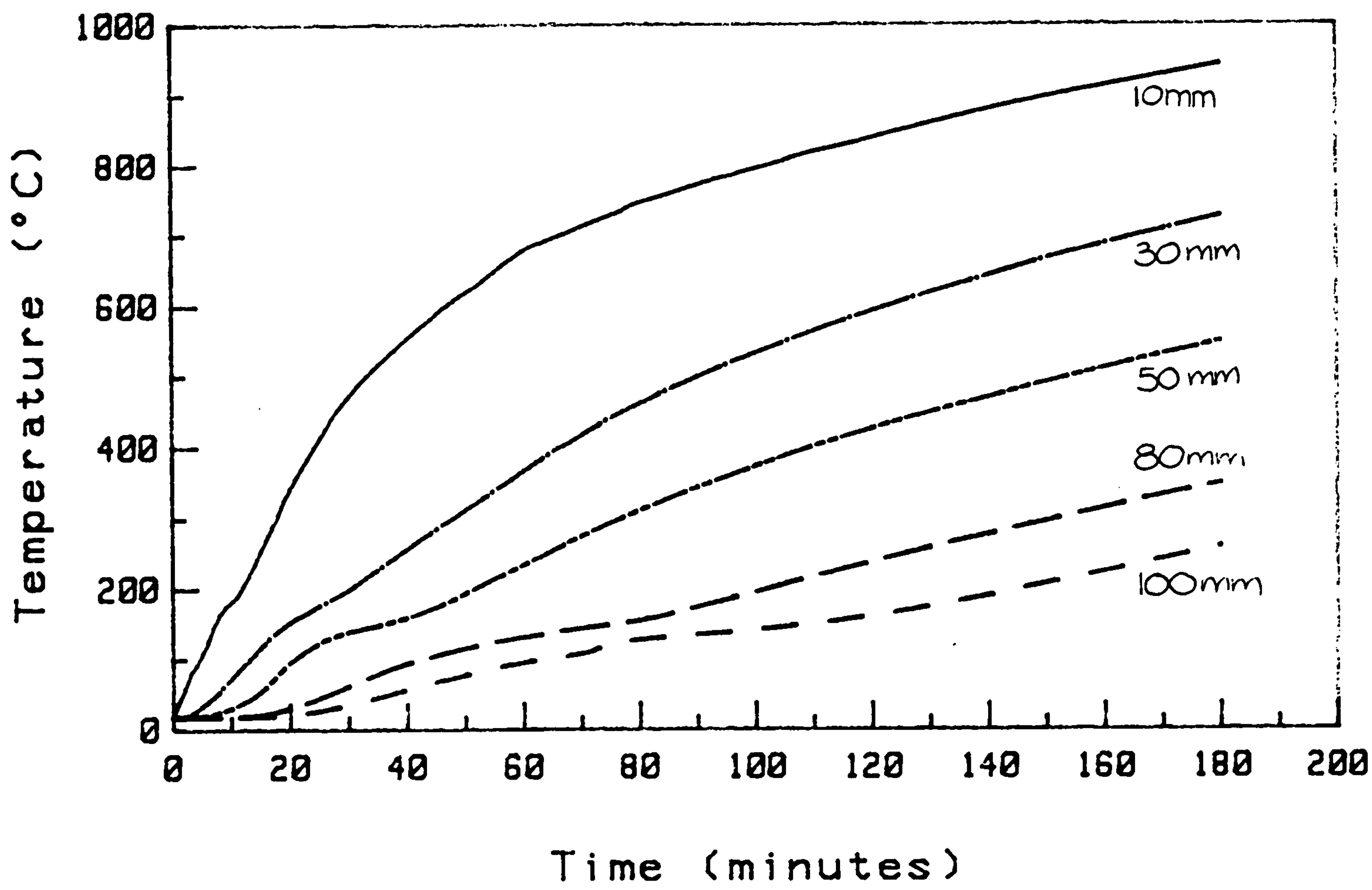


Fig 29: Quarried Basalt. Internal Temperature

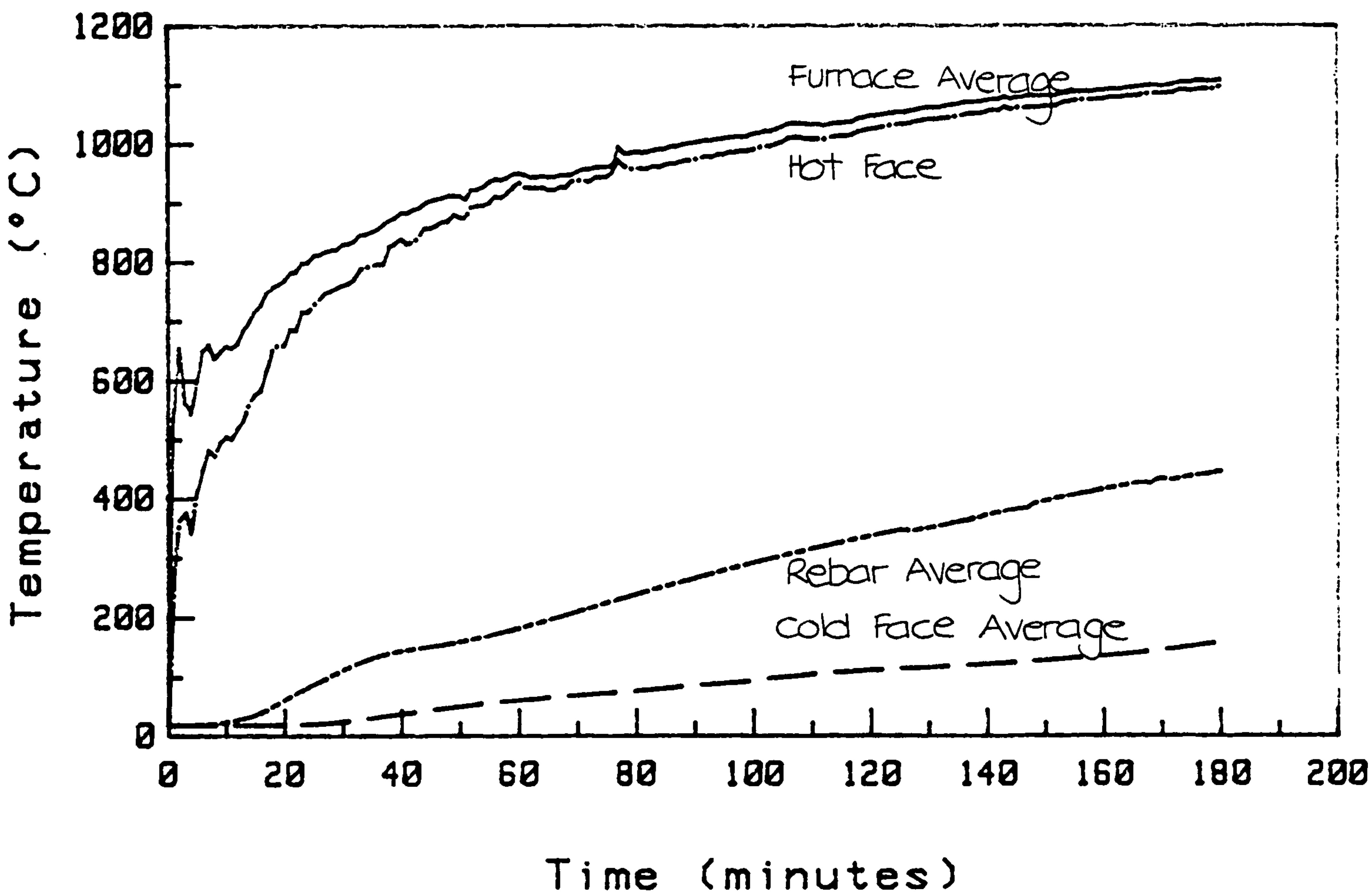


Fig 30: Quarried Basalt. External and Rebar Temperatures

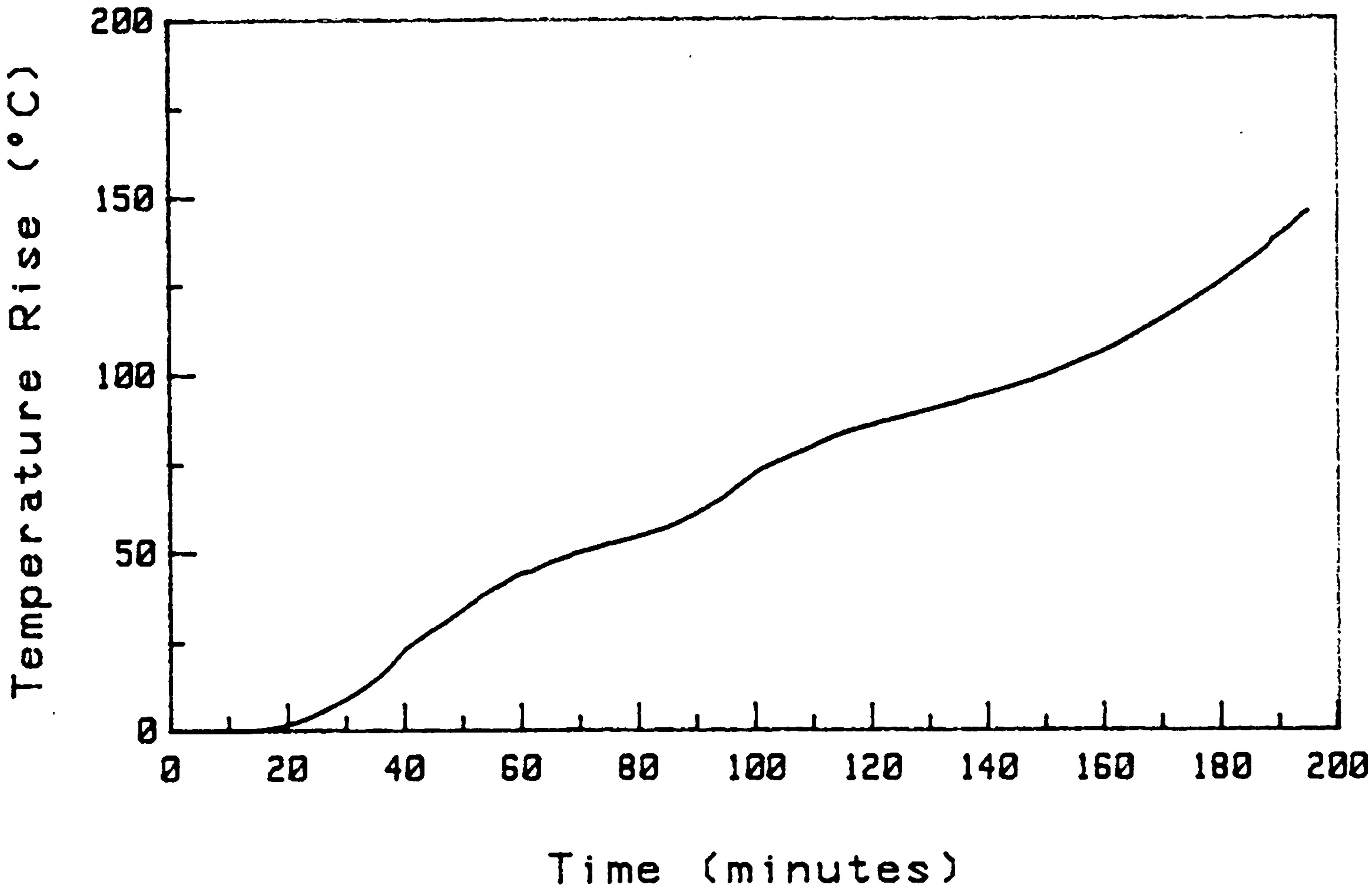


Fig 31: Phonolite. Unexposed Face Temperature

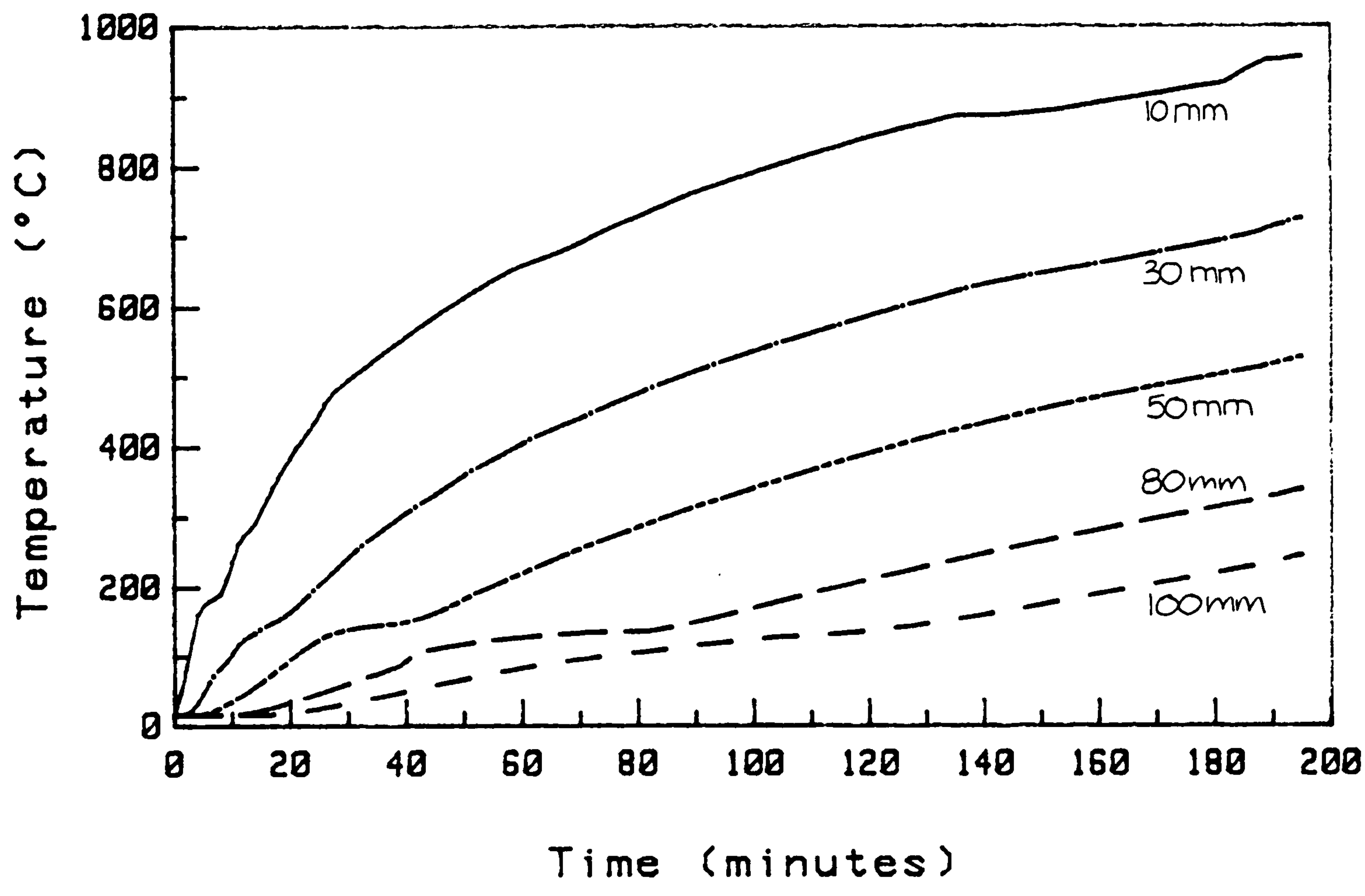


Fig 32: Phonolite. Internal Temperatures

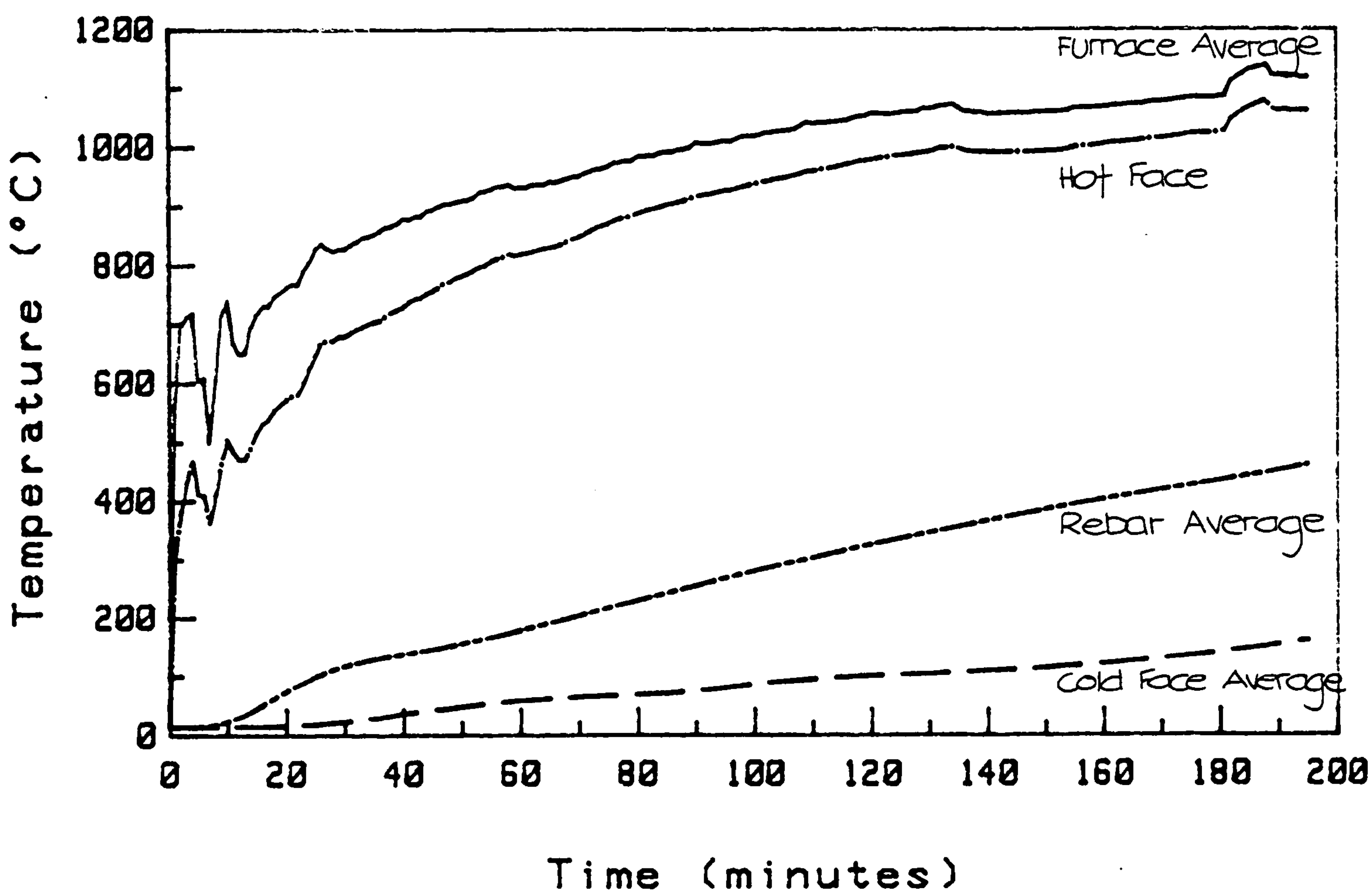


Fig 33: Phonolite. External and Rebar Temperatures

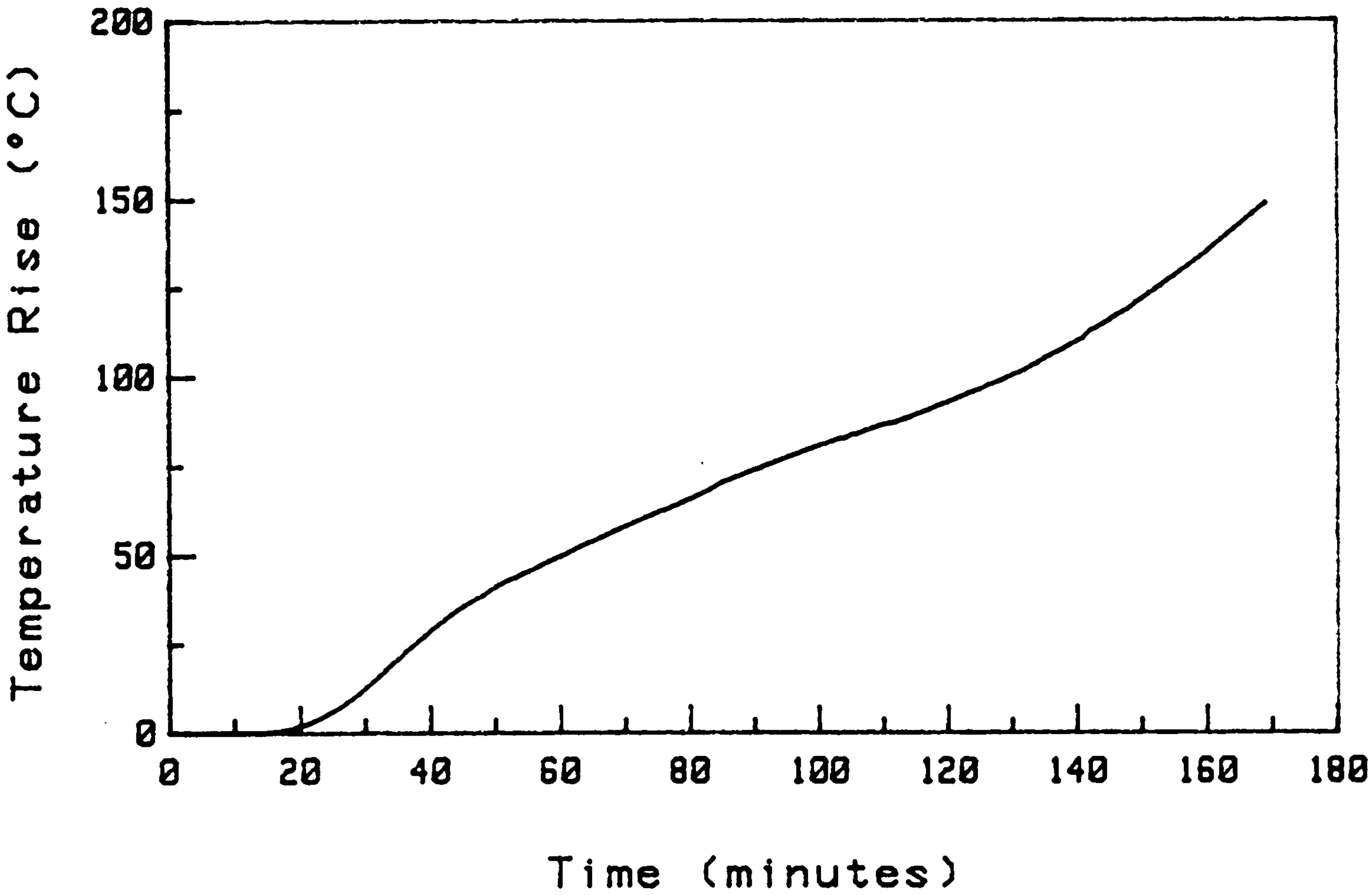


Fig 34: Alluvial Greywacke. Unexposed Face Temperature

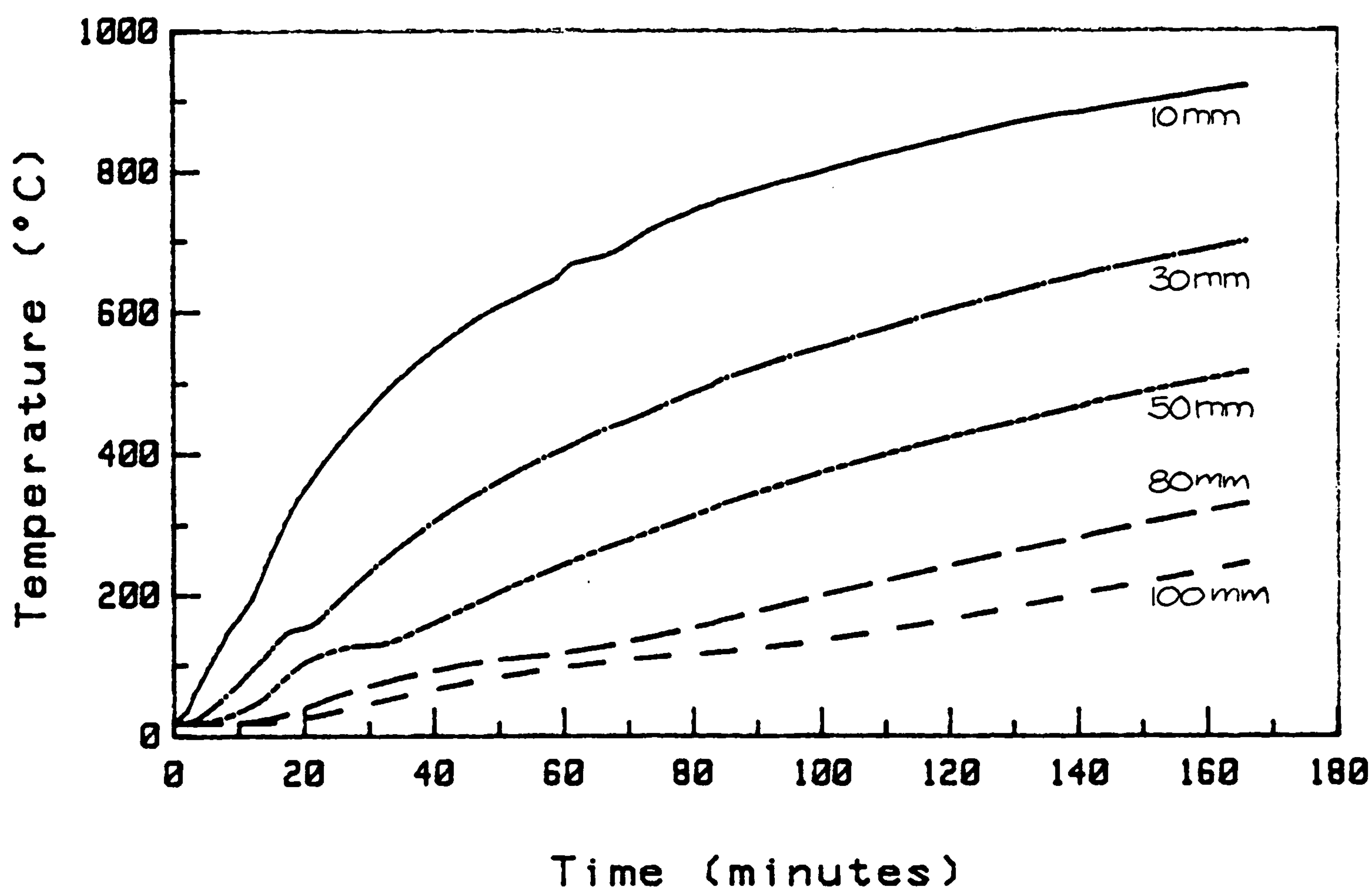


Fig 35: Alluvial Greywacke. Internal Temperatures

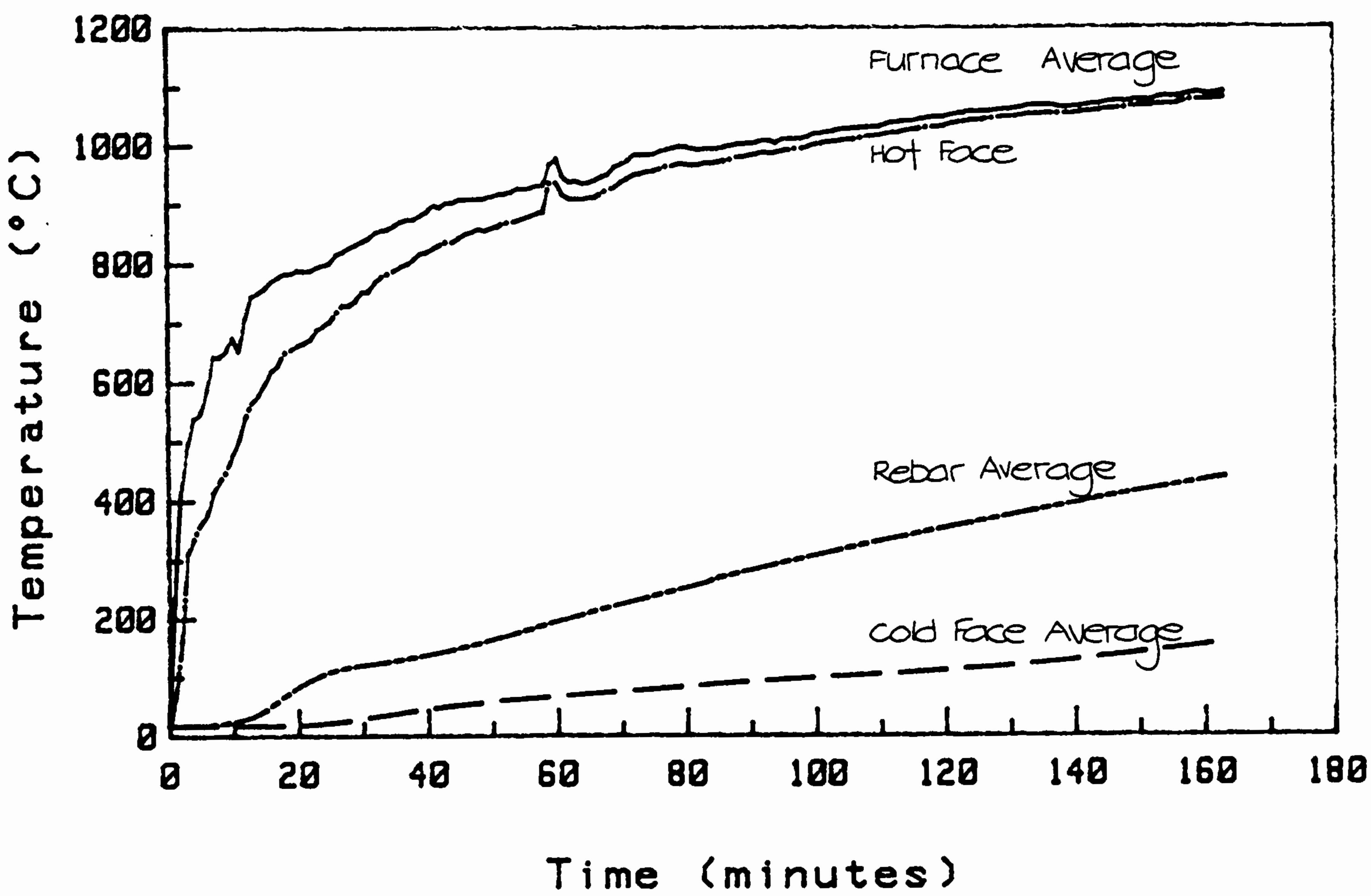


Fig 36: Alluvial Greywacke. External and Rebar Temperatures

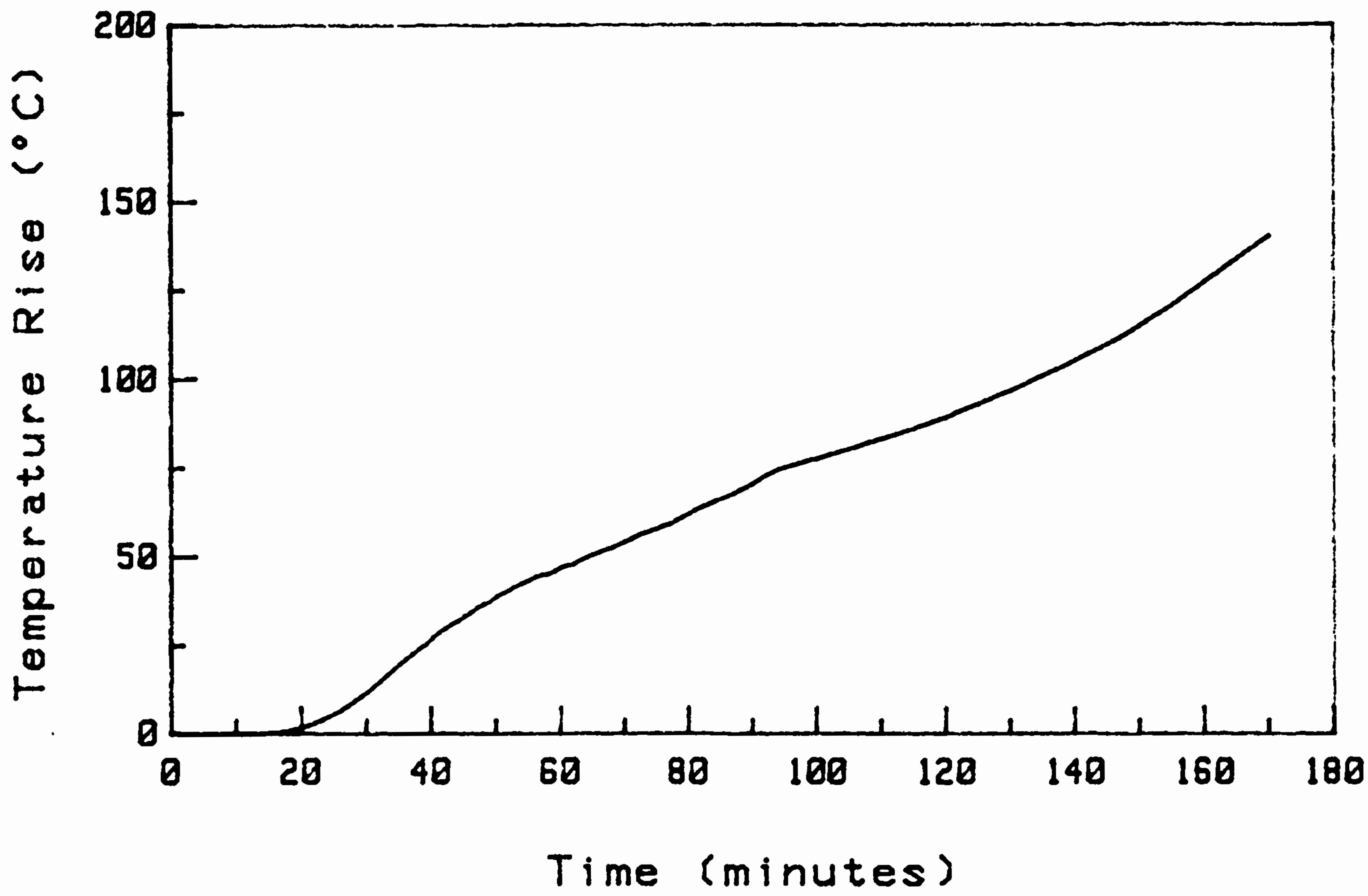


Fig 37: Quarried Greywacke (1). Unexposed Face Temperature

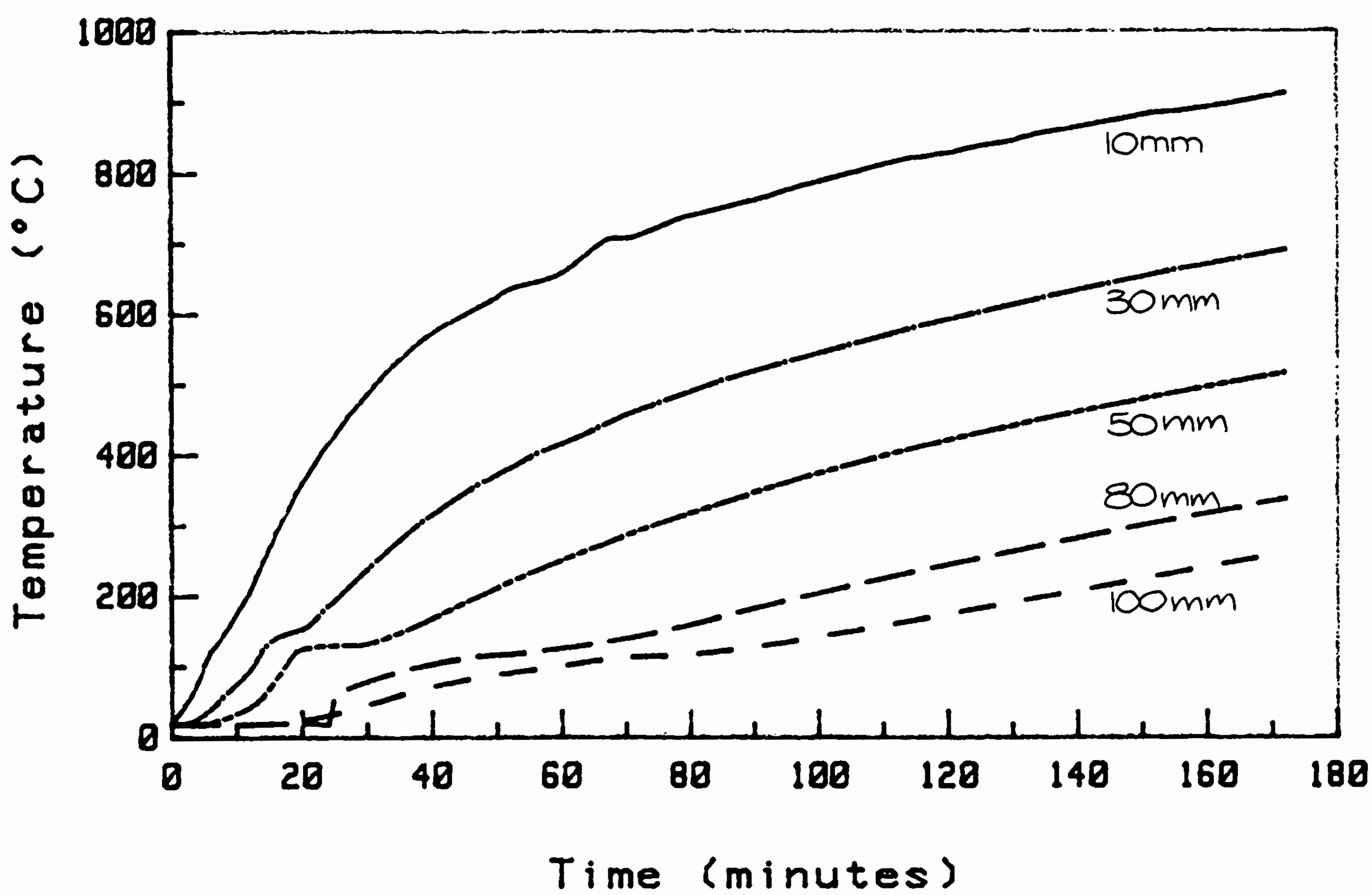


Fig 38: Quarried Greywacke (1). Internal Temperatures

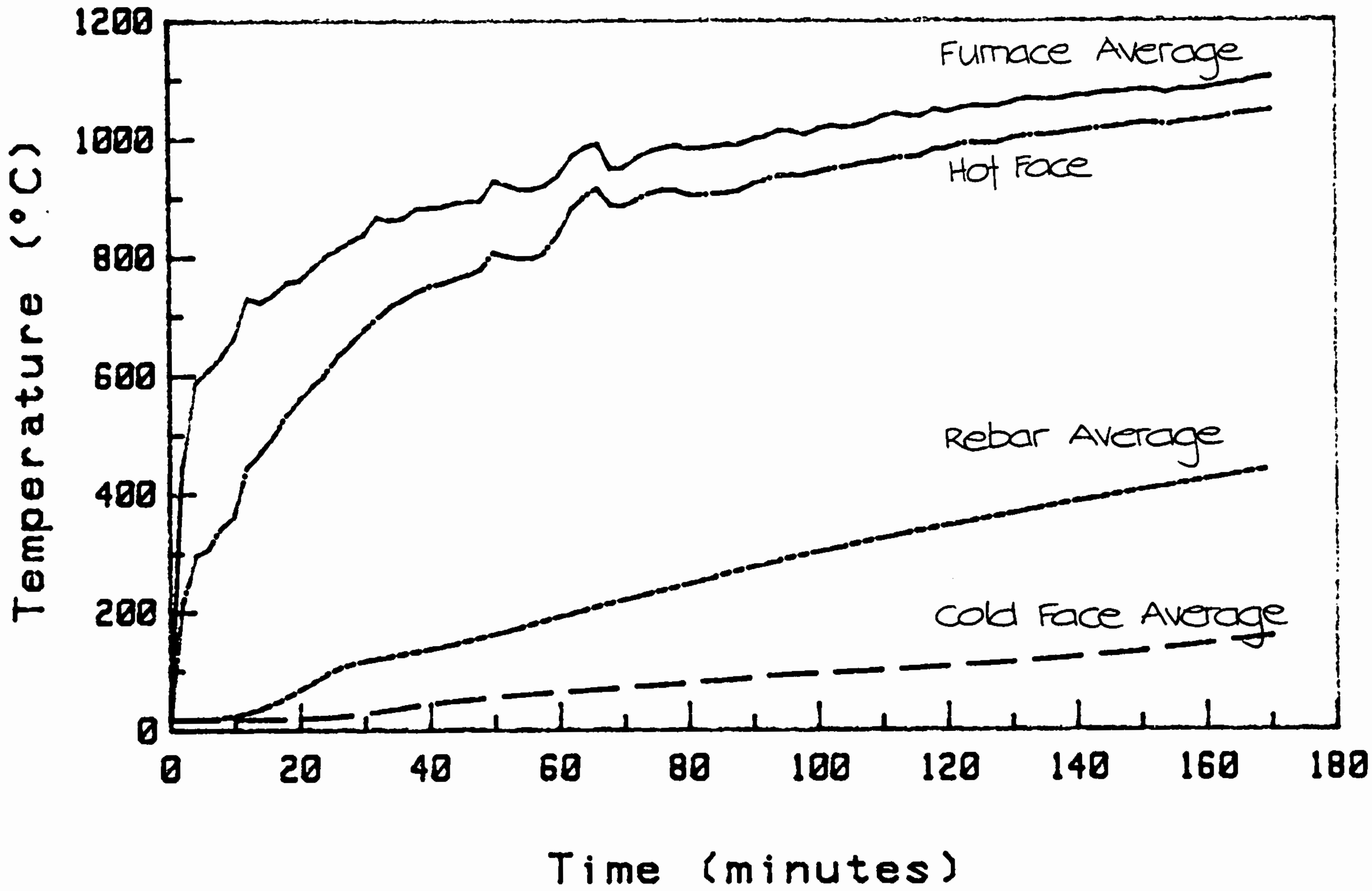


Fig 39: Quarried Greywacke (1). External and Rebar Temperatures

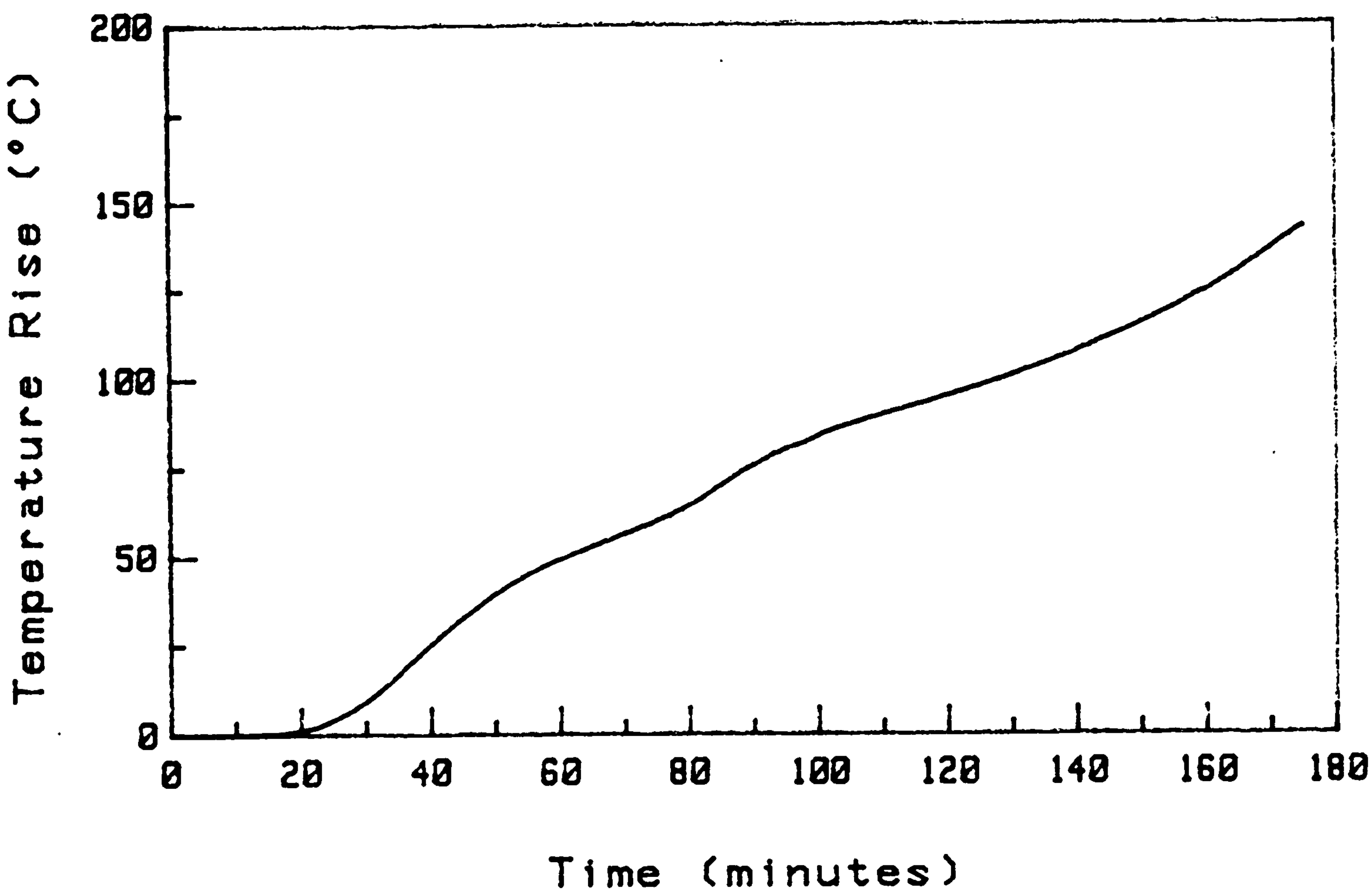


Fig 40: Quarried Greywacke (2). Unexposed Face Temperature

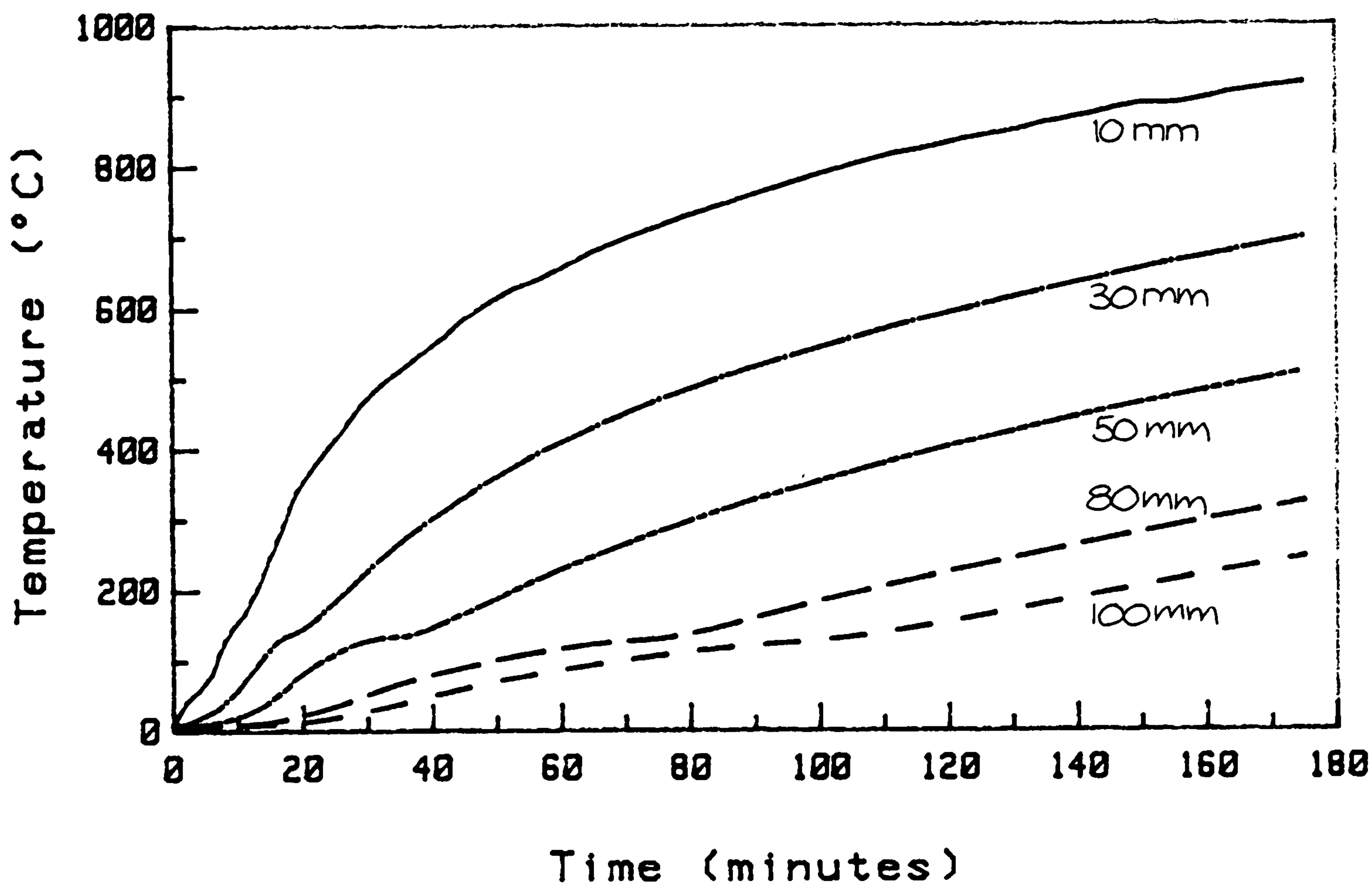


Fig 41: Quarried Greywacke (2). Internal Temperatures

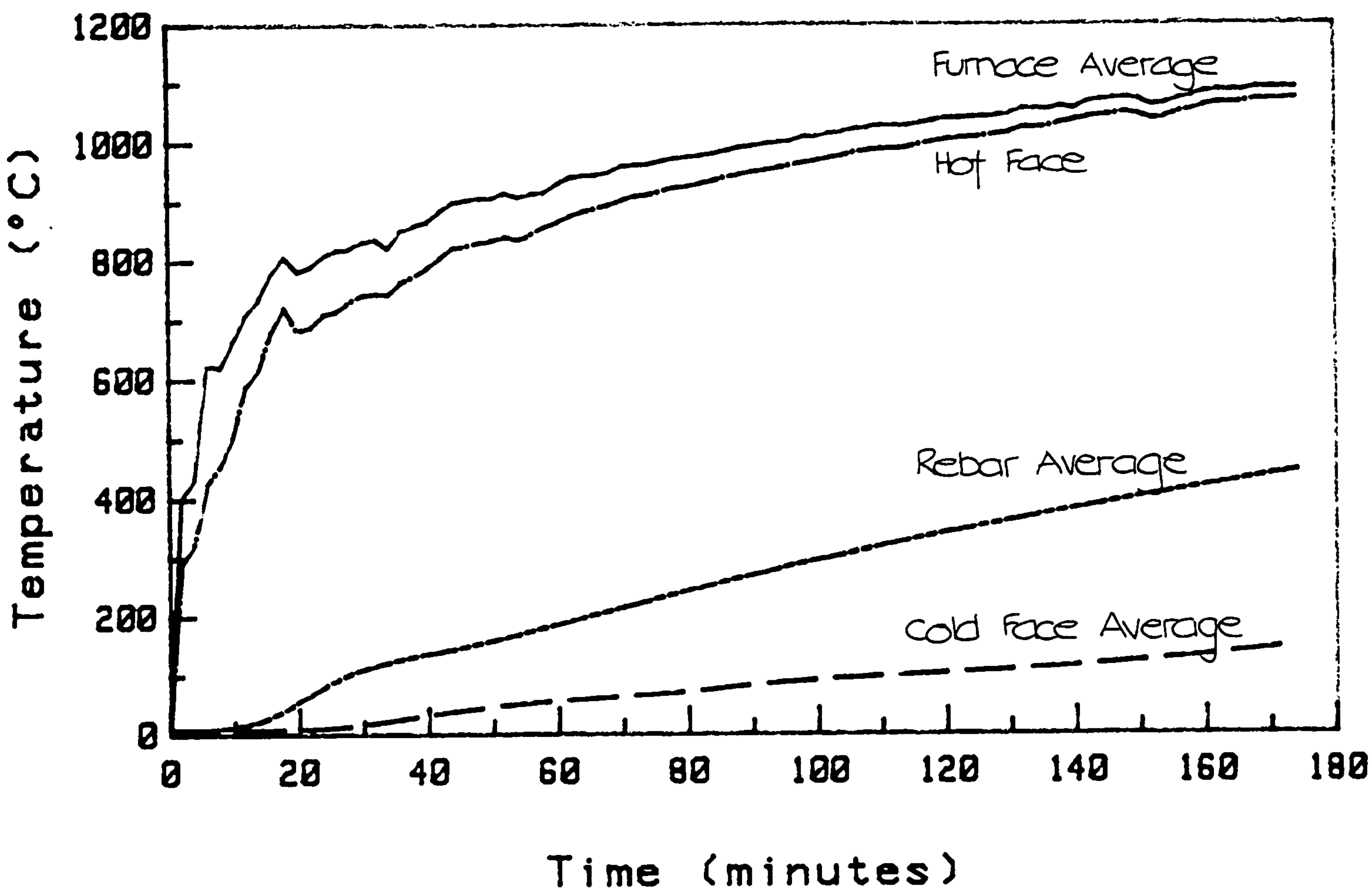


Fig 42: Quarried Greywacke (2). External and Rebar Temperatures

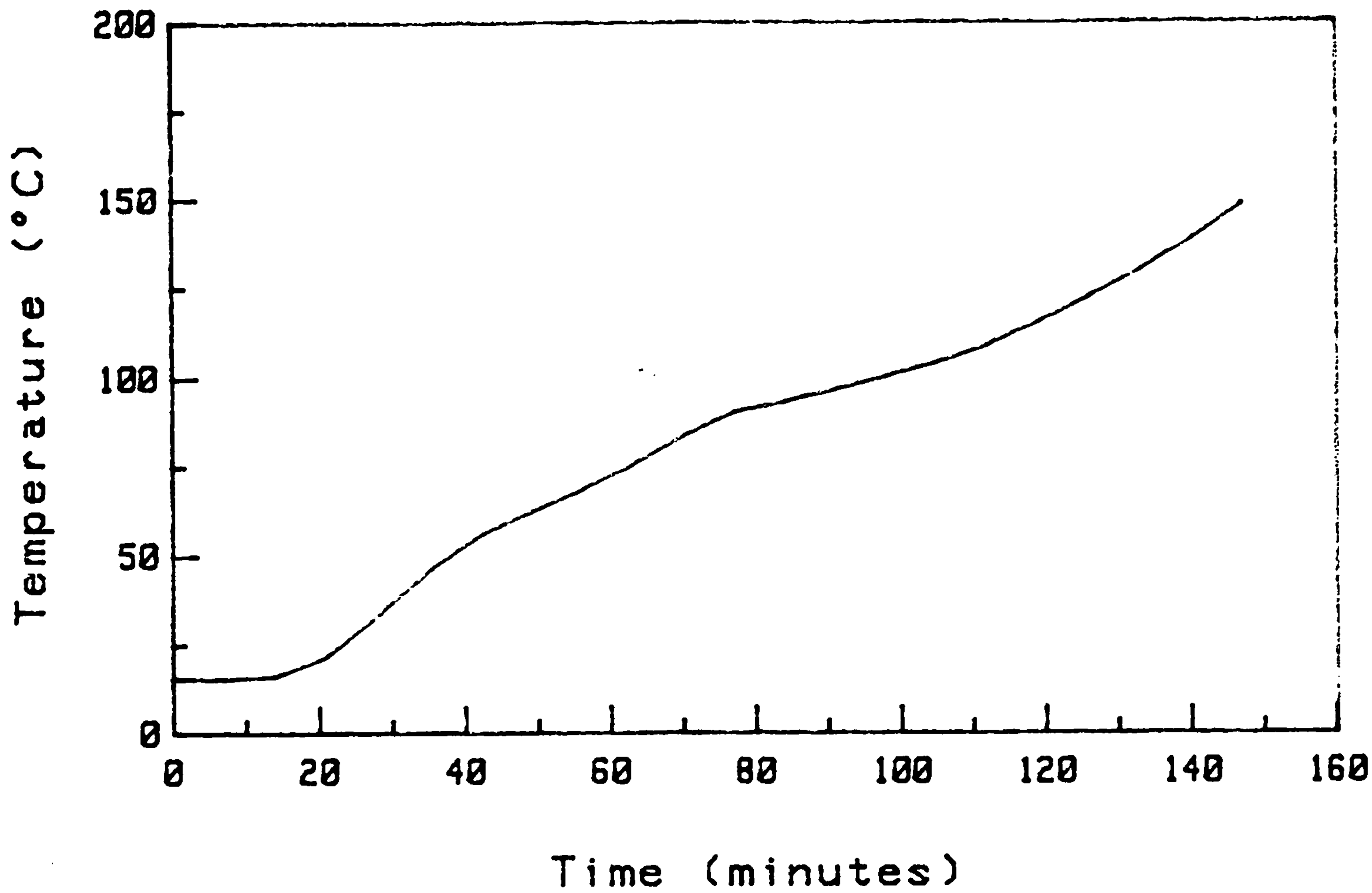


Fig 43: Alluvial Quartz. Unexposed Face Temperature

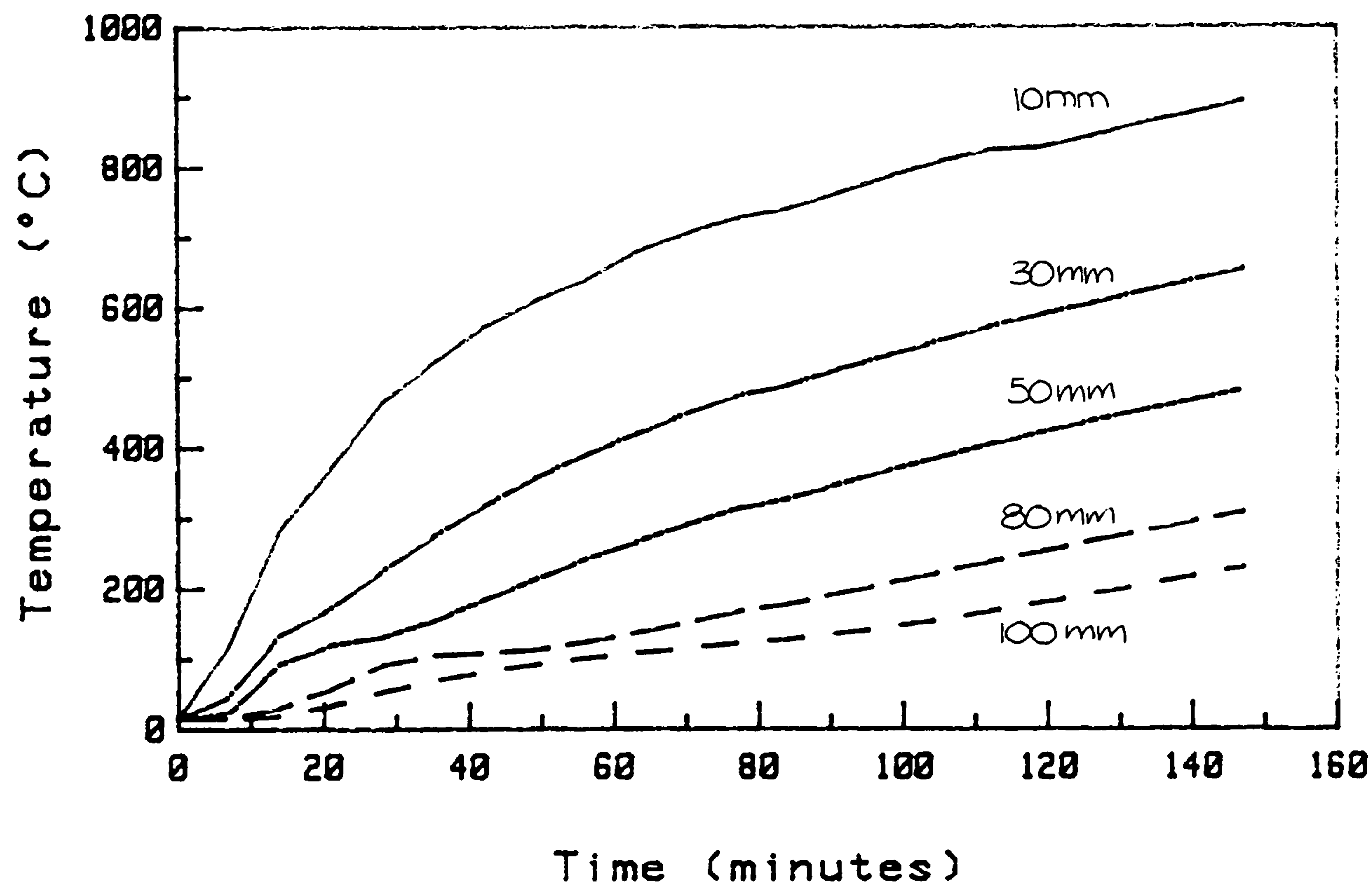


Fig 44: Alluvial Quartz. Internal Temperatures

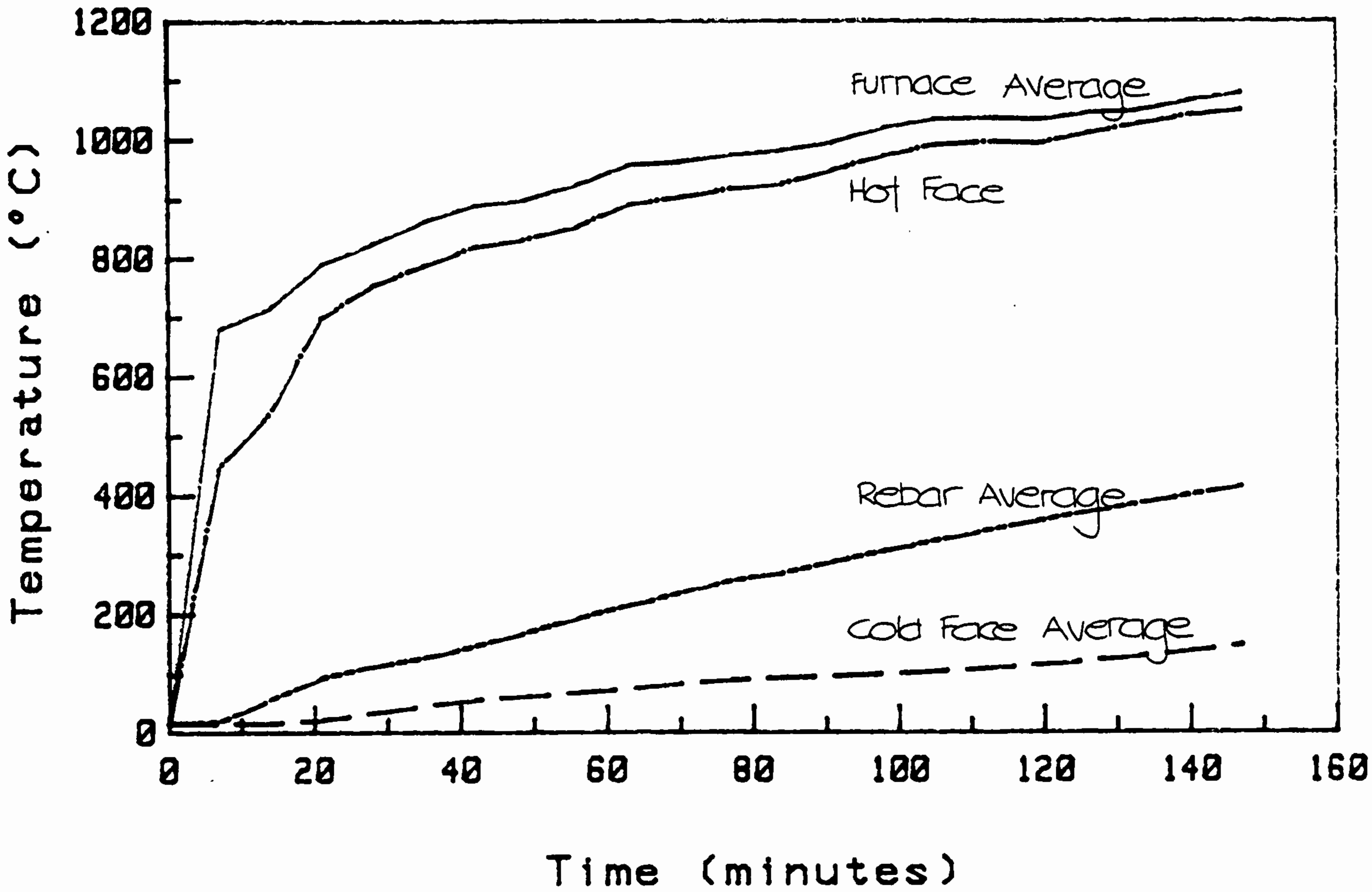


Fig 45: Alluvial Quartz. External and Rebar Temperatures

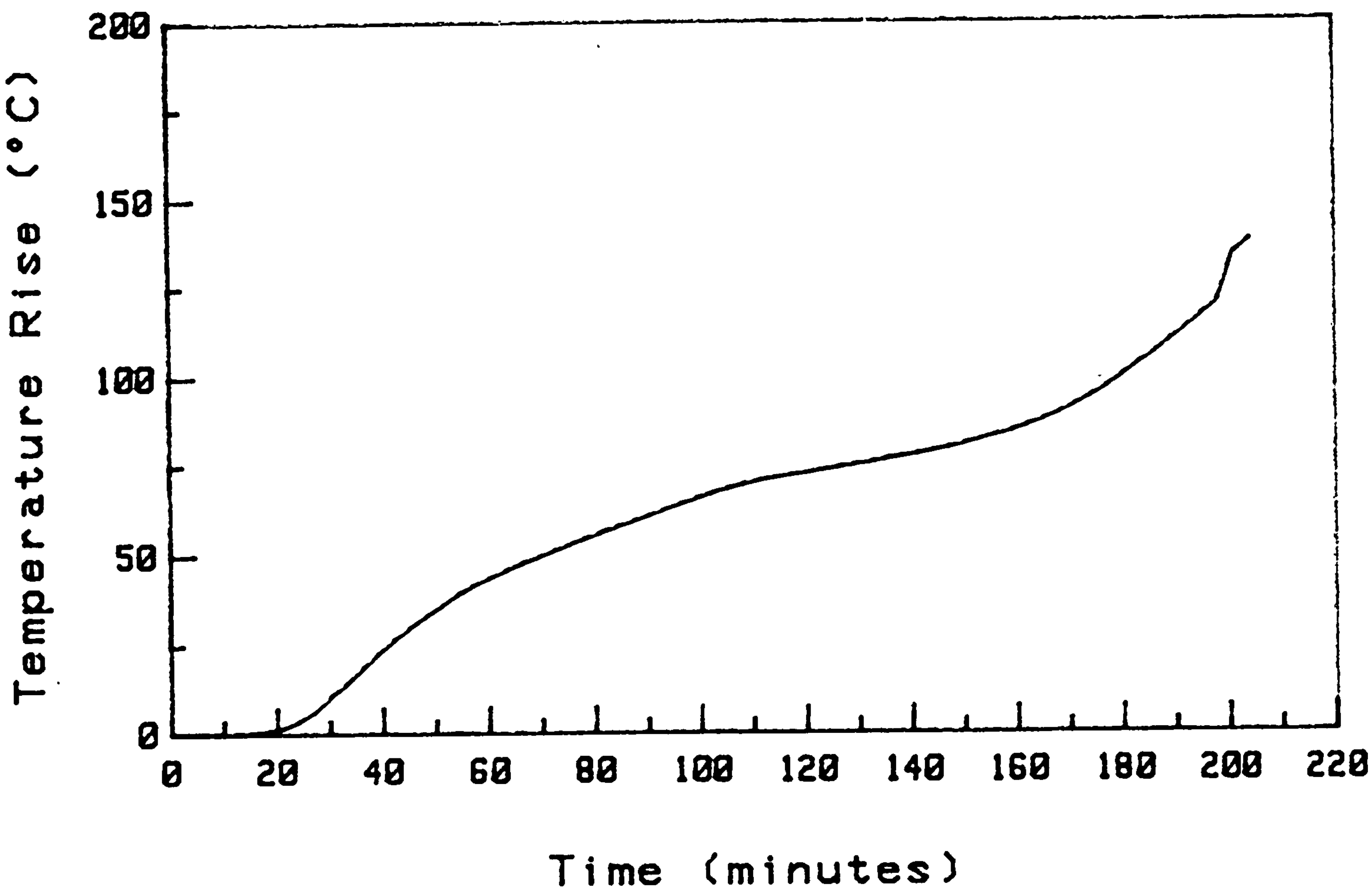


Fig 46: Coquina Limestone. Unexposed Face Temperature

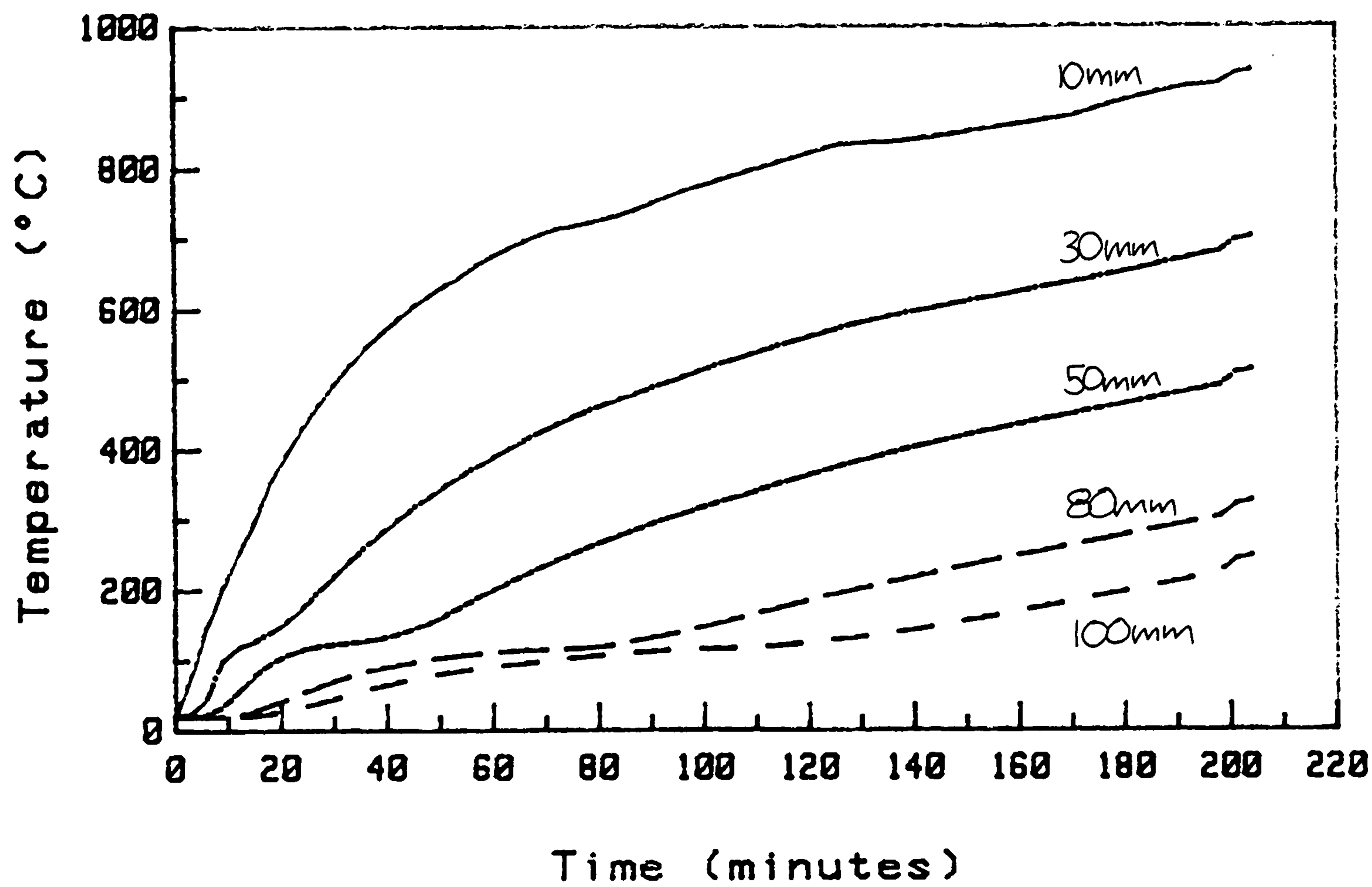


Fig 47: Coquina Limestone. Internal Temperatures

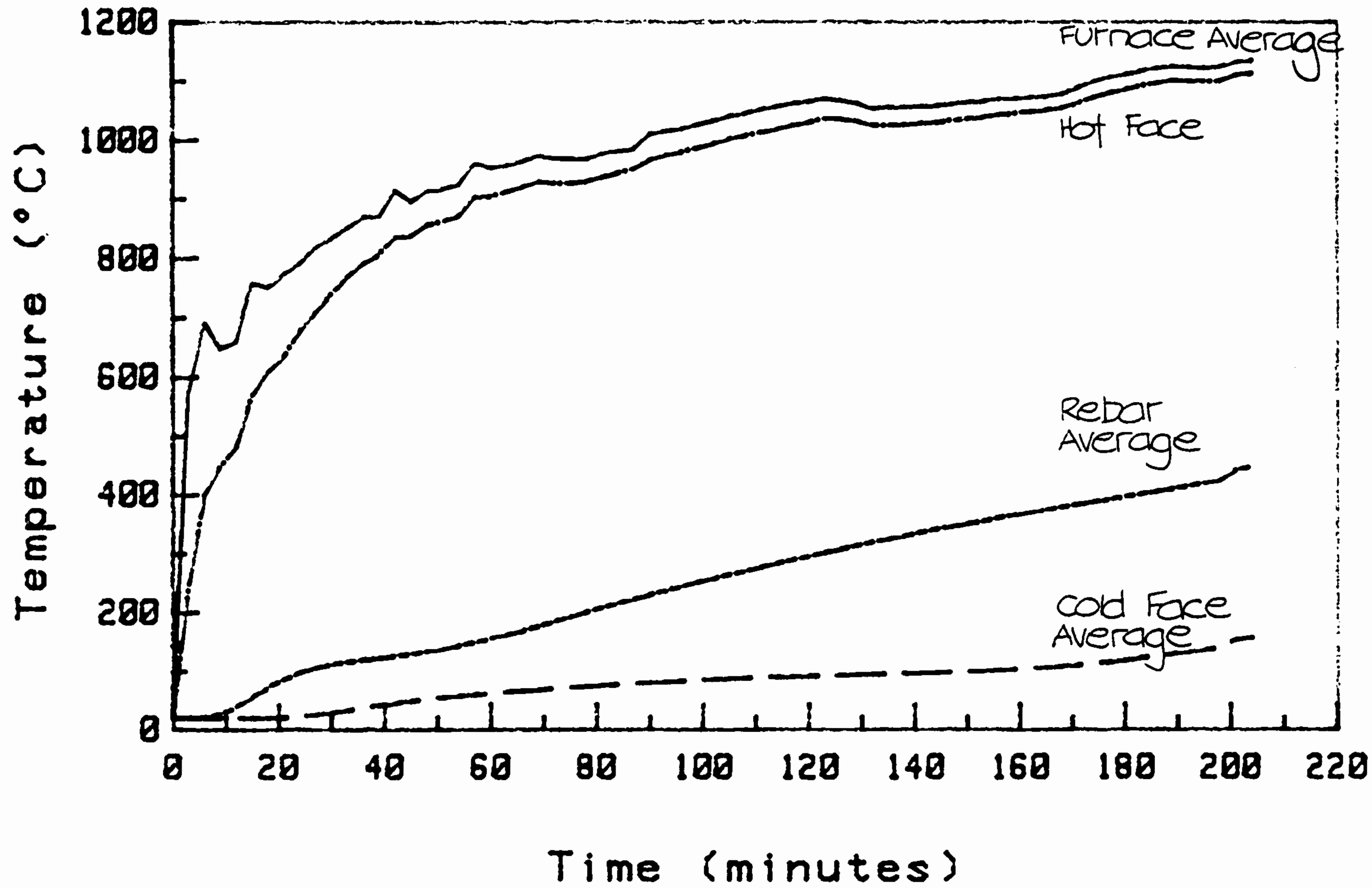


Fig 48: Coquina Limestone. External and Rebar Temperatures

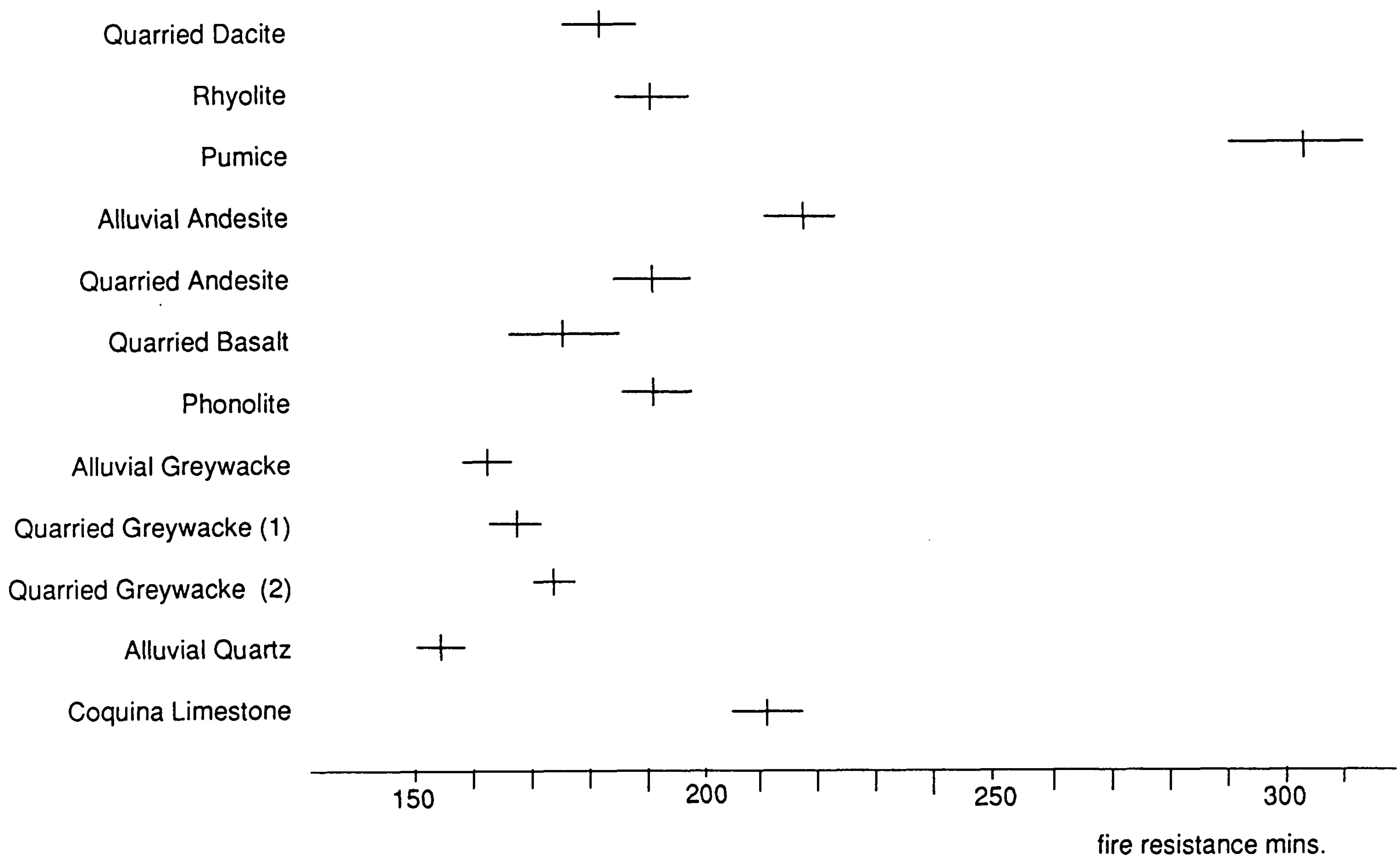


Figure 49: 95% Confidence Levels for Fire Resistance of 130 mm Thick Slabs (Corrected to 75% RH) - *see table 5*

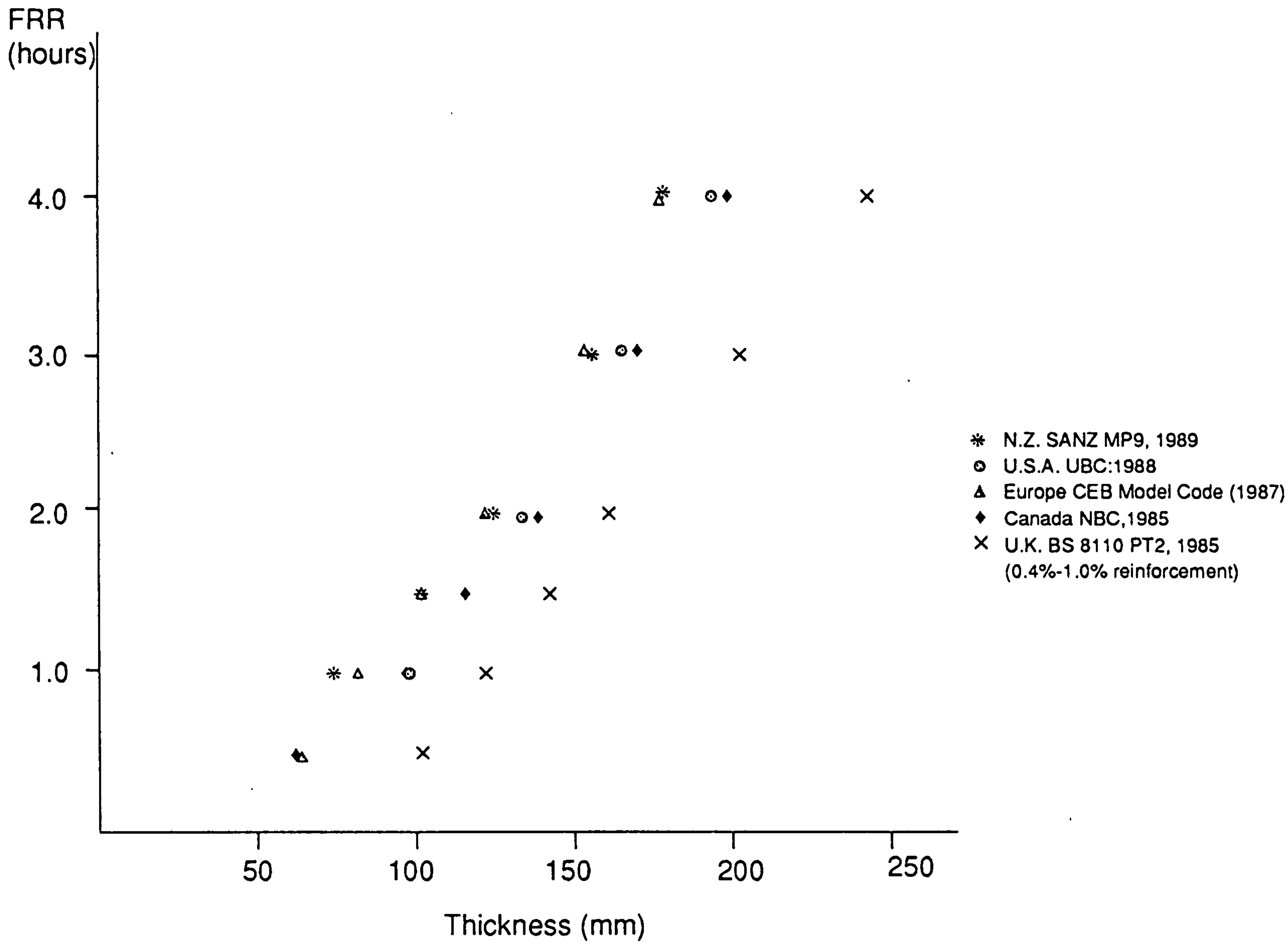


Figure 50: FRR for Reinforced Concrete Non-loadbearing Dense or Siliceous Aggregate Walls.

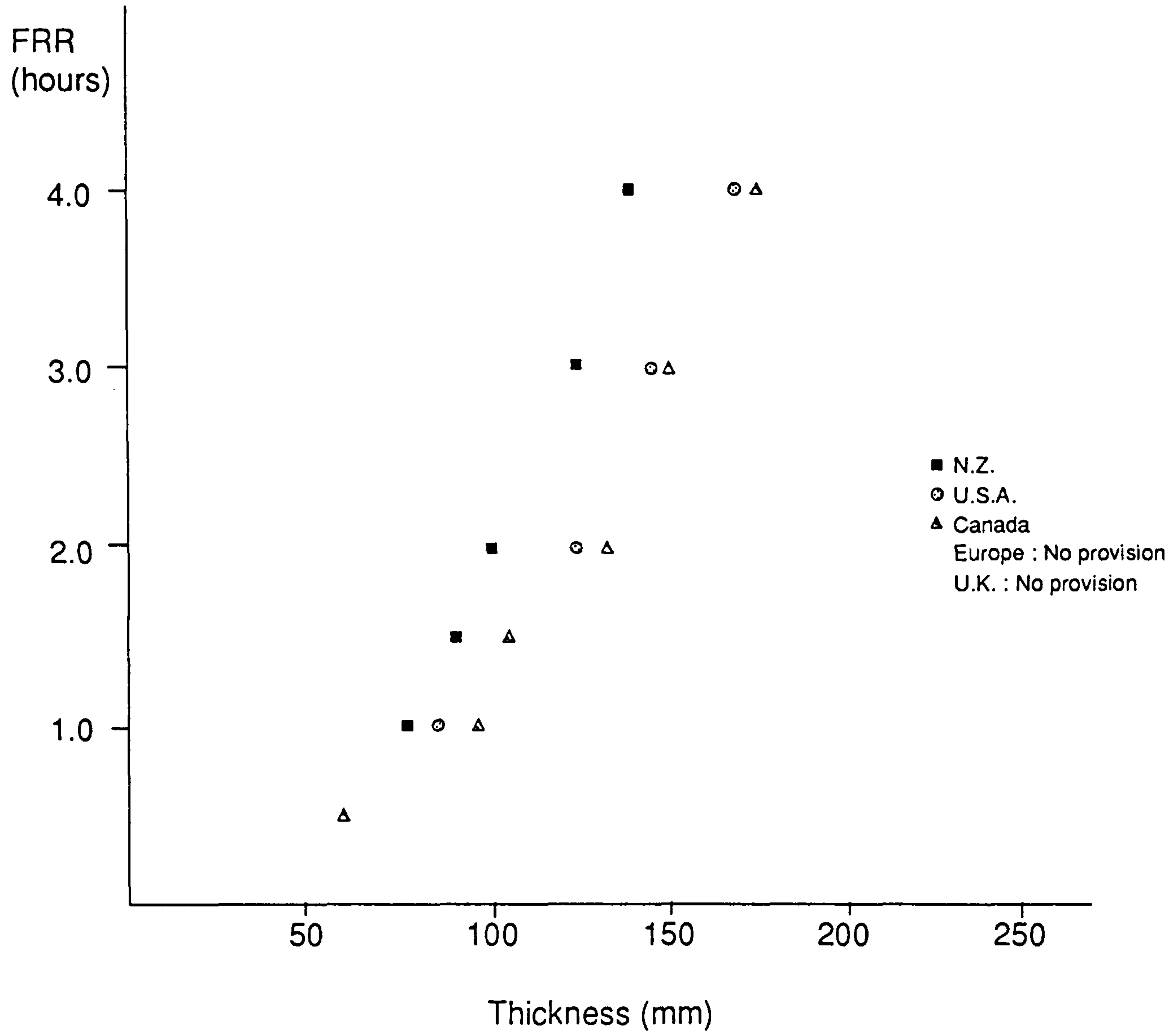


Figure 51: FRR for Reinforced Concrete, Non-loadbearing Calcareous or Limestone Aggregate Walls

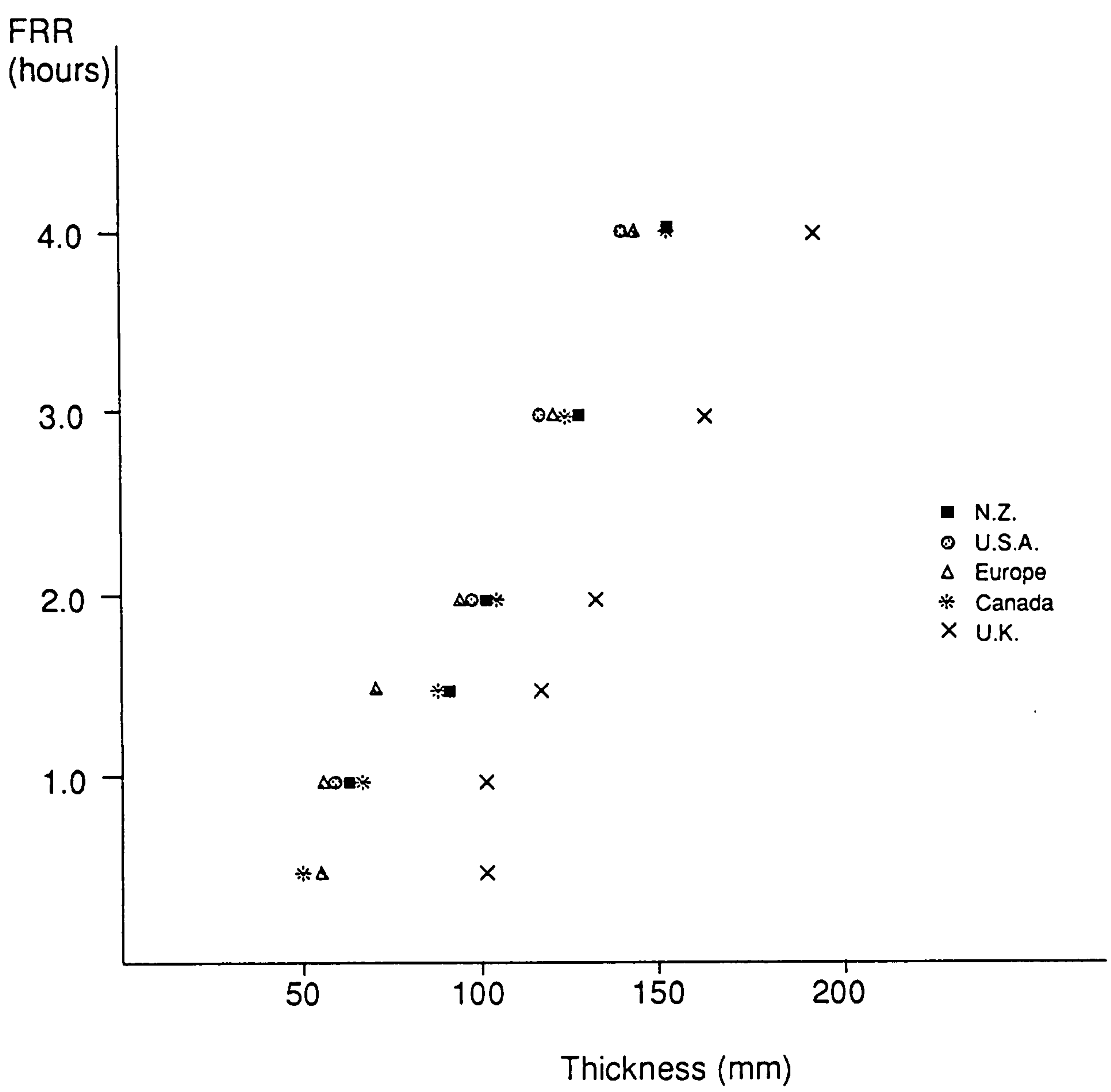


Figure 52: FRR for Reinforced Concrete Non-loadbearing Lightweight or Pumice Aggregate Walls.

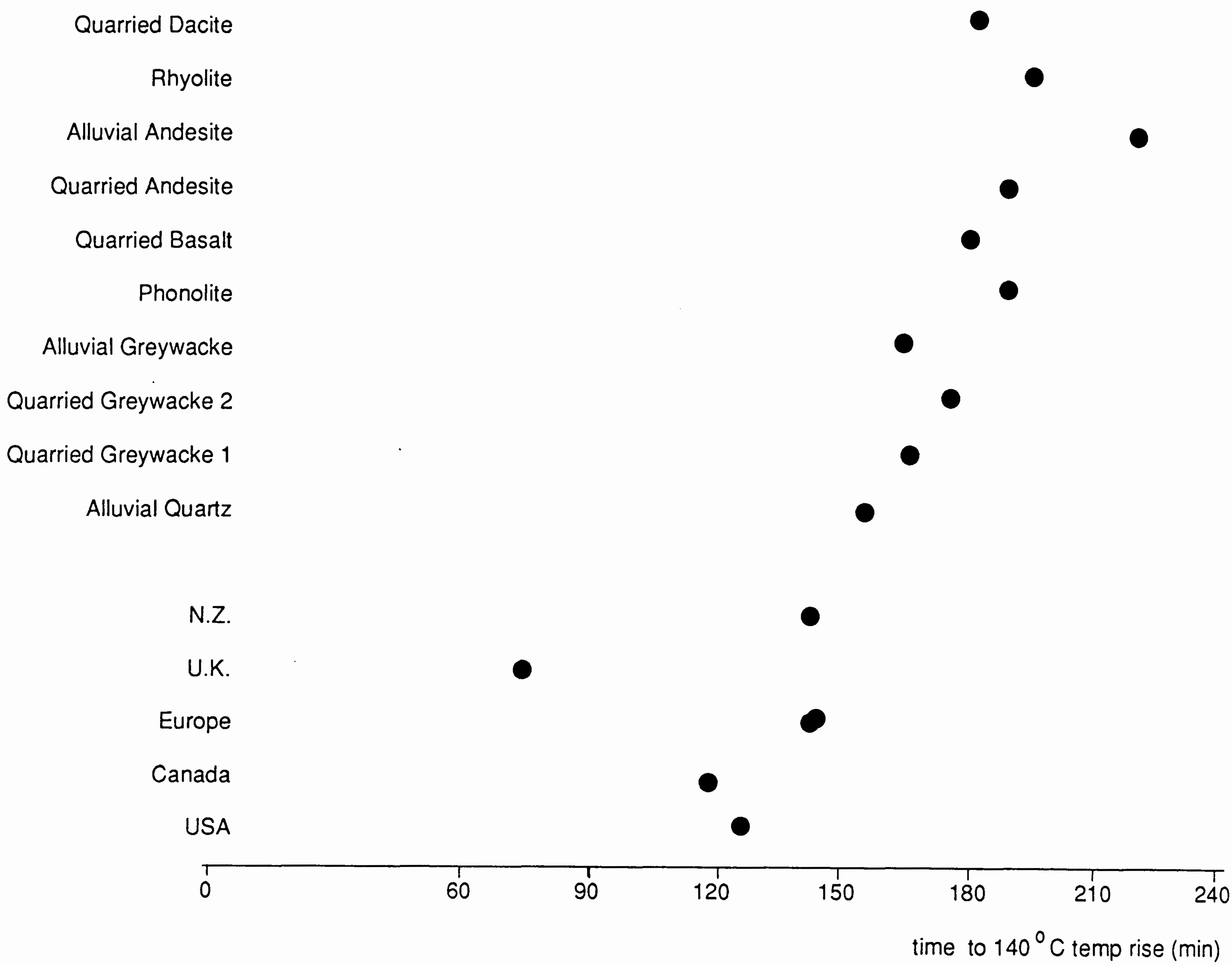


Figure 53: Comparison BRANZ Tests with Overseas Standard Recomendations of Fire Resistance for 130 mm Thick, Reinforced, Non-loadbearing, Dense Aggregate Concrete Walls

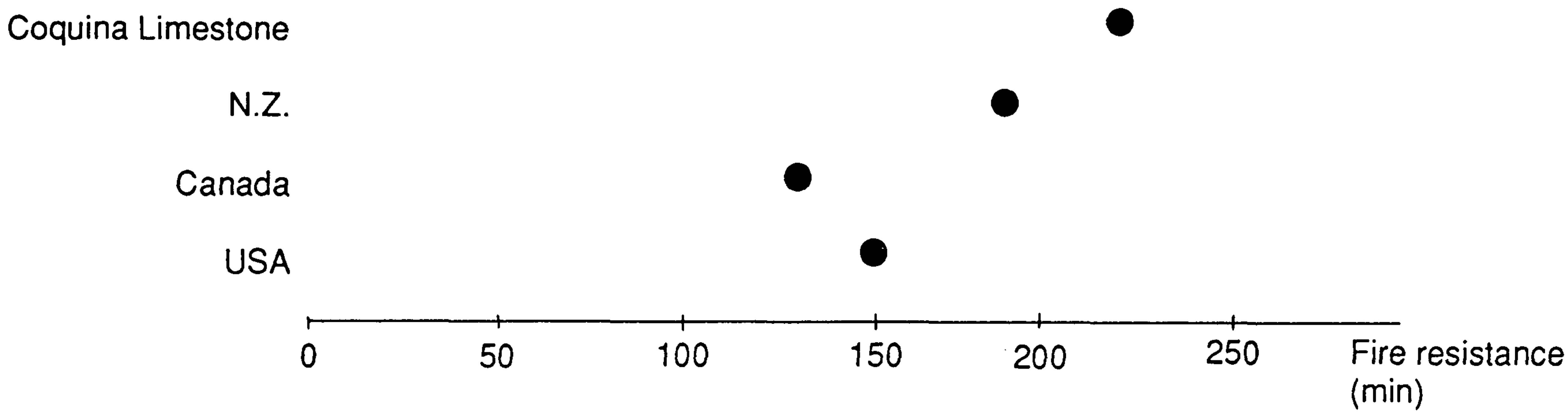


Figure 54: Fire Resistance for 130 mm Thick Reinforced Concrete Non-loadbearing Limestone Aggregate Walls

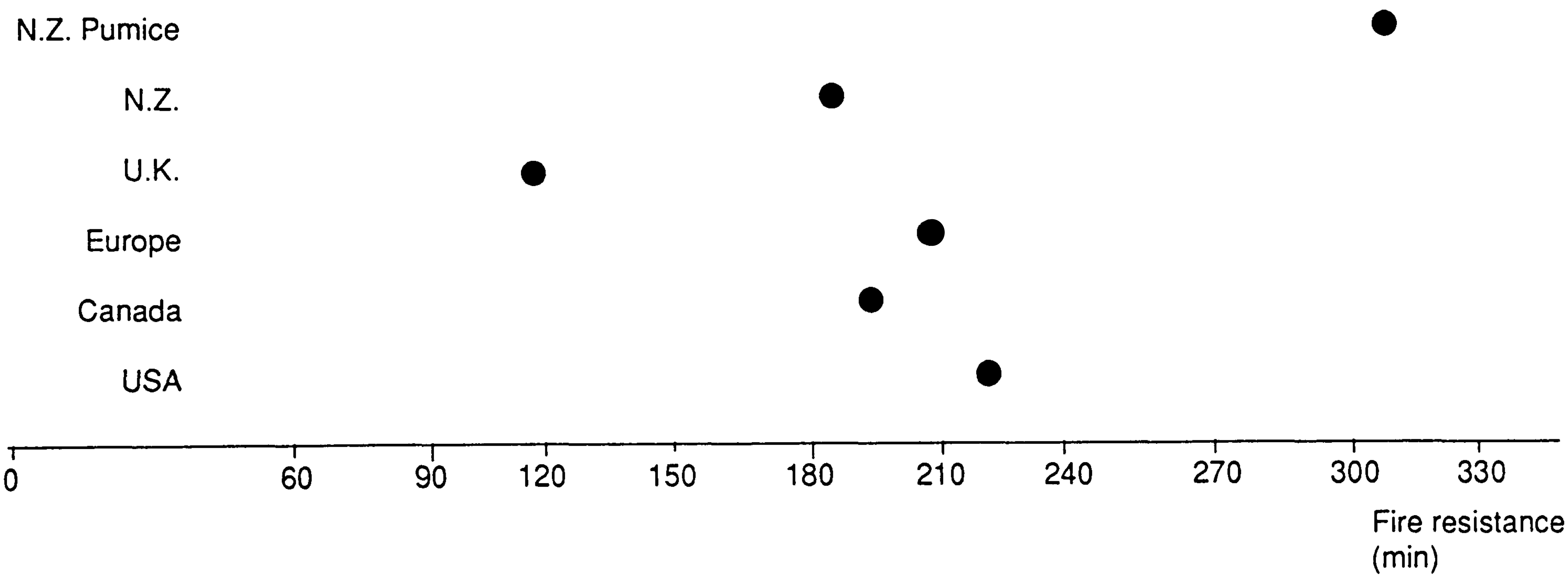


Figure 55: Fire Resistance for 130 mm Thick Reinforced Concrete Non-loadbearing Pumice / Lightweight Aggregate Walls

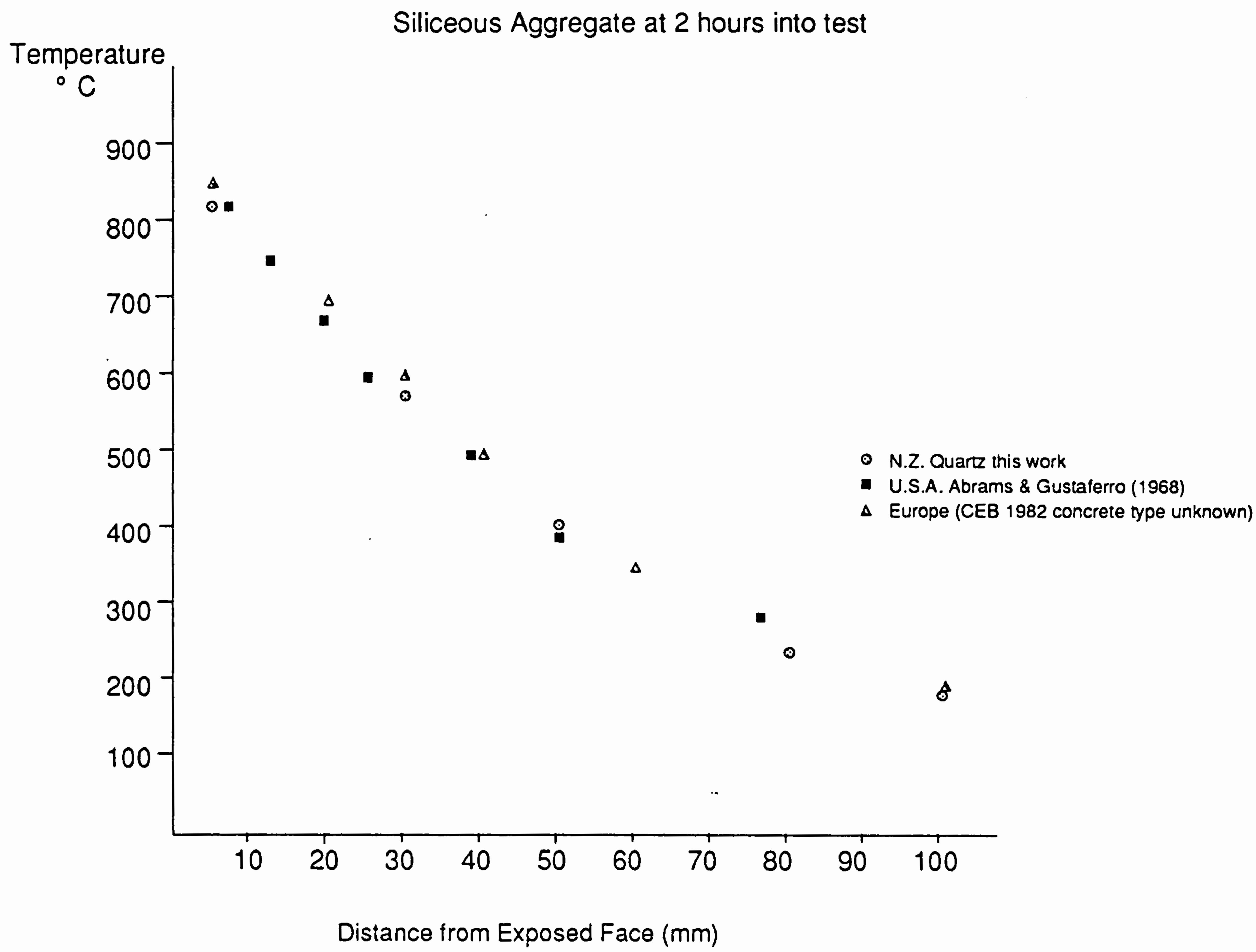


Figure 56: Temperatures Within Concrete During Fire Tests

Fig 57: Fire-exposed Face of Pumice Concrete Showing Surface Spalling

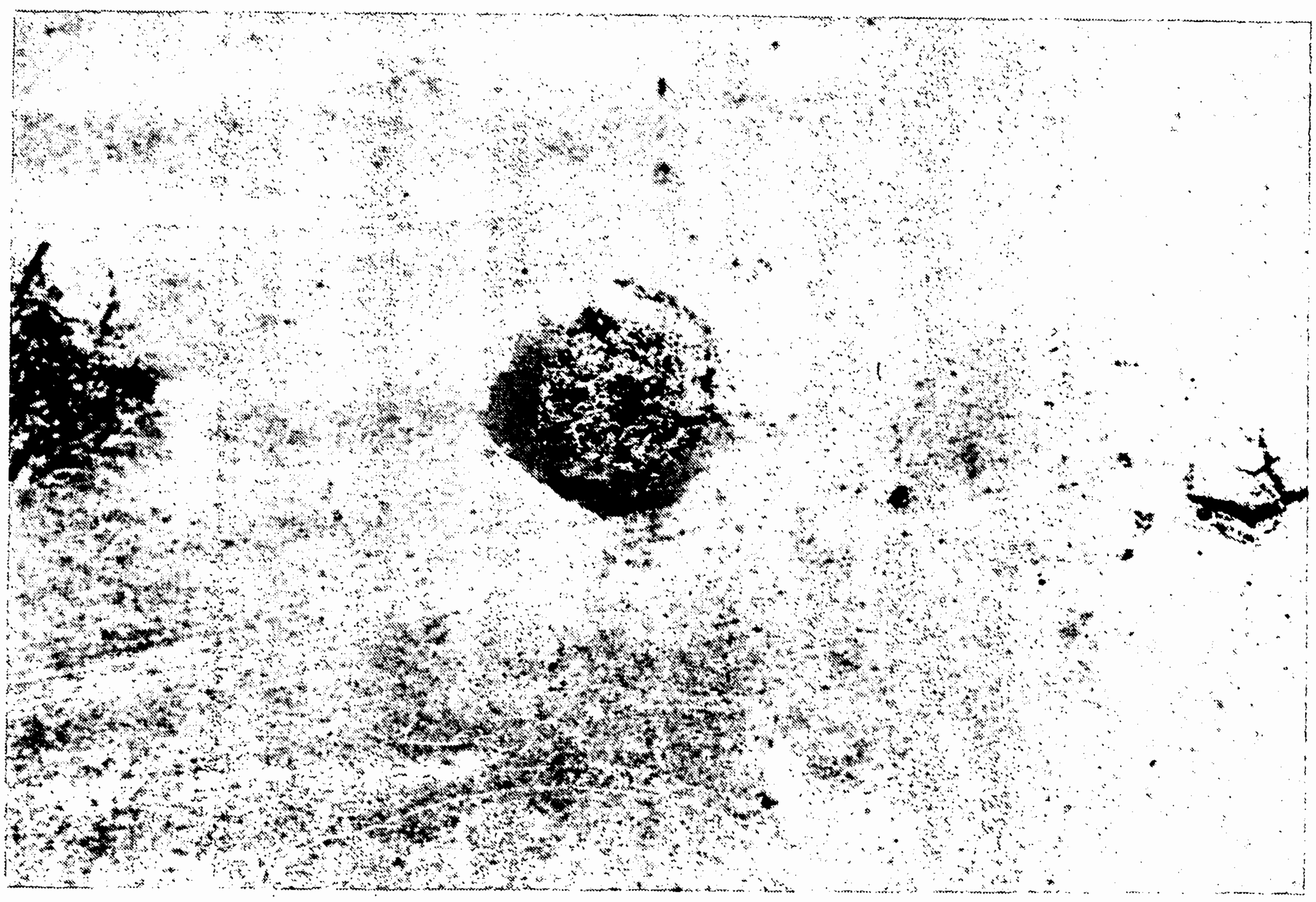


Fig 58: Detail of Surface Spalling of Pumice Concrete

Fig 59: Fire-exposed Face of Limestone Concrete

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