LOW TEMPERATURE TRANSVERSE CRACKS IN ASPHALT PAVEMENTS IN INTERIOR ALASKA

Final Report

by

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CONTENTS

ABSTRACT

The formation and dynamics of low temperature transverse cracks in asphalt pavement have been investigated. Caliper measurements of crack movements showed that the average movement was 14.21 mm with a maximum movement of 27.86 mm for a one year period. A lag in crack response to air temperatures suggests that the movement of the cracks is controlled, not only by the asphalt pavement, but by all the material in the top 2 m of the embankment. The cracks returned to within 1 mm of their previous widths over a period of a year. Average crack spacing was 23.6 m with a general trend of increased crack movement with an increase in average crack spacing. Pavement surface elevations near cracks were about 1.1-1.2 cm higher (relative to the nearby pavement) in February as compared to the following June. The linear thermal expansion coefficients of twelve samples recovered from asphalt pavement ranged from 17.2 to 22.3 x 10^{-6} $(°C)^{-1}$ with an average value near 19.8 x 10^{-6} $(°C)^{-1}$.

Predicted pavement cracking temperatures for three pavements in the Fairbanks area are colder than -50°C. Crack observations in the Deadhorse Airport runway suggest that the position and severity of low temperature transverse asphalt pavement cracks can be controlled by introducing transverse zones of weakness into the pavement and/or underlying embankment.

I. INTRODUCTION

Low temperature transverse cracks are the most common form of asphalt pavement damage observed on Alaskan highways and airport runways, taxiways, etc. These cracks, which constitute a pavement failure, cause the annoying cyclic bumps so familiar to the Alaskan driver. Transverse cracks are defined, for the purposes of this report, as those cracks roughly perpendicular to the pavement direction which extend from one side of the pavement to the other. These cracks often extend to the shoulder of the road and sometimes continue across ditches, bike paths, frontage roads and all lanes of divided roadways. The cracks may transect concrete curbing, sometimes damaging it. Figures 1 and 2 illustrate transverse pavement cracks on the Parks Highway.

Although the formation of transverse thermal cracks has been ascribed to a variety of physical processes, those occurring in Interior Alaska are thought to be caused by thermal contraction of the asphalt pavement, roadbed and possibly the underlying soils or permafrost. However, very little is known regarding the material properties and physical mechanisms governing the formation, growth and dynamics of these cracks.

The thermal pavement cracking problem has been studied in west Texas by Carpenter and Lytton (1977). They suggest that freeze-thaw cycling is a major factor in crack formation and development, and that the cracks are a direct result of volume contraction of the base course on freezing. This mechanism cannot be assumed to be applicable to the Alaskan situation because of different soil conditions (especially clay content), more extreme environmental conditions and the fact that most of the roads in Interior Alaska are underlain by permafrost.

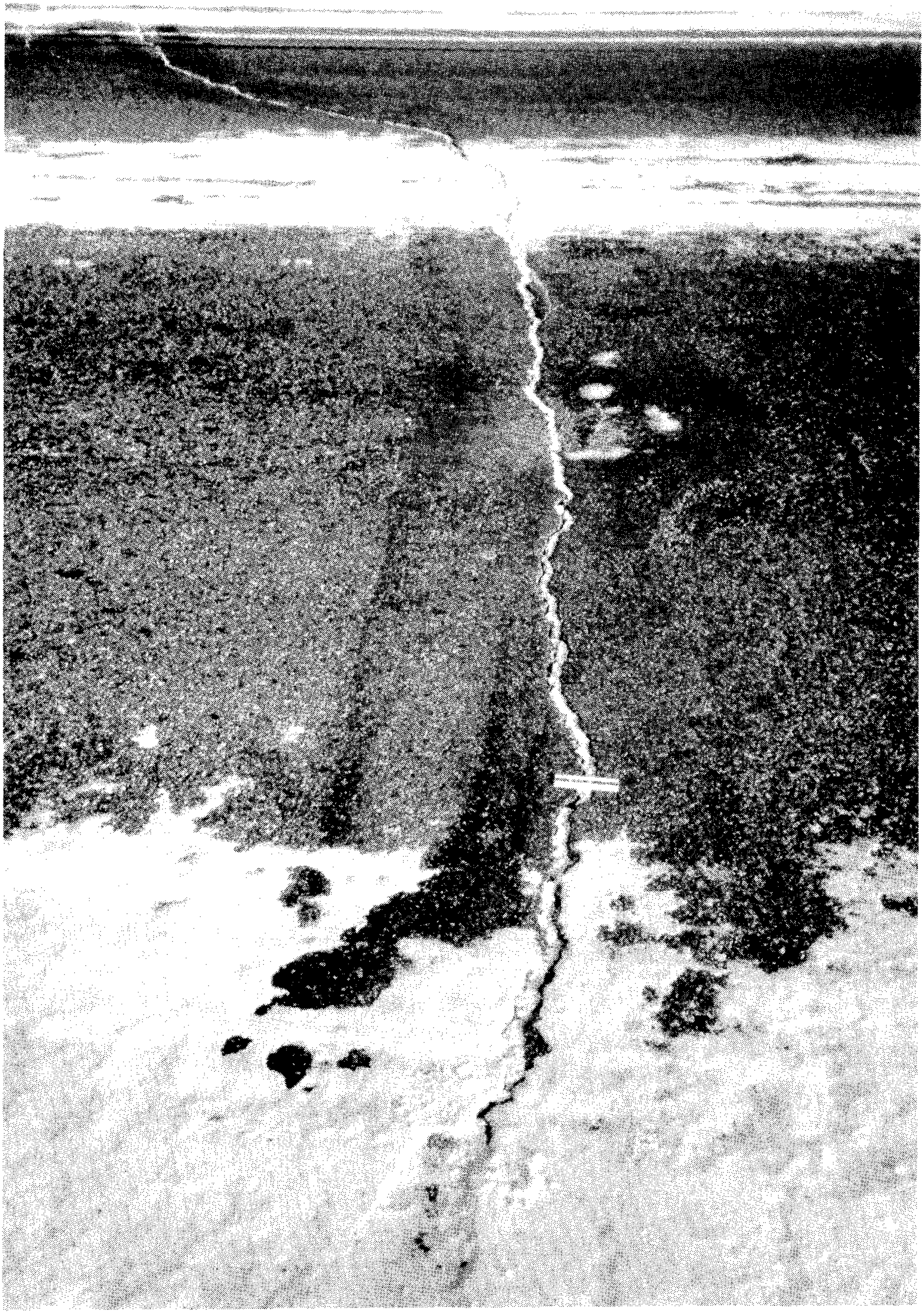


Figure 1: Transverse pavement crack on the Parks Highway (crack number 12, Table 1) during winter. The scale left of center is ≈ 17 cm in length.

Figure 2: Transverse pavement crack on the Parks Highway during summer.

McHattie et al. (1980) have surveyed major transverse cracks in roads near Fairbanks. They suggest that cracks occurring in the Fairbanks area are due to the thermal contraction of the entire pavement structure and are related primarily to the cold climate. Their measurements also suggest that crack movements (opening and closing of the cracks) correlate with air temperatures in a general way. They assumed, based on field observations, that the cracks are controlled principally by the thermal response of the soils rather than the asphalt pavement layer.

The rough ride associated with the presence of transverse thermal cracks is, perhaps, the least serious of the problems associated with them. Cracks can act as a conduit for surface water to enter the road bed as shown in Figure 3. On hills, water has been observed entering and running out of the cracks (Figure 4). The strength of the road bed can be reduced by the presence of the water, and the area around the crack may become a weak point. Figure 5 shows a general depression of the pavement surface and multiple cracks which may have been formed by this process. Note the pavement break and shoulder erosion on the near side of the crack. Localized potholes and broken pavement can also be associated with cracks as shown in Figure 6. During the freezing seasons, the wetted road bed may be subject to frost heaving which elevates the pavement surface adjacent to the crack (Section VI). On thawing, there could be a loss of pavement support, due to thaw consolidation effects, leading to a depression of the pavement surface adjacent to the crack.

The widespread occurrence of major cracks, their presence in other structures such as runways, parking aprons, parking lots, gravel roads etc. and in permafrost terrain (contraction cracks, polygonal ground) suggests that reasonable design modifications may not prevent this type

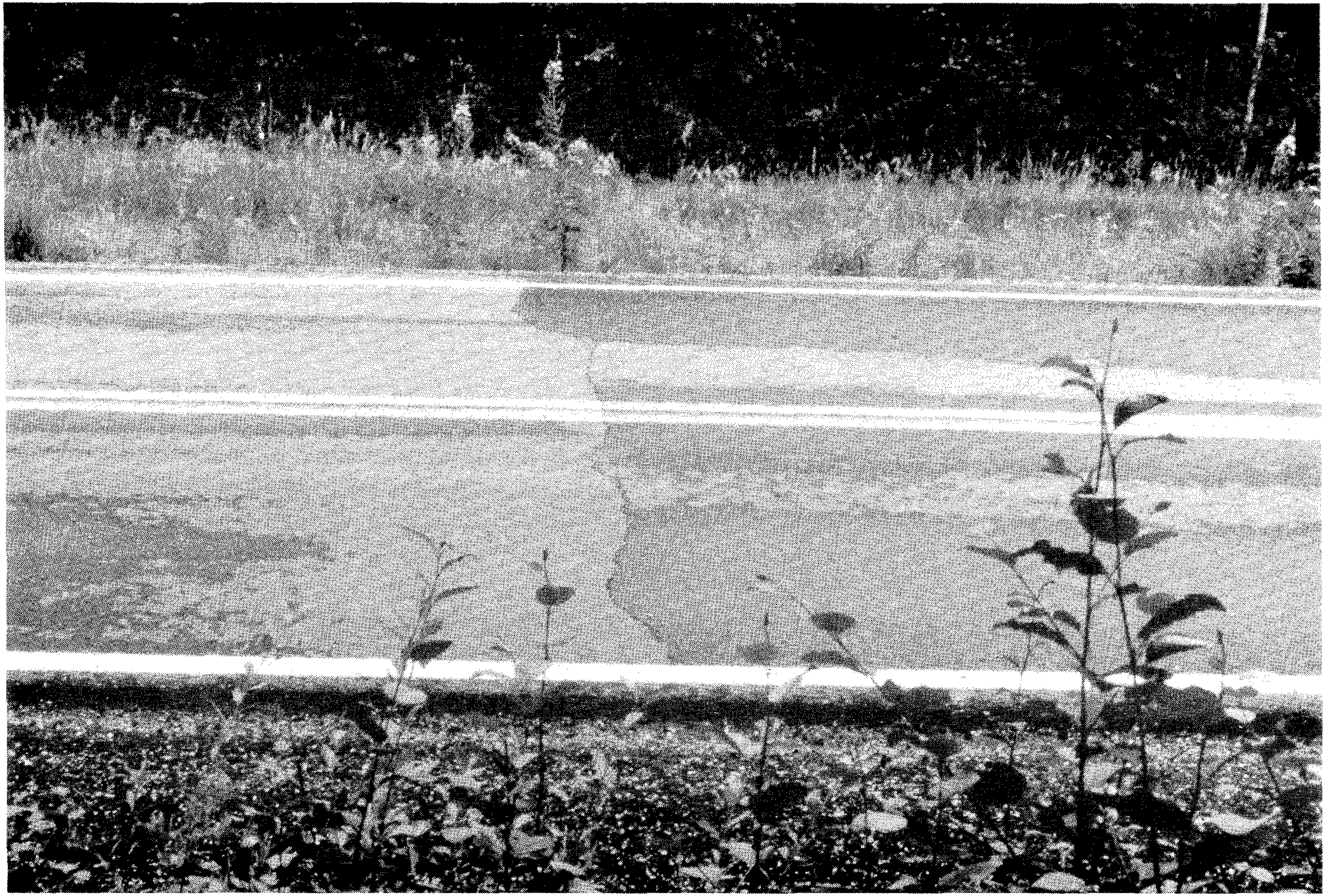


Figure 3: Transverse pavement crack after a shower showing the wet pavement on the uphill (right side) of the crack and the dry pavement on the downhill side.

Figure 4: View looking uphill on the Elliott Highway showing water coming out of a transverse crack and the running into another crack.

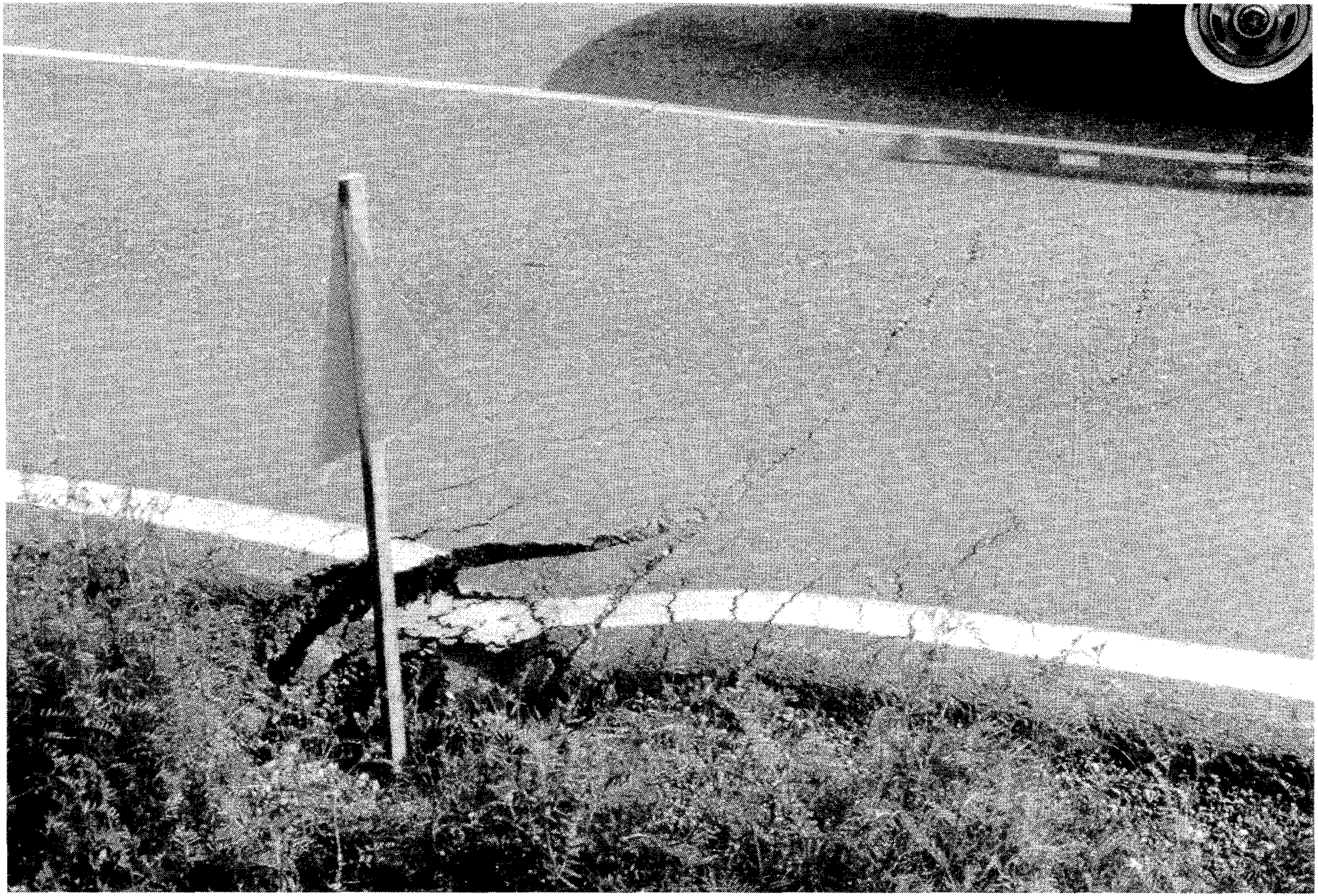


Figure 5: Cracks and associated damage to the edge of the pavement.

Figure 6: Pothole beginning to form in a crack.

of pavement failure. However, potential solutions to the crack problem, mitigative methods, and crack maintenance procedures cannot be evaluated because so little is known about the physical nature of the cracking problem.

The primary objective of this research program has been to investigate the formation, growth and dynamics of cracks in an effort to develop an understanding of their physical nature and of the physical mechanisms that govern their dynamics. This report focuses on crack characteristics, their seasonal variation, vertical pavement movements, and laboratory measurements of the linear thermal expansion coefficients of asphalt pavement samples. Supporting measurements included air and pavement surface temperatures and a large number of visual observations and inspections of cracks. Physical and mathematical models of crack formation and dynamics have also been investigated.

II. CALIPER MEASUREMENTS OF CRACK MOVEMENTS

Visual observations suggest that transverse pavement cracks open in the winter and close in the summer. McHattie et al. (1980) measured crack movements over a three year period in roads in the Fairbanks area. They determined the range of seasonal movements and correlated the crack movements with air temperature in a general way. Six of the cracks studied by McHattie et al. (1980) and a number of additional sites in the Fairbanks area were instrumented to obtain measurements on the opening and closing of transverse pavement cracks in a variety of settings. These included primary and secondary roads in both permafrost and non-permafrost areas. Measurements were made at these sites for a period of about a year. These included short-span caliper measurements across cracks, long-span extensometer measurements over both cracked and uncracked pavement, and measurements of the air and pavement surface temperatures. This section summarizes the results of the caliper measurements of crack widths.

The heads of masonry nails (1/4" x 1 1/2") were drilled with 1/8" diameter holes, 1/8" deep and the nails were then pounded into the pavement on each side of a crack separated by about 5-25 cm. A vernier caliper (0.001"), fitted with centerline gauges, was used to measure the distance between the holes in the nail heads as shown in Figure 7.

Air temperatures were measured with a thermistor thermometer to a precision of about ± 1°C. This thermometer was also used occasionally to measure the temperature ≈ 15 cm deep in the cracks. A radiation thermometer was used to measure the pavement surface temperature at selected sites to an accuracy of a few degrees Celsius (see Section VIII D for these measurements).

Table 1 gives some of the pertinent information for the sites where the crack measurements were made.

The crack movement data vs. time have been graphed and are presented with the air temperature vs. time measurements for each site in Appendix A. These data show that each site appears to have some peculiarities, however, they appear to follow a general trend. Figure 8 illustrates the crack movement vs. time for the Alder Creek site #2 with the associated air temperature from the University Experiment Station (10 km distant).

The measurements show conclusively that the crack movements are correlated with air temperatures, a fact suggested by McHattie et al. (1980) based on their less-frequent measurements. With cold air temperatures during winter, cracks open to their maximum widths and, with warm summer temperatures, cracks close to their minimum widths.

TABLE 1: Pertinent data for the transverse cracks investigated in the Fairbanks area.

Site Number	Location, Description	Nail Placement	Setting	Estimated Pavement Age	Maximum Crack Movement	Average Crack Spacing
1	Alder Creek Parks Hwy Mile 350	Nails on white line, southbound lane	Deep (≈ 10 m) gravel over Alder Creek, permafrost	8 years	1.529 cm	33.1 m
2	Alder Creek Parks Hwy. Mile 350	Nails 6' from white line, southbound lane	"	8 years	1.585 cm	33.1 m
3	University Ave. 30 m south of Chena River bridge	Nails 6' from concrete curb, northbound lane	Flood plain permafrost (?)	18 years	0.934 cm	10.3 m
4	Farmer's Loop Rd. # 1, 60 m north of College Rd. intersection	Nails 6' from white line, southbound lane	Flood plain, permafrost	7 years	1.303 cm	20.0 m
5	Farmer's Loop Rd. Mile 6	North 6' from white line, eastbound lane	Permafrost	18 years	0.585 cm	36.6 m
6	Farmer's Loop Rd. Mile 3	Nails 6' from white line, eastbound lane	Permafrost	18 years	0.874 cm	17.3 m

TABLE 1 (cont'd)

Site Number	Location, Description	Nail Placement	Setting	Estimated Pavement Age	Maximum Crack Movement	Average Crack Spacing
7	Farmer's Loop Rd. Mile 2.5	Nails 6' from white line, eastbound lane	Permafrost	18 years	0.665 cm	37.2 m
8	Trainer Gate Rd. and F Street	Nails 1' from white line, eastbound lane	Floodplain, permafrost	21 years	0.510 cm	17.3 m
9	Trainer Gate Rd. and D Street	Nails 21" from white line, westbound land	"	21 years	0.645 cm	16.0 m
10	Miller Hill Rd. West of intersection with Jankovich Rd.	Nails in northbound lane, uphill site	Shaded south-facing slope, No permafrost	≈ 20 years	0.783 cm	14.6 m
11	Miller Hill Rd. West of intersection with Jankovich Rd.	Nails in northbound lane, downhill site	"	≈ 20 years	0.851 cm	14.6 m
12-1	Parks Highway Approx. 500 m west of Geist Rd. intersection	Nails 1.85 m from white line, eastbound lane	Flood plain, permafrost	10 years	2.304 cm	33.6 m
12-2	"	Nails on white line, eastbound lane	"	"	2.522 cm	"

TABLE I (cont'd)

Site Number	Location, Description	Nail Placement	Setting	Estimated Pavement Age	Maximum Crack Movement	Average Crack Spacing
12-3	"	Nails 1.71 m from white line. Approx. center of east-bound lane	"	"	2.786 cm	"
12-4	"	Nails on center line	"	"	2.527 cm	"
12-5	"	Nails 1.60 m from white line, west bound lane	"	"	2.758 cm	"
12-6	"	Nails on white line, west bound lane	"	"	1.715 cm	"
12-7	"	Nails 0.52 m from white line west bound lane	"	"	0.706 cm	"

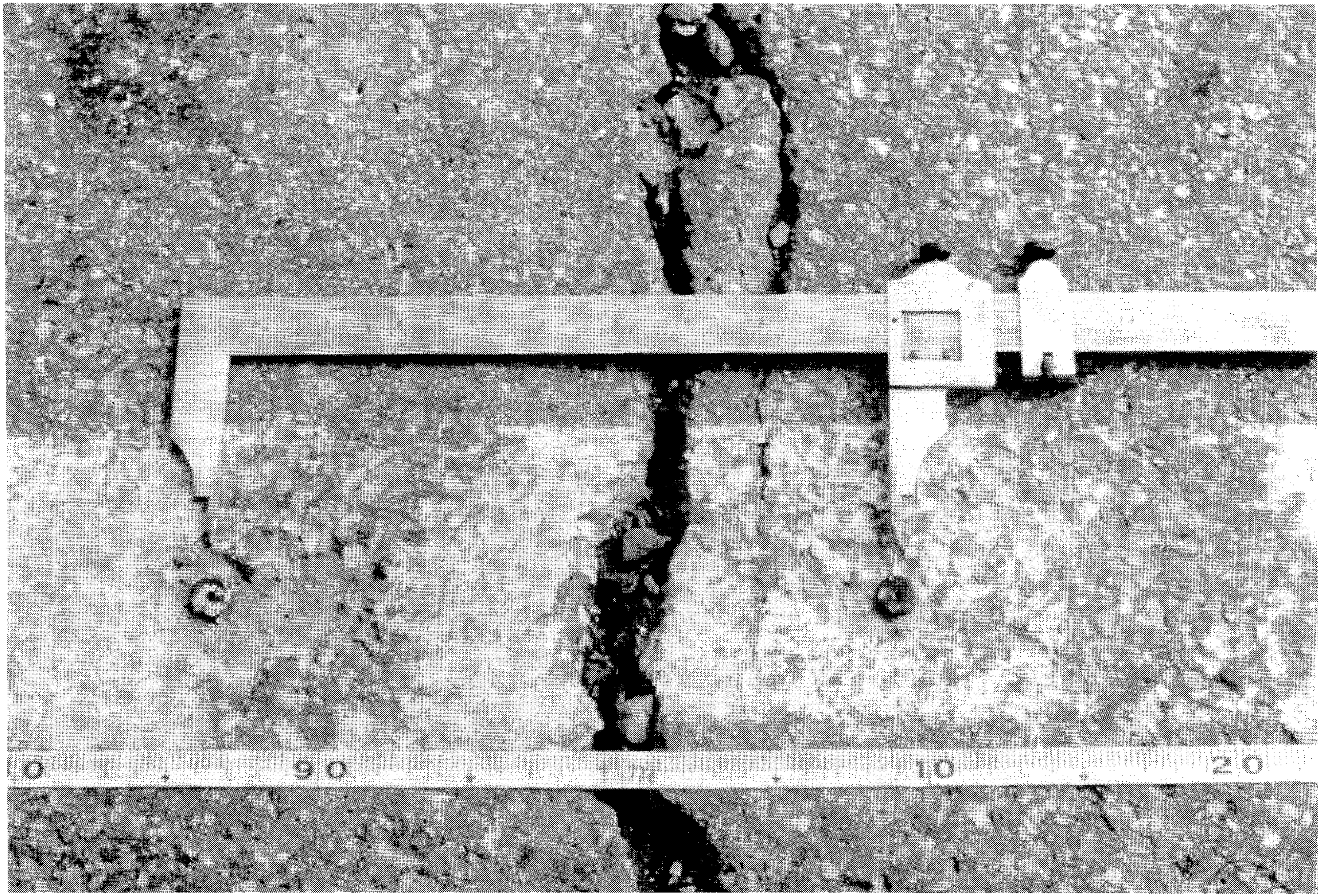


Figure 7: Vernier calipers and modified nails used to measure crack movements.

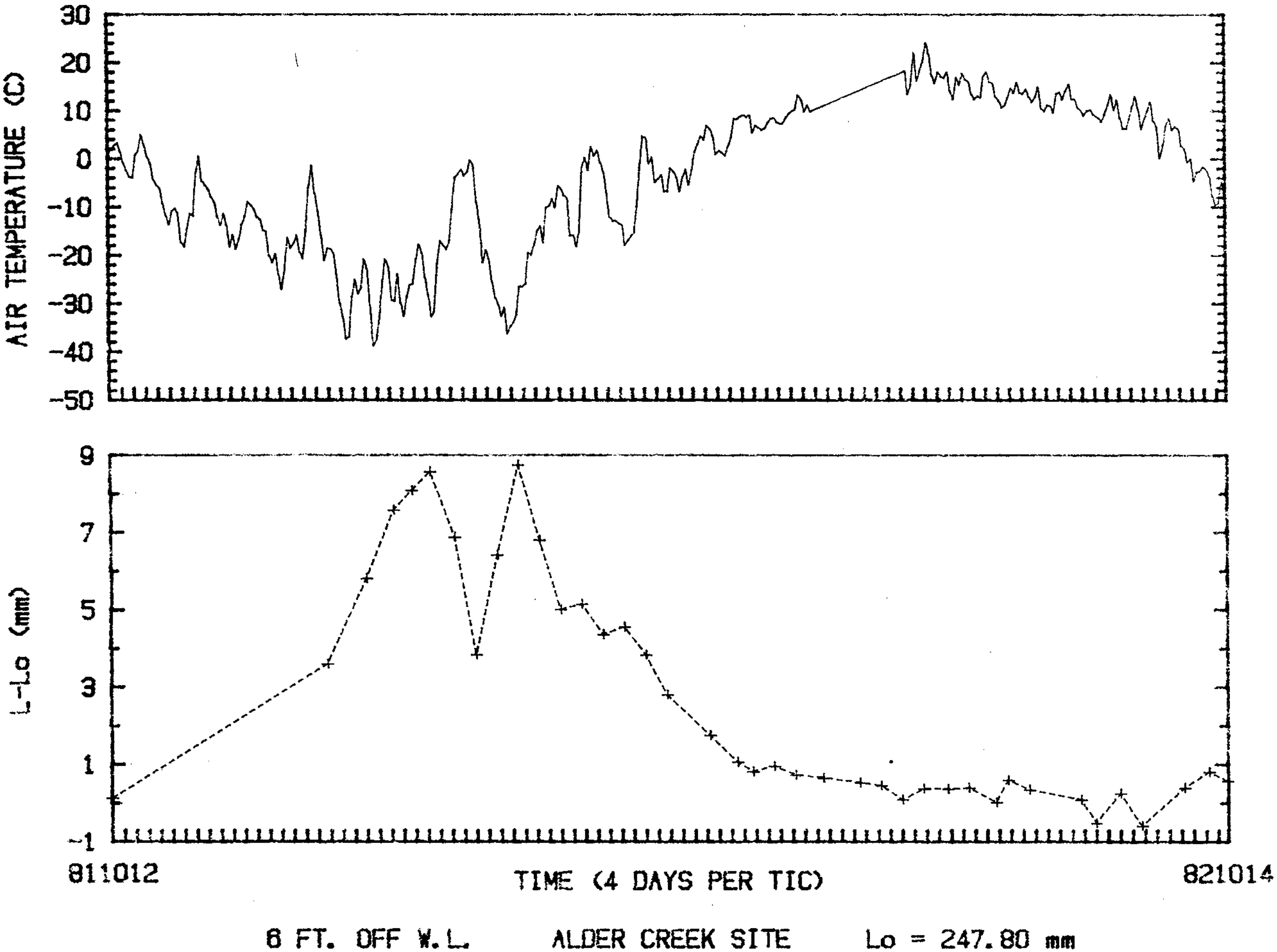


Figure 8: Crack movement vs. time for the Alder Creek site #2 with the associated air temperatures for the University Experiment Station.

McHattie et al. (1980) found a maximum crack movement of 22.9 mm at their site 9 (our site 2) on the Parks Highway in 1979. This crack, which was in a deep fill over Alder Creek opened 15.8 mm in 1982. The maximum crack movement measured in these studies was 27.86 mm at site 12-3 on the Parks Highway. The average maximum crack movement was 14.21 mm with a standard deviation of 8.22 mm.

An examination of the crack movement data (Fig. 8 and Appendix A) suggests that there is a lag in crack movement with respect to changes in air temperature. The amount of lag cannot be determined accurately with these data because it appears that the lag is shorter than the frequency of observations. The frequency of the measurements ranged from six days to periods longer than two weeks and the lag appears to be on the order of a day to a week. The amount of lag is an important point since, given the lag, a rough estimate of the depth that controls the crack movement can be made as follows. The effects of a step change in surface temperature can be obtained, under certain conditions, from the time constant (Osterkamp, 1984)

$$t_o = X^2/4K \qquad (2.1)$$

where X is the thickness of the layer affected by the temperature change and K is its thermal diffusity. For example, for dry coarse-grained road bed materials with $K \approx 70\ m^2\text{-}y^{-1}$, values for t_o for various X are given in Table 2.

TABLE 2: Values for the time constant, t_o, for various depths, X, with $K = 70\ m^2\text{-}y^{-1}$.

X(m)	0.5	1	1.5	2.0	3.0
t_o(days)	0.3	1.3	2.9	5.2	11.7

Table 2 suggests that a lag of a day to a week would be associated with the temperature changes at depths up to about 2 m. This suggests that movement of the cracks is controlled, not only by the asphalt pavement, but by all the material in the top 2 m of the road.

Changes in air temperatures between warm and cold periods were fairly rapid during the winter and produced rapid and large changes in the opening and closing of the cracks. However, gradual air temperature variations during spring and summer of about the same magnitude did not generally produce similar changes in the cracks.

By the beginning of spring, cracks were generally closing steadily with occasional minor increases in width associated with cooler weather periods. Crack widths were generally constant or decreasing slowly over the summer. Air temperature variations over the summer produced only small variations in crack widths. These small changes in crack width over the summer may reflect smaller variations in air temperatures but may also be a result of the crack having closed or of debris in the crack that may have retarded its movement.

At several sites, where data on crack movements were obtained for a full year, there was 1 mm or less difference in crack widths between October of 1981 and 1982.

The simplest model for crack movement based on contraction suggests that there should be a correlation between the crack movement and crack spacing since the crack movement, ΔL, should, in principle, be related to the spacing by

$$\Delta L = \alpha \cdot \Delta T \cdot L \qquad (2.2)$$

where α is the linear thermal expansion coefficient, ΔT is the change in temperature of the road and L is the spacing between cracks. Figure 9 is a graph of ΔL vs. the average crack spacing (on either side of the measured crack). Figure 9 suggests increased movement with an increase in average crack spacing. This result will be discussed further in Section V.

III. EXTENSOMETER MEASUREMENTS OF PAVEMENT AND CRACK MOVEMENTS

A tape extensometer was used to investigate pavement movements over long spans (up to 11 m). These measurements were carried out at the Parks Highway site 12-2. The reference points (concrete anchor bolts set into the pavement) were on a line crossing site 12-2. Measurements were made from both ends of the line (10 anchor bolts) across the uncracked pavement to the far side of the crack.

The tape extensometer consists of a steel surveyor's tape fitted with a caliper for precise measurements over distances of 20 m. Sensitivity in the laboratory was about ± 0.07 mm. The tape was transported in a heated vehicle and then removed from the vehicle to make the measurements at each site. Under extremely cold air temperatures, the accuracy may be as low as a few mm primarily due to contraction of the tape. Since the tape temperatures are unknown and were probably varying with time it is difficult to obtain reliable corrections for these changes. Consequently, the accuracy of the tape extensometer data at low temperatures is poor.

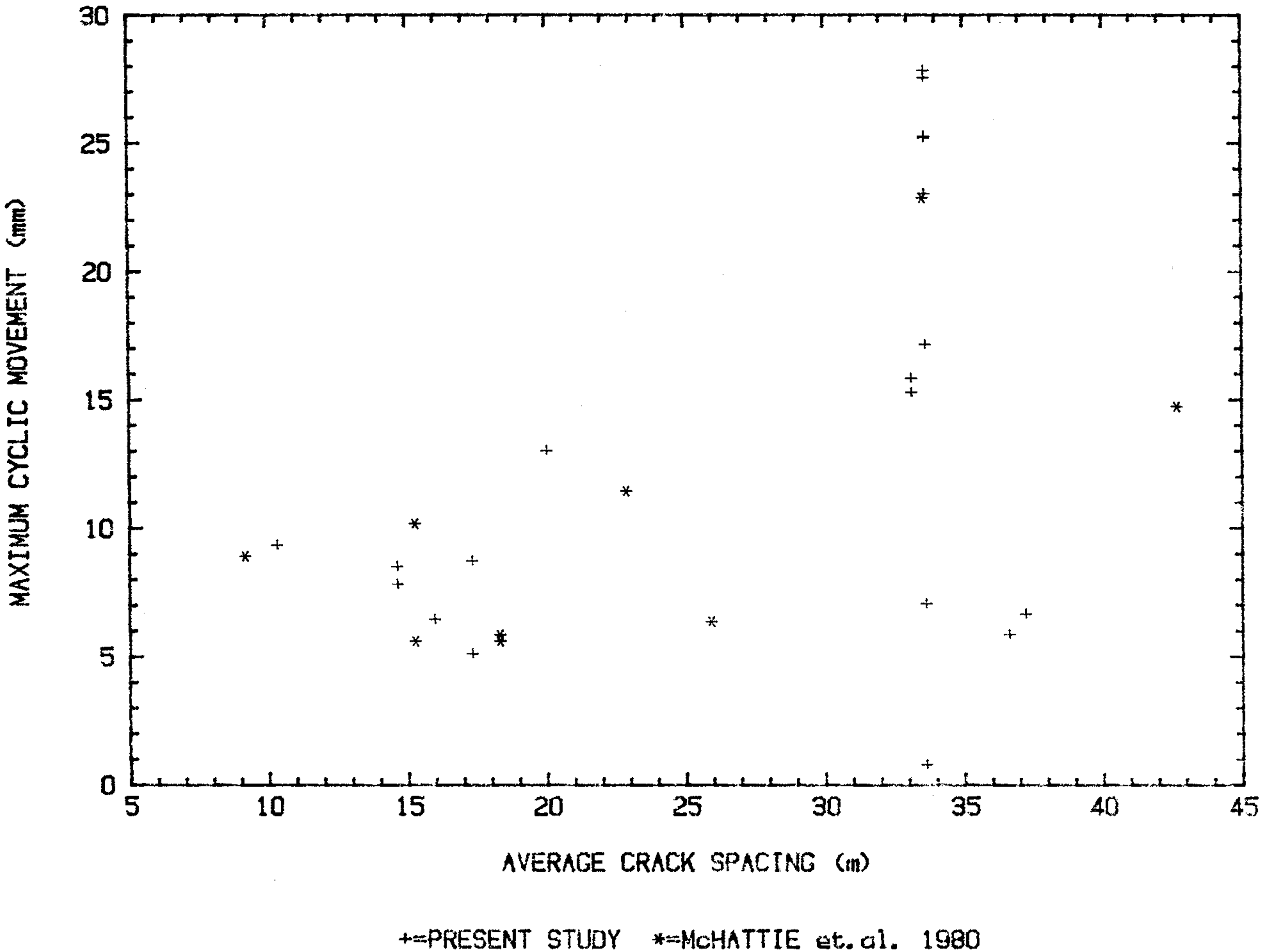


Figure 9: Maximum cyclic movement of the cracks vs. the average crack spacing for the present study and that of McHattie et al. (1980).

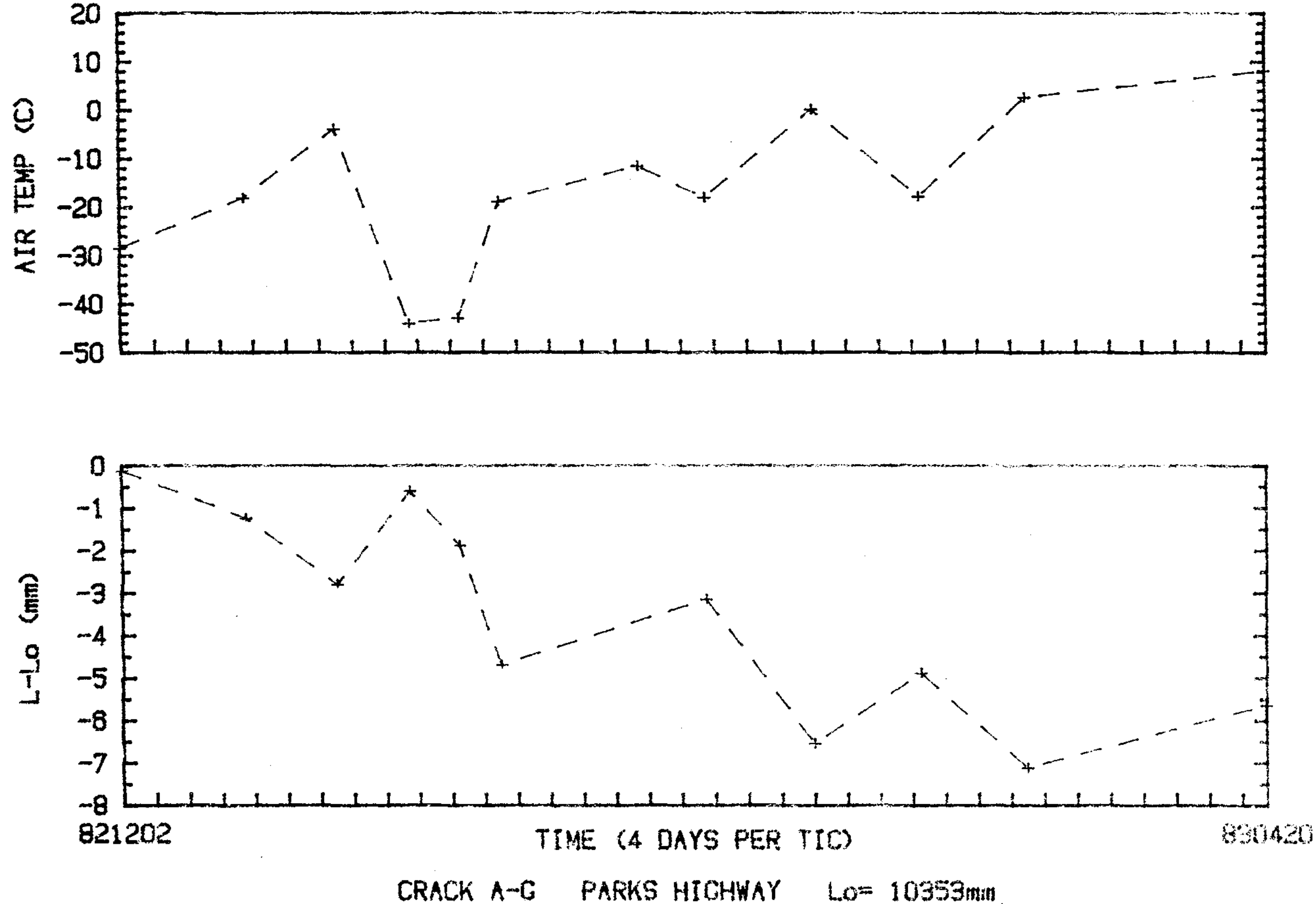


Figure 10: Pavement movement vs. time for span AG (site 12, Table 1) with the associated air temperatures for the University Experiment Station.

An example of the extensometer measurements with time and the associated air temperatures for span AG is shown in Figure 10. The full data in graphical form are given in Appendix B.

An estimate of the thermal expansion coefficient, α, measured with the extensometer can be obtained from Eq. 2.1 in the form

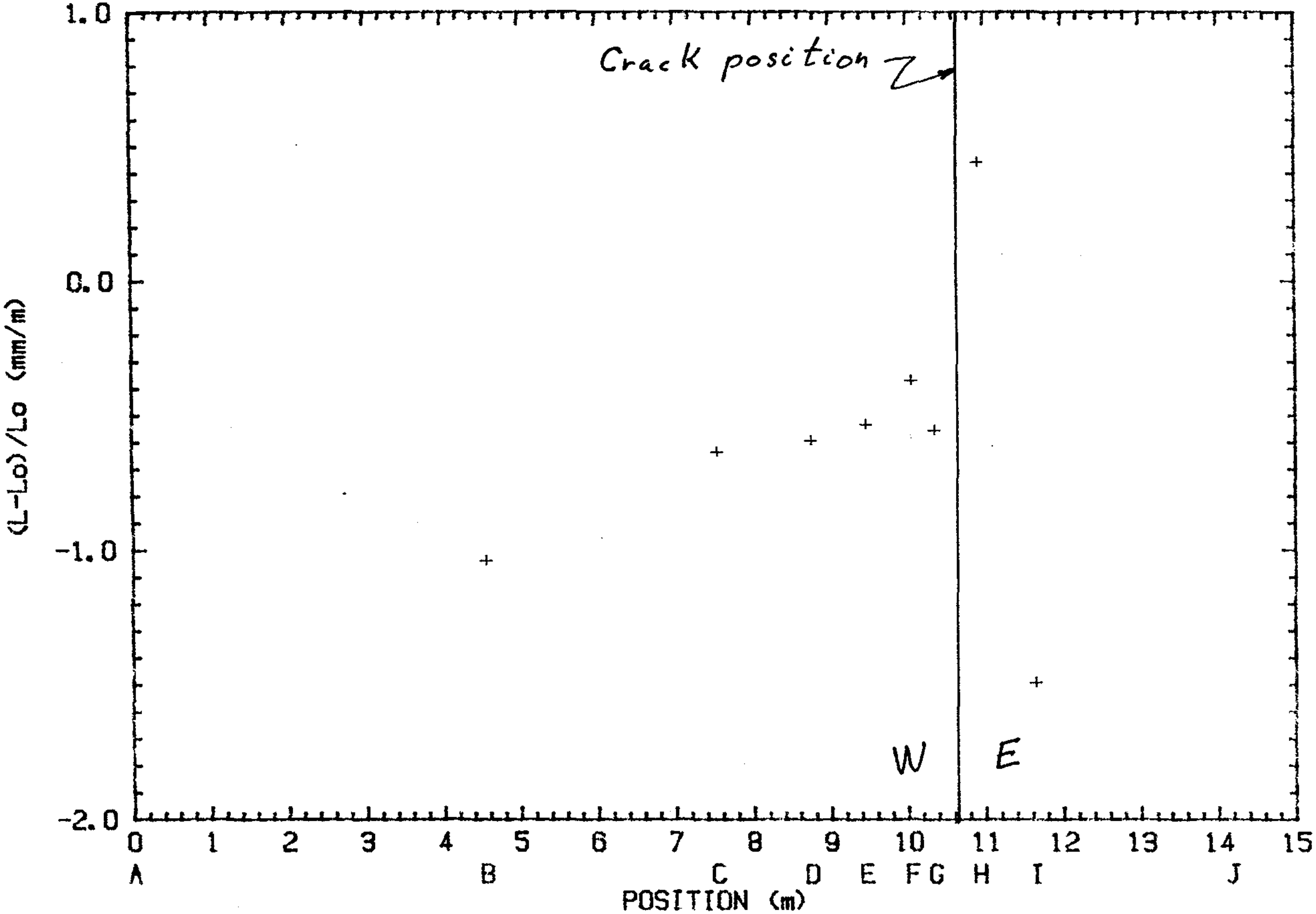
$$\alpha = (1/L_o) \left(\Delta L/\Delta T\right) \tag{3.1}$$

where L_o is the length between the anchor bolts and ΔL is the change in length produced by a change in temperature ΔT. ΔT is not known but can be estimated from the air temperature. For span AG, $\alpha \approx 27 \times 10^{-6}$ ($°C^{-1}$), which is about one-third greater than the average coefficient for asphalt (Sec. VII). For span JH, $\alpha \approx 62 \times 10^{-6}$ ($°C^{-1}$), which is about two and one-half times the average coefficient for asphalt pavement (see Sec. VII). Thus, the agreement is good for span AG and poor for span JH which may be the result of several factors such as a variable stress distribution in the blocks between cracks, effects of the underlying road bed materials etc. In this regard, measurements on the expansion coefficients of the road bed materials would be useful to determine whether or not this last value is characteristic of them. Additional discussion of this point is given in Appendix F.

Equation 3.1 in the form

$$\Delta L/L_o = \alpha \cdot \Delta T \tag{3.2}$$

suggests, with simplifying assumptions, that the strain, $\Delta L/L_o$, should be constant for a given value of ΔT. Figure 11 is a graph of strain vs. position in the uncracked pavement near site 12-2. The strain is nearly constant on the west side and within 3 m of the crack but not at



PARKS HIGHWAY
821112 TO 830420

Figure 11: Strain, $\Delta L/L_0$, as a function of position for the period from December, 1982 to April, 1983 for the Parks Highway crack.

other points. However, it now appears that there were a few hairline cracks in the pavement between the A and G positions. If these hairline cracks were present and opened during the measurements then a more detailed analysis of this point is not warranted.

IV. EXTENSOMETER MEASUREMENTS OF FINE-GRAINED PERMAFROST

It was initially proposed to measure the thermal expansion coefficients of fine-grained permafrost samples in the laboratory. However, by mutual agreement these laboratory measurements were replaced by a field experiment. In this experiment, the tape extensometer described in Sec. III was used to measure the thermal contraction and expansion of fine-grained (silt) permafrost near the junction of Sheep Creek Road and Miller Hill Road near Fairbanks. Two perpendicular spans (≈ 17 m in length) with a common corner were set up with one span in the east-west direction and the other in the north-south direction. One set of measurements were made between pipes set 0.8 m into the ground. Another set of measurements were made between pipes set 3.45 m into the ground with a casing set over these pipes to a depth of 0.8 m to shield them from movements in the top meter of ground. The data are given in Appendix C. The data from the deep pipes extends from December, 1982 to July, 1983. However, measurements at the shallow sites had to be suspended in April, 1983 because of deformation of the soil around the pipes due to thawing after this time.

Movements of both the deep and shallow pipes were contrary to expectations. For example, during the cold period of the first half of January, 1983, all four spans showed an expansion relative to measurements made at a time of warmer temperatures in early December, 1982. Temperatures at the 0.65 m depth cooled from about -1 to -3 to -6°C on December 17th, Janury 6th, and January 17th, respectively. Therefore, a phase lag does not seem likely to be responsible for the deeper pipe movements. Measurements of the deep pipes in July, 1983 showed a substantial contraction relative to the early December, 1982, measurements. The magnitude of the contraction was about 35 mm for the east-west span and < 14 mm for the north-south span. Ground temperature measurements at this site suggest that the temperature variation at the 1-3 m depth did not exceed 7°C. In December, 1982 and July, 1983, the temperatures at these depths probably differed by only 1 or 2°C. According to Eq. 3.1, a very large negative expansion coefficient would be required to fit these data. The expansion coefficient of Fairbanks silt is not known and, while not likely, could be negative at this site. Other possibilities could involve moisture transport in the permafrost, distortion of the anchor pipes under the influence of a ground temperature gradient and contraction of the tape and tension spring at cold air temperatures that resulted in an apparent expansion between the anchor points. Corrected expansion coefficients, which allow for the contraction and expansion of the tape in a crude way, were computed assuming a linear expansion coefficient for steel tapes equal to 11.7×10^{-6} °C^{-1}. About half the corrected coefficients were between 4×10^{-6} °C^{-1} and 13×10^{-6} °C^{-1}. A few coefficients were smaller and positive, but about one-fourth of the corrected coefficients were negative. It would be desirable to obtain laboratory measurements of natural samples of Fairbanks silt permafrost to help interpret these findings or to carry out a more detailed field investigation.

V. CRACK SPACING MEASUREMENTS

At each crack site that was investigated in this study, the crack spacing was measured with a surveyors tape over a distance of about 200 m, 100 m or more on each side of the crack. Only transverse cracks that were more or less perpendicular to the road and which transected the road

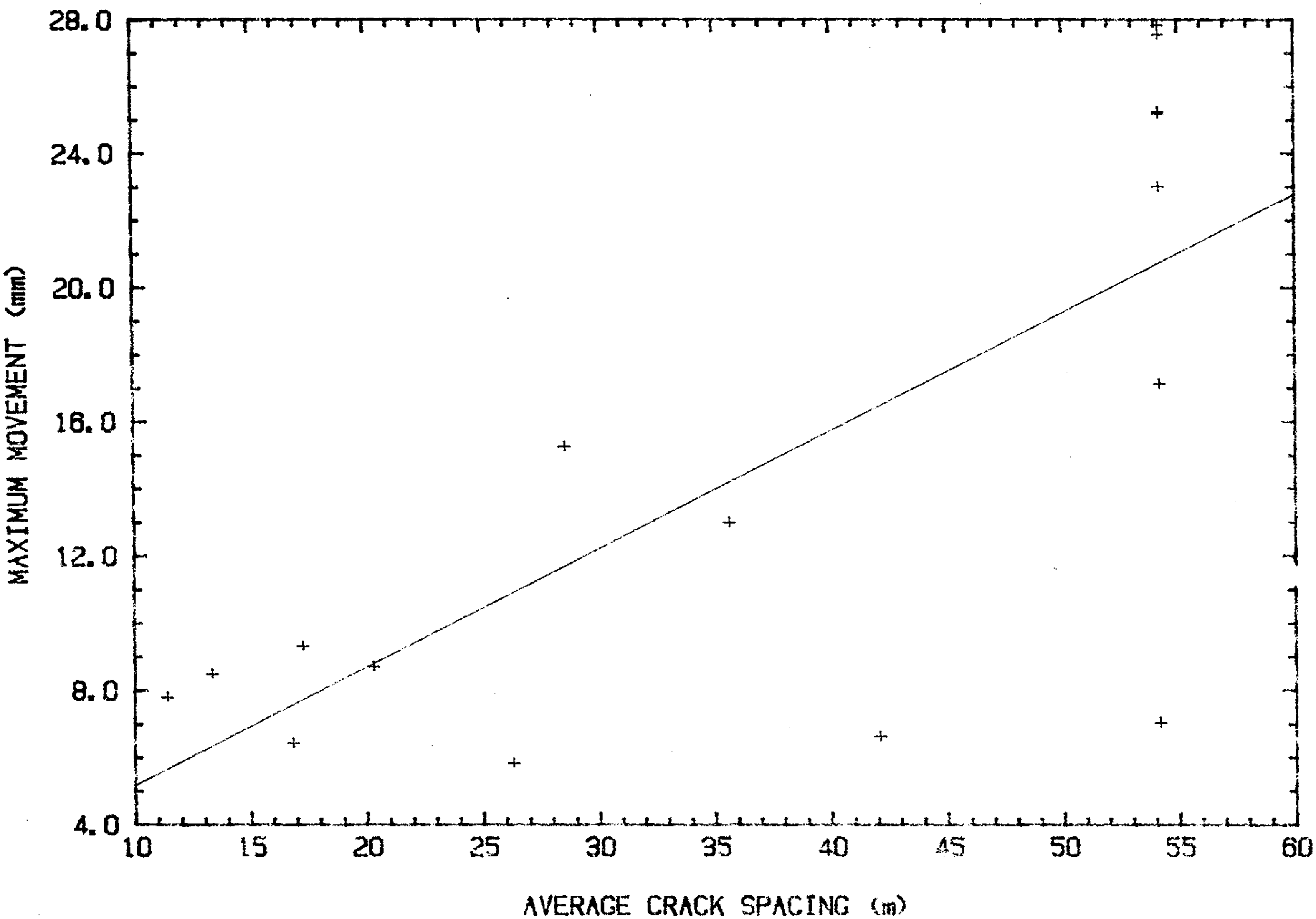


Figure 12: Maximum crack movement vs. the average spacing of the two cracks immediately adjacent to the measured crack. The solid line is a least squares fit to the data.

from side to side were counted. Obviously, a certain amount of judgement enters into the measurements reported in Table 1 (p. 13). The average crack spacing was 23.6 ± 10.3 m. McHattie et al. (1980) found an average crack spacing of 22.3 m.

Figure 12 is a graph of the maximum crack movement with respect to the average crack spacing for the two cracks on either side of the measured crack. Equation 2.2 can be modified under simplifying assumptions to show that

$$\Delta L = \alpha \cdot \Delta T \cdot (L_1 + L_2)/2 \qquad (5.1)$$

where $(L_1 + L_2)/2$ is the average crack spacing for the two cracks on either side of the measured crack. Equation 5.1 shows that there should be a correlation between ΔL and $(L_1 + L_2)/2$ which is illustrated in Figure 12. However, these data are not directly comparable since some of the cracks were measured over different time spans. In addition, the mechanisms controlling crack movement are still uncertain. Therefore, no physical meaning can be attached to the least squares line through the data.

VI. VERTICAL PAVEMENT DISPLACEMENTS NEAR CRACKS

Qualitative observations suggest that the road surface, near major transverse pavement cracks, tends to move up during winter and down during the summer. In some cases, the edges of the cracks were observed to curl up (winter) and to drop down (summer) relative to the adjacent road surface. It is hypothesized that the water that runs into cracks during the thawed period, wets the soil under the pavement and then causes the pavement to heave as it freezes. During the thaw period the wetter soils near the cracks would be weaker tending to allow greater deformation from traffic.

Measurements were made of the elevation of the road surface at two sites on Miller Hill Road from February to June, 1982, to quantify the magnitude and nature of the movement of the road surface near cracks. The position of the road surface was measured relative to the top of an aluminum bar supported on each end by bolts which were reduced to 1/8" diameter at the bottom. The supports were fitted into the drilled heads of concrete nails (Sec. II) which were driven into the pavement. The distance between the holes in the nails was ≈ 1.226 m. The elevation of the road surface relative to these nail heads was measured from the top of the bar, through holes drilled in the bar, with a vernier caliper as shown in Figure 13.

Elevation data for the road surface is shown in Appendix D. Figure 14 shows the pavement surface elevations during February and June, 1982, relative to the nail heads in the pavement on either side of the crack which was located at the 50 cm position. During February, the pavement surface elevation was at about 0.9 to 1.0 cm and, during June, it was at about -0.1 to -0.2 cm which gives a total change in elevation of about 1.1 to 1.2 cm between the winter and summer condition. Figure 14 shows that the position of the support pins changed between February and June suggesting that the supports should have been farther apart. Therefore, the total change in elevation was probably greater than the 1.1 to 1.2 cm noted above. Another feature of the data in Figure 14 is that it shows that the road was relatively smooth during winter with surface elevation variations of < 0.2 cm near the crack while it was relatively rough during summer with variations of about 1.0 cm near the crack.

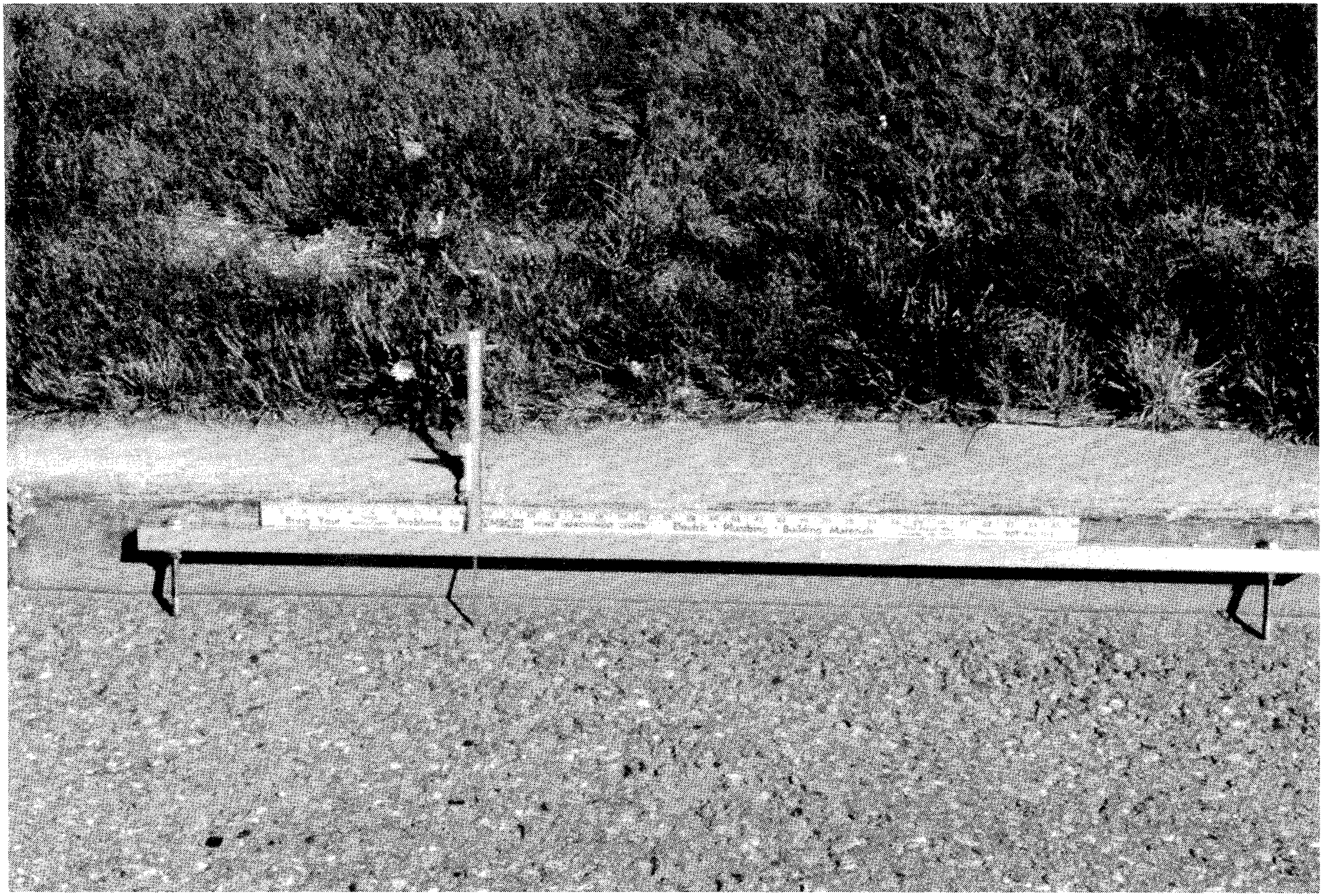


Figure 13: Aluminum bar, support legs and calipers used to measure the vertical pavement displacements.

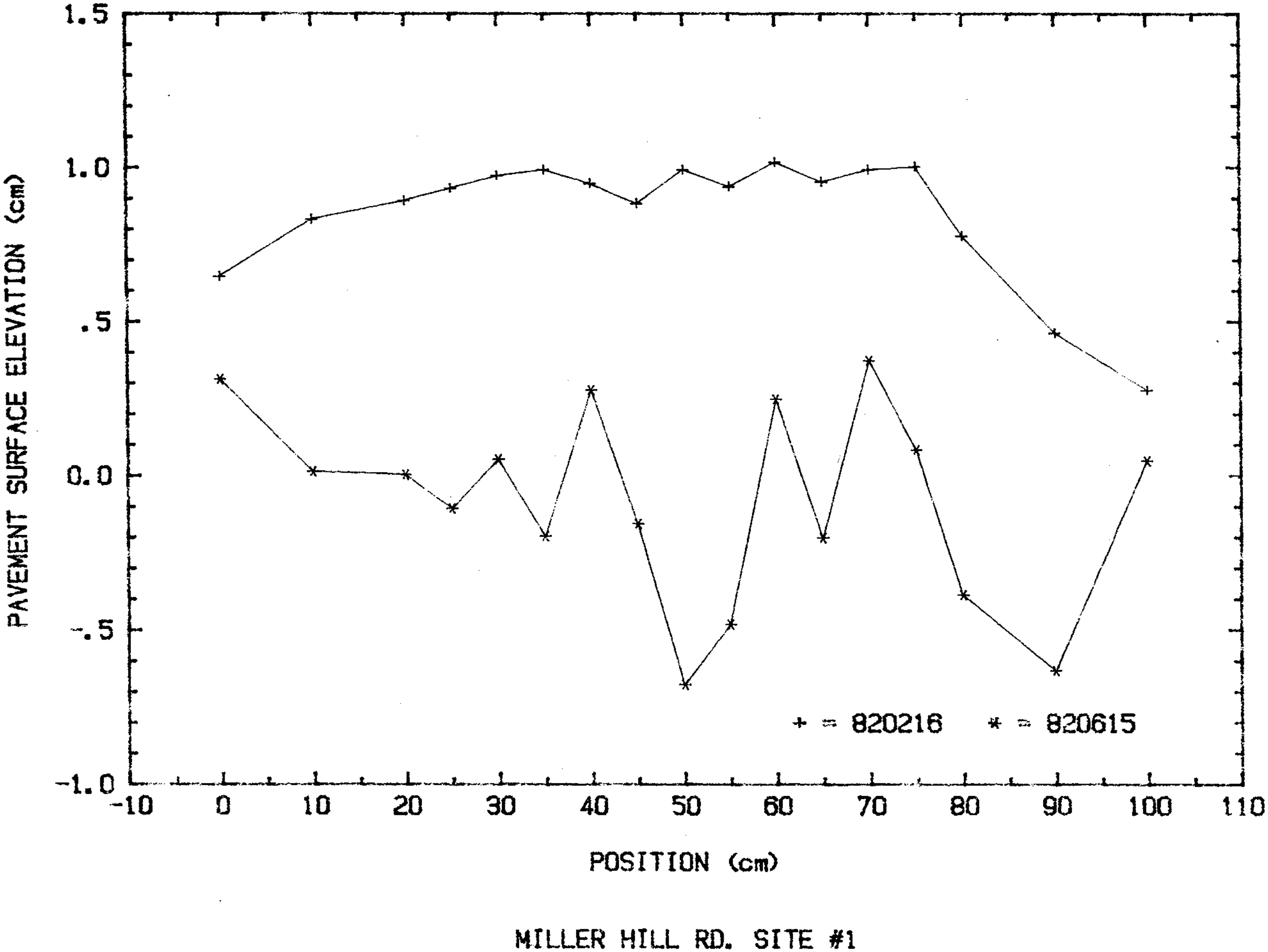


Figure 14: Pavement surface elevation vs. position for the Miller Hill Road site number 1 for February and June, 1982.

VII. LABORATORY MEASUREMENTS OF THE LINEAR THERMAL EXPANSION COEFFICIENTS OF ASPHALT PAVEMENT AT LOW TEMPERATURES

Improved design techniques to minimize transverse cracking in asphalt pavement subjected to low temperatures require information on the linear thermal expansion coefficients of the pavement (Asphalt Institute, 1981). The available data on the expansion coefficients of asphalt pavement mixes has been obtained from laboratory samples (Littlefield, 1967; Jones et al. 1968). This section reports the results of measurements of the linear thermal expansion coefficients of asphalt pavement samples obtained from several pavements near Fairbanks, Alaska.

The samples were cut from the pavement with a diamond core barrel at the locations shown in Table 3. These core samples were returned to the laboratory and cut into rectangular bars parallel and perpendicular to the road direction. The bars were then cut to their final size and the ends squared with a circular diamond rock saw. Thin stainless steel sheets (#304, 0.605 mm thickness) were glued with Loctite Super Bonder 495 to each end of the samples to prevent the dilatometer push rod from indenting them. The sample dimensions were measured at room temperature with vernier calipers. Sample analyses (Table 3) were carried out by the Alaska Department of Transportation and Public Facilities on the pavement material after sampling.

Contraction and expansion of the samples were measured with a linear voltage displacement transformer (LVDT) and a precision push-rod type dilatometer (± 0.0001"). The tripod dilatometer was mounted on fused quartz glass legs as shown in Figure 15. A metal base plate was used to support the tripod and sample, which was placed in a cylinder between the legs of the tripod. This apparatus was placed in a well-stirred low

TABLE 3: Pertinent data for the asphalt pavement samples used in measurements of the thermal expansion coefficients.

Sample Designations	Location, Age	% Passing #10 Sieve	Asphalt[1] %	Specific Gravity	Viscosity[2] 275°F	Viscosity[2] 140°F	Penetration[3] 77°F	Penetration[3] 39.2°F
MH-1,2,3,4	Miller Hill Rd, ≈ 20 years	42	5.9	2.40	167.8	417.7	142	37
A-1,2,3,4	Airport Way ≈ 18 years	44	5.8	2.37	534.5	3882.5	31	14
K-1,2,3,4	Kantishna Drive, 4 years	24	7.9	2.21	161.5	330.3	224	32

1. % by weight of aggregate. 2. Kinematic viscosity @ 275°F in centistokes, absolute viscosity @ 140°F in poises. 3. Asphalt cement penetration in tenths of a mm.

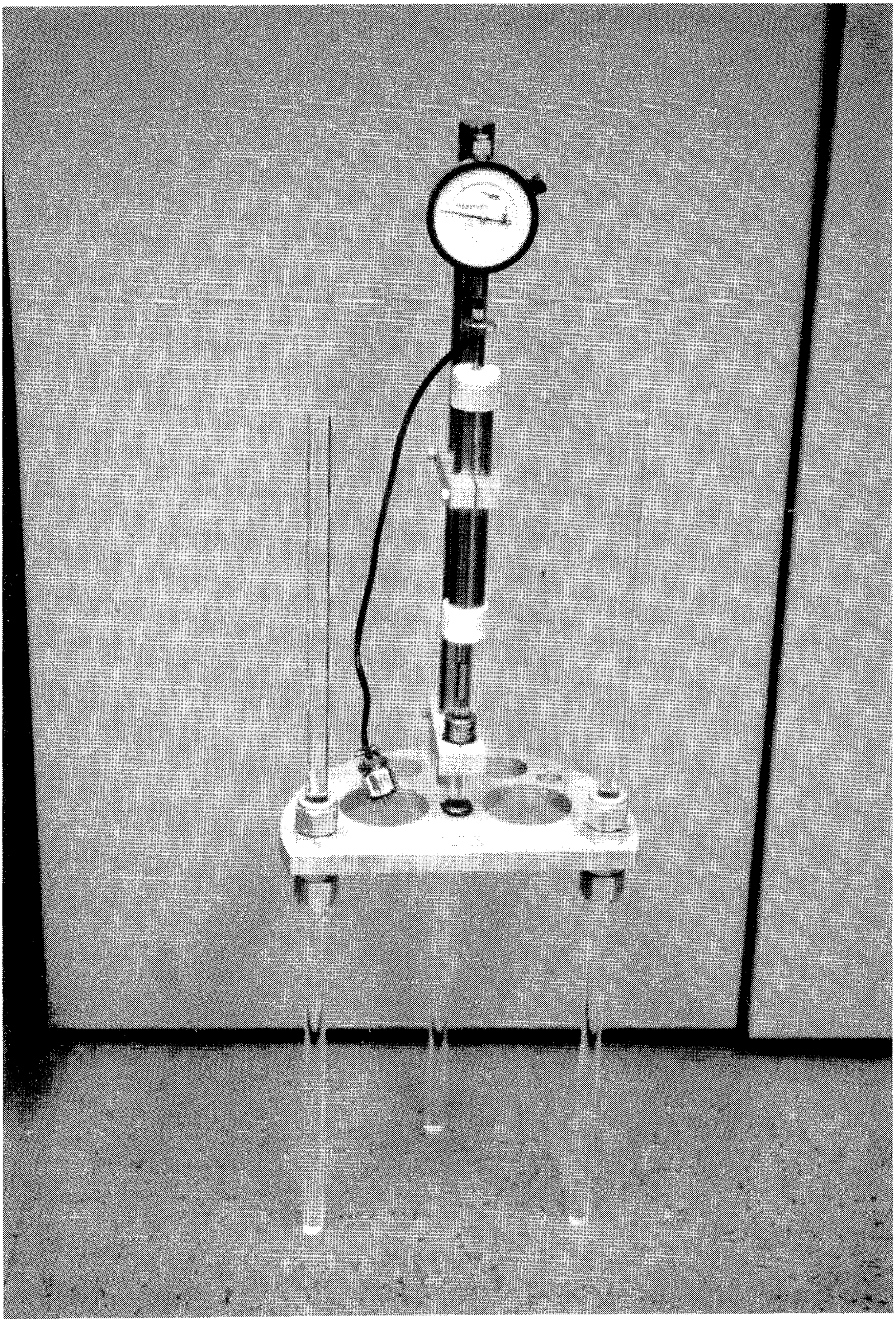


Figure 15: Tripod dilatometer used to measure the linear thermal expansion coefficients of asphalt pavement at low temperatures.

temperature bath. Sample temperature was measured by a thermistor (± 0.1°C) taped about midway along the side of the sample.

Calibration of the dilatometer system was accomplished by measuring the expansion coefficient of an aluminum bar (Alcoa TG51-6060), 3.0" in diameter and 3.828" in length (76.2 mm x 97.2 mm) which has an expansion coefficient comparable to that of the asphalt pavement samples.

During the initial measurements, it was found that the samples were being deformed under the applied load of the push rod and the spring-loaded dial gauge at the starting temperature (room temperature). This problem was avoided by using a starting temperature near 10°C or colder.

The experimental procedure involved placing the sample in the apparatus and allowing it to equilibrate, usually for a day or more, at the starting temperature. When the LVDT and dial gauge measurements were stable, the bath was lowered at a rate of about 7°C per hour to the low temperature (near -55°C). The sample was allowed to equilibrate, as indicated by stable length measurements, and then the bath was heated at a rate of about 5°C per hour to the starting temperature where the sample was again allowed to equilibrate. Three to six of these cycles were carried out for each sample.

The instantaneous linear thermal expansion coefficient, α_i, defined by (Touloukian et al., 1978)

$$\alpha_i = \Delta L/(L' \Delta T) \qquad (7.1)$$

where ΔL is the change in length associated with a change in temperature, ΔT, and L' is the sample length measured at room temperature ($\approx$ 20°C), was calculated for both the contraction and expansion part of the cycles. Mean values and their standard derivations are reported in Table 4.

TABLE 4: Measured values for the instantaneous linear thermal expansion coefficients, α_i, of asphalt pavement samples at low temperatures.

Sample	Length (mm)	Average Temp (°C) High	Low	Mean value and standard deviation for $\alpha_i \times 10^6$ (°C^{-1})
MH-1	85.24	+10.1	-53.8	21.2 ± 0.7
MH-2	85.29	+9.2	-54.1	17.2 ± 0.4
MH-3	155.30	+7.4	-54.7	18.8 ± 1.2
MH-4	150.83	+7.1	-55.2	18.4 ± 0.6
A-1	83.31	+16.2	-48.6	19.0 ± 1.0
A-2	86.26	+9.9	-53.4	20.1 ± 2.0
A-3	152.45	+6.4	-55.8	17.4 ± 1.0
A-4	151.59	+6.7	-56.1	21.7 ± 1.8
K-1	79.20	+8.6	-53.8	19.7 ± 1.1
K-2	85.70	+7.9	-52.9	22.3 ± 0.2
K-3	152.15	+6.4	-56.1	21.5 ± 1.9
K-4	154.31	+6.5	-55.9	20.6 ± 0.3

These values are useful for determining the pavement cracking temperature (Asphalt Institute, 1981)(see Sec. VIII F).

VIII. MISCELLANEOUS INVESTIGATIONS

A. Crack Characteristics

Several cracks were excavated, using a pick and shovel, on the shoulder of the road to try to determine the depth of penetration of the crack. It was not possible to trace these cracks beyond the 1 m depth because of debris created during the excavation. Wires were also pushed down into cracks but generally failed to penetrate more than 0.5 to 1 m. Based on these sparse observations, it is thought that some of the major transverse pavement cracks must extend to at least 1 m below the pavement surface.

B. Cracking in Resurfaced Asphalt Pavements

The northern entrance to the UAF from Farmer's Loop Road was formerly a gravel road for many years. A number of major transverse cracks were observed (Osterkamp, unpublished) during the 1972-1975 period on a nearly daily basis. It is his impression that, after the road was paved, it cracked the first winter at the same places. Some of these cracks extend across the shoulder of the road, through a ditch and then through a bicycle path.

Prior to resurfacing Miller Hill Road, the crack positions were marked by stakes placed at the edge of the right-of-way on both sides of the road. The first winter, the road cracked at the same sites as the original cracks which was determined by sighting between the stakes.

C. A Crack Observation in Annually Frozen Ground

In February, 1982, we received a report of a crack in the yard of a home near the dog mushers field on Farmers Loop Road. The crack was about 30 m from the house of Mr. T. Keltner. It appears to have originated on one side of his ploughed drive, extended across the drive and under the snow cover along the side of it for a total distance of 5.3 m in the northerly direction. At this point, under the snow cover, it turned east for about 2.6 m and then disappeared. The eastern portion was S-shaped. Maximum width of the crack was about 2 cm and the maximum depth, measured with a wire pushed into the crack, was about 0.5 m. According to Mr. Keltner, the crack occurred at night, suddenly, and sounded like the cracking of a large wooden beam. A check for seismic activity at the Geophysical Institute Seismic Station showed that there was none at the time the crack formed. While the ground in the area was thawed under the annually frozen zone, it is believed that permafrost was probably present at depth.

D. Air and Pavement Surface Temperatures

Air temperatures were measured at all sites with a thermistor thermometer as reported in Section III. In addition, during the summer and fall of 1982, pavement surface temperatures were measured at eleven sites (Appendix E). These surface temperatures were measured with a hand-held Barnes PRT-10 infrared radiation thermometer. The sensitivity of this instrument is about ± 0.5°C. An estimated accuracy of about ± 3°C was achieved by using a radiation calibration standard.

Typically, during the summer, pavement surface temperatures were much higher than the air temperatures and, during the fall, they were slightly lower than the air temperatures. For example, in July, maximum pavement surface temperatures averaged about 10°C warmer than the air temperature with a maximum difference of about 17°C at Miller Hill Road - a sloped surface facing south. In late September and early October, pavement surface temperatures were typically 1-2°C colder than the air temperatures, a reversal of the summer condition.

At Peger Road, in Fairbanks, the white and yellow surfaces were typically colder by several degrees Celsius than the air temperature from August into October with the white surface slightly colder (a degree or two) than the yellow surface. On July 21st, the air, white surface, yellow surface, and unpainted asphalt surface temperatures were about 22°C, 34°C, 36°C, and 40°C, respectively. For the same day, Berg (1985), reports respective values of 18.1°C, 20.2°C, 21.5°C, and 25.6°C, however, his measurements are the average for the day while the above are spot measurements taken at a time of total cloud cover. Another difference is that Berg's (1985) pavement measurements were made about 3 mm below the surface of the pavement. Equation 2.1 suggests that the response of the asphalt pavement to changes in air temperature should be fairly rapid, on the order of an hour. Therefore, with the air temperatures in reasonable agreement, it is difficult to understand the cause for the large differences in the pavement surface temperature measurements. On August 10th, our measured temperatures were + 20°C, 15°C, 16°C, and 20°C while Berg (1985) found 13.7°C, 16.1°C, 16.7°C, and 19.4°C, respectively. At this time, the air temperatures do not agree as well (20°C compared to 13.7°C) but the pavement surface temperatures are in excellent agreement.

E. Crack Observations in the Deadhorse and Old Man Airport Runways

The Old Man Airport runway is located near the Old Man pipeline camp just south of the Arctic Circle. It is an unpaved, gravel, runway about 1600 m in length and about 70 m in width. The gravel fill appears to be 2-3 m in thickness. Major transverse cracks were observed in this runway during July, 1985 (Figure 16) at an estimated spacing of 30-60 m. There

Figure 16: A major transverse crack in the gravel runway of Old Man Airport.

are several areas of the runway that appear to be settling by thaw consolidation of the underlying ice-rich permafrost and by thawing of ice wedges. Water is pooled at a number of places on the uphill (north) side of the runway. Some of these cracks appear to be associated with the polygonal ground (ice wedges).

The Deadhorse Airport runway near Prudhoe Bay is an asphalt paved runway with a paved parking area. It is about 2000 m in length and 84 m in width with a gravel fill 2-3 m in thickness. The runway is not insulated and there are no obvious areas of major settling due to permafrost degradation. However, there are a few places in the ramps and parking areas that are obviously settling (Figure 17).

There are landing lights along the edges of the pavement at 60 m intervals. Additional directional landing lights are arranged along lines across the runway spaced at 30 m intervals with seven lights per line for the southwest half of the runway. These lights are apparently installed by trenching across the runway to depths of 0.3-0.5 m before paving.

The major transverse cracks along the southwest part of the Deadhorse Airport runway are along the same lines as the directional landing lights and have the same spacing of 30 m (Figures 18 and 19). These cracks appeared to be much less prominent (much smaller in width) than those across the northeast part of the runway. Some of the cracks extended through the gravel apron along the side of the paved portion of the runway. Cracks across the northeast part of the runway were more randomly spaced with an estimated spacing of 40-75 m. These cracks had just been filled with asphalt and were very prominent (Figure 20). The asphalt patches were estimated to be 5-15 cm in width. Many of these cracks

Figure 17: Pooled water on a ramp of the Deadhorse Airport which shows the areas subjected to settling due to the thawing of the underlying ice-rich permafrost.

line for the southwest half of the runway. These lights are apparently installed by trenching across the runway to depths of 0.3-0.5 m before paving.

The major transverse cracks along the southwest part of the Deadhorse Airport runway are along the same lines as the directional landing lights and have the same spacing of 30 m (Figures 18 and 19). These cracks appeared to be much less prominent (much smaller in width) than those across the northeast part of the runway. Some of the cracks extended through the gravel apron along the side of the paved portion of the runway. Cracks across the northeast part of the runway were more randomly spaced with an estimated spacing of 40-75 m. These cracks had just been filled with asphalt and were very prominent (Figure 20). The asphalt patches were estimated to be 5-15 cm in width. Many of these cracks extended through the gravel apron and had a 5-10 cm deep trough at the gravel surface. There did not appear to be any correlation between the position of ice wedge polygons and the cracks.

It is hypothesized that trenching the runway for the directional landing lights created lines of weakness across the runway and that, after paving, the runway and pavement cracked along these lines. Another possibility is that the wiring caused local warming which would have become lines of weakness in the frozen soils. If either or both of these hypotheses are true, it suggests that the position of the cracks can be controlled and that the severity of the cracks can be reduced (i.e., by causing a greater-than-natural number of cracks to form, with correspondingly less movement associated with each crack).

Following a different line of reasoning that led to the same conclusion, McHattie (personal communication) recently sawed partially through a

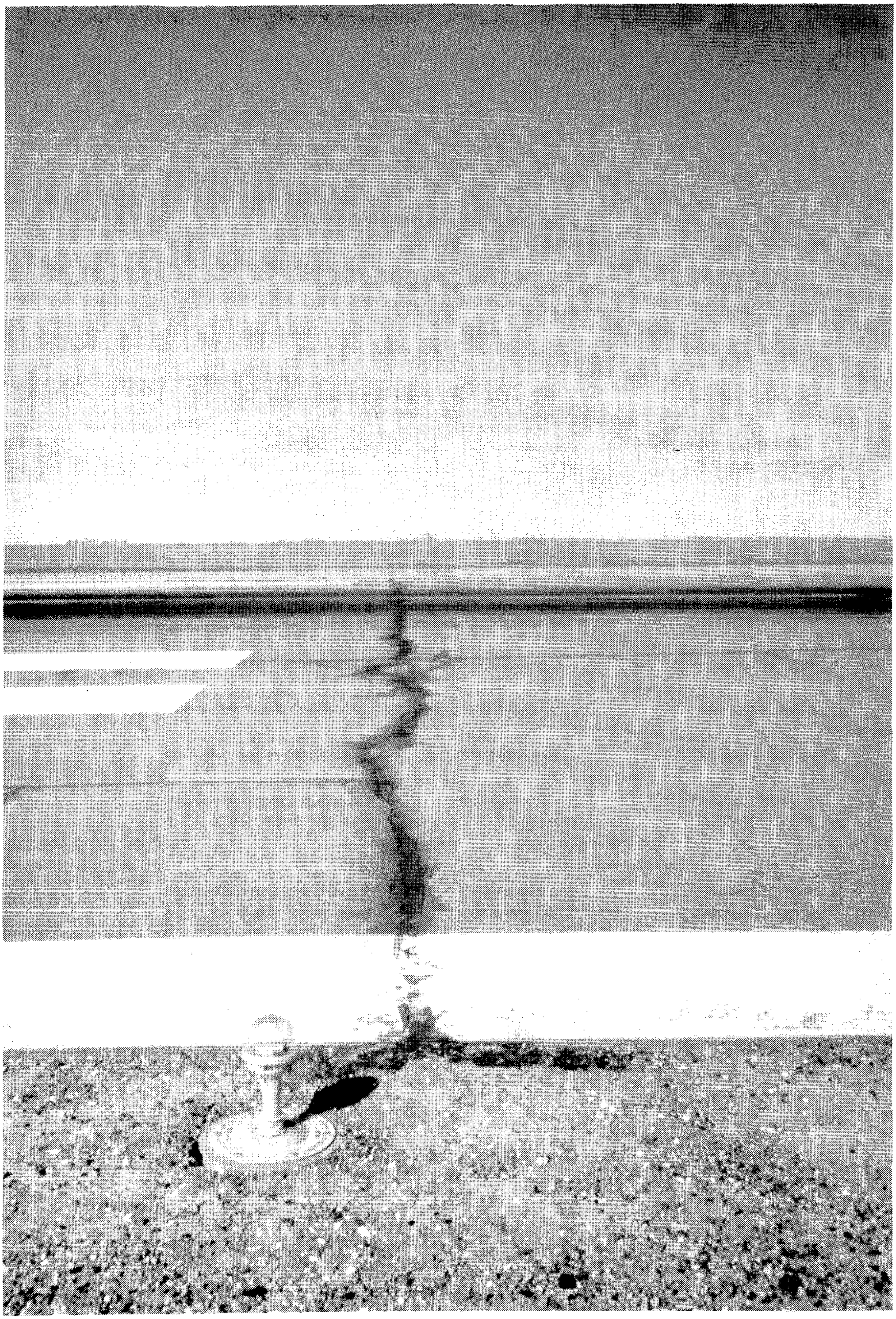


Figure 18: Major transverse crack in the pavement of the Deadhorse Airport Runway showing its relation to the landing lights along the edge of the runway.

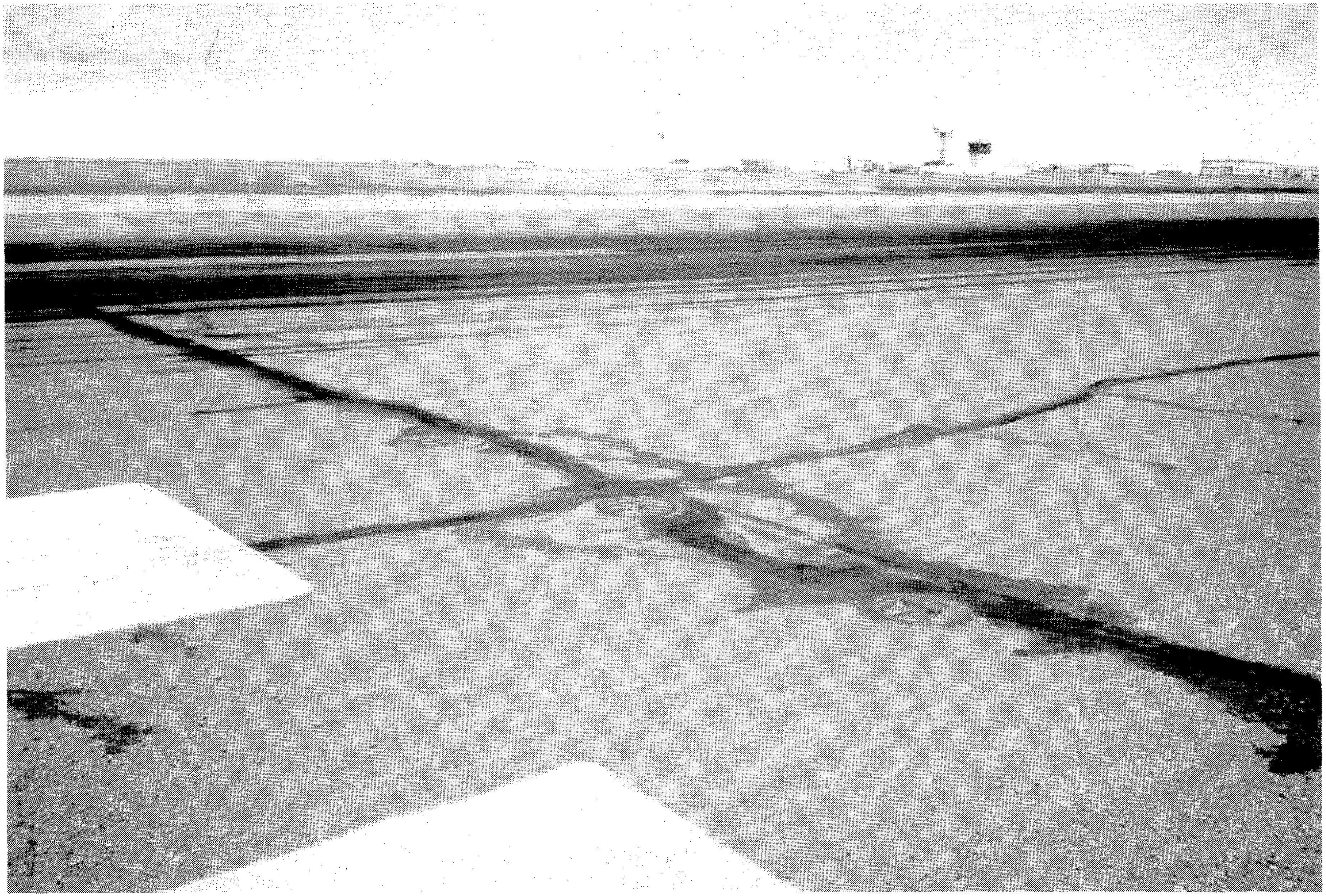


Figure 19: Major transverse crack in the pavement of the Deadhorse Airport Runway showing its relation to the directional landing lights in the runway.

Figure 20: Major transverse crack in the pavement on the northeast end of the Deadhorse Airport Runway which has been patched recently. The cracks in this end of the runway were much more prominent than in the southwest end (Figures 18 and 19).

extended through the gravel apron and had a 5-10 cm deep trough at the gravel surface. There did not appear to be any correlation between the position of ice wedge polygons off the edge of the runway and the cracks.

It is hypothesized that trenching the runway for the directional landing lights created lines of weakness across the runway and that, after paving, the runway and pavement cracked along these lines. Another possibility is that energy losses from the wiring caused local warming which would have become lines of weakness in the frozen soils. If either or both of these hypotheses are true, it suggests that the position of the cracks can be controlled and that the severity of the cracks can be reduced (i.e., by causing a greater-than-natural number of cracks to form, with correspondingly less movement associated with each crack).

Following a different line of reasoning that led to the same conclusion, McHattie (personal communication) recently sawed partially through a section of new pavement near Fairbanks at 15 m intervals along the road. The purpose of this experiment is to try to control the position of the cracks. If either or both of the above hypotheses are true, then there is a high probabiity that the saw cuts in the pavement will not only control crack position but also the severity of the cracks. If this experiment is not successful then a second experiment where the saw cuts penetrate the pavement and on the order of 0.3 m of the underlying embankment should be tried.

Unfortunately, the above ideas are based on only a few observations and are somewhat conjectural. Additional experimental data are needed, especially on the phase lag of crack movements, to fully define the physical processes associated with crack formation and crack dynamics. Nevertheless, these are new and promising ideas which should be explored.

F. Pavement Design

Design techniques to minimize low temperature transverse cracking in roads with asphalt pavement have focused primarily on the pavement layer (Asphalt Institute, 1981). The basic steps in a low temperature pavement design procedure should consist of the following:

1. Determine minimum pavement design temperature.
2. Predict pavement cracking temperature.
3. Asphalt grade selection for low temperature performance.
4. Asphalt selection for optimum overall pavement design.

The pavement design temperature can be determined from the minimum ambient air temperature. However, curves developed for use in temperate climates may not be applicable in Alaska. Air and pavement temperature data in Sec. VIII D suggest that the pavement may be colder than the air temperature during winter. Further research during the coldest winter months, using radiation thermometers, is desirable to resolve this question.

Predictions of pavement cracking temperatures can be accomplished by using a nomograph that relates asphalt penetration at 5°C and 25°C to the cracking temperature. Using the data in Table 3 (Sec. VII), a pavement cracking temperature colder than -50°C is predicted for our pavement samples on Airport Way, Miller Hill and Kantishna Drive. Unfortunately, the nomograph does not extend to temperatures lower than -50°C. However, it is our understanding that the State of Alaska presently selects the softest grade of asphalt available so that having a proper nomograph may not contribute to the design effort.

IX. MODELS FOR CRACK AND PAVEMENT MOVEMENTS

An evaluation of several models for thermal cracking and crack propagation in pavements is under preparation (Berg, unpublished report) and should be consulted for a detailed assessment of the models when it is available. However, the application of these models to Alaskan pavements calls for attention to several particular shortcomings in the models.

In general, the models emphasize the effects of accumulated cyclic thermal stress rather than unstable fracture due to sudden temperature change. The accumulated cyclic thermal stress effectively characterizes the aging history (or the "hereditary integral") of the asphalt. The primary failure mode of pavement in Alaskan conditions is not known with any degree of certainty. Therefore, it is not clear whether the prevalent hereditary integral approach is more suitable than a simpler mechanical model. Furthermore, the aging models do not appear to account for asphalt variability, calculating the aging for all asphalts similarly. This may be invalid in Alaska where very soft asphalts are normally used.

It is worth noting that the prediction of crack formation and growth as well as crack spacing, is sensitive to the prediction of ground temperature in the base course. Several models (Carpenter and Lytton, 1977; Shahin and McCullough, 1972) use a modified form of the "Barber" equation for predicting the diurnal variation in ground temperatures. The equation specifies an analytic solution for ground temperatures assuming diurnal variation in short wave radiation and air temperature. Unfortunately, the above models use an overly simplified version of the "Barber" equation which assumes that short wave radiation and air temperature are in phase, that the short wave radiation is a local average, and furthermore, that the lag period between maximum air temperature and surface temperature is constant and always equal to 1.1 hours. The inaccuracy of these assumptions can be seen in Figures 7 and 8 of a recent report by Berg (1985) where the lag between surface and air temperatures varies between zero and three hours, and air temperature consistently lags short wave radiation. The unmodified Barber equation suggests that the lag between short wave and ground temperature maximums is variable and decreases as wind speed increases which appears to be the case in Figure 7c of the same report.

If the models are to be used in a statistical manner for the prediction of the probabilities of crack formation and spacing, then mean ground temperature variations are adequate parameters. However, model verification or the comparison of several different models to evaluate their predictive capabilities with actual Alaskan pavement cracking requires more accurate information on ground temperatures. The implementation of the complete Barber equation is a relatively straightforward modification of the Carpenter and Lytton (1977) model which would substantially improve ground temperature prediction. Furthermore, an analytic extension of the Barber equation which permits a linear temperature gradient in the underlying soil is relatively simple to obtain, and may be directly programmed into the models.

X. SUMMARY AND RECOMMENDATIONS

A. Summary

The formation and dynamics of low temperature transverse cracks in asphalt pavement have been investigated in an effort to develop an understanding of their physical nature and of the physical mechanisms that govern their movements. Caliper measurements of crack movements showed that the average movement was 14.21 mm with a maximum movement of 27.86 mm for a one year period. There is a lag in crack movement with air temperature on the order of a day to a week. This suggests that the movement of the cracks is controlled, not only by the asphalt pavement, but by all the material in the top 2 m of the embankment. Crack movements were rapid and large during winter and gradual and small during summer. The cracks returned to within 1 mm of their previous widths over a period of a year. Average crack spacing was 23.6 m. There was a general trend of increased crack movement with an increase in average crack spacing. An approximate method was used to correct for the thermal contraction and expansion of the tape. However, even with this correction, some negative values of the thermal expansion coefficient of the pavement and the permafrost were found, contrary to expectations. Pavement surface elevations near cracks were about 1.1-1.2 cm higher (relative to the nearby pavement) in February as compared to the following June. These changes in surface elevation of the pavement near cracks are thought to be caused by wetting of the material under the pavement by water intrusion through the cracks with subsequent frost heaving and thaw consolidation. The linear thermal expansion coefficients of twelve samples recovered from asphalt pavement ranged from 17.2 to 22.3 x 10^{-6} (°C)$^{-1}$ with an average value near 19.8 x 10^{-6} (°C)$^{-1}$.

Cracks in resurfaced asphalt pavements recurred at the positions of the old cracks. One crack that reportedly formed suddenly in the frozen ground of a ploughed driveway was examined. Summer daytime pavement surface temperatures averaged about 10°C warmer than the air temperatures. Pavement surface temperatures measured in the Fall were typically 1-2°C colder than the air temperatures suggesting that this condition might prevail during Fall, Winter and early Spring. White and yellow pavement surfaces were significantly cooler than the black pavement surface during summer.

Predicted pavement cracking temperatures for three pavements in the Fairbanks area are colder than -50°C. Crack observations in the Deadhorse Airport runway suggest that the position and severity of low temperature transverse asphalt pavement cracks can be controlled by introducing transverse zones of weakness into the pavement and/or underlying embankment. These zones could take the form of trenches, pavement cuts, chemical treatment etc.

B. Crack movement studies

This research has shown that the lag in crack movements with air temperatures is much smaller than believed, probably about a day or so, from our measurements. Therefore, additional research on crack movements (opening and closing) is needed to determine the lag relationship between crack movement and air temperature which will provide information on the physical processes controlling crack movement and the movement rates which will be useful in evaluating potential solutions (e.g., by comparison to the flow rates of crack sealers). These studies will require, at the least, daily measurements on the cracks and air temperatures as well as measurements of the temperature profiles through the pavement to depths of several meters. Continuous measurements would be the most desirable.

C. Pavement movement studies

The strain, $\Delta L/L$, between cracks should be nearly constant. However, interpretation problems with the data make this difficult to verify. More careful measurements on a daily basis, taking into account the temperature of the tape extensometer are needed. These measurements should contribute additional information to our understanding of the physical processes that govern crack dynamics. This type of study should be carried out in conjunction with B above.

D. Thermal expansion of permafrost

This study suggests that the thermal expansion, contraction and cracking of permafrost may be contrary to expectations. Therefore, it would be desirable to conduct studies on permafrost, similar to those suggested in B and C above, but with improved instrumentation, to confirm or reject these results. In addition, laboratory measurements of the linear thermal expansion coefficients of permafrost are needed to provide basic physical data for interpretation purposes.

E. Crack spacings

Crack spacing measurements may be valuable in developing procedures for reducing the impact of cracking on pavements (e.g., by the use of saw cuts). If these procedures prove useful in reducing the severity of the transverse cracking problem then it would be desirable to use measured crack spacings on a number of roads and runways, statewide, for determining the required spacing of the saw cuts.

F. Vertical pavement displacements near cracks

The results from the data of this study show that the road surface near road cracks may rise on the order of a centimeter during winter and settle by several millimeters during the summer. These motions are probably associated with frost heaving and thaw settlement. One problem with the measurements was the short span used over the crack. It would be useful to make measurements over a longer span at a few sites for a full year since it is known that the long-term maintenance problems with major thermal cracks results from settlements adjacent to these cracks.

These results also suggest that it would be desirable to devote some engineering effort to effective methods for sealing the cracks against the intrustion of water during break-up and the thaw season.

G. Air and pavement surface temperature

It is necessary to define the relationship between air and pavement surface temperatures to determine the pavement design temperature. A knowledge of this relationship would also be useful in numerical models of embankment temperatures. It is recommended that a thorough study of this relationship be made which should include meteorological, pavement surface and both shallow and deep ground temperature measurements. One such study (Berg, 1985) has been made in Fairbanks but other sites in the state should be investigated. The pavement surface temperature should be determined with a radiometer rather than a contact thermometer.

H. Control of crack position and severity

This study suggests that both the position and severity of transverse pavement cracks could be controlled by introducing transverse zones of weakness into the pavement and/or embankment. These zones of weakness could take the form of pavement and/or embankment cuts or trenches. A knowledge of crack spacing and crack movement in the general area of the site will be required to properly design the experiment. Monitoring should include measurements of pavement cracking and crack movements. Special attention should be given to sealing the saw cuts in the pavement if these are used. Two sites should be chosen, one in a new embankment and one in an embankment that has been repaved. McHattie (personal communication) has already initiated this type of study.

XI. ACKNOWLEDGEMENTS

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XII. REFERENCES, FIGURES AND TABLES

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LIST OF FIGURES

LIST OF TABLES

XIII. APPENDICES

A. Caliper measurements of crack movements with time.

B. Extensometer measurements of pavement and crack movements with time.

C. Extensometer measurements in fine-grained permafrost.

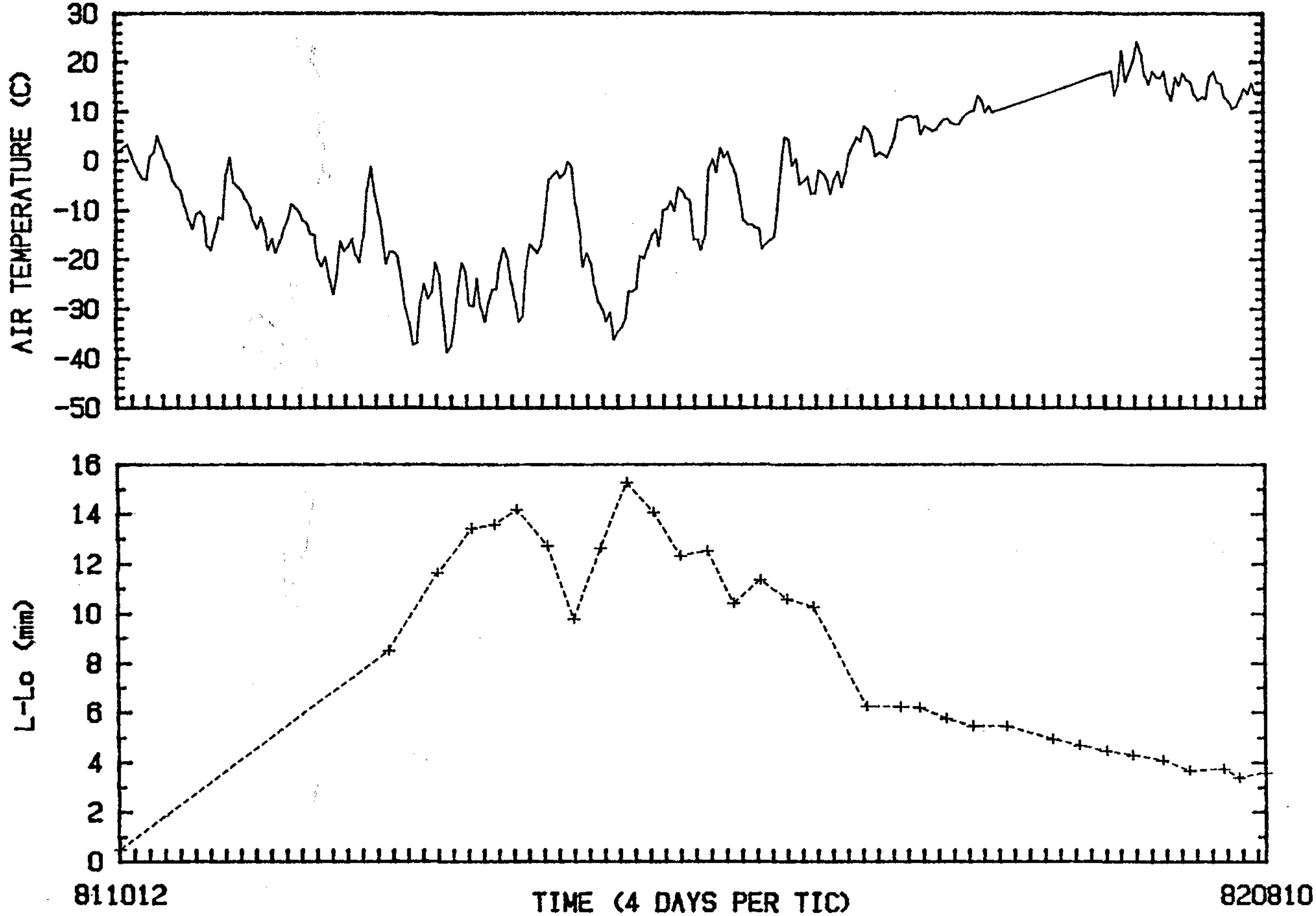
D. Measurements of the vertical pavement displacements near cracks.

E. Air and pavement surface temperature measurements.

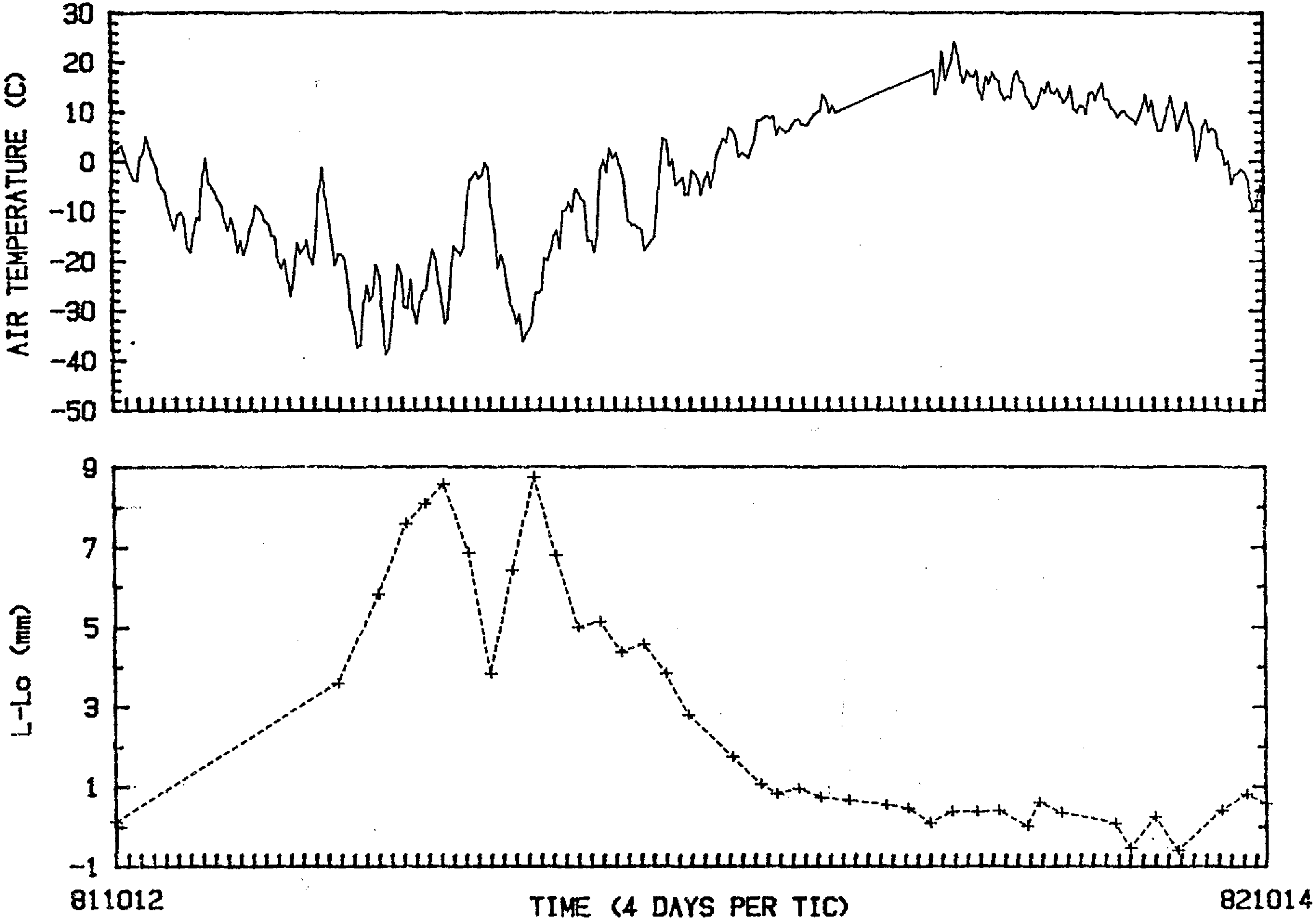
F. Estimating thermal expansion coefficients from extensometer measurements.

APPENDIX A

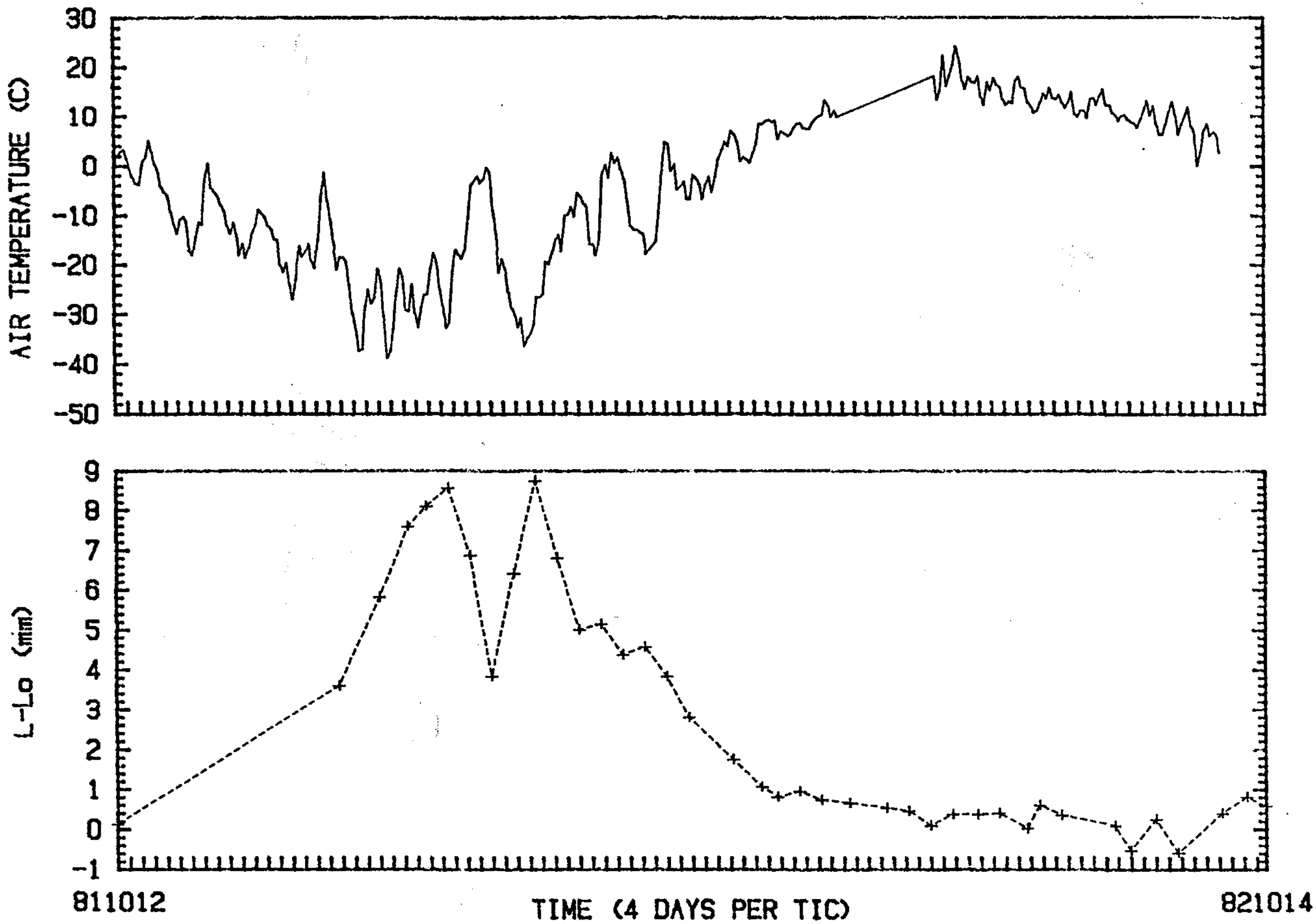
Caliper measurements of crack movements with time



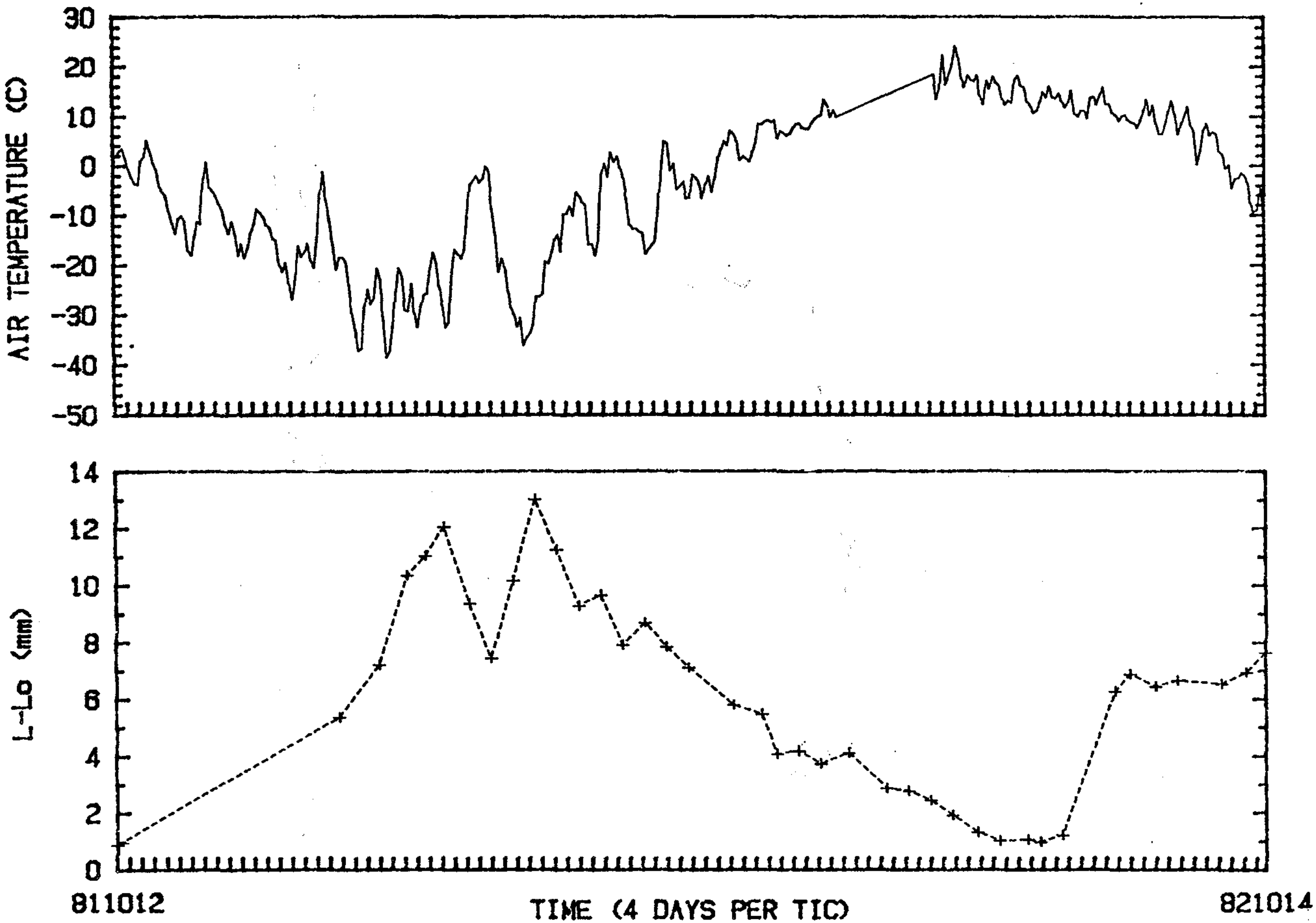
WHITE LINE PINS ALDER CREEK SITE Lo = 211.84 mm



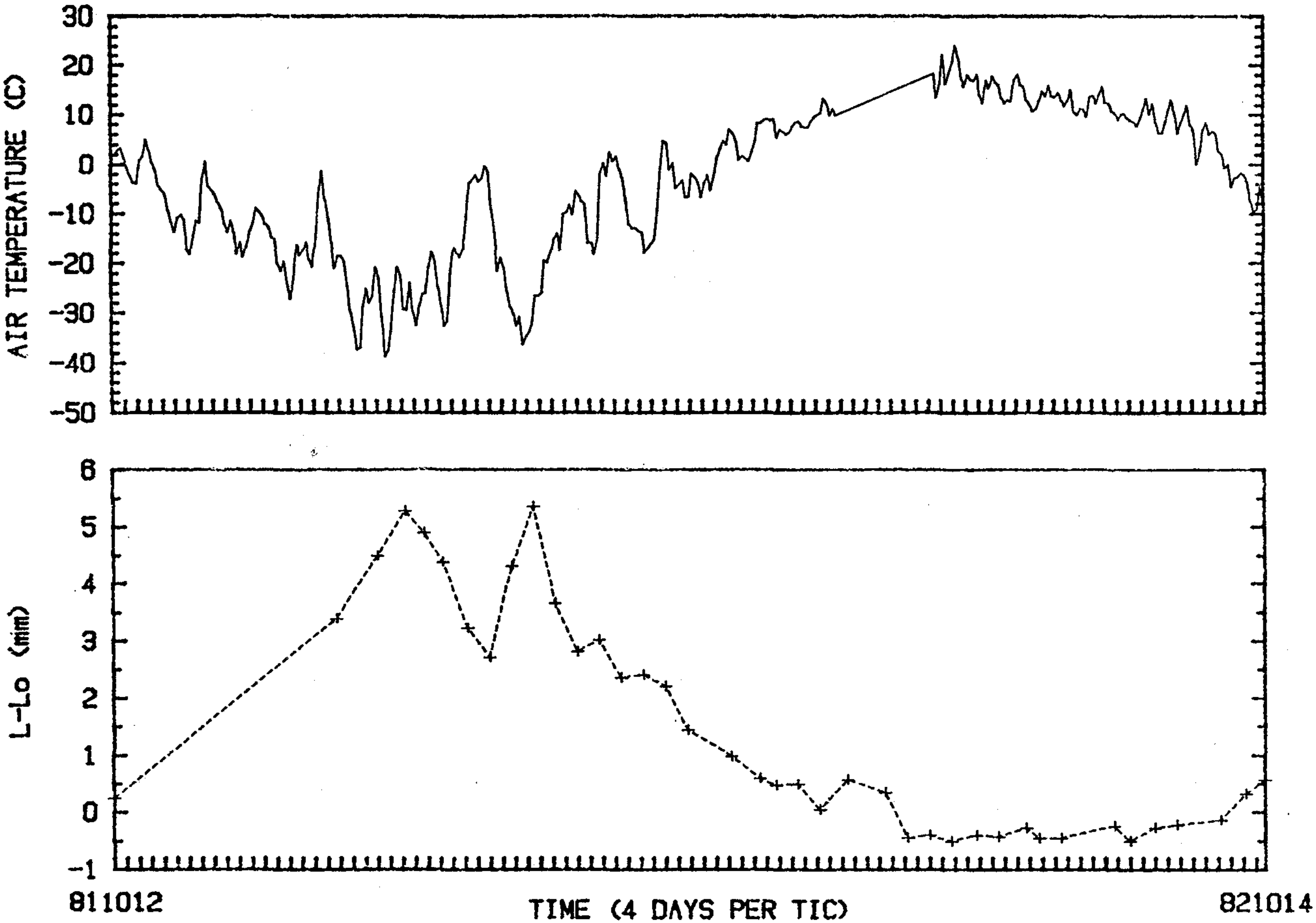
6 FT. OFF W.L.   ALDER CREEK SITE   Lo = 247.80 mm



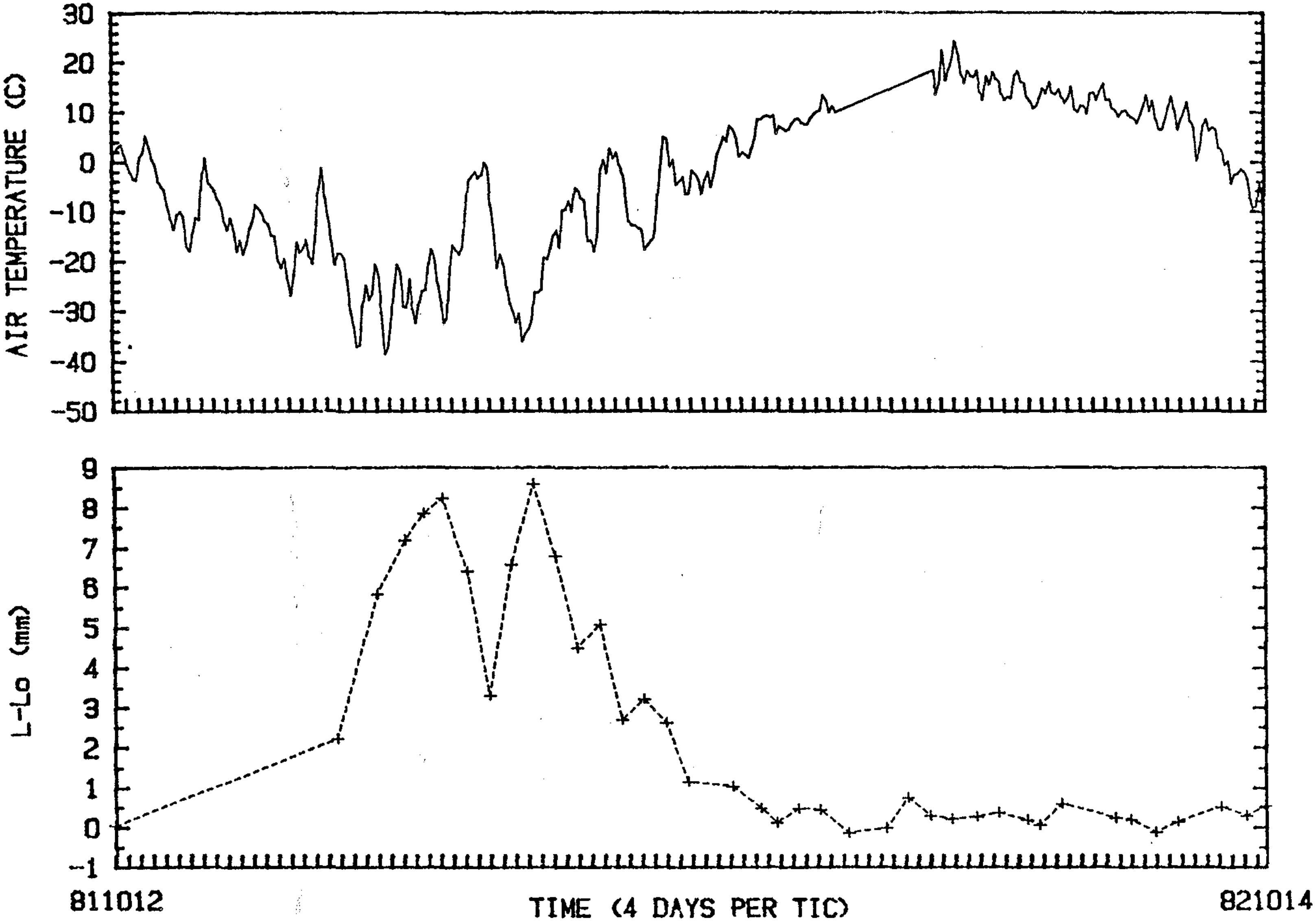
8'OFF WHITE LINE   UNIVERSITY AVE.   Lo = 247.80 mm



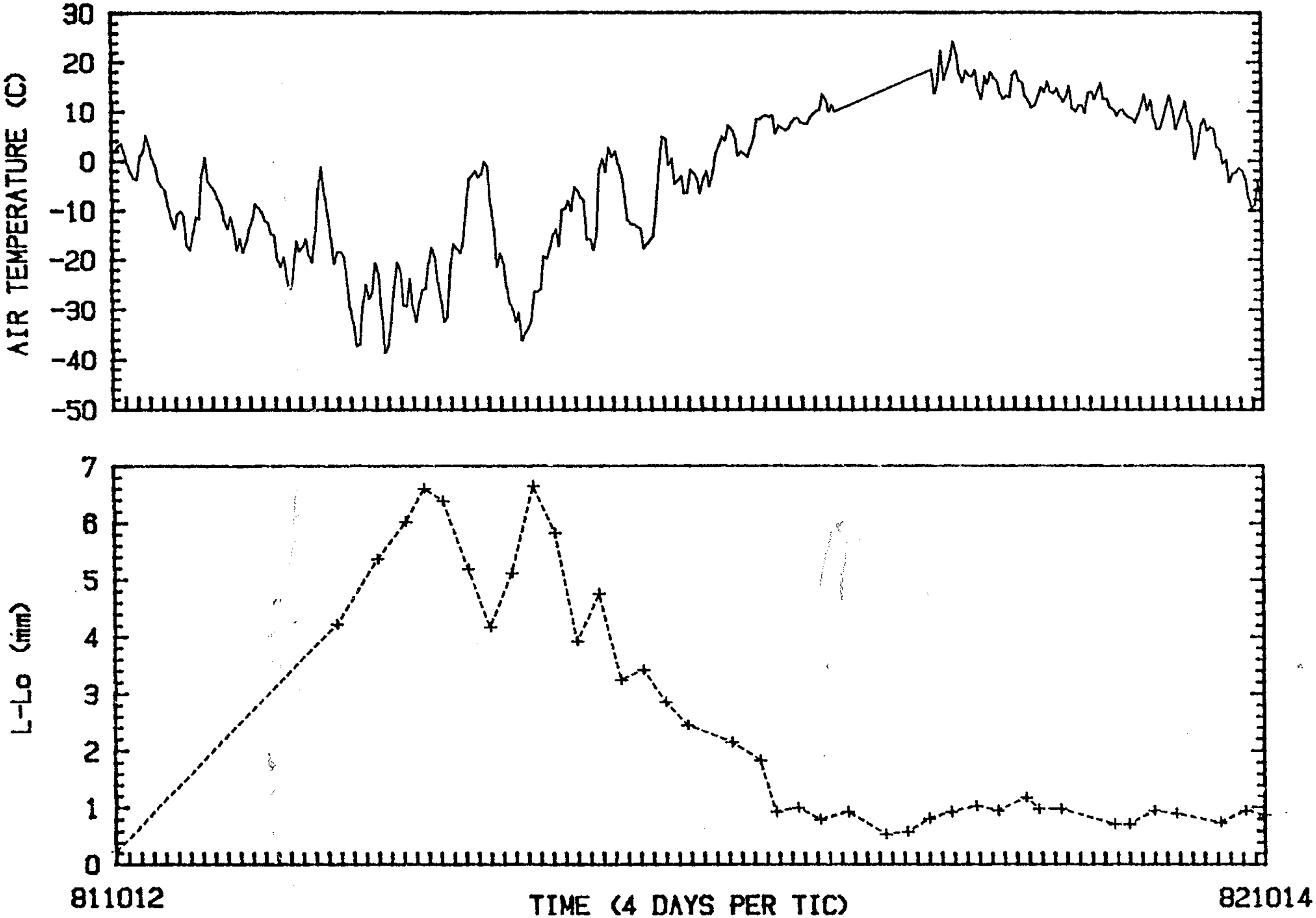
SITE 3 FARMERS LOOP #1 Lo = 207.18 mm



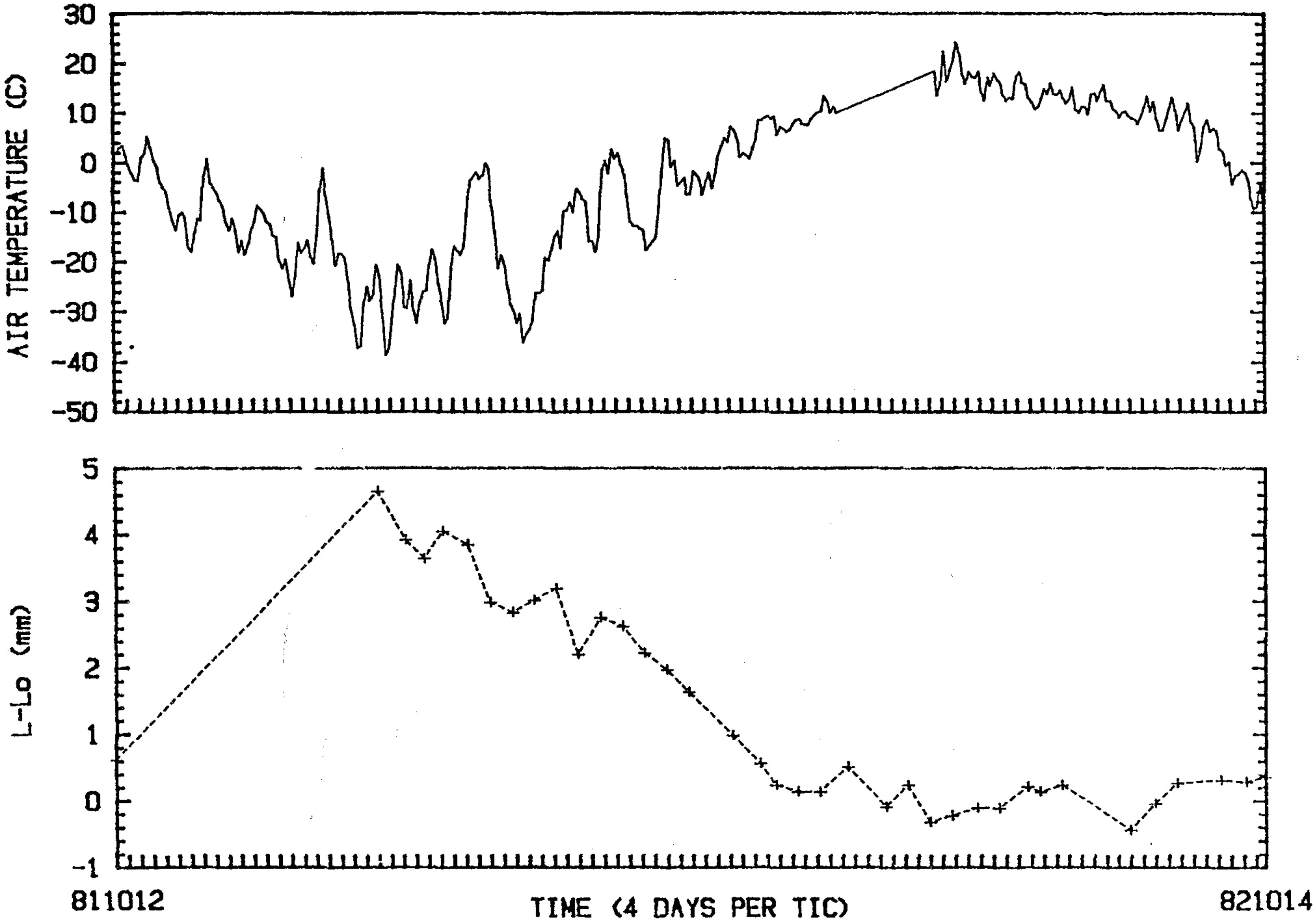
SITE 4 FARMERS LOOP #2 Lo = 213.06 mm



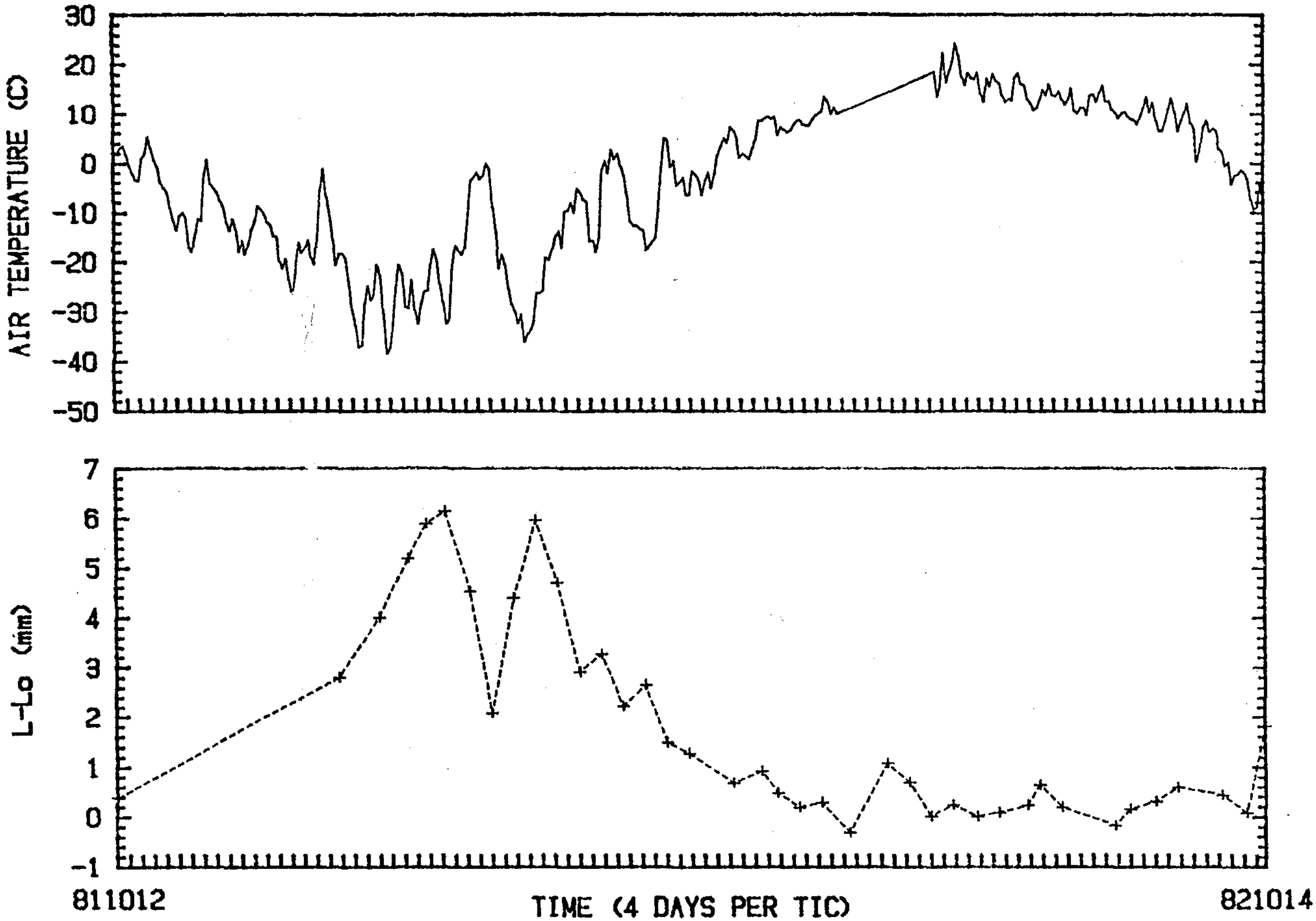
SITE 5 FARMERS LOOP #3 Lo = 201.55 mm



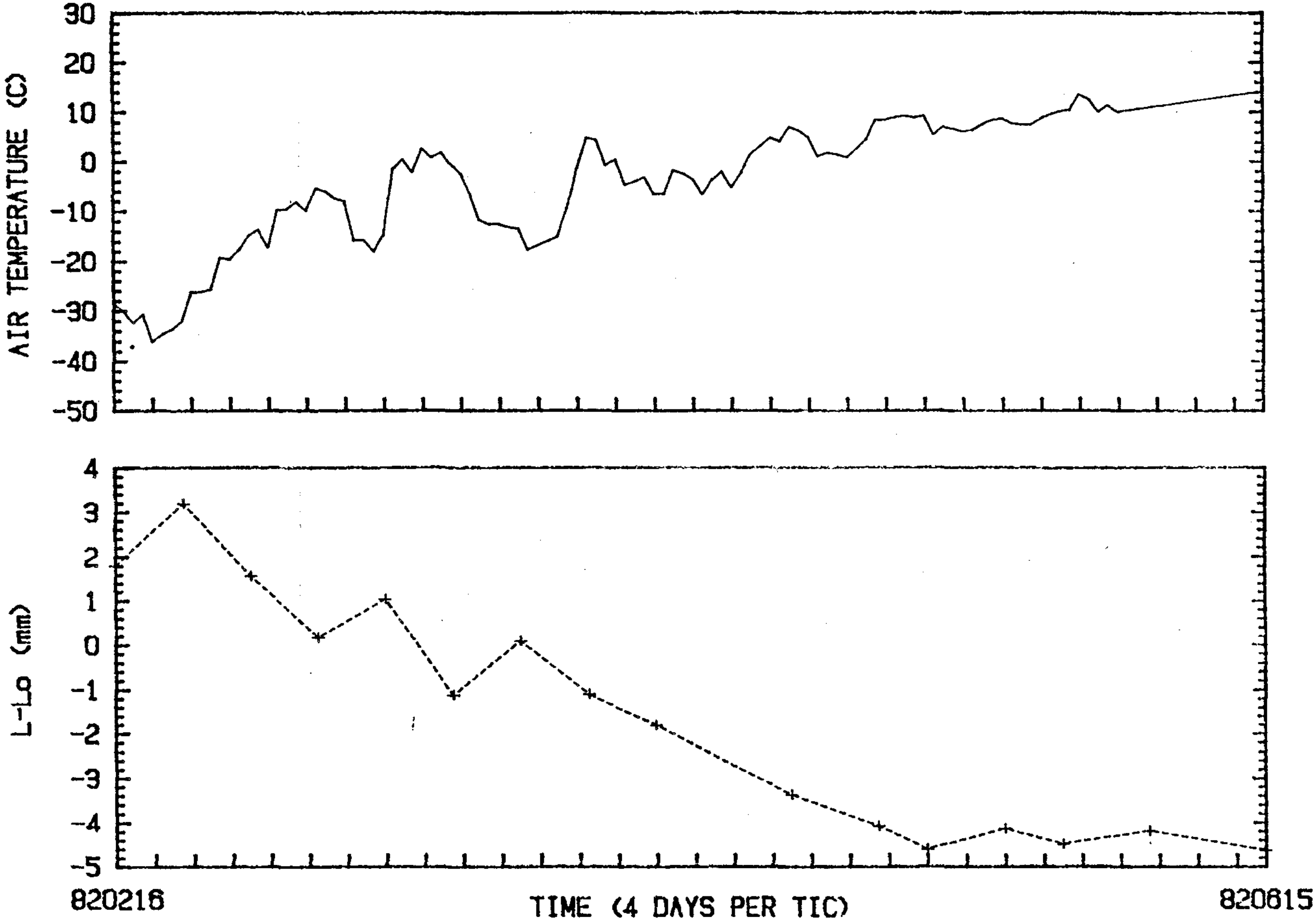
SITE 8   FARMERS LOOP #4   Lo = 204.57 mm



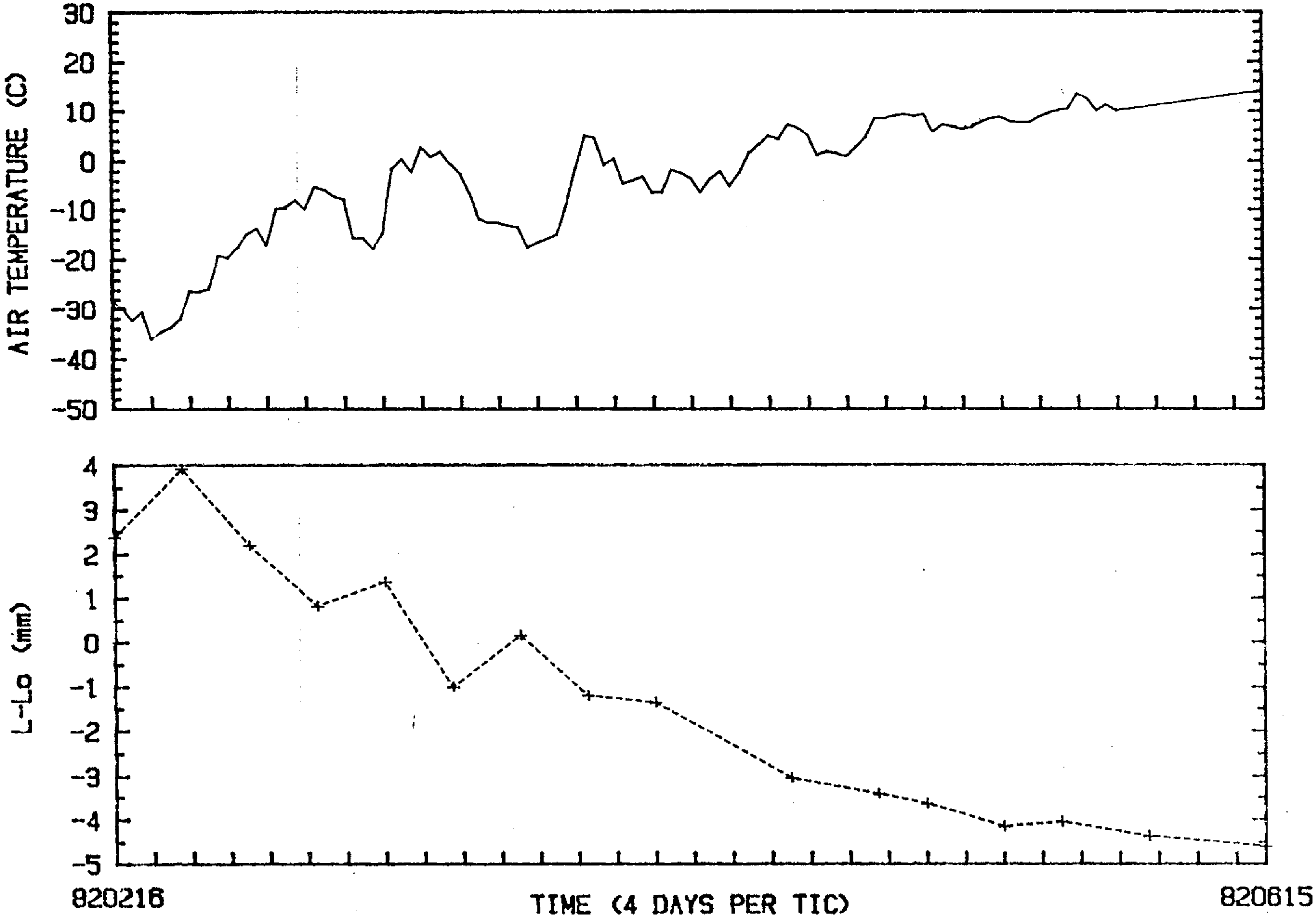
SITE 7 TRAINER GATE #1 Lo = 210.69 mm



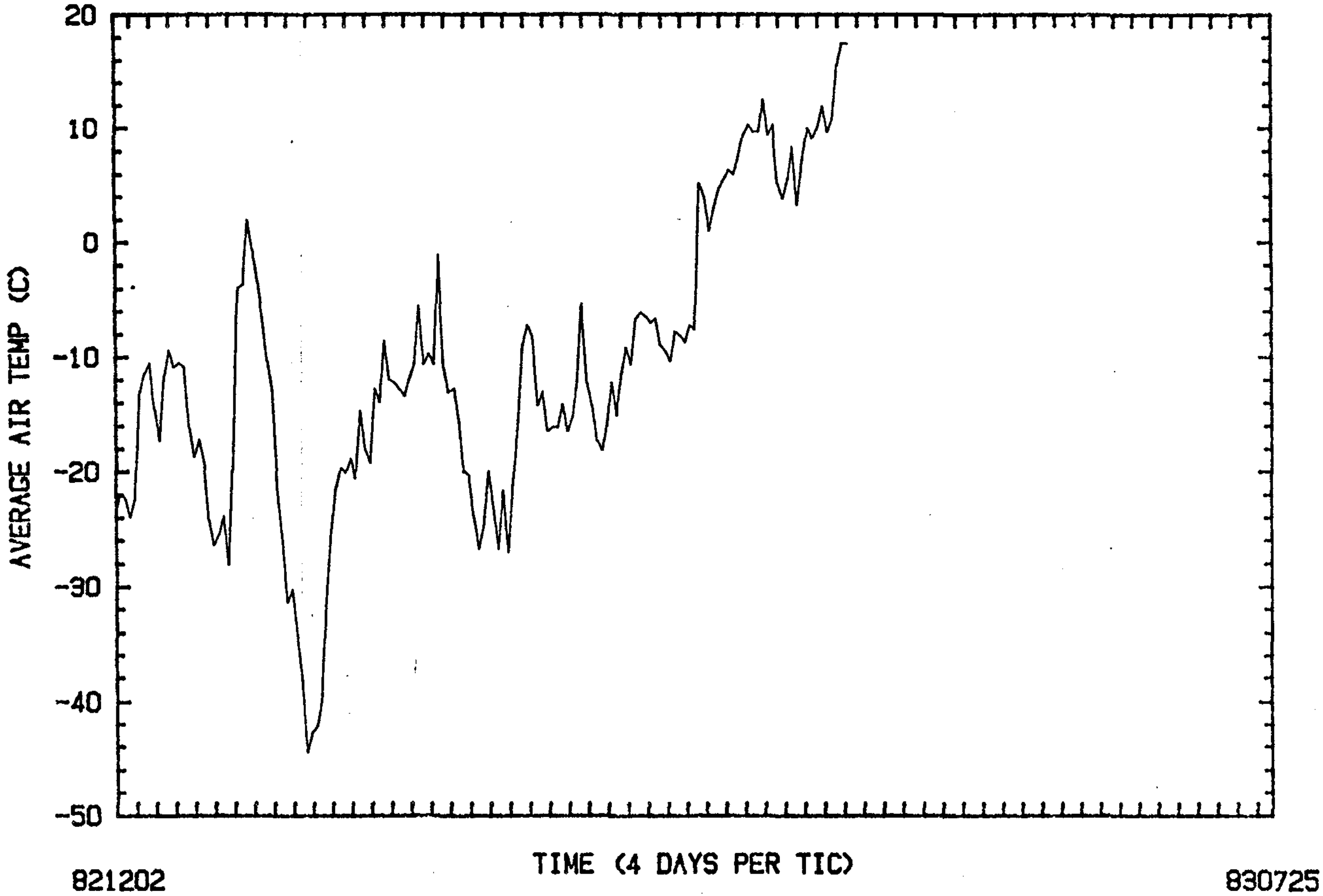
SITE 8 TRAINER GATE #2 Lo = 238.45 mm



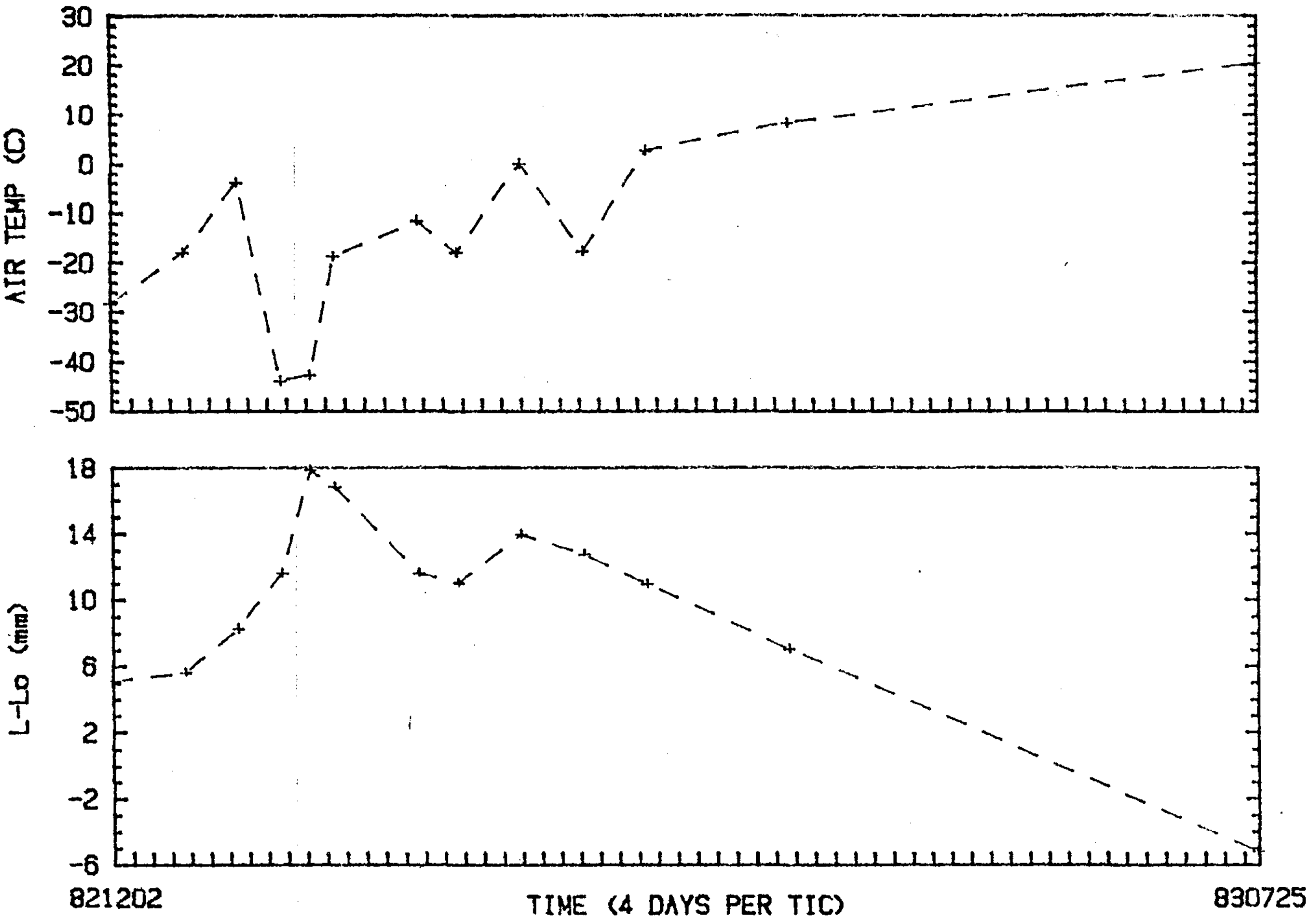
UP HILL MILLER HILL Lo = 242.19 mm



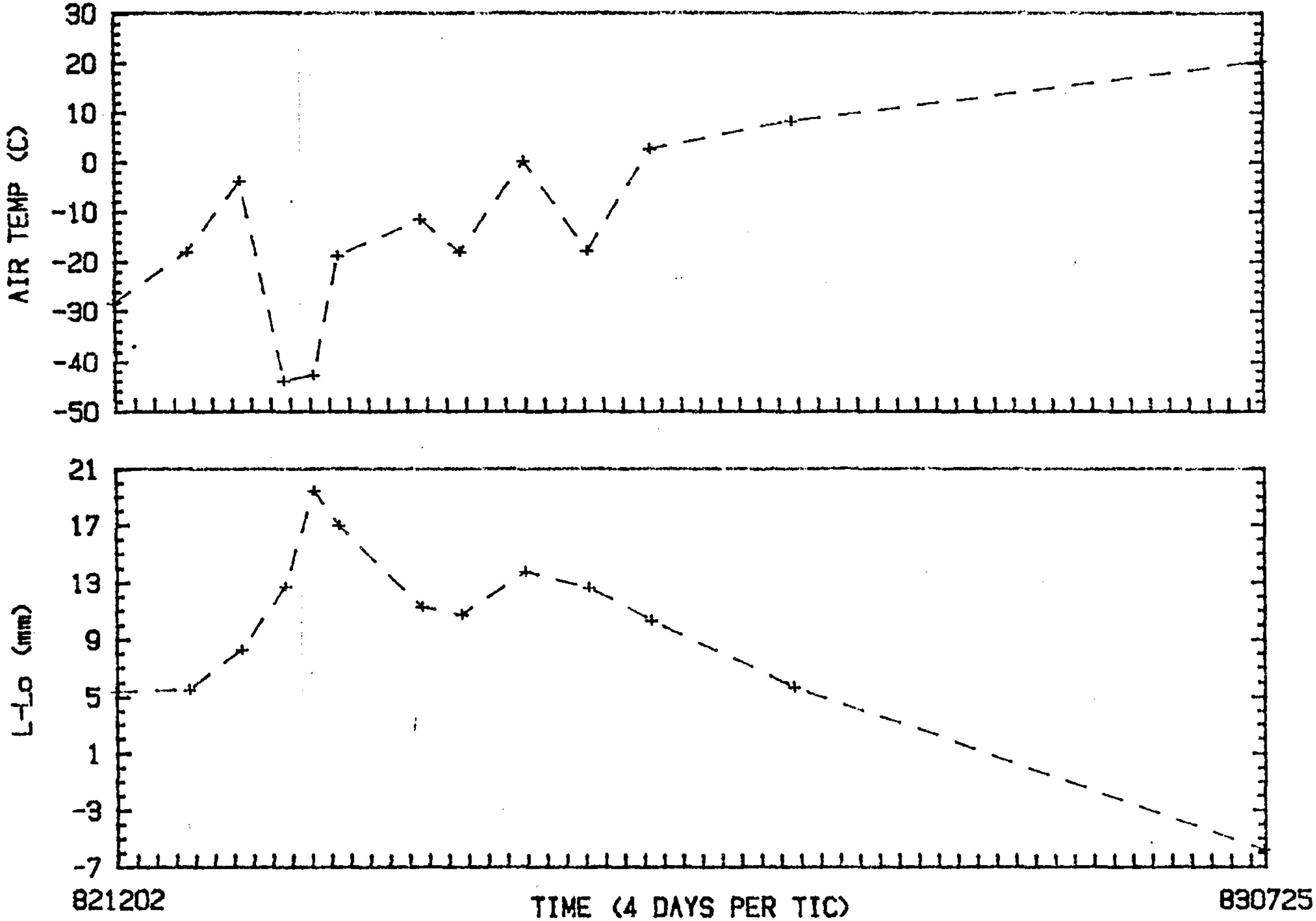
DOWN HILL MILLER HILL Lo = 232.46 mm



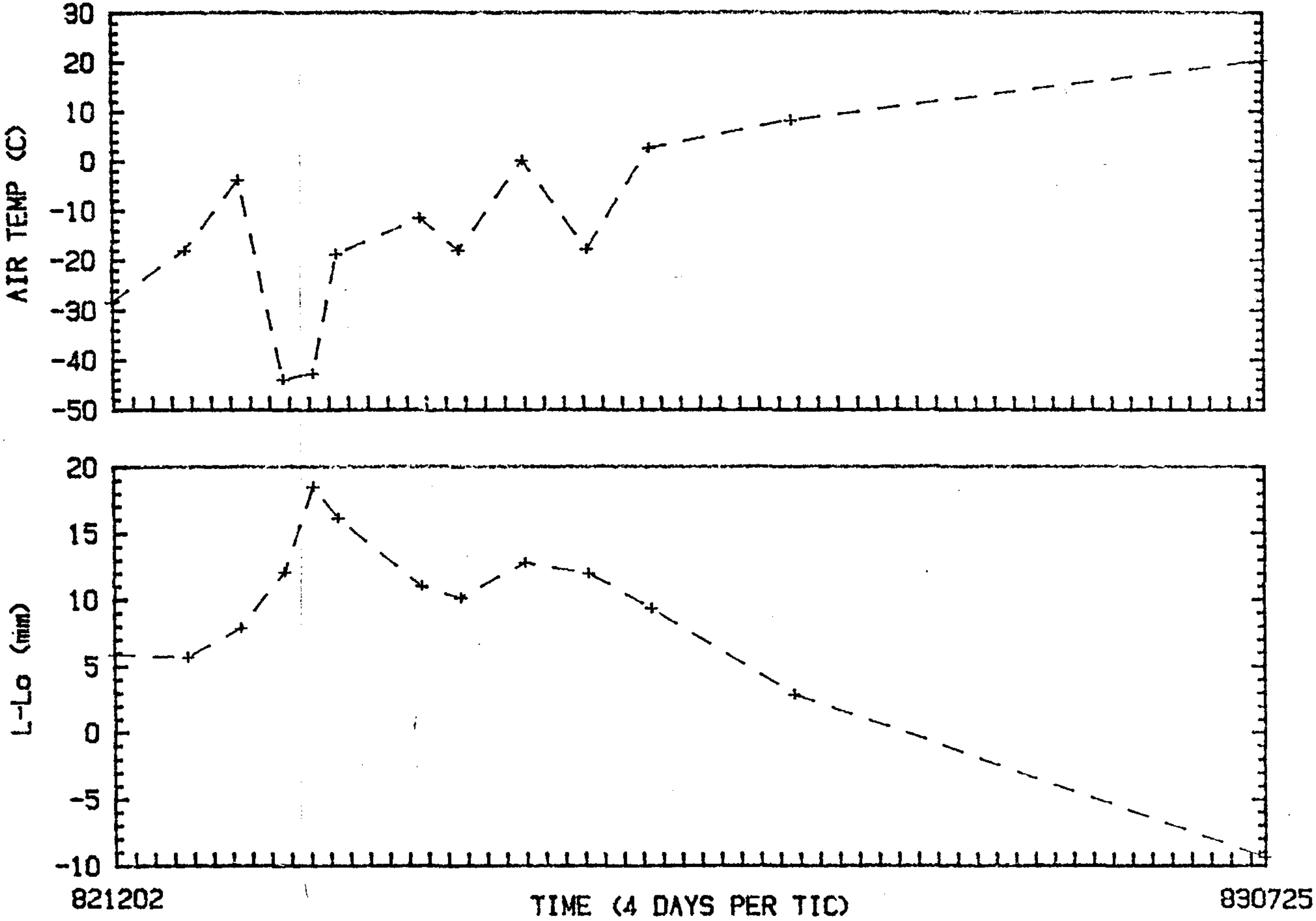
UNIVERSITY EXP STATION



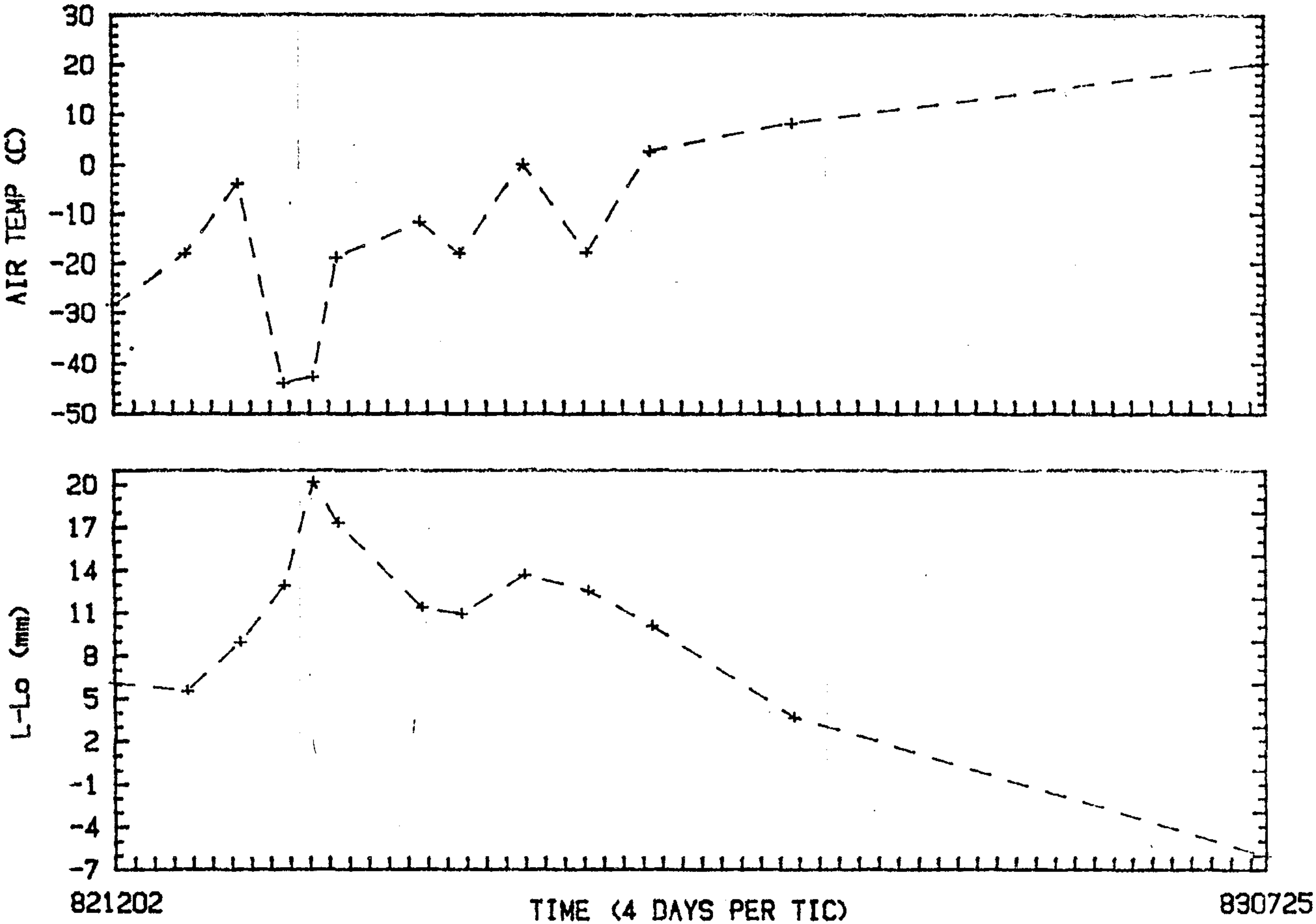
CRACK #1 PARKS HIGHWAY Lo= 217.39 mm



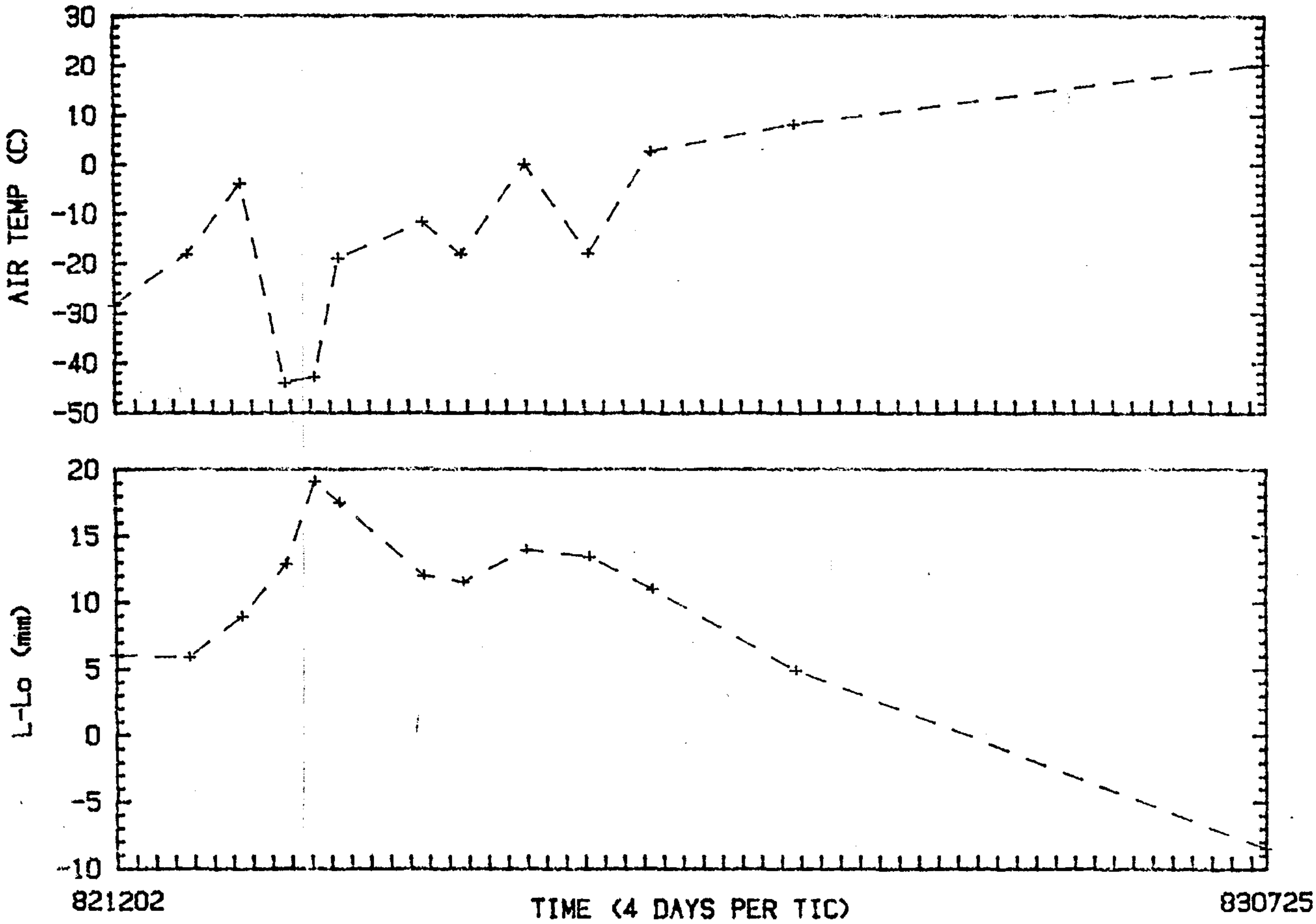
CRACK #2 PARKS HIGHWAY Lo= 233.05 mm



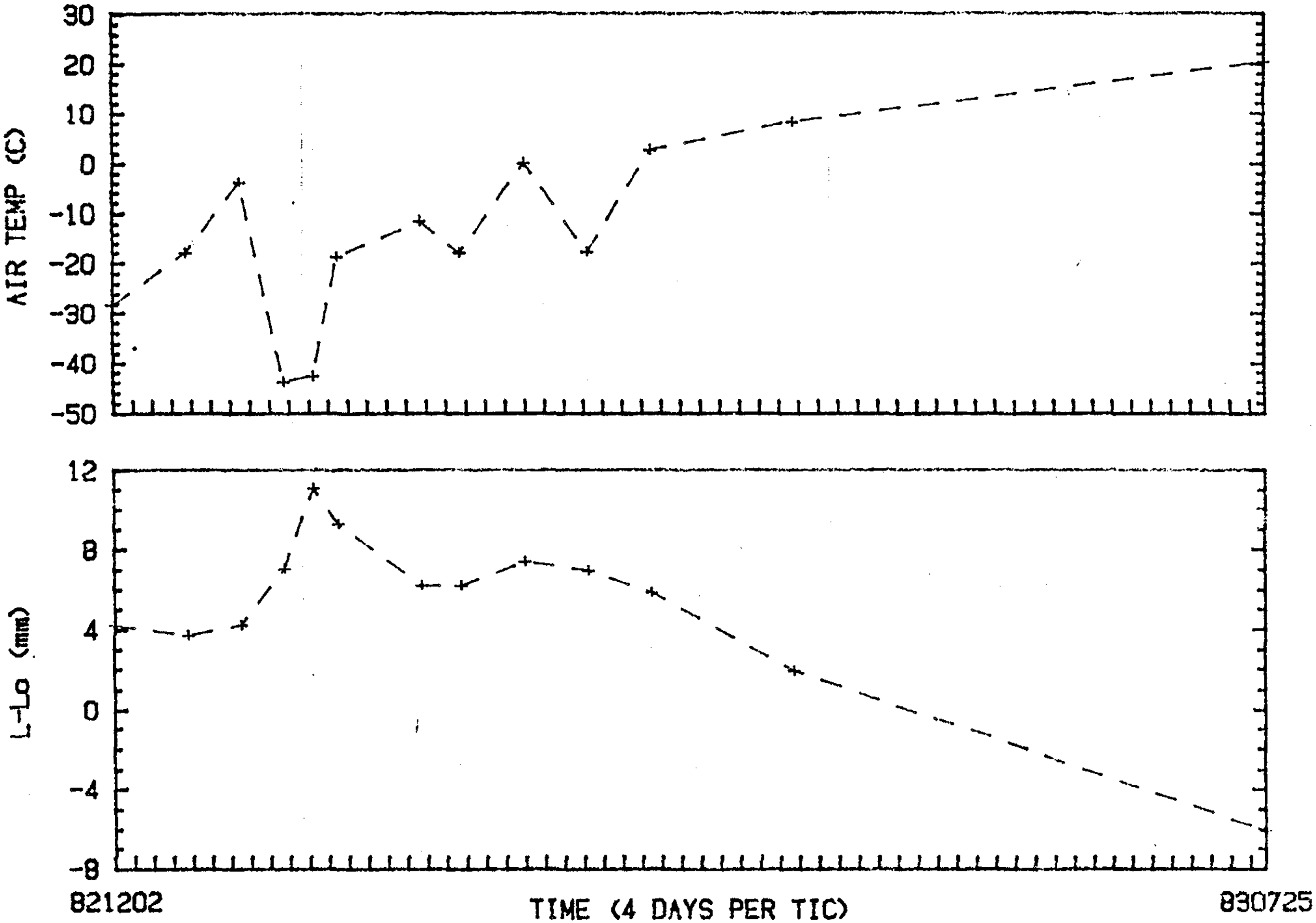
CRACK #3 PARKS HIGHWAY Lo= 200.96mm



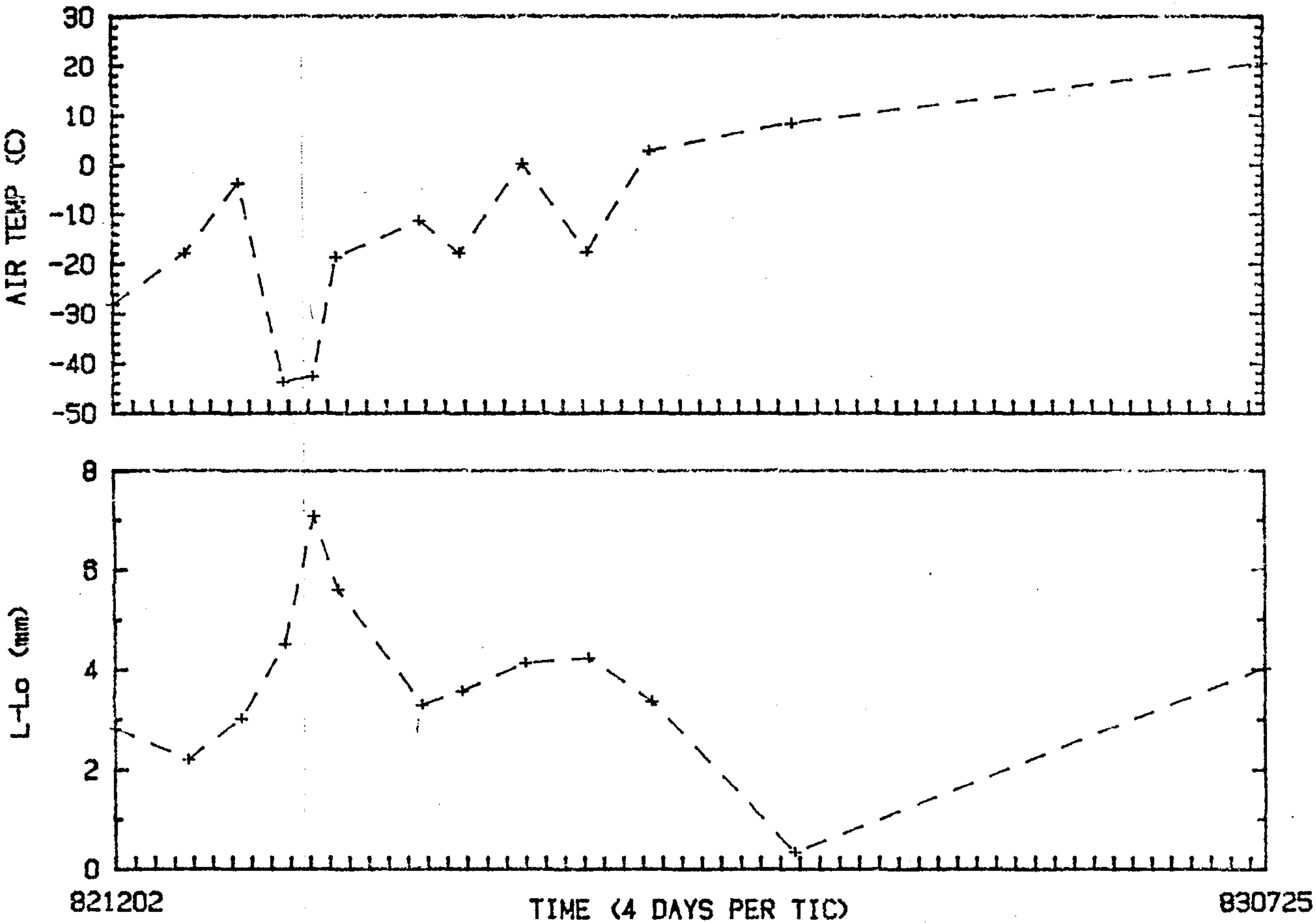
CRACK #4 PARKS HIGHWAY Lo= 189.66 mm



CRACK #5 PARKS HIGHWAY Lo= 226.00 mm



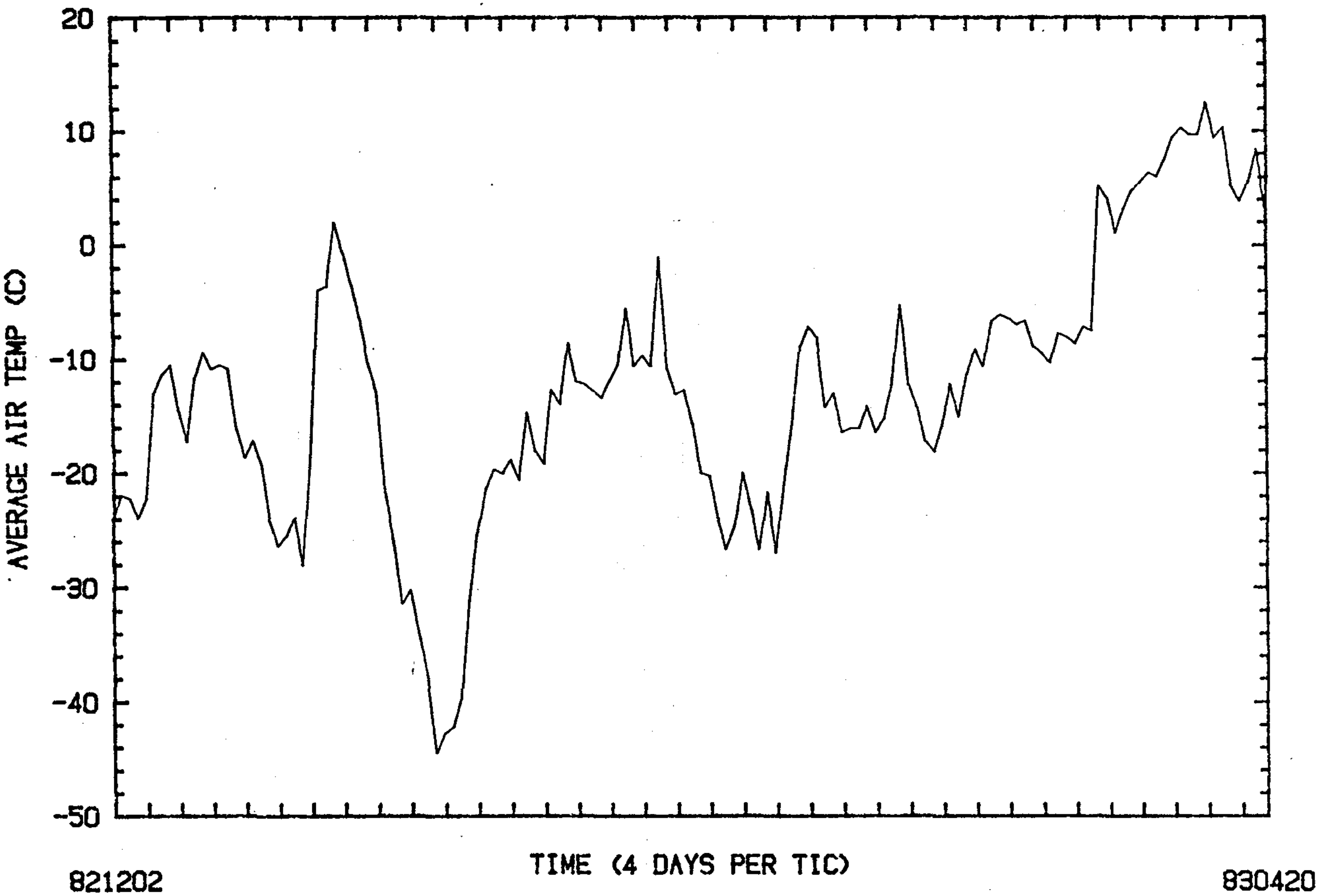
CRACK #8 PARKS HIGHWAY Lo= 215.47 mm



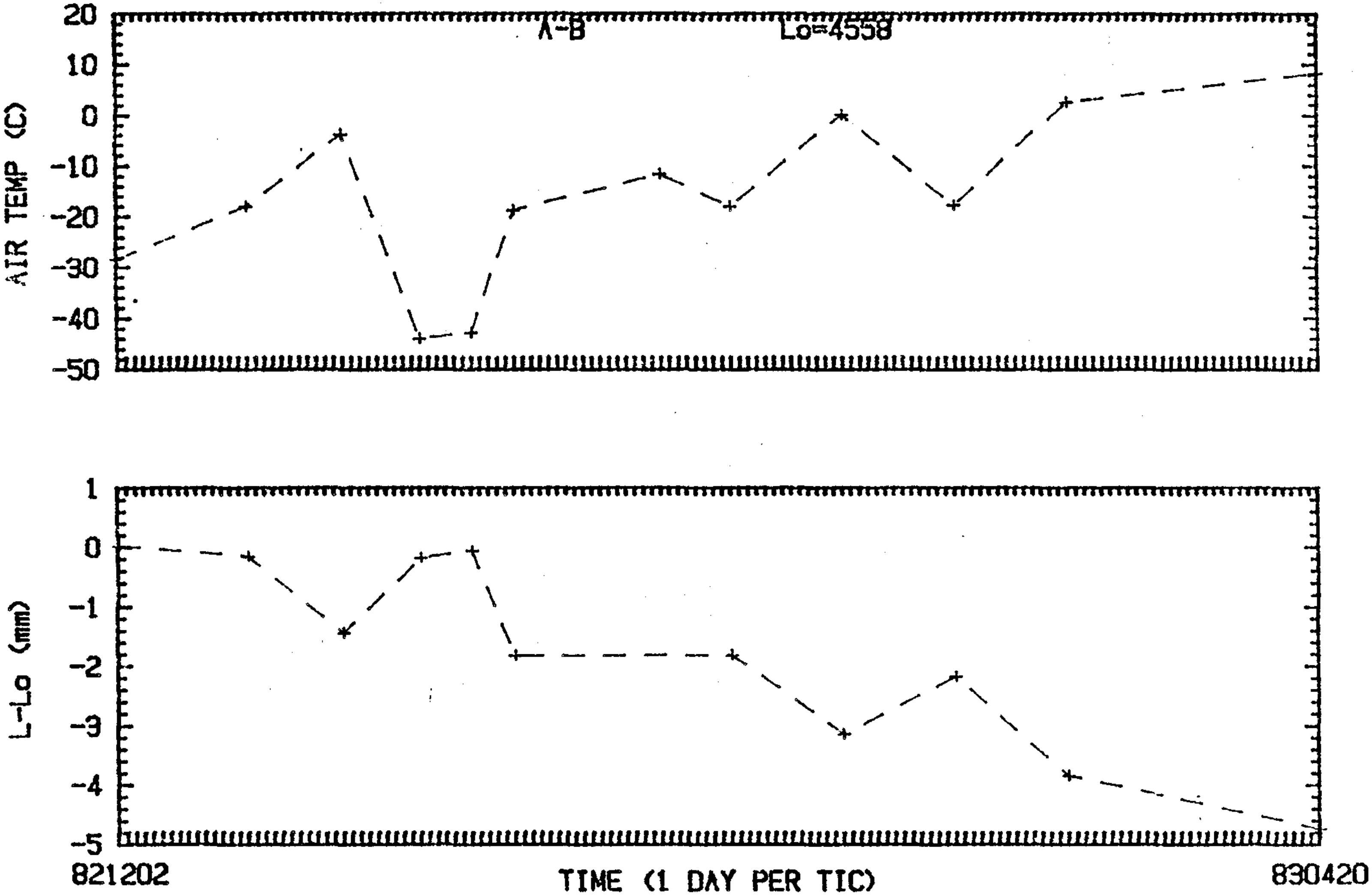
CRACK #7 PARKS HIGHWAY Lo= 208.58 mm

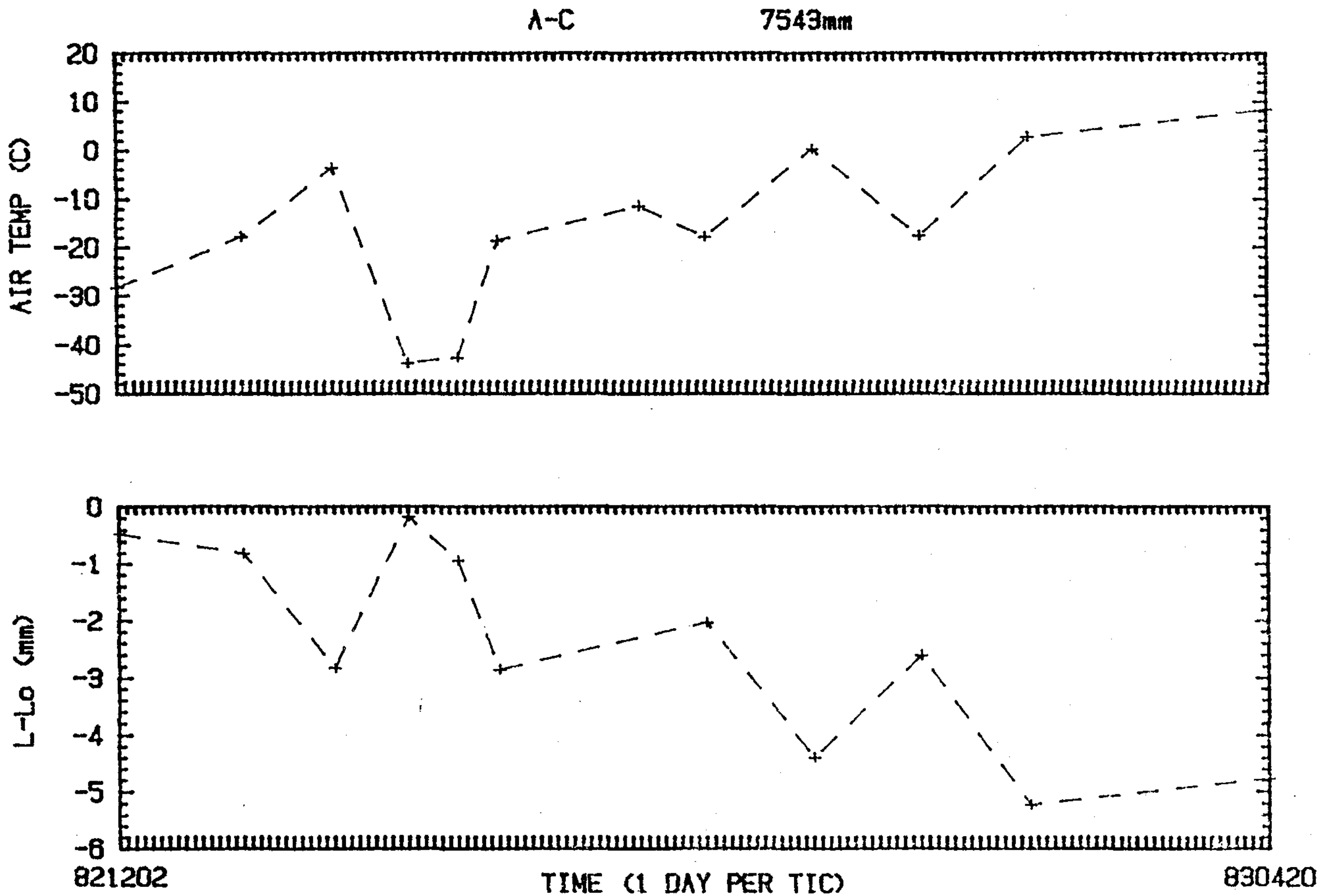
APPENDIX B

Extensometer measurements of pavement and crack movements with time

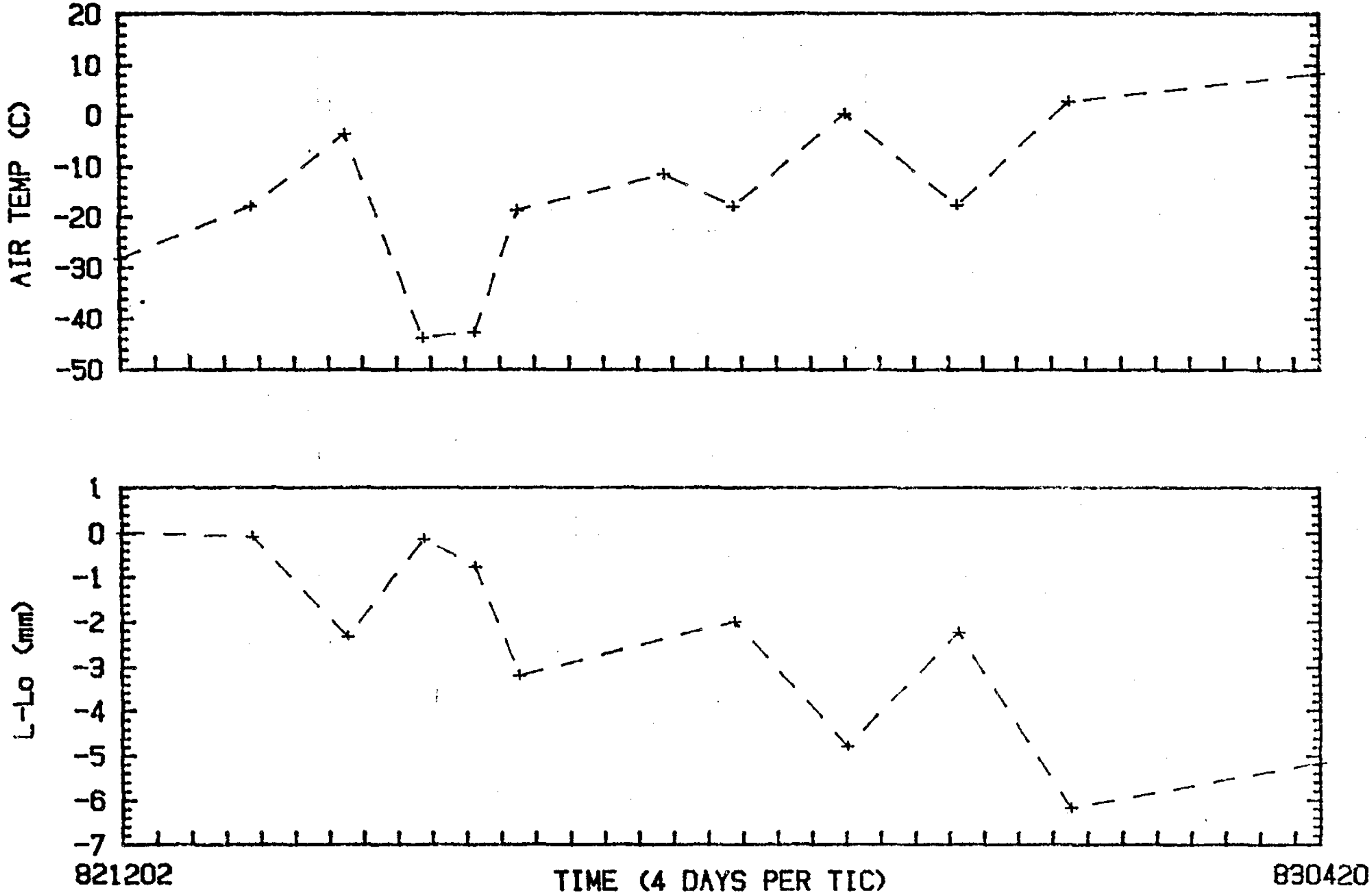


UNIVERSITY EXP STATION

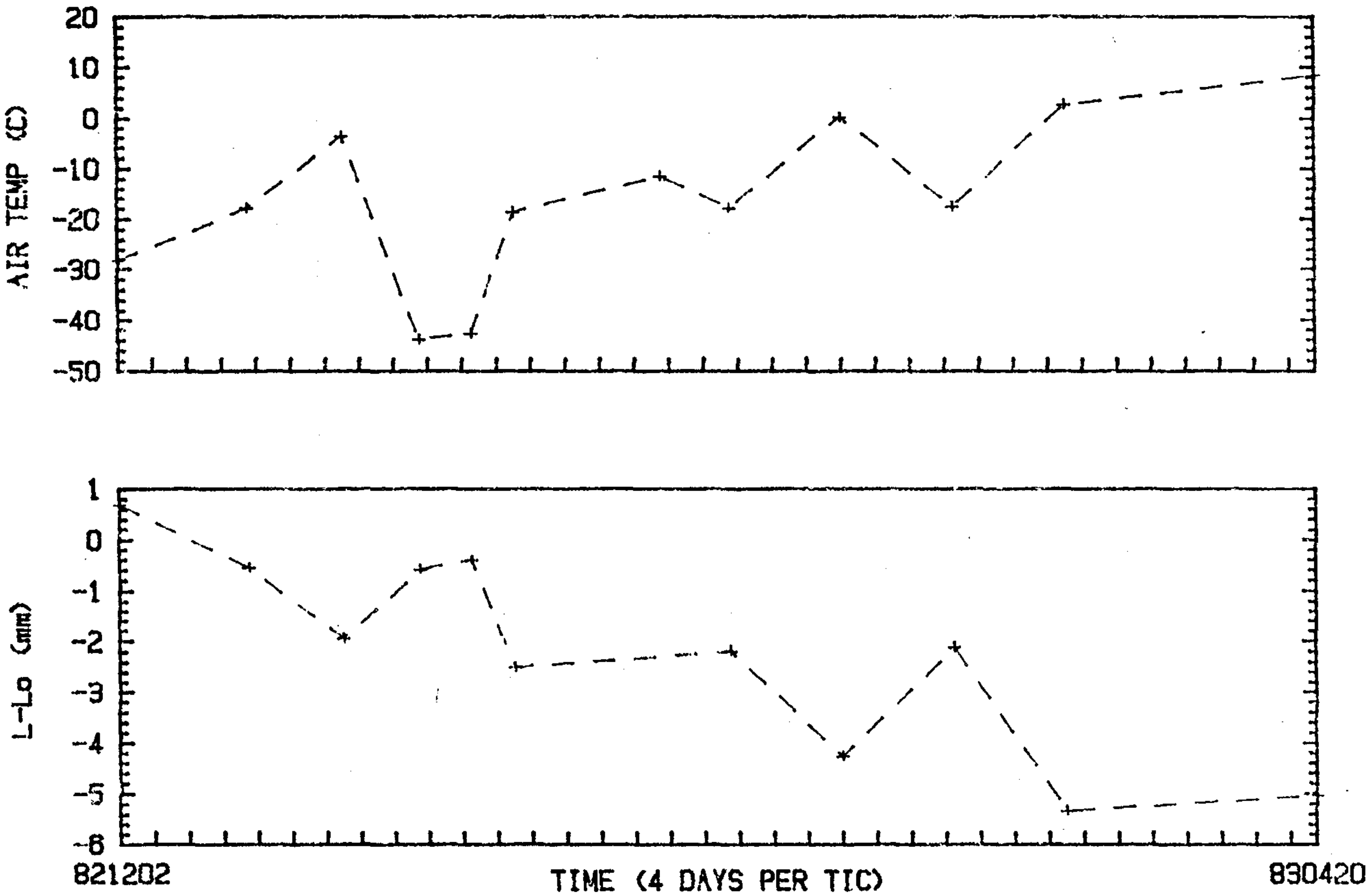

Λ-B
Lo=4558
AIR TEMP (C)
20
10
0
-10
-20
-30
-40
-50
L-Lo (mm)
1
0
-1
-2
-3
-4
-5
821202
TIME (1 DAY PER TIC)
830420



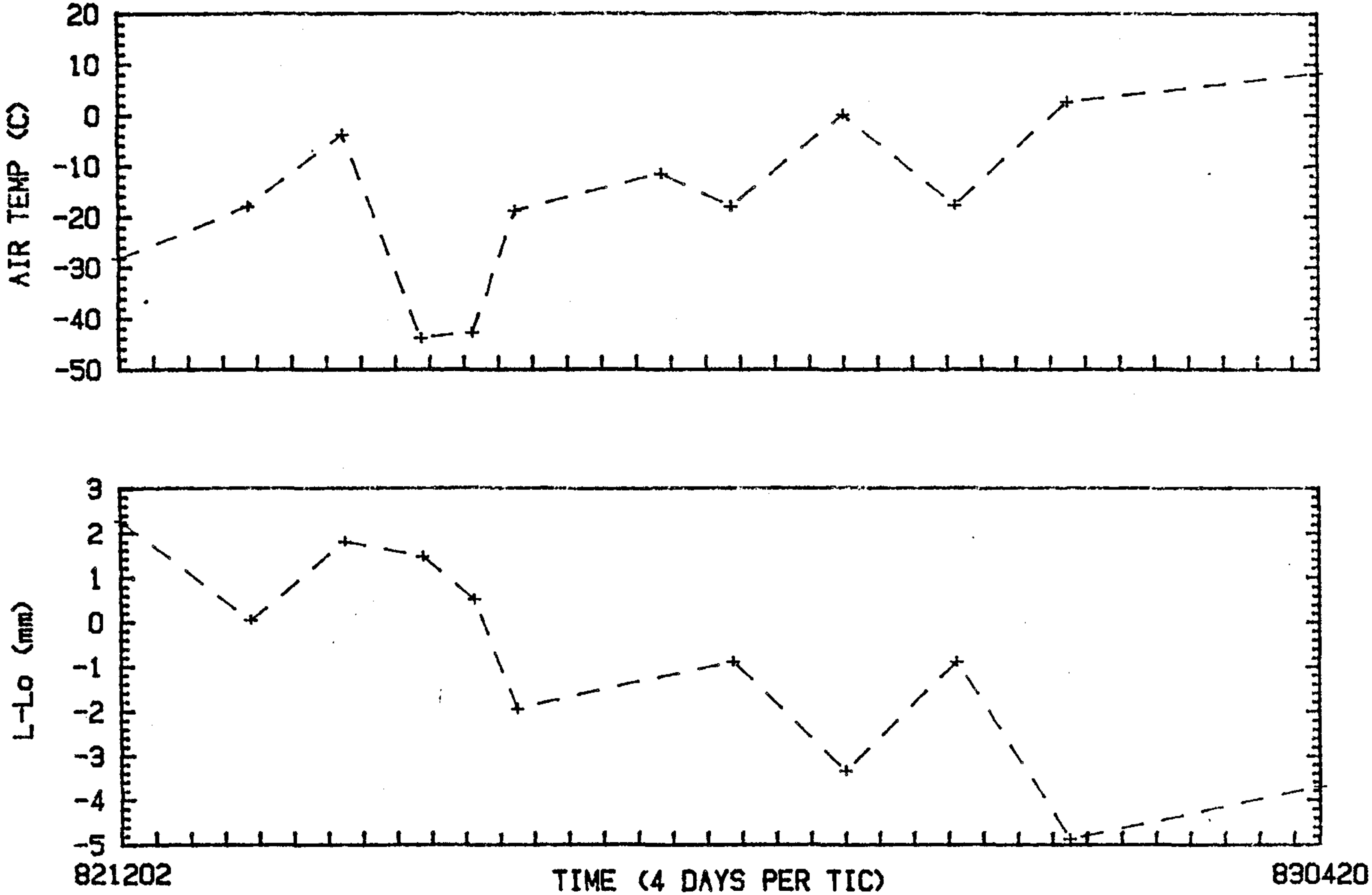
Λ-C
7543mm
AIR TEMP (C)
20
10
0
-10
-20
-30
-40
-50
L-Lo (mm)
0
-1
-2
-3
-4
-5
-6
821202
TIME (1 DAY PER TIC)
830420



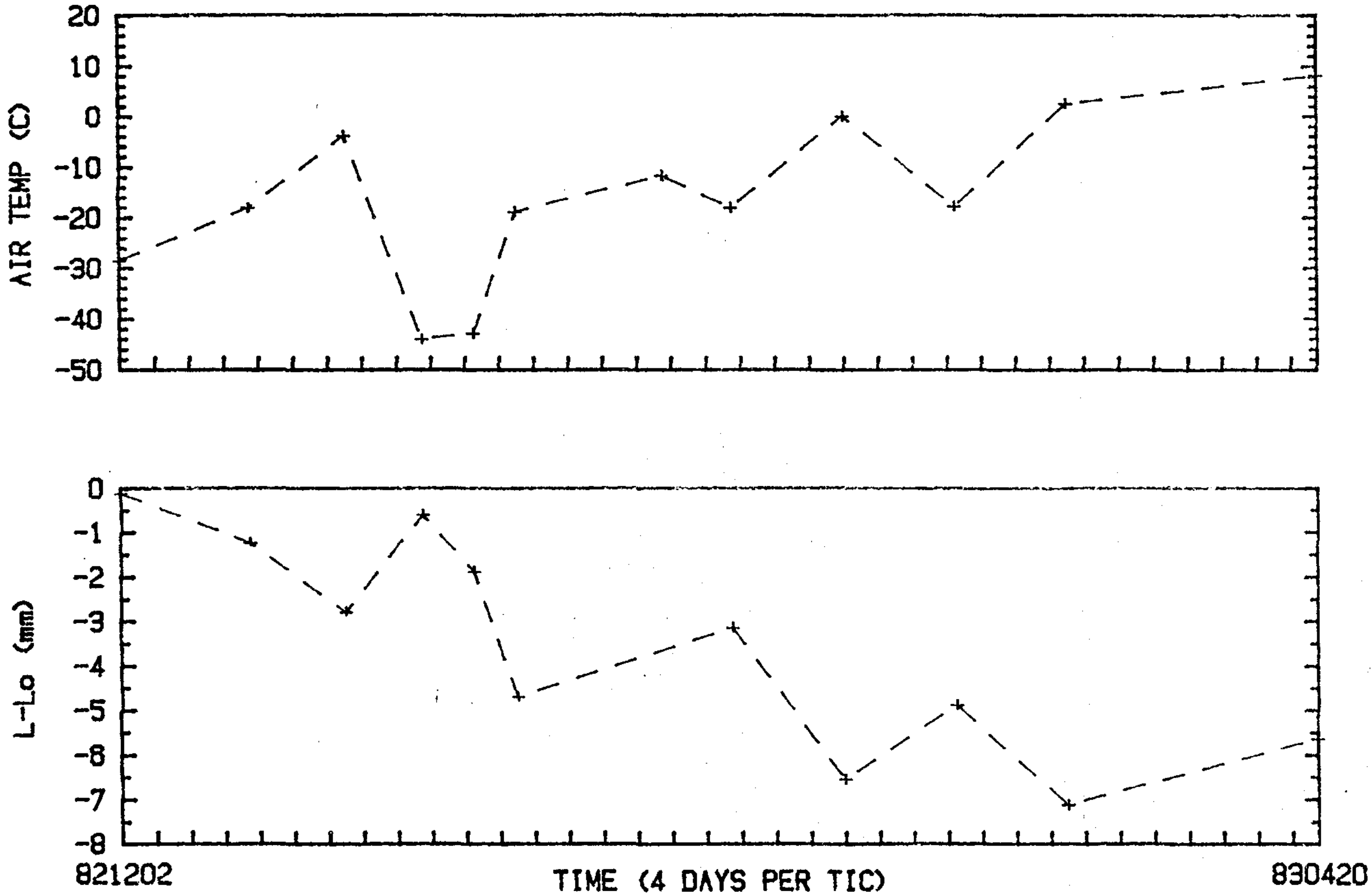
A-D PARKS HIGHWAY 8750



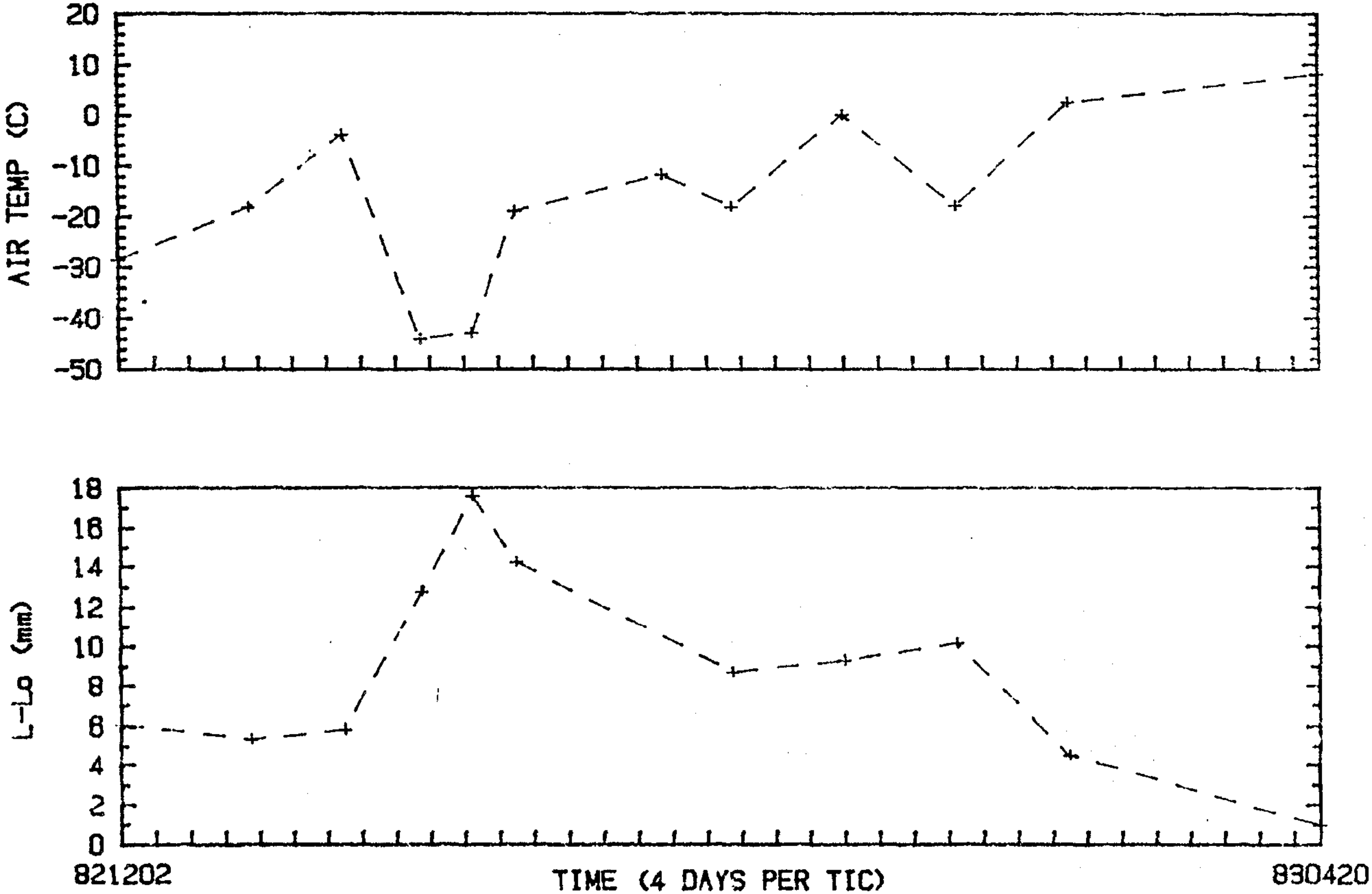
CRACK A-E PARKS HIGHWAY Lo= 9465



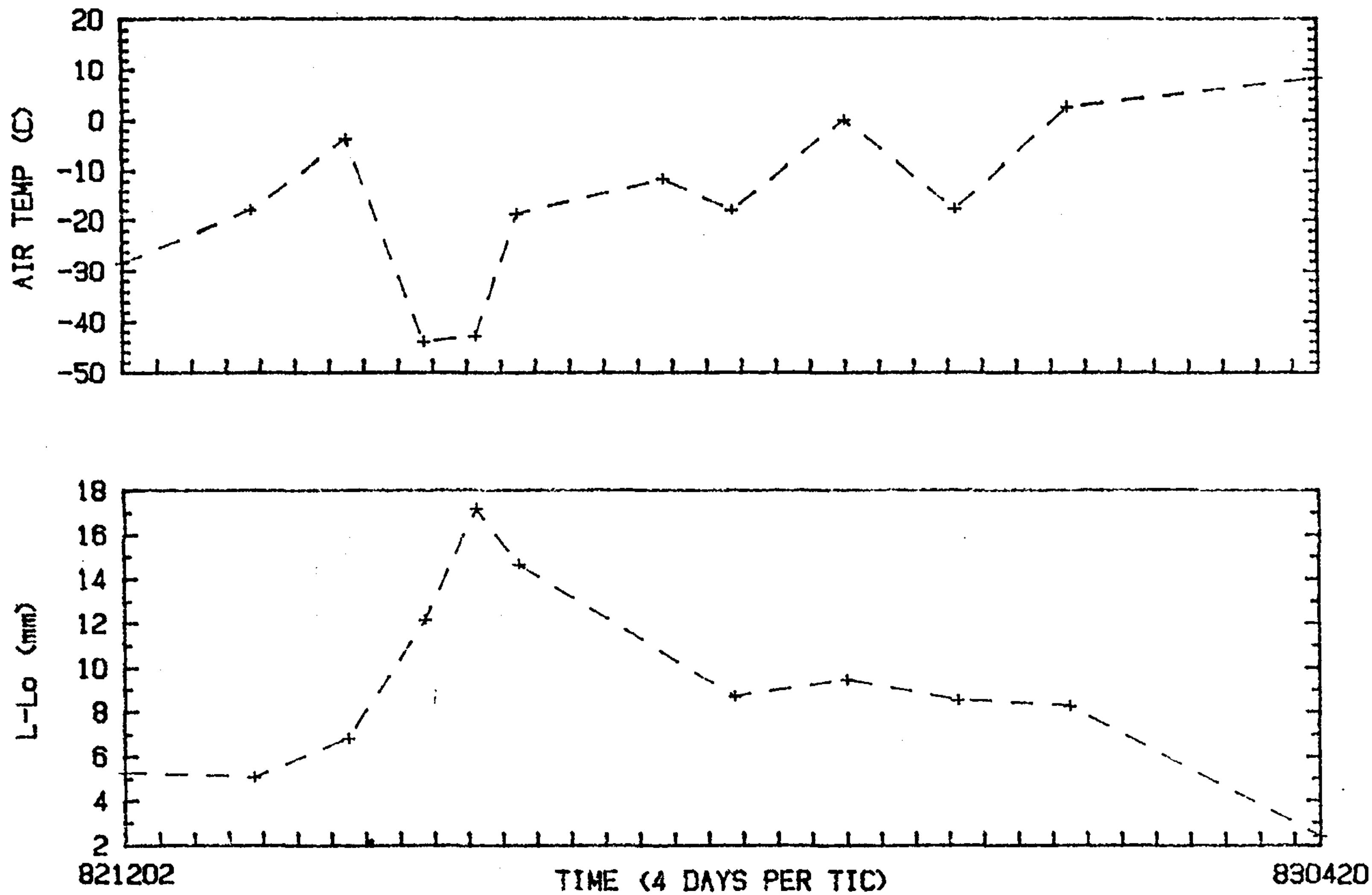
CRACK A-F PARKS HIGHWAY Lo= 10060



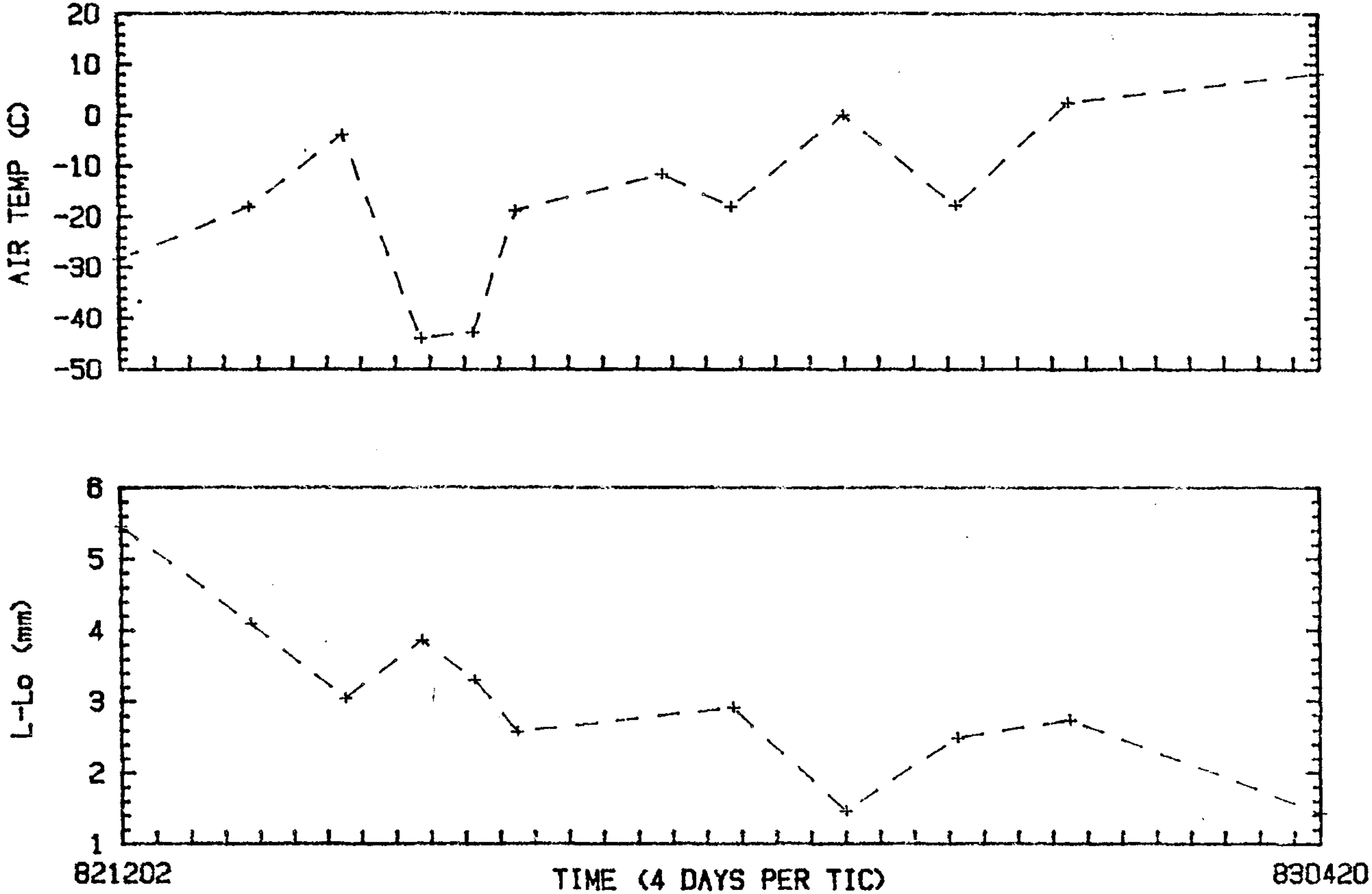
CRACK A-G PARKS HIGHWAY Lo= 10353mm



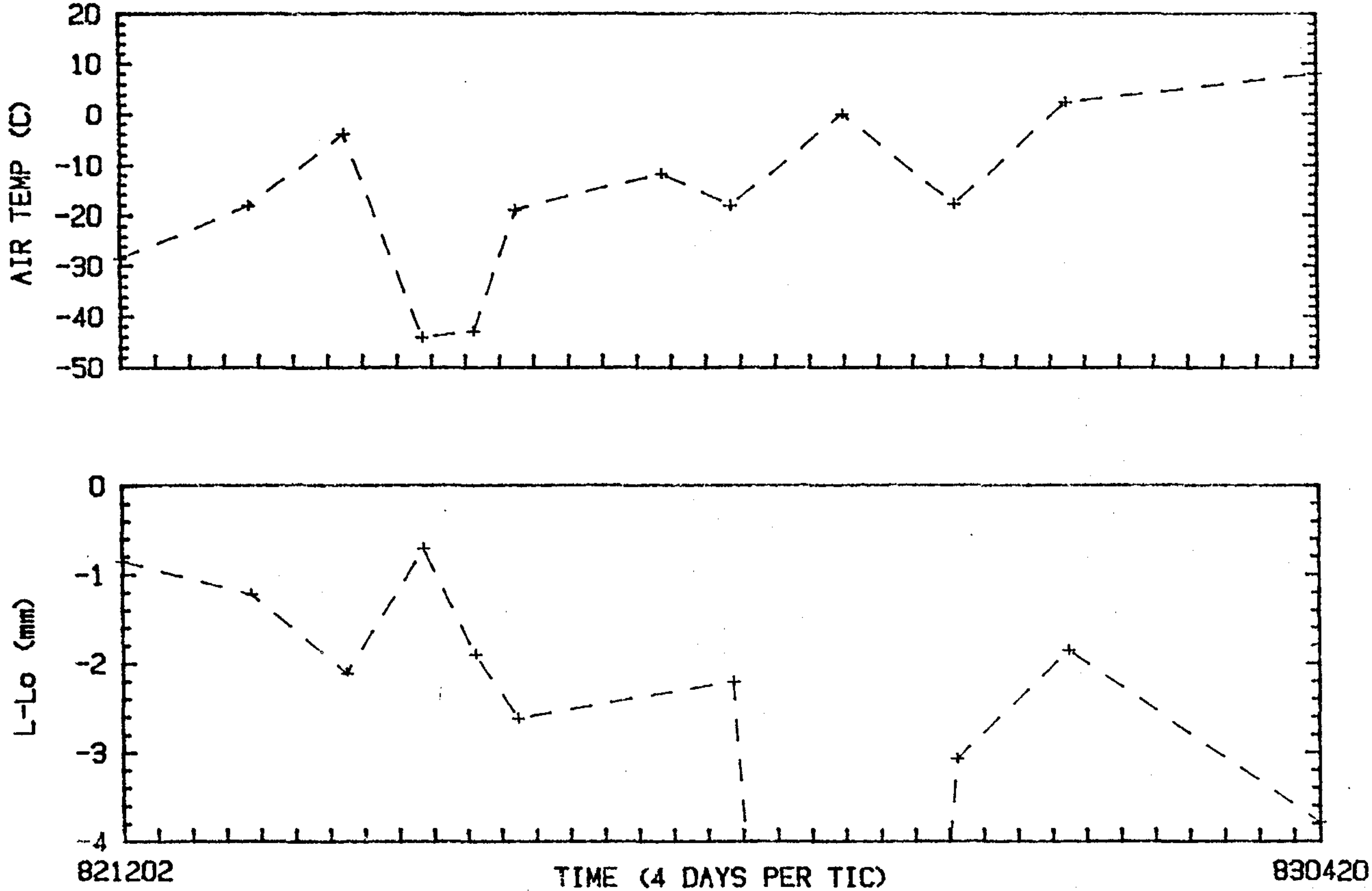
CRACK A-H PARKS HIGHWAY Lo= 10930mm



CRACK J-G PARKS HIGHWAY Lo= 3833mm



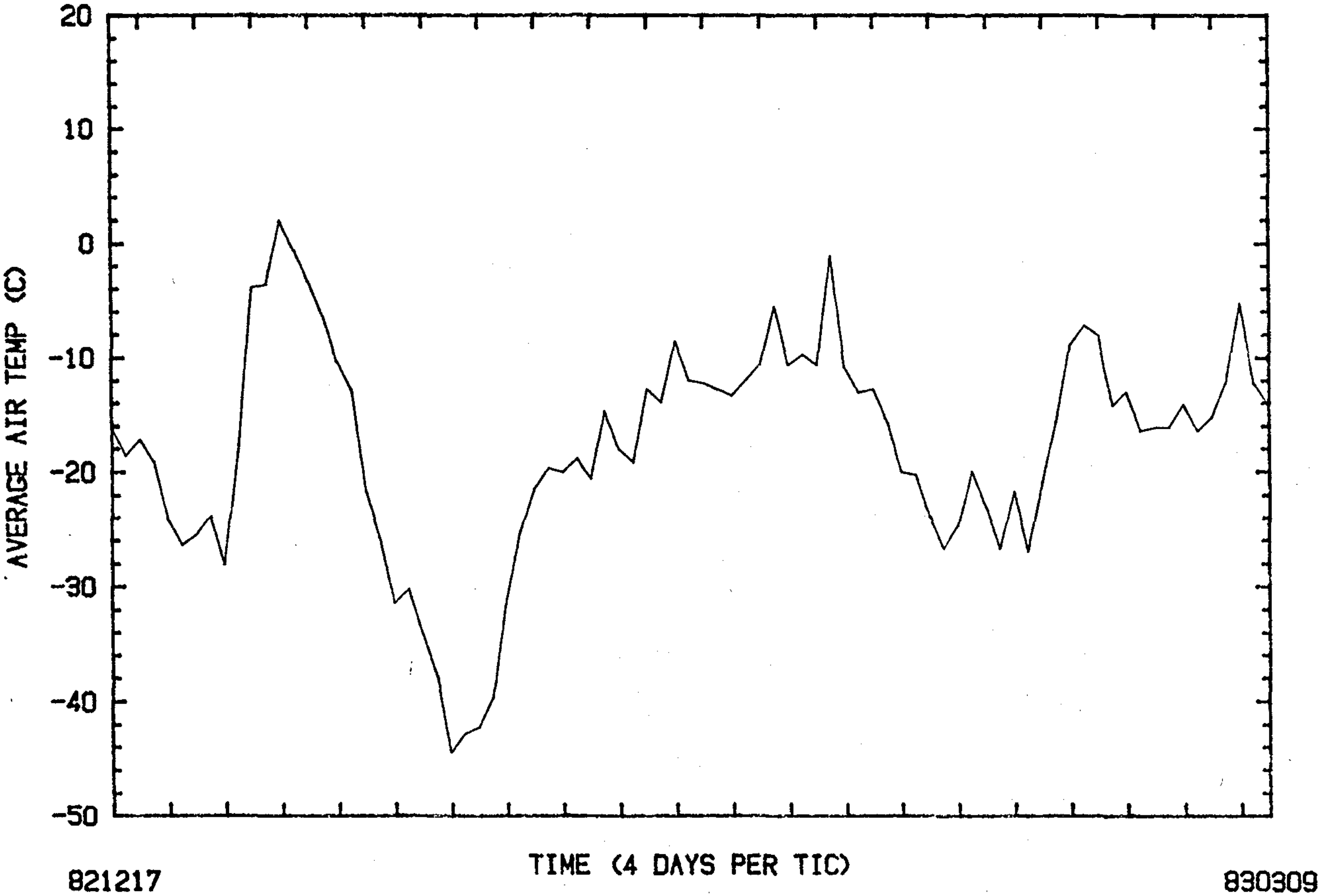
CRACK J-H PARKS HIGHWAY Lo= 3251mm



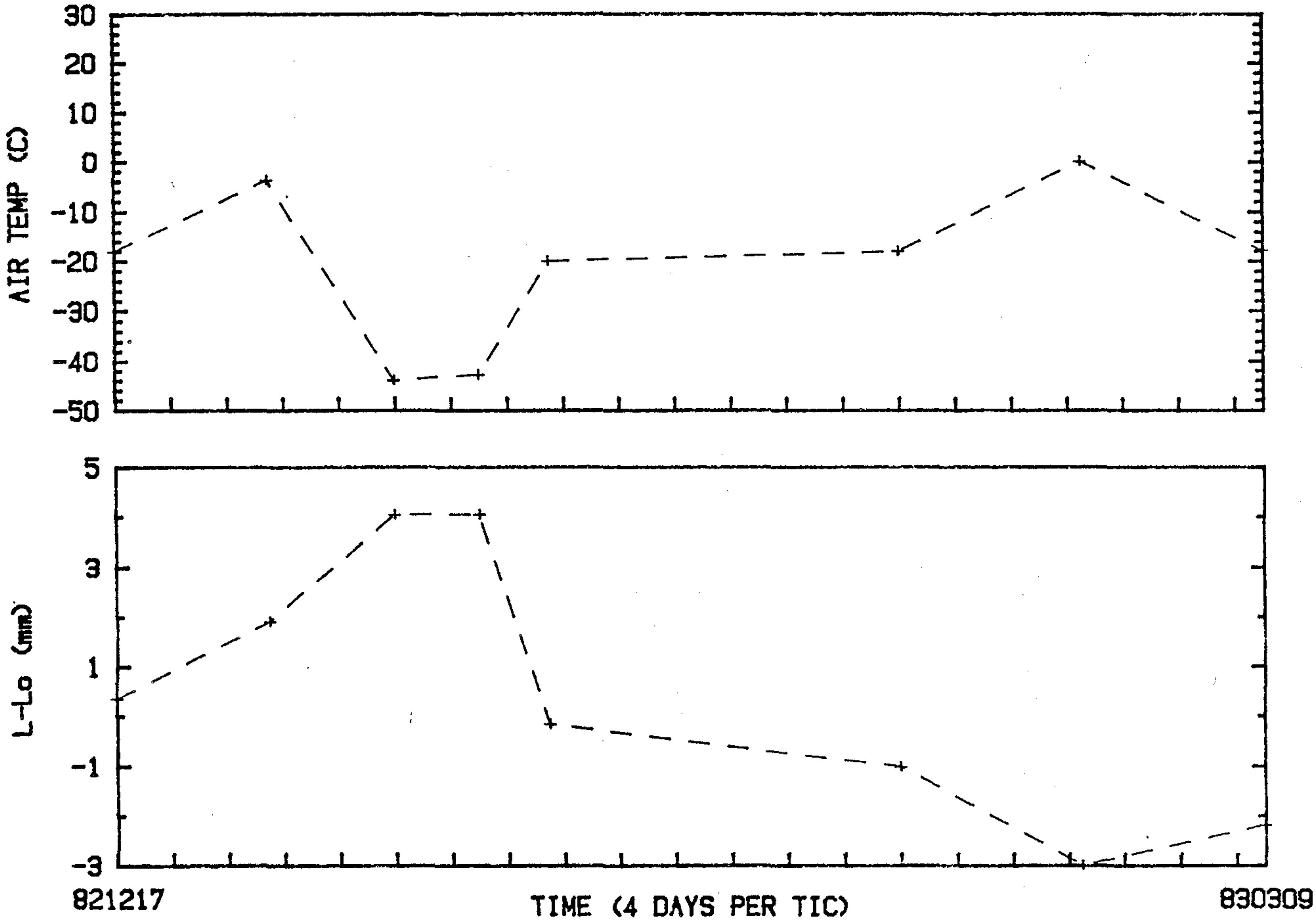
CRACK J-I PARKS HIGHWAY Lo= 2532mm

APPENDIX C

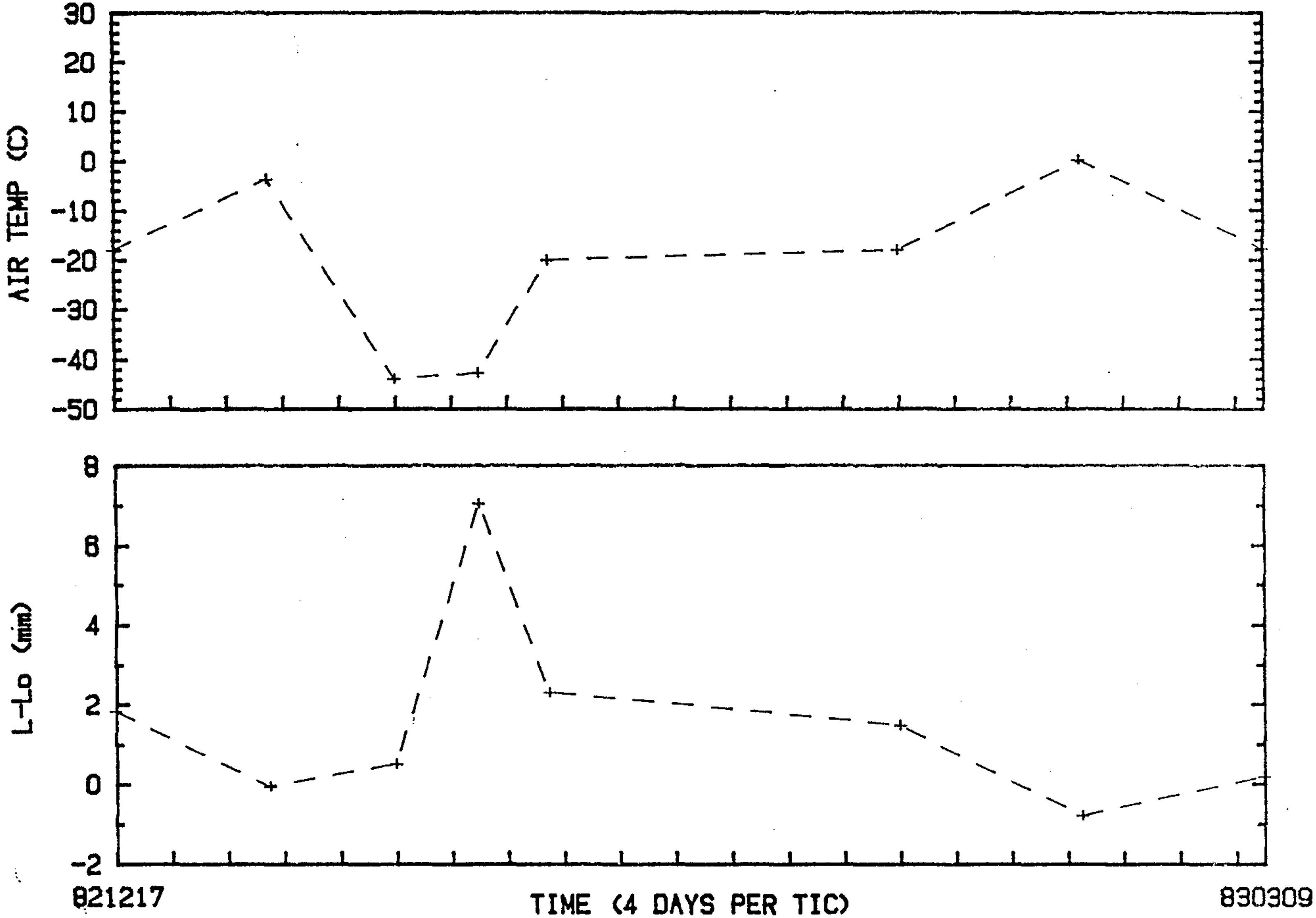
Extensometer measurements in fine-grained permafrost



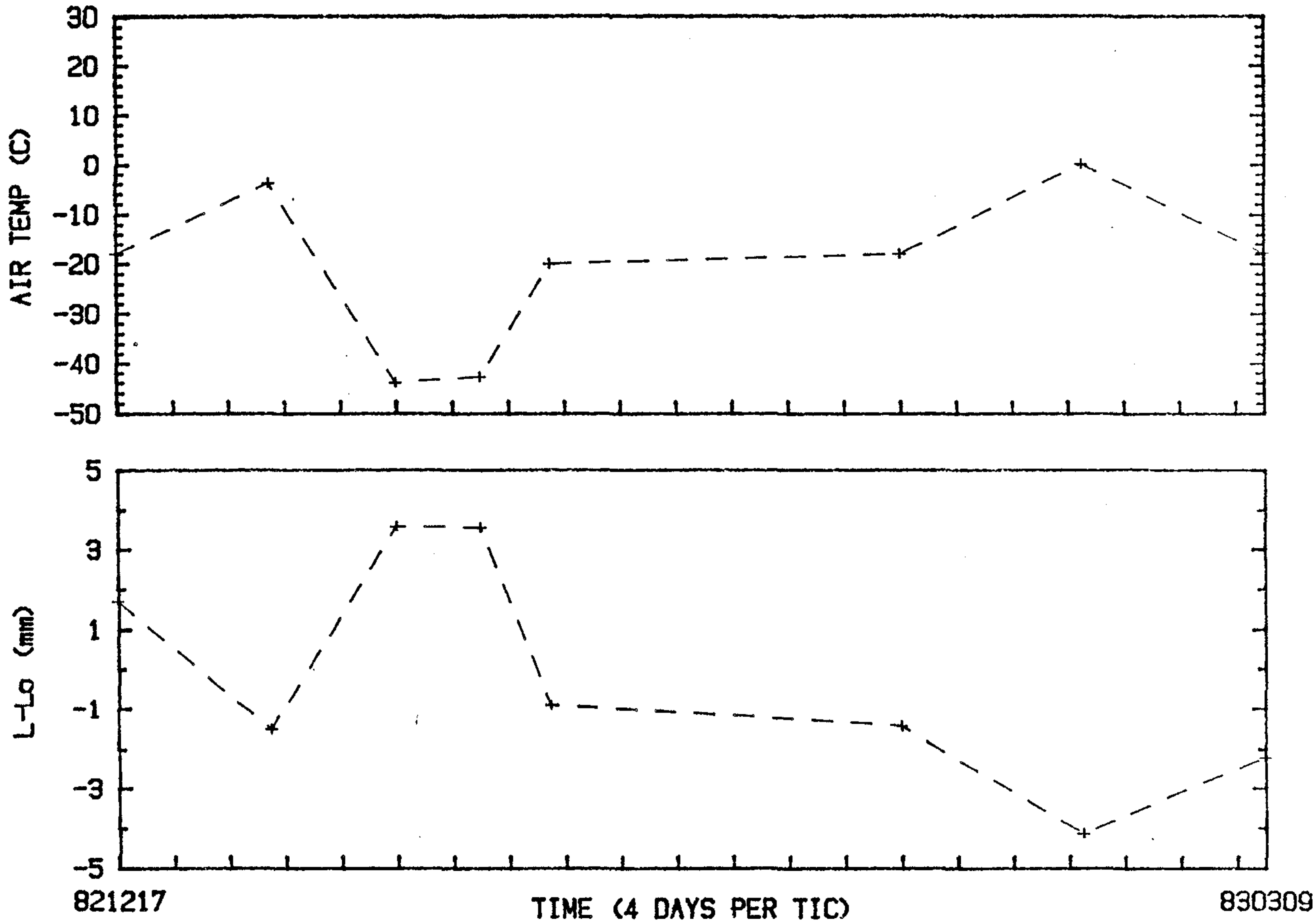
UNIVERSITY EXP STATION



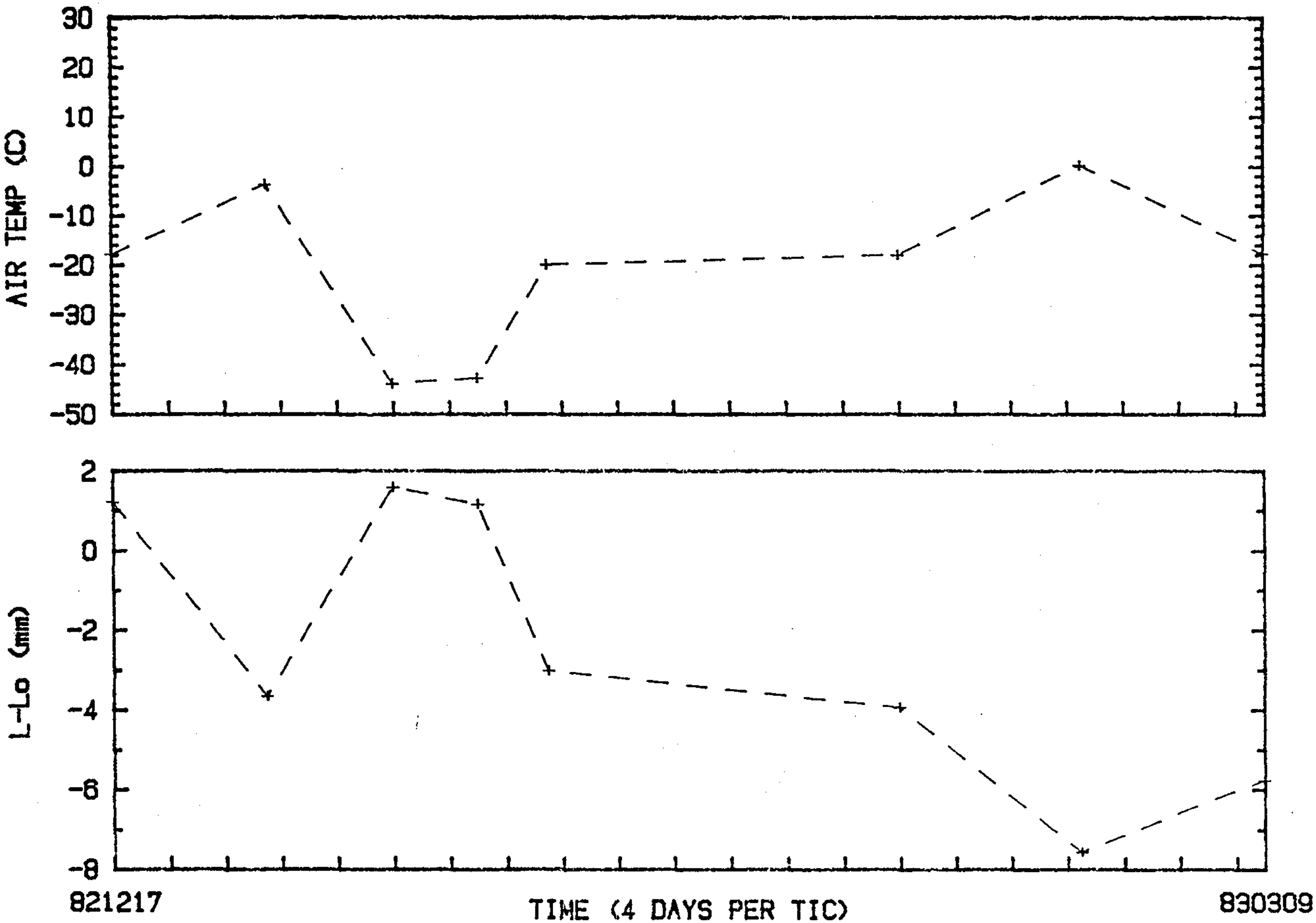
MEASUREMENT N-S SHALLOW SHEEP CREEK SITE Lo= 16.08776 m



MEASUREMENT N-S DEEP SHEEP CREEK SITE Lo= 17.58036 m



MEASUREMENT E-W SHALLOW   SHEEP CREEK SITE   Lo= 16.57041 m

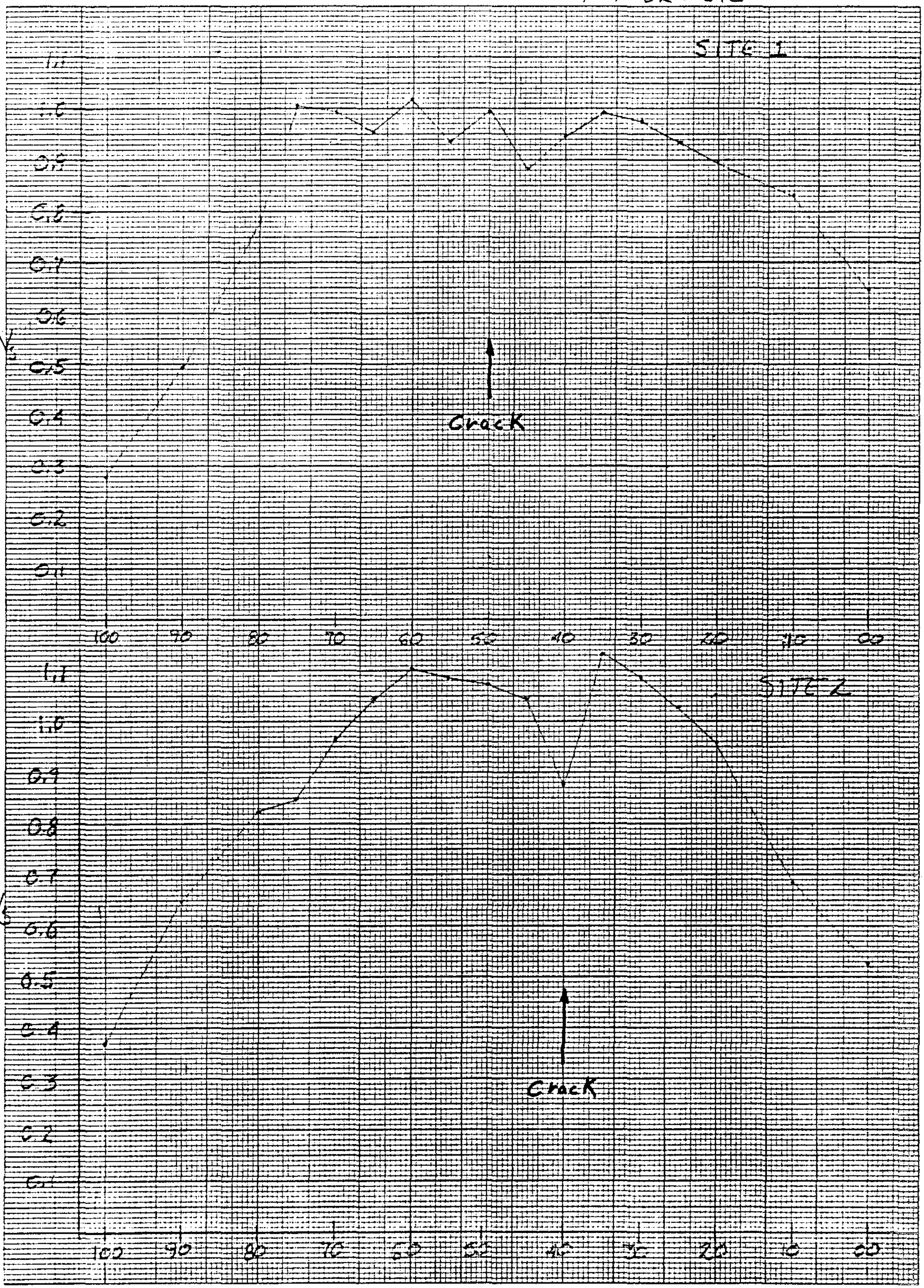


MEASUREMENT E-W DEEP SHEEP CREEK SITE Lo= 17.42282

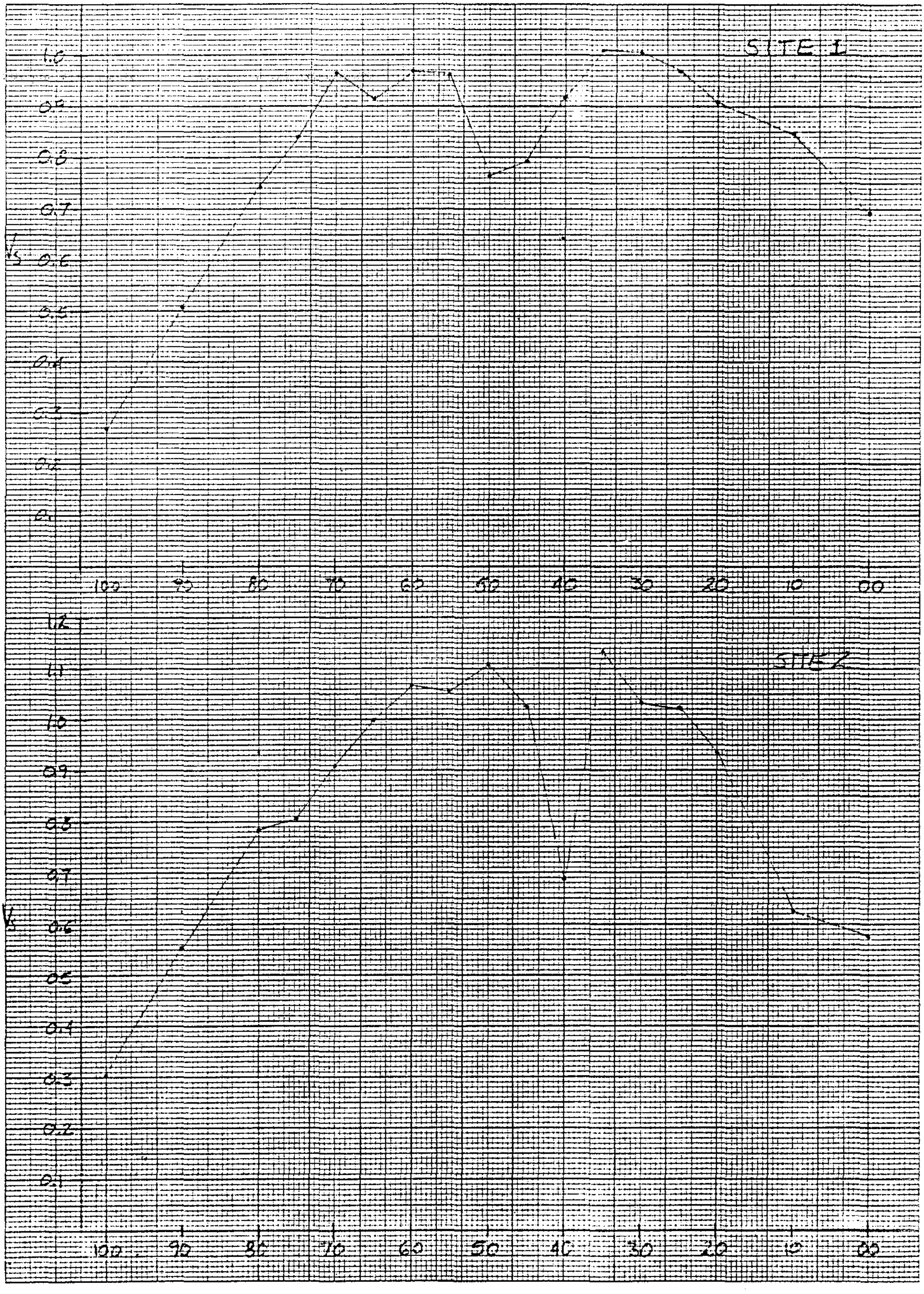
APPENDIX D

Measurements of the vertical pavement displacements near cracks

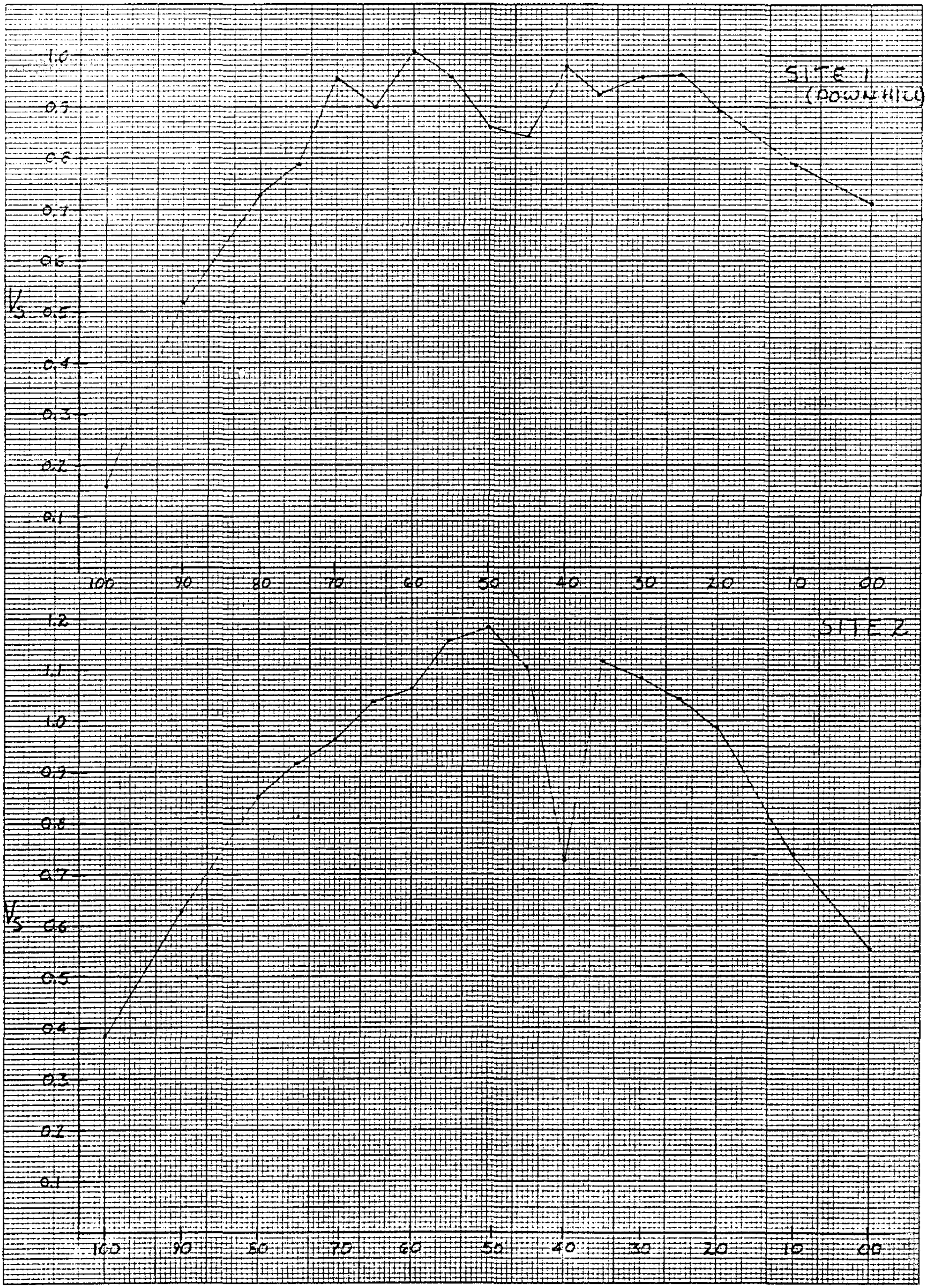
VERTICAL CRACK MEASUREMENTS
FOR 820216



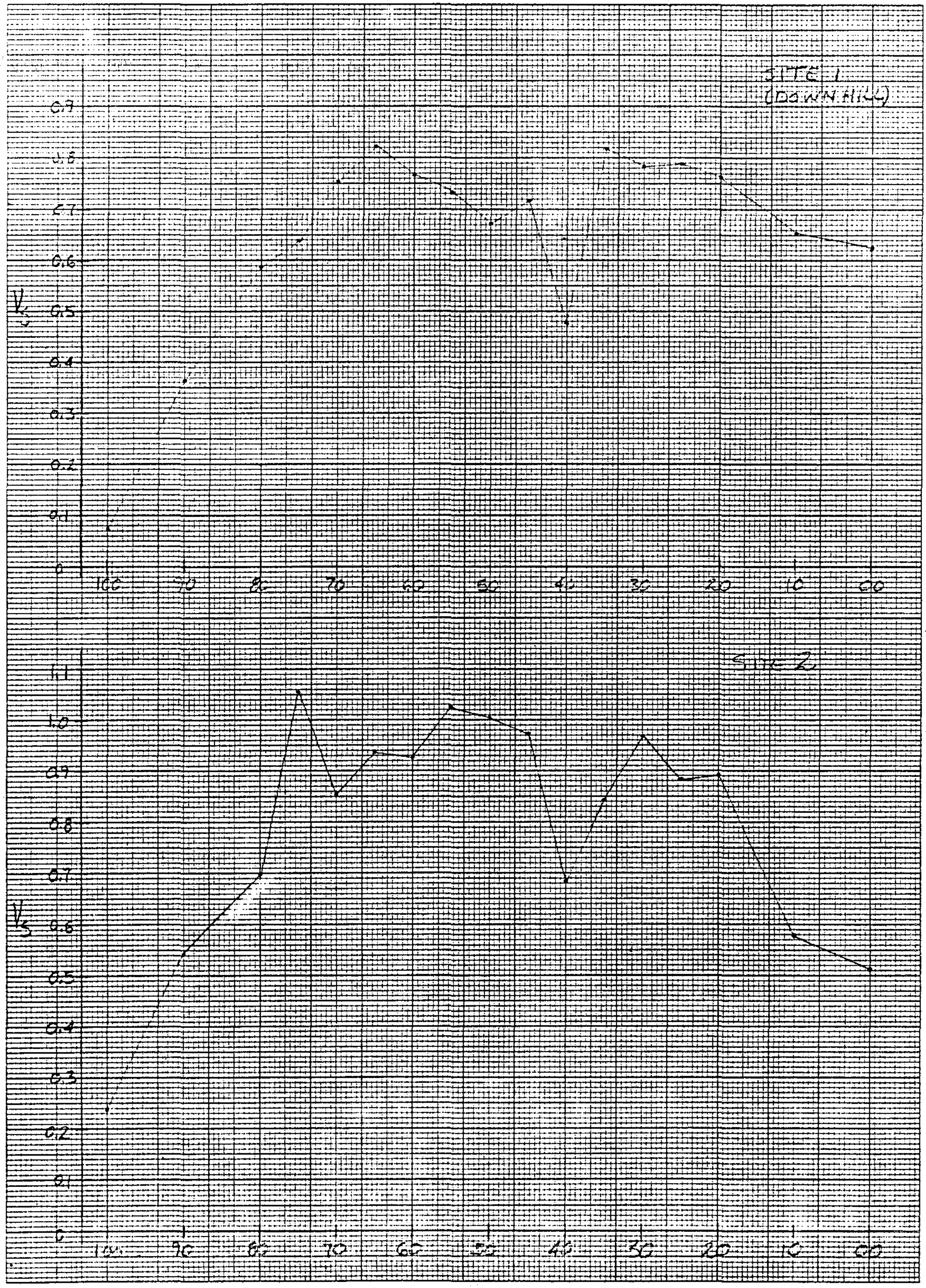
VERTICAL CRACK MEASUR
FOR 820223



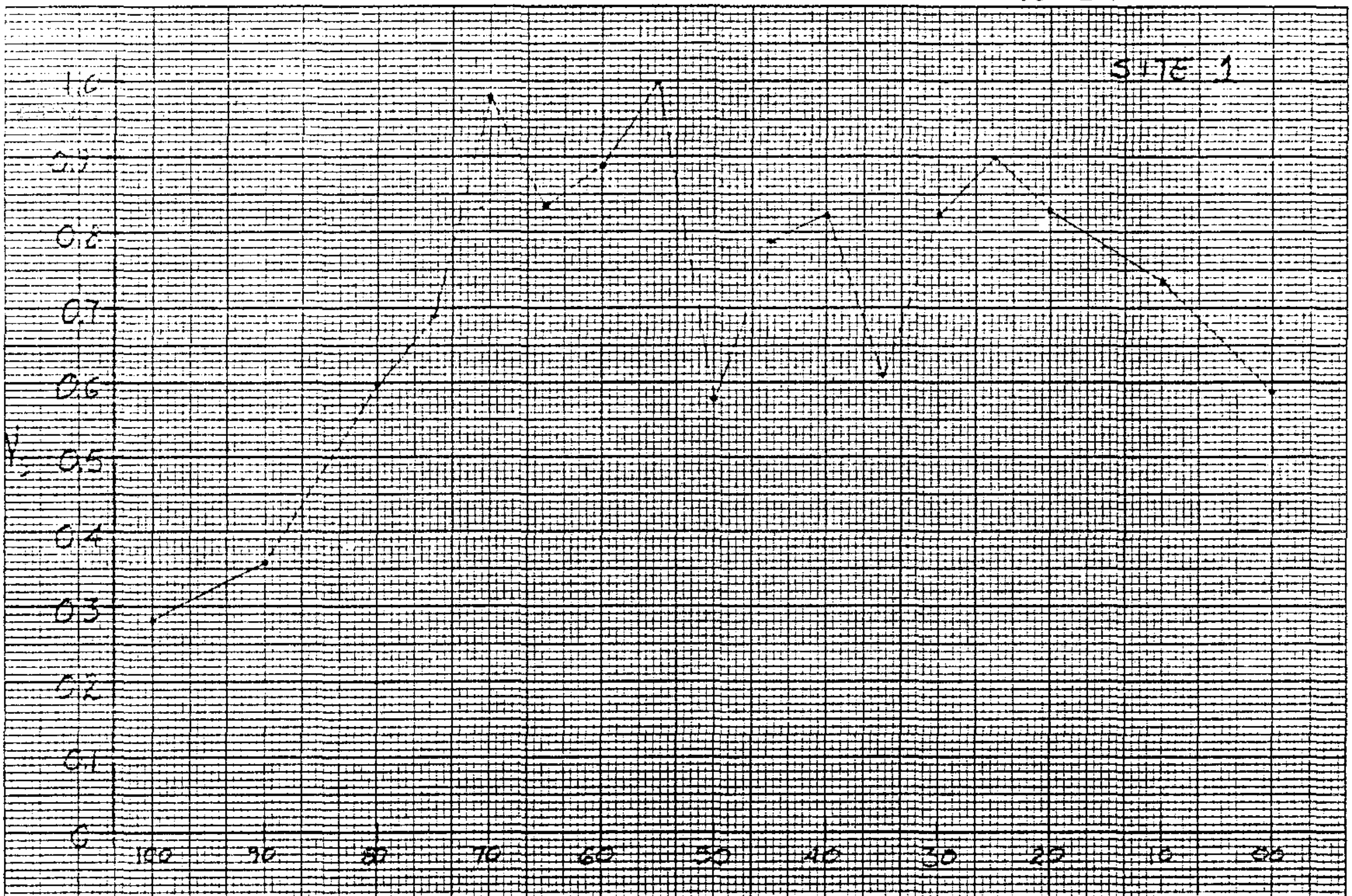
VERTICAL CRACK MEA.
820302

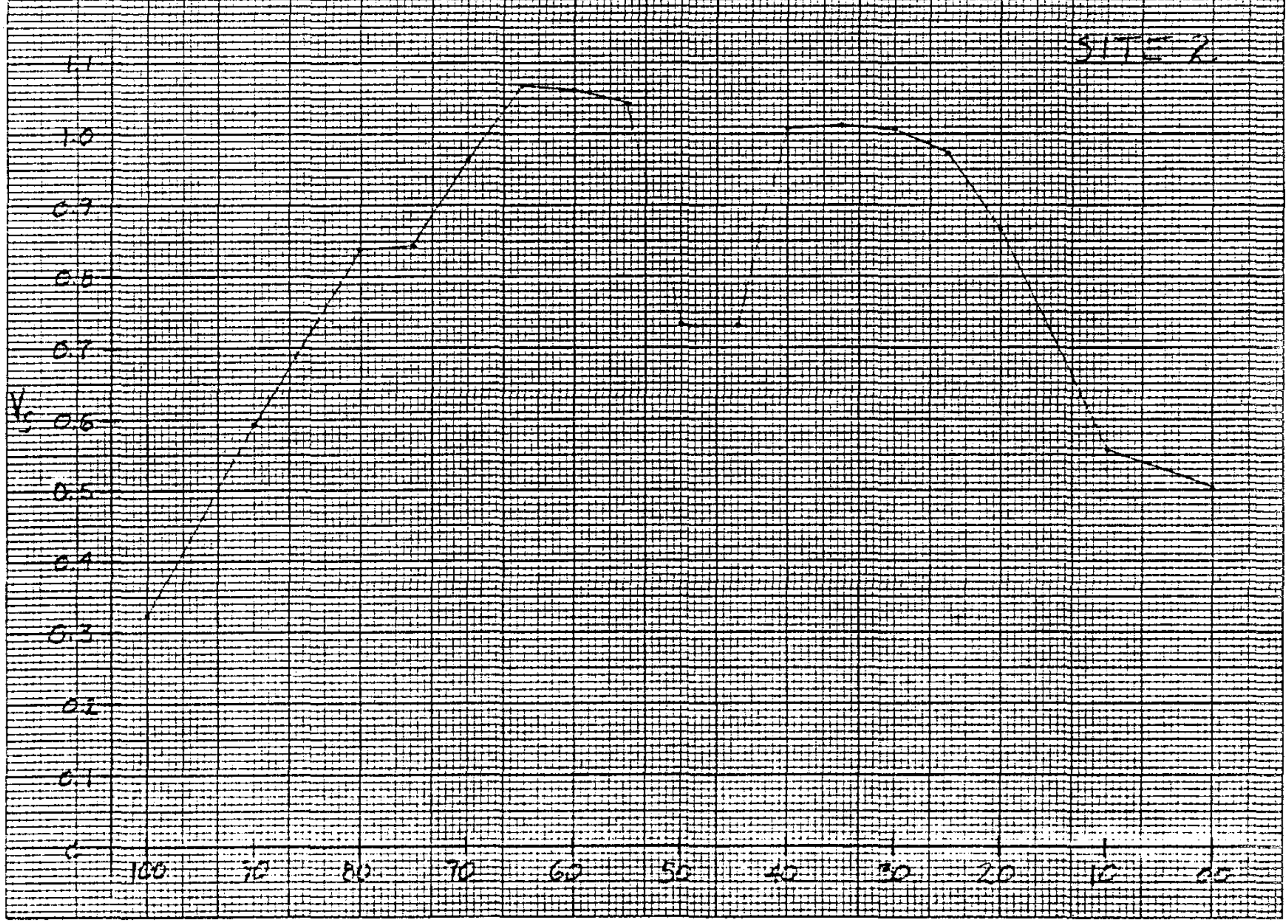


VERT. CRACK MEA.
82 03 09

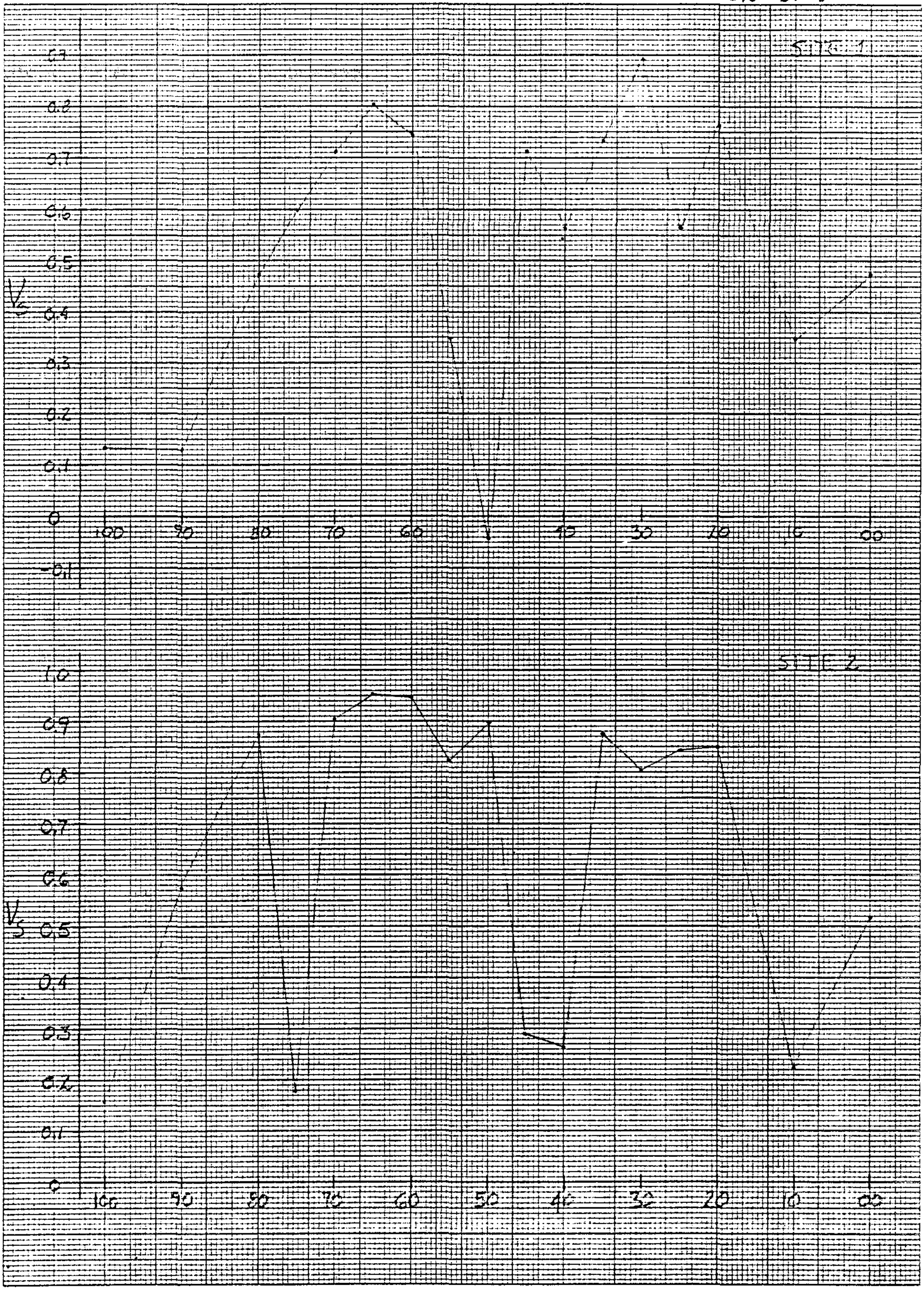


VERTICAL CRACK MEA
820316

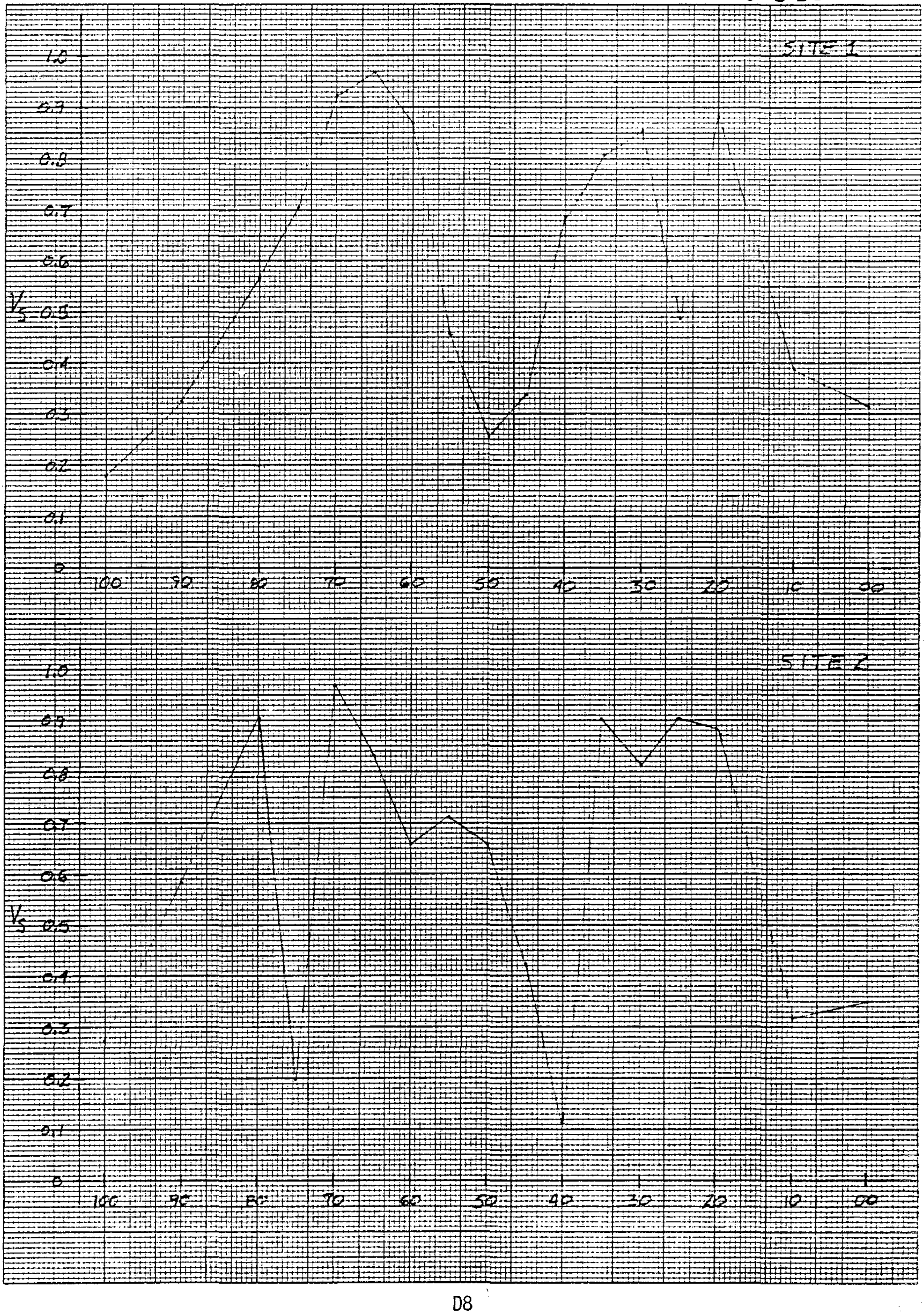




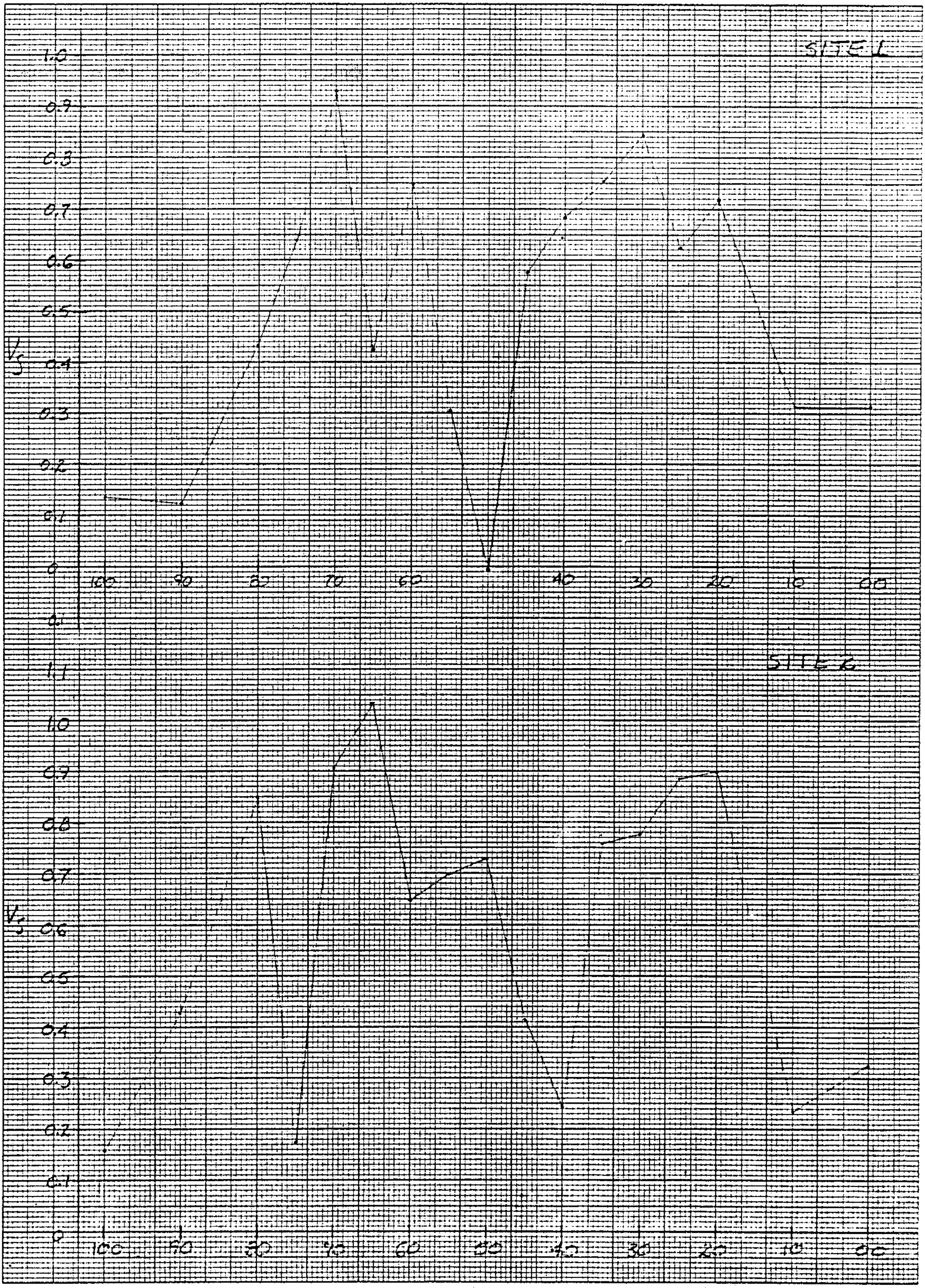
VERTICAL CRACK M
82 0323

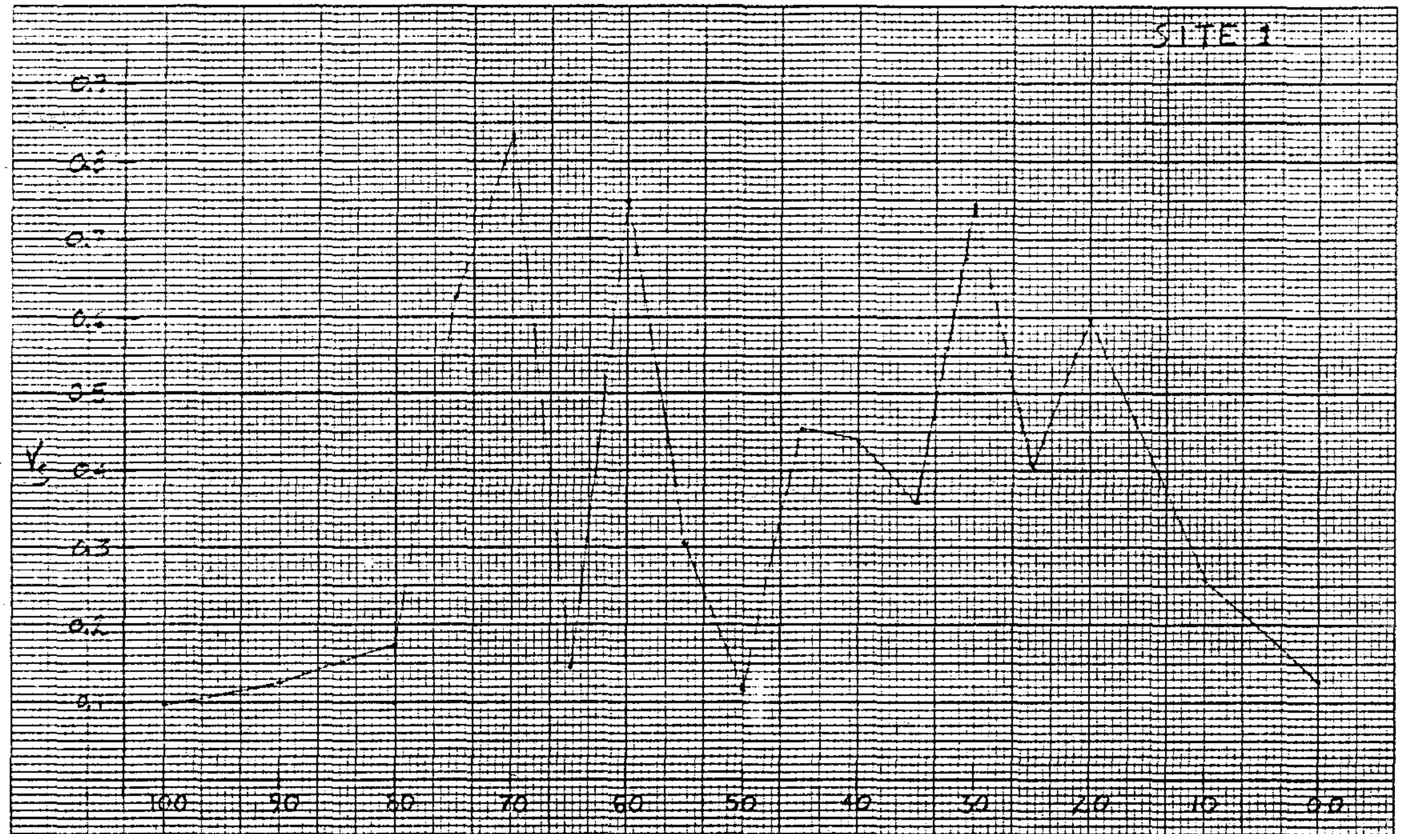


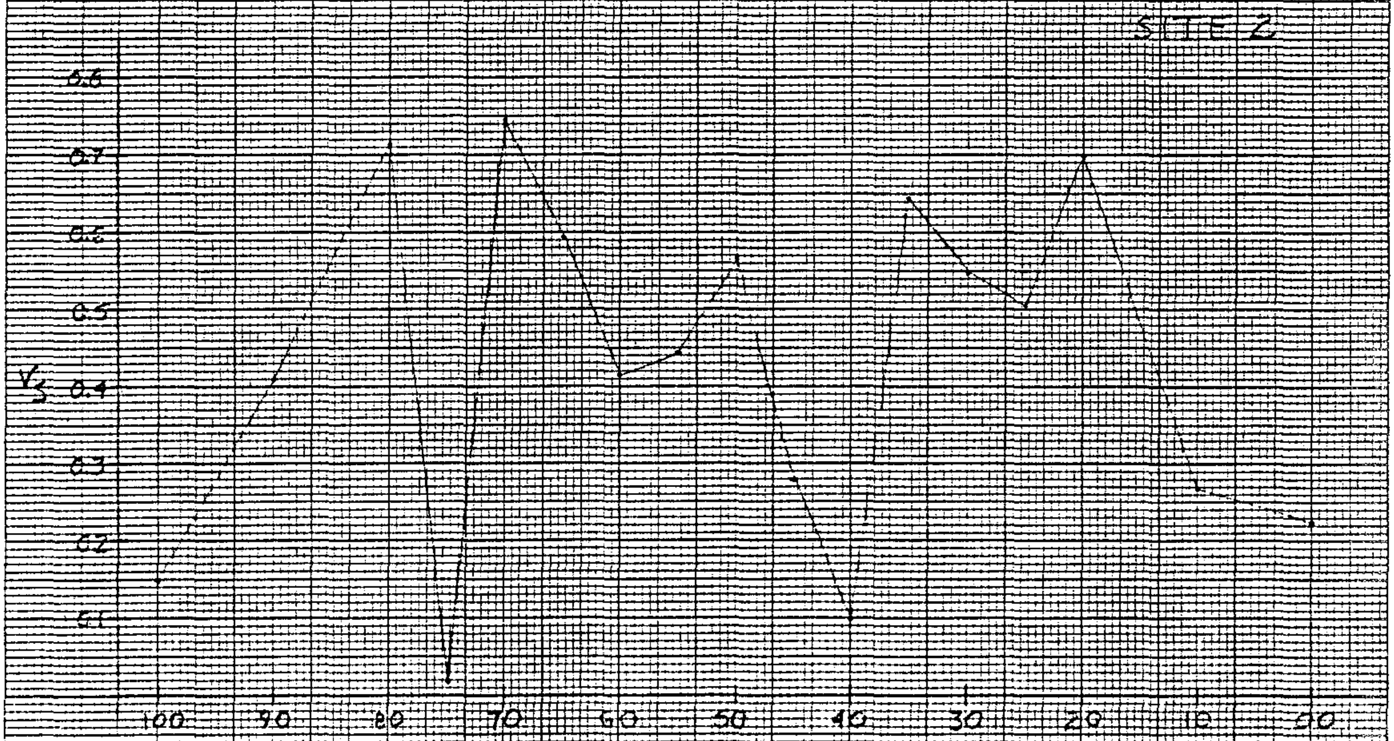
VERTICAL CRACK MEA.
820330



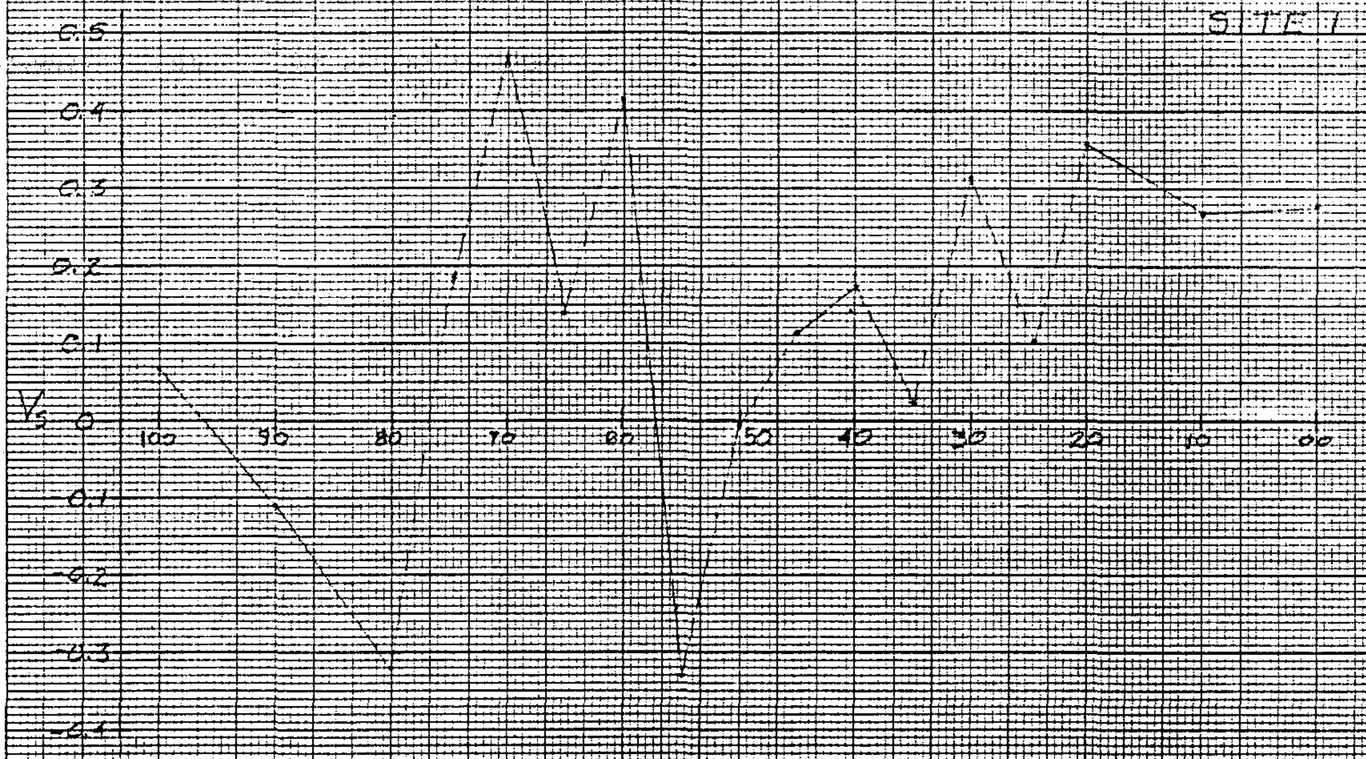
VERTICAL CRACK M
82 0406

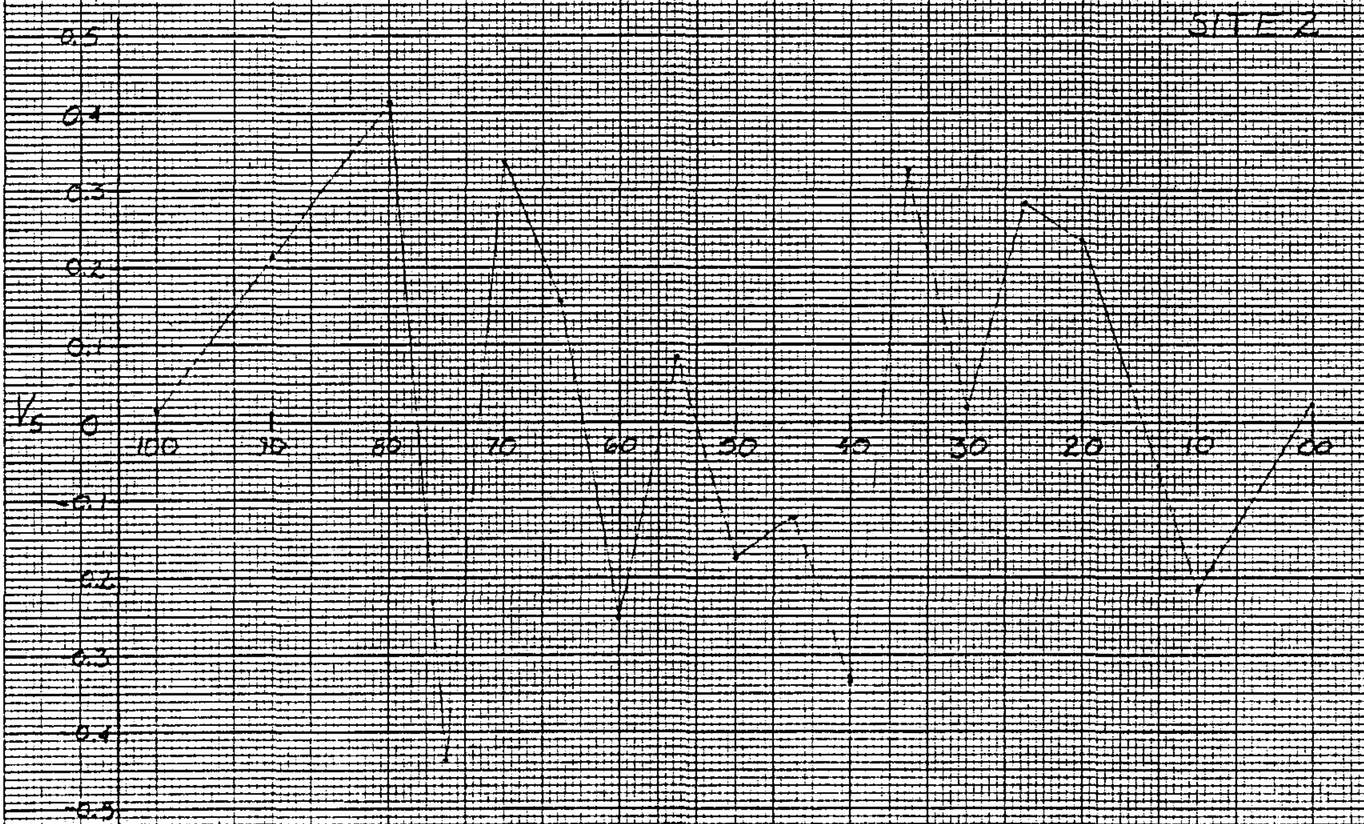



SITE 1
V_s
0.1
0.2
0.3
0.4
0.5
0.6
100
90
80
70
60
50
40
30
20
10
00

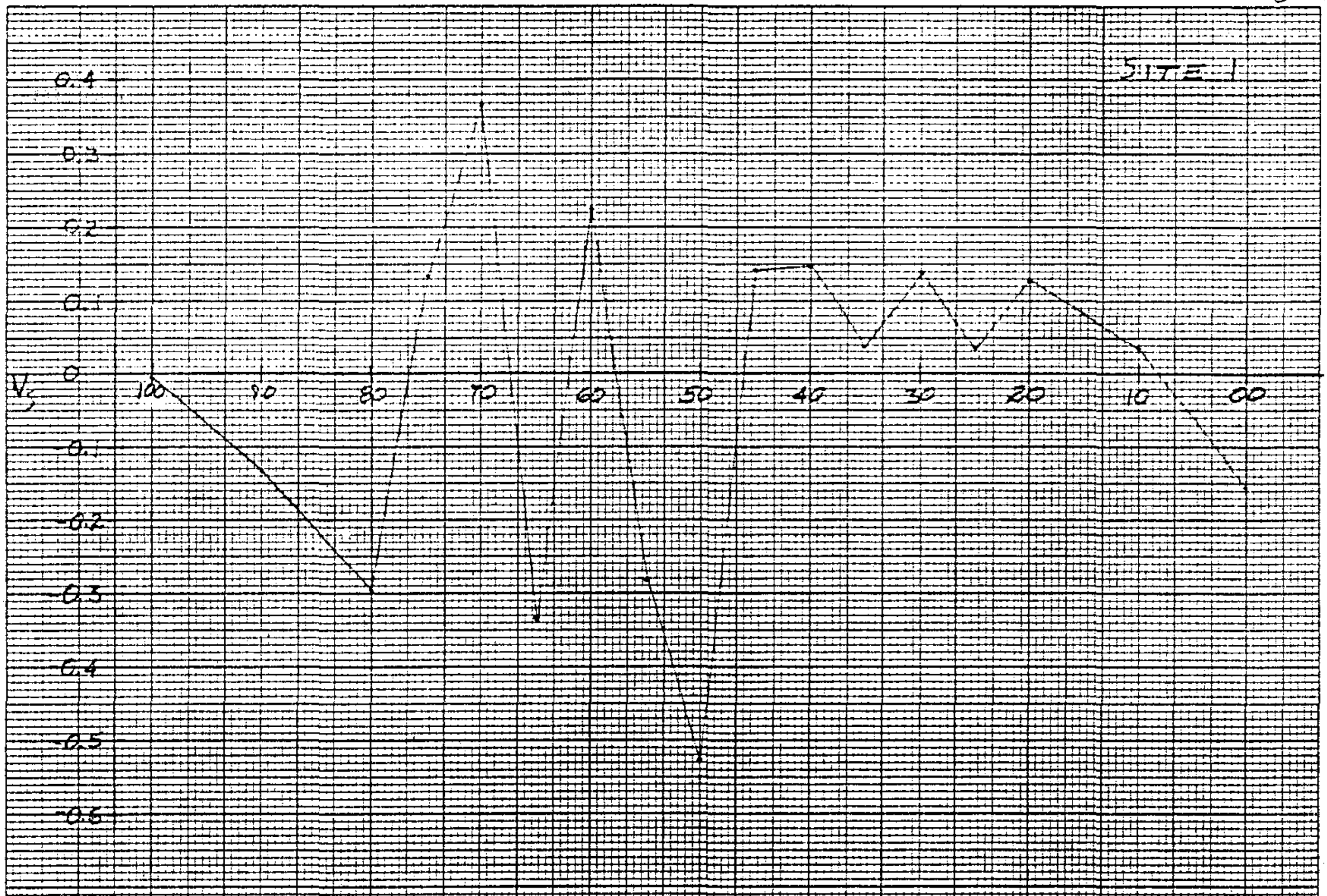

SITE 2
V_s
0.1
0.2
0.3
0.4
0.5
0.6
100
90
80
70
60
50
40
30
20
10
00

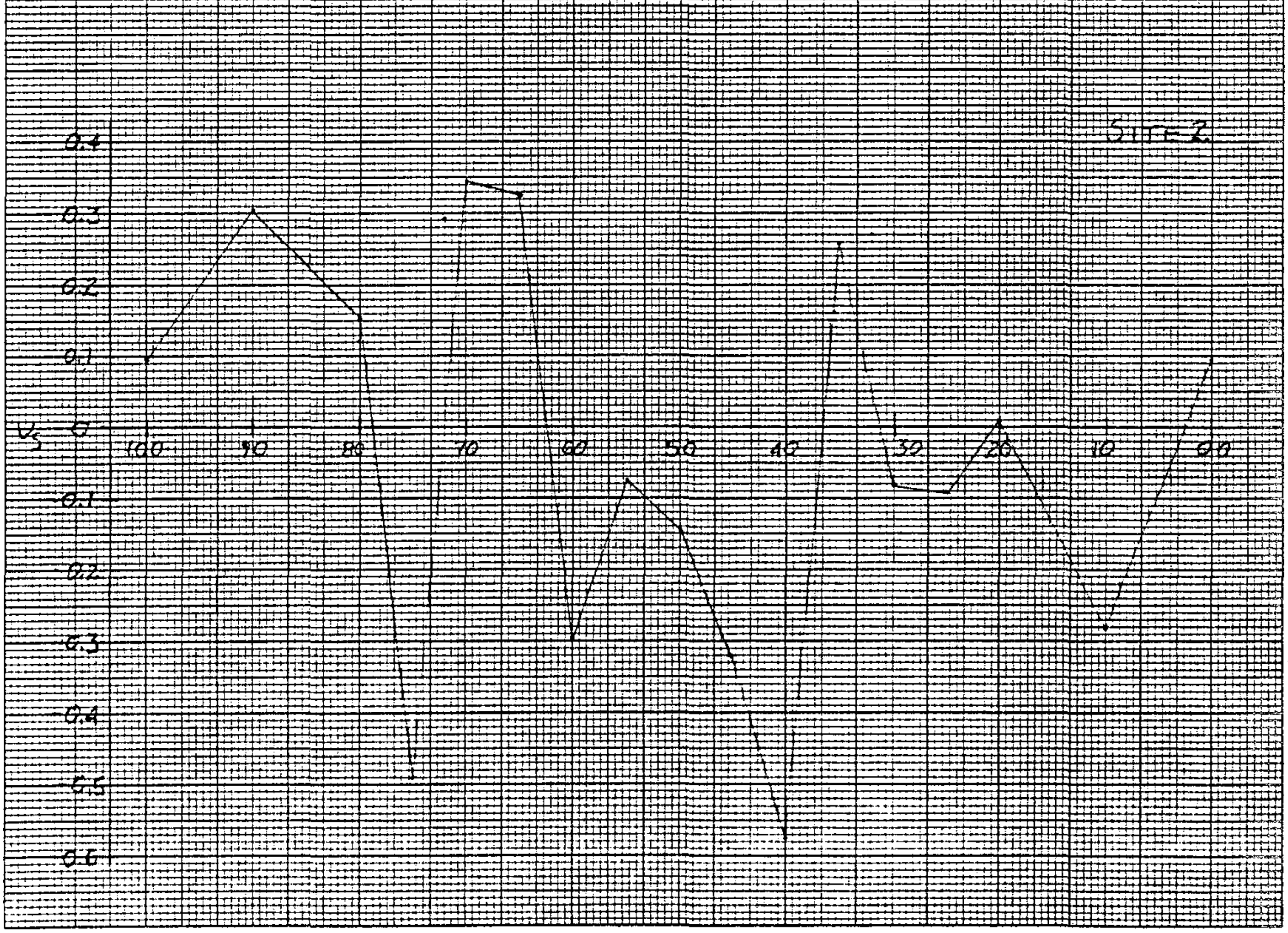
VERTICAL MEA.
82 09 27





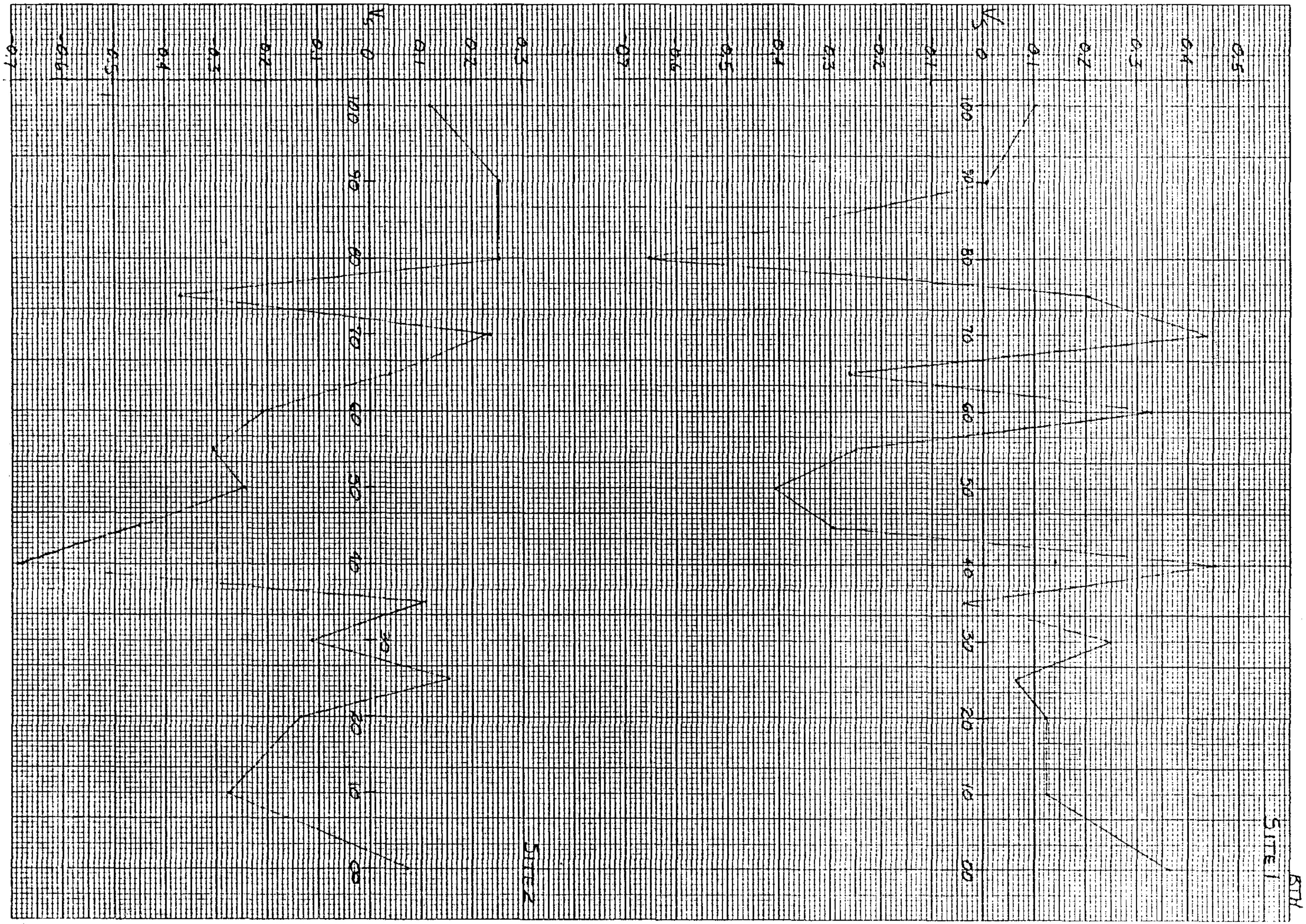
MILLER HILL RD
820506 BTH





K+E 10 X 10 TO THE CENTIMETER 18 X 25 CM.
KEUFFEL & ESSER CO. MADE IN U.S.A.

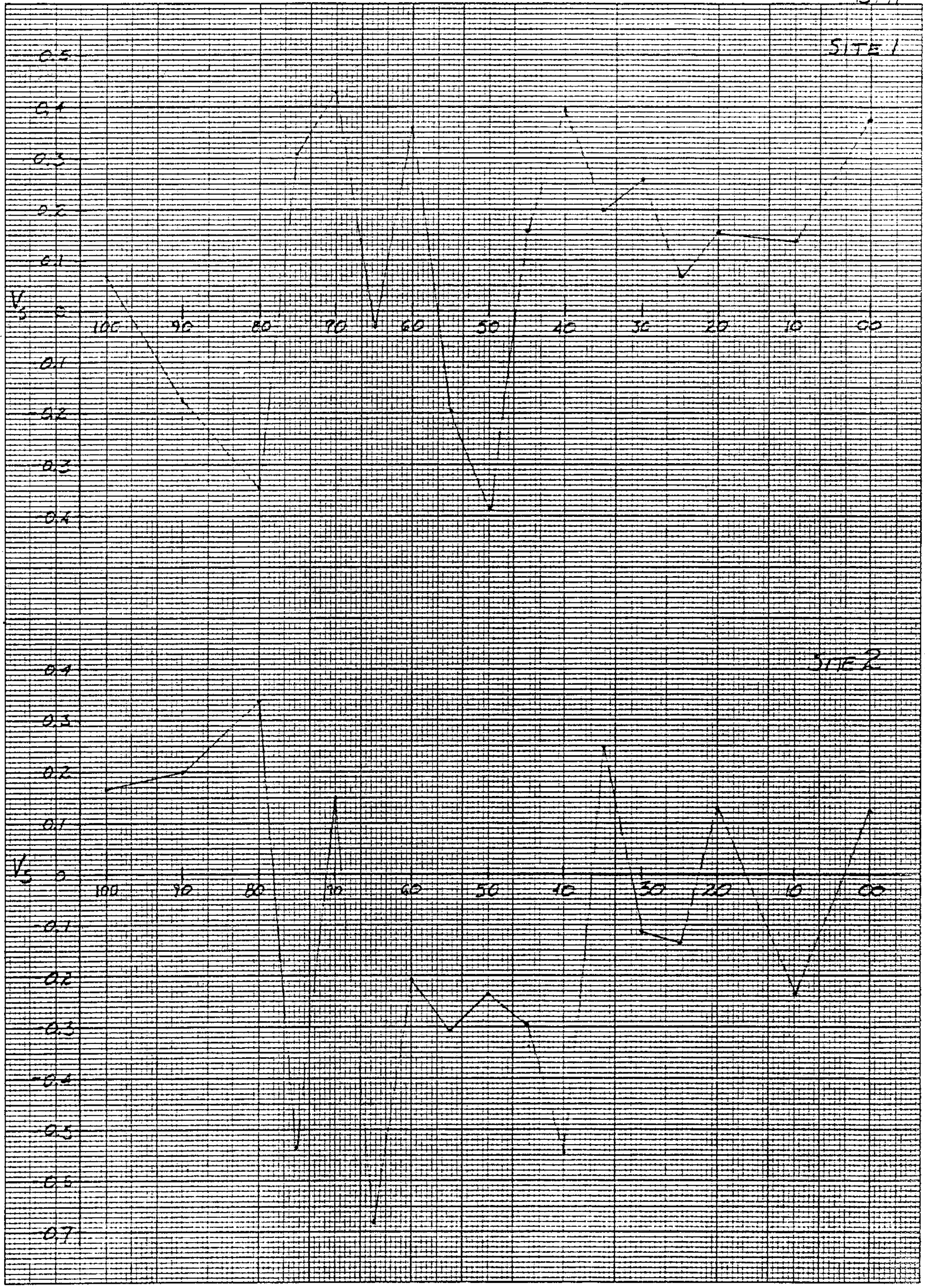
46 1512



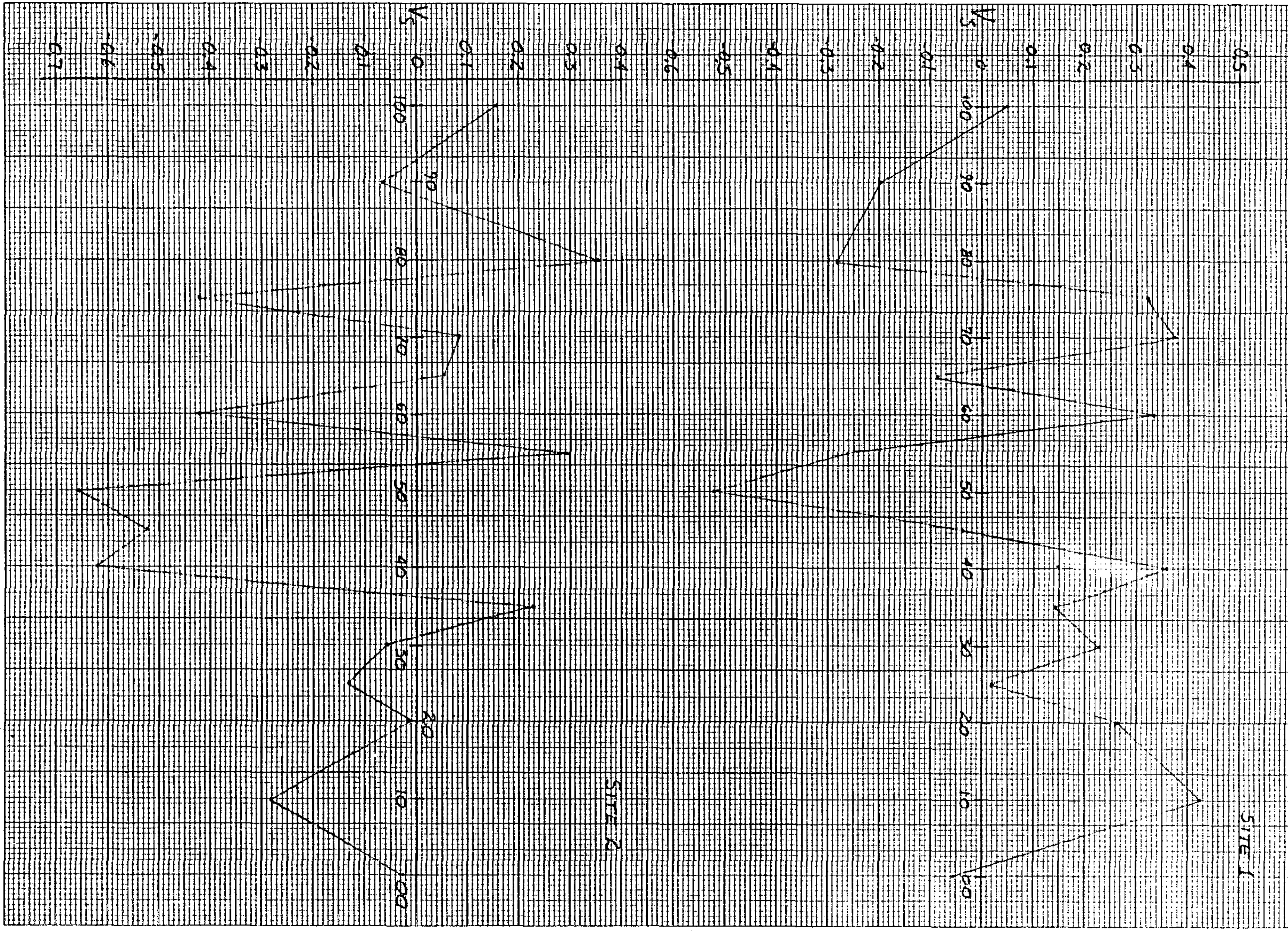
D13

820511

MILLER HILL RD
820519



46 1512

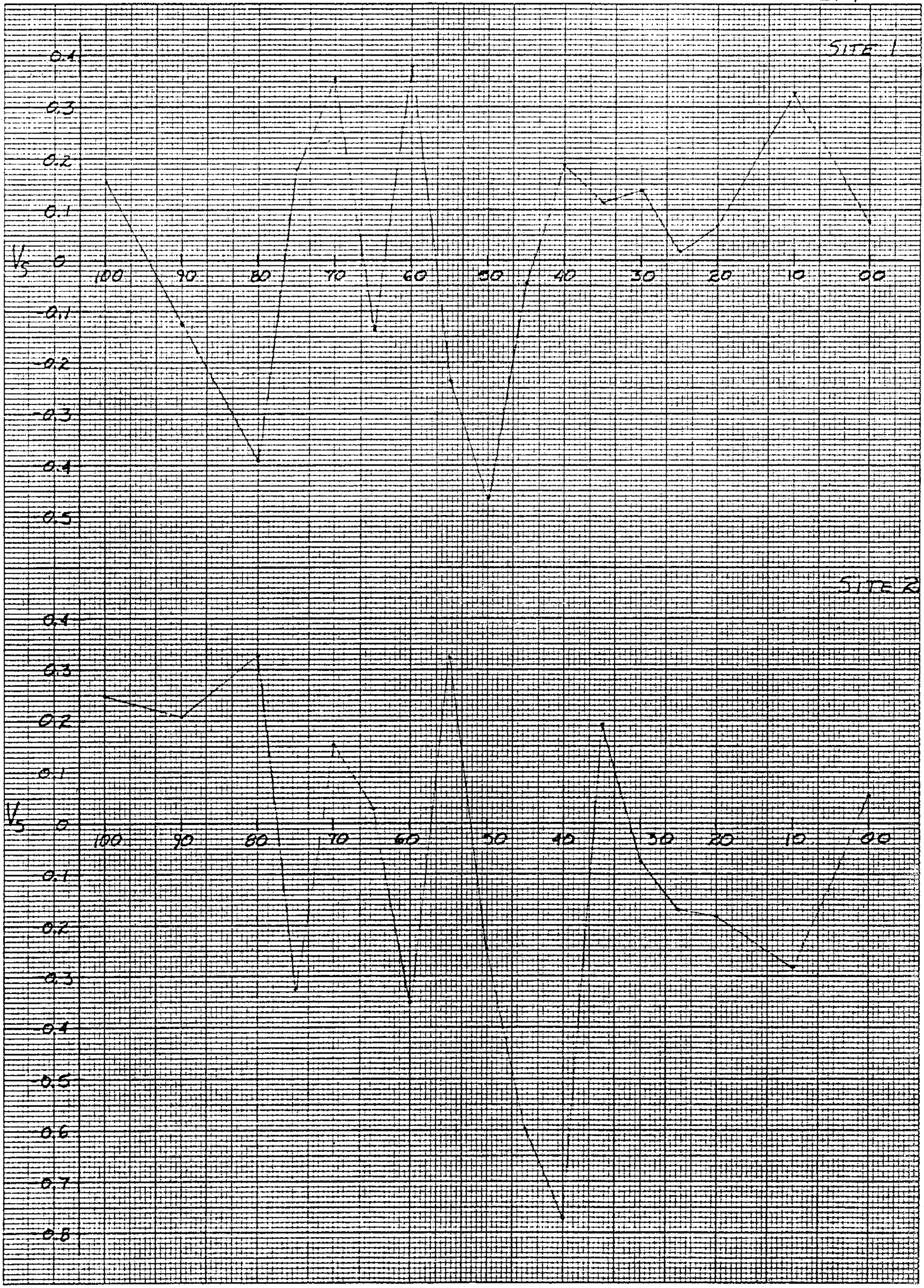


D15

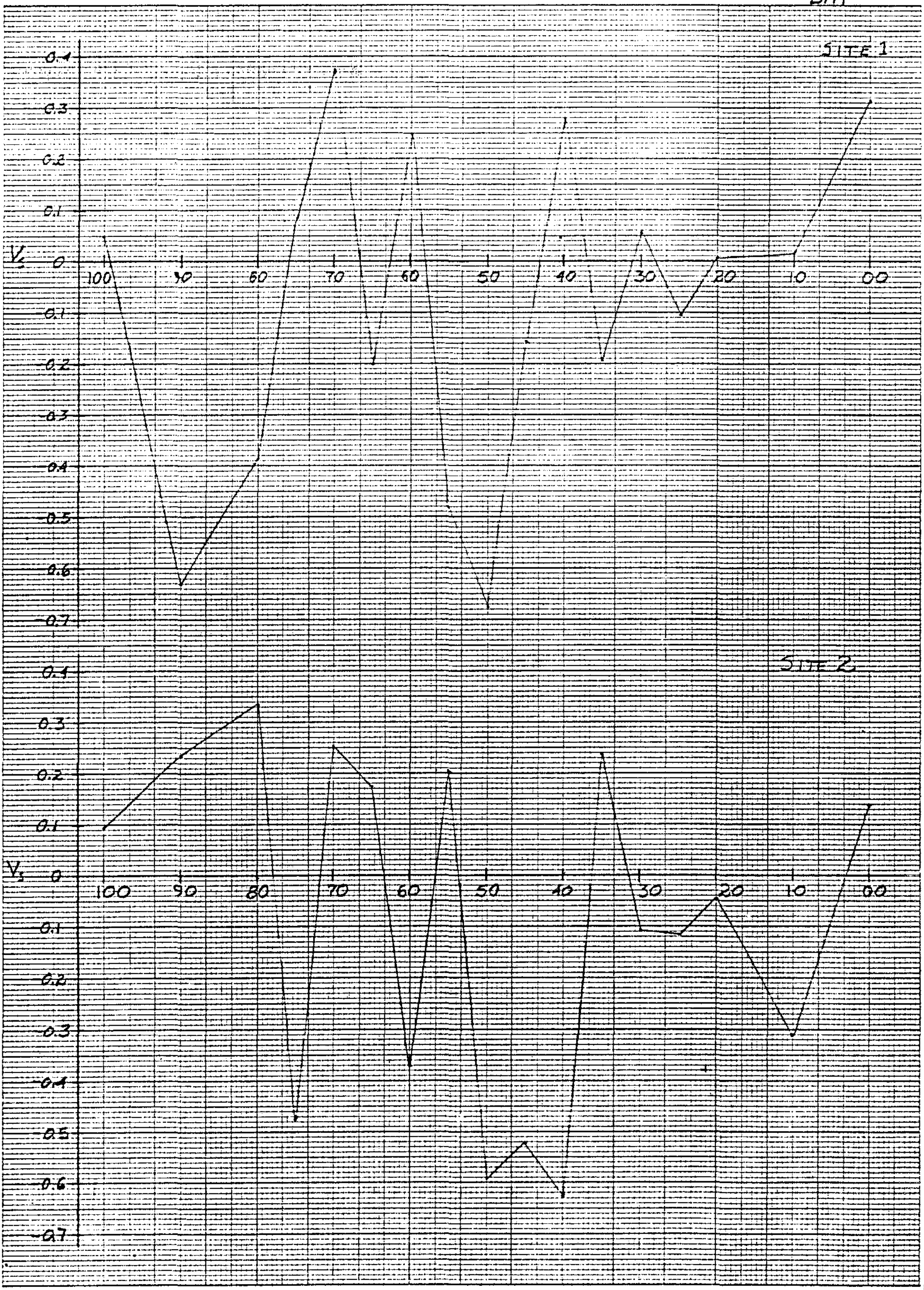
MILLER HILL RD.
820525

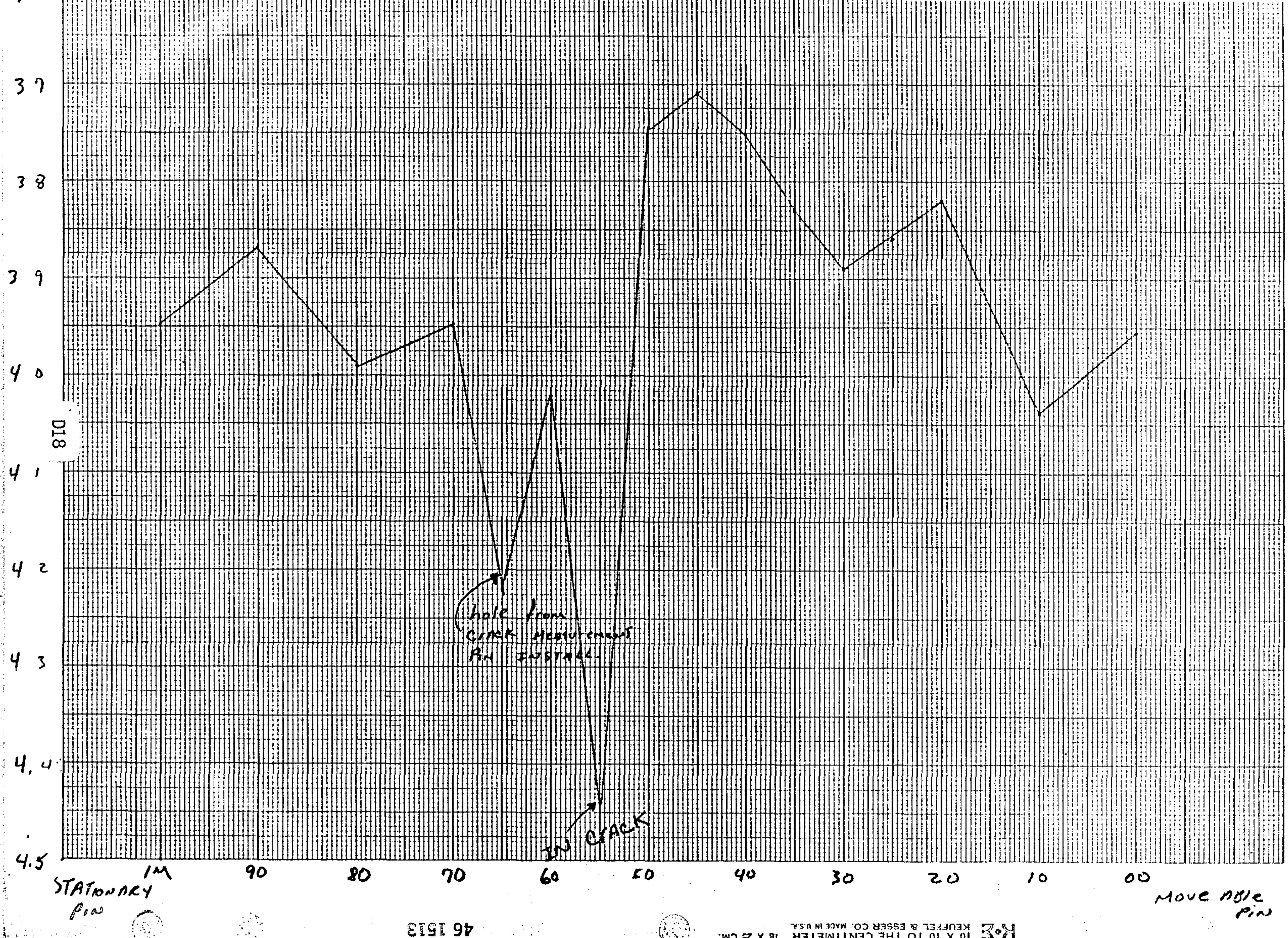
BTH

Miller Hill Rd.
820603
BTH

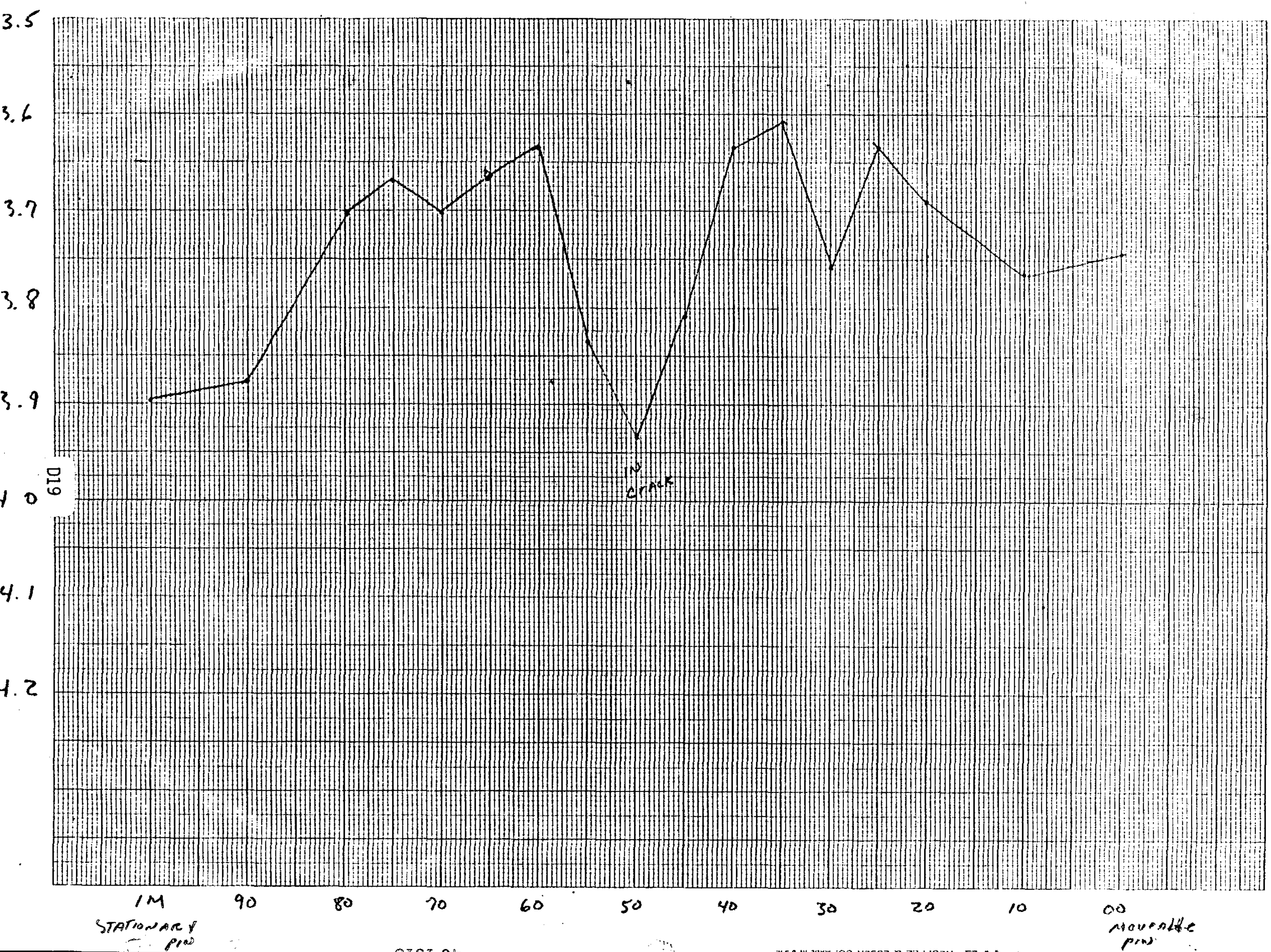


MILLER HILL RD.
820615
BTH





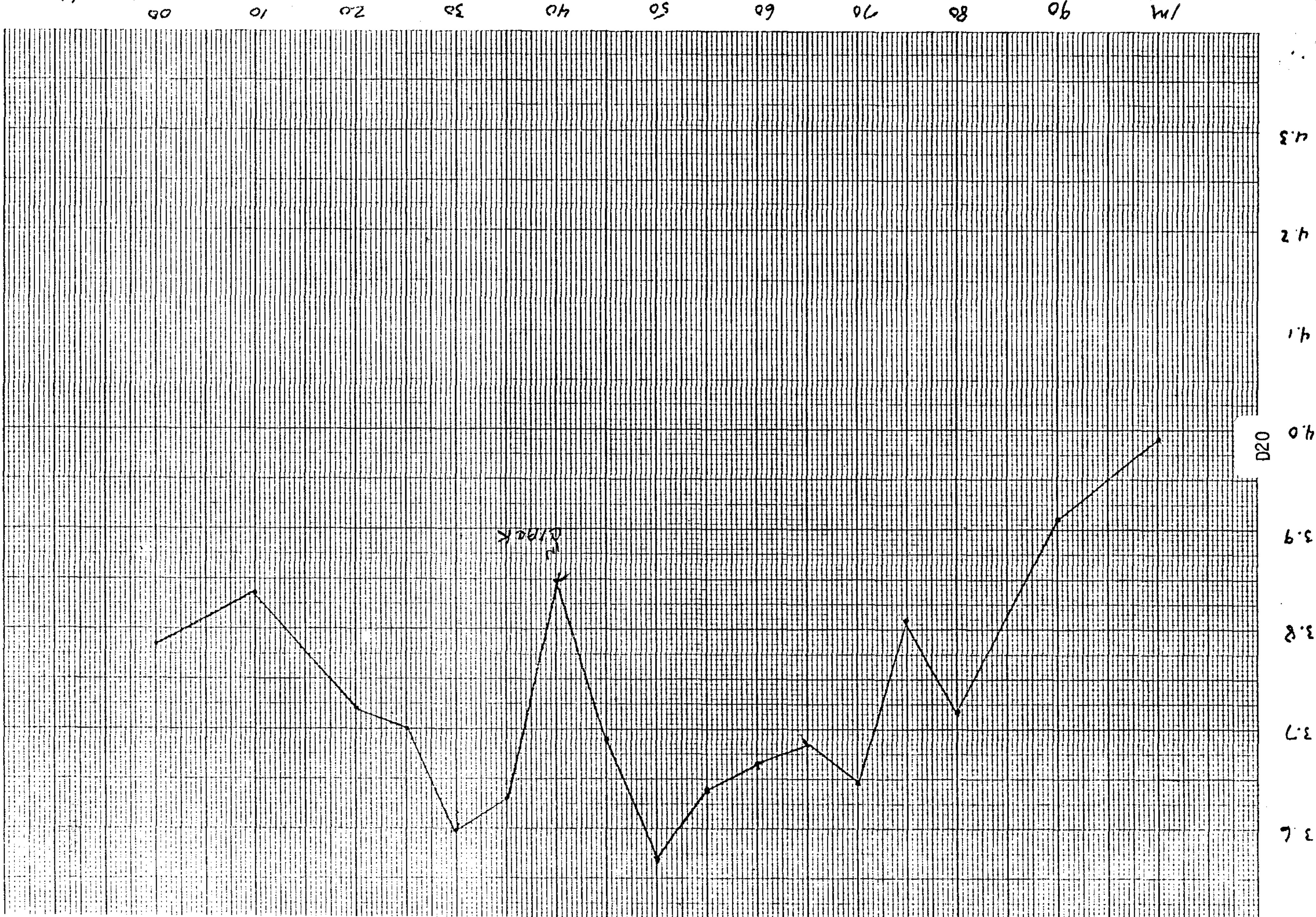
D18
3 7
3 8
3 9
4 0
4 1
4 2
4 3
4, 4
4.5
1M
90
80
70
60
50
40
30
20
10
00
STATIONARY PIN
MOVEABLE PIN
hole from crack measurement pin install.
IN CRACK


3.5
3.6
3.7
3.8
3.9
4.0
4.1
4.2
IN CRACK
1M
90
80
70
60
50
40
30
20
10
00
STATIONARY PIN
MOVEABLE PIN

D19

EAST
movable pin

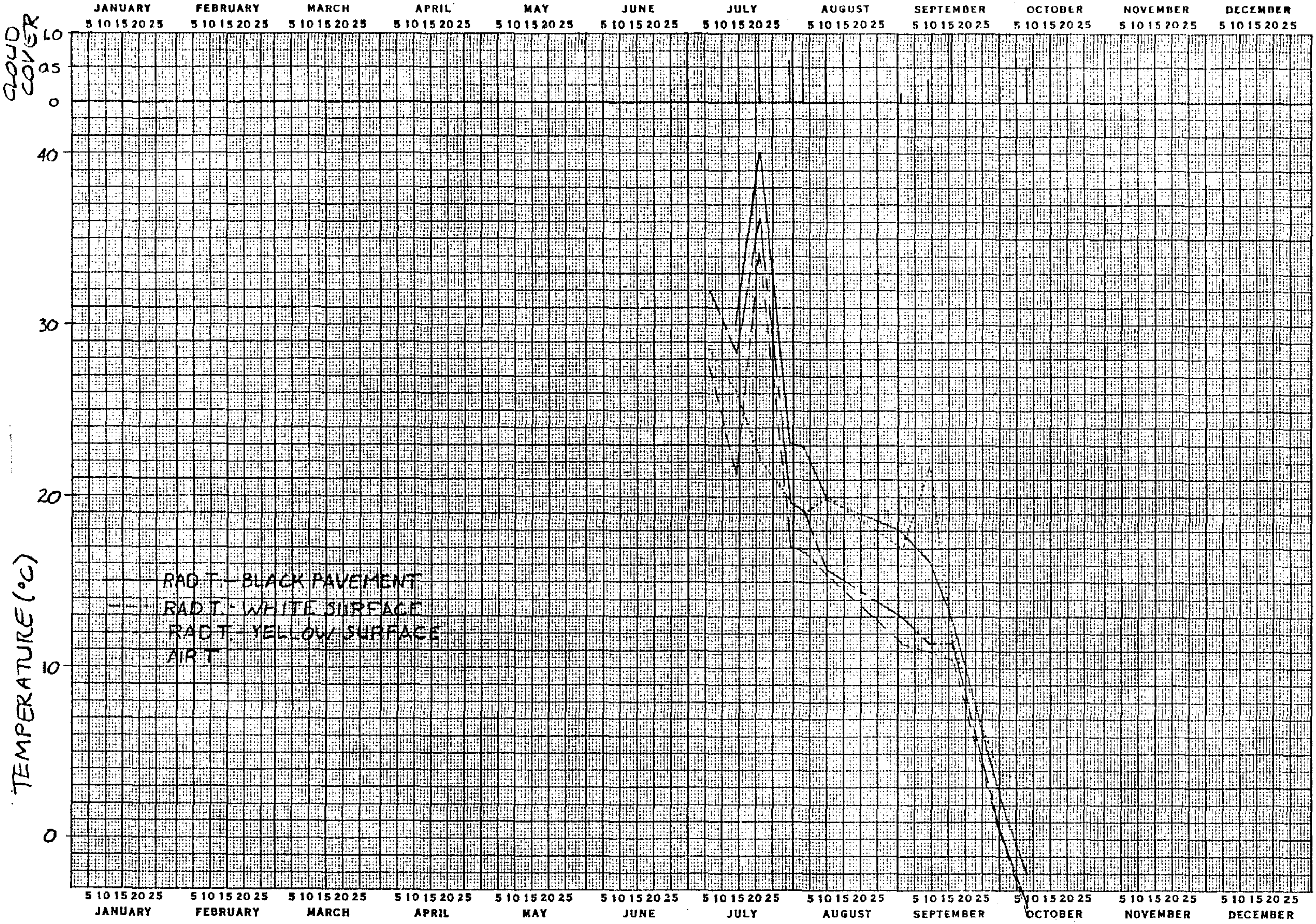
STATIONARY pin
WEST



APPENDIX E

Air and pavement surface temperature measurements

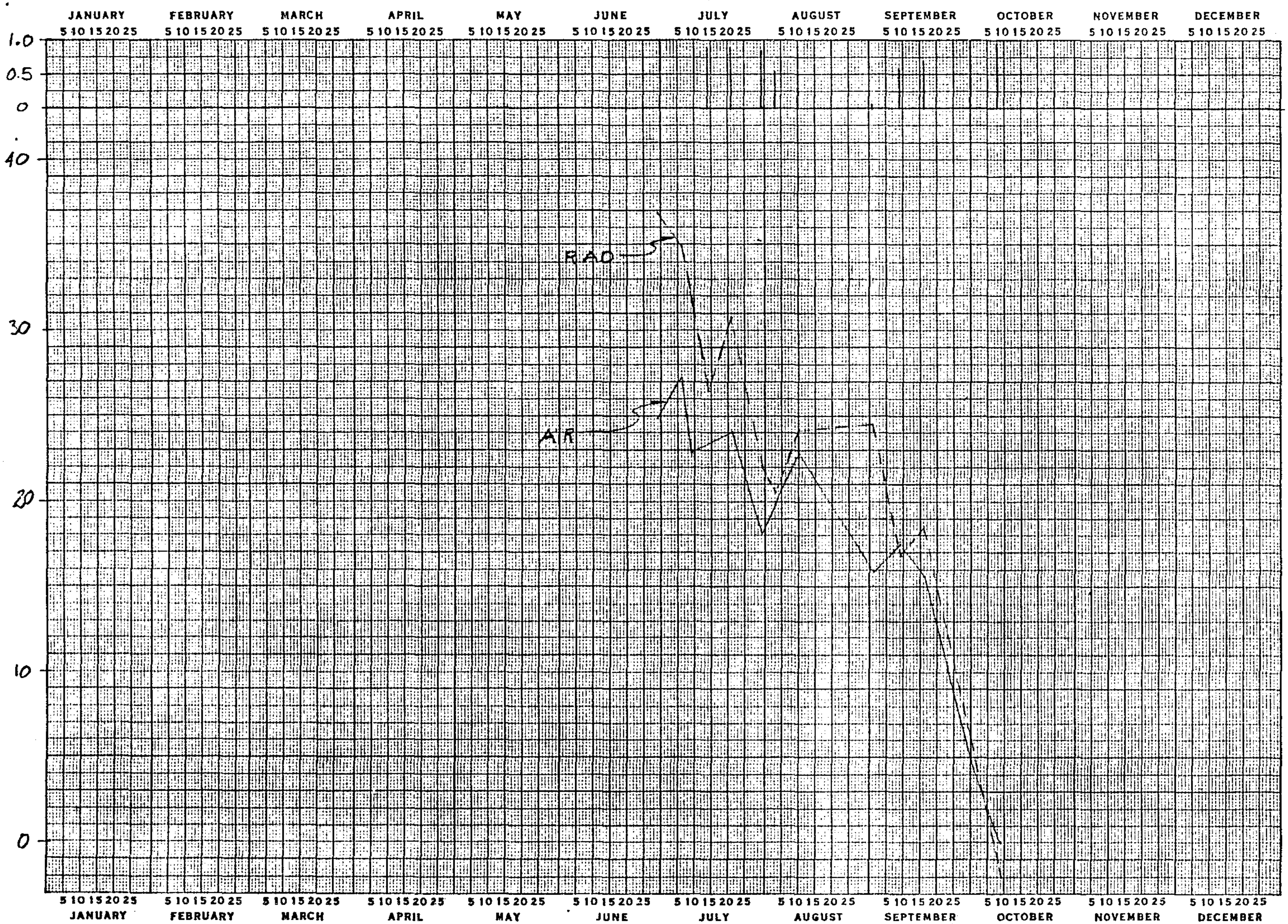
PEGER ROAD



1982

K+E 1 YEAR BY DAYS X 250 DIVISIONS 46 2893 MADE IN U.S.A.
KEUFFEL & ESSER CO.

SHEEP CREEK RD.

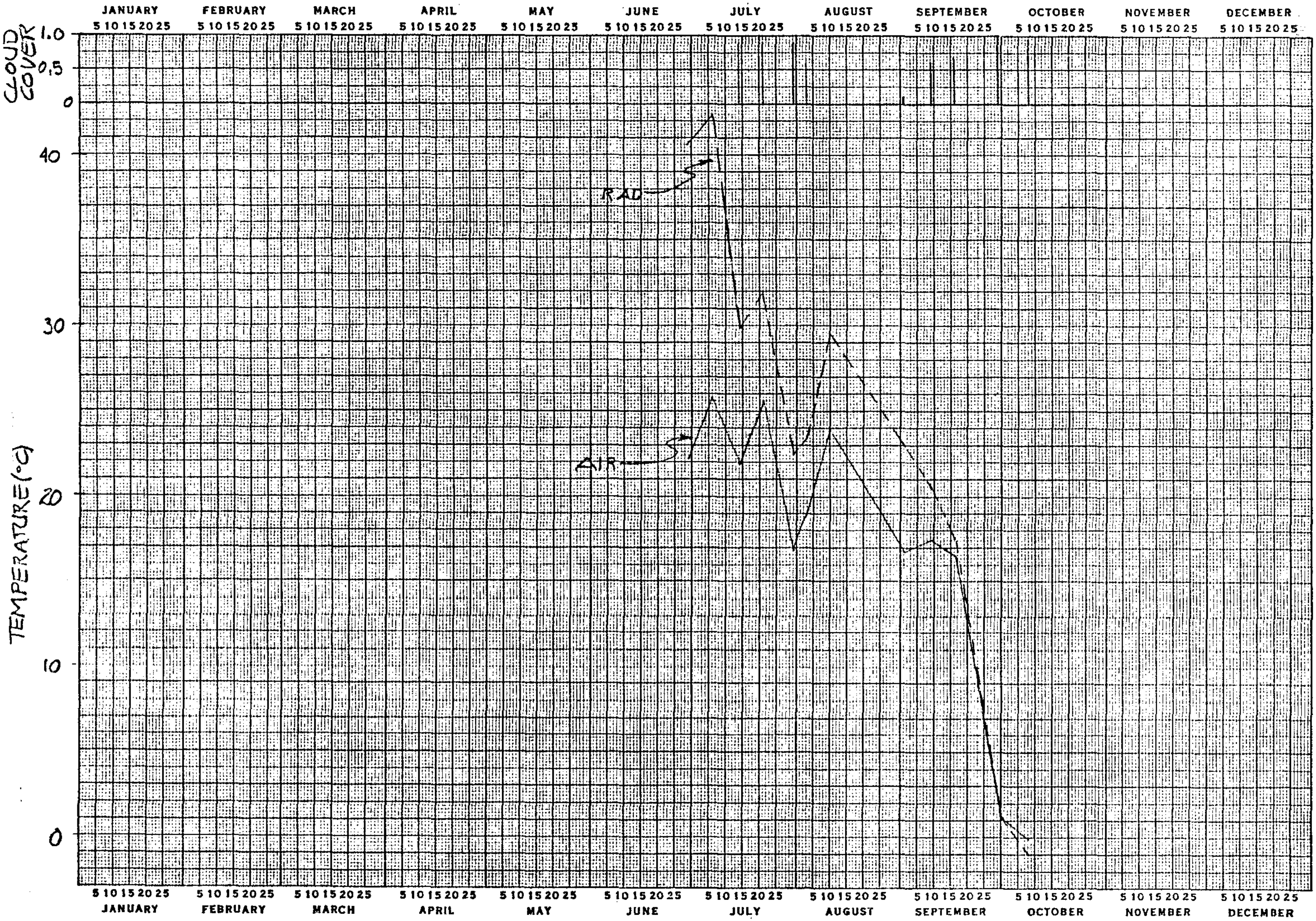


E3

EZ1015

1982.

MILLER HILL RD

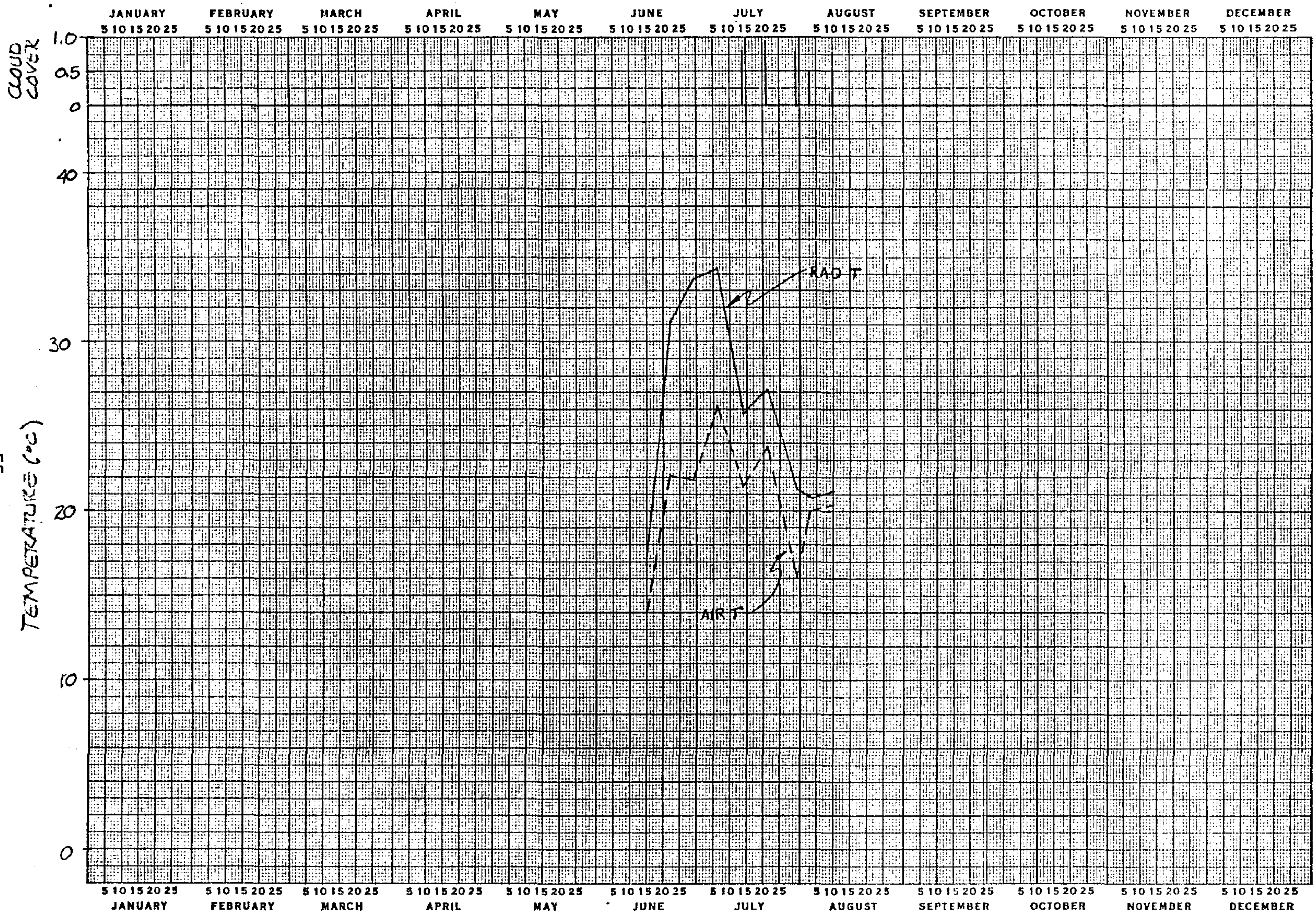


E4

ALDER CREEK ON PARKS HWY

SITE 1

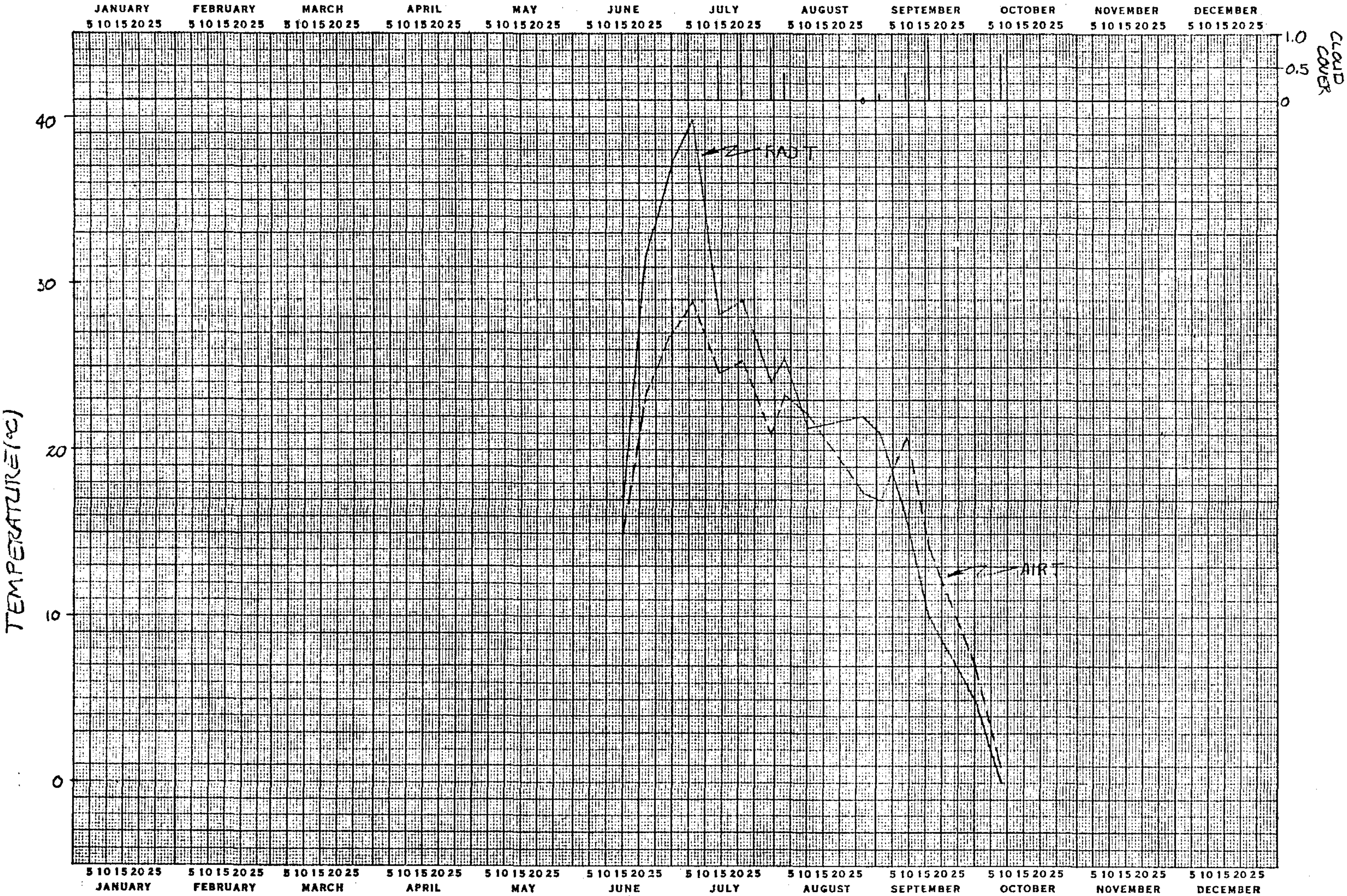
E5



1982

120128

UNIVERSITY AVE. BY CHENA RIVER SITE 2

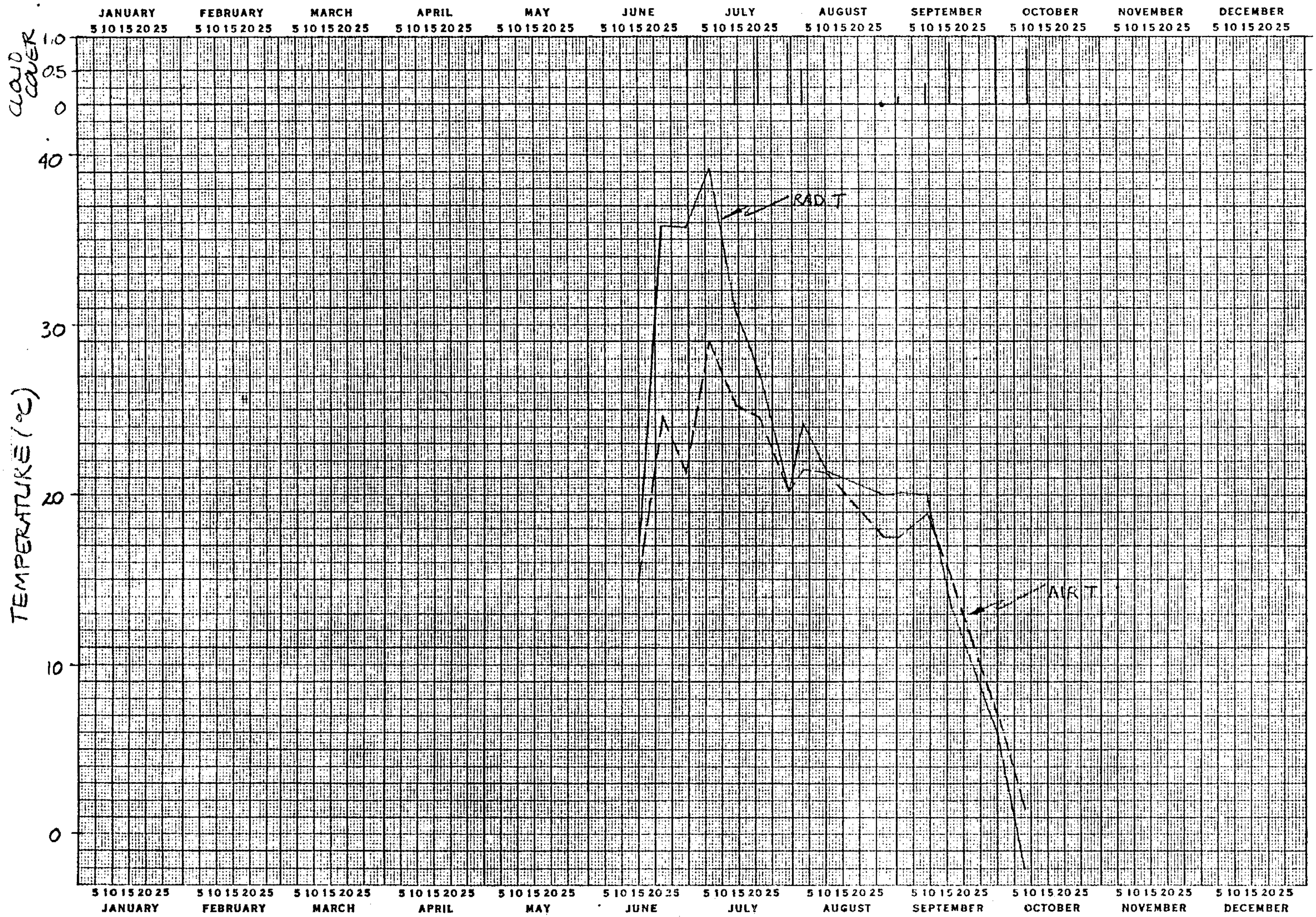


1982

821021

FARMERS LOOP RD ACROSS FROM BANK

SITE 3

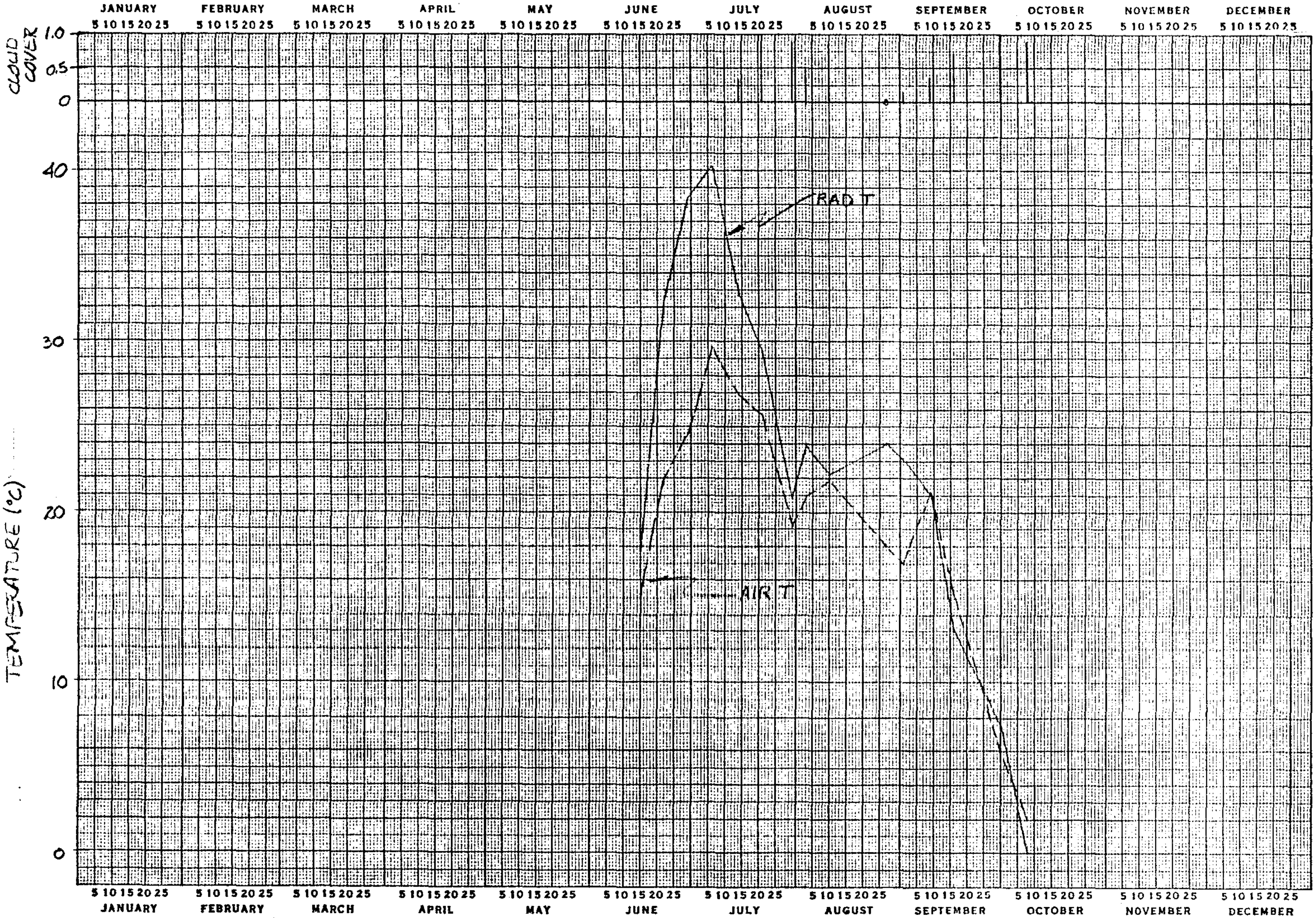


E7

1987

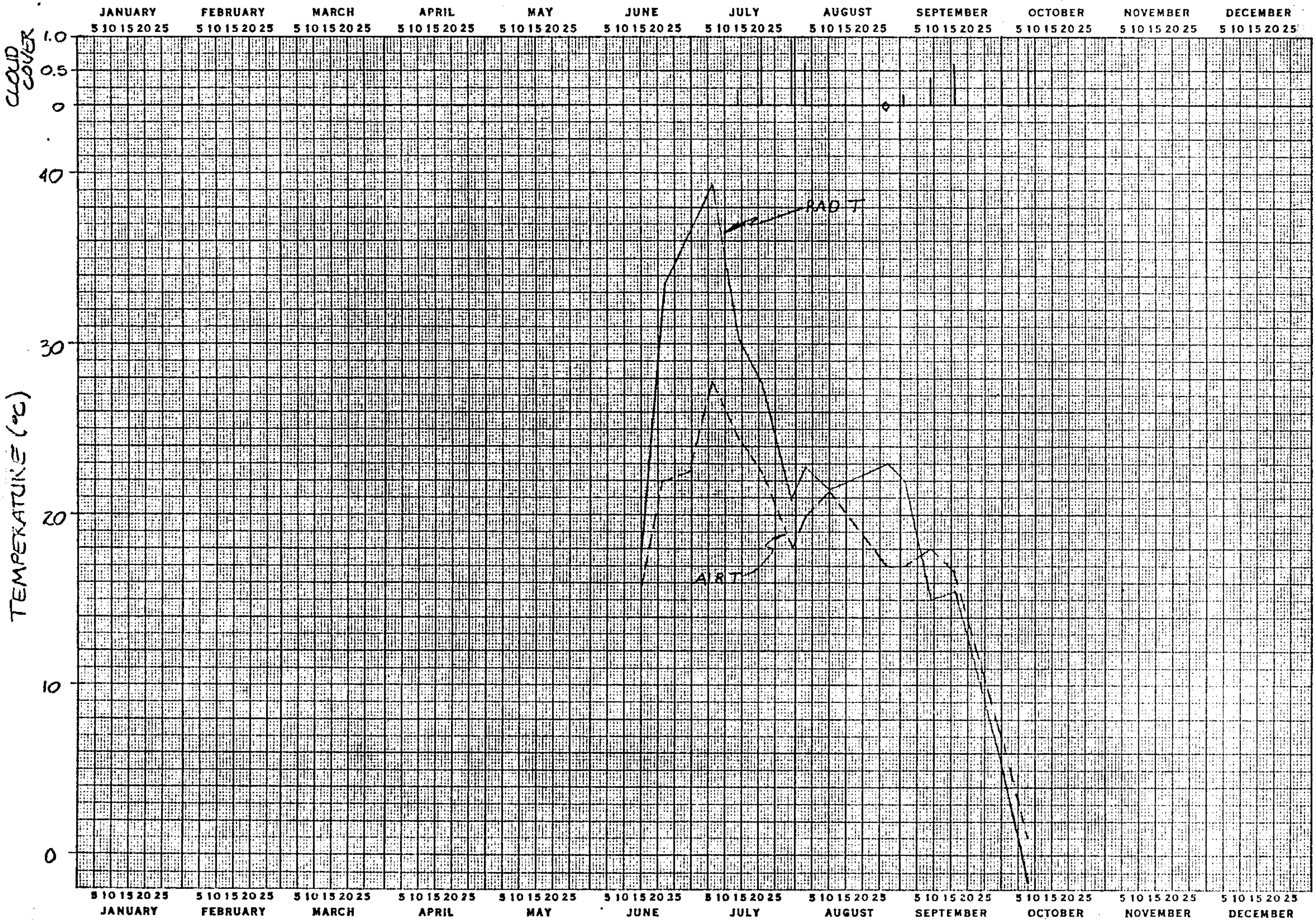
FARMERS LOOP RD BY BLUE HOUSE

SITE 4



1982

FARMERS LOOP RD - WEST ENTRANCE TO DOG MUSHERS FIELD SITE 5

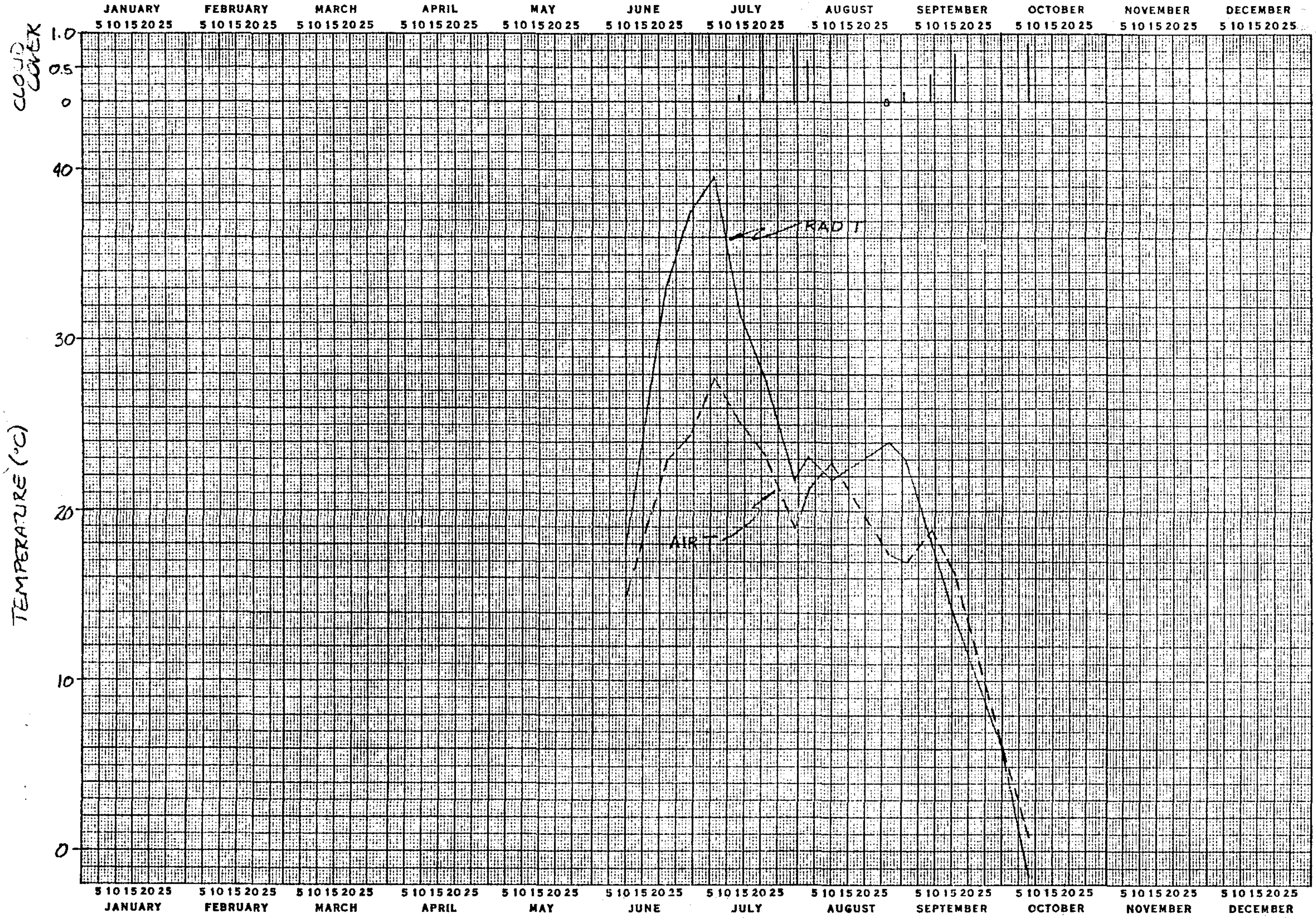


1982

821021

FARMERS LOOP ROAD BY GREEN QUONSET HUT

SITE 6

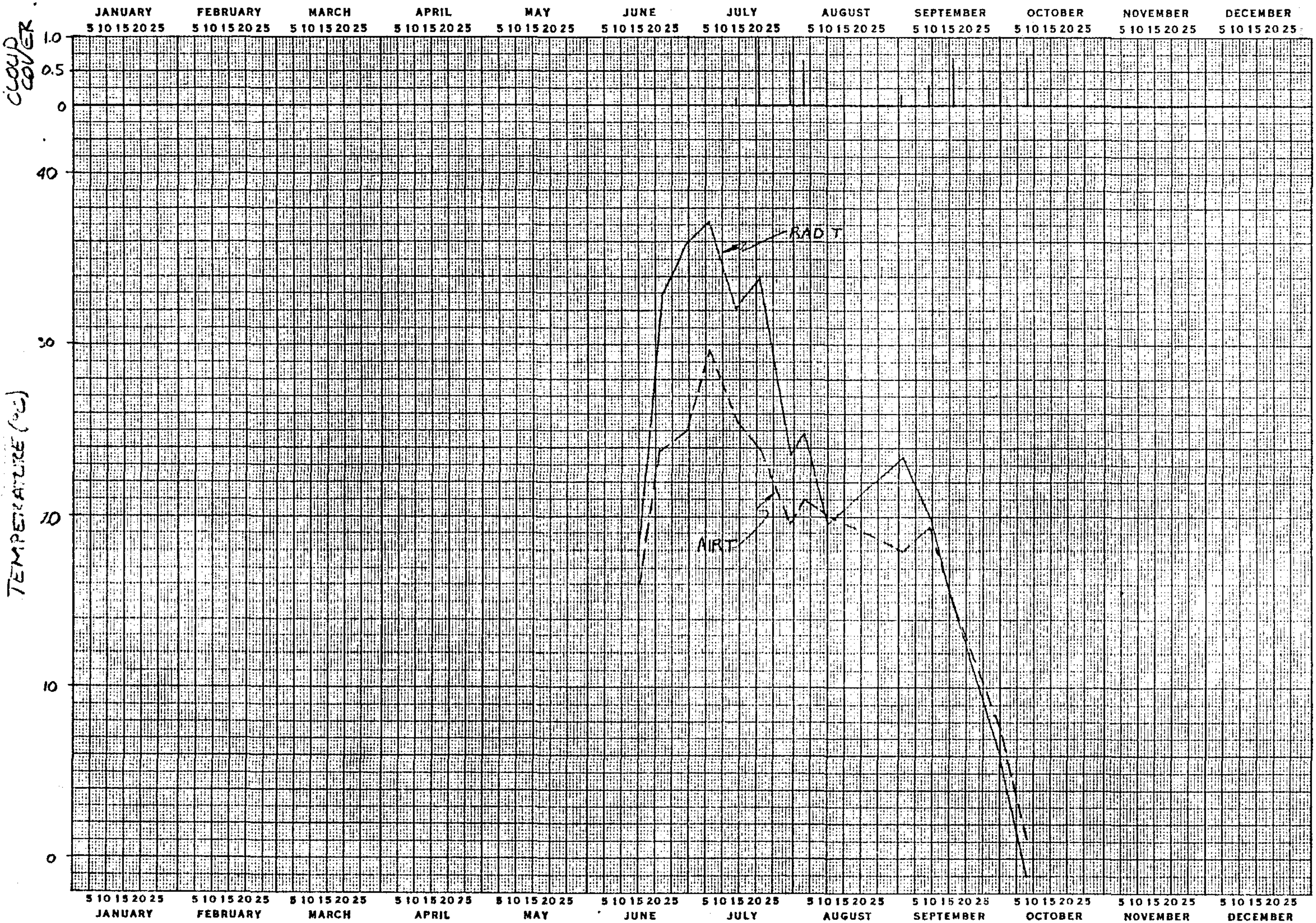


E10

121028

TRAINOR GATE RD & F ST

SITE 7

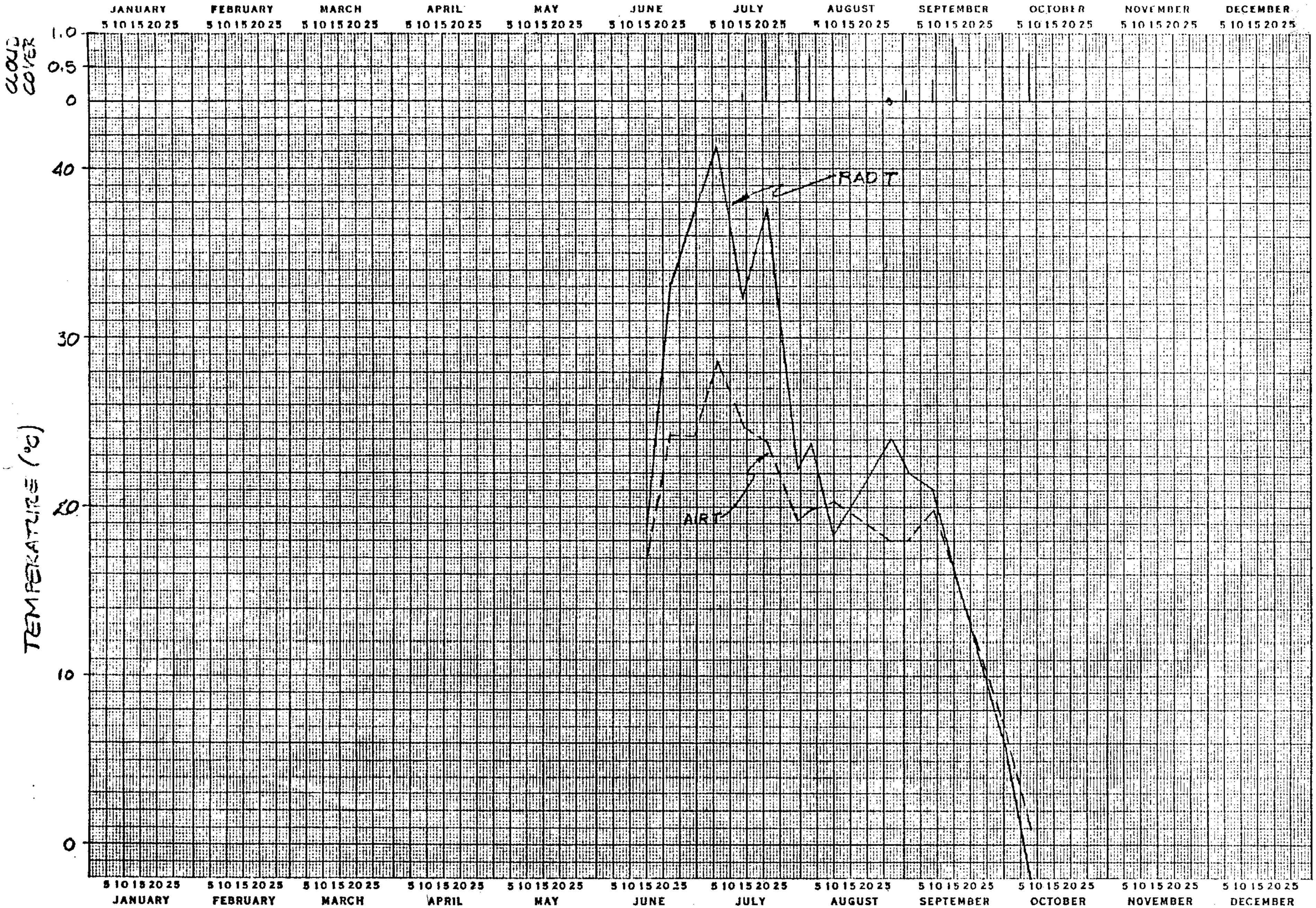


1982

E11

120128

TRAINOR GATE RD AND D ST. SITE 8



E12

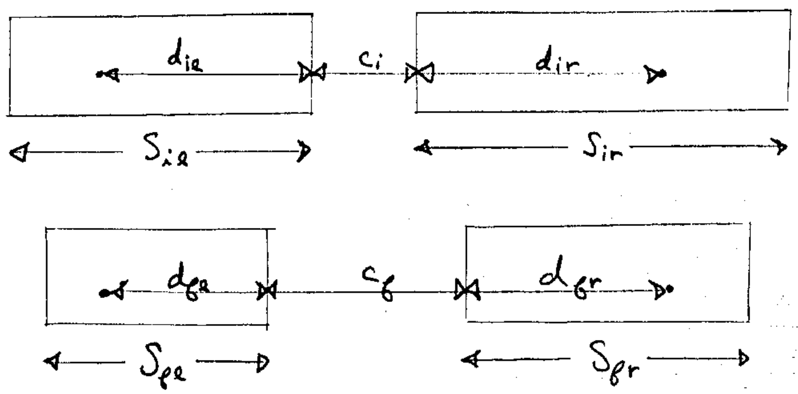
120128

APPENDIX F

Estimating thermal expansion coefficients from extensometer measurements

The appropriateness of equation 3.1 relating the thermal expansion coefficient to measured variations in the distance between anchor bolt positions depends upon the actual crack spacing. Equation 3.1 becomes more reliable as the actual crack spacing decreases or as the distance between anchor bolts increases. This can be seen from the following analysis.

Figure F1(a) depicts the initial conditions and Figure F1(b) a later condition after crack extension



F1(a)

F1(b)

where c_i = initial crack width

c_f = final crack width

d_{ir} = initial distance from crack edge to anchor bolt on right

d_{il} = initial distance from crack edge to anchor bolt on left

d_{fr} = final distance from crack edge to anchor bolt on right

d_{fl} = final distance from crack edge to anchor bolt on left

S_{ir} = initial uncracked pavement length on right

S_{il} = initial uncracked pavement length on left

S_{fr} = final uncracked pavement length on right

S_{fl} = final uncracked pavement length on left

ΔL = change in measured distance between anchor bolts

We further assume that the midpoints of the uncracked pavement sections remain fixed during crack extension. Then the dimensionless pavement strain ε is given by

$$\varepsilon = \frac{S_{fr} - S_{ir}}{S_{ir}} = \frac{S_{fl} - S_{il}}{S_{il}}$$

$$\varepsilon = \frac{d_{fr} - d_{ir}}{d_{ir}} = \frac{d_{fl} - d_{il}}{d_{il}}$$

and

$$c_f - c_i = \frac{S_{fr} - S_{ir}}{2} + \frac{S_{fl} - S_{il}}{2}$$

Since $\varepsilon = \alpha\,\Delta T$,

$$c_f - c_i = \frac{\alpha \Delta T S_{ir}}{2} + \frac{\alpha \Delta T S_{il}}{2}$$

and

$$\Delta L = (d_{fr} + c_f + d_{fl}) - (d_{ir} + c_i + d_{il})$$

$$\Delta L = \alpha \Delta T\,[d_{ir} + d_{il} + \frac{S_{ir} + S_{il}}{2}]$$

The estimate of thermal expansion coefficient found in section III (say α') was determined by dividing ΔL by the original distance between anchor bolts:

$$\frac{\Delta L}{L_o} = \frac{\alpha \Delta T\,[d_{ir} + d_{il} + (S_{ir} + S_{il})/2]}{d_{ir} + d_{i\ell} + c_i} = \alpha' \Delta T$$

which implies

$$\alpha = \alpha' \frac{(d_{ir} + d_{il} + c_i)}{d_{ir} + d_{il} + (S_{ir} + S_{il})/2}$$

This suggests that the actual thermal expansion coefficient is appreciably less than the crude estimate, with the proportionality factor r given by

$$r = \frac{1 + c_i/(d_{ir}/d_{il})}{1 + (S_{ir} + S_{il})/2(d_{ir} + d_{il})}$$

Since $c_i \ll d_{ir} + d_{il}$, and assuming $S_{ir} \approx S_{il} = S$ and $d_{ir} \approx d_{il} = d$

$$r \approx 1/(1 + S/2d) \qquad \text{(F2)}$$

For the span AG, where $1/r \approx 1.3$, we find $S/d = 0.6$. For the span JH, where $1/r = 2.5$, we find $S/d = 3.0$. Actual crack spacing for these locations was not recorded so it is not known whether this simple one-dimensional analysis is appropriate. For these extensometer measurements $d \approx 6.5$ m suggesting a crack spacing of about 4 m near span AG and 20 m near span JH.

From equation F2 it can be seen that the estimate for the thermal expansion coefficient using the anchor bolt locations becomes more reliable as the crack spacing S decreases and as the distance between anchor bolts d increases.