FROST HEAVING IN THE WALLINGFORD-MERIDEN SECTION OF I-91

by

Kent A. Healy - Assistant Professor
Grant Meitzler - Graduate Assistant

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Background:

The first indication of trouble in the I-91 shoulders was the appearance of differential heaving shortly after the road was opened on January 6, 1966. The shoulder had heaved as much as three inches above the adjoining concrete.

In addition, pumping at the shoulder joints has been observed during periods following even light rain. Recent performance indicates that pumping is becoming more severe.

Field Observations:

Eyewitnesses report a maximum shoulder heave of three inches above the concrete. The events leading to this heaving must be pieced together from other available information.

Three important field observations are available from witnesses who saw the heaving. These observations are:

1. The asphalt rose three inches above the concrete.
2. There was thin ice encapsulating stones excavated from the shoulder when the highway department sampled the materials on February 7, 1966.
3. There was a void underneath the shoulder near the shoulder joint.

Since solid concrete will not heave, simple frost heaving in the adjacent soil under the shoulder would result in a differential heaving. Freezing in the soil beneath the bottom of the concrete should result in equal heaving under both shoulder and roadway. There will still be heaving, but there will be minor differential heaving.

The observation that thin ice was encapsulating small stones in the gravels indicates a large amount of segregated ice present. It is the observed nature of these soils both in the laboratory and in the field to form very thin ice lenses which are virtually impossible to see with the naked eye. The fact that any segregated ice was seen at all is surprising.

The presence of the void underneath the shoulder indicates that a partial thawing had occurred at the time of observation. Random wheel loads forcing the edge of the asphalt paving down into the void has caused crescent shaped cracks along the shoulder.

The longitudinal shoulder joint may admit water to the shoulder; efforts should be made to seal this joint. But, it must be kept in mind that continual action due to temperature changes and traffic loading make it a virtual certainty that water will be able to enter the joint. Provision must be made for the effective draining of this water from the base layers.

There can be no question at this point but that the soils are frost susceptible. The typical classification of these soils is SM, (under the Unified Soil Classification System), and group F-2 (Corps of Engineers Frost Classification). On the basis of the F-2 frost classification, the soil could be expected to heave between 2 and 4 mm per day, being classed as medium frost susceptible.

The field report submitted after the shoulders were first opened on February 7, 1966, notes that there was a lack of segregated ice which was incompatible with the observer's experience. Laboratory tests conducted on these particular soils have shown that the ice lenses which are formed in these soils are typically so fine that very careful dissection is necessary to spot them. The report did note that the stones which were there were coated with a thin sheet of ice. This is an indication of considerable segregated ice which is generally present in such thin layers that it is not readily visible.

There was ample evidence of ice segregation in the field during the winter of 1966-1967. This evidence ranged from a few isolated points of minor shoulder heaving, to the uncovering of a one eighth inch thick ice lens in the "gravel" base of the shoulder (at the Northbound entrance ramp to I-91 from the new Route 72). Frozen base soils were seen to have either definite ice lenses or thin sheets of ice about the pebbles in the soils.

Aside from any indirect evidence, the most important thing to remember is that there was a heaving condition. If the soil was not frost susceptible, it would not have heaved.

It has been observed in the field that the road area is in a generally wet condition. During November of 1966, pumping was seen in several locations twenty-four hours after a light rain. There was ample evidence of water flowing in the open shoulder joint. Field notes from the Highway Department after the February 7, 1966 sampling of the shoulder soils, indicate that salt stains were present on the underside of the pavement one foot and two feet away from the open crack. In addition, it was noted that water was seen to enter the shoulder joints at one point and exit from the joint farther down the slope of the roadway. This was also observed during the 1966-1967 winter and is clear evidence of the lack of drainage capability in the shoulder materials.

The wet condition in the field can be further appreciated in the light of the fact that the precipitation for both the 1965-1966 and the 1966-1967 winter was below normal. During the specific period of the heaving, the precipitation was 29.9% below normal. However, rain did occur at the opportune times of the temperature cycle for significant frost heaving.

The specific temperature conditions of the freezing period during January of 1966 provided optimum heaving conditions. There was a combined freezing period from January 8, 1966 to February 10, 1966. This was made up of two freezing periods broken by a short period during which the temperature averaged about freezing. The first period followed a long warm spell and served to establish conditions for the heaving to occur during the second cold period. The temperatures were generally mild freezing temperatures which permitted slow build up of segregated ice and maximum heave. Since the heave was reported about February 1, 1966, the later period in early February has not been considered. The significant temperature period was that of January 24, 1966 to February 1, 1966, which followed significant precipitation and progressed slowly from mild freezing temperatures to a two day period of very cold weather. This short cold period can easily account for the large and sudden movement which was noted by observers (see calculation D of the amount of heaving which occurred during this period). It must be emphasized that the temperatures of the 1965-1966 winter were not unusual and can be expected to recur in the future.

These soils have a typically high percentage of fines, and their dry strength is quite high. It is very difficult to break down the dried flocs of soil that remain unbroken through the duration of the test. This is a basic shortcoming in the dry sieve analysis which must be recognized. The percentage of fines indicated by dry sieving is always less than the actual fine percentage which is present in the soil.

The requirement for the plasticity limit of the fine material as per section M.02.07 paragraph two, is that the material passing the one hundred sieve cannot have enough plasticity to permit the running of the standard plastic limit test. This test was run on all the samples and each was found to be plastic (to the extent that it was just possible to perform the test).

The soft particles test as specified in section M.02.07 paragraph three, is indefinite, with results that bear little relation to the properties in question, or the field performance of the soil. For what value it might bear, the soils were seen to be acceptable under this requirement.

The soils which were tested did not meet the existing state specifications. Had they been within the existing specifications the likelihood of significant heaving would have been reduced.

Additional Tests:

We are specifically interested in various properties which have direct bearing on the extent of the frost heaving problem as well as the pumping problem.

PERMEABILITY

Tests were run to establish permeability values. This value permits evaluation of the tendency of the soils to hold water by capillarity, as well as their drainage capability.

The permeability values range from $4x10^{-}$ ft/min down to $1x10^{-}$ ft/min. These values correspond to typical values for a silt. The material cannot be considered free draining. Since these "gravels" will be relatively dense in the field, the possibility exists that the in situ permeability is even lower than the test values stated above.

The material which has been used under the name "gravel" has certain properties which classify it as a silt. This is consistent with the material's having been too fine to pass the state specifications. It is also consistent with the frost and pumping performance of the soils.

DEGRADATION

The "red gravels" are related to the red shales and sandstones of the Meriden-Wallingford area. These shales and sandstones are typically weak and the possibility exists that they deteriorate under the compacting rollers. The scratch tests makes an attempt to evaluate this tendency but, it falls short because of the nature of the rocks in question. There is little doubt on a siltstone which has a fairly uniform distribution of particles of uniform hardness. The pass or fail decision becomes difficult with the sandstones because of their structure. The sandstone is typically non-uniform with hard particles which will pass the scratch test individually, embedded in soft cementing material. The soft matrix will not pass the test. Yet, the individual hard particles will result in a clear streak. It was these sandstones that were seen to deteriorate most readily during the course of the tests which were performed.

The degree of degradation was determined using the standard proctor apparatus to apply known amounts of compaction in combination with alternate freezing and thawing cycles. The results of these tests show an increase in the amount of fine material, increasing with the amount of energy applied in compaction.

It is concluded, however, that there would not be any increase in percent of any size of more than two percent during field operation.

It is also pertinent that the change in particle distribution occurs as a decrease in particles larger than two millimeters. This decrease in coarse material re-appears as an increase in the weight retained between two millimeters and two tenths of a millimeter. There was no discernible change in the percentages of fines (less than 0.2 millimeters), even at 13,000 blows. If this is true at 13,000 blows, it will certainly be true at ninety percent of the optimum proctor of twenty-five blows.

These soils are too impermeable to act as base draining materials. Investigation of the degradation effects on the soils indicates the soils are not significantly changed from their original bank composition, with respect to heaving.

Frost Tests:

Frost heaving is the result of interaction of soil and water under freezing conditions. Provided certain conditions exist, the freezing of a saturated soil mass will result in frost heaving. The general conditions most conducive to heaving are:

1. ample water supply - saturation due to high or 'perched' water table, poor drainage.
2. fine grained soil - finer average soil grains mean more likelihood of heaving.
3. slow freezing and uniform temperature will result in the worst heaving situation.

The above are statements of degree. There is no clear demarcation between acceptable and unacceptable conditions. Understanding the three statements and their inter-relationship leads to an understanding of the problem and it is worthwhile to make this relation clear.

If the ground is saturated, a clear possibility of frost heaving exists. The road section would not be saturated unless the base materials were too fine to permit adequate drainage. A coarse soil would be permeable enough to allow the drainage of the water away from the road base. A fine soil has enough capillarity to hold the water that has been admitted through any open cracks or joints. A coarse soil is not frost susceptible; a fine soil is susceptible. A non-susceptible soil is not likely to exist in a saturated condition because of the excellent drainage properties of the material. A frost susceptible soil is likely to be saturated because of its propensity for water. There is a distinct relation between drainage capability and frost susceptibility. The nature of a frost susceptible soil is such that it is most likely to exist under conditions that will encourage heaving.

In the field the mere presence of adverse water conditions is a good indication of frost susceptible soil. The frost heaving and pumping phenomena are warm and cold weather manifestations of a year round water problem.

Tests have shown that there is a wide range of behavior for a given soil, depending on changes in temperature and moisture conditions. The frost phenomenon is the result of the delicate balance of thermal conditions with water conditions. The source of energy for the heaving phenomenon is a suction built up in the soil due to the phase change of the water from liquid to solid - being the latent heat of fusion for water-ice phase change (80 cal/gram). This suction draws available water towards the freezing zone in the soil. The permeability of the soil governs the speed at which water can be supplied to the freezing zone. If temperature conditions are such that water is being frozen (removed) more quickly than it can be supplied, there is a resistance to the build up of ice in the soil and heaving will cease. If water can be supplied easily, the ice will continue to build up indefinitely, as long as a balance is maintained. The frost heave results because the ice freezing in the soil forms as layers parallel to the ground surface. This ice may form as a few thick layers or as a large number of closely spaced microscopic layers which in sum are equal to the single thick lens. The upward growth of the soil due to the ice layers may be quite large - and the time required short. A period of two or three days can account for a large part of the heaving in this case (see calculation D).

Figure one: <u>Cumulative Degree-days</u>

This curve shows the duration of the freeze in January preceding the reported heave. The next year had about the same overall freezing index but no such single cold period.

Figure Two: <u>Mechanical Analysis</u>

Note that the similarity between the I-91 A&B and Glastonbury silt is only in their similar percentages of fine materials. Their frost behavior is very similar. This indicates that the coarse material has only a secondary effect on the frost behavior, if any.

Figure Three: <u>Percent finer than 0.02mm vs. Percent Heave</u>

Following the observation that coarse material has little effect on frost action, a correlation is made between the 0.02 mm percentage and the percent heave observed during the frost tests.

An approximation of the relation is:

% heave = 1.6 (0.02mm %) average

% heave = 2.2 (0.02mm %) maximum

Figure Four: <u>Moisture Content vs. Percent Heave</u>

A distinct difference in the behavior of well graded soils - I-91 A&B - and uniform soils - Glastonbury Silt and Rocky Hill Sand - is shown; there is a higher percentage of water in the silt and sandy soil. This is because of the higher dry unit weight of the well graded soil.

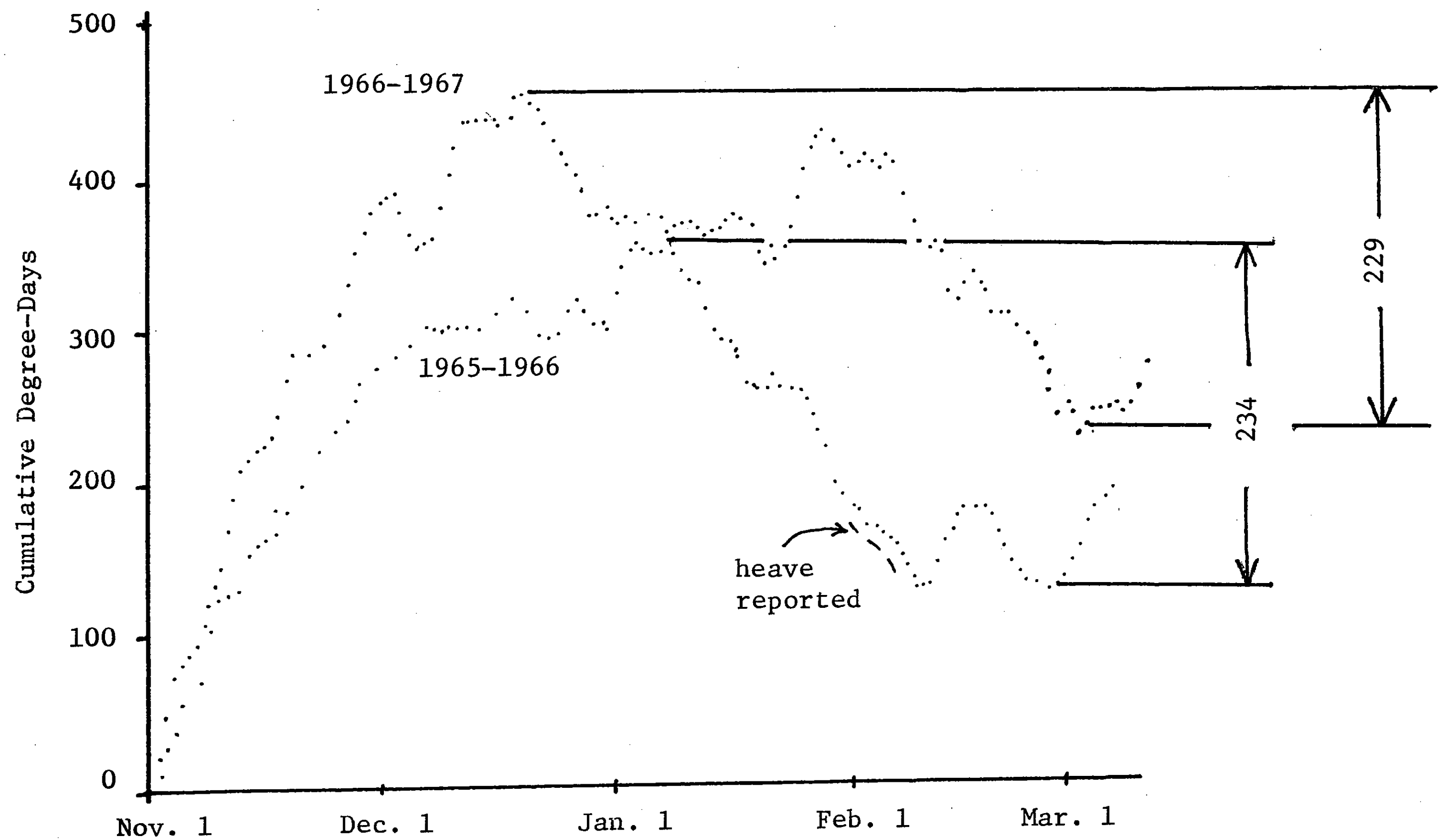


FIGURE ONE: Cumulative Degree-Days showing Freezing Index for 1965-1966 and 1966-1967

FIGURE TWO: Mechanical Analysis

Percent Passing Given Size vs. Particle Diameter

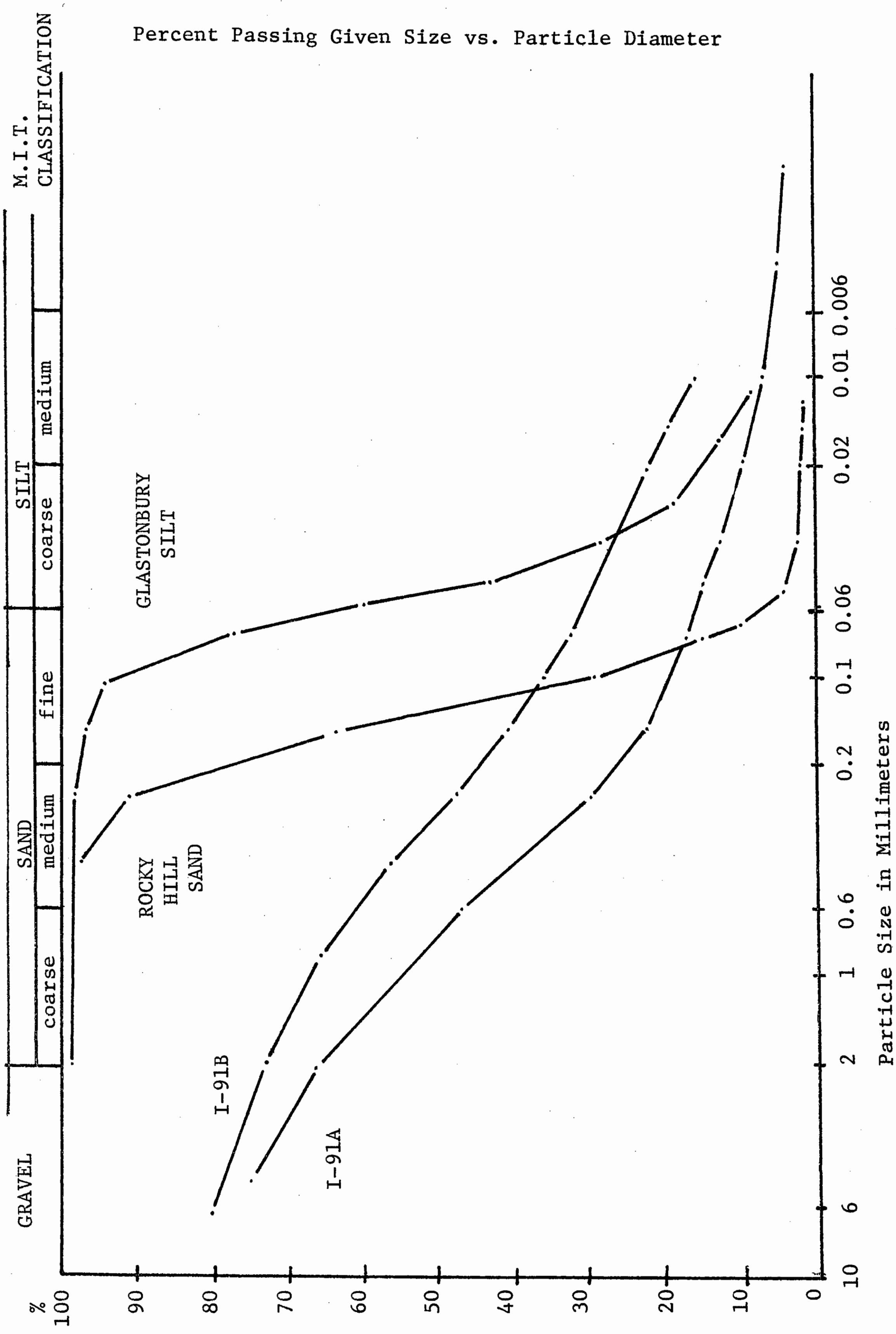


FIGURE THREE: Percent Finer Than 0.02 mm vs. Percent Heave

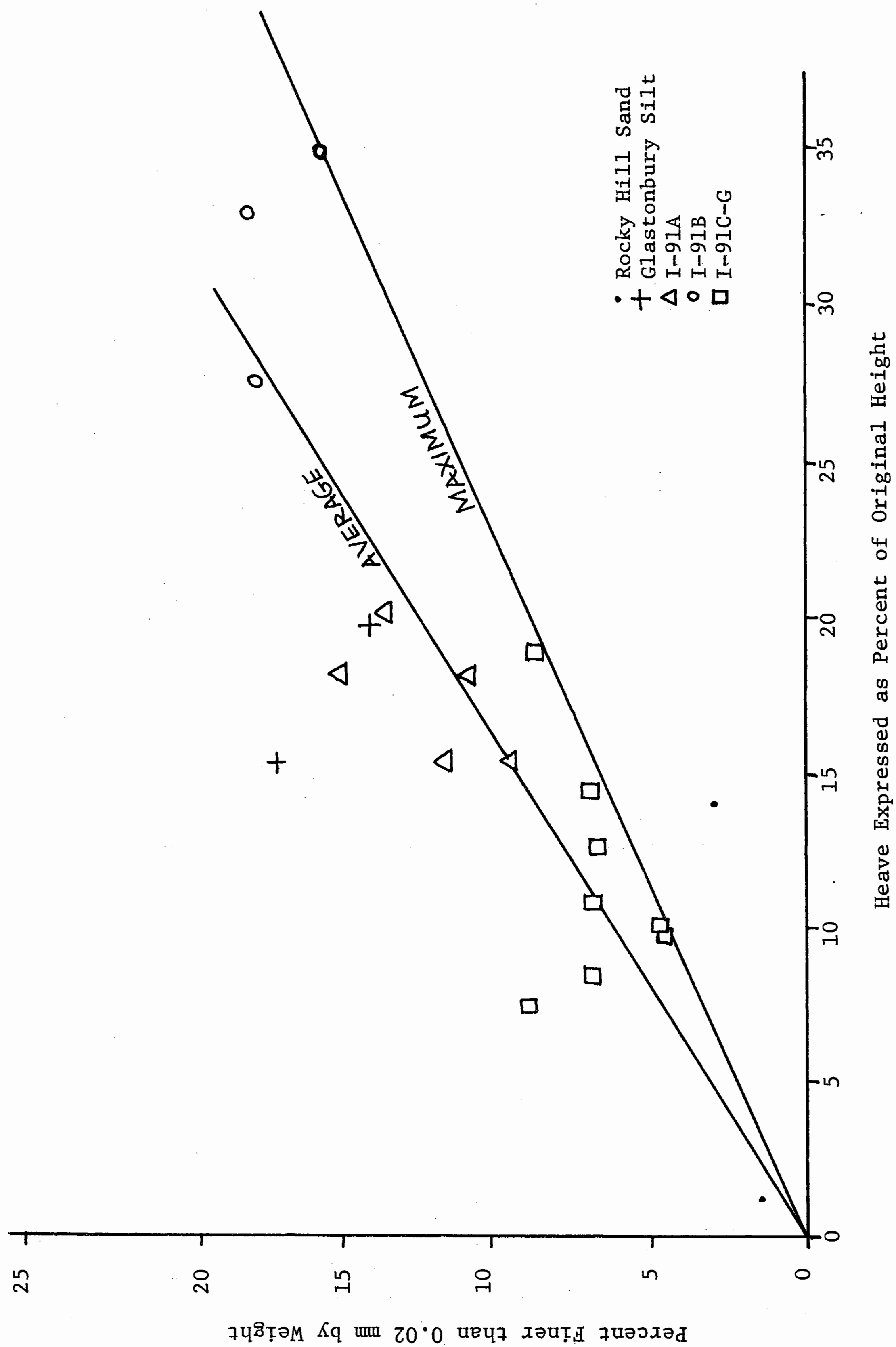


Figure Five: Maximum Rate of Heave vs. Temperature Gradient

This figure is used as the basis of one of the calculations to be performed in the next section. Increasing temperature gradient is seen to cause higher rates of heave. There is also a distinct tendency for higher rates of heave to occur in soils with higher percentages of material finer than 0.02mm.

Using the data which have been presented, calculations of the amount of heave can be made. In the next section, an attempt is made to perform calculations of the amount of heave which could be expected of these soils.

FIGURE FOUR: Moisture Content vs. Percent Heave

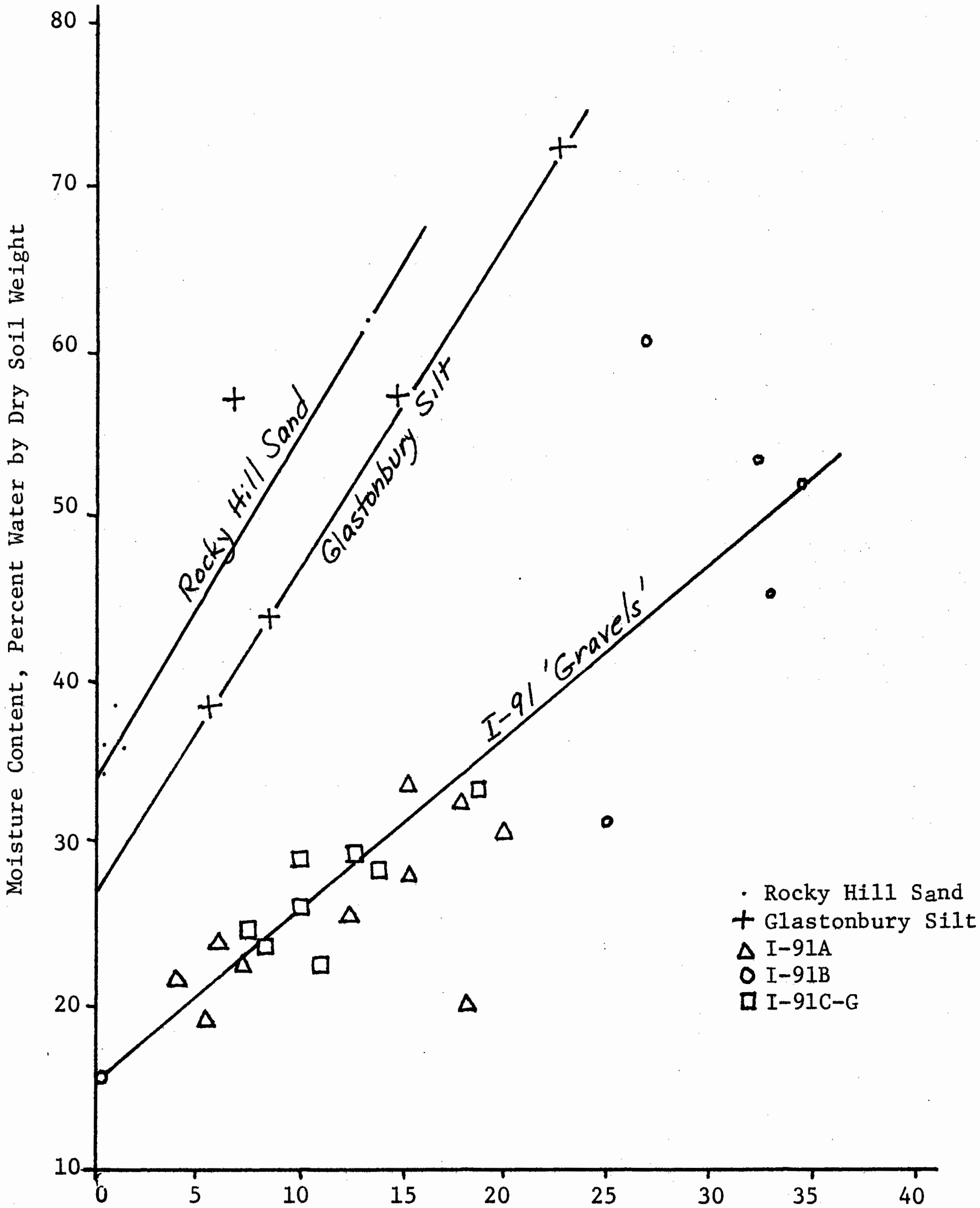
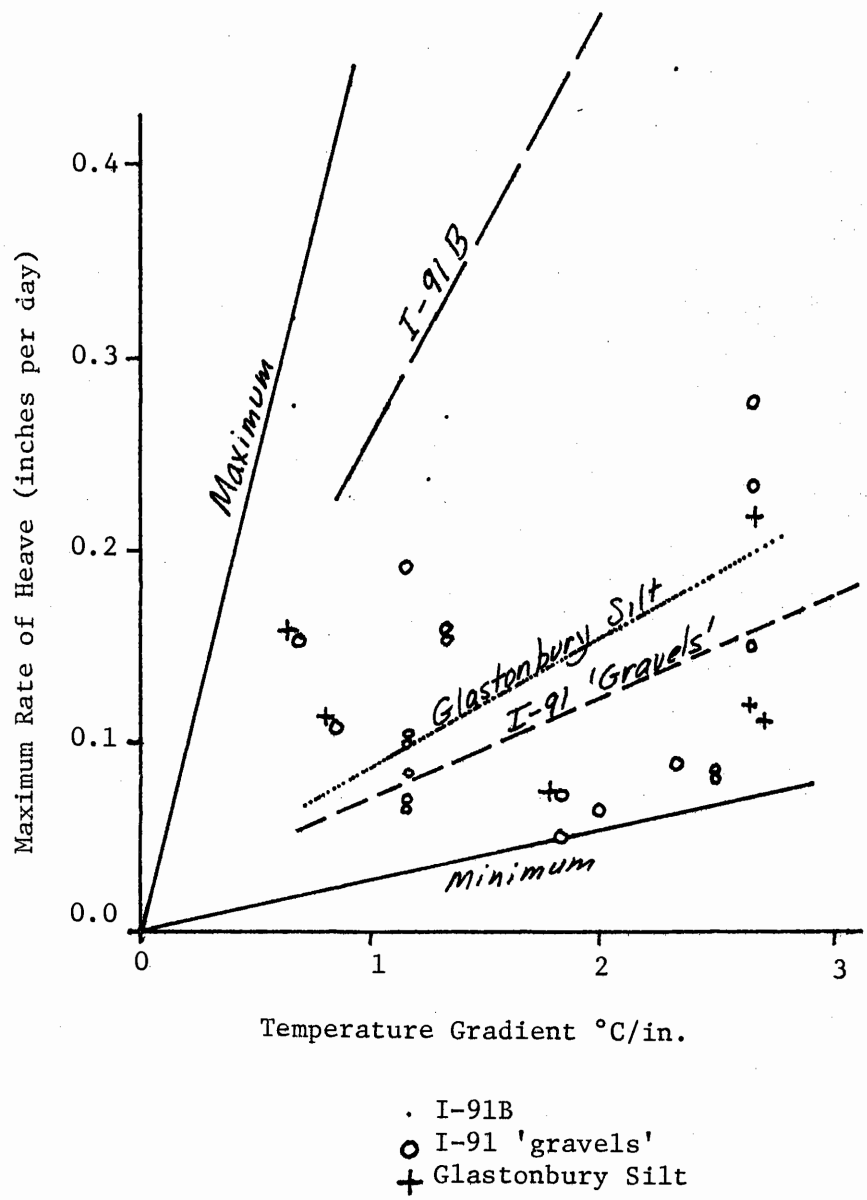


FIGURE FIVE

Maximum Rate of Heave vs. Temperature Gradient



CALCULATIONS

A. This calculation is based on the results of tests presented in Linell, Hennion, Lobacz (3) which categorize the behavior of soils from the different U. S. Corps of Engineers Frost Groups.

1. These soils typically fall in the medium frost susceptibility group. This classification is based on tests performed by the Corps and the classification 'medium' refers to soils which showed average rates of heave between two and four millimeters per day.
2. The weather records show a significant freezing period which extends from January 24, 1966 to February 1, 1966. The freezing period is taken as nine days, 1/24/66 to 2/1/66.

The amount of heave which could be anticipated to have occurred in that nine days is:

minimum: = 2 (mm/da)x9 (da) = 18 mm = 0.709"

maximum: = 4 (mm/da)x9 (da) = 36 mm = 1.42"

The range of values for this calculation are within a reasonable range for the average behavior of the field soils. The maximum observed heave has been cited as three inches. This difference in the maximum may be accounted for by the following:

1. varying soil properties
2. varying water conditions
3. varying temperature conditions

Varying soil properties - The calculations made above are based on the average soil encountered in the field. The tests performed on these soils show that for very subtle differences in the soil conditions, large variation in the heaving behavior can be encountered.

Typically the soils were of the F2 group, but in all probability there are small local areas where the soils would fall in the F3 group, which would show higher frost activity.

Varying water conditions - The Corps of Engineers frost test is conducted with a water table about six inches below the soil surface. If water was more readily available in the field, the heaving could be expected to be greater than indicated by the calculations.

Varying temperature conditions - Once the freezing face penetrates to the bottom of the concrete pavement, the shoulder and the roadway will rise together. For this reason the extremes of temperature will have no worse effect on the differential heaving than will the mild freeze. Because the shoulder and concrete pavement will rise together once the frost is deep, the effect of variations in temperature is limited.

If the material under the concrete is different than that under the shoulder and not frost susceptible, differential heaving would continue as the depth of frost passed the bottom of the concrete pavement. This would account for low predicted heave magnitudes.

The discussion of the variables which has just been presented applies to all of the calculations in this section.

B. This calculation is based on figure seven, Percent Heave vs. Percent Finer Than 0.02 mm. It is based on tests performed on the actual soils.

1. The soils were observed to heave between 7.4% and 34.8% of their unfrozen height.
2. The frozen height of the soil is the layer of soil beneath the asphalt shoulder paving and the bottom elevation of the concrete slab. This thickness is between 5.5" and 6.5", considering variation in the three inch asphalt layer and the nine inch concrete slab.

The amount of heave which could be expected to have occurred is:

minimum: = .074 x 5.5" = 0.406"

maximum: = .348 x 6.5" = 2.26"

C. This calculation is also based on figure seven; the percent heave is determined from the observed percent-finer-than-0.02 mm in the soils rather than being taken directly from the tests, as in the previous calculation.

1. The percent finer than 0.02 mm lies between 4.9% and 18.0% based on hydrometer tests run on the soils tested.
2. The frozen depth of soil contributing to the differential heave is taken as 5.5" to 6.5" as was discussed in calculation B.

Based on the data presented in figure seven, the relation between the 0.02 mm percentage and the amount of heave expressed as a percent is:

average percent heave = 1.6 (.02 mm percentage)

maximum percent heave = 2.2 (.02 mm percentage)

The amount of heave which can be expected is the percent heave times the frozen depth of soil or:

average heave = 1.6 (.049)(5.5") = 0.431 ins.

maximum heave = 2.2 (.396)(6.5") = 2.57 ins.

Inasmuch as this is the closest estimate so far for the maximum heave, it can be deduced that the average test tends to represent a less critical condition than the maximum field conditions.

D. This calculation is a day by day summation during the freezing period of the daily heaving, determined from figure eight, using the maximum and minimum curves as shown in the figure.

1. The soil at a depth of nine inches is assumed to be at the monthly average temperature.
2. The soil at the bottom of the asphalt is assumed to be at the daily average temperature.
3. The gradient of temperature through the soil is assumed to be constant.
4. The gradient equals the daily average temperature minus the monthly average temperature, divided by the soil thickness which is six inches.
5. The total heave equals the daily average rate of heave (x 1 day) summed for each day during the freezing period.
6. The freezing period is assumed to be the period from 1/24/66 to 2/2/66.

TABLE NUMBER ONE

TABULATED DAILY GRADIENT

DAILY AVE. °F	MONTHLY AVE. °F	DIFF. °F	GRADIENT °F/in.	GRADIENT °C/in.	DATE
- 2	-4.2	-	-	-	1/24/66
- 8	"	3.8	0.632	0.351	25
-12	"	7.8	1.30	0.721	26
- 8	"	3.8	0.632	0.351	27
-19	"	14.8	2.47	1.37	28
-19	"	14.8	2.47	1.37	29
- 6	"	1.8	0.300	0.167	30
-13	"	9.8	1.63	0.906	31
- 7	"	2.8	0.466	0.259	2/ 1/66
- 5	"	0.8	0.133	0.074	2

TABLE NUMBER TWO

TABULATED CALCULATION D.

GRADIENT (°C/in.	MINIMUM HEAVE RATE ("/da.)	MINIMUM HEAVE AMOUNT (ins.)	MINIMUM HEAVE CUMULATIVE (ins.)	MAXIMUM HEAVE RATE ("/da.)	MAXIMUM HEAVE AMOUNT (ins.)	MAXIMUM HEAVE CUMULATIVE (ins.)
-	-	-	-	-	-	-
0.351	0.011	0.011	0.011	0.164	0.164	0.164
0.721	0.020	0.020	0.031	0.340	0.340	0.504
0.351	0.011	0.011	0.042	0.164	0.164	0.668
1.37	0.037	0.037	0.079	0.616	0.616	1.284
1.37	0.037	0.037	0.116	0.616	0.616	1.900
0.167	0.007	0.007	0.123	0.075	0.075	1.975
0.906	0.024	0.024	0.147	0.408	0.408	2.383
0.259	0.009	0.009	0.156	0.120	0.120	2.503
0.074	0.003	0.003	0.159	0.034	0.034	2.537
	minimum =		0.159"	maximum =		2.537"

TABLE NUMBER THREE

TABULATION OF THE HEAVE CALCULATIONS

CALCULATION	MINIMUM HEAVE	MAXIMUM HEAVE
A	0.709"	1.42"
B	0.406"	2.26"
C	0.431"	2.57"
D	0.159"	2.54"

The fact that the observed maximum heave was larger than the maximum calculated above indicates that the test conditions were somewhat different than in the field. It may also be that the assumed freezing period was too short. The water table in the laboratory was only two inches below the surface - it is hard to imagine the field water conditions as being worse than this. The magnitude of the temperature conditions is fairly well known. The difficulty in fixing the exact freezing period along with possible variation in soil properties account for the low calculated values for the maximum heave.

No attempt is made to offer these calculations as rigorous mathematical operations but, it is interesting to note that each of the procedures gives similar results. This indicates that the heaving in the field is a predictable phenomenon.

While it is true that the use of good soils and good drains represents added initial cost, the ideal of a long-lived, low maintenance roadway is well worth the initial investment.

Conclusions:

The soils should not have been used.

a) They did not meet the existing specifications.

b) They are frost susceptible.

c) They are impervious.

The soils were initially accepted because of poor dry sieving technique. Dry flocs of small particles were not sufficiently broken down.

The acceptance of soils that do not meet the specifications, through poor technique, points up the need for change in the specification.

The heaving was due to the heaving tendencies of the soils, in combination with readily available water. There would have been ample water available even without the open shoulder joint.

Recommendations:

With respect to the occurrence of heaving:

a. Locate and mark all trouble spots - both pumping and heaving.

b. These areas should be watched and as damage becomes apparent in break-up, the shoulders should be opened and the poor material should be replaced with acceptable material extending clear to the side drains.

c. Consideration should be given to the installation of lateral drains on the longer grades where it is apparent that "ponded" water at the base of the grade is causing Water damage.

With respect to future recurrence in other construction:

a. The dry sieving technique is inadequate to establish the true percentage of fines in a soil, although the present gradation specification is realistic. Wet sieving should be used in all cases.

b. The following specification is suggested:

   1. Wet sieving shall be performed on all samples to determine the particle size distribution

   2. Hydrometer analysis is required when the percentage of fines (#100 sieve) is greater than or equal to eight percent. The allowable percentage of material finer than 0.02 mm, shall be no greater than three percent.