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16. Abstract

This study compares the benefits during construction of a lime-modified embankment to an unmodified embankment and after the placement of subsequent pavement layers. Adverse weather conditions did not exist during construction to create the type of observations desired. Deflections, on the average, were lower and more uniform on the lime-modified embankment than on the unmodified embankment. Deflections on the CAM subbase were equal, but those on the lime-modified embankment again were more uniform. With the addition of the 10-inch pavement, no difference in average deflection or in uniformity of deflection could be detected. The subbase on the lime-modified embankment had fewer shrinkage cracks per unit length than did the subbase on the unmodified embankment. The overall smoothness of the pavements was equal for both types of embankment.

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DEPARTMENT OF TRANSPORTATION
Division of Highways
Bureau of Materials and Physical Research

LIME-MODIFIED SOIL TO INCREASE SUBGRADE STABILITY

by

R. J. Little

Final Report
Research Study IHR-409

A Research Project Conducted by
Illinois Department of Transportation
in cooperation with
U.S. Department of Transportation
Federal Highway Administration

May 1983

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LIME-MODIFIED SOIL TO INCREASE SUBGRADE STABILITY

INTRODUCTION

Lime added to Illinois soils can improve their engineering properties. Previously, lime has been used extensively to stabilize soil in known problem areas. This study is directed at modifying the embankment to increase subgrade stability over a large area to determine if the improvement is sufficient to justify statewide use in subgrade construction. An improvement may take one or more forms, such as improved support for paving equipment, less down time due to inclement weather, less difficulty in obtaining subbase density, and improved pavement support from the subgrade.

Lime-reactive soils are those soils that react with lime to improve their engineering properties, particularly for highway construction. Much research has been conducted in the field of lime-reactive soils and the types of lime used. Two reports of interest to this study, which cover in depth the subject of lime-reactive soils in Illinois, are The Significance of Soil Properties in Lime-Soil Stabilization[1] and Lime Stabilization of Soils for Highway Purposes[2].

OBJECTIVES

The general objective of this study is to obtain data needed to determine whether statewide use of lime modification in subgrade construction would be beneficial.

Specific objectives are:

(1) to determine the relative benefit lime modification has in subgrade compaction and stability

(2) to observe and to document any benefit lime modification affords the contractor during subbase construction

(3) to determine whether the lime-modified subgrade improves the stability of the subbase

(4) to determine whether the pavement derives any immediate and any long-term benefit from the lime-modified subgrade

(5) to determine whether there is a difference in difficulty of installing edge drains in the lime-modified subgrade

EXPERIMENTAL FEATURES

The study site is located on Interstate 55 near Dwight, Illinois. The construction designation is FAI Route 55, Sections 32-3(A,B-1,B-2), Project I-55-5(79)215. The project starts at the Grundy-Livingston County line, Station 304+46, and extends into Grundy County to Station 462+00, for a total length of 15,754 feet.

The experimental feature specifically involves the subgrade under the northbound pavement and paved shoulders. The portion modified with lime was the top 12 inches of embankment at a width of 42 feet. A good portion of both embankments under the northbound and southbound pavements was constructed in 1978. Part of the southbound embankment was constructed to grade in 1978. The remaining portion of the southbound embankment and all of the northbound embankment were completed to grade in 1979. The 4-inch CAM subbase and 10-inch continuously reinforced pavement were constructed in 1980.

The embankments were constructed mostly from borrow out of two pits located on the job site. They are identified as Roeder and Danker pits. Analysis of the soil from the pits was made by the local district. The textural class of the borrow pits was identified in the field as clay. Hydrometer test results are summarized in Table 1.

The textural class of the B horizon is similar to C; therefore, the design lime content for C_1 should be effective in modifying the B horizon soil mixed in with the C_1 and C_2 tills in the upper 12 inches of the subgrade. The top 12 inches of the northbound embankment was constructed with soil from Roeder pit. While most of this soil came from the C_2 horizon, some of the C_1 horizon soil was used. In the southbound embankment, the top 12 inches from Station 304+46 to Station 370+00 was constructed with soil from Danker pit, and from Station 370+00 to Station 462+00 the soil came from Roeder pit. Because of the close proximity of the two pits it was decided that duplication of tests was not necessary and the engineering properties would not change appreciably in either the vertical or horizontal direction. Therefore, physical tests were conducted on only the C_1 and C_2 horizons of the Roeder pit.

Preliminary Tests

Polyhydrated lime from the same source as that used on the job was used in all tests. A pH of 12.3 is needed for pozzolanic reaction between the lime and the aluminates and silicates in the soil. Figure 1 shows the pH for C_1 and C_2 horizon tills with increasing percent of lime. The curve is flat at 11 percent because the limit of the pH test strip was 11. However, a pH rise from 6 to 11 with 2 percent indicates that strength gain from chemical reaction is probable.

Summarized in Table 2 are the Mechanical Analysis, Plasticity Index, Standard Dry Density, Textural Class, and AASHTO Classification of the tills in the Roeder pit.

TABLE 1. SOIL CLASSIFICATION BY PIT AND HORIZON

BORROW PITS

ROEDER	DANKER	HORIZON	DEPTH
C A-7-6(26)	SiC A-6(16)	B	1.5 - 3.0
Clay Till A-7-6(12)	Clay Till A-6(12)	C_1	3' - 9'
Clay Till A-6(11)	Clay Till A-6(12)	C_2	9' - 48'

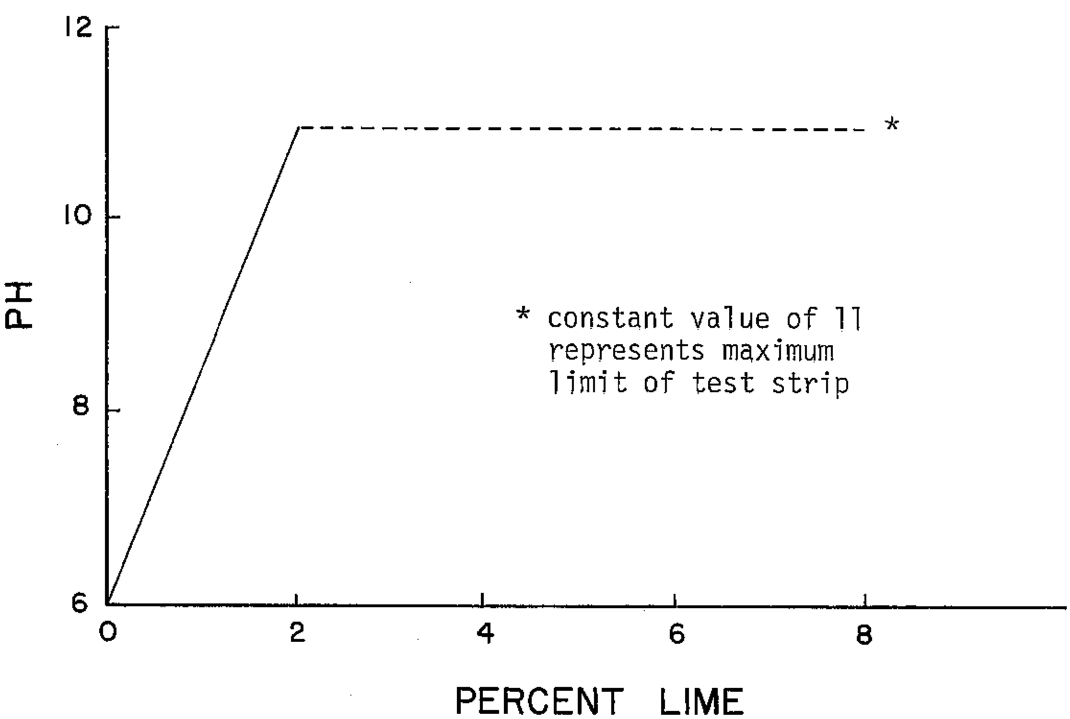

Figure 1. Soil pH versus lime content for C_1 and C_2 horizon tills

TABLE 2. MECHANICAL ANALYSIS OF SOIL

ROEDER BORROW PIT

SAMPLE C_1 DEPTH 3' - 9'		SAMPLE C_2 DEPTH 9' - 48'	
Clay Till A-7-6(12)		Clay Till A-6(11)	
% Sand	10.0	% Sand	13.6
% Silt	37.5	% Silt	38.4
% Clay	52.5	% Clay	48.0
LL	40.0	LL	35.4
PL	18.8	PL	17.0
PI	21.2	PI	18.4
Standard Dry Density: 108.4 lb/cu ft @ 17.7% H_2O; 105.3 lb/cu ft @ 19.2% H_2O @ 5% lime		Standard Dry Density: 116.1 lb/cu ft @ 14.7% H_2O; 109.4 lb/cu ft @ 17.4% H_2O @ 5% lime	

To determine if the C_1 and C_2 soils are reactive with lime, a 5 percent lime-soil proctor curve was developed for both soils. From this optimum moisture curve the moisture content was adjusted up or down 1/2 percent for each 1 percent increase or decrease in lime content.

Due to a limited amount of sample material, three cylinders were made for unconfined compression tests instead of four. Three 2-inch by 4-inch cylinders were made for unconfined compression tests at lime contents of 0, 2, 4, 6, and 8 percent. The cylinders were cured for 48 hours and each specimen tested to failure. The average compressive strength of the specimens was compared to the strength of the untreated specimens. A strength gain of the treated specimens over the untreated specimens of 50 PSI indicates a reactive soil. Both soils had such a gain, and a plot of the average compressive strength for the C_1 and C_2 versus the lime content is seen in Figure 2. Based on these tests, the approximate amount of lime to be used was set at 4 percent.

CONSTRUCTION

Work on the lime-modified top 12 inches of the northbound embankment started on July 9, 1979. This top 12 inches was to be constructed in two 6-inch lifts. The soil for the first lift had been placed prior to this date.

The lime was trucked to the job site and transferred to the distributor truck for placement on the embankment. The distributor truck had three augers that conveyed the lime to canvas tubes, which allowed the lime to drop on the embankment with a minimal amount of dusting. Construction was done by blocks because the lime arrived in truckloads.

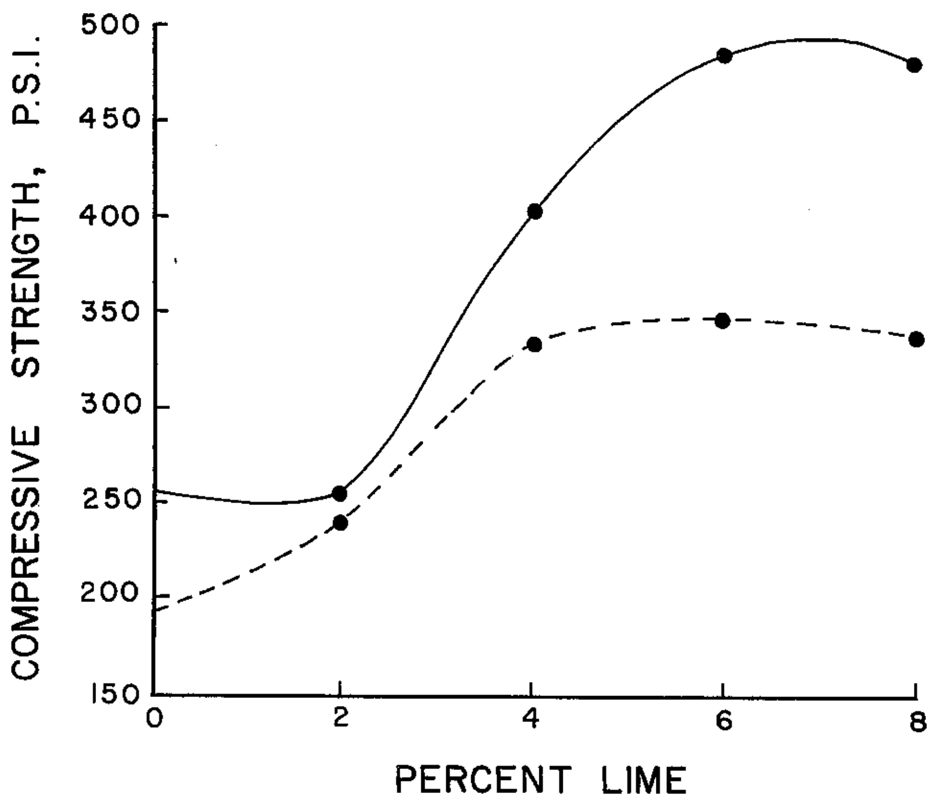

Figure 2. Plot of compressive strength versus lime content

The block lengths varied, depending on the amount of lime on the truck and the thickness of the soil in place to be modified.

Figure 3 shows the method of transferring the lime from the tanker to the distributor truck. Figure 4 is a view of the canvas tubes used to place the lime on the embankment and Figure 5 shows an overall view of lime being placed on the embankment.

When a load of lime arrived the contractor checked the load ticket for weight. He then used a table prepared by the Resident Engineer which gave the limits of the embankment to be treated based on the weight of the load. Laths were used to set the limits for that load, and spreading operations started.

After the dry lime was placed, water trucks moved in to wet the lime prior to mixing with the soil. Normally, two water trucks were used to wet the lime. The number of loads ranged from 1 1/2 to 6 per block. Water was applied until the lime was well saturated. Some lime was lost due to over-watering, causing the lime to be washed off the embankment. Figure 6 shows a typical condition of the lime prior to mixing.

Mixing was done at first by a tiller. The tiller did an excellent job of mixing the lime and soil, but was slow and broke down twice during the first three days. During the third day of operations the contractor switched to a disc to mix the lime and soil. As a comparison, the production rate from the tiller was slightly less than 3 feet per minute and the rate for the disc was slightly more than 4 feet per minute for the 42-foot-wide embankment. Figure 7 shows the tiller used and Figure 8 is a close-up view of the tiller blades.

Figure 3. Transport tanker loading distributor truck
for placement of lime on embankment

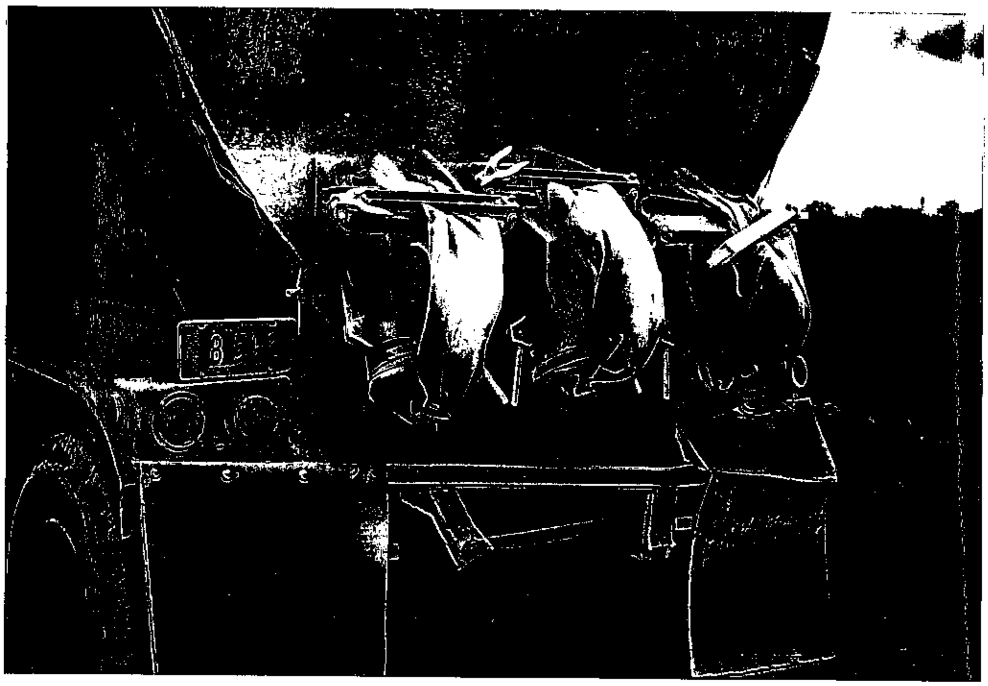

Figure 4. Rear end of distributor truck

Figure 5. Lime being placed on embankment

Figure 6. Water truck on embankment at completion of wetting operation

Figure 7. Tiller used at beginning of construction

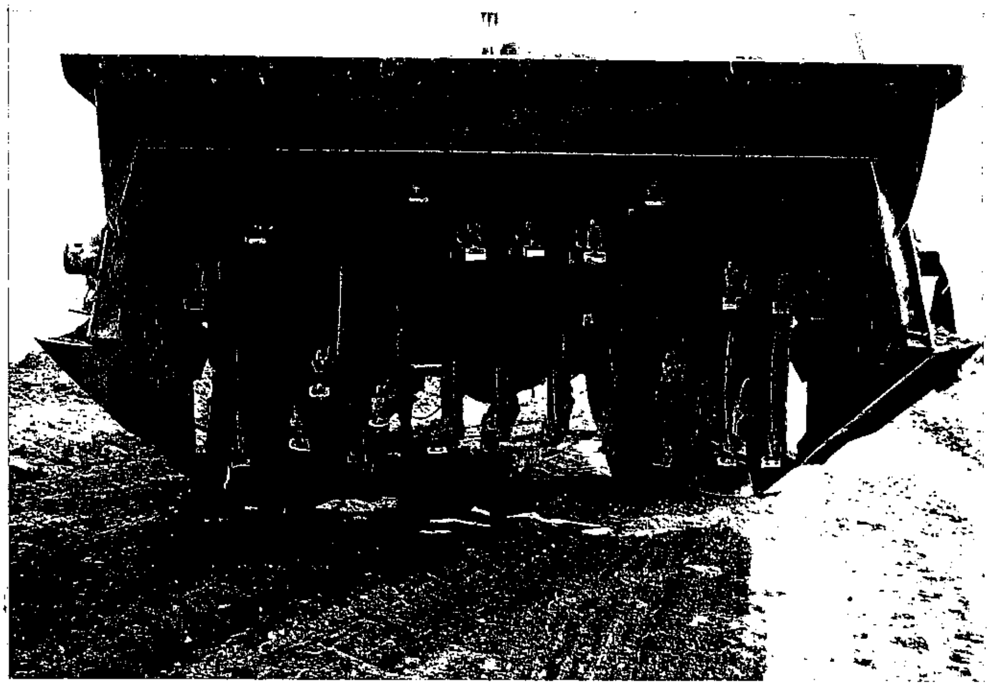

Figure 8. View of tiller blades

One pass by the tiller was sufficient to obtain mixing. The tiller overlapped about 1 foot to assure complete coverage. Discing had no set pattern and was continued until the inspector was satisfied that mixing was complete. Discing was confined to one block until the next block was ready. At that time the disc would work two or three blocks at a time. This procedure was helpful because it eliminated turning around at the beginning and end of each block. Mixing by discing in the turnaround areas is not accomplished as easily as when the disc is operating in a straight line.

Due to the nature of the reaction of lime and soil, the process cannot be accelerated. Concentrated discing in one block did not produce a better-mixed soil and lime faster than when two or more blocks were disced as one. Therefore, discing two or more blocks at a time was beneficial. Figure 9 is a picture of the disc.

Rolling was done by a set pattern. In general, the roller would go up and back in the same track and then move over approximately 3 feet. This pattern was continued until the block was compacted. When compaction was completed on each block, the blade on the roller was dropped and the block was backbladed. Figure 10 is a picture of the roller.

In general, the same construction procedure was followed for each lift. The soil for the first lift was placed as much as several weeks before the lime was added. This created two problems. First, the soil was very dry and this required more water than if the soil had been placed a few hours before mixing with lime. The second problem was in mixing. At some locations, where construction traffic was heavy, the soil had to be

Figure 9. Disc used for majority of study

scarified to aid the disc in breaking up the soil for mixing. Figure 11 is a picture of a motor grader scarifying the soil.

Construction of the lime-modified portion of the embankment started on Monday, July 7, 1979, and was completed on Friday, July 27, 1979. During that period there were two rains. The first, on Friday night and Saturday, July 13 and 14, amounted to 1.15 inches and came during the construction of the first lift.

Blocks 4 and 5, constructed late Friday afternoon, were not completely rolled. On Monday morning, rolling was completed on these two blocks without difficulty. The roller operator was surprised that he could roll those blocks after the rain, and doubted that he could have rolled the embankment in a similar situation without the lime.

The second rain came late on Tuesday, July 24 and early on Wednesday, July 25 in the amount of 2.7 inches during the construction of the second lift. All construction was stopped on Wednesday, July 25 and resumed on Thursday, July 26. Figure 12 shows standing water caused by a windrow of loose material at the edge of the modified embankment. There was more standing water on the modified embankment than on the unmodified embankment due to the windrows.

A comparison of the most extreme conditions can be seen in Figures 13 and 14. These pictures were taken about 36 hours after the rain stopped. The lime-modified embankment provided a firm roadbed, while the unmodified embankment had areas that were nearly impassable.

Controlling the depth of mixing the soil and lime was fairly easy with the tiller. The depth of mixing with the disc was not quite as controllable. In order to determine the depth of mixing, an indicator was

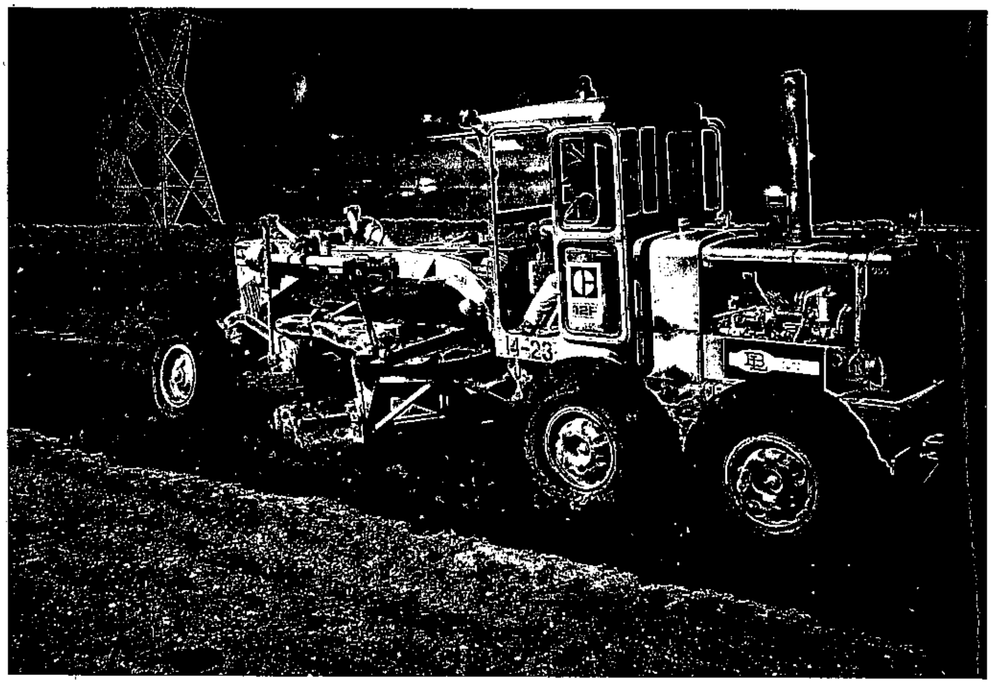

Figure 11. Motor grader scarifying soil

Figure 12. Water standing on lime-modified embankment after 2.7-inch rain

used. The indicator was a mixture of phenolphthalein and alcohol. When the mixture comes in contact with lime the result is a marked color change to a light purple. By carefully digging a hole in the mixed soil and using the indicator, the depth of mixing could be determined. A picture of this method is seen in Figure 15. Several locations were tested the first two days after the disc was used in order to be sure that mixing was deep enough. After the first two days of using the disc, one location per block was selected to check mixing depth. A mixing depth of 9 inches to 10 inches before compaction was not hard to accomplish.

Mixing efficiency was determined from one or more samples taken each day. The samples were delivered to the District 3 soils laboratory for testing. The mixing efficiency is defined as the field mix q_u (unconfined compressive strength) divided by the lab mix q_u times 100. The mixing efficiency for the samples are listed in Table 3.

The mixing efficiency ranged from a low of 69 percent to a high of 103.4 percent. The average of 89 percent is well above the 70 percent figure considered as a good mixing efficiency.

Compaction of the lime-modified soil presented no problem. All but one block passed the required density test with the rolling pattern established. Additional rolling of the one block the next day brought the density up to the passing range.

During construction of the first lift, 37 nuclear density tests were made. The average was 101.1 percent of standard dry density as determined by the Proctor method, with the lowest of 93.1 percent and the highest of 106.4 percent. The second lift had an average of 98.9 percent of maximum, with a range from 95.2 percent to 104.2 percent. For comparison, the

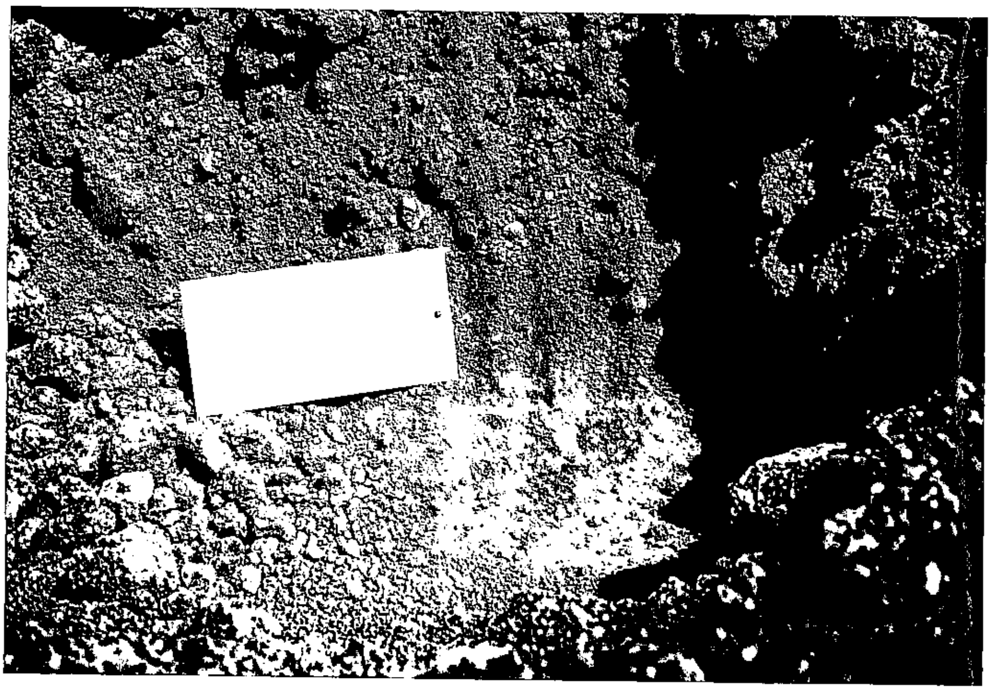

Figure 15. Indicator showing depth to which lime was mixed

TABLE 3. MIXING EFFICIENCY

SAMPLE NO.		EFFICIENCY
1		79.8
2		90.6
3		98.6
4		87.7
5		75.8
6		86.7
7		87.8
8		94.0
9		93.4
10		88.9
11		92.7
12		69.0
13		91.3
14		81.5
15		94.6
16		103.4
17		86.2
18		93.3
19		97.8
	Average	89.0

average density at completion of the unmodified embankment was 107.8 percent of the standard dry density.

When fine grading of the embankment was completed in May 1980, another set of density tests were made. These tests showed that, on the average, the density of the lime-modified embankment increased by 1.3 percent while the unmodified embankment was reduced by 1.7 percent. There is no explanation for why this happened, except that construction traffic may have been heavier on the modified embankment.

Before fine grading was started, pictures of each embankment were taken. Figure 16 shows the lime-modified embankment and Figure 17 shows the unmodified embankment at the same time. Both pictures were taken from about 4 feet above the embankment. The circle scratched on the surface, for reference, is about 12 inches in diameter. The number and width of the cracks point out the difference in shrinkage between the two embankments.

FIELD TEST

In order to determine the stability of the embankment, a series of Road Rater deflection tests were planned. Deflection measurements were made on both embankments at the completion of each stage of construction. Table 4 lists the dates, applied load, and stage when deflection measurements were made.

The first three series of Road Rater measurements were made on the embankment. In each case, as seen in Figures 18, 19, and 20, the modified embankment had the least variability of deflections and the lowest overall average. The areas of higher deflections are fairly well defined in the unmodified embankment.

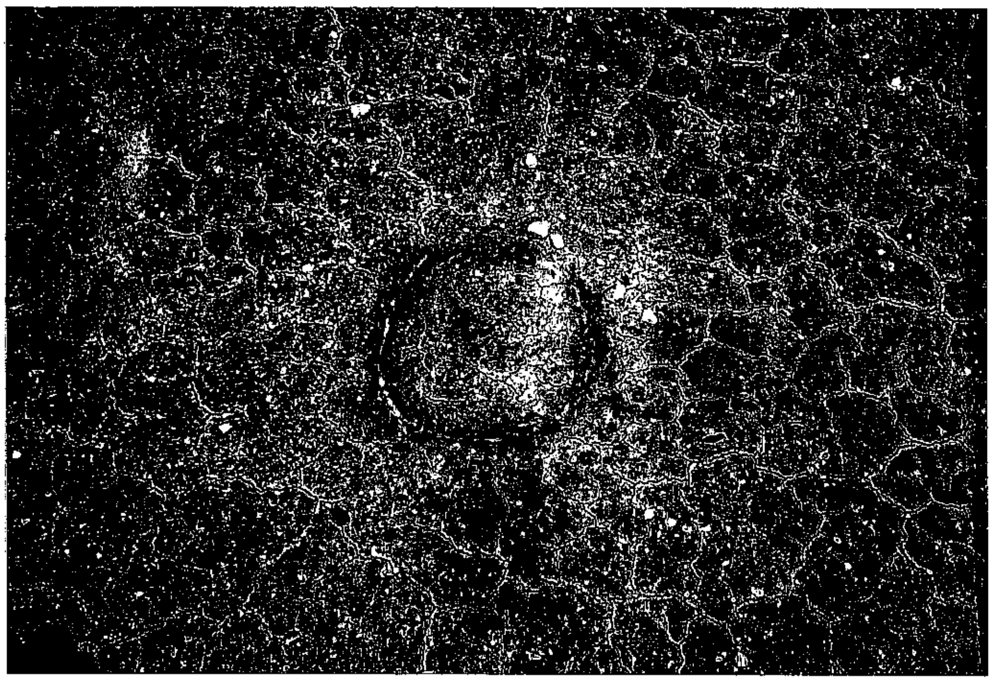

Figure 16. Shrinkage cracks on lime-modified embankment

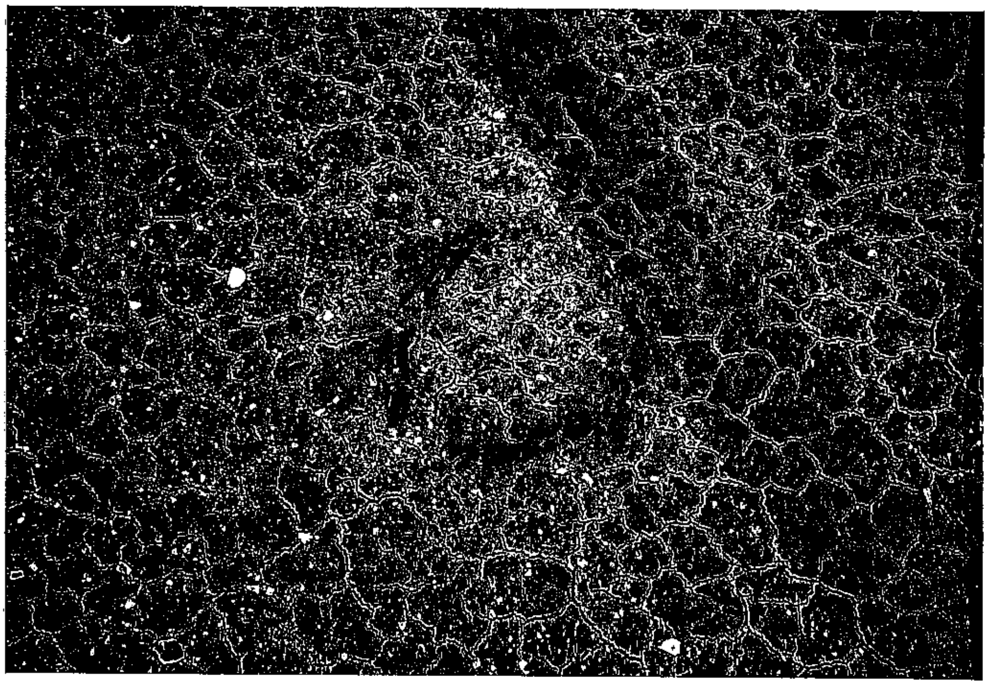

Figure 17. Shrinkage cracks on unmodified embankment

TABLE 4. DATES OF ROAD RATER DEFLECTION MEASUREMENTS

DATE	APPLIED LOAD (pounds)	STAGE OF CONSTRUCTION
09-17-79	2000	Completion of embankment
04-23-80	2000	Prior to fine grading
05-15-80	2000	After fine grading
06-04-80	2000	Completion of CAM subbase
10-31-80	8000	Fall deflection on 10-inch CRC pavement
04-14-81	8000	Spring deflections on pavement
10-07-81	8000	Fall deflections on pavement
04-23-82	8000	Spring deflections on pavement

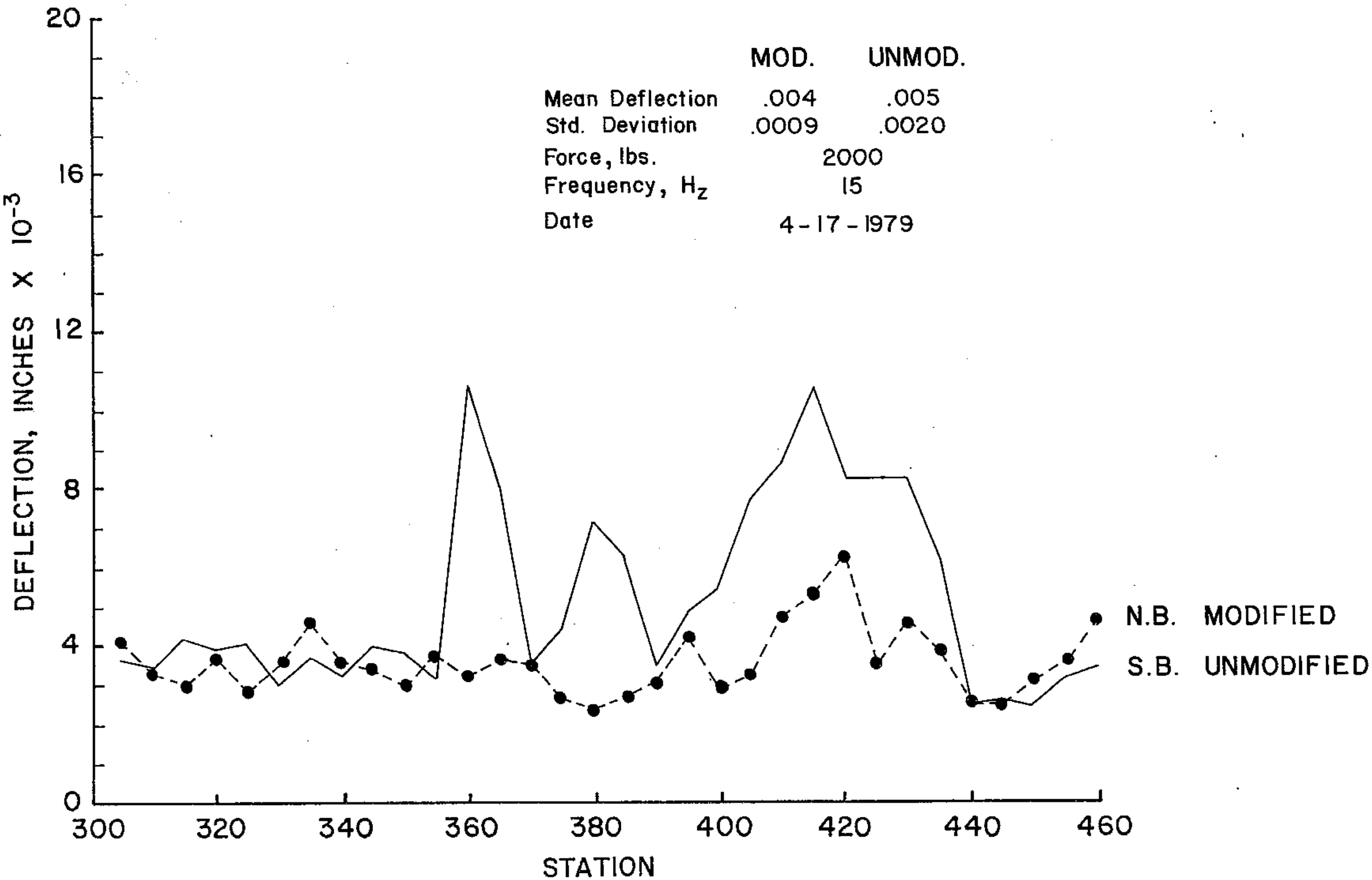

Figure 18. Plot of deflection measurements taken at completion of embankment

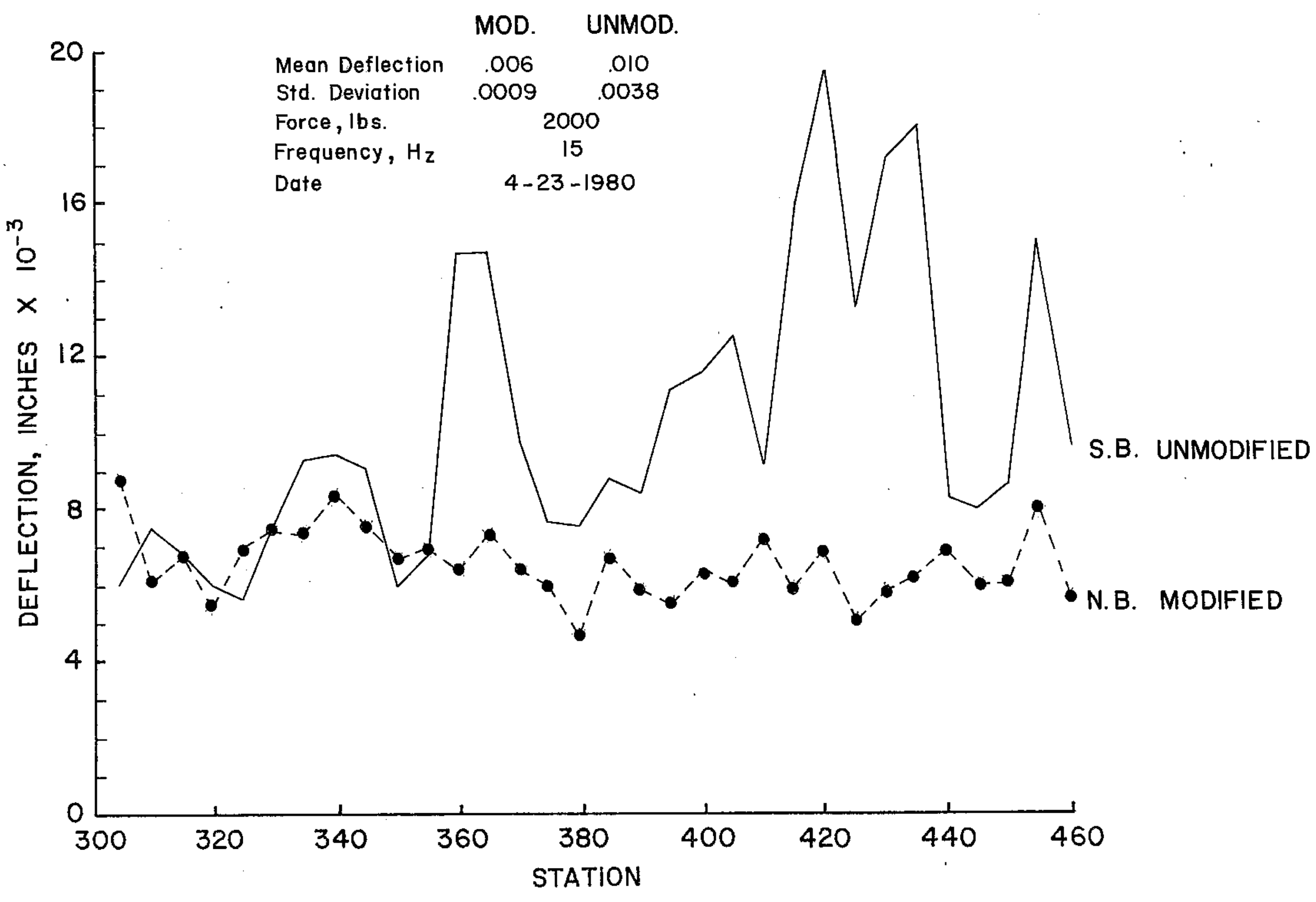

Figure 19. Plot of deflection measurements prior to fine grading

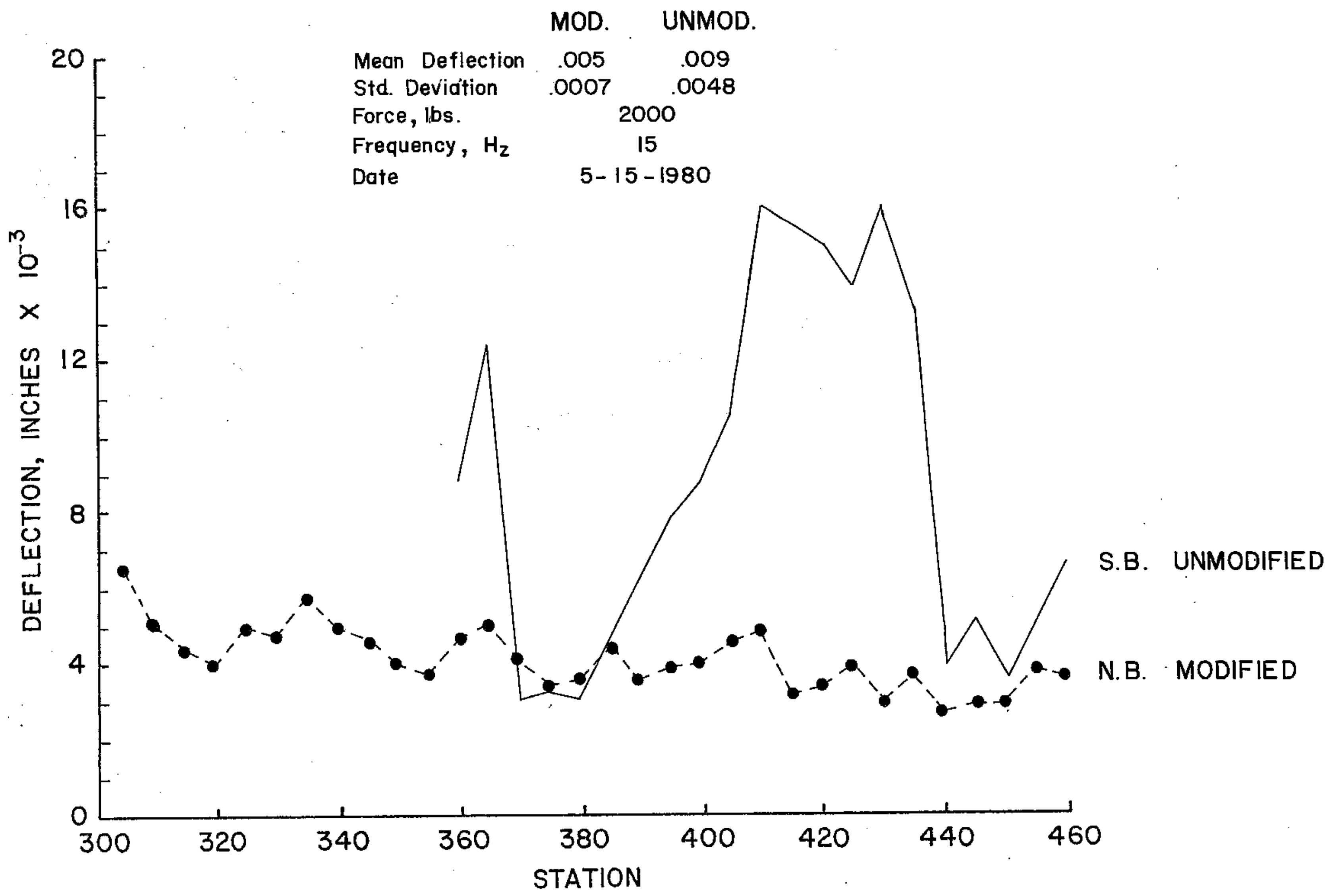

Figure 20. Plot of deflection measurements after fine grading

Figure 21 is a plot of the deflection measurements made on top of the CAM. With the addition of the 4 inches of CAM, the average deflection for both embankments is equal, with a slightly higher variability occurring on the unmodified embankment.

Figures 22, 23, 24, and 25 are the plots of two fall and two spring sets of deflection measurements made on the completed 10-inch CRCP. The force for these deflections was increased to 8,000 pounds. The difference in deflection and variability is not significant.

Two sets of deflections were measured on the 10-inch-thick bituminous concrete shoulders. The fall deflections, seen in Figure 26, have equal mean values, with the unmodified embankment having a slightly higher variability. The following spring deflections, represented in Figure 27, show a marked difference, with the modified embankment having the highest mean deflection and much higher variability. The reason for the higher spring deflections and variability in the modified embankment is unknown at this time.

Trenching Observations

In order to determine if trenching was more or less difficult in the lime embankment, several hundred feet of the tiling operation in both embankments was observed. The machine was operated in the same gear ratio and at the same throttle setting for both embankments. The rate of operation in the modified embankment was 59 feet per minute, and in the unmodified embankment it was 46 feet per minute. The tile machine operator stated that he could not tell the difference in the operation of the tile machine on either embankment until it rained. The operator's observation then was that immediately after a rain, which occurred while trenching both

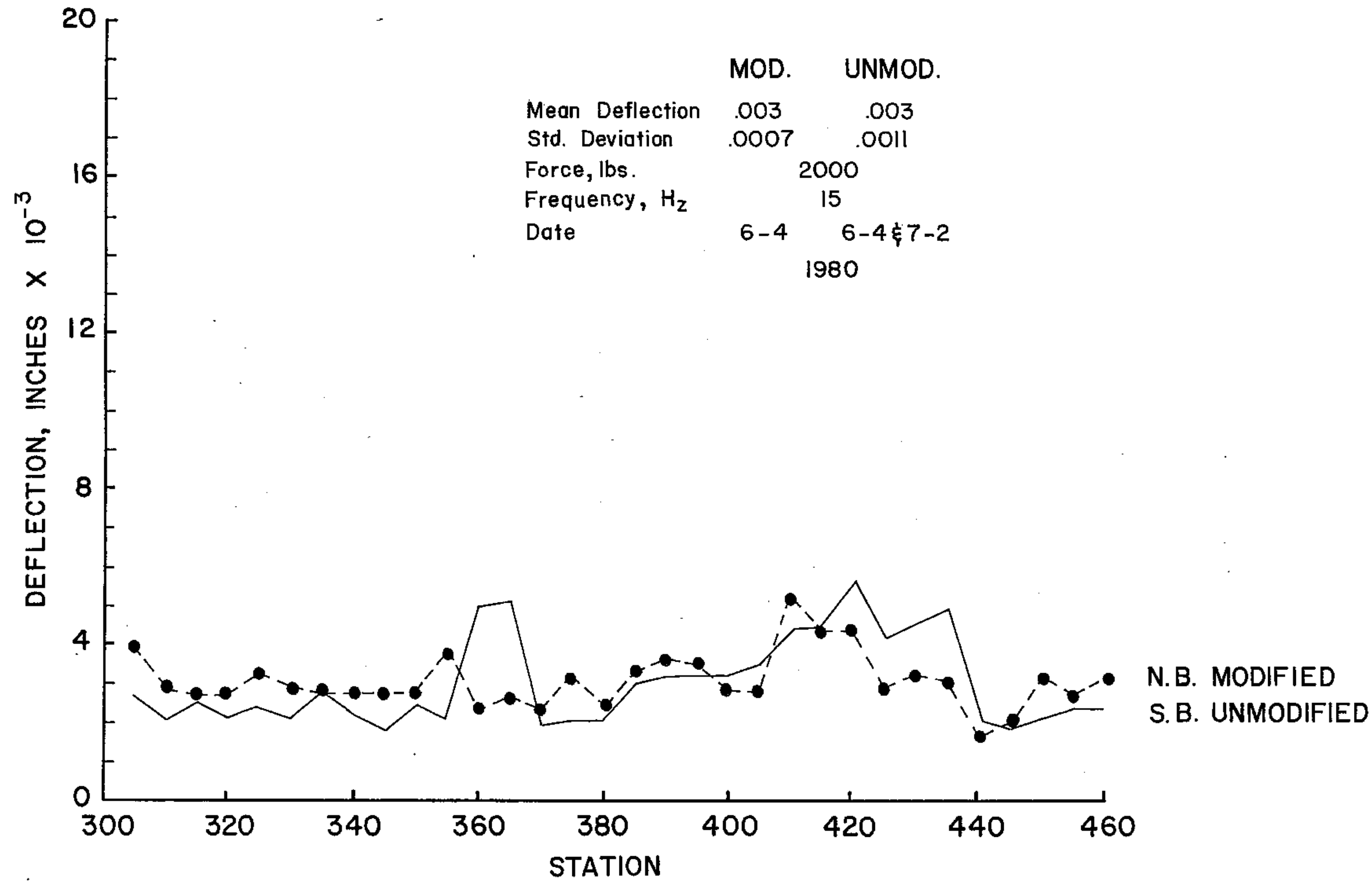

Figure 21. Plot of deflection measurements after completion of CAM subbase

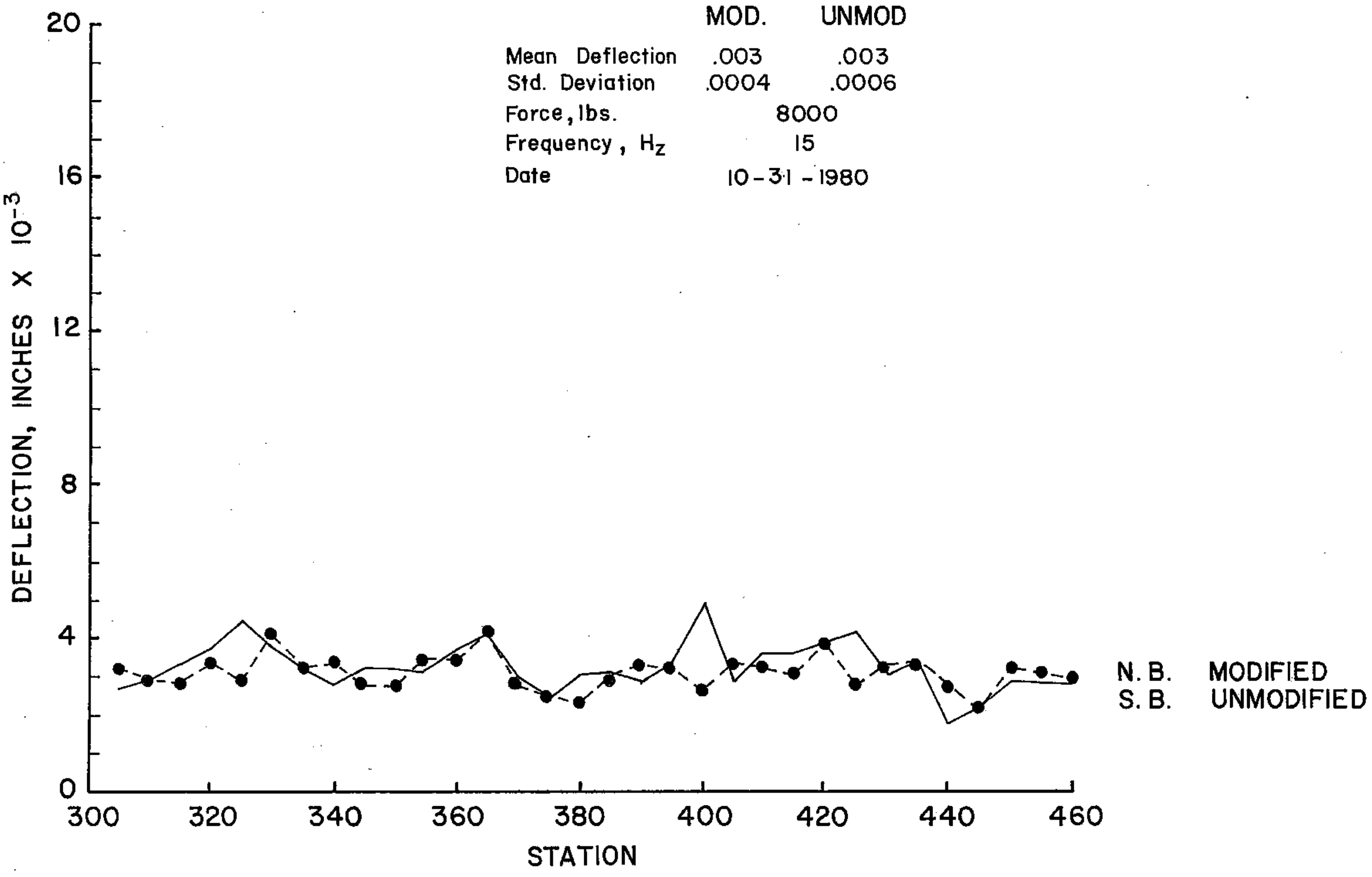

Figure 22. Plot of fall deflection measurements after completion of 10-inch CRC pavement

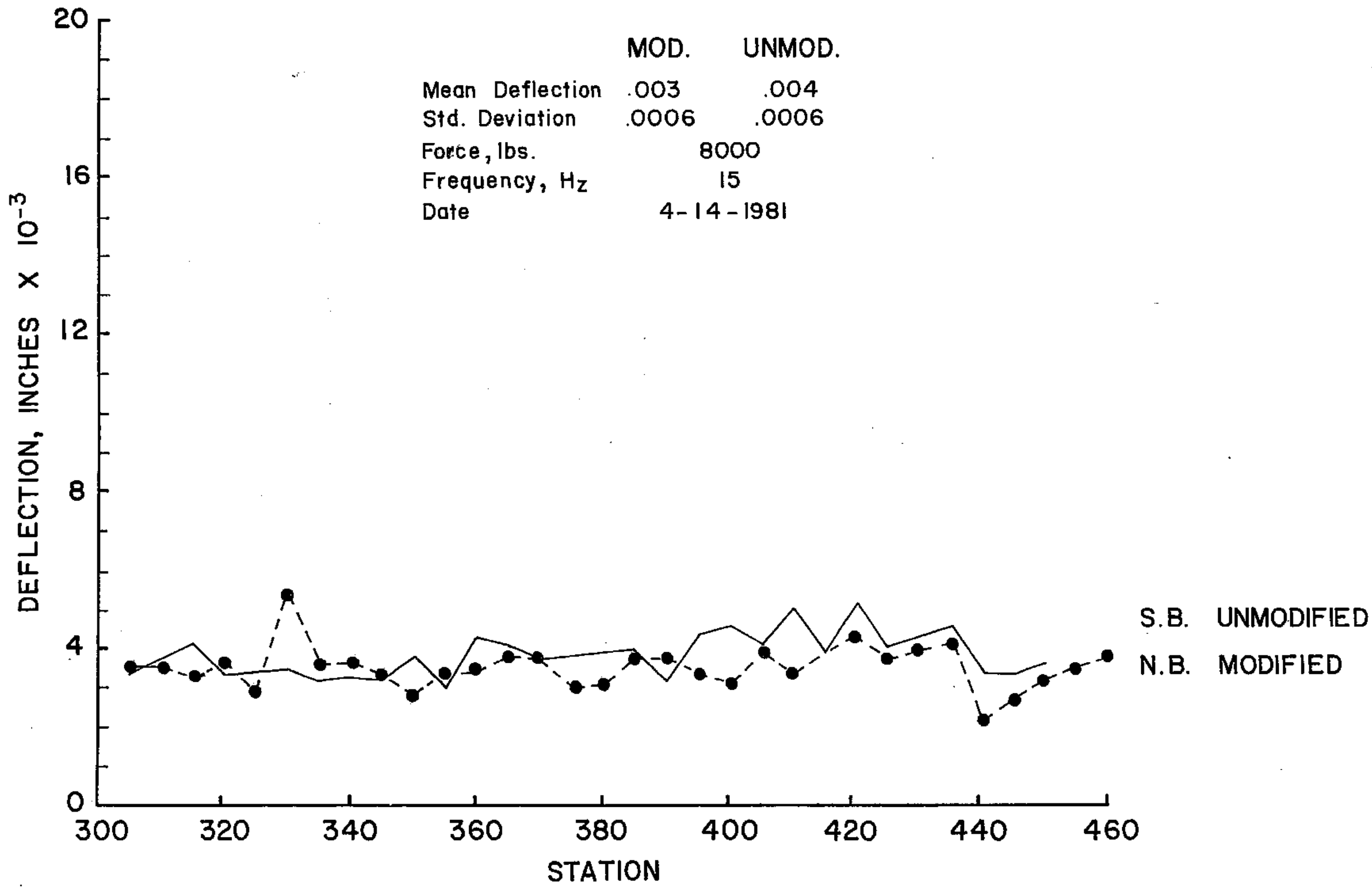

Figure 23. Plot of spring deflection measurements

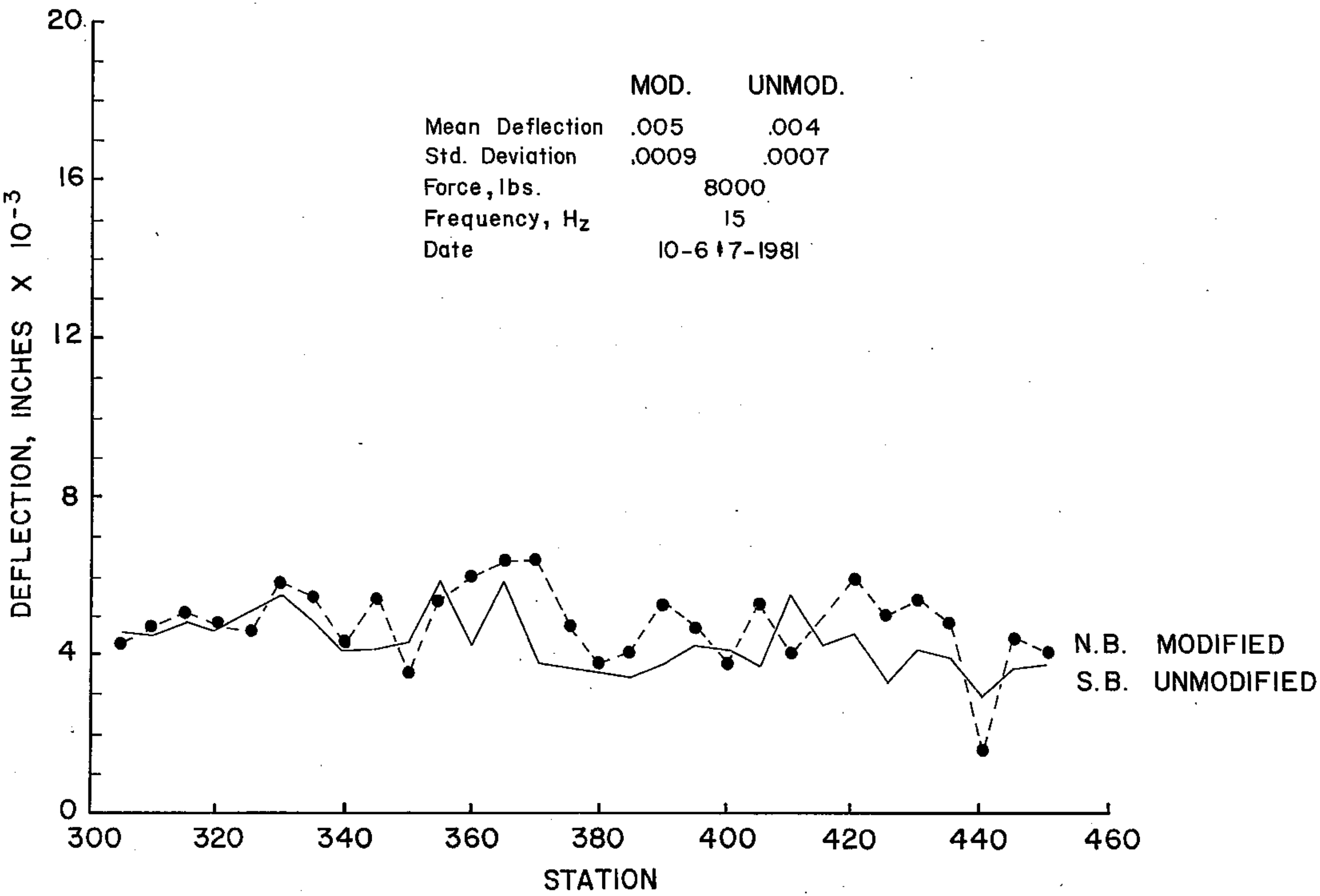

Figure 24. Plot of second fall deflection measurements

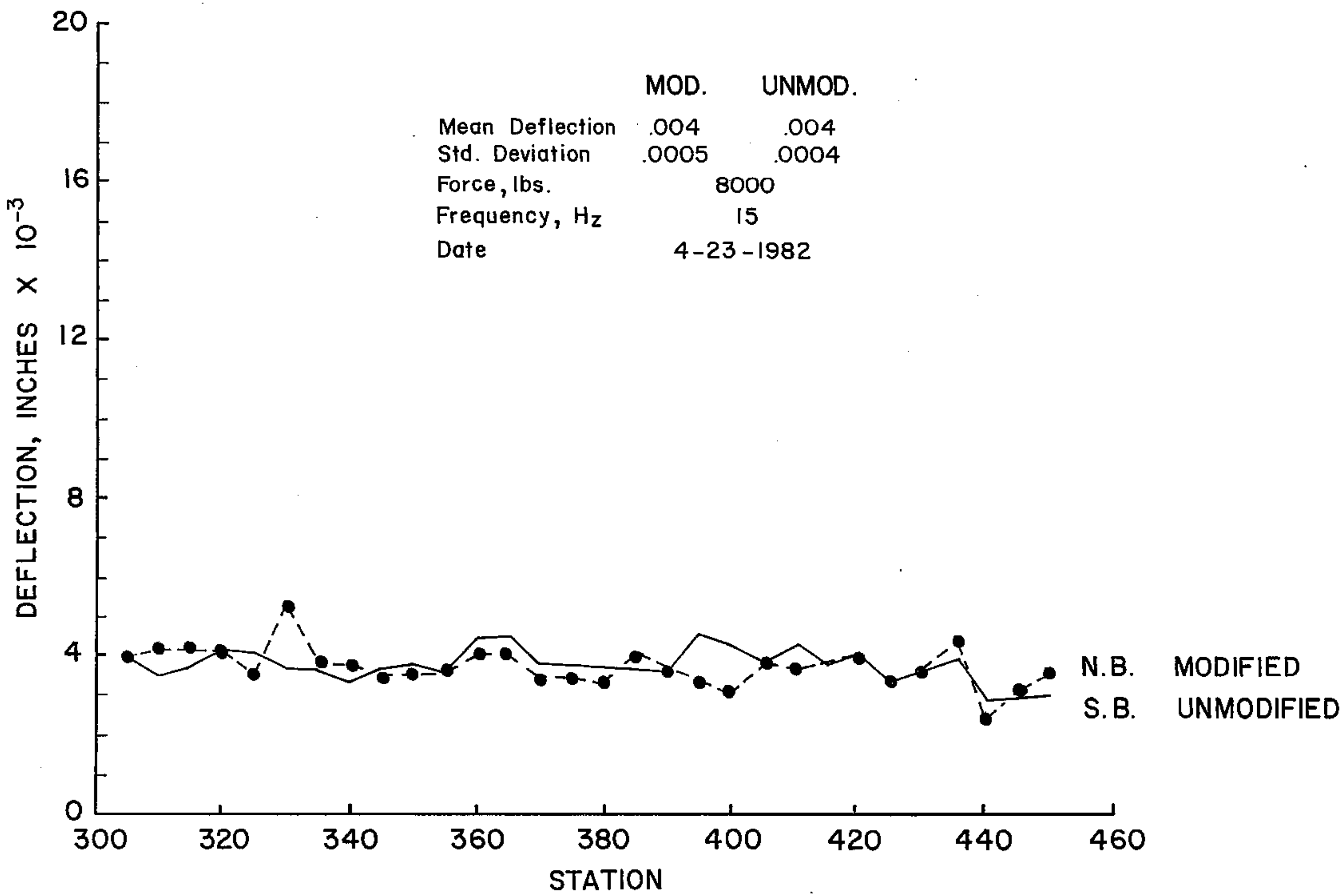

Figure 25. Plot of second spring deflection measurements

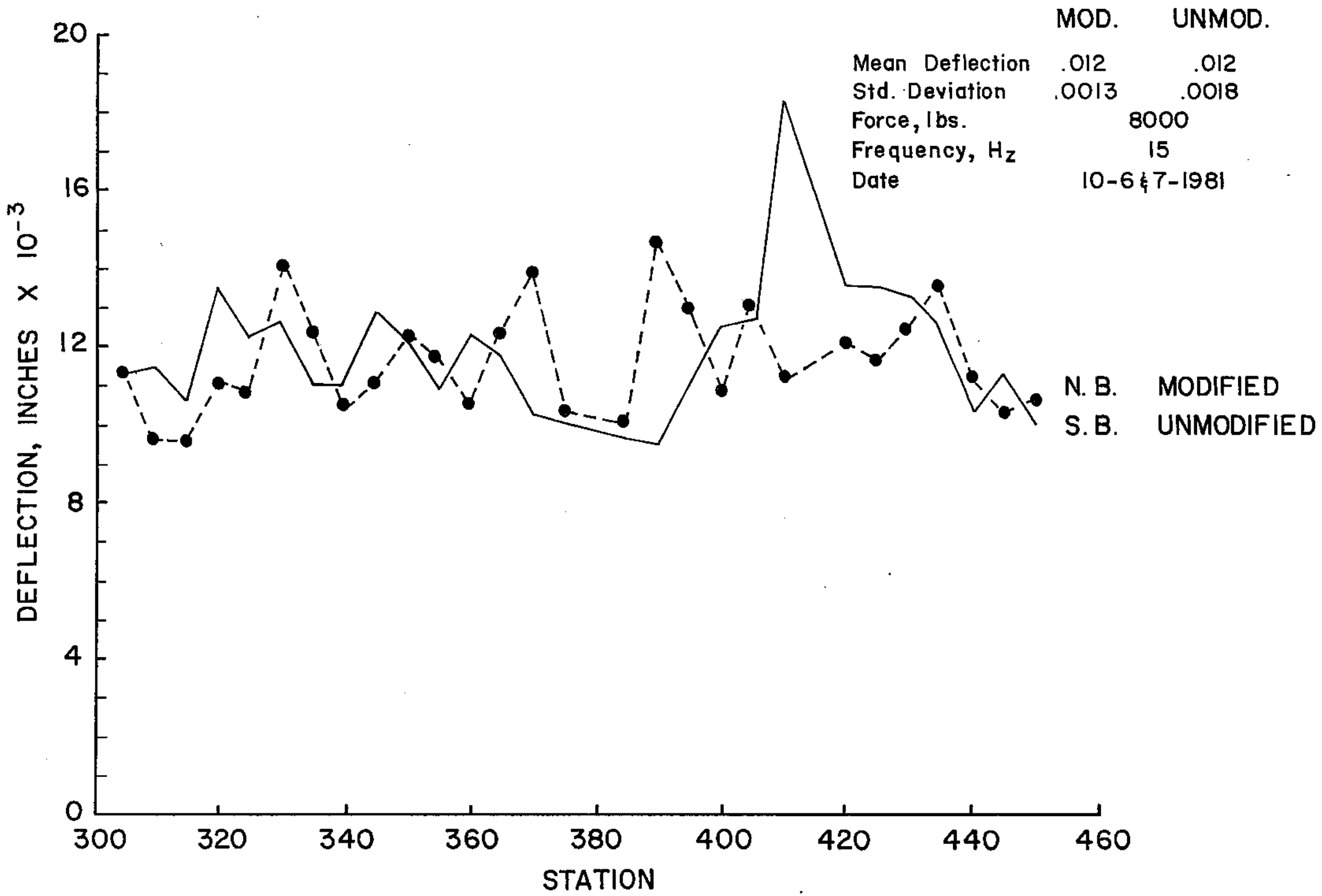

Figure 26. Plot of fall shoulder deflection measurements

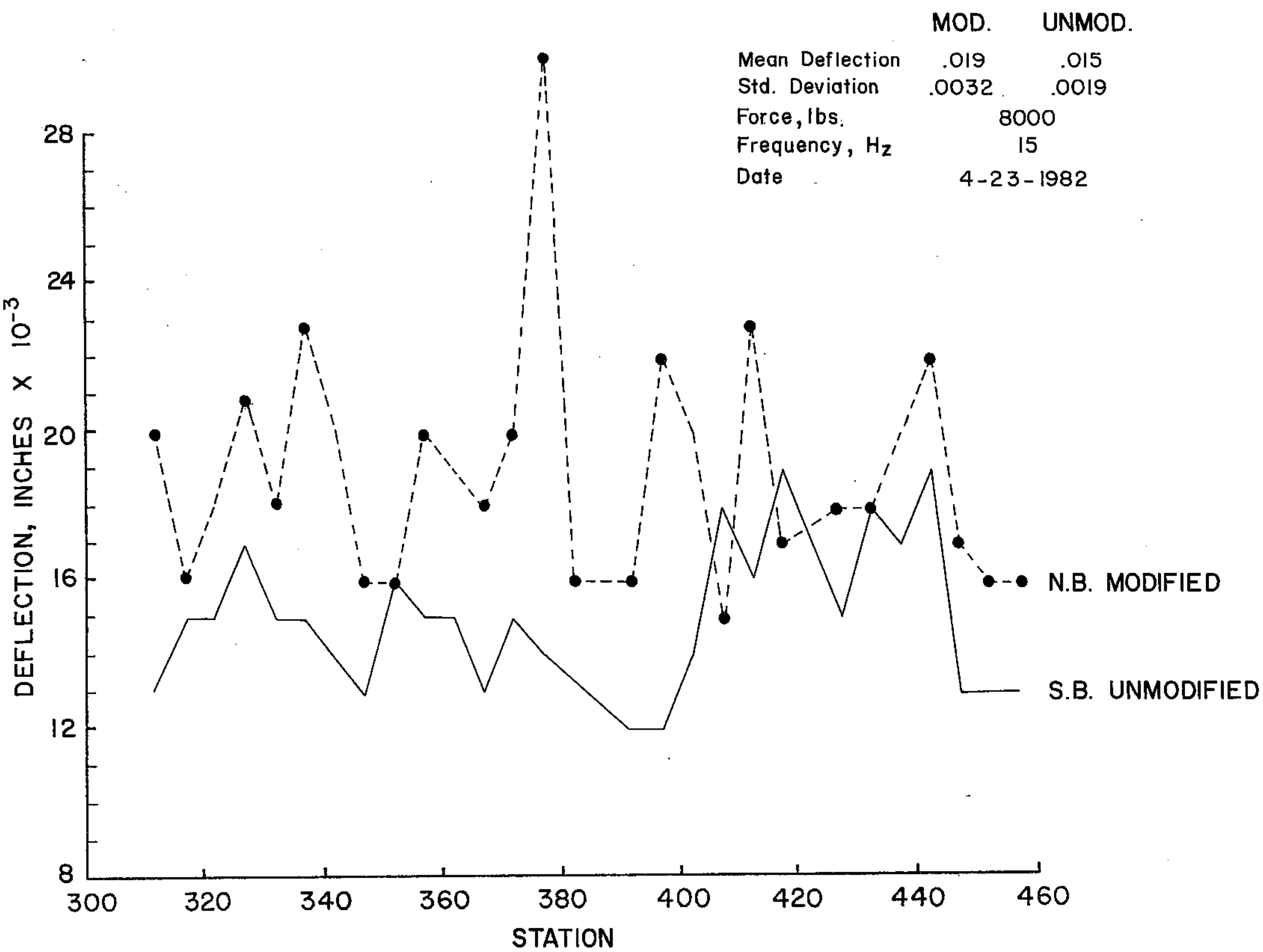

Figure 27. Plot of spring shoulder deflection measurements

embankments, the modified embankment was much better on which to work. The unmodified soil was gummy while the modified soil was not.

On June 4, 1980, while running the Road Rater, several shrinkage cracks were seen in the CAM subbase. A survey of the CAM was made on that day. An asphalt curing membrane had been applied which may have obscured some cracks. The cracks were transverse and from 3 feet to full width long. For a summary of the survey see Table 5.

The crack spacing is greater than would be expected for a CAM subbase. Due to the curing membrane, only the wider cracks may have been seen. If this is true, the CAM on the modified embankment had fewer wide shrinkage cracks. The CAM that was surveyed on the unmodified embankment was in the area of highest deflections. Just what significance, if any, this survey has is not known. However, it could shed some light in the future if early distress in the pavement is noted in this area.

Road smoothness tests revealed that both pavements were constructed essentially equal. The traffic lanes of both pavements had identical Roughness Indexes, which gave them an adjective rating of "smooth." The passing lanes had a difference of 5 in Roughness Index, which gave them an adjective rating of "very smooth."

DISCUSSION

For a study of this kind, unfavorable weather conditions would have been preferred. Unfortunately, the weather favored the contractor and it rained only twice during construction of the lime-modified embankment. Therefore, any benefits to the contractor from the use of lime during construction could not be documented. The only evidence of benefit to the contractor is shown in Figures 12 and 13. During a wet year, the benefits

TABLE 5. SHRINKAGE CRACK SPACING IN CAM SUBBASE

Total Length of CAM Surveyed ft.	Average spacing between cracks: N.B. Lime-Modified ft.	S.B. Unmodified ft.
15,524	192	-
5,400	-	126

could be significant. The only rain that fell on the project during construction of the subbase was on May 28, 1980. This was a small amount and did not affect the embankment in any way.

During the study the observer came in contact with one contractor who had experience with the use of lime for embankment construction. This contractor felt that the use of lime in wet areas was to his advantage. It was further stated that they liked it so well that they are considering bidding all soil jobs for the use of lime.

The benefit lime modification has on compaction was also undetermined from this study. Only one test on the modified embankment failed under the set roller pattern, and no failure of compaction on the unmodified embankment was reported.

The workability of the soil was noted on two occasions and both by equipment operators. In both areas, each operator thought the modified soil worked better than the unmodified soil, especially after a rain.

The amount of lime recommended included a slight amount for loss due to dusting. Dusting was no problem, and only a very small amount was lost due to dusting. A greater amount was lost due to the wetting operation. All the required water was applied before discing started. This caused some lime to be lost due to runoff, especially in areas where there was no windrow of soil along the edge. The amount of lime lost could not be determined. However, the total amount of lime lost by dusting and runoff was believed to be less than was anticipated. Therefore, the net amount of lime incorporated in the soil is believed to be in the range of 3 1/2 to 4 percent.

The deflection measurements point out one of the most obvious advantages of the lime modification that was observed. The difference in mean deflection of the two embankments is slight. The greatest difference is in uniformity. The modified embankment was much more uniform than the unmodified embankment. The three sets of measurements directly on the embankments point out this difference. With the addition of the CAM subbase, the deflections are naturally reduced. Even so, the less stable areas on the unmodified embankment are identifiable. With the placement of the 10-inch CRCP, the areas of higher deflections are no longer identifiable and each embankment has essentially the same mean deflection. The effect on the behavior of the pavement will not be known for some time. A reasonable rationale would suggest that distress caused by a weak embankment would show up first on the unmodified embankment within the area from Stations 355+00 to 370+00 and from Stations 405+00 to 435+00.

CONCLUSIONS

Of the five specific objectives, the first two were only partially answered. No clear-cut relative benefit of lime modification on subgrade compaction and stability was established. Neither was there a clear-cut benefit lime modification afforded the contractor during subbase construction.

Based on the deflection measurements and shrinkage cracks, the lime-modified subgrade improves the stability of the subbase, but the improvement is very slight and hard to determine because of the favorable construction conditions.

The immediate benefits derived by the pavement from the use of lime-modified embankment could not be determined due to reasons already assigned. The long-range benefits are beyond the time frame of this study. However, the uniformity of deflections would suggest that the pavement on the lime-modified embankment would perform better than the pavement on the unmodified embankment.

The lime-modified embankment presented no difficulty in installing edge drains.

RECOMMENDATIONS

It is recommended that a condition survey be made in five to seven years in order to see if any distress difference exists between the pavement on each embankment. If there is a difference, then it is also recommended that a set of deflection measurements be made during the fall and following spring to see if there is a difference at that time.

REFERENCES

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