Report No. 224

EVALUATION OF THREE METHODS
TO PREVENT FURTHER DIFFERENTIAL SETTLEMENT
ON I-10 THROUGH MCELROY SWAMP

FINAL REPORT

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ABSTRACT

About an eight mile long section of Interstate 10 through McElroy Swamp in Ascension and St. James parishes have been undergoing excessive differential settlements for the past decade. The embankment was constructed by removing soft clay deposits of the fresh water swamp to the elevation of the underlying hard Pleistocene marine clay layer which is located 15 feet to 20 feet below natural ground level. The soft clay was replaced with sand pumped from the Mississippi River. This sand was placed in water and was not compacted except for the top layers above the ground water level. Tests conducted by the Louisiana Transportation Research Center (LTRC), the Department of Transportation and Development (DOTD), and Louisiana State University (LSU) showed that the differential settlement found in this stretch of road is due to lack of density in the fill itself. Various methods of in-situ densification of the sand fill were tried. It was demonstrated that the embankment can be stabilized, and the differential settlements can be eliminated by using compaction grouting. Furthermore it was demonstrated that such could be achieved without destroying the pavement and without having to close I-10 to traffic at considerable savings as compared to other alternatives.

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INTRODUCTION

Interstate-10 between U.S. 61 (Airline Highway) and Blind River was constructed on a hydraulically placed sand fill from September 1967 to June 1970. This section began to settle differentially to a noticeable extent after approximately eight years of use. Investigations by the Department of Transportation and Development (DOTD) Materials Laboratory, the Louisiana Transportation Research Center (LTRC), and Louisiana State University concluded that the settlement was due to movement within the sand fill itself and not because of trapped soft material (1).

LTRC engineers estimated that there may be a maximum of two feet of settlement if the sand in the fill were in its loosest state and if it were eventually compacted to its densest state. It is unlikely that conditions are such that the full two feet of settlement would be realized anywhere in the troubled length of roadway; but since the sand is in a loose state, it does show that differential settlement will continue for some time to come.

Several meetings were held by the DOTD, LTRC, and the Federal Highway Administration (FHWA) to determine means to alleviate the situation. Among solutions discussed were: abandoning the roadway and constructing an elevated, pile-supported roadway over McElroy Swamp; using a sheet-pile coffer dam type of structure; removing the pavement and using dynamic compaction or vibrofloatation to compact the fill; compactive blasting; dewatering; injecting lime/fly ash and compaction grouting. The do-nothing alternative was also discussed and discarded because of expected continuing differential settlement. The DOTD recently spent $4 million to overlay the distressed section to improve rideability. It is a possibility that a similar amount must be spent in equivalent dollars every three to four years if nothing is done to correct the deficiency of the sand embankment. Because I-10 is the major link between the two large cities in

Louisiana, Baton Rouge and New Orleans, as well as an important interstate route, the group agreed that the method of rehabilitation should interfere with traffic flow as little as possible as well as be economically and technically feasible. There were three methods of rehabilitation that met the above criteria: dewatering, lime/fly ash injection, and compaction grouting. It was decided to select test areas in the roadway and median in the troubled area and evaluate these methods. It was further decided that the primary means of evaluation would be by the use of the electronic cone penetrometer.

METHODOLOGY

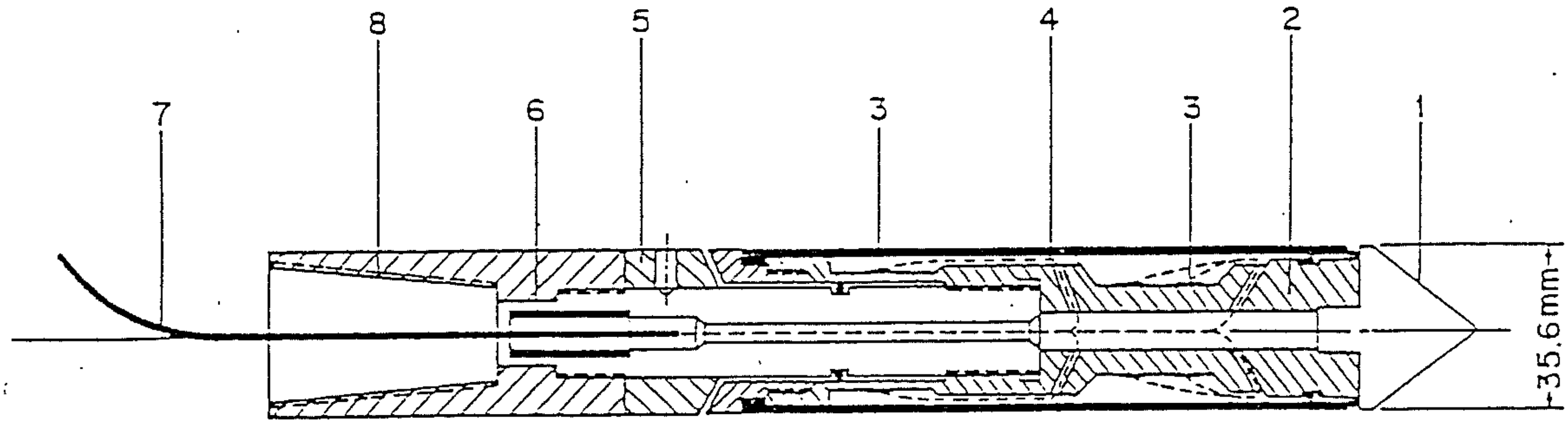
OPERATION OF THE ELECTRONIC CONE PENETROMETER

The electronic cone penetrometer testing for the entire project was performed under contract by Fugro Inter Inc., of Houston, Texas.

The Fugro cone penetrometer testing (CPT) system is housed in an environmentally controlled cabin mounted on a 20-ton (deadweight) truck. Sounding information is obtained by hydraulically pushing the electric penetrometer into the soil. The standard penetrometer (Figure 1) has a 60-degree apex angle and a 1.55 square inch cone. A straight cylindrical shaft above the cone has the same diameter as the cone and a surface area of 3.875 square inches. With the use of a hydraulically operated system, the penetrometer is able to penetrate the ground at a constant rate of 0.79 inches per second. During penetration, continuous measurements of tip resistance (q_c) and sleeve friction (f_s) are recorded on a strip chart. Data is also recorded on a diskette every 0.79 inches of penetration. An onboard computer and plotter produce the formatted results including friction ratio (the ratio of sleeve resistance (f_s) to tip resistance (q_c)), which are immediately useable in the field. (2)

CRITERION FOR ACCEPTABLE IMPROVEMENT

In order to define success in achieving an adequate increase in density, a point of reference had to be established. The relative density method as described by Dr. J. H. Schmertmann (3) was chosen because most of his work for his study was done using Mississippi River sands similar to those used for the fill in McElroy Swamp. It was decided that an increase of the relative density to 70 percent as determined by the electronic cone penetrometer would be regarded as satisfactory.



1 Conical point (10 cm^2)
2 Load cell
3 Strain gages
4 Friction sleeve (150 cm^2)
5 Adjustment ring
6 Waterproof bushing
7 Cable
8 Connection with rods

Electric Friction-Cone Penetrometer Tip

Figure 1

DEWATERING

The dewatering test section was located near a reliable water source, a two-pipe culvert undercrossing of I-10, at milepost 191.1. A large volume of water was needed to use in jetting the wellpoints in place. The test site was 1,000 feet long, centered at milepost 191.1. LTRC prepared a matrix for cone penetrometer exploration and drilled four-inch-diameter holes through the pavement on 50-foot centers in the center of the outside lane on both roadways throughout the test section. Cone penetrometer tests were also conducted in the unpaved median at 50-foot centers.

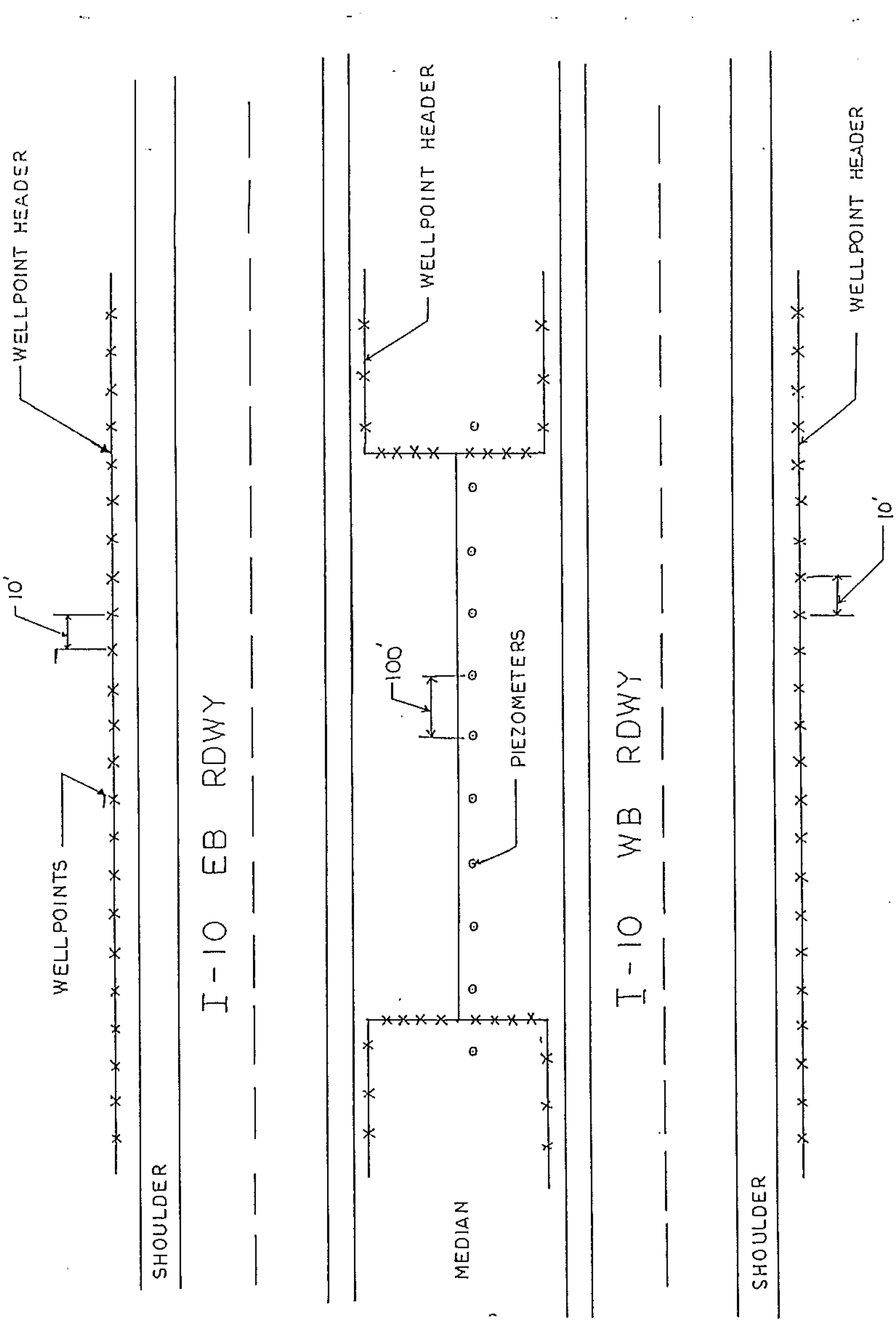
It was thought that dewatering the fill beneath the roadways would increase effective stress within the fill and might cause sufficient compaction to stop differential settlement. It was hoped that the water table could be drawn down by 15 feet using a wellpoint system.

The dewatering was done by contract with Stang Inc., of Mobile, Alabama.

Wellpoints were set on the outside shoulders of the eastbound and westbound roadways on 10-foot centers for a distance of 1200 feet. In the median, wellpoints were set in a "U" shape with 100-foot legs to stop recharge from the ends of the project (Fig. 2).

Each wellpoint was installed by washing and driving a two-foot diameter steel casing to the desired depth, placing the plastic wellpoint in the center of it, filling the annular space with sand to act as a filter, and then withdrawing the casing. The wellpoints were set with their intakes just above the hard marine clay layer which underlies the entire area.

The wellpoints were 2.5-inch diameter PVC pipe with 0.016-inch wide slots cut in them to a height of 2.5 feet above the tip.



N.T.S.

Dewatering Wellpoint Layout

Figure 2

Each wellpoint was connected through a valve and flexible hose to an eight-inch PVC header pipe (manifold) that ran the length of the project.

Three vacuum-assisted, 1500-gallon-per-minute, diesel-powered pumps were connected to the header systems. Each shoulder header had a pump and the two median interception headers shared the third.

In order to determine the effectiveness of dewatering, LTRC installed an observation well system every 100 intervals along the median centerline. To monitor surface roadway performance, level rods were installed along both shoulders at 100-foot intervals for sighting by a surveyor's level.

The pumps started on November 30, 1987, and ran continuously until December 18, 1987. Water levels were taken at the wells and elevations checked on an hourly basis, 24 hours per day during pumping.

The water table in the median was lowered 5.5 feet and could not be lowered any further. At this point in the test, the valves at the wellpoints were all nearly shut off because air was entering into the system, indicating maximum drawdown at the wellpoint.

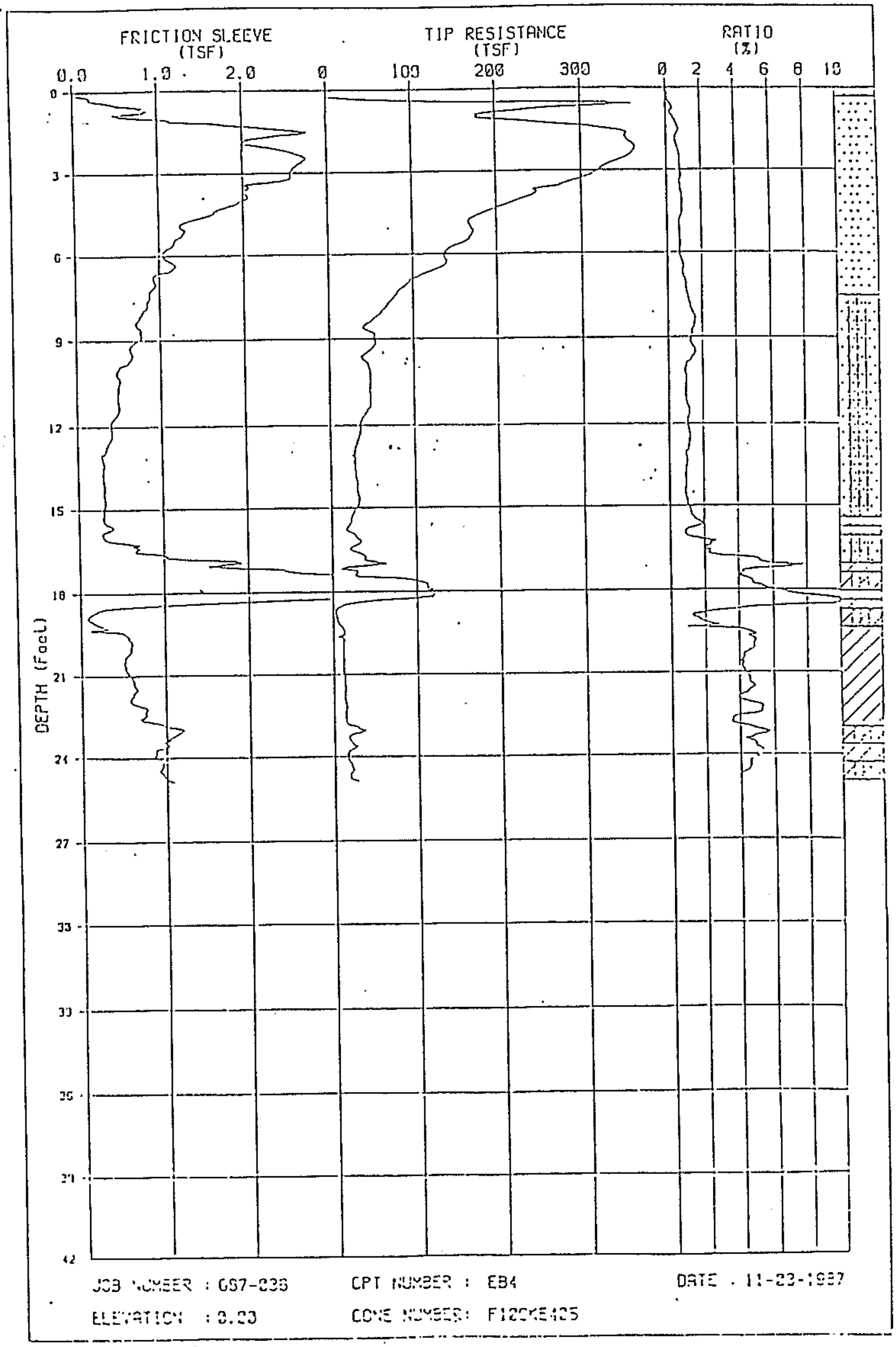
It was felt that recharge of the fill was happening in spite of the wellpoints. Water could have flowed along the interface of the marine clay and sand fill between the wellpoints in sufficient quantity to prevent further drawdown than was achieved. It is also possible that debris (tree limbs, stumps, etc.) left in the dredging process provided channels for water to recharge the sand fill faster than it could be pumped out. There is a also the possibility that the sand fill, while pumpable to a degree, was not permeable enough to allow dewatering to the desired degree. The test indicated that a single line of wellpoints was insufficient to dewater the fill effectively and that the drawdown that was achieved was insufficient to cause consolidation of the fill. Cone penetrometer testing also indicated that no improvement was achieved by dewatering. Figures 6, 7, and 8, show penetrometer results of both before and after dewatering.

The cone penetrometer results (Figures 3, 4, 5) are typical for data taken through the fill. The characteristically high cone resistance and high friction are present on initial penetration for approximately three feet, followed by a rapid drop down to the elevation of the water table, where resistance values are very low. These results are indicative of density achieved by construction equipment operating during construction of the embankment and of traffic vibrations after construction.

The composite figures (Figures 6, 7, 8) show a weakening of the fill after dewatering in the soundings taken through the pavement. This is the result of the cone being inserted in the same hole drilled through the pavement before and after dewatering. The soil, being previously disturbed, was weaker. The same pattern is not evident in the median comparison, where the penetrations were not in identical locations.

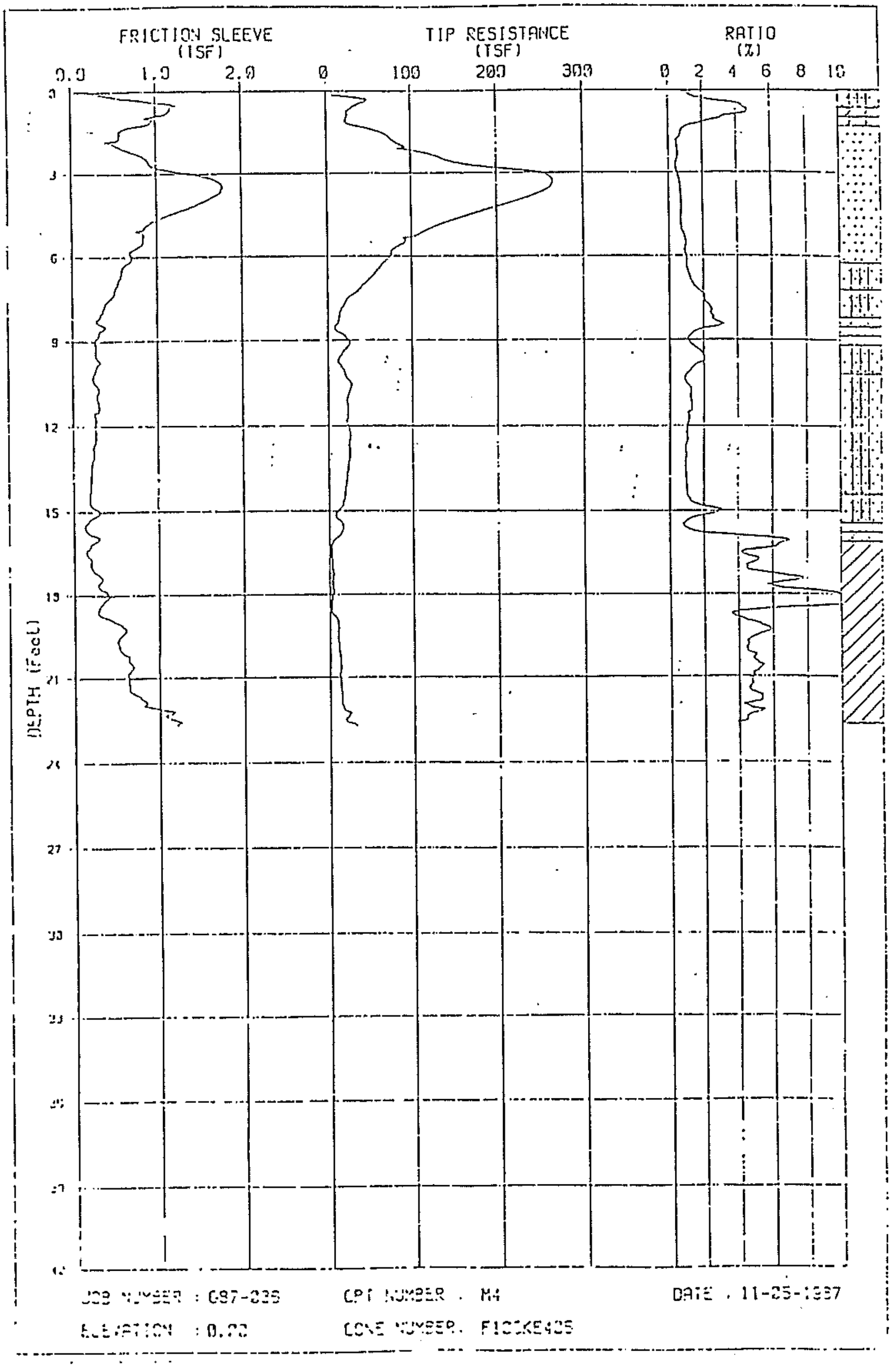
The cone penetrometer contractor took 63 soundings before and after dewatering. The soundings were taken on 50-foot centers. Typical results of the soundings are shown in Figures 3,4,5,6,7, and 8.

After reviewing the dewatering results, it was agreed to leave the dewatering equipment in place and to inject lime/fly ash in a portion of the dewatered test area as an additional test to determine the effect of the reduction of a resisting hydraulic head in the soil on the penetration of injection slurry. The dewatering equipment was shut down until January 11, 1988, when it was restarted preparatory to the arrival of the injection contractor on January 19, 1988.



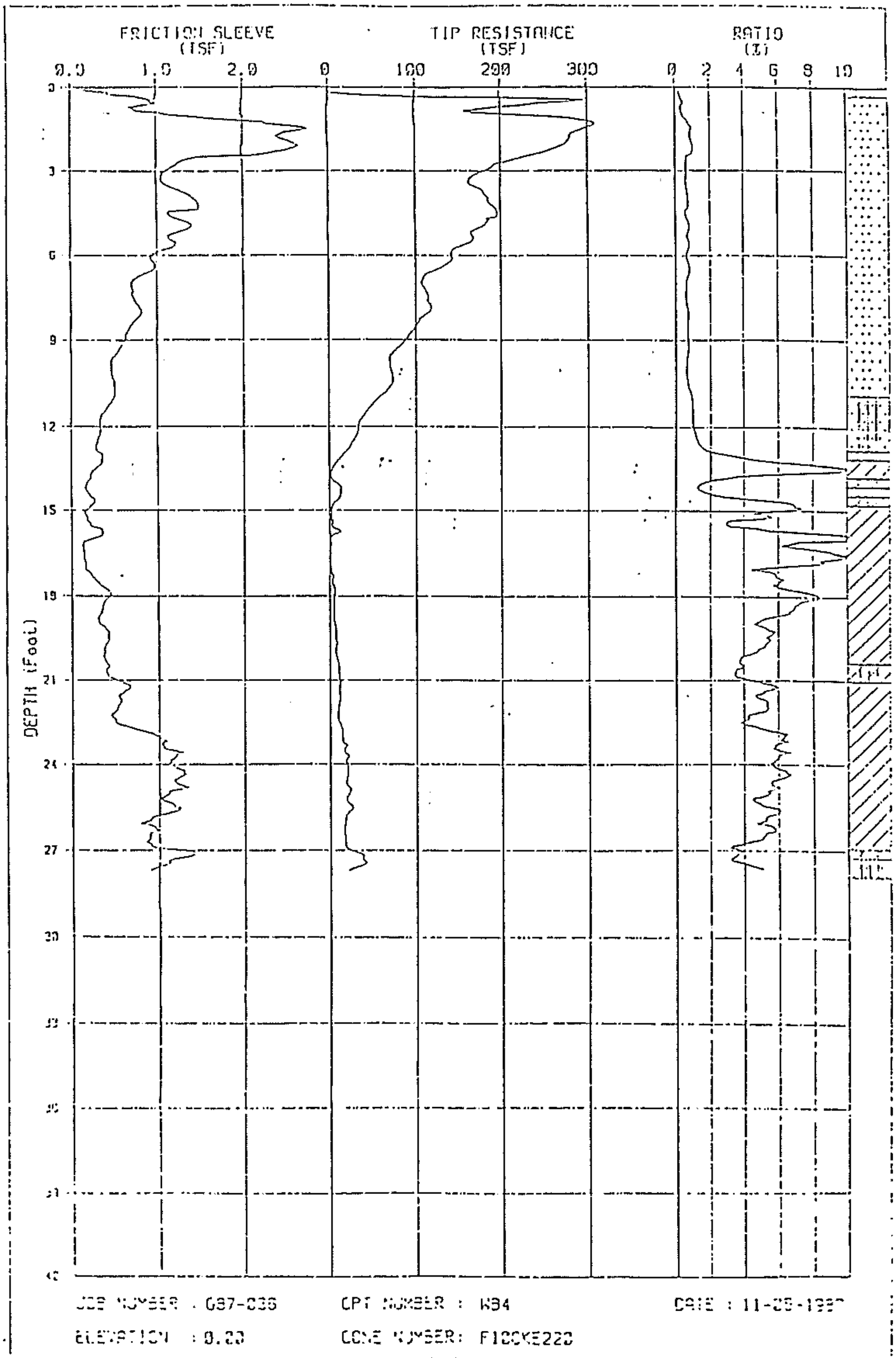
Before Dewatering - Eastbound Lanes

Figure 3



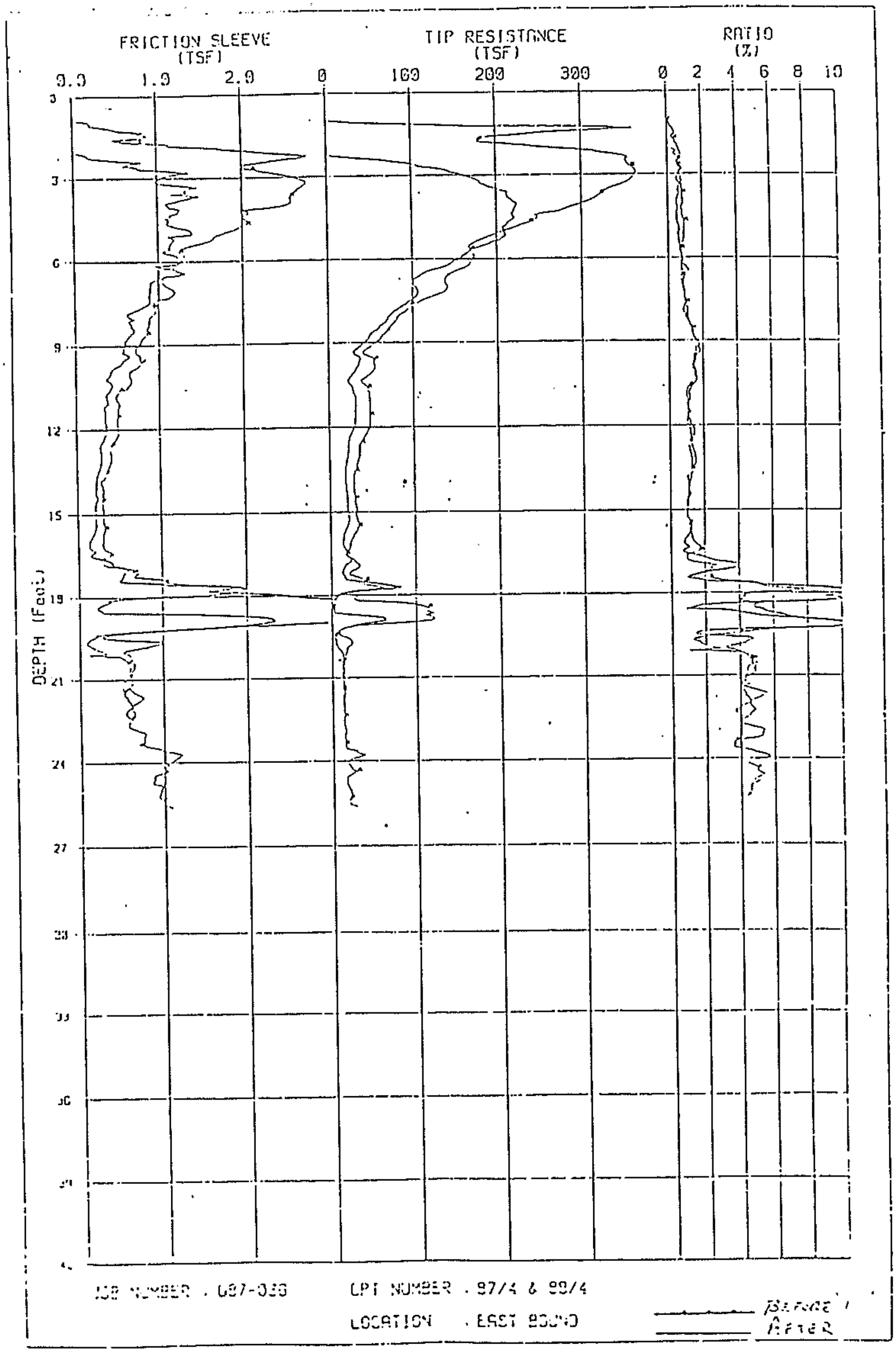
Before Dewatering - Median

Figure 4



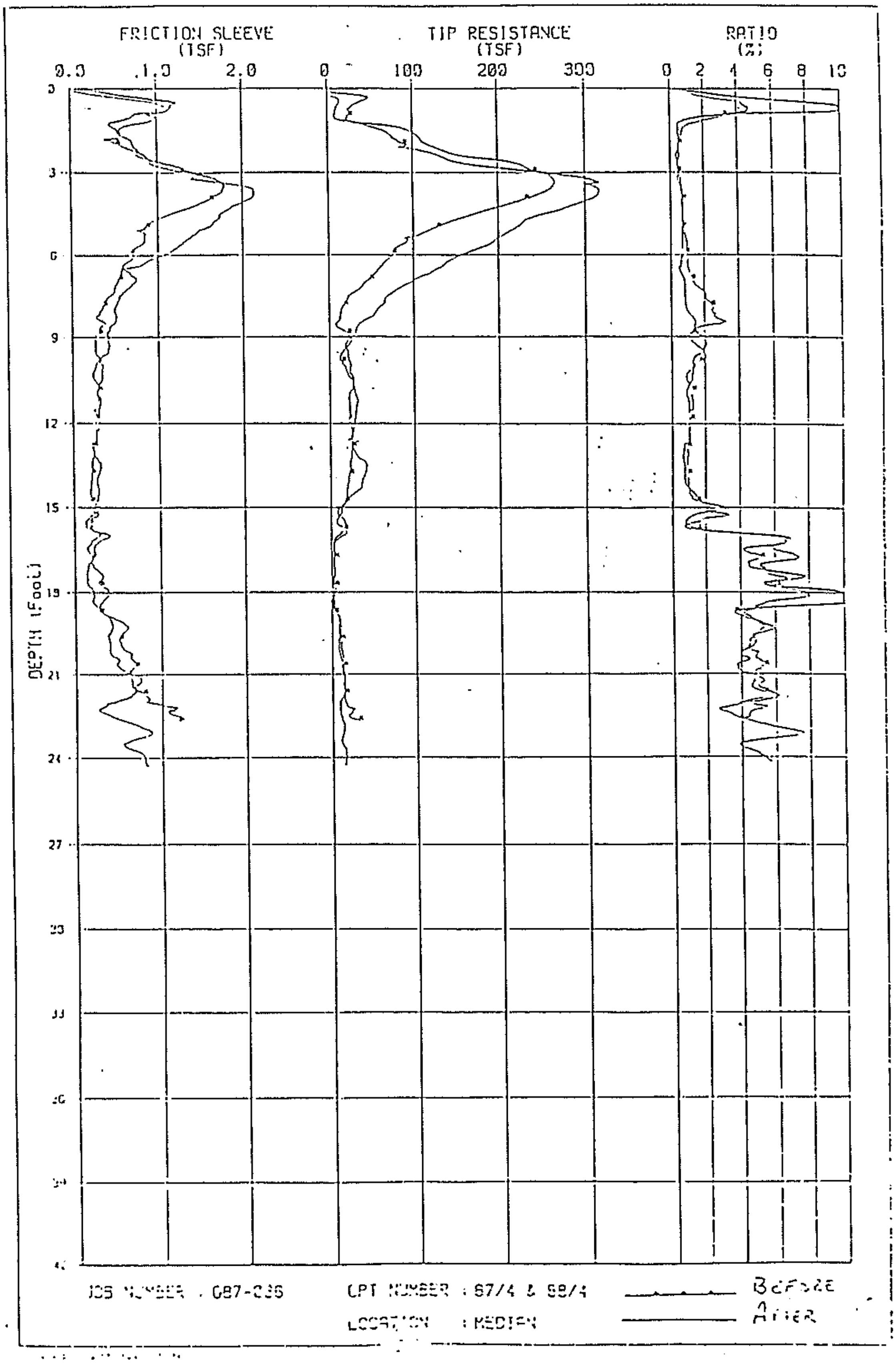
Before Dewatering - Westbound Lanes

Figure 5



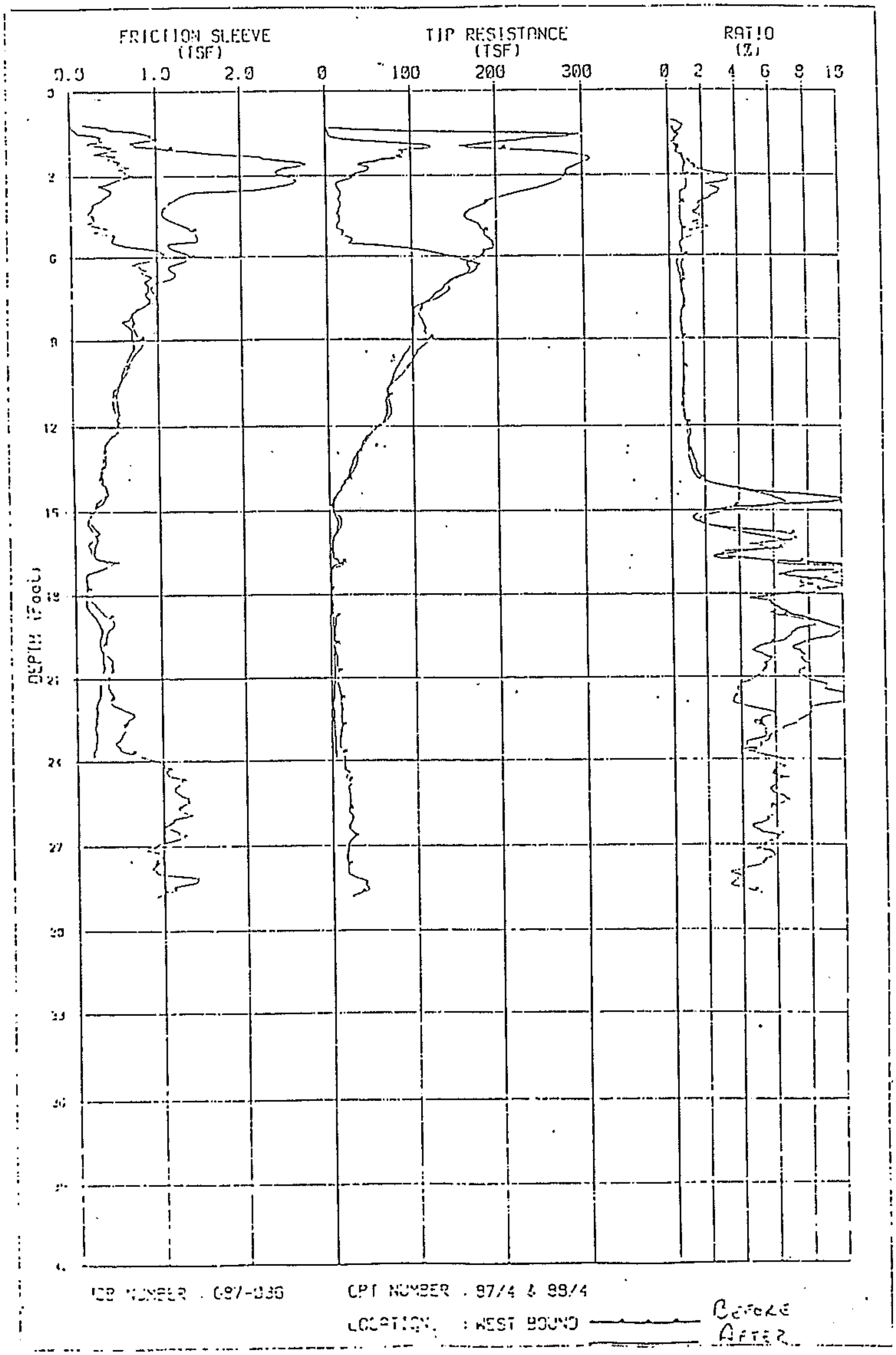
After Dewatering - Eastbound Lanes (Composite)

Figure 6



After Dewatering - Median (Composite)

Figure 7



After Dewatering - Westbound Lanes
(Composite)

Figure 8

LIME/FLY ASH INJECTION

The lime/fly ash injection was performed under contract by GKN Hayward Baker Woodbine Inc.

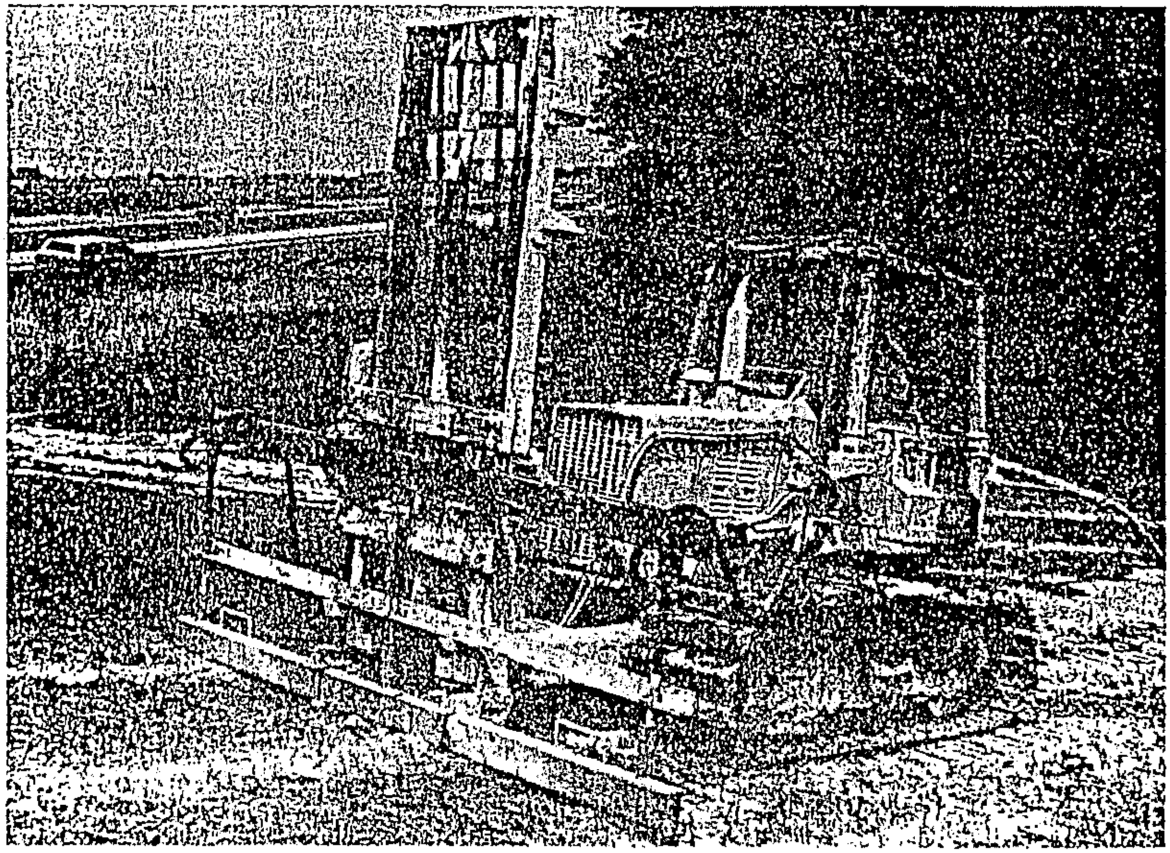
The injection device was mounted on a tracked tractor (Figure 9). Four probes, on four-foot centers, were advanced simultaneously into the soil to a depth of 20 feet.

A mixture of three parts of fly ash to one part of lime was injected. The injection slurry contained about 35% solids and 65% water. Control of the water/solid ratio was accomplished by periodically measuring the specific gravity of the mixture. The initial specific gravity was 1.40. The mixture could be varied, depending on the judgement of the operator. Injection was considered sufficient when material flowed freely to the surface around the injection tubes, indicating that the slurry was not penetrating the fill anymore but was taking the easier path along the tubes.

A 50-foot by 50-foot section of the dewatering site and three 50-foot by 50-foot sections in the median at milepost 189.0 were injected with lime/fly ash slurry. It was hoped that the lime/fly ash would react with materials in the fill and densify it by filling voids and hardening. Testing was restricted to the median because it was not desirable to perforate the pavement for the experiment.

The three injection sites at milepost 189.0 were treated as follows:

A. Four-foot centers with continuous injection to a depth of 20 feet.
B. Four-foot centers with penetrations alternating between ten- and 20-foot depth.
C. Five-foot centers with continuous injection to a depth of 20 feet.



Injection Rig Advancing on Injection Pattern

Figure 9

The dewatering site was to receive continuous injection on four-foot centers to a depth of 20 feet with the dewatering system in full operation.

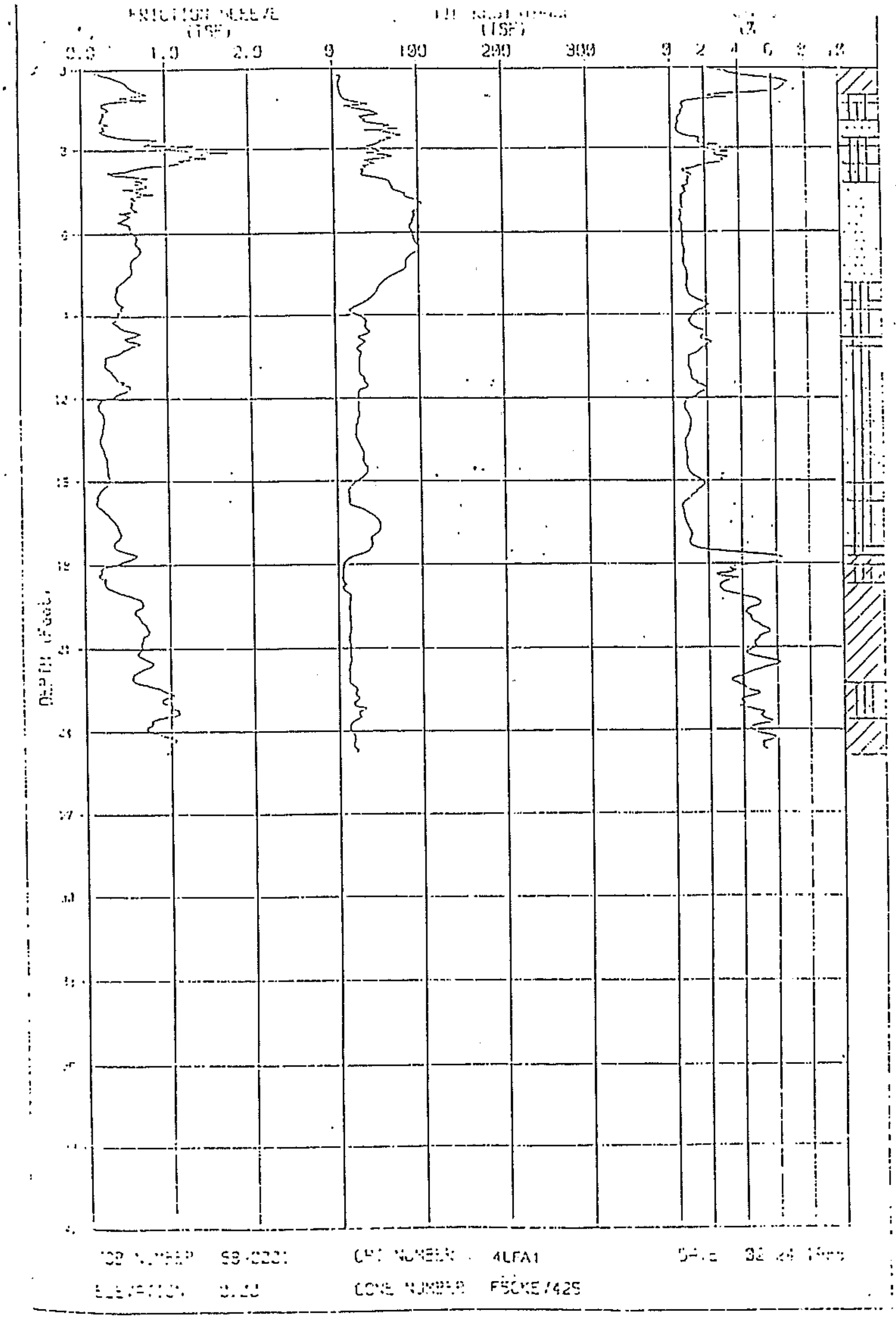
The contractor had a great deal of difficulty penetrating the dense layer of fill with four probes. He reduced the number of probes to two. The contractor was then able to penetrate the dense layer, but only with difficulty. He attached a vibrator to the probes, which helped somewhat, but production was still very slow, with each push taking from 15 to 20 minutes. As a last resort, an air-powered auger was used to drill holes through which the probes could be inserted. This effort sped production considerably.

So much of the slurry was forced up around the probes that it was impossible to estimate the volume that was being injected into the fill. At times, the entire median around or near the operation was flooded with the lime/fly ash slurry.

Cone penetrometer soundings taken at both the dewatered and unaltered sites showed no improvement as a result of the lime/fly ash injection. Figures 10 and 11 are depictions of cone resistance and friction after injection. These figures should be compared to Figure 4, which is a sounding made in the median with typical results.

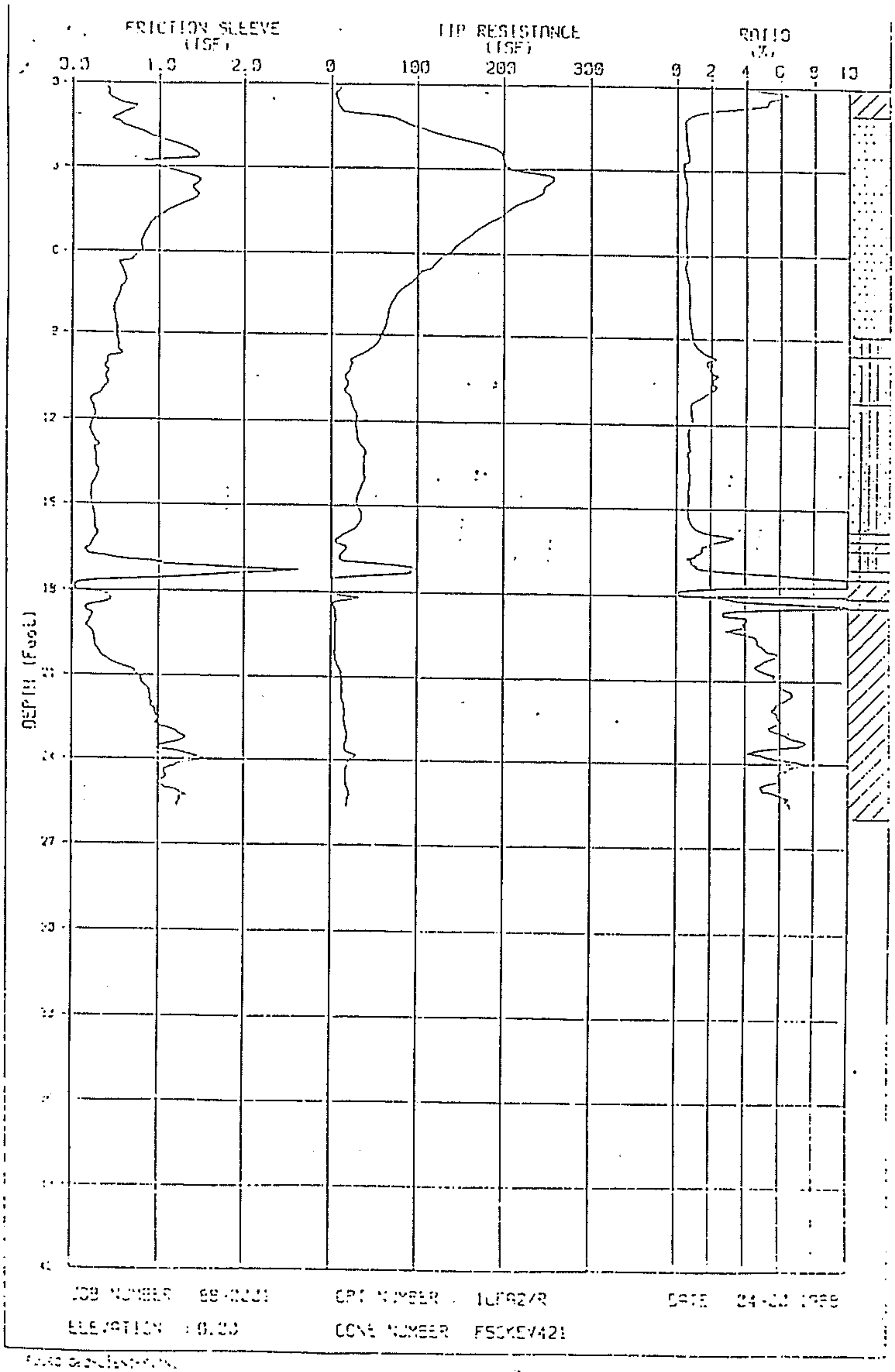
As shown in the after-injection data, there is virtually no change in cone resistance or friction. Cone soundings were also made 70 days after injection in order to give the material time to set up, but no change was evident after this period either.

It was believed that the reasons for the failure of this procedure are that either the slurry did not react with the material in the fill, or dispersion of the slurry was not sufficient, or a combination of these. As expected, the slurry take was greater in the dewatered area than in the other sites.



After Lime/Fly Ash Injection - Dewatering Site

Figure 10



After Lime/Fly Ash Injection - Site

Figure 11

COMPACTION GROUTING

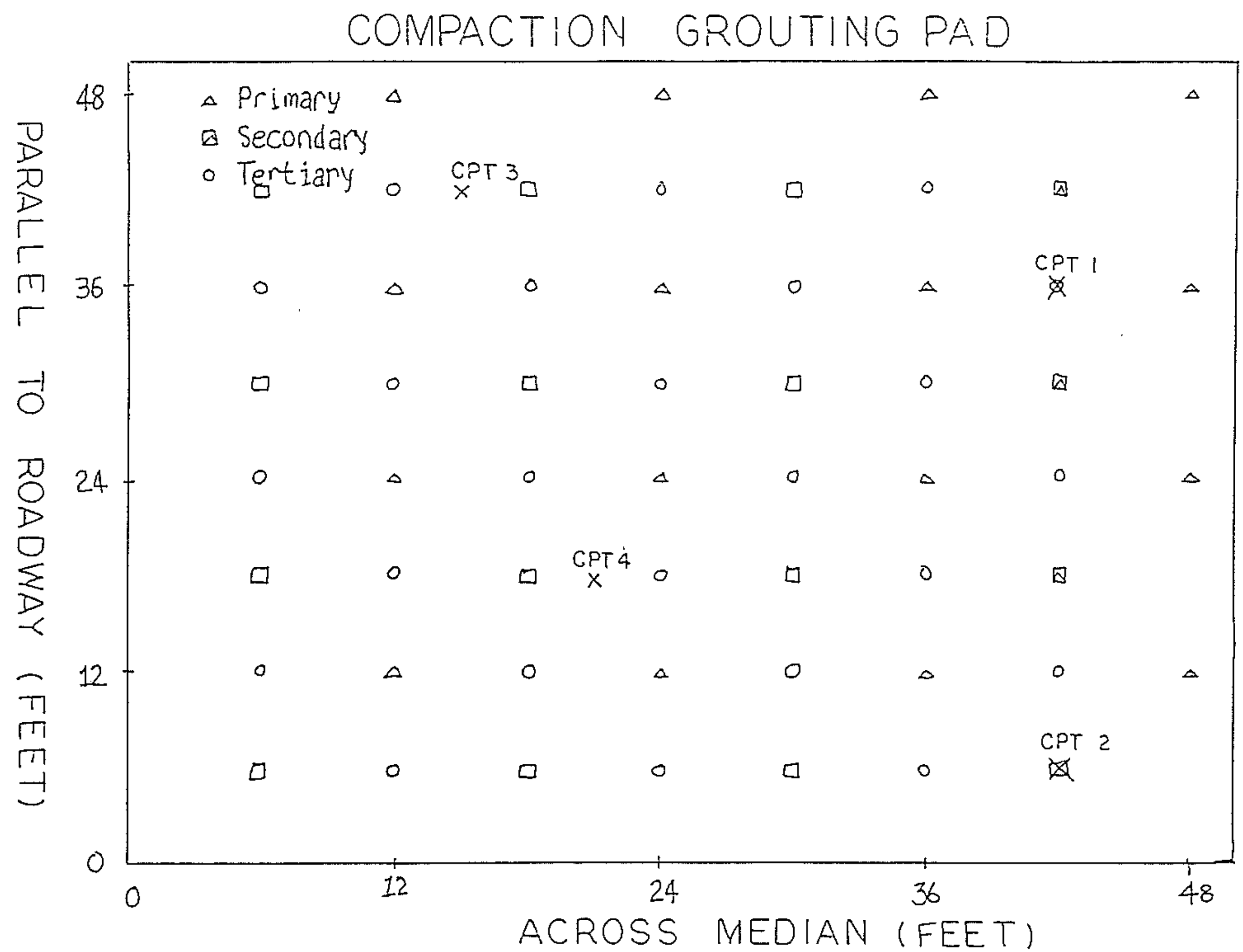
Compaction grouting was done under contract by GKN Hayward Baker.

The grout to be injected was made up of a mix of 500 pounds of fly ash, 200 pounds of cement, and 2400 pounds of sand. The mix was very stiff, with a slump of 0 to 1 inch allowed.

Injection pressures were expected to be on the order of 200 to 600 pounds per square inch (psi). In some instances, pressures of 1000 psi were used.

Grouting vertically through the pavement into the loose sands was technically the easiest and most straightforward approach. Unfortunately, this approach would have placed large amounts of heavy equipment and men on the heavily travelled roadway which, at best, would have been a major inconvenience for the travelling public and, at worst, an extremely dangerous undertaking. It was felt that the desired objective of compacting the loose fill could be achieved by grouting through the embankment slopes to a point that would be under the pavement. To do this, grout tubes were angled at various inclinations. This process is called angle grouting. Angle grouting would enable the work to take place with a minimum exposure of men and equipment to passing traffic.

The initial test was a 50-foot by 50-foot section in the median, as shown in Figure 12. The primary grout pattern was done on 12-foot centers. At the end of primary grouting, a cone sounding was taken at the center of the pattern. A secondary pass of grout was placed which reduced the spread to approximately eight feet. At the completion of this pass, two cone soundings were taken. A tertiary pass was then made which reduced the spread still further to six feet. A final sounding was taken at the center of one of these six-foot spreads.
Figure 13 clearly shows that initially, the relative density of the fill is considerably below the acceptable criterion.



Compaction Grout Layout

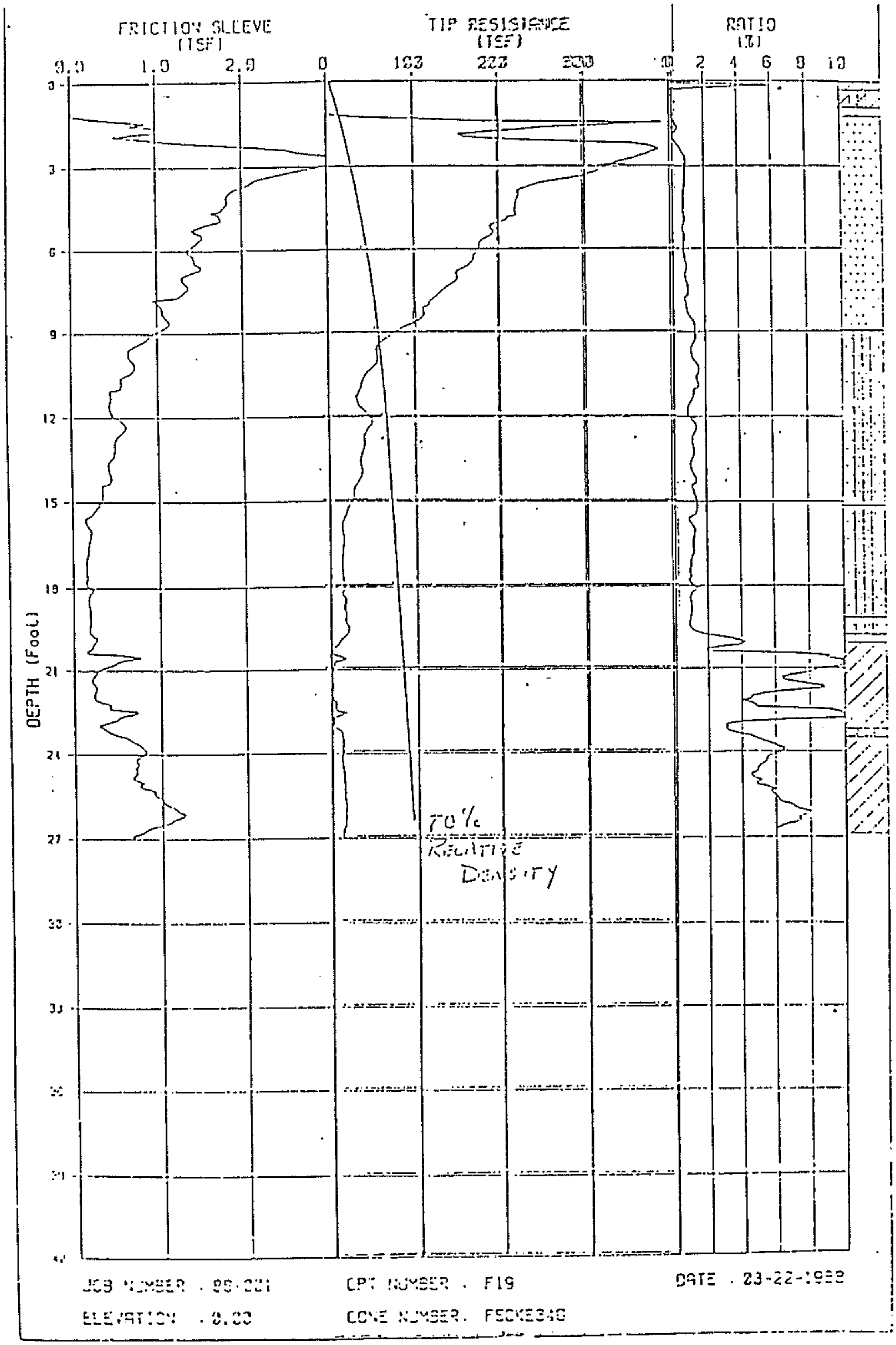
Figure 12

Similarly, Figure 14, after one grout injection, also shows values below the 70 percent relative density line.

The next three Figures, 15, 16, and 17, show the marked improvement in relative density obtained as a result of grouting. All values show an increase in relative density to the 70 percent line, with only a negligible number of points below that line. Where the tip resistance line falls below the 70 percent relative density line, low tip resistance is attributed to lack of grouting at that point.

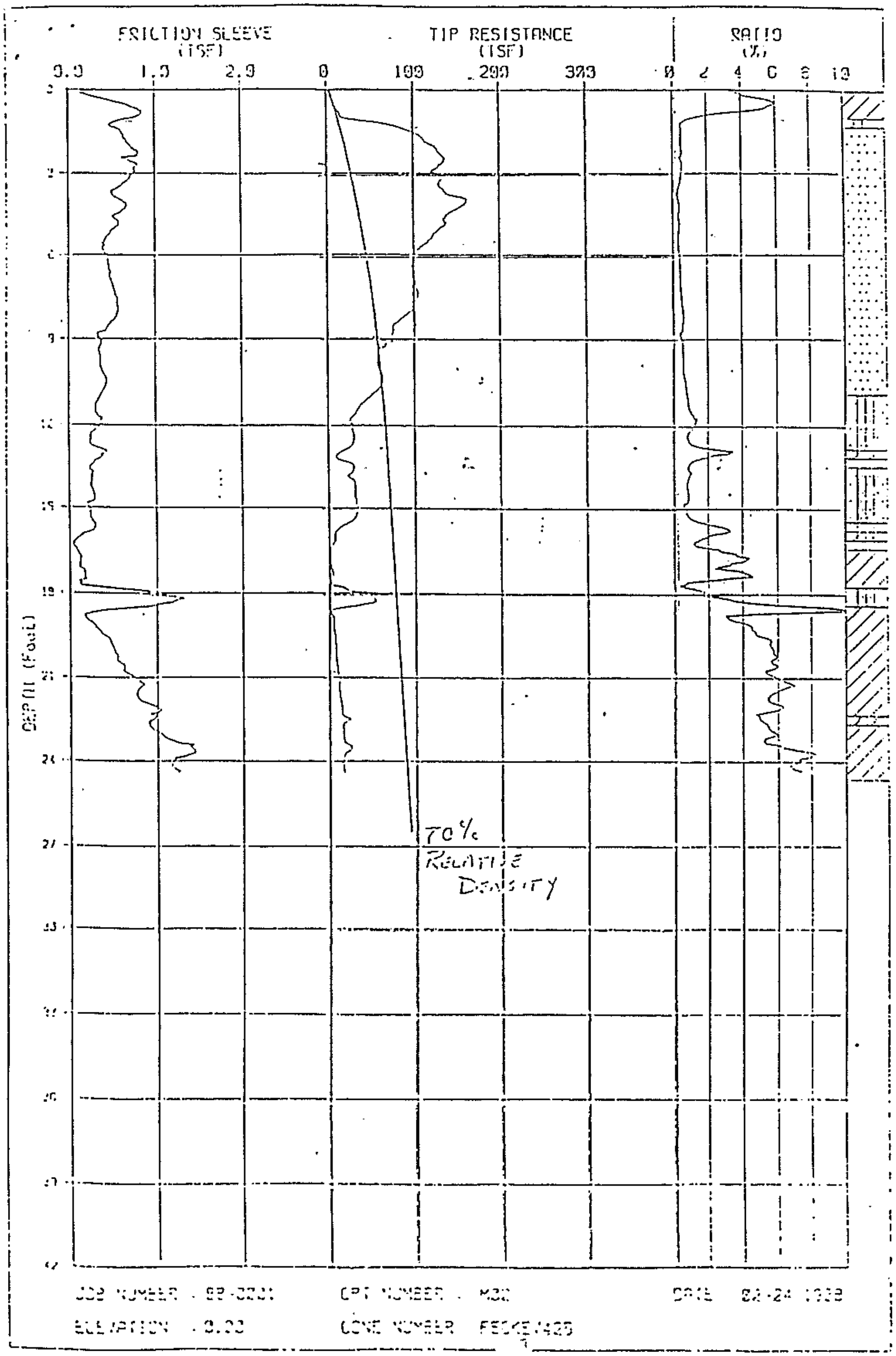
Angle grouting beneath the pavement presented some difficulty because of limitations of the grout placement machine. This machine was totally air-operated, including the hydraulic system. It used a tall mast which enabled it to press up to 20 feet of grout casing into the ground before more casing must be added (Figure 18). To angle the mast sufficiently for grouting to reach an angle of inclination of up to 40 degrees rendered the machine unstable. The contractor's solution was to drill the angle holes with a large auger and set casing using the hydraulic press from the grout placement machine. Casing joints of five-foot length were used because they had to be set by hand. This particular arrangement, while effective, was considerably more unwieldy and slower than had the grout machine been modified. Modifications to the equipment such as the addition of an extended arm to overcome the moment force of the angled mast or an anchoring system were considered desirable. However, this modification was impractical once operations commenced.

Once the initial test appeared successful, two additional compaction grouting tests were done under the roadway. Figure 19 shows the layout of the angle grout holes on the shoulder of the outside westbound lanes, and Figure 20 shows a different layout on the inside westbound lanes. Each side used an initial angle of approximately 40 degrees from the vertical, with a casing length of 26 feet for the hole nearest the travel



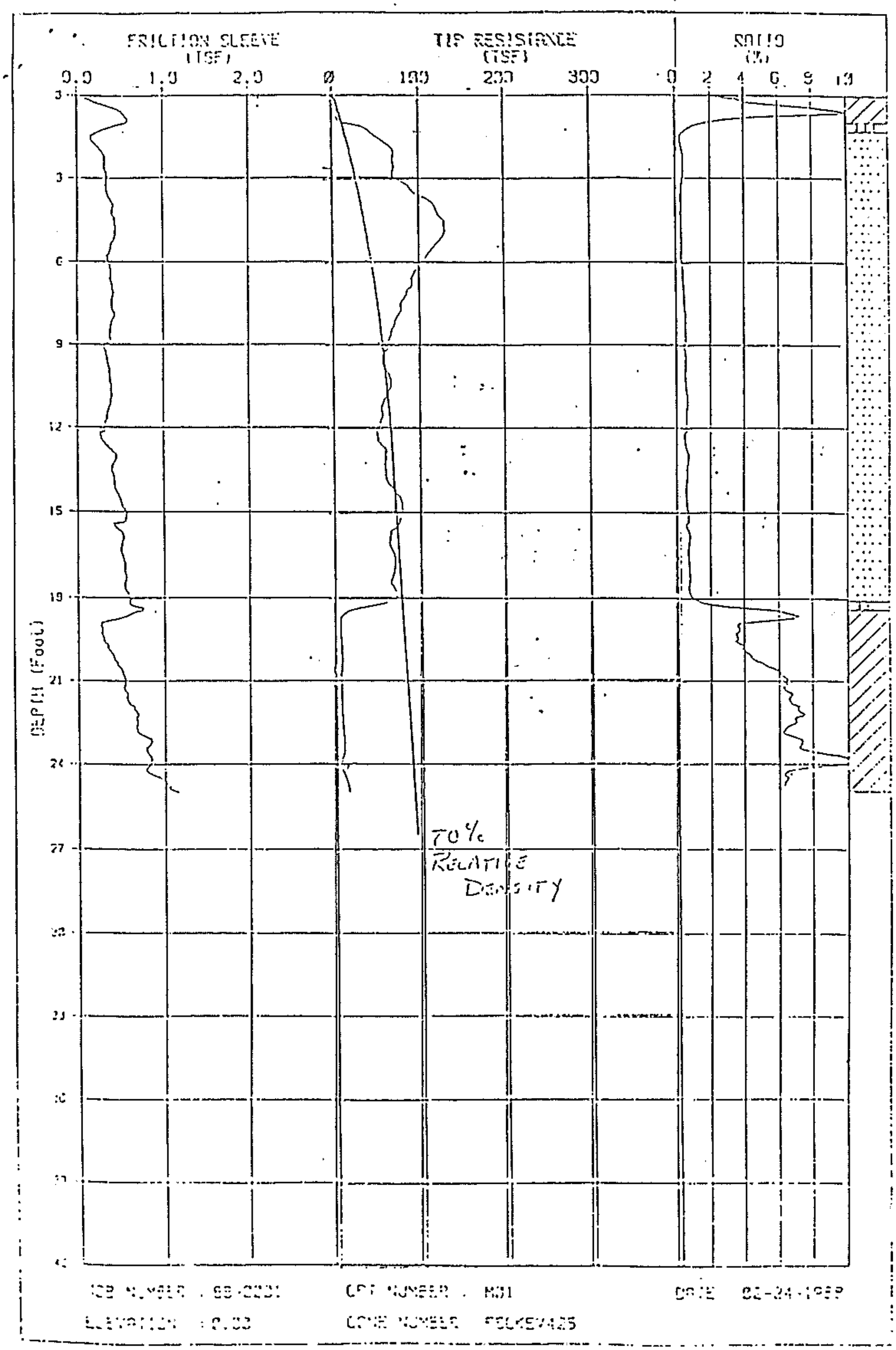
Before Compaction Grouting - Site 1

Figure 13



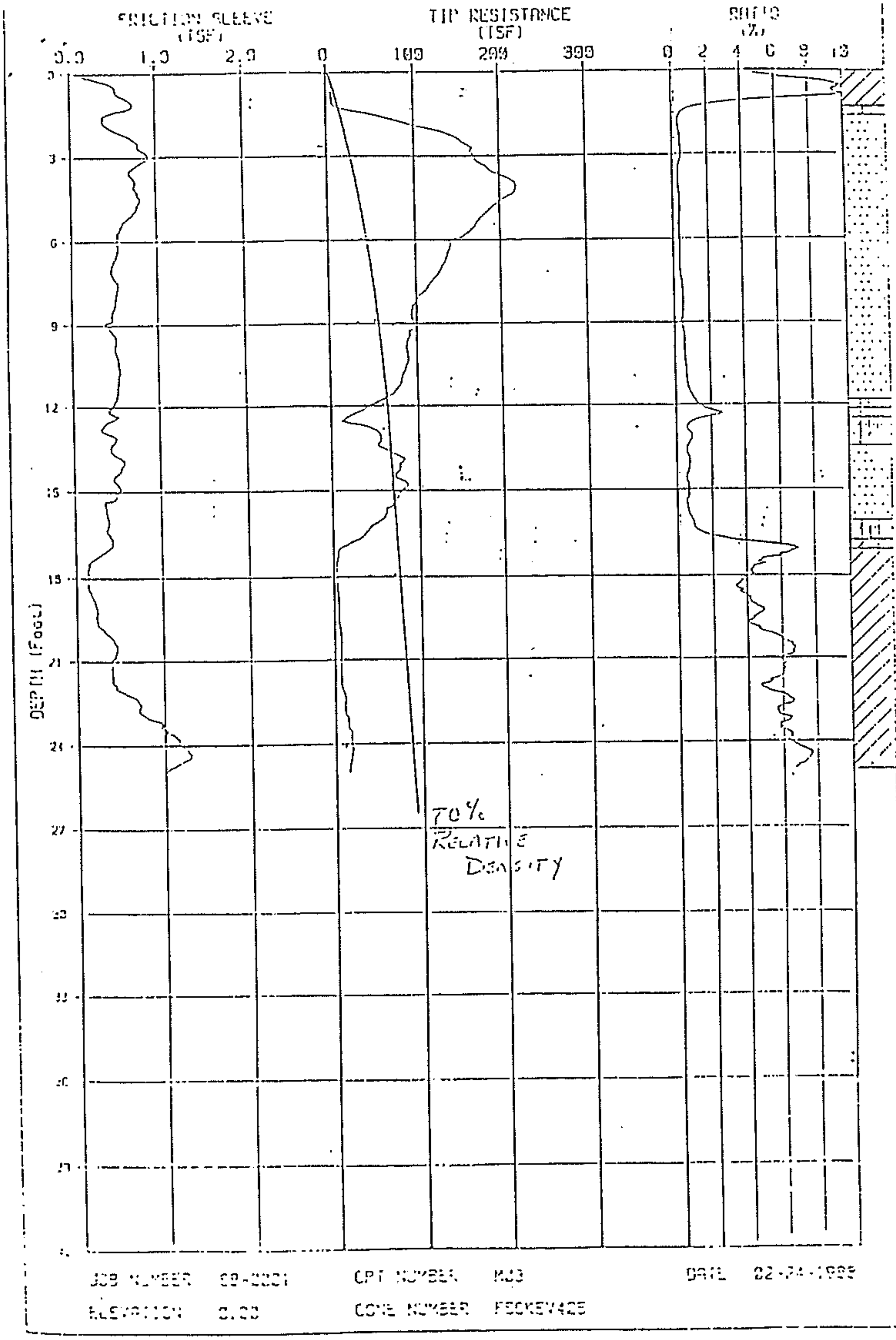
After Primary Grouting - CPT 1

Figure 14



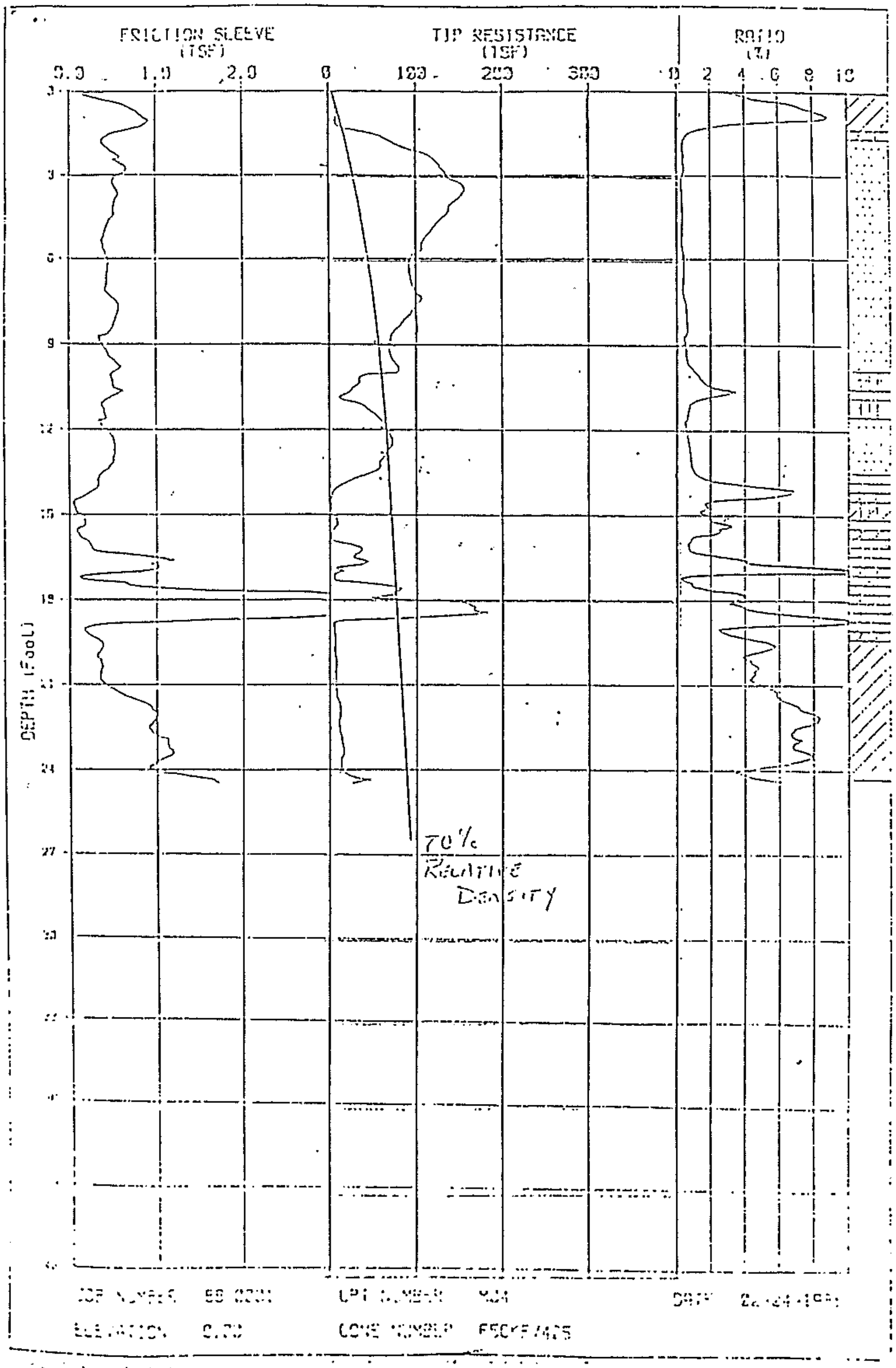
After Primary & Secondary Grouting - CPT 2

Figure 15



After Primary & Secondard Grouting - CPT 3

Figure 16



After Primary, Secondary, and Tertiary Grouting - CPT 4

Figure 17

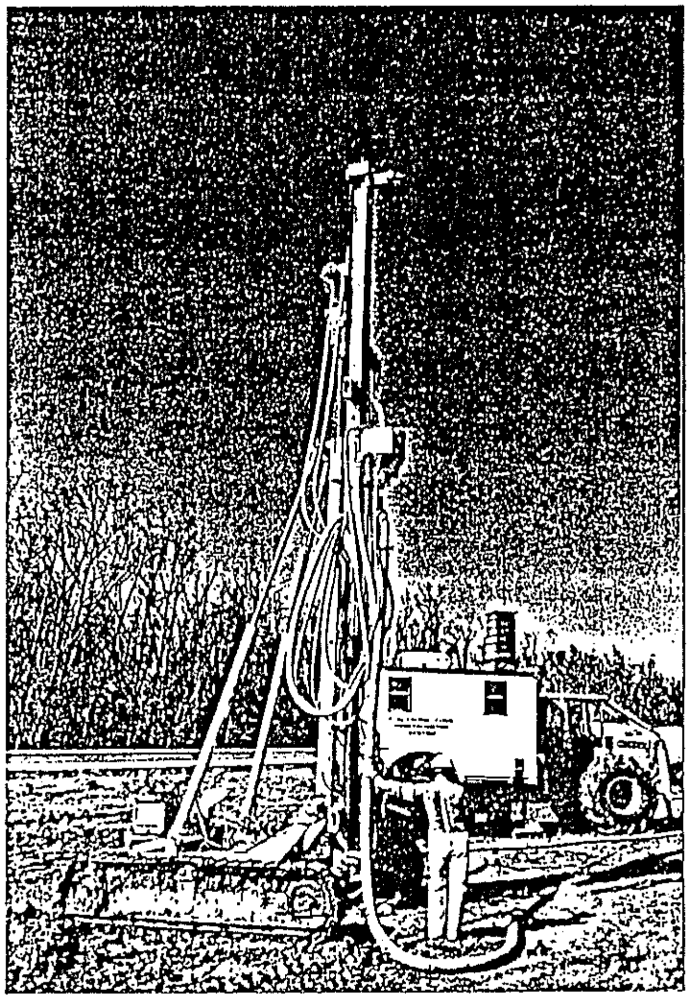
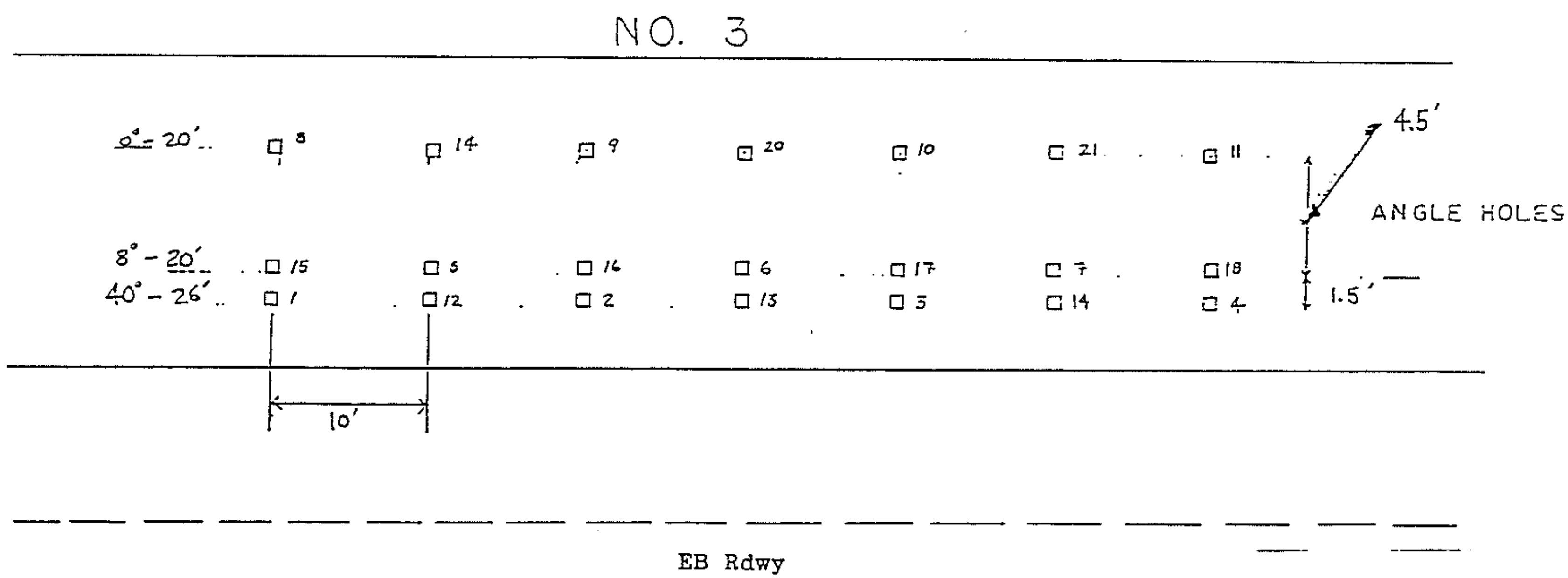
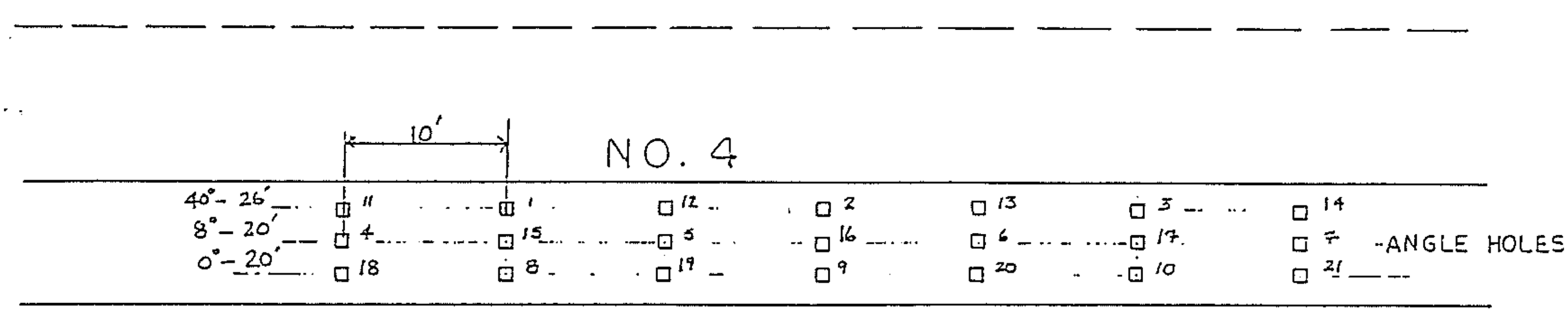


Figure 18. Grouting Rig



Outside Shoulder Angle Grouting

Figure 19



Inside Shoulder Angle Grouting

Figure 20

lanes. The next hole towards the shoulder was angled at 8 degrees from the vertical, with 20 feet of casing. The hole farthest from the travel lanes was vertical, with 20 feet of casing. Spacing for each line of angle grouting was 8 feet on center, parallel to the roadway.

Figure 21 shows the result of cone penetrometer testing after angle grouting. The cone tip resistance is clearly reaching the 70 percent relative density line. This method was considered a success.

ECONOMICS

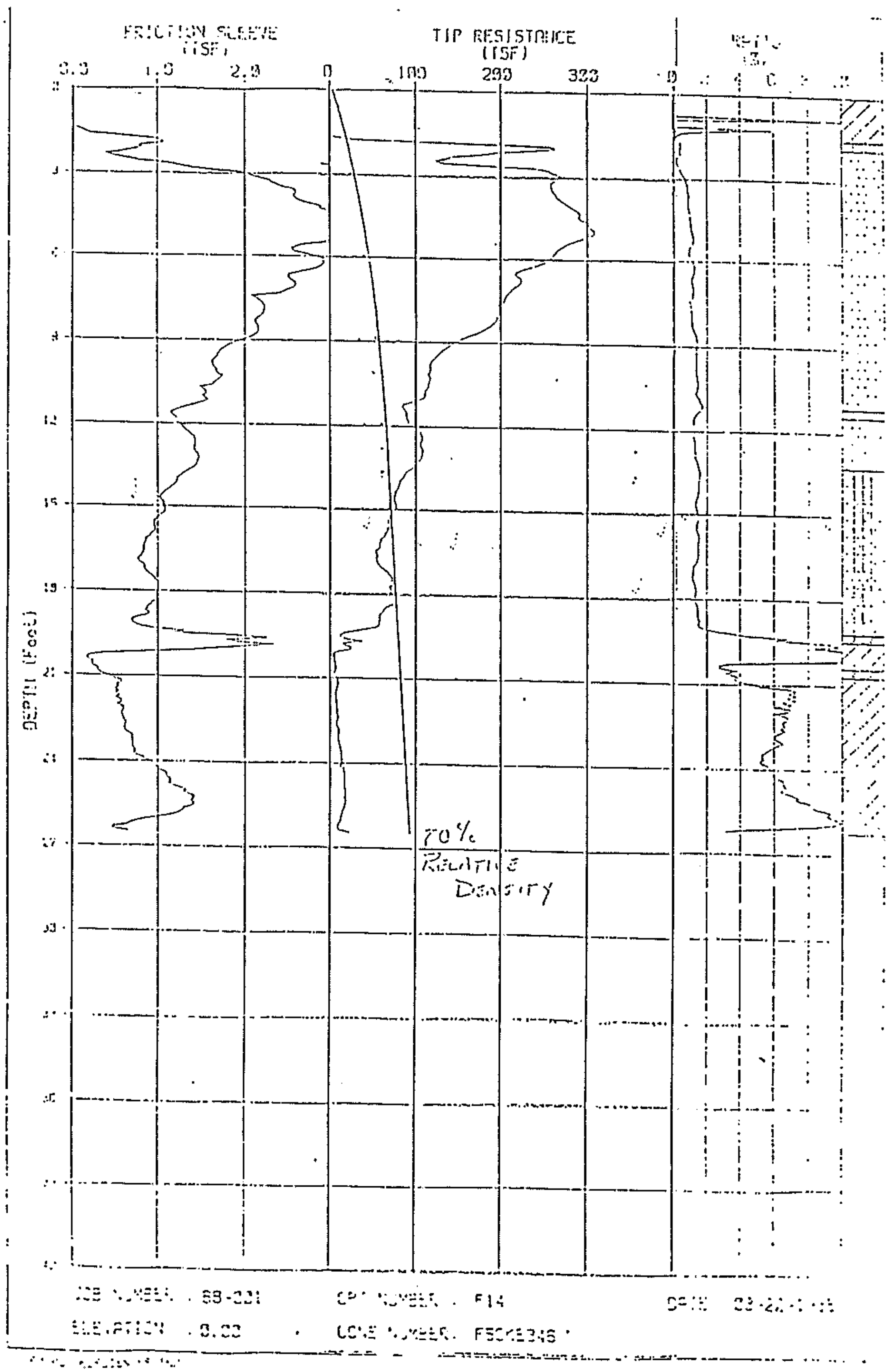
The principal object of this study was to find a technically feasible, economical, permanent solution to the lack of initial densification of the embankment fill of I-10 through McElroy Swamp. The following table is a cost comparison of permanent solutions:

METHOD	COST PER MILE	TOTAL COST
Elevated highway	* 10,000,000	87,700,000
Reconstructed fill and highway	* 5,000,000	43,850,000
Compaction grouting	1,800,000	15,800,000

* Engineering Estimates

The cost estimate of $1,800,000 per mile for the compaction grouting estimate was based on the experimental project. Verbal estimates by the contractor at the job site were considerably higher--in the range of $3 million per mile; however, they were based on an extremely high loss of time due to poor weather experienced during the experimental project. It is believed that the contractor's estimates are considerably higher than actual costs would be experienced with reasonable weather conditins.

Compaction grouting economically compares very favorably with other permanent solutions to the problem, but how does it compare in cost with a periodic asphalt overlay?



After Angle Grouting Beneath Pavement

Figure 21

To make this comparison, one must use a time-value of money approach. The cost figure of $15,800,000 for the one-time, permanent compaction grouting method will be used to compare with a four-year, repeated, overlay program, using the 1987 $4 million cost as reference. Overlay program will be necessitated because of the expected continuing settlement.

If $4,000,000, adjusted for inflation*, were spent every four years for overlays over a 25 year period, a total of $54,200,000 would be disbursed. This would be equivalent to investing $24,000,000 into a "maintenance fund" at the beginning of the project. It can readily be seen that the $15,800,000 cost for compaction grouting compares very favorable with the overlay alternate.

The present worth of $16,000,000 makes overlaying a close competitor of compaction grouting but the actual amount required to allow for four-year disbursement does not.

A more graphic example may be found on Figure 22. In this instance, the initial cost for the compaction grouting method, $15,800,000 was used as a datum. This amount was allowed to inflate at 5 percent for four years, then an adjusted $4 million was subtracted from it. This same process was repeated every four years until 25 years had elapsed. The $15.8 million is used up completely in the sixteenth year (third hot mix application), leaving a $436,575 deficit. Also, for the last two overlays (needed to complete the 25 year life cycle), will cost an additional $23,513,000.

From an economic standpoint, it is evident that compaction grouting is a better choice than repeating levelling overlay every four years.

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*5-percent annual inflation rate, 1987 as year 0, 25 year project life

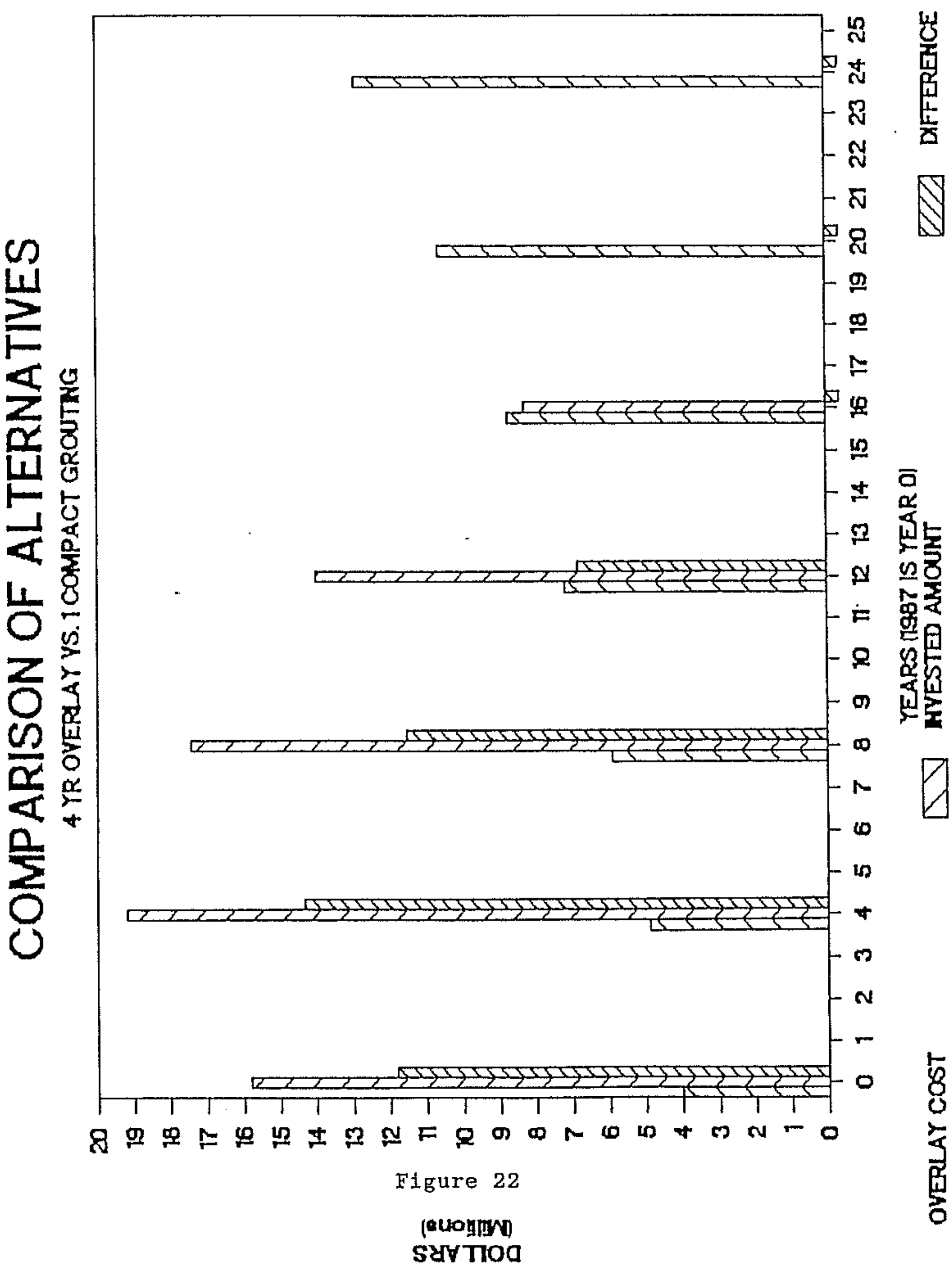
COMPARISON OF ALTERNATIVES
4 YR OVERLAY VS. 1 COMPACT GROUTING
DOLLARS (Millions)
YEARS (1987 IS YEAR 0)
OVERLAY COST
INVESTED AMOUNT
DIFFERENCE

Figure 22

It is also evident that if damage from differential settlement takes longer than four years, the economic advantage shifts to the overlay program. Also, if damage occurs in less than four years,the compaction grouting alternative is even more attractive.

While not an economical argument, it will eventually be impossible to continue a simple overlay program. It will become impossible to place a thin enough layer of asphalt that will stay in place at transitions to structures. Maintaining desired side slopes from shoulders will be extremely difficult as the roadway height changes relative to the underlying embankment. At some point, a permanent rehabilitation method will have to be chosen.

CONCLUSIONS

From our investigation the following conclusions may be drawn:

1. Dewatering using a single line of wellpoints is ineffective in removing enough water to increase effective stress to a beneficial level.

2. Injection of lime/fly ash is an ineffective means of stabilizing this fill, either because there is no reaction of fill material with the slurry, or the slurry can not be adequately dispersed through the fill, or both.

3. Compaction grouting is an economically attractive, technologically feasible means of successfully dealing with the problem of differential settlement in this fill.

RECOMMENDATIONS

As part of LTRC's involvement in this project, a long-term monitoring program has begun. The test and control areas are being surveyed monthly for movement and the roadway is tested by May's ridemeter on a six-month basis. As of this writing, January 1989, no movement has been noted.

The primary recommendation is to watch and wait. While we believe that differential settlement of the roadway is inevitable unless remedial measures are taken, there is no need to be premature in applying them.

We further recommend that once the roadway begins to measurably settle, and depending on the elapsed time, the remedial method of compaction grouting should be employed. The contract should be flexible, perhaps based on quantity of grout used, so that we can measure and adjust the angle grout spread to achieve maximum economy and still reach the desired relative density.

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