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COMPACTION GROUTING IN TALUS SLOPES OF GLENWOOD CANYON

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16. Abstract

Construction of I-70 through Glenwood Canyon in Colorado has required several viaduct structures to be constructed on the talus slopes of the canyon. The talus has a high percentage of voids, and there is much concern that settlement could occur under pier and abutment foundations. To overcome this problem, compaction grouting was utilized to treat the foundation soils. The procedure involved injecting, under high pressure, a very stiff (3-inch slump or less) soil-cement mortar to displace and thus compact the adjacent soils. Pressuremeter tests were used before and after the compaction grouting to evaluate the effectiveness of this treatment method.

The results indicate that the ground densities have improved significantly after the treatment, and it is evident that the compaction grouting is effective in talus material. The bridge was monitored for more than a year, and it has been evidenced that the maximum settlements of the bridge piers and abutments did not exceed 0.96 inches.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF PHOTOGRAPHS

COMPACTION GROUTING

IN

TALUS SLOPES OF GLENWOOD CANYON

INTRODUCTION

Construction of I-70 through Glenwood Canyon in Colorado has required several viaduct structures on the West half of the Canyon.

Many viaduct structures are planned to be located along steep hillsides that are composed of deep talus slopes. The talus has a high percentage of voids, and there is much concern that settlement could occur under the pier and abutment foundations. A variety of alternatives including piling and drilled shafts were considered to minimize the possible settlements. Piling could not be driven into the coarse grained colluvium. Drilled shafts and hand excavated shafts were evaluated by hiring three firms to prepare estimates for performing the work. It was the conclusion of each firm that drilled and hand excavated shafts would be prohibitively expensive alternatives. Grouting was then considered as an alternative. As part of the preliminary geological investigation, Colorado Department of Highway personnel attempted consolidation grouting at a selected pier location using a cement and fly-ash mixture. The permeability of the talus was such that the grout migrated well beyond the proposed footing perimeter.

Compaction grouting was evaluated by hiring the Hayward Baker Company of Maryland to treat a selected pier location. This method appeared to be successful; however, a before and after cross-hole seismic study performed by Dr. Woods of the University of Michigan to verify the increase in density of the treated area could not be completed due to the disruption of the monitoring casings during the grouting operation. Therefore, this research study was initiated to evaluate the effectiveness of compaction grouting in talus material.

OBJECTIVES OF THE STUDY

The main objectives of this study were as follows:

1. Determine the effects of compaction grouting in talus material using pressuremeter test results,
2. Evaluate the long-term performance of grouted bridge piers and abutments on talus.

Pressuremeter tests were to be performed before and after the compaction grouting operation to determine the increase in ground material densities. The bridge piers and abutments were then monitored for more than a year to evaluate the postconstruction settlements after completion of the bridge structure.

PROJECT LOCATION AND GEOLOGY

The construction of I-70 through Glenwood Canyon is a 300 million dollar project that includes a variety of structures to complete the construction of I-70 through a narrow canyon. The Canyon is 12 miles long, just east of Glenwood Springs in Western Colorado, and about 150 miles west of Denver as shown in Figure 1. The viaduct structures are part of this project to expand the present two-lane traffic road to four lanes in the near future.

Based on geological investigations, two bridge sites were originally selected to be treated by the compaction grouting method. The bridge sites are located on colluvial slopes inclined at between 12 and 40 degrees to the south. The slopes consist of relatively stable talus composed of angular fragments of quartzite ranging from gravel to boulder size, intermixed with

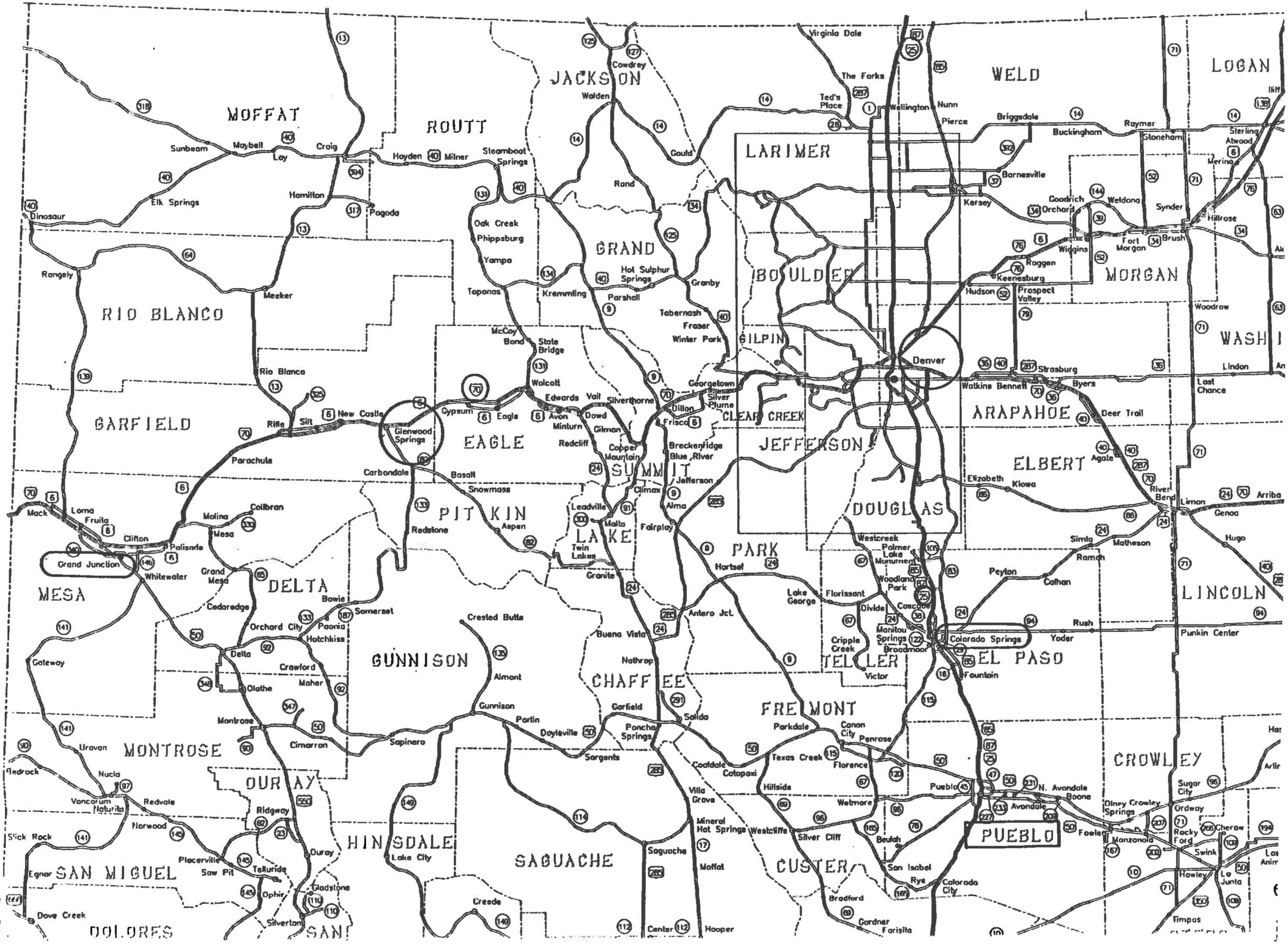


FIGURE 1 GENERAL MAP OF THE STATE OF COLORADO

silt and sand-size particles. The talus contains areas of empty or very loosely filled interconnected voids ranging up to several inches across. Underlying the talus at depth between 30 and 50 feet are denser deposits of river terrace sands and gravels. Depth to bedrock varies from 21 to more than 100 feet.

Project I-70-2(112) was selected for compaction grouting treatment. A long-term monitoring program along with a series of pressuremeter tests were also included to determine the effectiveness of compaction grouting in talus material.

Figure 2 illustrates the plan and profile of the bridge selected for monitoring. All pressuremeter tests were conducted in holes drilled at piers No. 2 and 4. This figure also includes information regarding the foundation soil profile. In general, the material in this site consisted of boulders, cobbles, gravel, sand, and very little silt. Photographs 1 and 2 show the talus slopes in Glenwood Canyon adjacent to the proposed bridge on Project I-70-2(112). Photograph 3 is a close-up view of the talus material.

COMPACTION GROUTING

Compaction grouting is a specialized ground modification process that has been used extensively over the last 20 years to provide in-place desification of soft or loose soils. The process involves the injection, under high pressure, of very stiff (3" slump or less) soil-cement mortar to displace and thus compact the adjacent soils as shown in Figure 3.

Compaction Grouting can be readily performed inside structures and other confined spaces, and its execution results in only minor interference with other operations, as large equipment is not required in the immediate injection area. Historically, its most extensive use has been in connection with settlement correction, however it has also been used for modification of

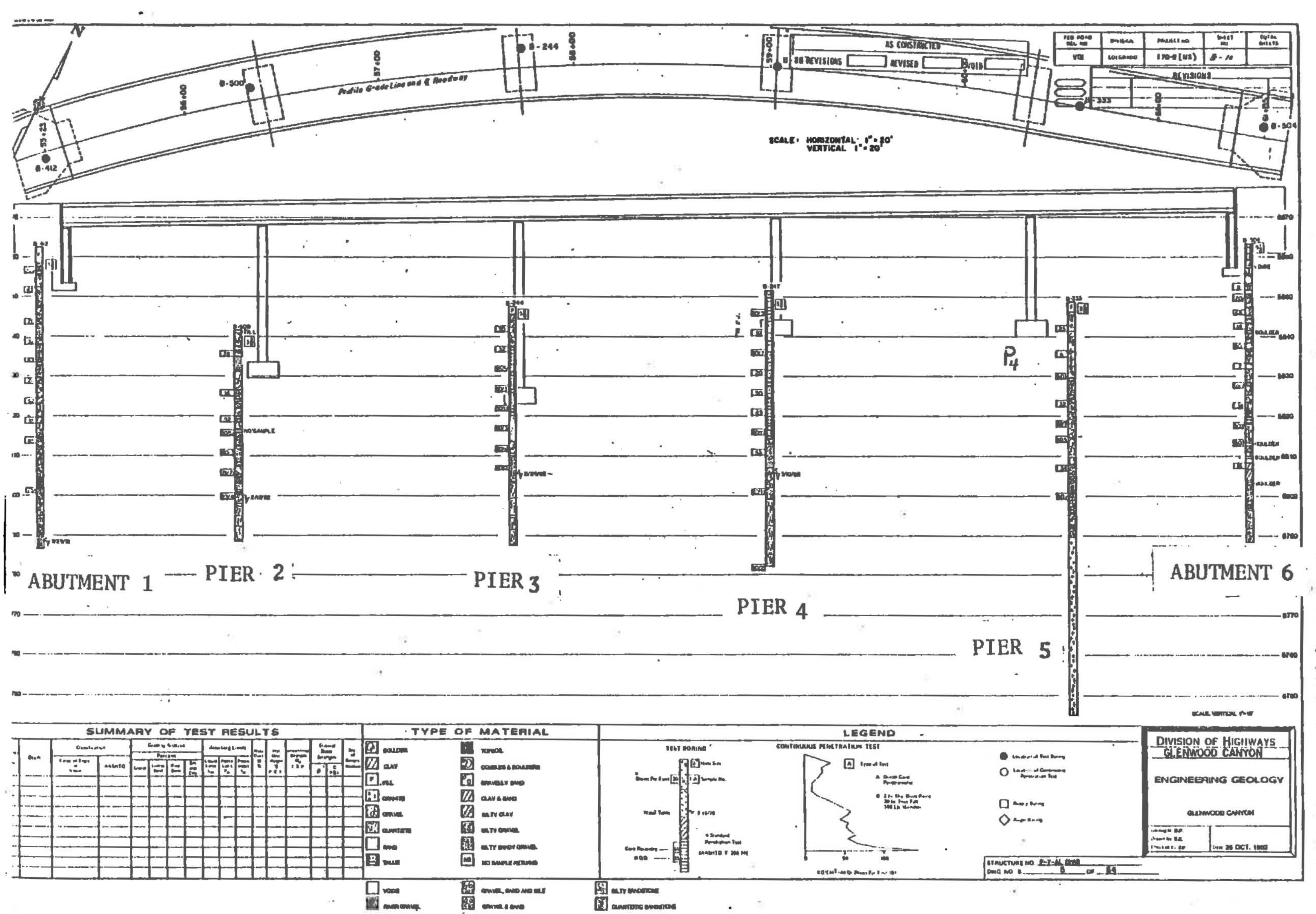
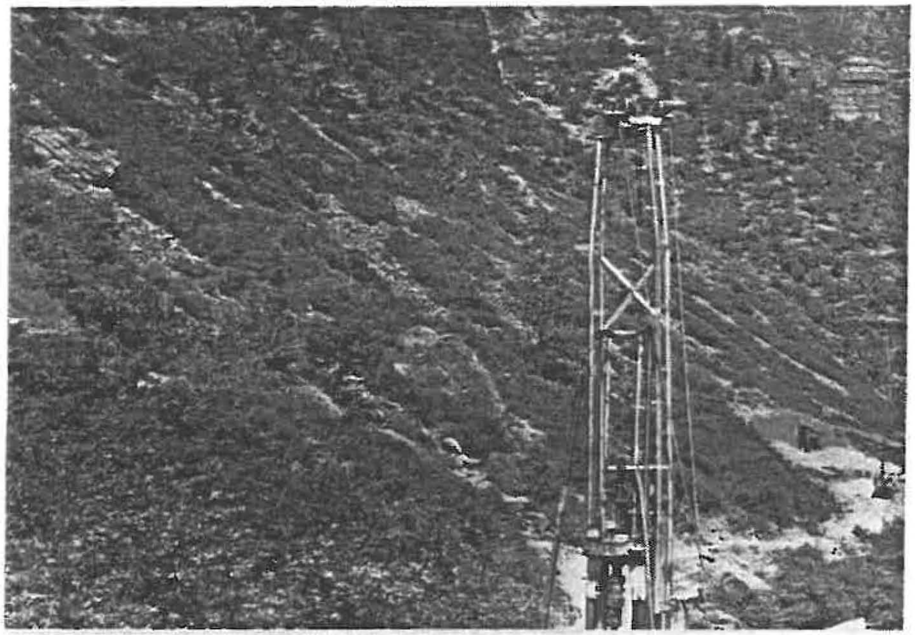


FIGURE 2 VIEW OF THE SELECTED BRIDGE FOR MONITORING



Photographs 1 & 2
Slopes adjacent
to the bridge
site.

Photograph 3
View of the
Talus Material

COMPACTION GROUTING

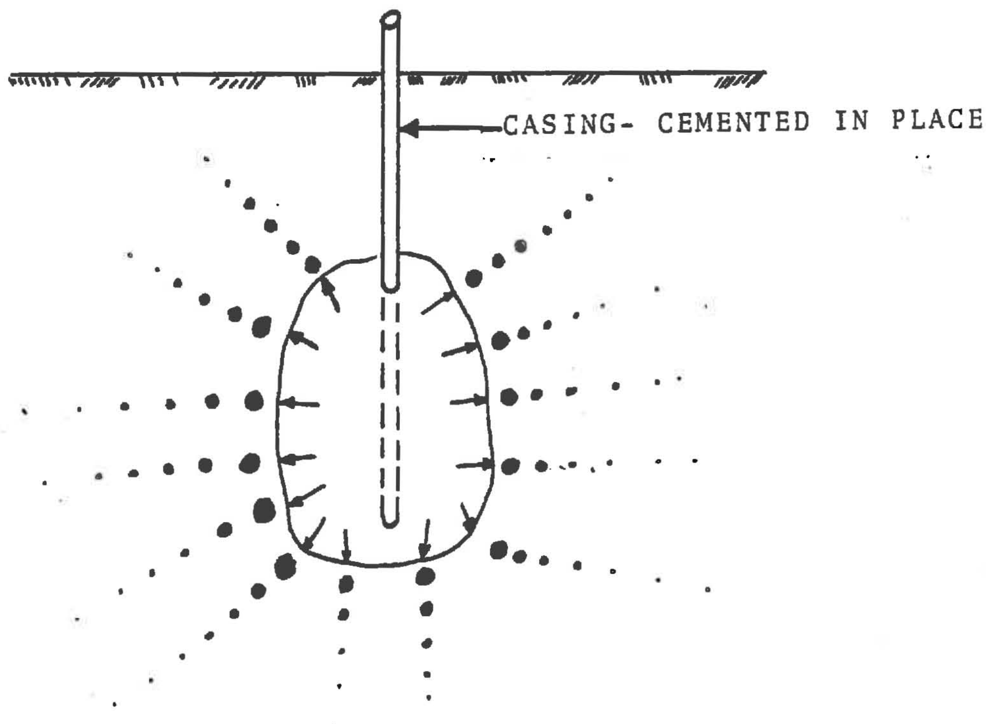


FIGURE 3: INCREASING MASS OF GROUT UNDER INJECTION PRESSURE CAUSES DISPLACEMENT AND COMPACTION OF THE SURROUNDING SOILS.

in situ soils to reduce the liquefaction potential during earthquakes (1), and as a "tool of construction" to limit ground movement during soft ground tunneling (2).

The procedure was first described by Graf (3) in 1969. Mitchell (4) presented comparisons of the procedure with other grouting methods, in particular differentiating between the "penetration" and displacement or "compaction" mechanisms, in 1970. Applicability of the procedure and the mechanics of injection, including a review of original research and development leading to the then current technology, were described by Brown and Warner (5) in 1973. Criteria for planning and performing compaction grouting projects were presented in 1974 by Warner and Brown (6).

Although the exact mechanism of soil modification is not thoroughly understood, a basic understanding of the injection effect is evolving. It has been fairly well established that the expanding mass of grout results in a complex system of radial and tangential stresses within the soil. Immediately adjacent to the expanding grout mass, shearing and plastic deformation will occur. In this zone the density of the in-place soil could actually be reduced as a result of the disturbance. As the distance from the soil-grout interface increases, the deformation will be essentially elastic, and an appreciable increase in density can be expected.

APPLICATION OF COMPACTION GROUTING

The compaction grouting subcontract was awarded to Geo-Con, Inc. Based in Pittsburgh, Pennsylvania. The contractor's field personnel included one superintendent, one driller, two laborers to assist in drilling and grouting, and one concrete-mobile mix operator.

Two-inch grout casings were used in each hole to keep the holes open during the operation. The treatment was advanced from bottom of the hole to the top. Grout with 3-inch maximum slump was injected on an approximately continuous basis throughout the length of the hole with the grout casing being withdrawn in increments of 10 to 11 inches. When one or more of the following criteria were met, the grout casing was raised to the next increment:

1. The grout pressure at the header gage exceeded 1000 PSI.
2. More than five cubic feet of grout had been injected per one foot interval.
3. Ground or slope movement had occurred, as determined by the CDOH field inspector

Figures 4 and 5 illustrate the grouting pattern for the abutment and pier footings.

PRESSUREMETER TEST

The pressuremeter was used to test the ground stiffness both before and after the compaction grouting treatment to determine the degree of improvement and consolidation in the talus material. This test is essentially an in situ lateral load test carried out in a borehole. Analysis of the resulting stress/deformation diagrams for each interval of penetration permits evaluation of the mechanical properties of the soil. For this study, the pressuremeter tests provided the values of deformation modulus both before and after the compaction grouting. These values were then compared to determine the degree of improvement of the ground densities due to the compaction grouting treatment.

Details of the pressuremeter test are presented in Appendix A.

GLENWOOD CANYON PROJECT
STRUCTURE F-17-AL (2W)
CONCRETE ALTERNATE

GROUTED INTERVAL	FROM	TO
HOLES 1-5	5825	5860
HOLES 6-11	5825	5855
HOLES 12-16	5825	5860

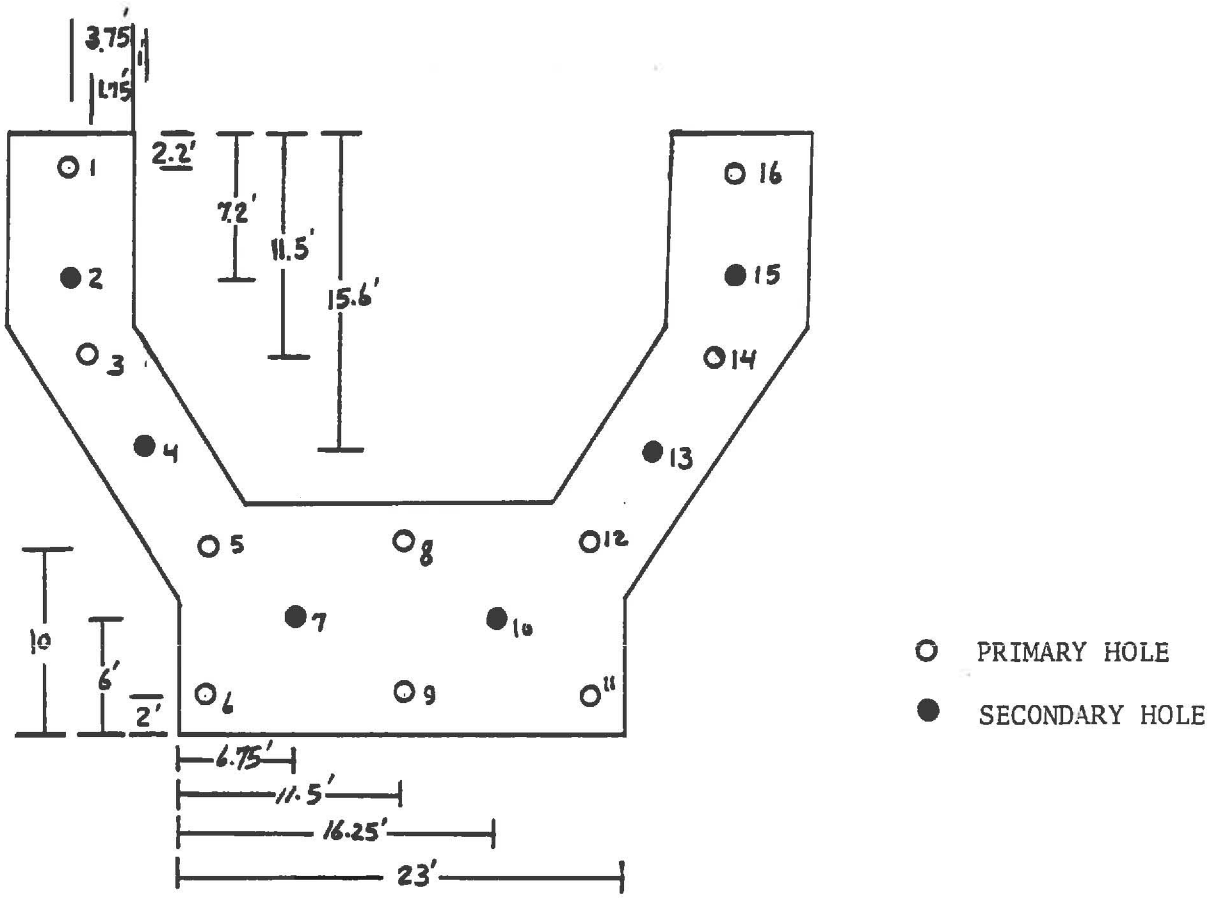


FIGURE 4 PLAN VIEW OF ABUTMENT 1 WITH LOCATIONS OF THE GROUTED HOLES

I 70-2(70)
GLENWOOD CANYON PROJECT
STRUCTURE F-17-AL (2W)
CONCRETE ALTERNATE

GROUTED INTERVAL

	FROM	TO
PIER 2	5815.5	5830.5
PIER 3	5810	5824
PIER 4	5821	5841
PIER 5	5818.5	5840.5

○ PRIMARY HOLE

● SECONDARY HOLE

ALL PIERS

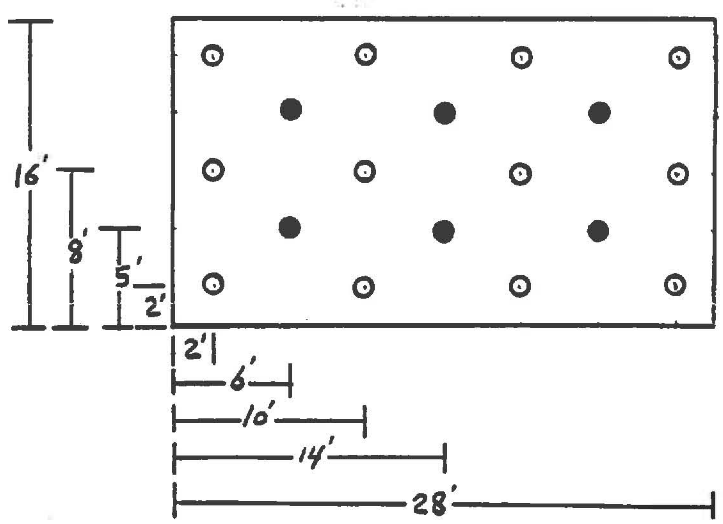


FIGURE 5 PLAN VIEW OF ONE OF THE BRIDGE PIERS WITH LOCATIONS OF THE GROUTED HOLES

RESULTS OF THE PRESSUREMETER TESTS

Results of pressuremeter tests conducted both before and after the compaction grouting treatment are presented in figures 9 through 15. At Pier No. 2, two tests were conducted successfully both before and after the treatment. At Pier No. 4, two successful tests were conducted prior to the treatment, and only one successful test was performed after the treatment. The value of the deformation modulus was then calculated for each test for comparison purposes. Table 1 is the summary of the pressuremeter test results both before and after the compaction grouting. These results suggest that the values of the deformation modulus have increased by a factor from 5 to 49 as a result of the treatment. This, in turn, indicates that the foundation soils have been consolidated and contain much higher densities.

Deformation modulus (Menard Modulus) Em is then related to Young's modulus, E, by the following expression:

$$E = E_m/\alpha$$

E = YOUNG'S MODULUS

E_m = MENARD DEFORMATION MODULUS

α = FACTOR RELATING E AND E_m

The value of α suggested by Menard for normally consolidated sand and gravel is 0.25. This is the closest approximation, and it is hoped to represent the talus material.

Once the value of Young's modulus is established, various formulas could be used to calculate the settlements. One of the more famous formulas is the Skempton's Expression:

$$S = \frac{\pi \cdot q \cdot d}{4\ E} (1 - \nu^2)$$

S = settlement

q = unit bearing stress

d = the diameter of the circle with an area which is equivalent to the area of the footing

= 2 (B /π) for a square footing of width B.

E = E /α

= Poisson's ratio

α = 0.25

The above formula implies that the value of Young's Modulus, E, and Menard Modulus, Em, are directly related. Therefore, if Young's modulus increases the Menard Modulus will increase; and therefore, the total settlements will decrease according to the above formula.

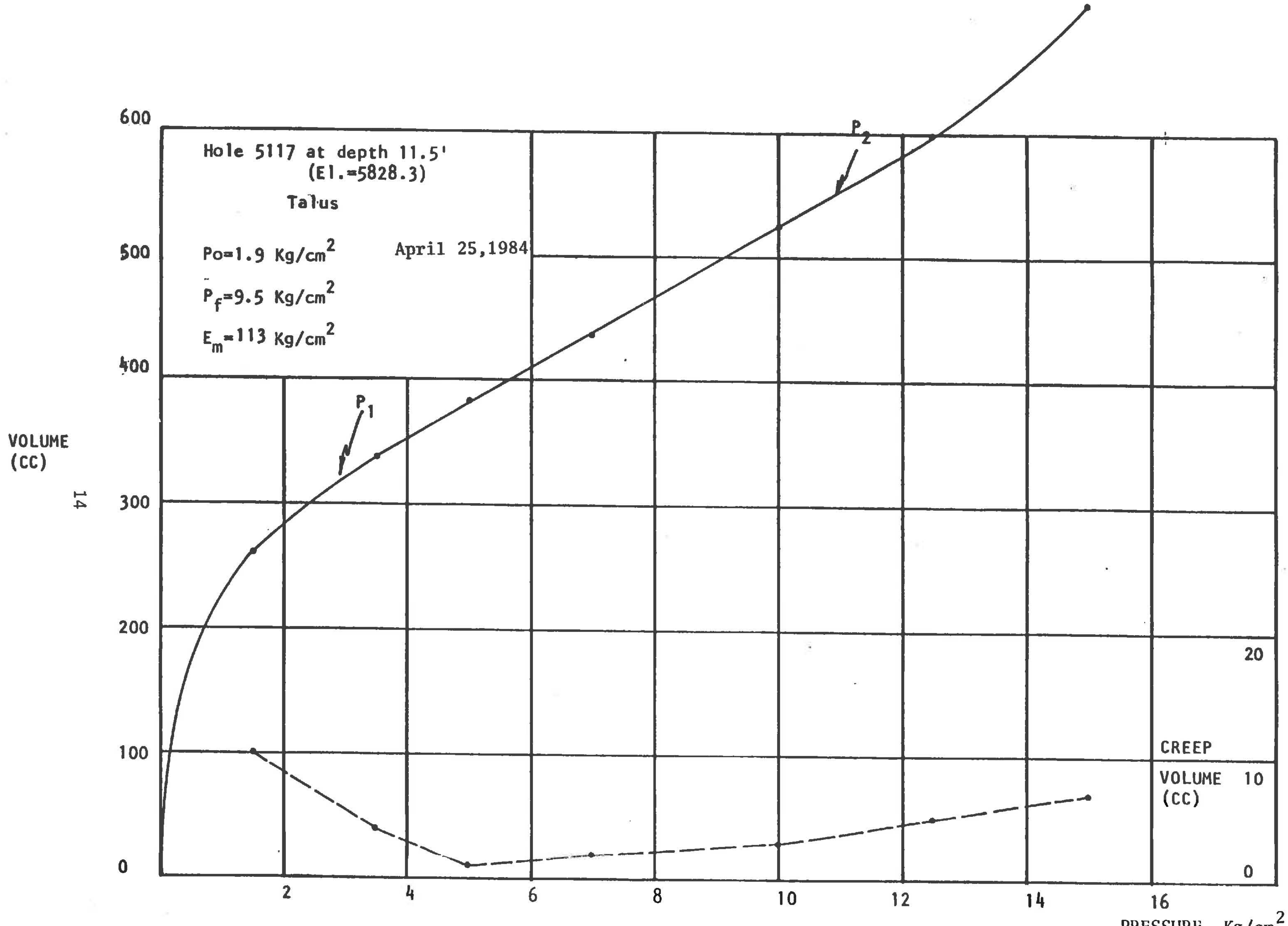


FIGURE 6 PRESSURE VS. VOLUME CHANGE AT PIER NO. 2 PRIOR TO COMPACTION GROUTING

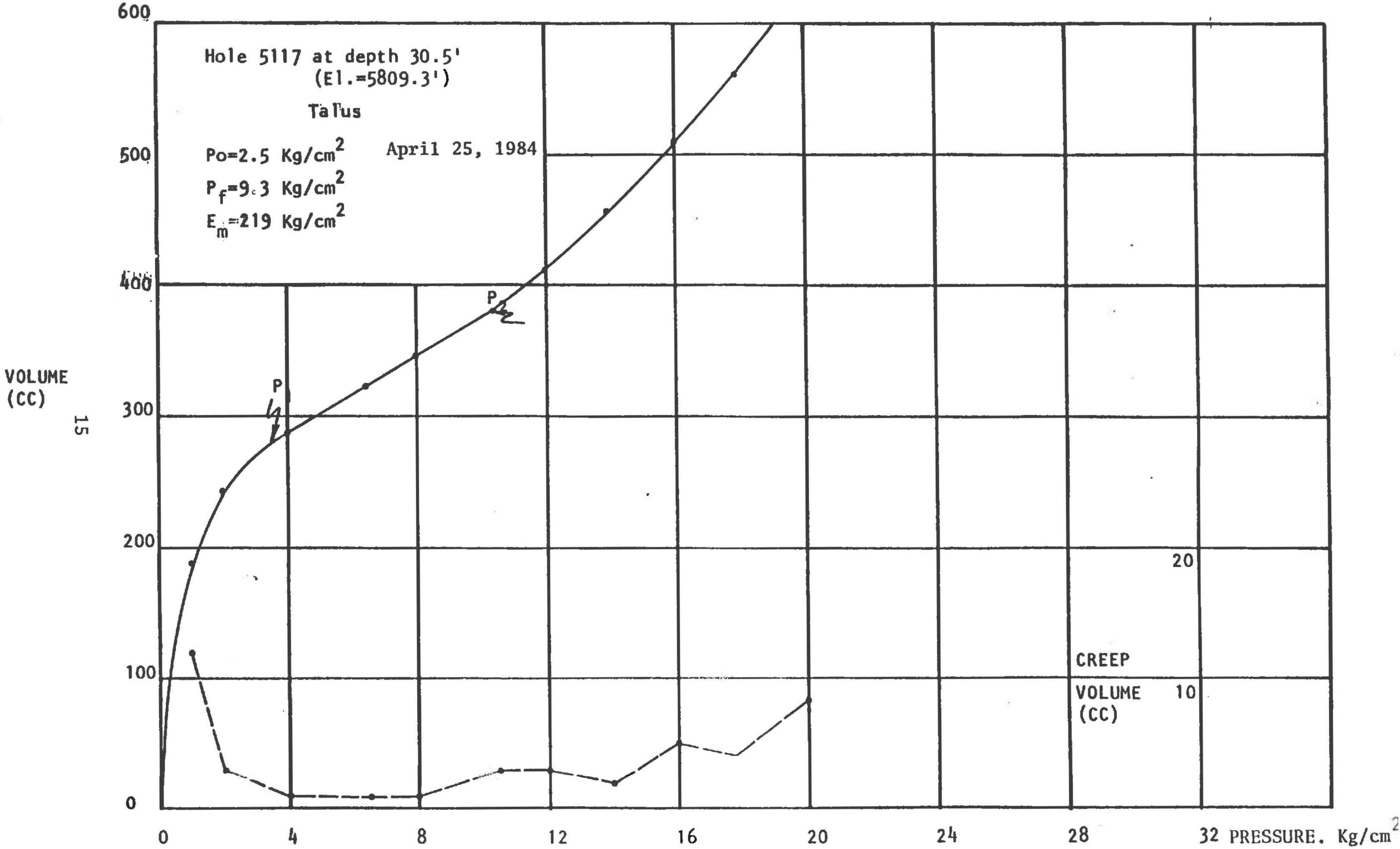


FIGURE 7 PRESSURE VS. VOLUME CHANGE AT PIER NO. 2 PRIOR TO COMPACTION GROUTING

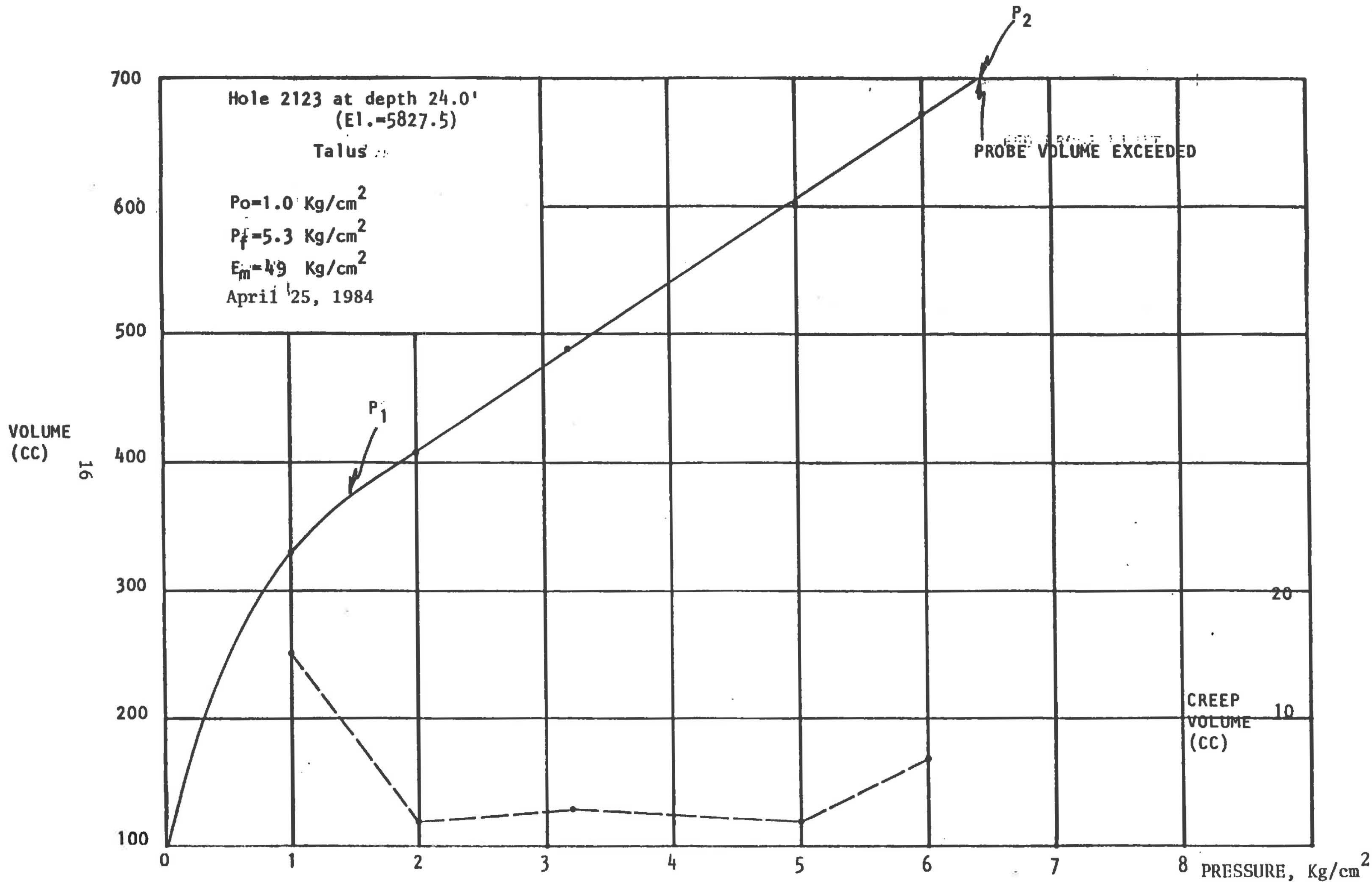


FIGURE 8 PRESSURE VS. VOLUME CHANGE AT PIER NO. 4 PRIOR TO COMPACTION GROUTING

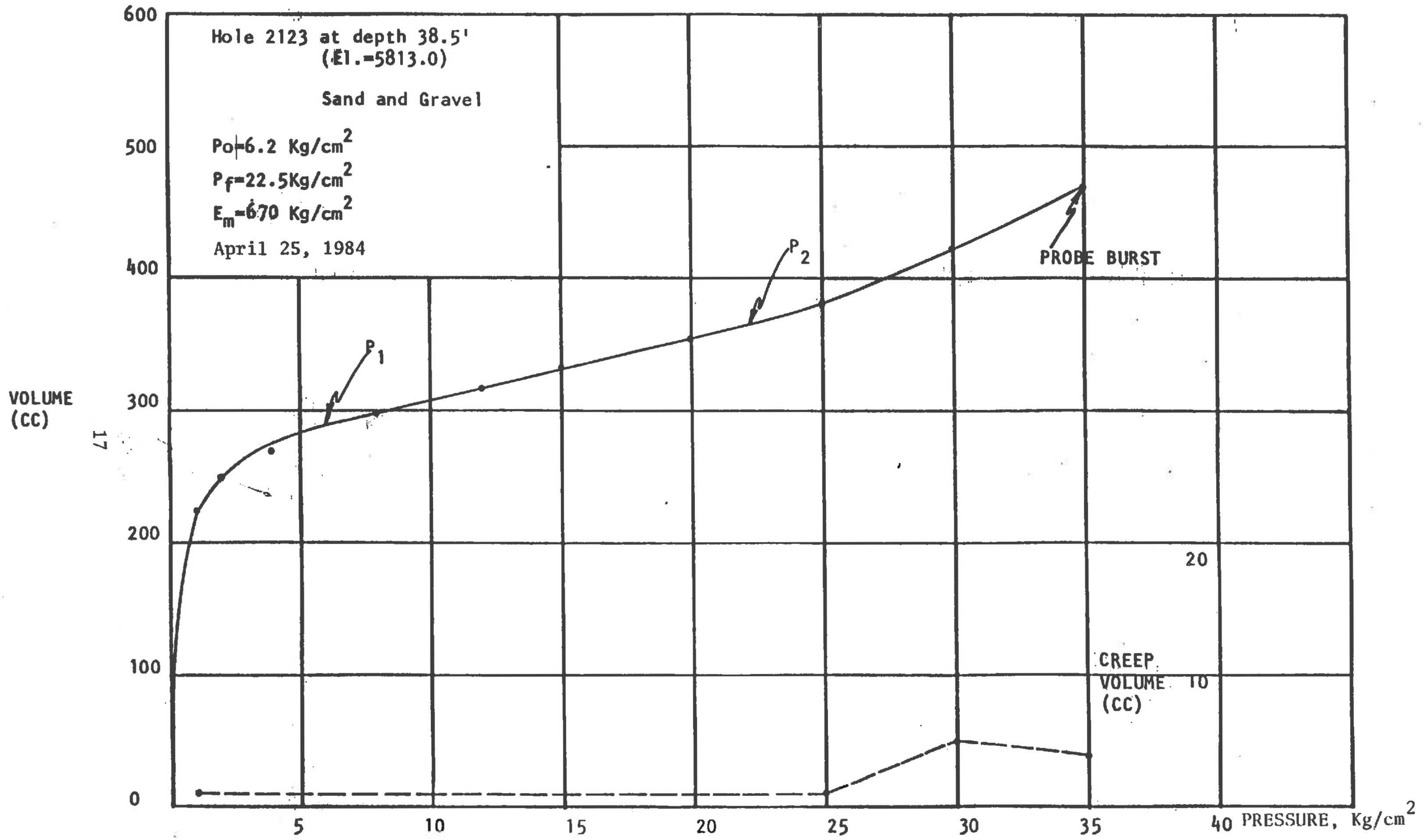


FIGURE 9 PRESSURE VS. VOLUME CHANGE AT PIER NO. 4 PRIOR TO COMPACTION GROUTING

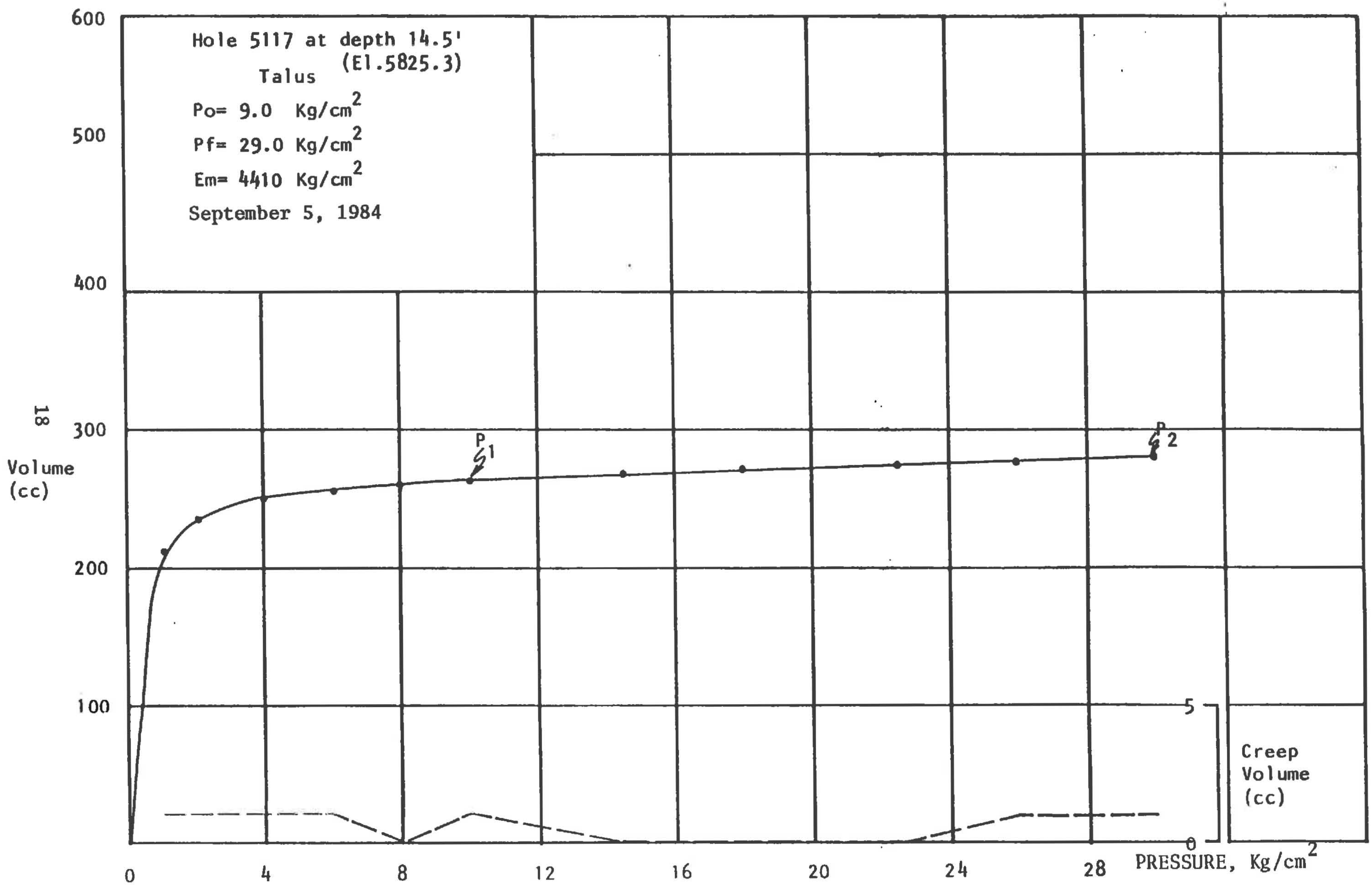


FIGURE 10 PRESSURE VS. VOLUME CHANGE AT PIER NO. 2 AFTER COMPACTION GROUTING

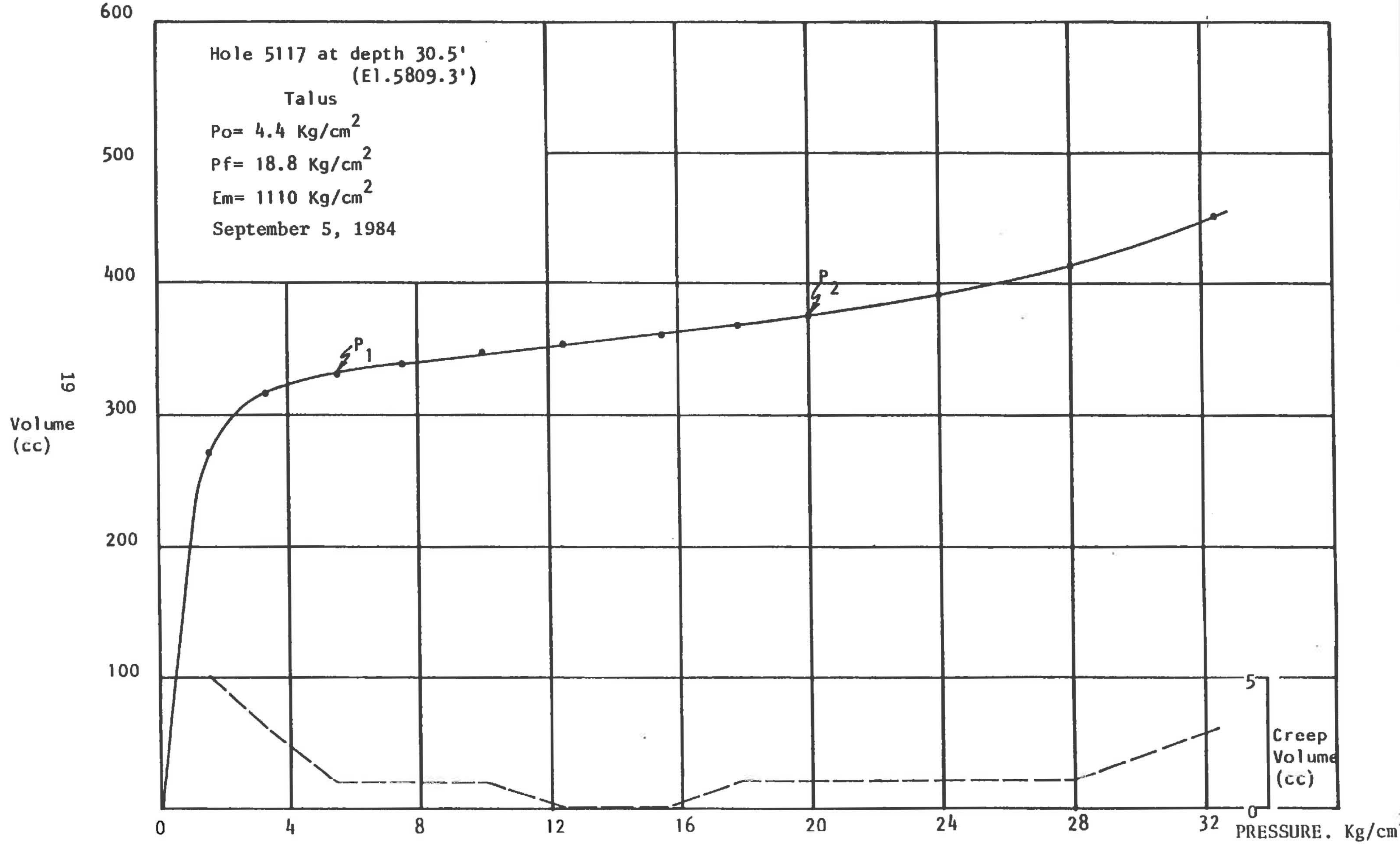


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FIGURE **11** PRESSURE VS. VOLUME CHANGE AT PIER NO. 2 AFTER COMPACTION GROUTING

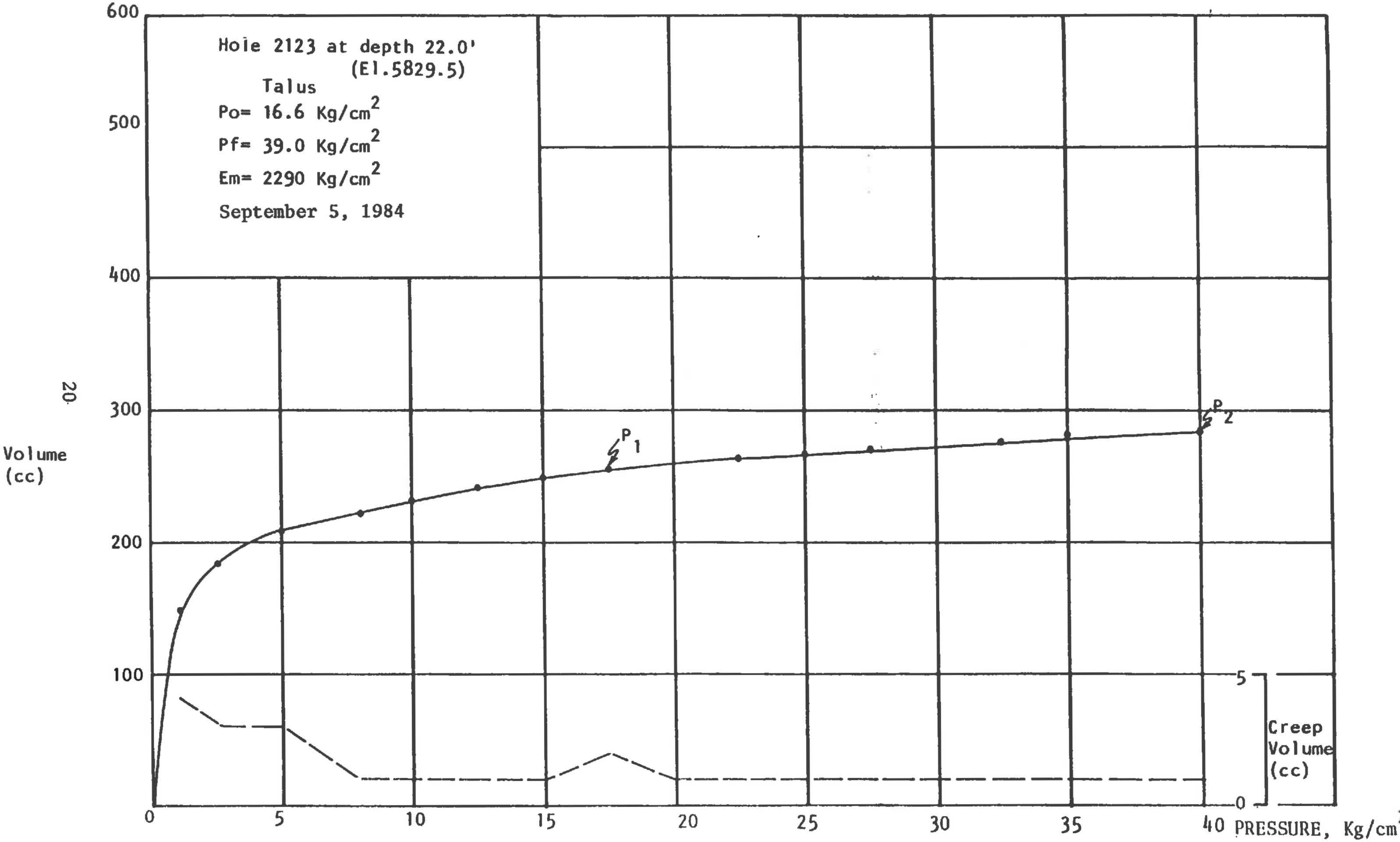


FIGURE 12 PRESSURE VS. VOLUME CHANGE AT PIER NO. 4 AFTER COMPACTION GROUTING

TABLE 1 SUMMARY OF THE PRESSUREMETER TEST RESULTS

	DEFORMATION MODULUS, E_m, kg/cm^2			
	Pier 2		Pier 4	
Depth (ft)	Before Treatment Apr. 85	After Treatment Sept. 85	Before Treatment Apr. 85	After Trea Sept 85
11.5	113	4410		
30.5	219	1110		
24.0			49	2290
38.5			670	-

CONCLUSION

The results obtained from the pressuremeter tests indicate that the values of Menard modulus, Em, increased 5 to 49 times after compaction grouting treatment. Thus, it is safe to assume that the total settlements after construction of the bridges will be reduced significantly compared to the values prior to compaction grouting.

The construction of the bridge was completed during 1985, and its performance has been evaluated by means of the surveying techniques. The results indicate that the bridge piers and abutments had negligible movements 18 months after the completion of the bridge structure. The maximum settlement took place at Pier No. 2 and it was measured to be 0.96 inches.

The excessive movements of the bridge structures built on top of the talus materials was of major concern to the engineers involved with these projects. The compaction grouting technique proved to be an effective treatment method, and its use is recommended for the stabilization of the foundations composed of talus materials.

APPENDIX A

DETAILS OF THE PRESSUREMETER TEST

DETAILS OF THE PRESSUREMETER TEST

The pressuremeter consists of three parts as illustrated in Figure A-1: The probe, the control unit, and the tubing.

In Figure A-1, the cell is at the bottom of the hole to insure that the cavity is constrained to expand only laterally. Water is used inside the probe to pressurize the measuring cell and to measure the resulting volume change at the control unit. The probe is impervious rubber bladder secured at top and bottom by guard cells which are inflated, usually by gas, to the same pressure as the measuring cell. The inflated guard cells effectively seal off the borehole and prevent the measuring cell membrane from expanding into the voids. Then with no pressure differential between the measuring cell and the guard cells there is no unbalanced force to cause the measuring cell to change in length. The rubber membrane is sufficiently flexible to ensure that a uniform pressure is applied to the walls of the hole, and the presence of the guard cells means that a longer length of borehole is pressurized than would be the case with just the measuring cell alone. Thus, plain strain conditions may be assumed in the soil around the measuring cell.

The control unit is located at a convenient spot on the ground surface close to the hole, as shown in Photograph A-1, and its function is to control and monitor the expansion of the probe. It does this by applying a given pressure to the probe and measuring the volume change of the measuring cell. The pressure source is a bottle of compressed gas; the flow of water to the measuring cell is monitored using a graduated cylinder called the volumeter.

Tubing is required to allow flow of water and gas between the control unit and the probe.

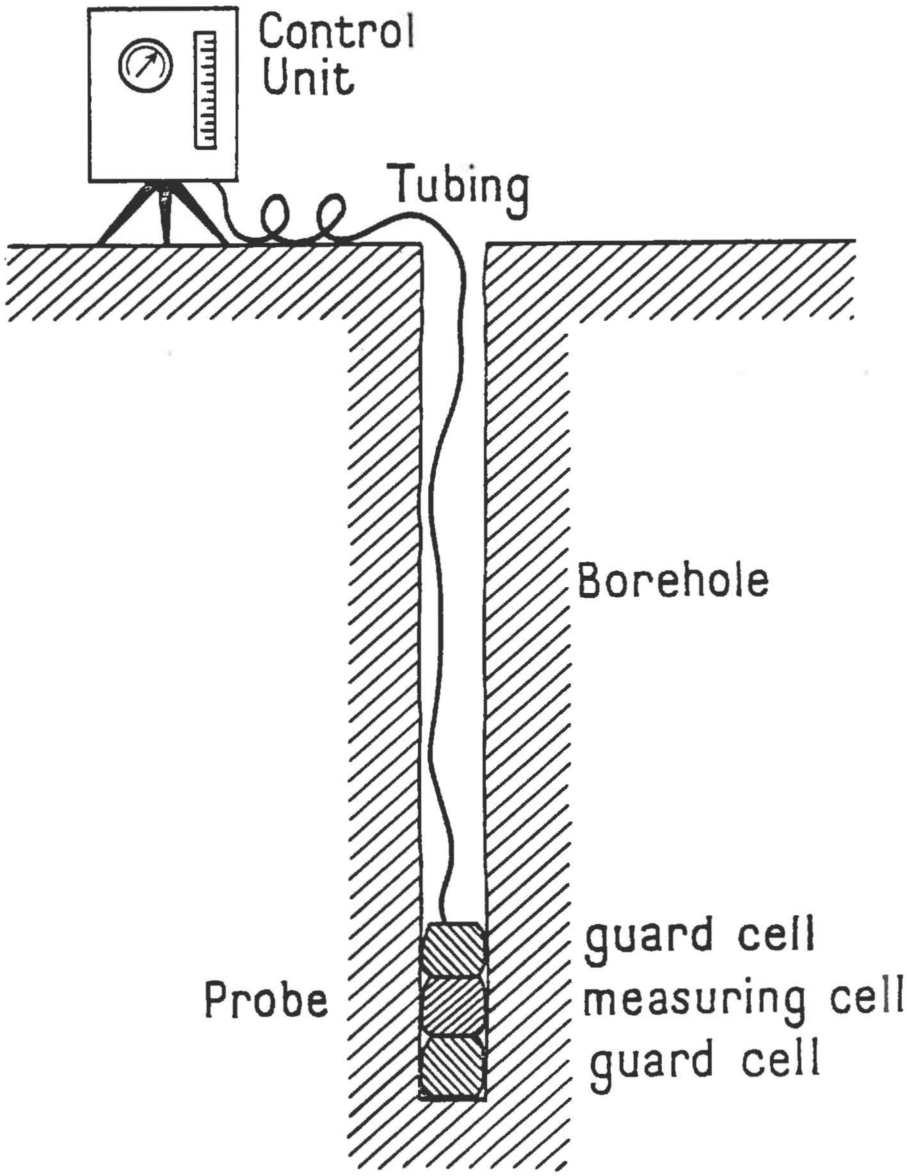
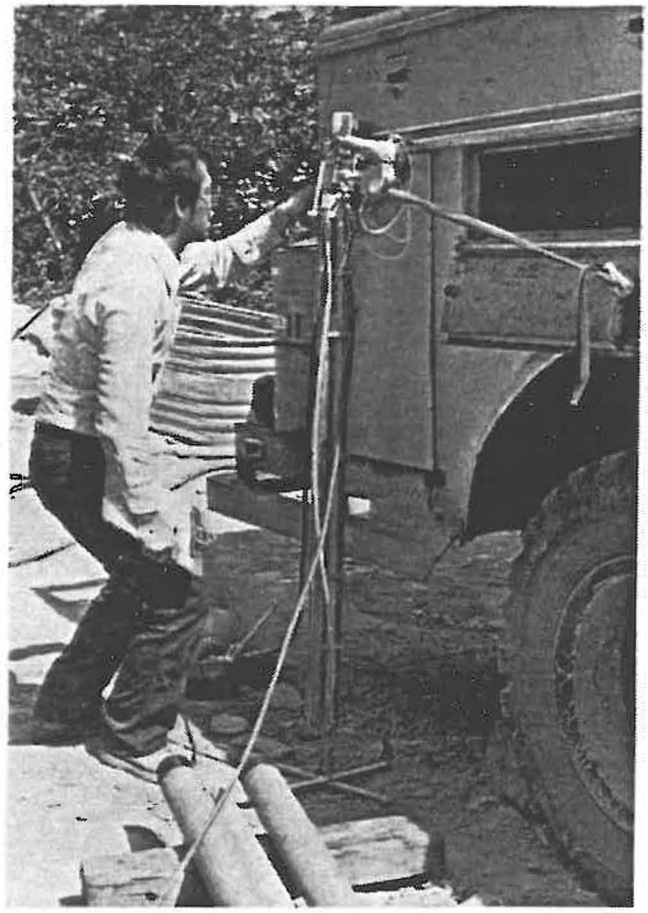


FIGURE A-1 BASIC PRINCIPLES OF THE PRESSUREMETER

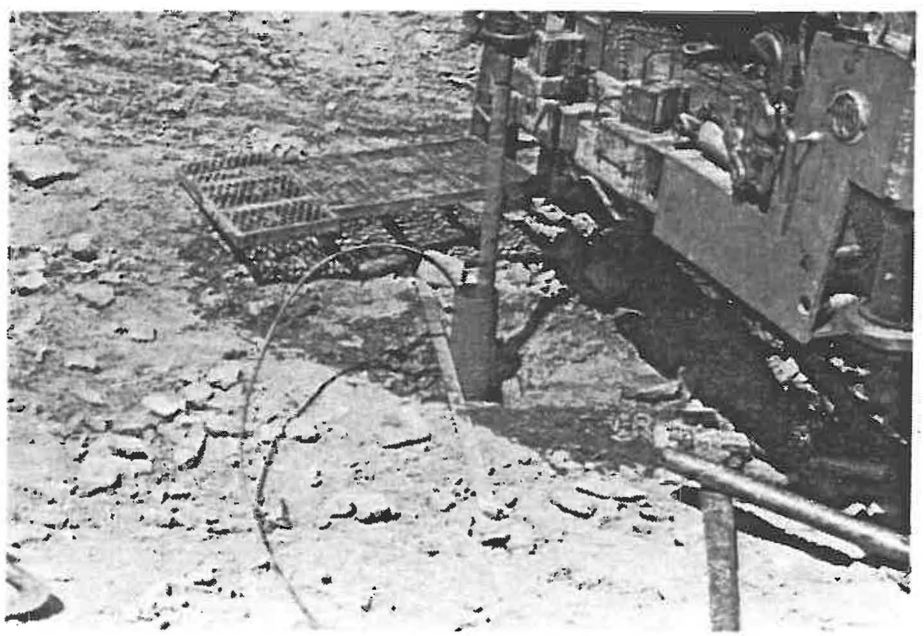
The pressuremeter tests were conducted with the Menard Pressuremeter Type 633-G using a standard NX diameter metallic sheath probe with approximately 145 feet of coaxial tubing. The NX probe was approximately 3 feet long with a measuring cell nine inches long. The pressuremeter test holes were drilled by the Colorado Department of Highways drilling crews and were advanced with three inch diameter tricon bit utilizing bentonite drilling mud as circulation fluid. All test holes were cased with steel casing to three feet above the test section. Figure A-3 shows the actual casing in place.

The standard pressuremeter test is carried out with 10 equal increments of pressure. On the 10th increment the limit pressure should be reached, that is to say the measuring cell should be doubled in size. At each pressure increment volume is recorded at 15, 30 and 60 seconds. Once the test is completed, a plot of pressure versus volume is plotted and the deformation modulus is determined. Figure A-4 is a typical plot of pressure versus volume and P_o, P_f, P_l and E_m are the parameters obtained from this curve.

The initial curved portion of the typical pressuremeter curve reflects the probe expansion as pressure is applied to the probe before the probe is in contact with walls of the Hole. P_o is the point on the low end of the pseudo-elastic portion of the curve. This is the point at which the borehole wall is theoretically restored to its original size and state of stress. P_o approximates the earth pressure at rest; however, it is generally not advisable to assume that P_o equals the at rest earth pressure. Disturbance to walls of the borehole can significantly affect the selection of P_o and there are generally not enough data points within the narrow pressure range on that portion of the curve to determine where the curve actually becomes linear. P_f is the pressure at the upper limit of the pseudo-elastic portion of the curve. For pressuremeter test, P_l or the limit pressure is defined

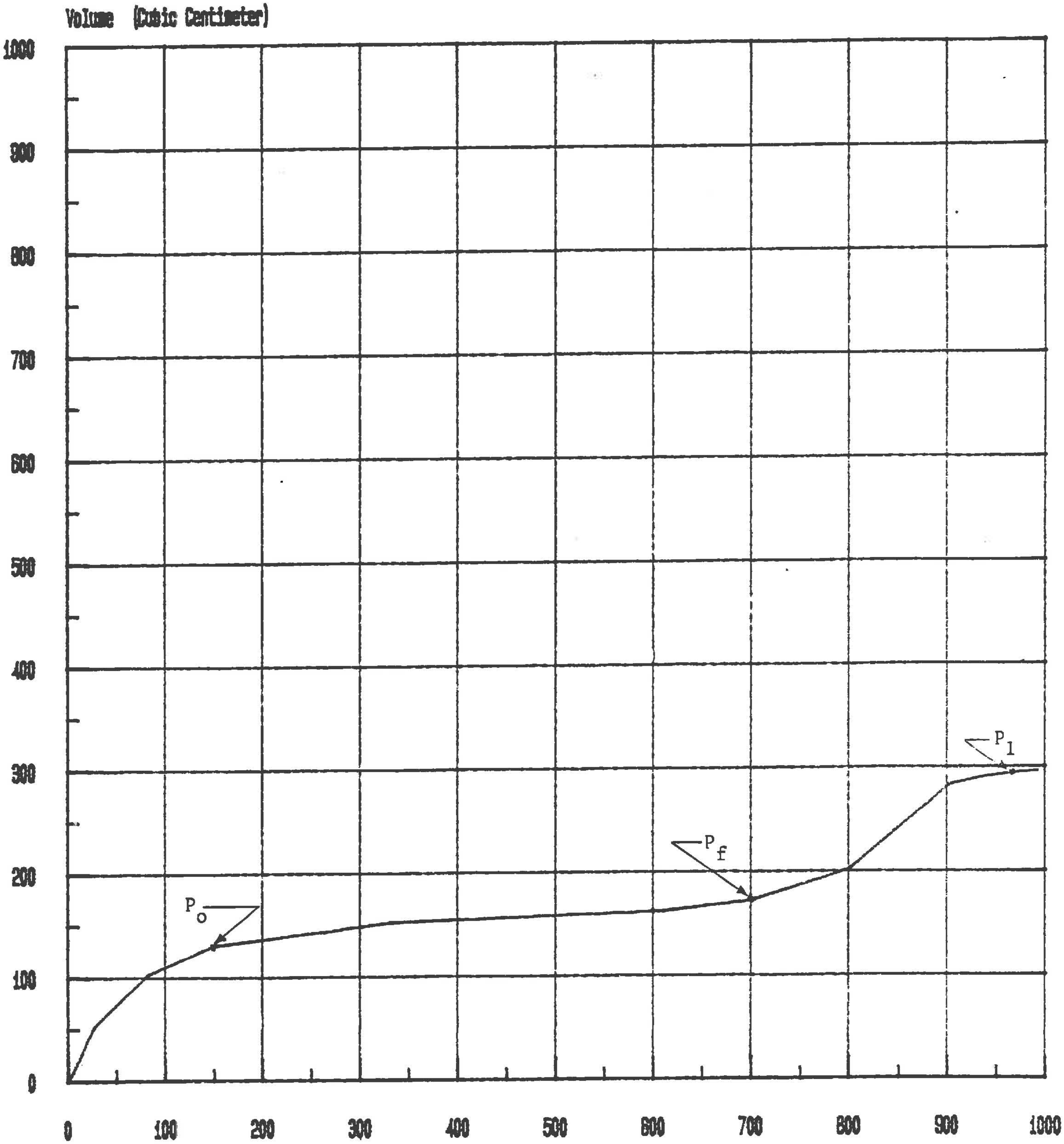


Photograph A-1 CONTROL UNIT OF THE PRESSUREMETER DURING OPERATION



Photograph A-2 CASINGS WERE USED TO KEEP THE HOLES OPEN DURING THE PRESSUREMETER TESTING

FIGURE A-2 PRESSUREMETER TEST RESULTS



pressure (Kilopaskal)

as being the pressure at which the initial volume of the cavity is doubled. This should be the approximate pressure at which the material's behavior is plastic and the hole expands with no additional application of pressure. E_m is the pressuremeter modulus and is directly proportional to the slope of the pseudo-elastic portion of the pressuremeter test curve.

Between P_o and P_f, the soil is said to behave as a more or less elastic material since the curve is approximately a straight line in this region. The equation for the radial expansion of a cylindrical cavity in an infinite elastic medium is (Lamb, 1952):

$$G = V.\ \Delta P/\Delta V$$

G = Shear Modulus

V = Volume of the cavity

P = Pressure in the cavity

By Convention V = Vm, where Vm is the midpoint of V_o and V_f. The quantity of V_m is calculated from the following formula:

Where $$V_m = V_c + (V_o + V_f)\ /2$$

V_m = Midpoint volume between V_o and V_f

V_c = Original volume of cavity with no pressure applied to walls.

V_o = Volume corresponding to P_o

V_f = Volume corresponding to P_f

In this case, the value of G is called G_m after Menard who first proposed that this procedure be used. Thus:

$$G_m = V_m \ \Delta P/\Delta V$$

to convert the shear modulus, G_m, to something roughly equivalent to Young's modulus the following relationship is used:

$$G_m = E_p / 2(1 + \upsilon)$$

E_p = Modulus of Deformation

υ = Poisson's Ratio

Menard, the inventor of the pressuremeter, assumed a constant value of 0.33 for the Poisson's ratio υ and called the resulting deformation modulus the Menard modulus E_m for a soil, where:

$$G_m = E_m / 2(1 + \upsilon)$$

$$E_m = 2\ (1+\upsilon)\ G_m$$

$$E_m = 2\ (1+\upsilon)\ V_m \ \Delta P/\ \Delta V$$

$$E_m = 2\ (1+\upsilon)\ [V_c + (V_o + V_f)/2]\ \Delta P/\Delta V$$

$$E_m = 2\ (1+\upsilon)\ [V_c + (V_o + V_f)/2]\ [(P_f - P_o)/(V_f - V_o)]$$

ΔP and ΔV are then corrected based on the calibration tests obtained in laboratory. The result is the following formula utilized to calculate the Menard (Deformation) modulus for various soils:

$$E_m = 2(1+\upsilon)\left(V_c + \frac{V_o+V_f}{2}\right)\left(\frac{\Delta P-\Delta Q}{\Delta V-\alpha\Delta P}\right)$$

υ = Poisson's Ratio = 0.33

ΔQ = System correction for pressures

α = System correction for volume, related to pressures

Therefore:

$$E_m = 2.66\left(V_c + \frac{V_o+V_f}{2}\right)\left(\frac{\Delta P-\Delta Q}{\Delta V-\alpha\Delta P}\right)$$

The value of the Modulus of Deformation is then used to calculate the settlements of the foundation material at specific locations according to the plans.

References

1. Warner, J., " Soil Modification to Reduce the Potential for Liquefaction," Proceedings, Second Joint Meeting, U.S./Japan Cooperative Research Program in Earthquake Engineering on Repair and Retrofit of Buildings and Lifelines, Tsukuba, Japan, May 1981.

2. Baker, W. H., MacPherson, H. H., Cording, E. J., "Compaction Grouting to Limit Ground Movements: Instrumented Case History Evaluation of the Bolton Hill Subway Tunnels, Baltimore, MD," Technical Report U.S. Department of Transportation, October 1980.

3. Graf, E. D., "Compaction Grouting Technique," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM5, Proc. Paper 6766, September 1969, pp. 1151-1158.

4. Mitchell, J. K., "In-Place Treatments of Foundation Soils," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM1, Proc. Paper 7035, January 1970, pp. 73-110

5. Brown, D. R., and Warner, J., "Compaction Grouting," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 99, No. SM8, Proc. Paper 9908, August 1973, pp. 589-601

6. Warner, J., and Brown, D.R., "Planning and Performing Compaction Grouting," Journal of the Geotechnical Engineering Division, ASCE Vol. 100, No. GT6, Proc. Paper 10606, June 1974, pp. 653-666

7. Lamb, B., "Sinking Building Gets Grout Injections to Strengthen Soil," Engineering News Record, Vol. 198, No. 4, January 27, 1977

8. Warner, J., "Compaction Grounting - A Significant Case History," Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT7, Proc. Paper 13897, July 1978, pp. 837-847