Report No. CDOH-DTP-R-84-8

DYNAMIC COMPACTION OF SANITARY LANDFILLS WITH HIGH WATER TABLES

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Final Report
February, 1985

Prepared in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

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Technical Report Documentation Page

1. Report No. CHOH-DTP-R-84-8	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Dynamic Compaction of Sanitary Landfills with High Water Tables		5. Report Date February 1986
		6. Performing Organization Code
7. Author(s) Majid Derakhshandeh		8. Performing Organization Report No. CHOH-DTP-R-84-8
9. Performing Organization Name and Address Colorado Department of Highways 4201 E. Arkansas Ave. Denver, CO. 80012		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. 1577A
12. Sponsoring Agency Name and Address Colorado Department of Highways 4201 E. Arkansas Avenue Denver, CO. 80012		13. Type of Report and Period Covered Final
		14. Sponsoring Agency Code

15. Supplementary Notes
Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. Abstract

Dynamic Compaction was selected to stabilize two test sections in highly compressible sanitary landfills with high ground water tables.

This method consisted of free fall dropping a 20 ton weight from 70 to 80 feet high on predetermined grid points. Both test sections were instrumented and monitored. The results indicated 2 to 4 hundred percent improvements in ground stiffness which exceeded the expectations.

This method was immediately implemented on a similar sanitary landfill along the future I-76 in northwest Denver. About 8.5 acres of surface area was treated and an additional site has been scheduled to be treated in the fall of 1984.

17. Key Words Stablization, Free Fall Drop, Grid Pattern, 20 Ton Weight, Deep Compaction, Cushion Material, Instrumentation	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	81	

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CHAPTER I

INTRODUCTION

Continuation of I-76 from I-25 to I-70 in northwest Denver had created growing concern among geotechnical engineers because of the need to construct highway embankments over soft foundation material. About one mile of roadway had to be constructed over five sanitary landfills with highly compressible material and high water table. Several alternatives including dynamic compaction (DC) were considered. A decision was made to determine the effectiveness of dynamic compaction on two sanitary landfill test sites with high water tables.

For some highways, the first and the most major decision is the choice between an elevated structure and embankment construction with or without foundation treatment. This, in turn, invloved a choice between methods having cost differentials which may be several million dollars per mile.

This report describes the results of dynamic compaction experiment which was performed in June 1983. Vara Construction Companies, Inc., was the prime contractor and Menard, Inc., the subcontractor, was to perform the DC of the two test sections.

CHAPTER II

LITERATURE REVIEW

2.1 Allowable Settlements and Various Treatment Methods

The NCHRP Report Number 29 dealing with the treatment of soft foundations for highway enbankments indicates that post-construction settlements during the economic life of a roadway as much as 1 foot are generally considered tolerable provided they (a) are reasonably uniform, (b) do not occur adjacent to a pile-supported structure, and (c) occur slowly over a long period of time.

Settlements up to 1 foot are considered tolerable even where rigid pavements are used, although in many areas flexible pavements are specified. Rigid pavements have undergone 1 foot of uniform settlement without distress or objectionable riding roughness. Where some doubt exists about the uniformity of post-construction settlements, flexible pavement is usually selected. This was also done in some states when predicted settlements exceeded six inches.

Various methods for treatment of soft foundations consist of (a) removal of soft foundation soils and replacement by suitable fill; (b) stabilization by dynamic compaction treatment; (c) preloading; (d) grouting; and (e) deep foundation.

For this project, removal and deep foundation methods were determined not to be cost-effective. The feasibility of the other methods were studied and Dynamic Compaction was chosen for treatment of two test sections in two sanitary landfills with high groundwater tables.

Dynamic Compaction was first introduced by Louis Menard in 1969 to compact a 26 foot deep hydraulic fill for a condominium development on the French Riviera. Dynamic Compaction is a method for improving the engineering properties of in-place soils at depth, both above and below groundwater level. Soil strength is increased and compressibility is decreased as a result of densification.

Dynamic Compaction consists of providing high energy impacts at the ground surface by dropping a heavy weight (10-200 ton) from a large distance (60-120 ft high) as shown in Figures 1, 2, and 3. The impacts produce shock waves that propagate to a great depth. The shocks cause compaction of any type of unsaturated material. In saturated granular soils, the shocks cause a partial liquefaction followed by rapid consolidation. A complete soil modification during a Dynamic Compaction treatment is shown in Figure 4. In this figure, the sudden increase in vertical stress, horizontal stress and pore water pressure corresponding to each drop of weight are presented.

The range of material that can be successfully treated is surprisingly broad. Compaction has been achieved above groundwater level in material ranging from rock-fill to clay, including organic soils, and building/domestic refuse. Below groundwater level, Dynamic Compaction is most effective in improving nonplastic granular soils.

Comprehensive field measurements are used to control the Dynamic Compaciton process so that the applied energy, the location, sequence and timing of the drops can be adjusted to achieve the desired results. Typical measurements involve level surveys to determine the induced settlement; excess pore water pressure measurements to determine the energy required to liquefy saturated soils and the time required for pore water pressure dissipation; and

FIGURE 1. 125 TON CRANE IS GENERALLY USED TO LIFT THE 20 TON WEIGHT.

FIGURE 2. 20 TON WEIGHT WAS USED FOR THIS EXPERIMENT.

FIGURE 3. 20 TON WEIGHT AT THE IMPACT POINT

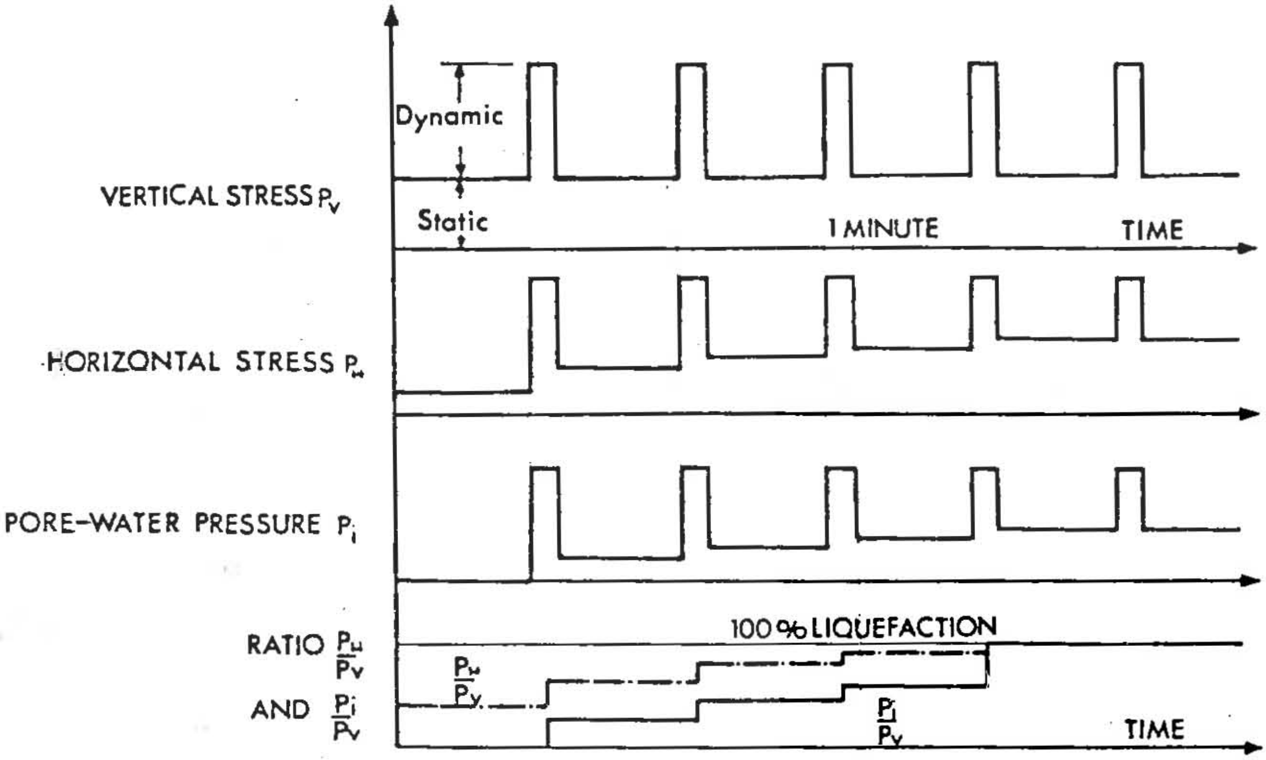


FIGURE 4 EFFECTS OF DYNAMIC COMPACTION ON ENGINEERING PROPERTIES OF SOILS.

in situ measurements to determine soil strength and compressibility.

The results of treatment by Dynamic Compaction are dramatic and immediate. Surface settlement is typically 3 to 7 percent or more of the thickness of the material being treated and is noticed immediately. Pore pressure increase is instantaneous, and dissipation occurs rapidly, often accompanied by a rising groundwater level or localized bubbling at the surface. Strength and compressibility of the material, as measured by in situ tests, are typically improved by a factor of 2 to 4.

This method was recently employed on two sanitary landfills in Denver, and the results of ground imporvements exceeded expectations.

2.2 Construction on Sanitary Landfills

Roads in urban areas must frequently be located on landfills and similar areas. In general, most of the landfills are utilized for construction without any major preconstruction treatment to the foundation material. The only requirement, in most cases, consists of using a heavy roller (50 ton) to compact the top soil prior to construction of the embankments. But, it has been proven that most of the embankments constructed on landfills without any foundation treatment experience prolonged differential settlements and require continuous maintenance work.

Post-construction settlements of sanitary landfills under embankments are difficult to predict and may be large. There is no available model to predict the behavior of trash under embankment loads. But, it is well documented that most landfills consist of large void volumes that needs to be treated and reduced prior to any construction. Therefore, stabilization methods such as Dynamic Compaction, Preloading, Grouting, or Excavation may be considered in order to increase the bearing capacity of the ground and reduce the post-construction settlements.

2.3 Application of Dynamic Compaction

Dynamic Compaction has been widely used in Europe for treatment of various foundation material for various structures. Even though North American applications have not reached levels comparable to those in Europe, the application in 1983 of the technique to large projects such as at the Kings Bay Military Base in Georgia and Highway I-65 in Jefferson County, Alabama, is notable. Also recent applications on various highway projects show the growing interest in utilizing Dynamic Compaction for treatment of soft foundation material. Table 1 show the history of Dynamic Compaction application in various countries for various projects. The geographical distribution of projects listed in Table 1 is presented in Figure 5.

TABLE 1

DYNAMIC CONSOLIDATION

PROJECTS REALIZED FOR

ROADWAYS, STREETS AND RELATED STRUCTURES

LOCATION AND NAME	SOURCE OF FOUNDATION SOILS	TYPE OF SOILS	YEAR	COMPACTION FOR	AREA m^2	REMARKS
ALVECHURCH HIGHWAY	HETEROGENEOUS FILL	DOMESTIC REFUSE	1975	HIGHWAY	28,000	
BOISSY ST-LEGER, RN-19	HETEROGENEOUS FILL	CLAY, SILT, CONST-WASTE, ETC	1973	HIGHWAY	4,000	BAKFILL-OLD GRAVEL PIT
BREMERHAVEN-CUXHAVEN, W-GERMANY, A-27	SELECTED FILL	SAND	1978	EXPRESSWAY	16,000	TEST: VERTICAL DRAINS & DYNAMIC CONSOLIDATION
OVERPASS ACCESS CHARLEVILLE-SEDAN HYWAY FRANCE	HETEROGENEOUS FILL	RANDOM MATERIALS	1971	HIGHWAY	17,000	
CHENEE-TILFF, BELGIUM, E-9	HETEROGENEOUS FILL	SILT	1972	EXPRESSWAY	1,000	TEST
EBEN-RENNWEG, AUSTRIA	NATURAL SOIL	MARSHY SOILS	1973	INTERCHANGE	59,000	
ETREMBIERES-LAVUACHE, FRANCE, A-42	HETEROGENEOUS FILL & NATURAL SOILS	SAND, GRAVEL, DOMESTIC REFUSE, CLAY	1980	HIGHWAY, ACCESS RAMP	7,000	
JOHANNESBURGH'S WESTERN BYPASS, SOUTH AFRICA	HETEROGENEOUS FILL & NATURAL SOILS	MINE SLIMES & MARSH	1979	EXPRESSWAY	95,000	
LEFAYET-LESHOUCHES, FRANCE, B-41	HETEROGENEOUS FILL	SAND, GRAVEL, BLOCKS	1977	HIGHWAY	6,600	
LEUCATE-BARCARES, NARBONNE, FRANCE, RN-9	HETEROGENEOUS FILL	CLAYEY MARL, CALCAREOUS BLOCKS	1972	HIGHWAY	13,000	COMPACTION OF 15m HIGHWAY EMBANKMENT SETTLING 10-15cm/YEAR
LISBON-PORTO HIGHWAY PORTUGAL	NATURAL SOILS	CLAY, SILTY SAND AND SOFT ALLUVIONS	1975 1976	EXPRESSWAY	80,000	
MAIZIERES-LEZ-METZ-INTERCHANGE, FRANCE	HETEROGENEOUS FILL	CLAYEY MATERIALS	1974	INTERCHANGE	110,000	BACKFILL OLD GRAVEL PIT: 15 TO 30% CLAY
PARIS-BRUXELLES, A-2 (COMBLES-HORDAIN SECTION)	HETEROGENEOUS FILL	CLAYEY SILT, CHALKY SOILS	1972	EXPRESSWAY	150,000	
TROARN-CLARBEC, FRANCE, A-13	HETEROGENEOUS FILL	SILT				
WEININGEN-URDOF, SWITZERLAND, RN-20	HETEROGENEOUS FILL	DOMESTIC REFUSE & RUBBISH	1980	EXPRESSWAY	14,000	OLD SANDPIT
VANNES, FRANCE, RN-165			1972	UNDERPASS	1,000	
NOYALO, FRANCE, RN-780			1973	HIGHWAY	2,000	
CHAMBERRY, FRANCE, CD-16A			1973	OVERPASS	2,000	
BEAUGENCY, FRANCE, A-10			1973	EXPRESSWAY	3,500	
MEULAN, FRANCE, CD-28			1975	ROAD	3,000	
MAUBRAY HIGHWAY, BELGIUM			1972	EXPRESSWAY	26,950	
WEVELGHEM HIGHWAY, BELGIUM			1973	EXPRESSWAY	10,000	
LISBON, PORTUGAL			1974	EXPRESSWAY	200,000	
SCHONBULL-BERN	NATURAL SOILS	PEAT	1976	EXPRESSWAY	80,000	
PLOCHINGEN INTERCHANGE WEST-GERMANY	HETEROGENEOUS FILL	DOMESTIC REFUSE	1977	INTERCHANGE	37,000	
NAGOYA, JAPAN			1978	EXPRESSWAY	50,000	DONE IN 2-25,000m^2 STAGES
GUILFORD, U.K., A-3			1979	EXPRESSWAY	9,000	
USTER, SWITZERLAND			1979	EXPRESSWAY	6,000	
U.K., M-25			1980	PERIPHERAL EXPRESSWAY	15,000	
ROUEN, FRANCE	HETEROGENEOUS FILL	GARBAGE CONSTRUCTION DEBRIS	1972	STREET-INDUSTRIAL PK	12,000	
ST-NAZAIRE, FRANCE	HETEROGENEOUS FILL	SAND & GARBAGE	1973	ACCESS ROAD	15,000	
MIMET, FRANCE			1975	ROAD EMBANKMENT	3,000	
ASNIERES, FRANCE	SELECTED FILL-NAT. SOIL	SAND	1975	ACCESS RAMP TO BRIDGE	6,000	
MONACO			1975	CITY STREETS	20,600	
LE VERDON, FRANCE	HYDRAULIC FILL	SANDFILL - SOFT CLAYEY SILT	1975	INDUSTRIAL ROAD	110,000	
HULL, CANADA	HETEROGENEOUS FILL	ROCKFILL, CONSTRUCTION DEBRIS & GARBAGE	1974	CITY BOULEVARD	9,000	
VERDUN, CANADA	HETEROGENEOUS FILL	CONSTRUCTION DEBRIS	1980	CITY STS. & SERVICES	17,250	
SPRINGDALE, ARKANSAS	HETEROGENEOUS FILL	DOMESTIC REFUSE	1982	U.S. ROUTE 71	14,200	REPORTED SAVINGS OF $3MM OVER EXCAVATION & REPL'NT
ALGODONES, NEW MEXICO	NATURAL SOILS	COLLAPSING SOIL	1980	SANTA FE TO ALBUQUERQUE RTE-25		THICKNESS COLLAPSIBLE SOIL 6M; SOIL DENSITY INCREASED FROM 1120 TO 1585KG/M^3
MONACO			1979	STREETS		
MONTESSON, FRANCE			1979	STREETS	4,000	
KINGS BAY, GEORGIA	NATURAL SOILS	SAND, SOME SILT	1983	ROAD ON NAVAL BASE	32,400	MIN. 60% RELATIVE DENSITY REQUIRED TO 12M
RIVIERE-AU-RENARD, CANADA	HETEROGENEOUS FILL	SCHIST, SAND & GRAVEL FILL	1977	RDS & SERV.-IND. PARK	6,500	FILL DUMPED INTO THE SEA
JEFFERSON COUNTY, ALABAMA	HETEROGENEOUS FILL	COARSE GRANULAR MINE SPOILS	1983	I-65 INTERSTATE	55,000	FULL SCALE FIELD TEST PRECEDED IN 1982

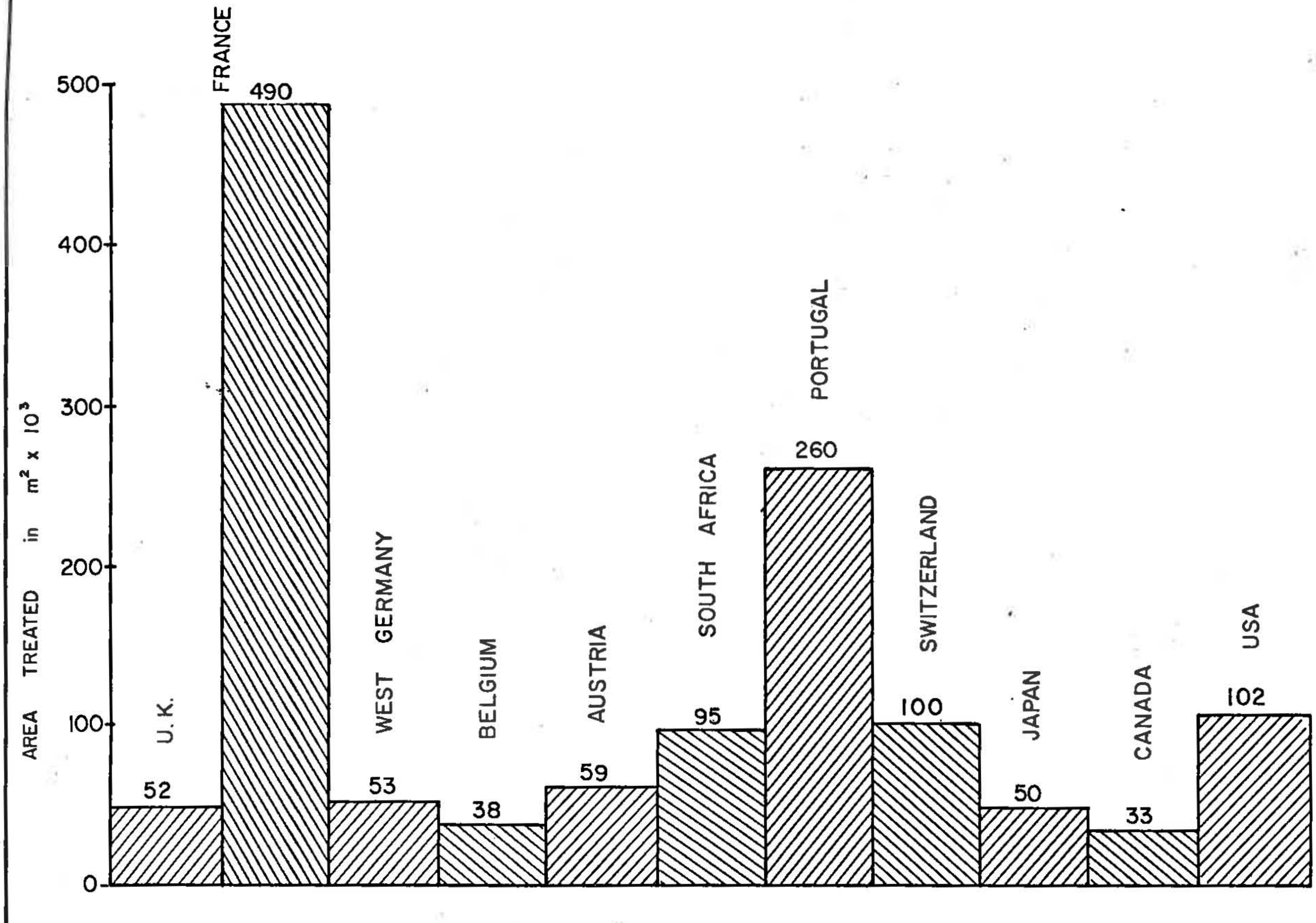


FIG.5 - HISTOGRAM SHOWING THE GEOGRAPHICAL DISTRIBUTION OF PROJECTS LISTED IN TABLE 1.

Chapter III

SITE INVESTIGATION

3.1 Site Location and Description

Two sites were selected as part of this study for Dynamic Compaction treatment. Site A with test section no. 1 was located on the south side of 62nd Street between Federal Boulevard and Pecos Street. Site B with test section No. 2 was located north of 64th Street between Pecos and Broadway. Figures 6 and 7 show the location of each sanitary landfill with shaded areas designated as the test sections.

Site A was wet and marshy with 3 to 4 feet of silty clay overburden material. The groundwater table was about three feet higher than the ground surface at the test section. For subsurface exploration as well as the safety of the crane, five feet of cushion material was placed on the designated test section.

Site B was dry and suitable for construction traffic. The specialty contractor inspected the site and suggested placing 2 to 3 feet of cushion material to ensure the safety of the crane during the Dynamic Compaction operation. The overburden material in this site consisted of 4 to 6 feet of unevenly distributed fly ash. The water table was 5 to 6 feet below the ground surface. Sites A and B are shown in Figures 8 and 9.

The test sections were 100 feet x 100 feet. They were selected such that they would represent the worst conditions in landfills with high water tables.

3.2 Subsurface Exploration

The subsurface exploration for both test sectons consisted of drilling, collecting samples and performing standard penetration tests to

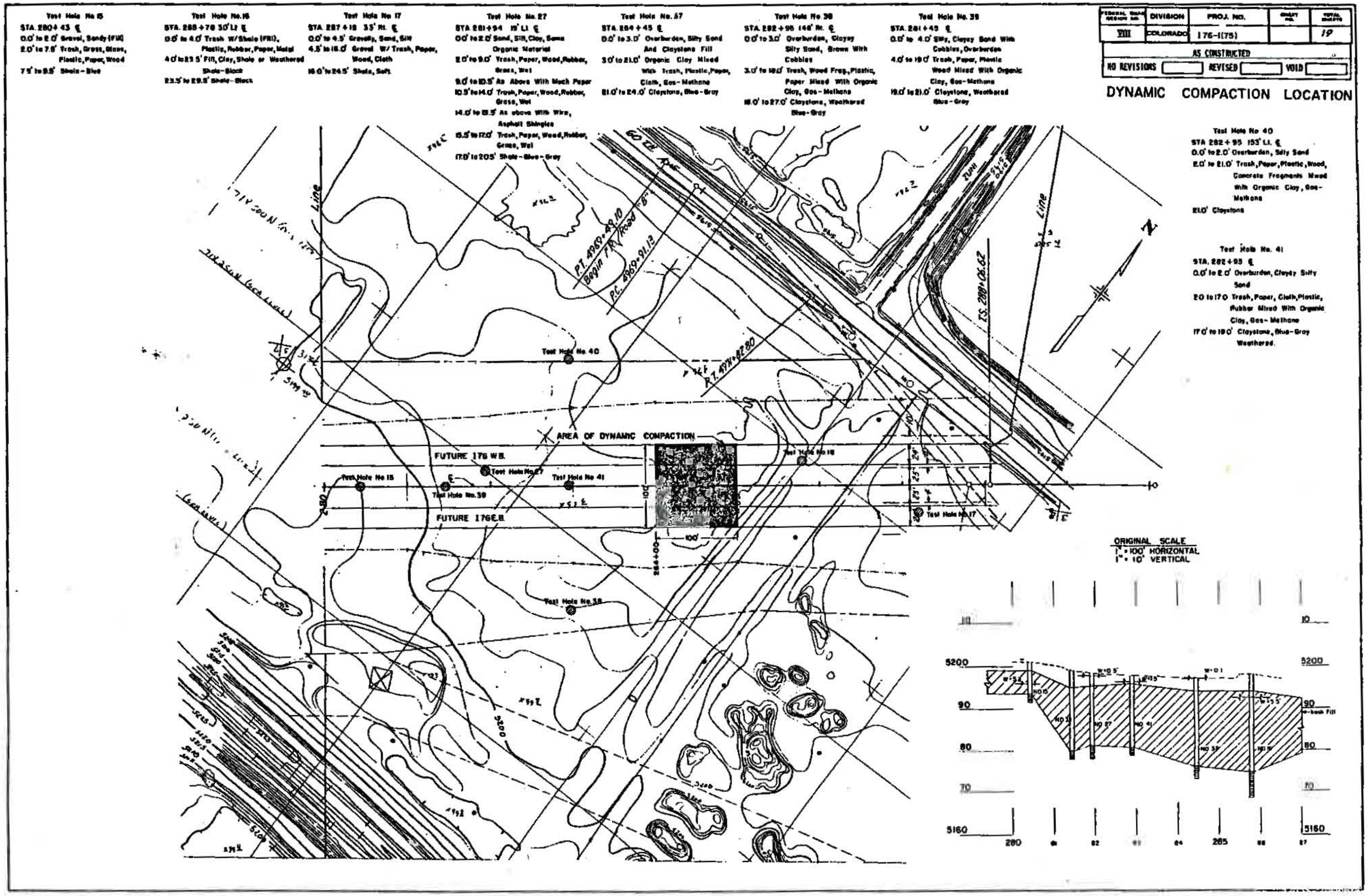


FIGURE 6 LOCATION OF SITE A WITH TEST SECTION NO.1

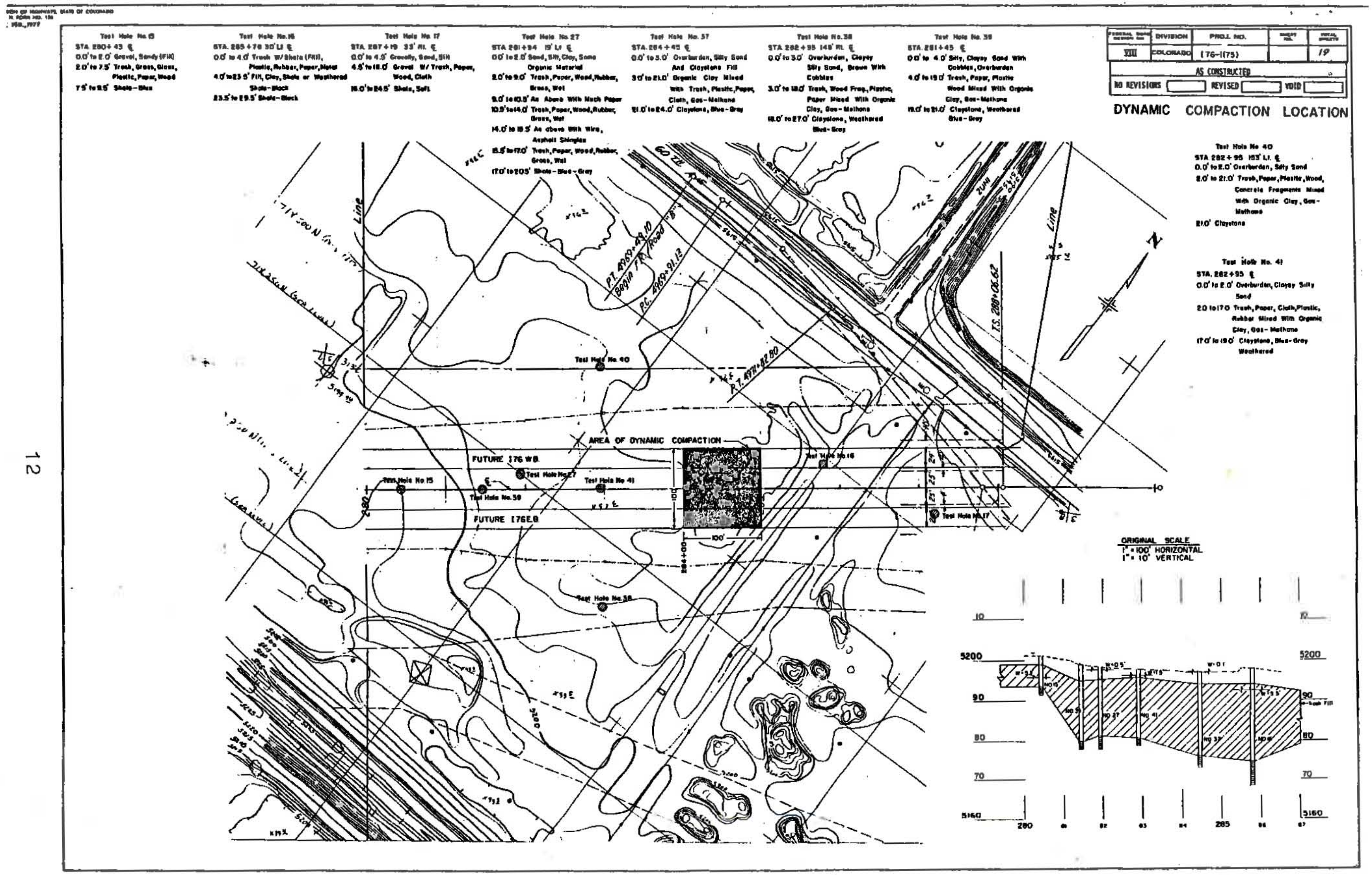


FIGURE 6 LOCATION OF SITE A WITH TEST SECTION NO.1

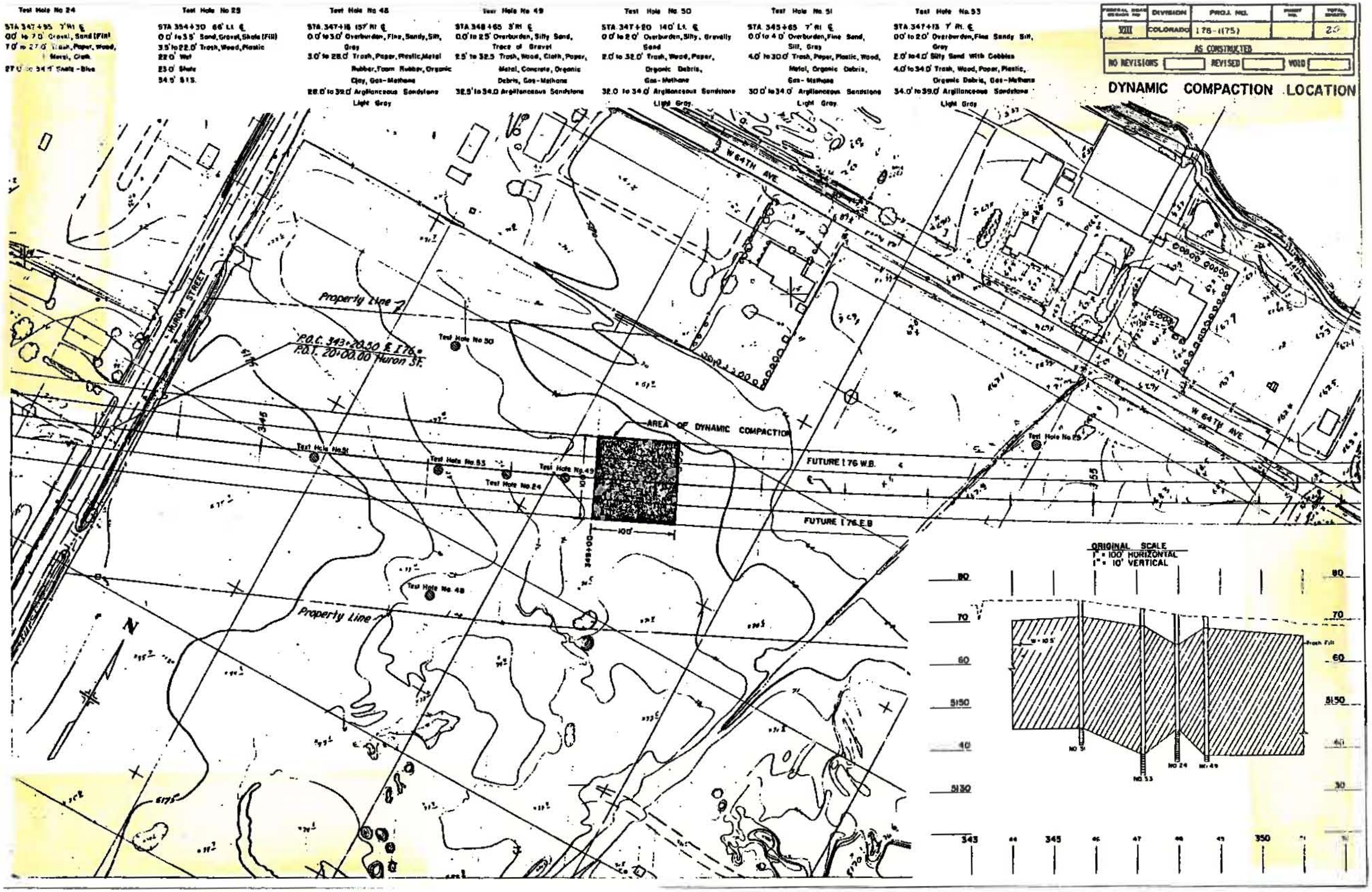


FIGURE 7 LOCATION OF SITE B WITH TEST SECTION NO. 2

FIGURE 8. SITE A WAS MARSHY AND WET WITH WATER TABLE AT 3 FEET ABOVE THE GROUND SURFACE

FIGURE 9. SITE B WAS DRY AND SOLID WITH NO APPARENT WATER ON THE SURFACE.WATER TABLE WAS AT 5 TO 6 FEET BELOW THE GROUND SURFACE.

determine the stiffness of the subsurface ground material. A total of 5 holes in test section number 1 and 3 holes in test section number 2 were drilled to determine the soil profiles. Tables 2 and 3 describe the nature of material obtained for each hole.

In general, the refuse material consisted of household trash, car bodies, tires, cloth, plastic, wood, concrete debris, etc. A study performed by the University of Colorado indicated that it is not feasible to determine the engineering properties of trash using the geotechnical laboratory equipment. This is mostly due to nonuniformity of the refuse material and the difference in material properties of trash compared to natural soils.

TABLE 2
PRELIMINARY BORE HOLE DATA
FOR TEST SECTION NO. 1

Depth	H #1
0.0' - 2.0'	Gravel, sandy (fill)
2.0' - 17.5'	Trash, metal, glass, plastic, paper, wood
17.5' - 19.5'	Shale (blue)
	H #2
0.0' - 4/0'	Silty Clayey sand with cobbles
4.0' - 19.0'	Trash, paper, plastic, wood, mixed wih organic clay
19.0' - 21.0'	Claystone weathered (blue gray)
	H #3
0.0' - 2.0'	Sand silt, clay, some organic material
2.0' - 4.0'	Trash, paper, wood, rubber, grass cuttings, wet
4.0' - 5.5'	Trash, paper, wood, rubber
5.5' - 10.5'	Trash, paper, wood
10.5' - 15.5	Wire, asphalt shingles
15.5' - 17.0'	Trash, paper, wood
17.0' - 20.5'	Shale (bluish gray)
	H #4
0.0' - 2.0'	Clayey silty sand
2.0' - 18.0'	Trash, paper, cloth, plastic, rubber, mixed with organic clay
18.0' - 19.0'	Claystone (blue-gray) weathered
	H #5
0.0' - 3.0'	Silty sand and claystone fill
3.0' - 21.0'	Organic clay mixed with trash, plastic, paper, cloth
21.0' - 24.0'	Claystone (blue-gray)

TABLE 3

PRELIMINARY BORE HOLE DATA
FOR TEST SECTION NO. 2

Depth	H #1
0.0' - 5.0'	Overburden Flyash
5.0' - 22.0'	Trash, paper, plastic, wood, metal
22.0' - 23.0'	Claystone, blue gray weathered
	H #2
0.0' - 4.0'	Overburden, Flyash
4.0' - 22.0'	Trash, wood, cloth, paper, plastic, box spring
22.0' - 24.0	Claystone (blue gray) weathered
	H #3
0.0' - 5.0'	Flyash
5.0' - 22.0'	Trash, rubber, paper, plastic, concrete debris
22.0' - 24.0'	Claystone (blue gray) weathered

CHAPTER IV

TESTING PROCEDURE

4.1 Scope of the Work

Dynamic Compaction was planned to be used to stabilize two test sections in sanitary landfills with high groundwater tables. A 125-ton crane was used to drop a 20-ton weight from 70 to 80 ft high. The entire operation was performed on top of a rockfill platform that was placed prior to the beginning of the experiment.

An extensive monitoring program was planned to investigate the subsurface ground material behavior before, during, and after Dynamic Compaction.

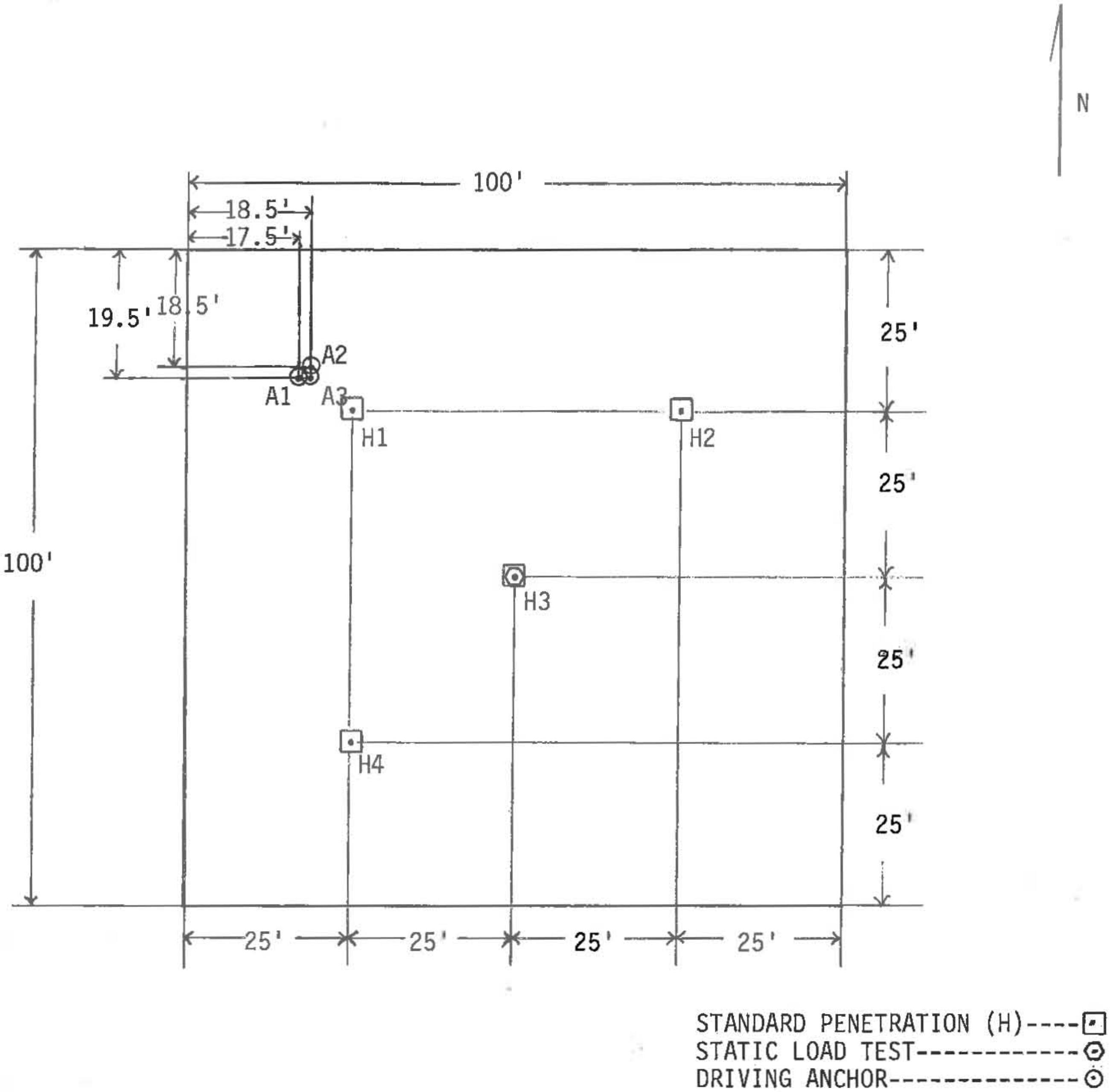
4.2 Monitoring Program

The monitoring program consisted of in situ testing as well as instrumentation to detect changes in densities and ground settlements (reduction of voids). The following methods and instruments were used to monitor the Dynamic Compaction operation:

1. Standard Penetration Test
2. Static Load Test
3. Seismic Test
4. Piezometer
5. Driving Anchor

4.2.1 Standard Penetration Test

Standard penetration test was used to measure the change in density of the sanitary landfill material caused by Dynamic Compaction. A total of nine holes were tested for this purpose. The distribution of holes are shown in Figures 10 and 11. Prior to beginning the experiment, the location of each



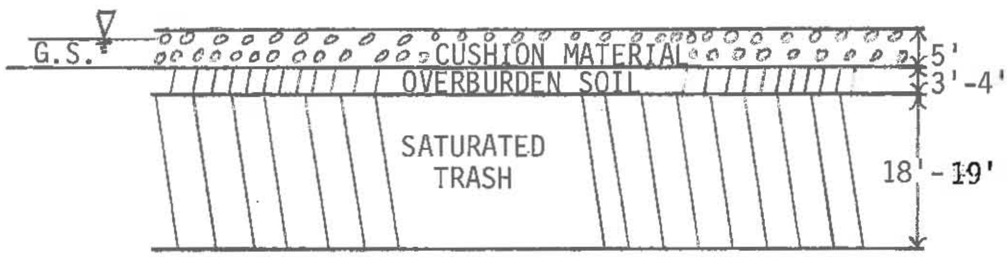
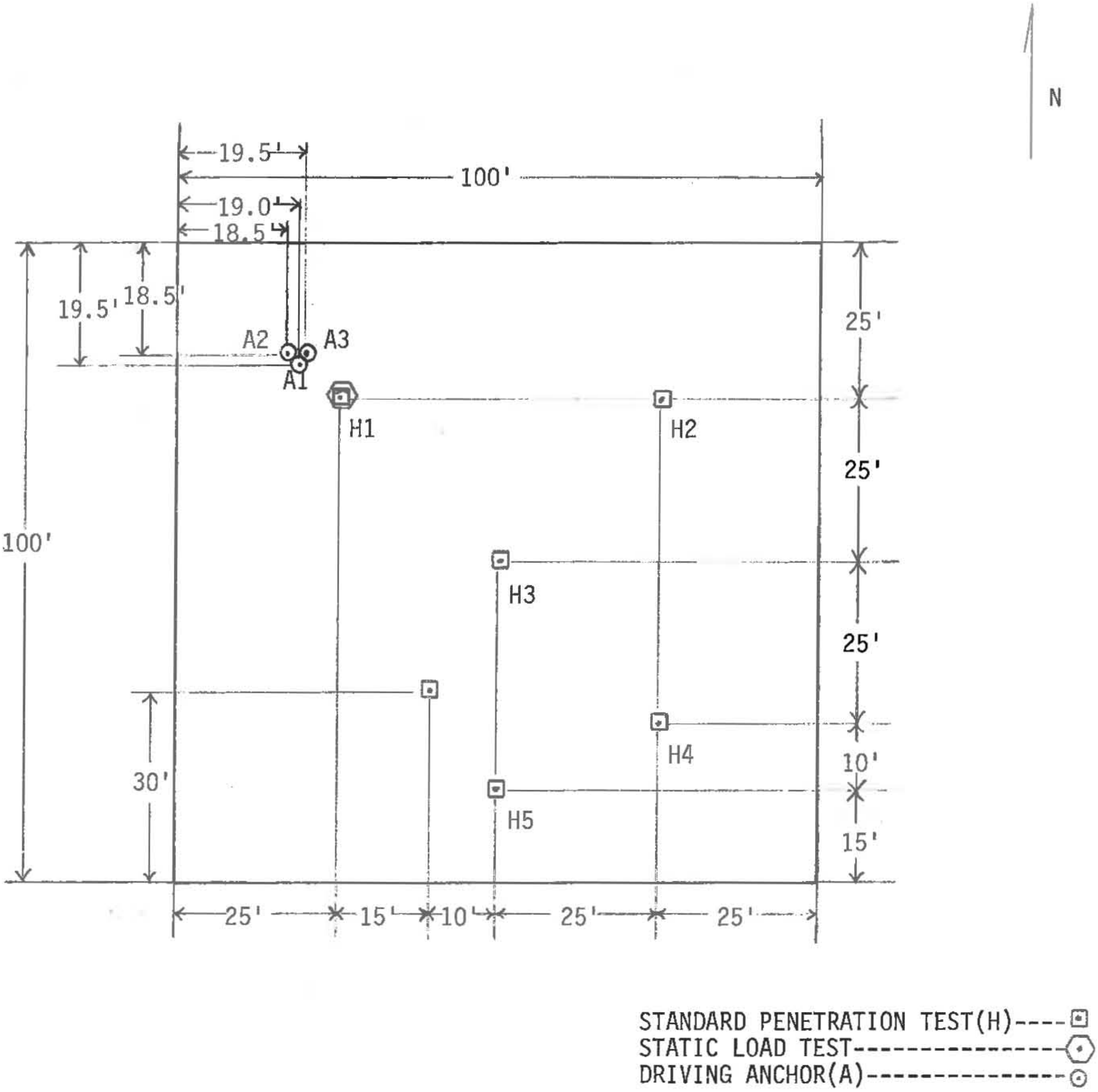


FIGURE 10 1st TEST SECTION ON SITE A



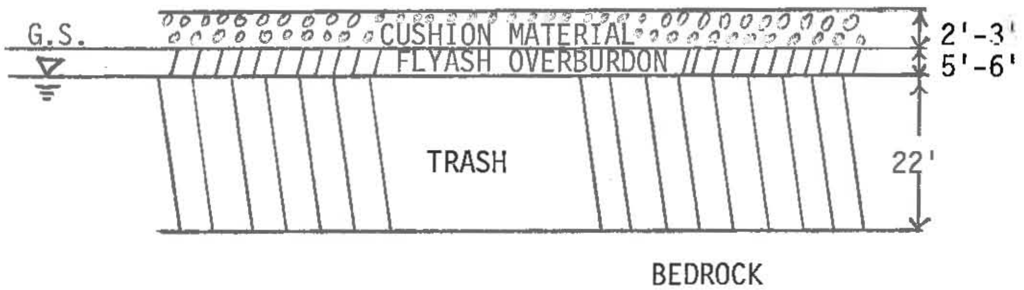


FIGURE 11 2nd TEST SECTION ON SITE B

point was layed out in each test section. Then, standard penetration tests were performed for each hole and the results were recorded. After the treatment, the same procedures were repeated and the results were compared.

4.2.2 Static Load Test

A Static load test was performed before and after Dynamic Compaction to determine the change in the rate of settlement. This test consisted of placing a conical static load 20 feet high and 40 feet in diameter at the bottom as shown in Figure 12. The rate of settlement was measured by monitoring the elevation of the extension tube attached to a settlement plate 3 ft x 3 ft at the base placed on the ground surface at the center of the static load. A conveyor was used to build the static load around the settlement plate. This test, if properly used, can determine the reduction in the magnitude of settlement due to increase in stiffness of the subsurface material.

Duration of the static load test varies with the nature of the compressible subsurface material. Based on the experience gained by the Arkansas Highway Department, all static load tests were monitored for a seven day period to obtain sufficient information on the magnitude of settlements before and after the treatment.

4.2.3 Seismic Testing

Shear wave velocities and amplitudes were measured and recorded. This was done for various distances from the impact point in all directions to obtain a good distribution of wave velocities and amplitudes. The shock waves were evaluated to determine their effects on the adjacent structures.

A seismograph was used to measure the shear wave velocities. The equipment is shown in Figure 13.

FIGURE 12. STATIC LOAD TEST. A CONICAL DEAD LOAD 20 FEET HIGH AND 40 FEET IN DIAMETER

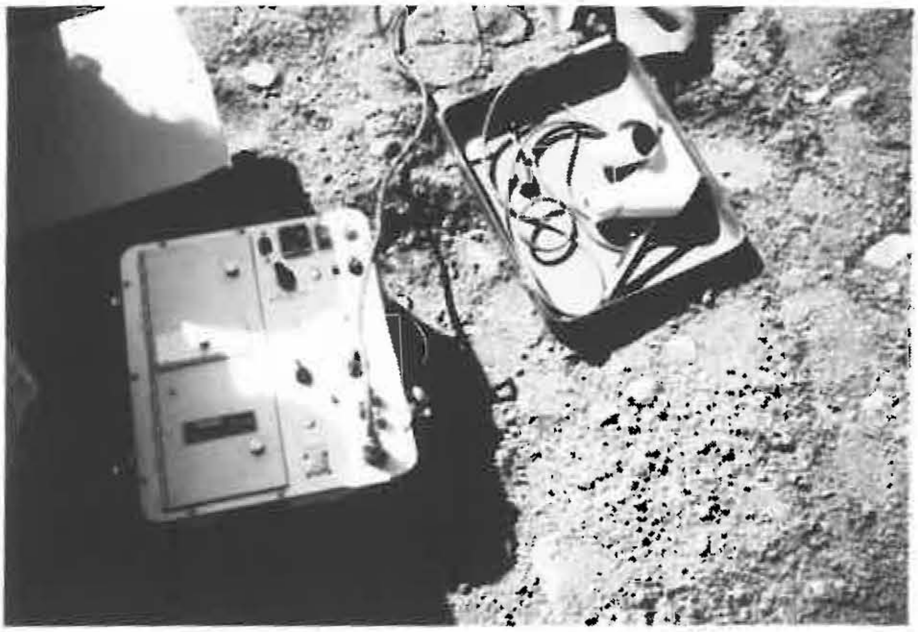


FIGURE 13. SIESMOGRAPH WAS USED TO MEASURE THE SHEAR WAVE AMPLITUDES PRODUCED DURING THE DYNAMIC COMPCTION PROCESS

4.2.4 Measurement of Pore Water Pressure

Dynamic Compaction induces tremendous amounts of energy to the subsurface material. As a result, the material under the ground surface become denser, and the total volume of voids reduces. This mechanism causes the water between the voids to be squeezed and its pressure to increase. The decrease in water pressure is directly related to settlement of the subsurface material. Piezometers are generally used to measure water pressures at different elevations below the ground surface. A total of eight piezometers were installed at different elevations below the ground surface. The distribution of piezometers for each test section are shown in Figures 14 and 15. A pneumatic piezometer and the corresponding read-out device are presented in Figures 16 and 17.

4.2.5 Driving Anchor

Driving anchors were used to measure the settlement of the subsurface material at various elevations. These anchors are hydraulically pushed into the ground and fixed at predetermined elevations by means of tapping the inner tube at the ground surface. The inner tube is connected to the anchors and the tapping action causes the anchors to expand and penetrate into the underground material. Figures 10 and 11 show the locations of these anchors in both sites. Figures 18 and 19 show a close-up of a driving anchor and final view of three anchors installed respectively.

4.3 Dynamic Compaction Testing Procedure

Dymatic Compaction operation consists of preliminary investigations and the actual treatment. Generally, a specialty contractor is hired to perform the entire operation in order to minimize errors and maximize the effects of the treatment.

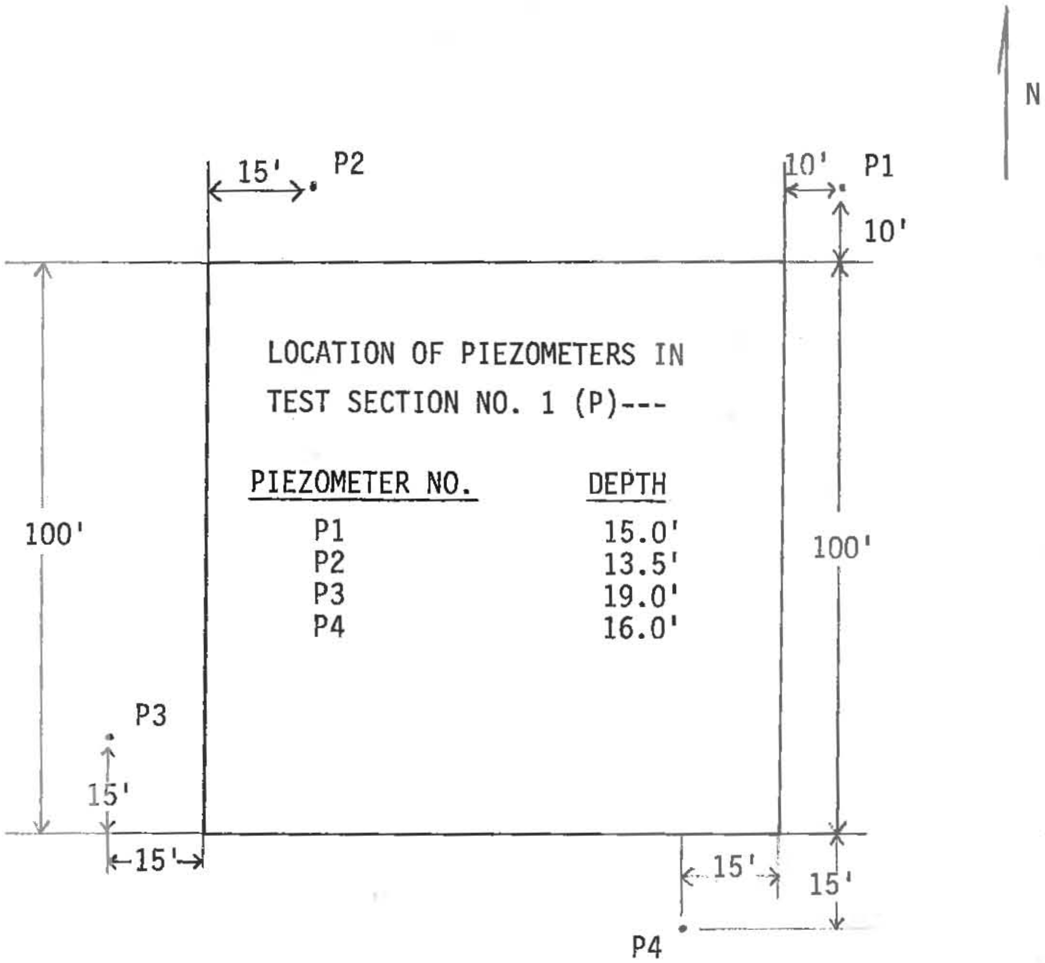


FIGURE 14 TEST SECTION NO. 1

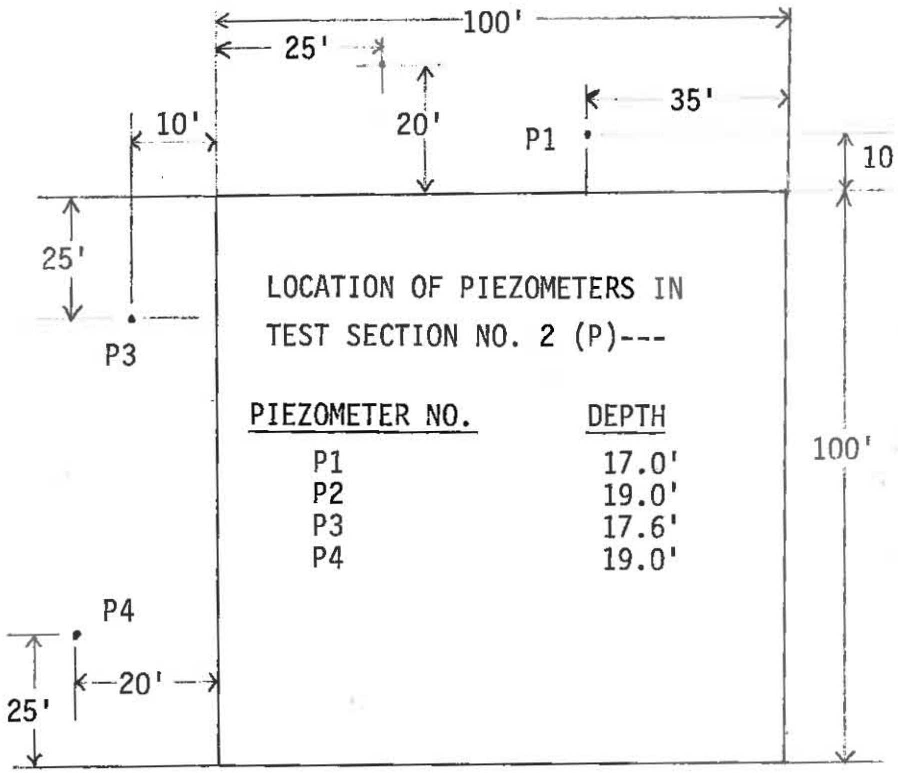


FIGURE 15 TEST SECTION NO. 2

FIGURE 16. PNEUMATIC PIEZOMETER MANUFACTURED BY THE SINCO COMPANY

FIGURE 17. READ-OUT DEVICE FOR THE PNEUMATIC PIEZOMETER

FIGURE 18. CLOSE UP VIEW OF A DRIVING ANCHOR

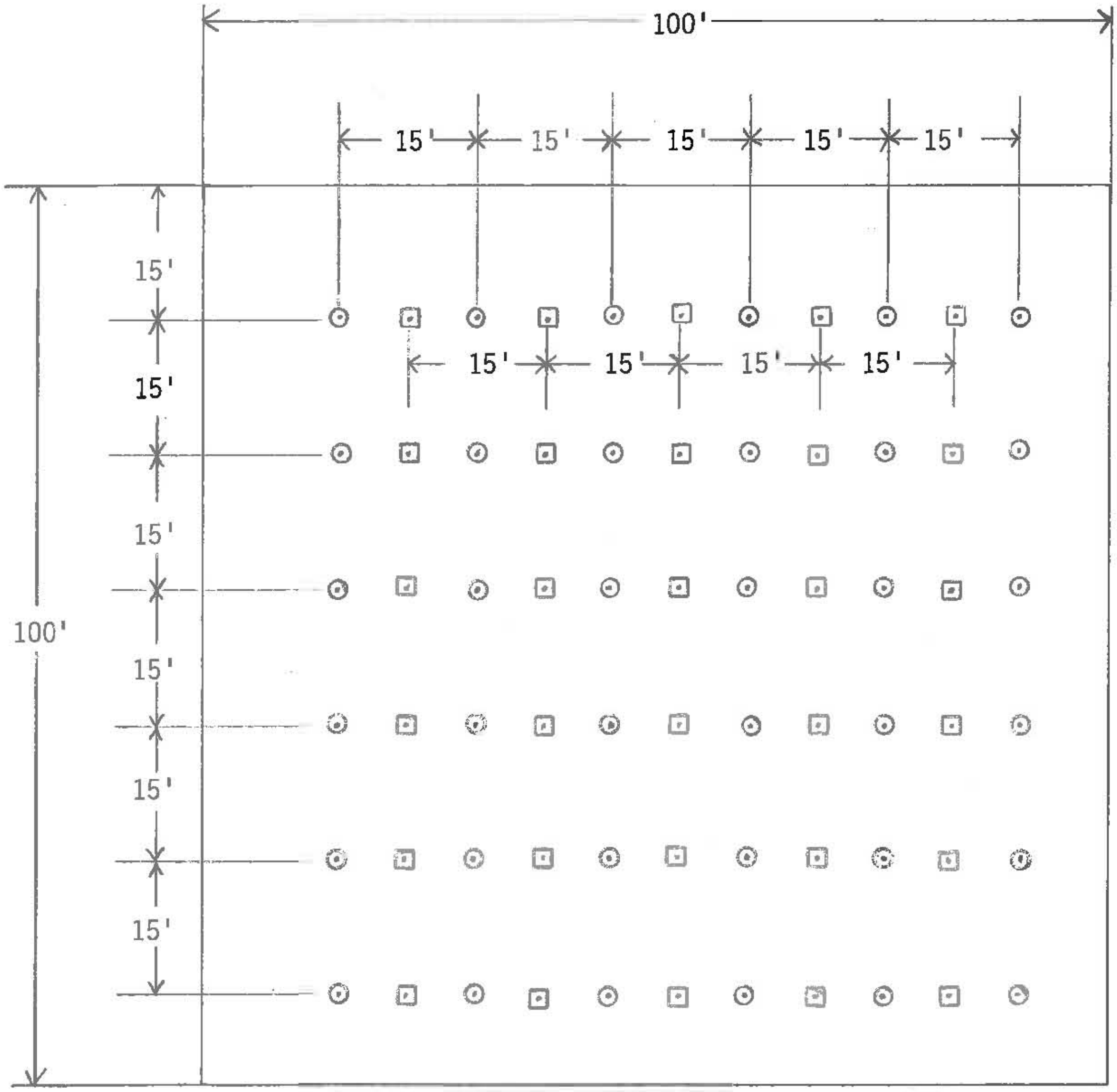
FIGURE 19. FINAL VIEW OF THREE ANCHORS INSTALLED IN TEST SECTION NO. 1. THE INNER TUBES WERE MONITORED FOR SUBSURFACE GROUND MOVEMENTS

The prime contractor selected Menard, Inc. to be the specialty contractor for all preliminary investigations. The Research Section from the Colorado Department of Highways was to coordinate with the specialty contractor to determine the optimum grid spacings, effective number of drops, various phases of compaction and instrumentation program.

Two test sections were provided on sanitary landfill areas. The project engineer from Menard, Inc. inspected the sites and recommended 15 foot grid point spacing for both test sections as shown in Figure 20. This number was selected based on contractor's years of experience and it is usually obtained on trail and error basis. The grid point spacing plays an important role during a Dynamic Compaction operation. The energy below the impact is dissipated in a conelike shape. If the grid spacing is too small, these cones will overlap high in the soil profile; therefore compacting first these layers, obstructing the energy applied during the subsequent phase and preventing it from reaching the deeper layers. The design of the first phase grid spacing is crucial for two important reasons: (1) To obtain optimum improvement, and (2) to minimize the cost of operation.

The field control of Dynamic Compaction treatment was accomplished by penetration and heave tests. Prior to each phase of the treatment, at least one penetration or heave test was performed to determine the effective number of drops for each phase of the treatment.

Penetration test was used to measure the depth of a particular crater at different levels of energy applied at the same impact location. The 20 ton weight was dropped from 60 to 80 feet height, and at two drop intervals the depth of craters were measure. The collected data was then plotted to obtain the relationship between the prenetration depth and the number of drops. From



LOCATION OF PRIMARY GRID POINTS--------⊙
LOCATION OF INTERMEDIATE POINTS--------⊡

FIGURE 20 GRID PATTERN DESIGNED FOR BOTH TEST SECTIONS this curve, the effective number of drops was determined and applied for the next phase of Dynamic Compaction treatment.

Heave test was a more time consuming but precise way of determining the effective number of drops. This test was used to measure the volume and heave around a particular crater. At two drop intervals, the diameter, depth and heave around the crater were measured using surveying equipment. The heave was obtained by surveying the ground surface elevations around the crater in four different directions, at 7, 10, 13 and 16 feet away from the center of the impact. The heave volume was then subtracted from the crater's volume to obtain the net volume at a particular energy level. The final step consisted of plotting the graph of net volume versus number of drops to obtain the effective number of drops.

The heave or penetration tests are strongly recommended to evaluate the effective number of drops prior to each phase of Dynamic Compaction treatment. If an arbitrary number of drops is chosen it could be either excessive or not enough. This, in turn may cause waste of time and money or lack of proper compaction.. Example of field penetration and heave tests are given in Appendix A.

Prior to beginning each phase of compaction test, prints were established and monitored to determine the appropirate optimum number of drops. Compaction of the entire area was then performed in various phases as described below.

Site A with test section number one, was compacted in three phases. Phase I was performed on 36 points (6x6), on a square gride pattern. The grid spacing was fixed at 15 ft. This phase was subdivided into two Sub-phases: (1) Phase I-1 where 10 drops per point were used, and (2) Phase I-2 where additional 10 drops per point were used on the same grid points. Phase II was performed on intermediate grid points located in between 4 points of phase I. A total of 15 drops per point was applied. Phase III or ironing phase consisted of dropping the pounder twice on a 5 ft. square grid.

On site B the grid spacings were the same as for site A. Phase I was divided into two Sub-phases (Phase I-1 and Phase I-2) of 10 drops per each primary grid point. Phase II was also subdivided into two sub-phases (Phase II-1 and Phase II-2) of 10 drops on each secondary (Intermediate) Grid point. Phase III or ironing phase consisted of two drops per point on a 5 ft. square grid.

Figure 21 through 26 illustrate the progress of Dynamic Compcation treatment.

FIGURE 21. CRANE LIFTING THE 20 TON WEIGHT

FIGURE 22. WEIGHT AT IMPACT POINT

FIGURE 23. CRATERS DURING THE PRIMARY PHASE. THE CRATERS WERE BACKFILLED AFTER COMPLETION OF EACH PHASE

FIGURE 24. CLOSE UP VIEW OF CRATERS AT THE END OF PRIMARY PHASE

FIGURE 25. CRATERS WERE BACKFILLED WITH GRANULAR MATERIAL

FIGURE 26. VIEW OF TEST SECTION NO. 1 AT THE END OF THE IRONING PHASE

CHAPTER V

RESULTS

The results of Dynamic Compaction are dramatic and immediate. The diameter of the average craters on the primary phase ranged from 10 to 12 ft, and their depths ranged from 6 to 8 ft. This was observed for both test sections with little variation throughout the entire operation. These dimensions were smaller in magnitude at each new phase of the Dynamic Compaction. The average diameter of the craters at the end of Phase III was 8 to 9 ft with maximum depth of 4 to 5 ft. Figures 27 and 28 show the approximate dimensions of a crater during the primary phase.

It was hoped to detect the behavior of the subsurface ground material by monitoring changes in pore water pressures. The pneumatic piezometers when confined in a sealed environment can detect any changes in the pore water pressures. During the compaction treatment, all piezometers were monitored during and after each phase of the operation. The results were poor and disappointing. All piezometers showed no change in pore water pressures. This observation is believed to be correct because all piezometers were embedded into the trash with large voids. Therefore, the excess pore water pressures were dissipated as soon as they were induced. This was also verified visually in site A where the groundwater table was 3 to 4 ft higher than the original ground. During the compaction, extensive bubbling was occuring within 10 to 15 ft. from the edges of the platform. This was a dramatic form of methane gas and pore water pressure dissipation. Therefore, it is highly recommended to avoid using piezometers in sanitary landfills. This also proved that waiting period were not needed between phases of

FIGURE 27. DIAMETERS OF CRATERS WERE MEASURED TO BE 10 TO 12 FEET AT THE END OF THE PRIMARY PHASE

FIGURE 28. SOME CRATERS WERE AS DEEP AS 8 FEET AT THE END OF THE PRIMARY PHASE

compaction for dissipation of the excess pore water pressures. Figures 29 and 30 show the actual water bubbling during the compaction.

The results of penetration and heave tests are presented in Figures 31 through 42. On Site A, Phase I (one) consisted of two sub-phases, I-1 and I-2, with 10 drops on each grid point. The effective number of drops was decided to be 10 by the specialty contractor based on the penetration tests performed prior to begining of each sub-phase. The results are presented in Figures 31 and 32. Then prior to Phase II, a set of penetration and heave tests were conducted to determine the optimum number of drops for this phase of compaction, the results are presented in Figures 33 and 34. After 10 drops, a sudden acceleration of settlement was observed. The specialty contractor decided to use 15 drops instead of 10 drops to ensure proper compaction for this phase of the treatment.

On site B, the overburden consisted of 5 to 7 feet of thick flyash. Therefore, the specialty contractor decided to conduct both penetration and heave tests more consistantly prior to each phase of compaction to increase the accuracy of his decisions on the optimum number of drops for each phase of compaction. Phase I (one) consisted of two sub-phases, I-1 and I-2. One set of penetration and heave tests were conducted prior to sub-phase I-1, and two sets of tests were performed prior to sub-phase I-2. The results are presented in Figures 35 through 40. The results were similar to those of cohesive soils (large volumes at low energy impacts). It was difficult to make a reasonable decision for the optimum number of drops for sub-phases I-1 and I-2. Finally, the specialty contractor decided to use 10 drops per grid point for each sub-phase of Phase I. The contractor became more conservative prior to second phase of the treatment. He decided to complete the second

FIGURE 29. DISSIPATION OF NETHANE GAS WAS DEMONSTRATED BY THE CONSTANT BUBBLING OF THE WATER AROUND THE LOADING PLATFORM

FIGURE 30. CLOSE UP VIEW OF WATER BUBBLING IN TEST SECTION NO. I

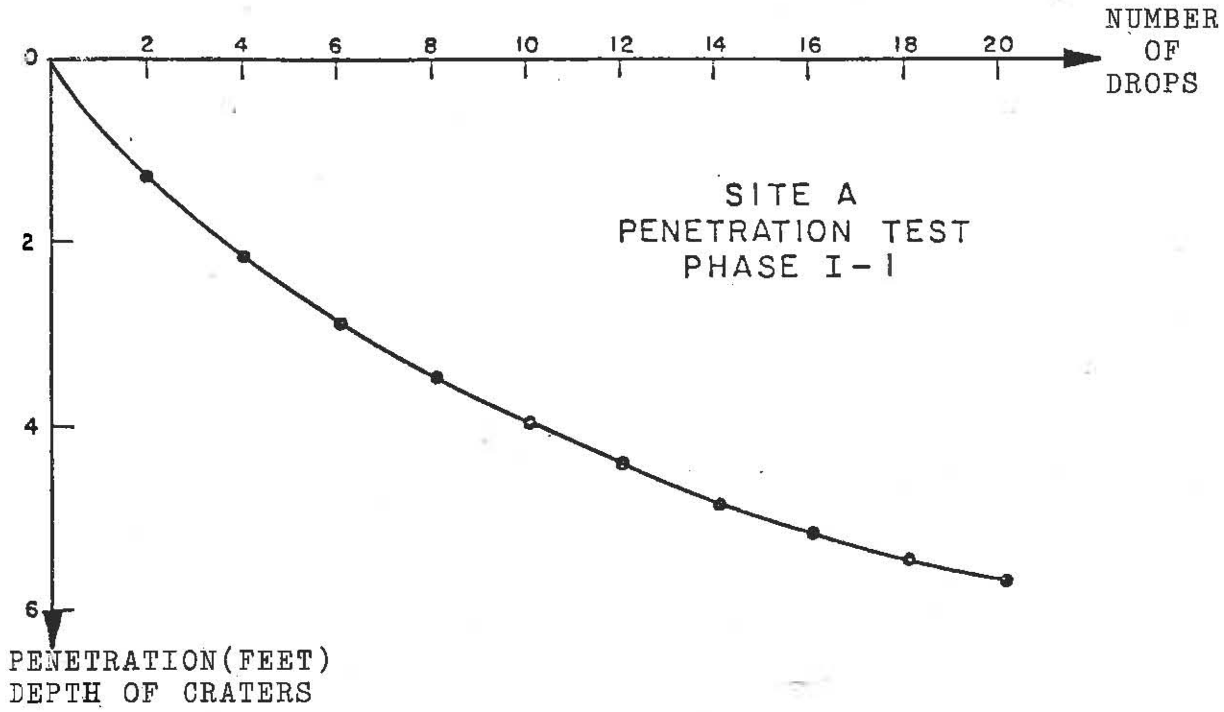


FIGURE 31- PENETRATION TEST ON SITE A PRIOR TO PHASE I-1

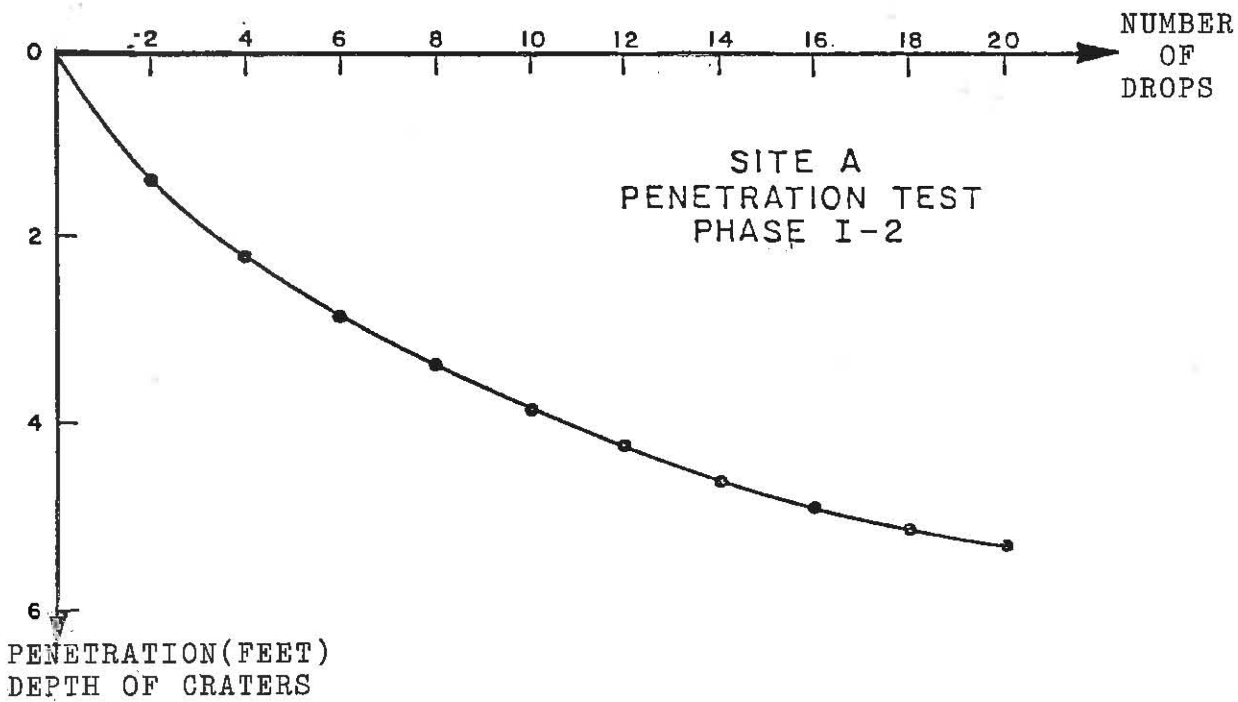


FIGURE 32- PENRTRATION TEST ON SITE A PRIOR TO PHASE I-2

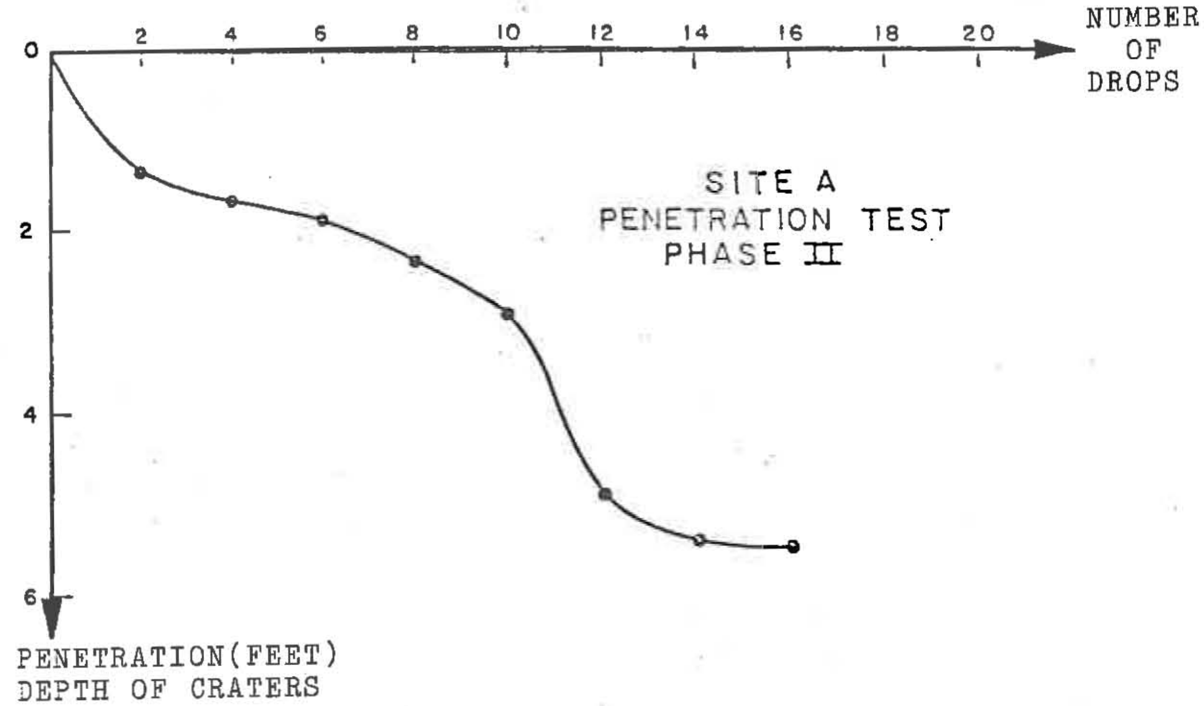


FIGURE 33- PENETRATION TEST ON SITE A PRIOR TO PHASE II

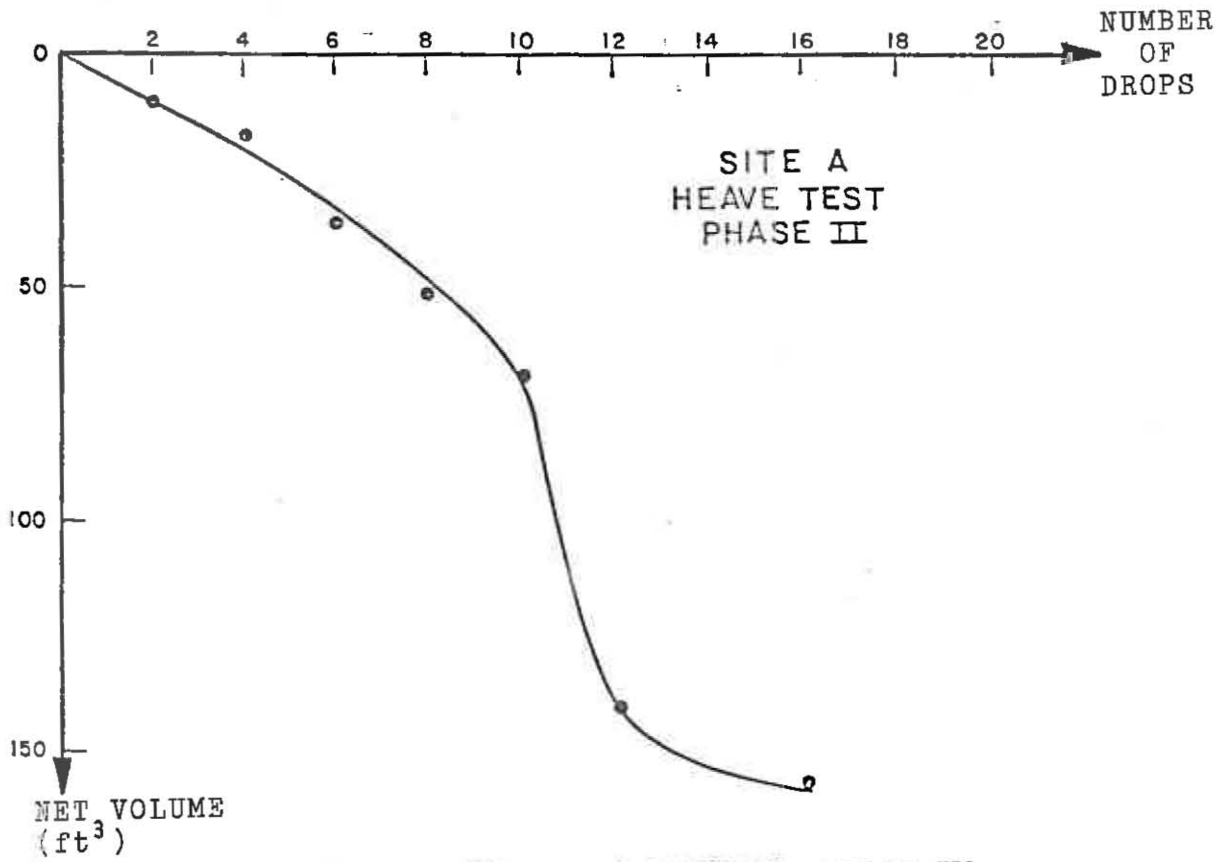


FIGURE 34- HEAVE TEST ON SITE A PRIOR TO PHASE II

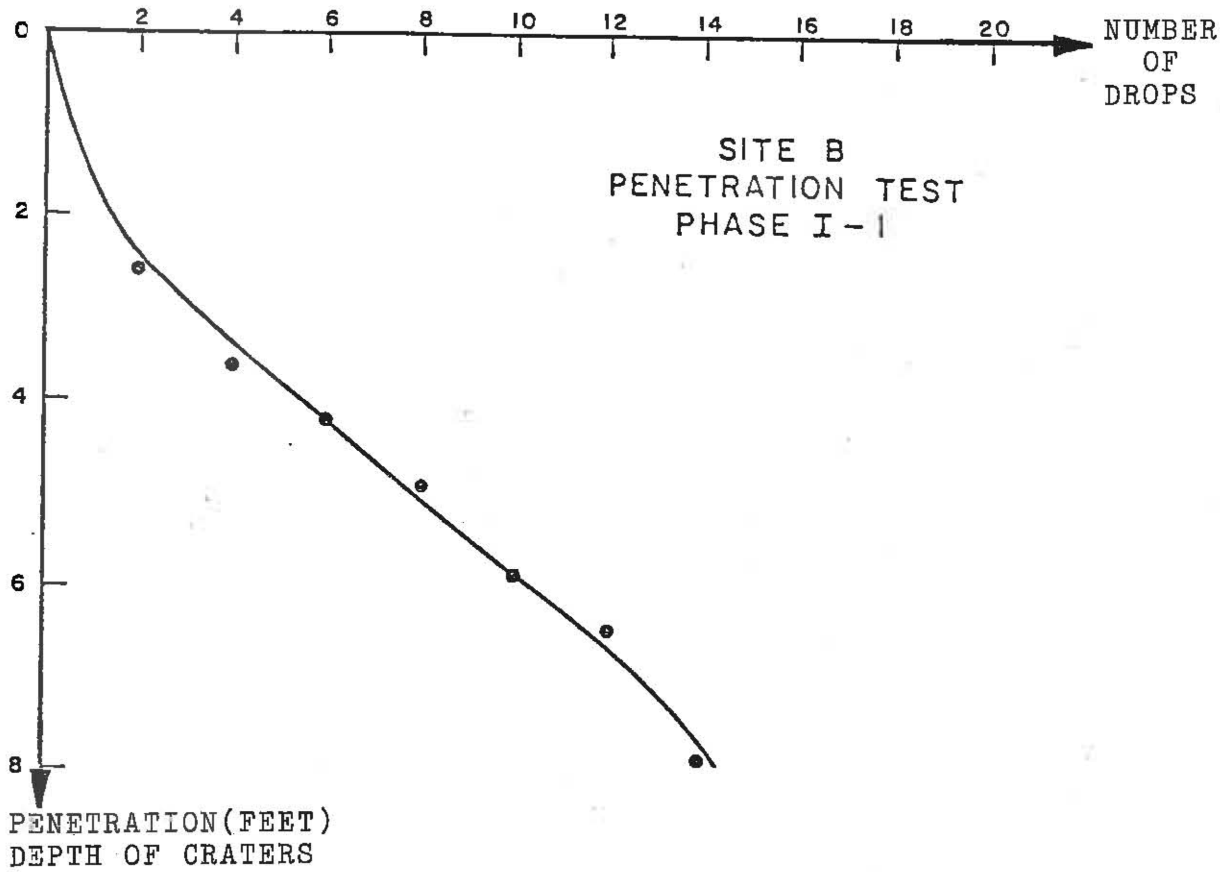


FIGURE 35- PENETRATION TEST ON SITE B PRIOR TO PHASE I-1

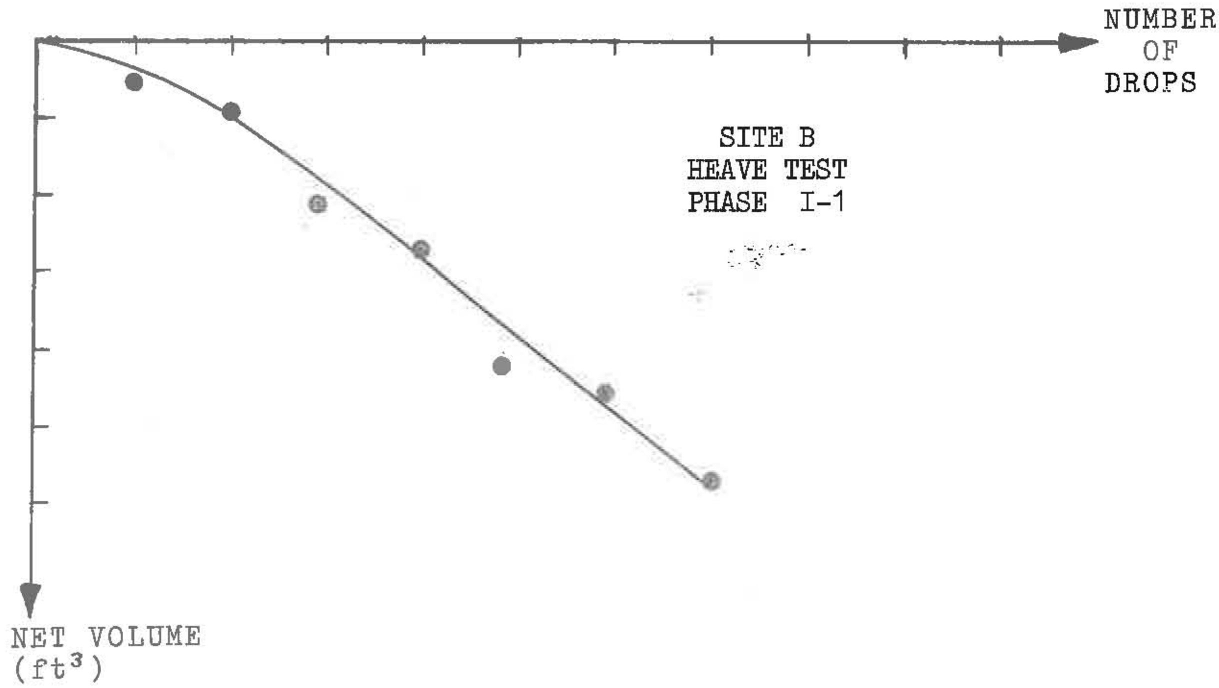


FIGURE 36- HEAVE TEST ON SITE B PRIOR TO PHASE I-1

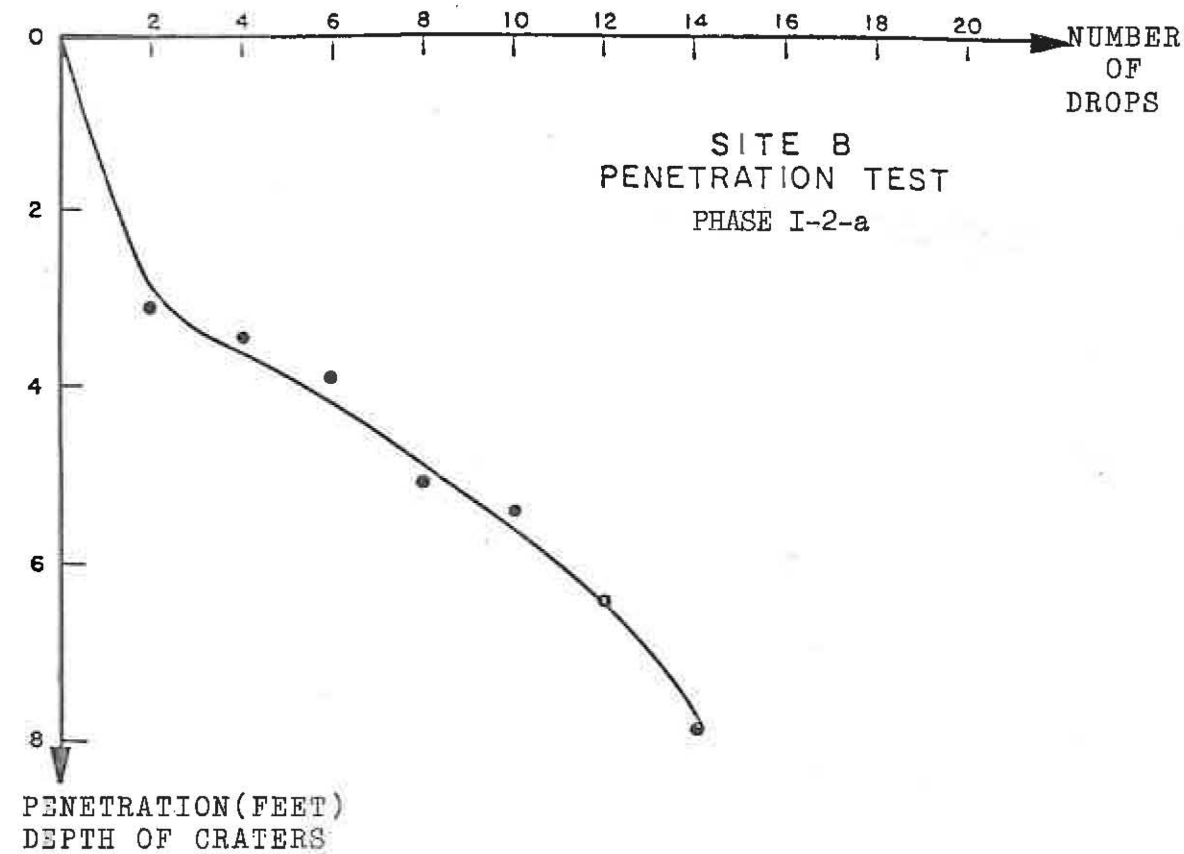


FIGURE 37- PENETRATION TEST ON SITE B PRIOR TO PHASE I-2-a

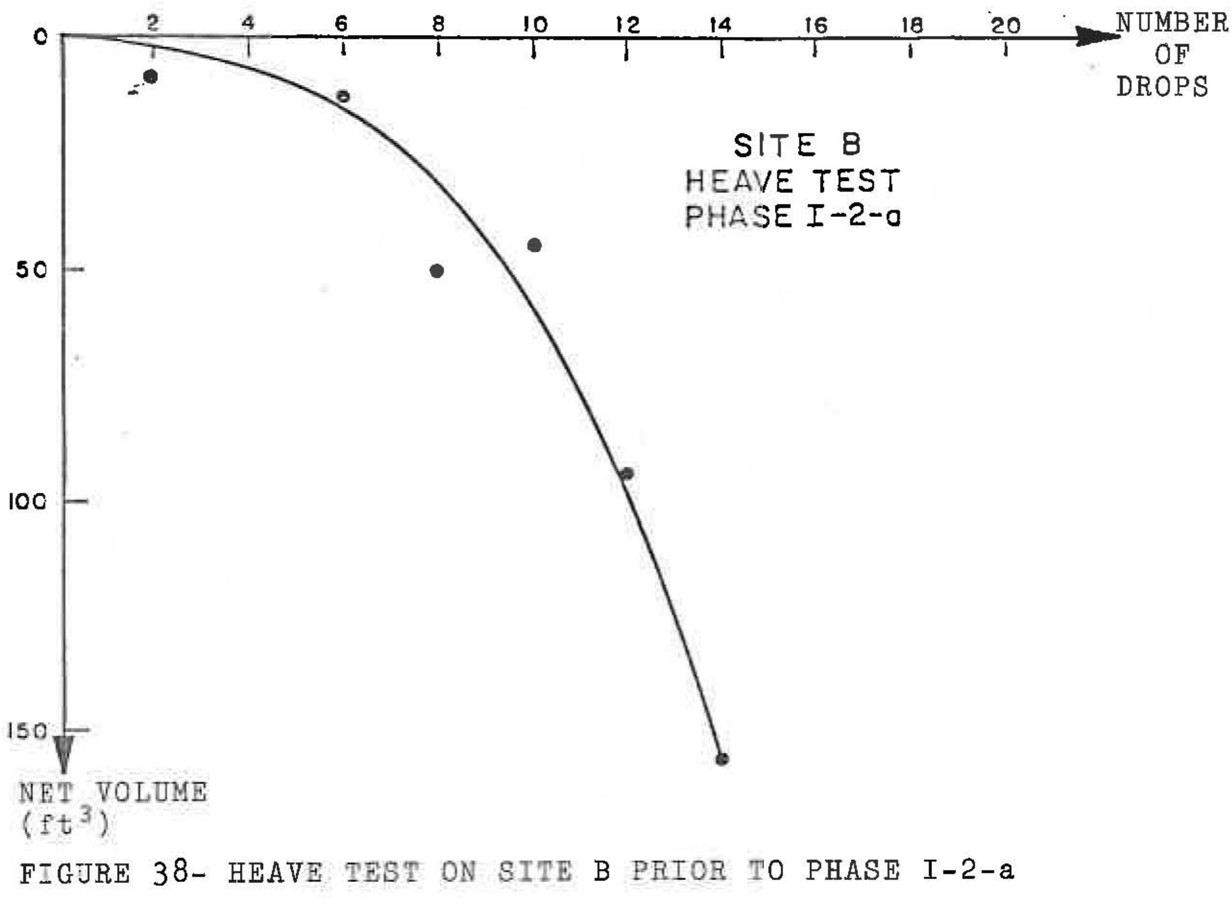


FIGURE 38- HEAVE TEST ON SITE B PRIOR TO PHASE I-2-a

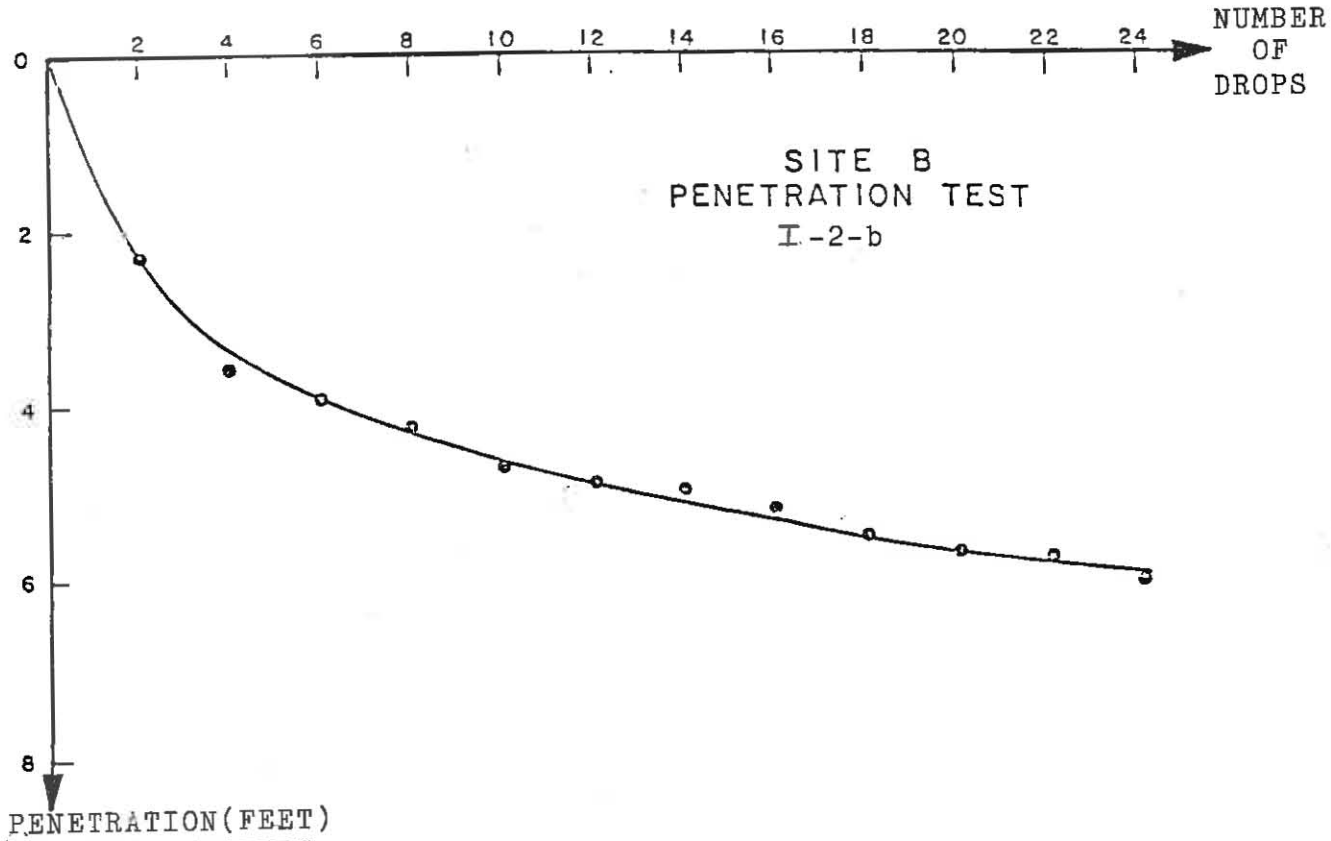


FIGURE 39- PENETRATION TEST ON SITE B PRIOR TO PHASE I-2-b

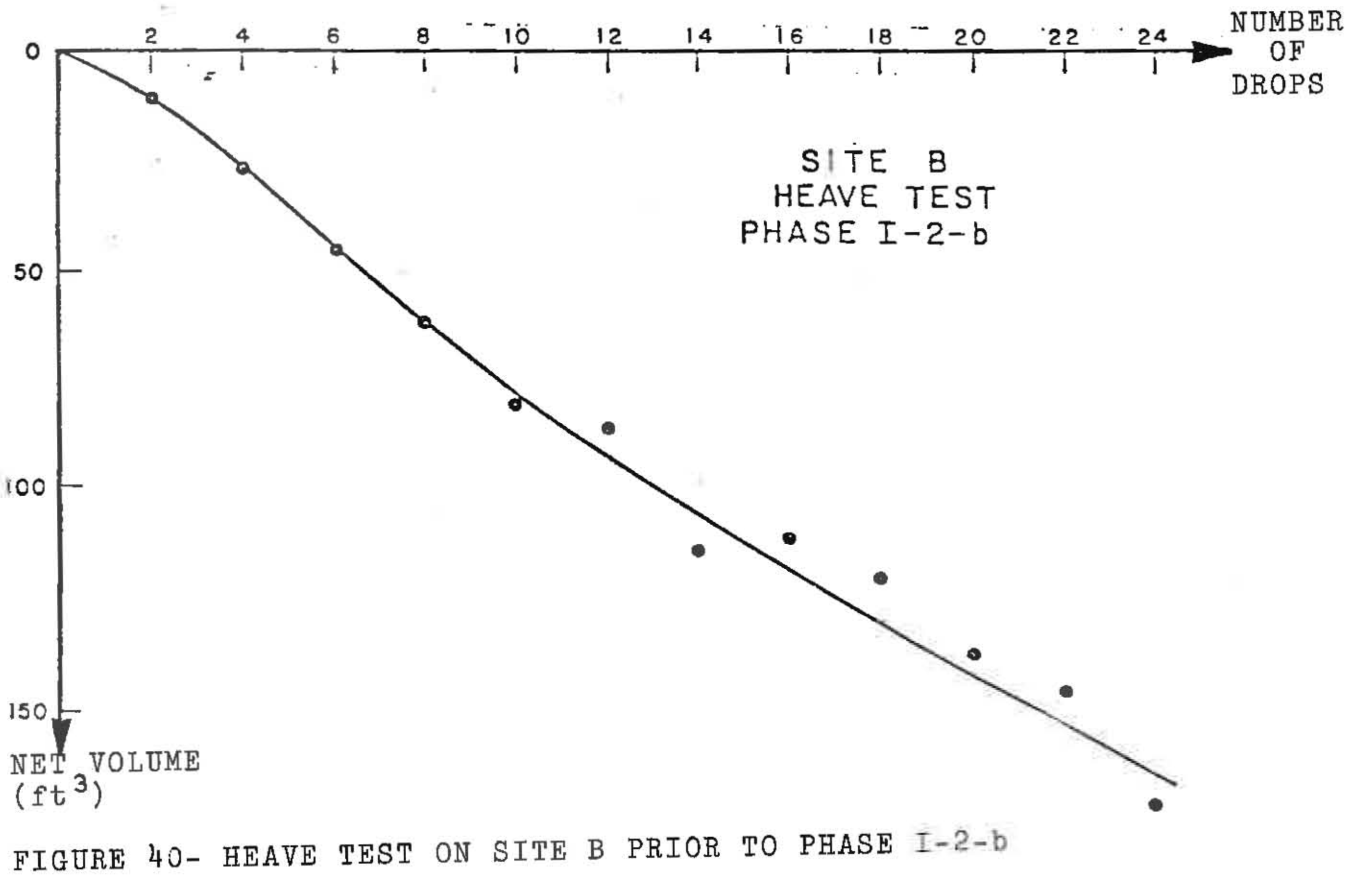


FIGURE 40- HEAVE TEST ON SITE B PRIOR TO PHASE I-2-b

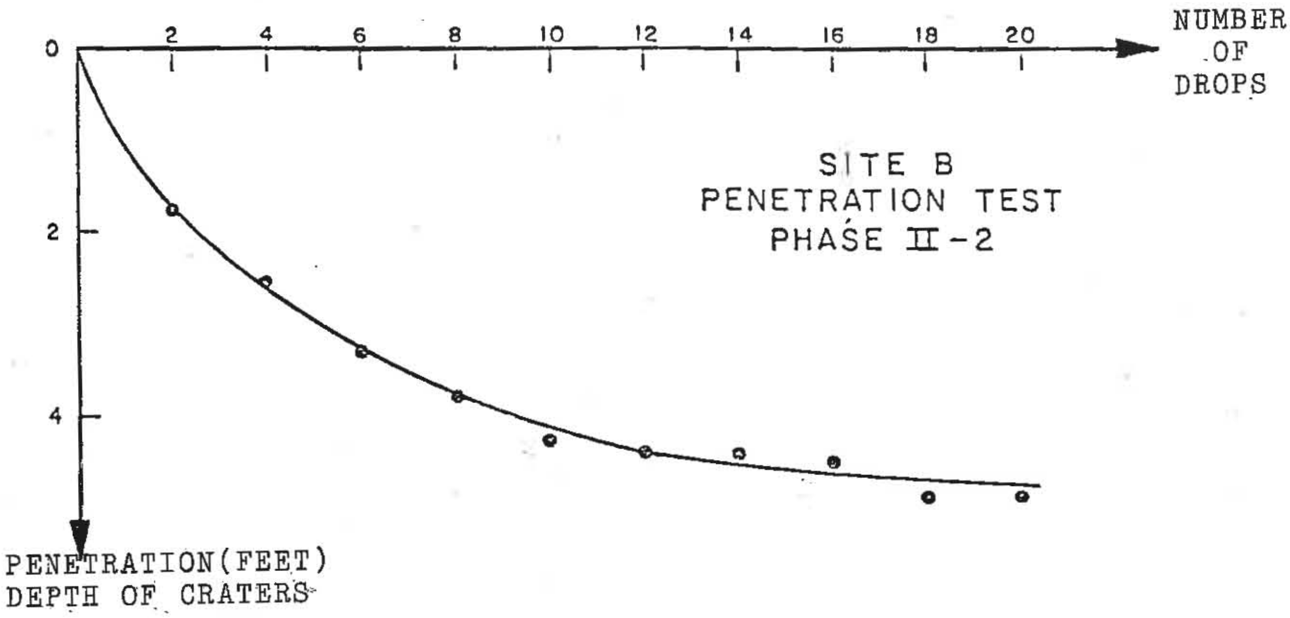


FIGURE 41- PENETRATION TEST ON SITE B PRIOR TO PHASE II-2

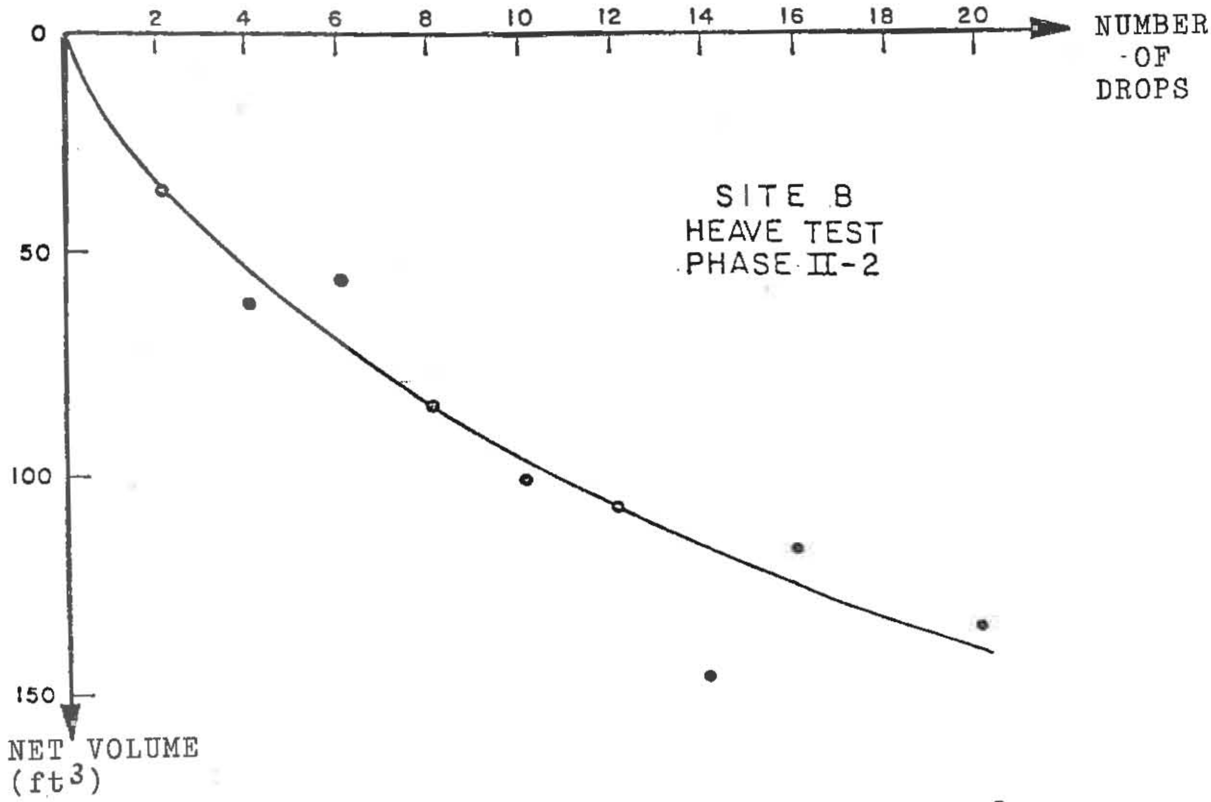


FIGURE 42- HEAVE TEST ON SITE B PRIOR TO PHASE II-2

phase of compaction with two sub-phases II-1, and II-2 with 10 drops on each grid point. Figure 41 shows the results of pentration test prior to sub-phase II-2. Based on this figure, the contractor used 10 drops as the optimum number of drops for this phase of compaction.

One way to investigate the effectiveness of the Dynamic Compaction is to measure volume of craters at the end of each phase of the treatment. The results are presented in Figures 43 through 49. Two important conclusions are obtained by plotting these graphs: (a) the reduction in volume of craters at the end of each progressive phase indicates that the ground material are becoming denser as the treatment progresses, and (b) Dynamic Compaction induced approximately 3 ft. of settlement on site A and 3.5 to 4 ft. on Site B. Settlements are estimated by averaging the total measured volume, reduced by the percentage of heave determined during the heave test and divided by the area concerned. The net enforced settlement is a good approximation of the amount of strain induced in deep layers.

An example of volume measurement and analysis is presented in Appendix A.

Standard Penetration tests were employed before and after Dynamic Compaction to determine the increase in stiffness of the subsurface ground material. The results are presented in Figures 50 through 58. There were a total of four test holes in Site A and five holes in Site B. Except for one hole at each site, the remainder of the tests suggested two to three times increase in stiffness of subsurface ground material at the end of compaction. This was based on increase in the number of drops (N values) obtained from Standard Penetration tests. The effective depth of penetration seemed to exceed beyond 20 ft. below the original ground surface.

	A	B	C	D	E	F	G	H	I	J	K
1	304		185		245		280		230		230
2											
3	230		210		185		160		255		250
4											
5	290		270		175* 270**		210		200		240
6											
7	320		250		280		200		210		235
8											
9	255		260		265		305		210		260
10											
11	280		280		230		310		320		280

SITE A

Phase I-1

Volume of Craters in Ft^3
10 drops per print

* after 9 drops
** after 20 drops

FIGURE 43

DYNAMIC CONSOLIDATION

PREPARED FOR

COLORADO STATE DEPARTMENT
OF HIGHWAYS

	A	B	C	D	E	F	G	H	I	J	K
1	235		206		222		177		317		226
2											
3	214		189		299*		140		290		211
4											
5	204		176		176		192		261		278
6											
7	208		177		168		203		146		177
8											
9	284		141		168		186		175		194
10											
11	380		226		170		206		199		186

SITE A

Phase I-2

Volume of Craters in Ft^3

*after 20 drops

FIGURE 44

DYNAMIC CONSOLIDATION

PREPARED FOR

COLORADO STATE DEPARTMENT OF HIGHWAYS

	A	B	C	D	E	F	G	H	I	J	K
1											
2		177		198		211		283		241	
3											
4		165		198		251		312		231	
5											
6		172		177		214		269		267	
7											
8		165		201		221		211		214	
9											
10		236		226		226		201		261	
11											

SITE A

Phase II

Volume of Craters in Ft^3

FIGURE 45

DYNAMIC CONSOLIDATION

PREPARED FOR

COLORADO STATE DEPARTMENT OF HIGHWAYS

	A	B	C	D	E	F	G	H	I	J	K
1	316		260		300		352		278		276
2											
3	289		251		345		321		295		271
4											
5	283		291		332		345		318		337
6											
7	221		241		337		256		241		251
8											
9	301		281		318		266		267		236
10											
11	286		386		323		278		236		231

SITE B

Phase I-1

Volume of Craters in Ft^3

FIGURE 46

DYNAMIC CONSOLIDATION

PREPARED FOR

COLORADO STATE DEPARTMENT
OF HIGHWAYS

	A	B	C	D	E	F	G	H	I	J	K
1	271		267		251		238		255		235
2											
3	207		245		220		241**		251		236
4											
5	243		225		286		294***		281		251
6											
7	241		202		383*		226		272		198
8											
9	310		251		295		201		246		198
10											
11	283		226		236		226		268		226

SITE B

Phase I-2

Volume of Craters in Ft^3

FIGURE 47

DYNAMIC CONSOLIDATION

PREPARED FOR

COLORADO STATE DEPARTMENT OF HIGHWAYS

*24 drops
**12 drops
***14 drops

	A	B	C	D	E	F	G	H	I	J	K
1											
2		389		269		340		398		289	
3											
4		265		371		269		353		331	
5											
6		331		377		300		353		353	
7											
8		309		436		331		331		331	
9											
10		380		376		289		331		331	
11											

SITE B

Phase II-1

Volume of Craters in Ft^3

FIGURE 48

DYNAMIC CONSOLIDATION

PREPARED FOR

COLORADO **STATE** DEPARTMENT
OF HIGHWAYS

	A	B	C	D	E	F	G	H	I	J	K
1											
2		190		221		231		288		283	
3											
4		255		278		274		322		305	
5											
6		208		337		243		379*		302	
7											
8		287		236		251		256		281	
9											
10		255		292		274		265		284	
11											

SITE B

Phase II-2

Volume of Craters in Ft^3

*20 drops

FIGURE 49

DYNAMIC CONSOLIDATION

PREPARED FOR

COLORADO STATE DEPARTMENT OF HIGHWAYS

FIGURE 50 - STANDARD PENETRATION TEST

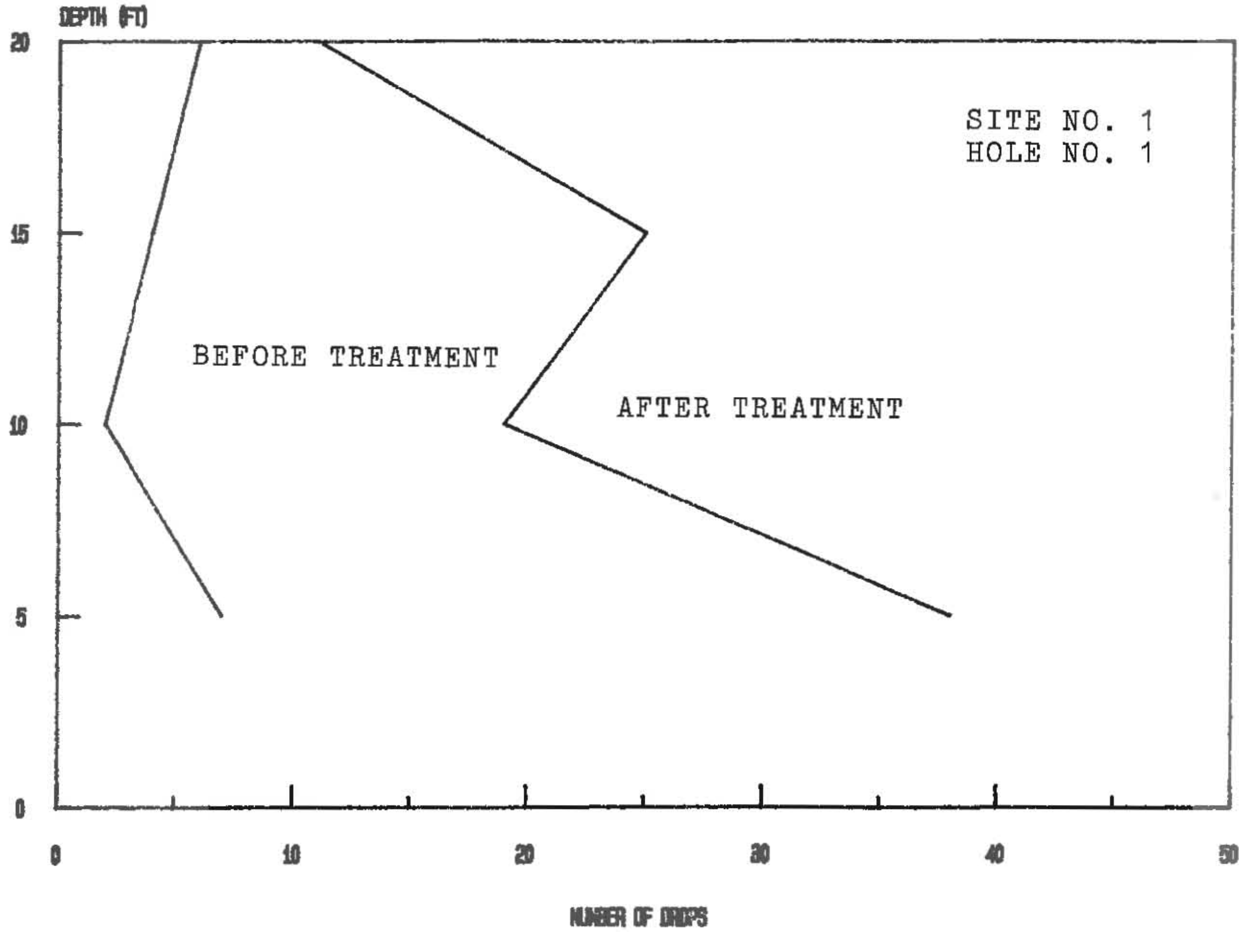


FIGURE 51 - STANDARD PENETRATION TEST

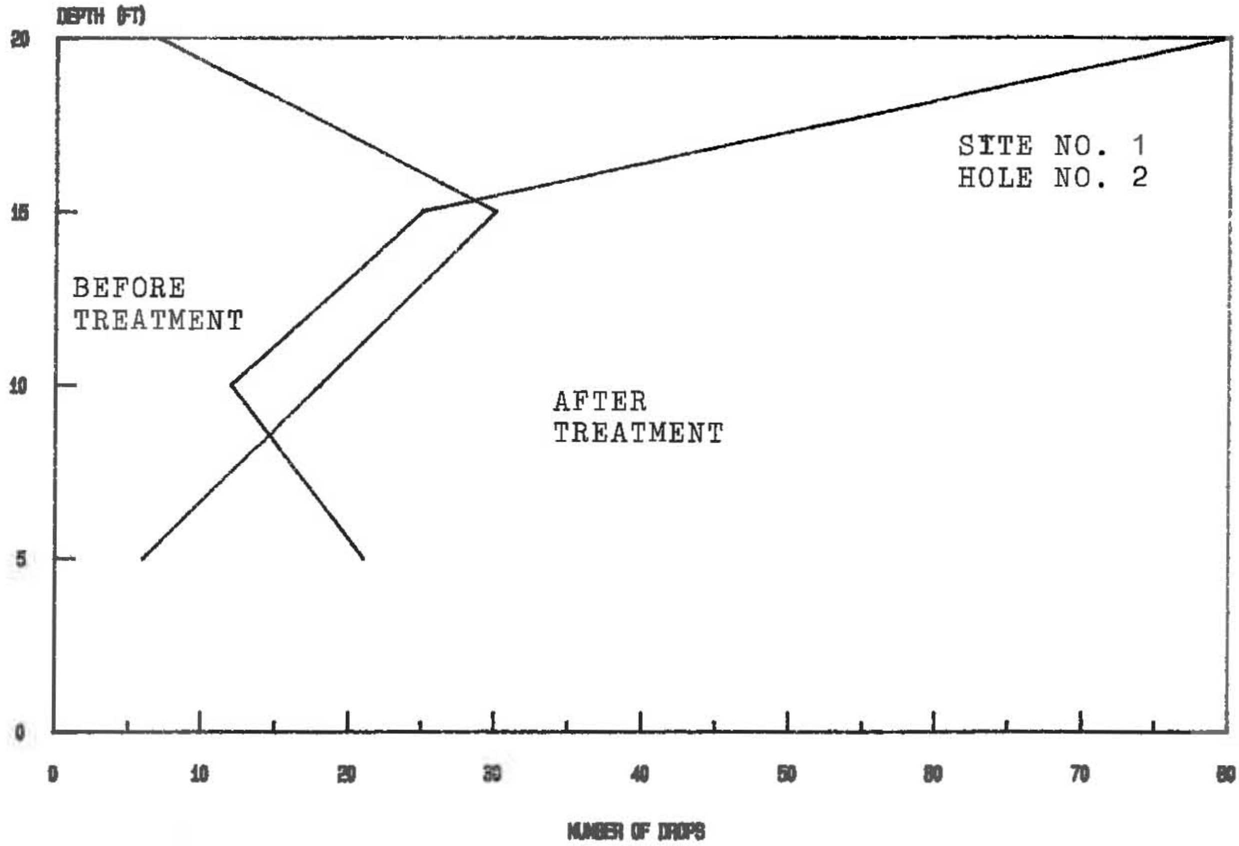


FIGURE 52 - STANDARD PENETRATION TEST

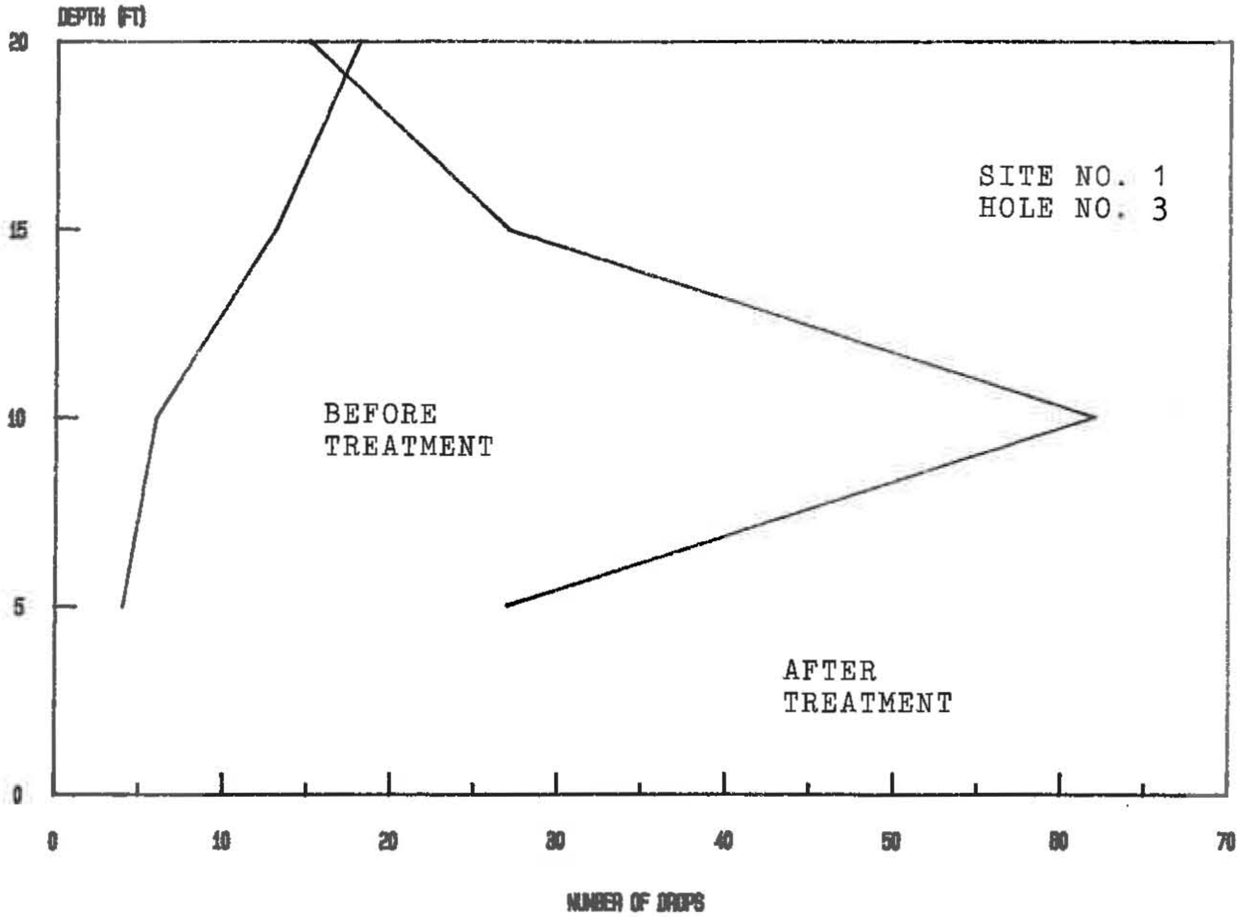


FIGURE 53 - STANDARD PENETRATION TEST

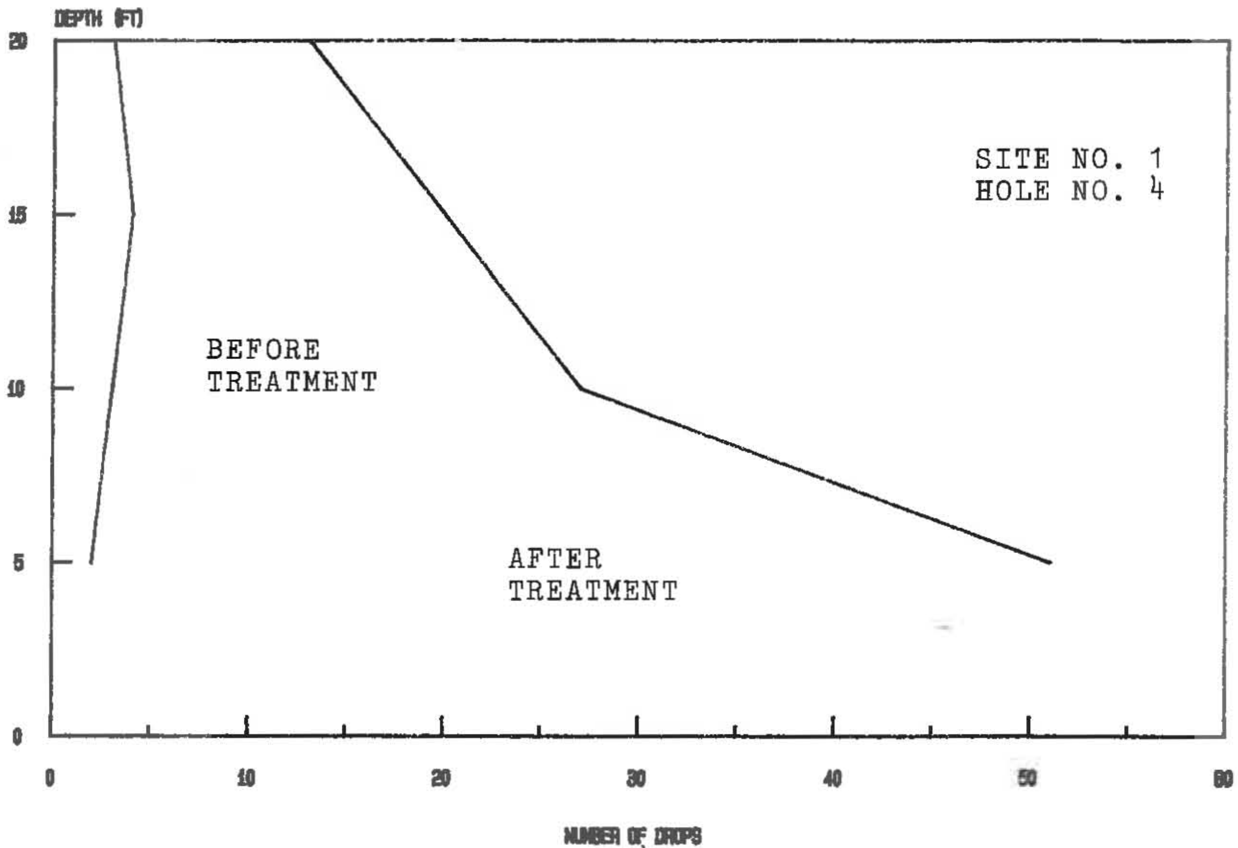


FIGURE 54 - STANDARD PENETRATION TEST

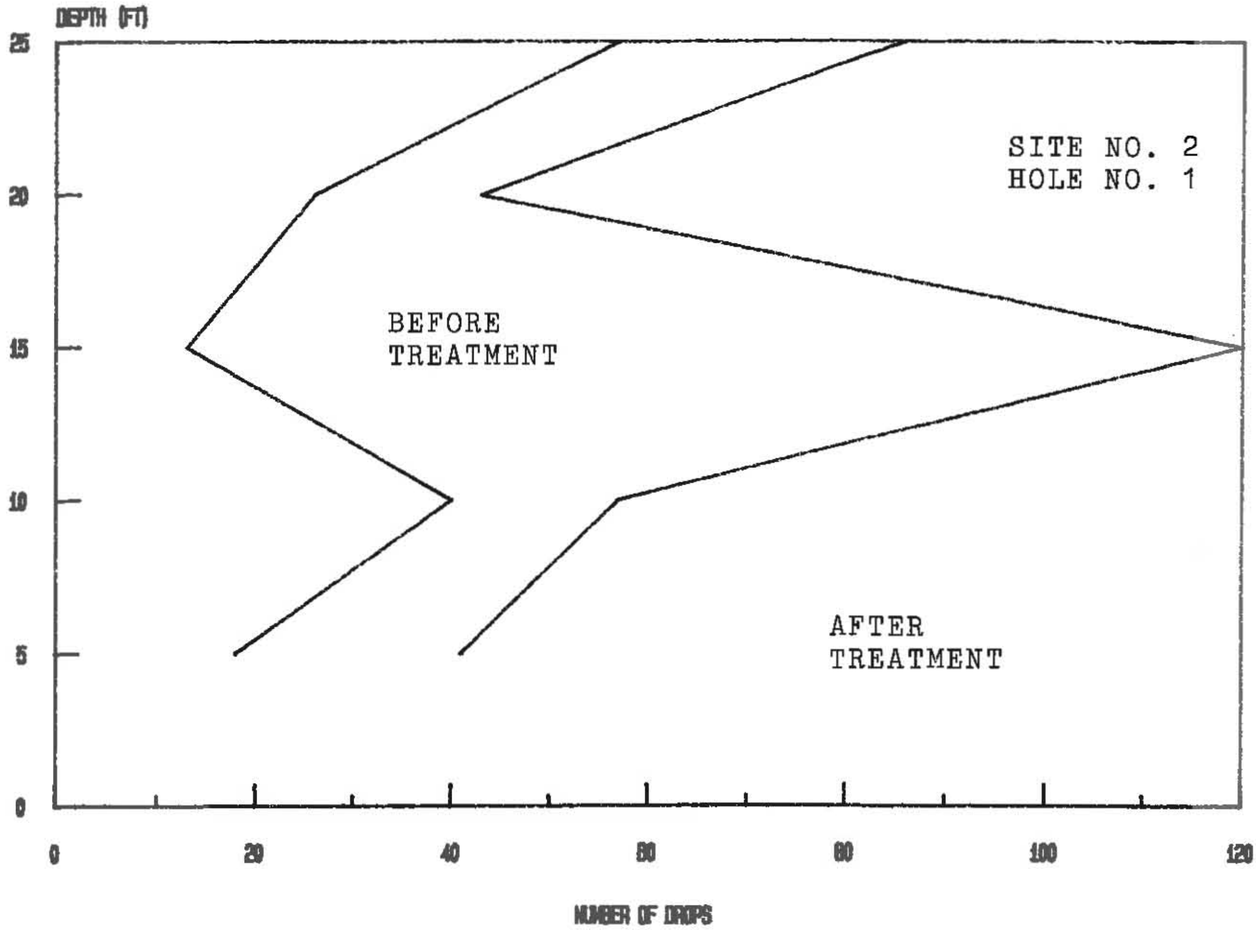


FIGURE 55 - STANDARD PENETRATION TEST

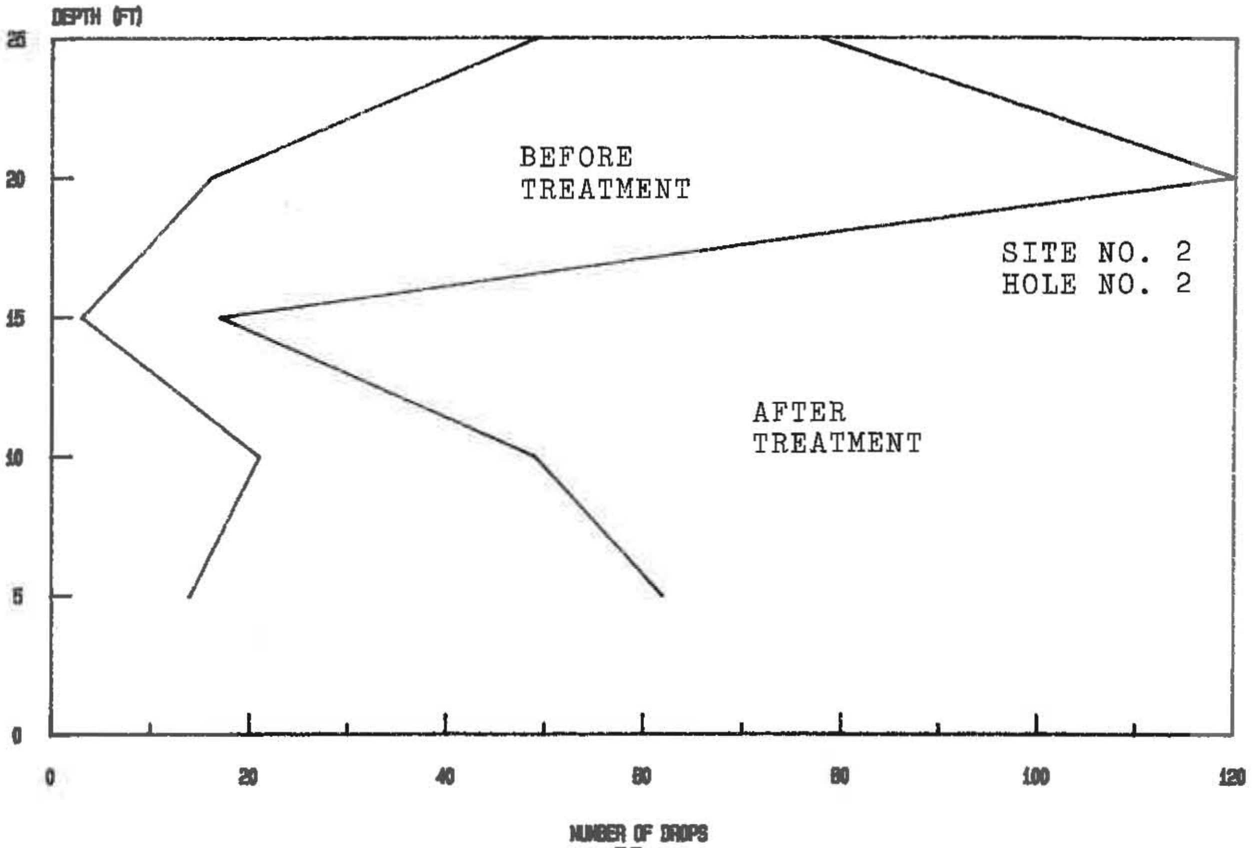


FIGURE 56 - STANDARD PENETRATION TEST

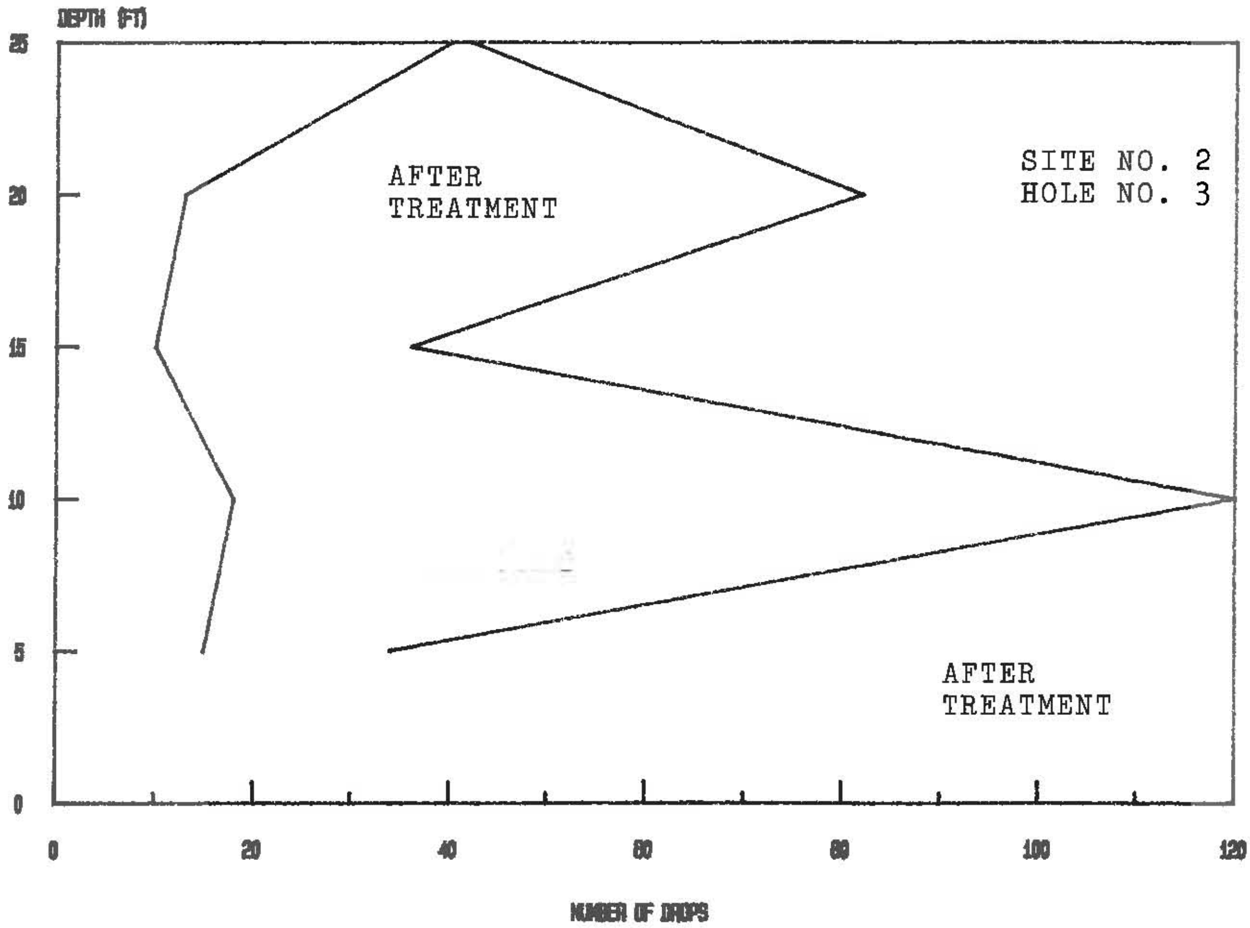


FIGURE 57 - STANDARD PENETRATION TEST

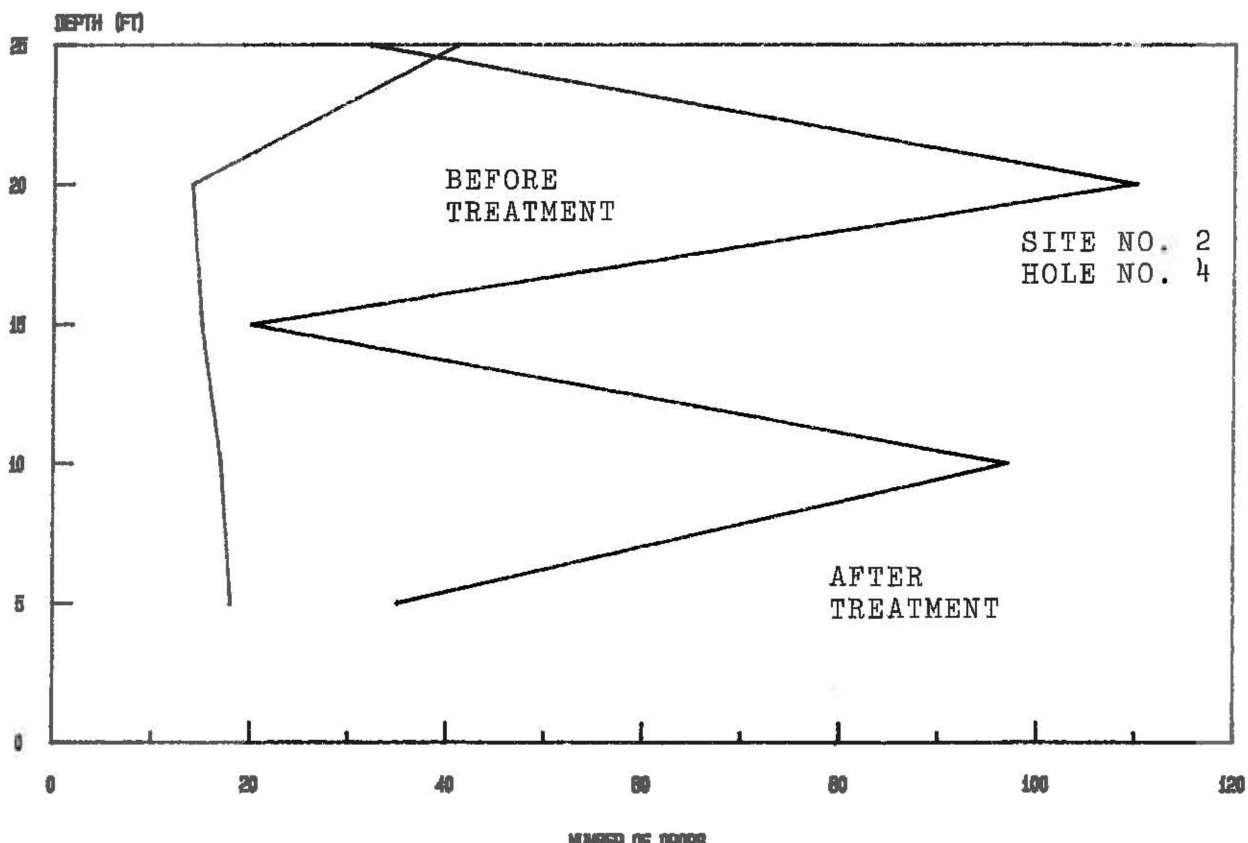
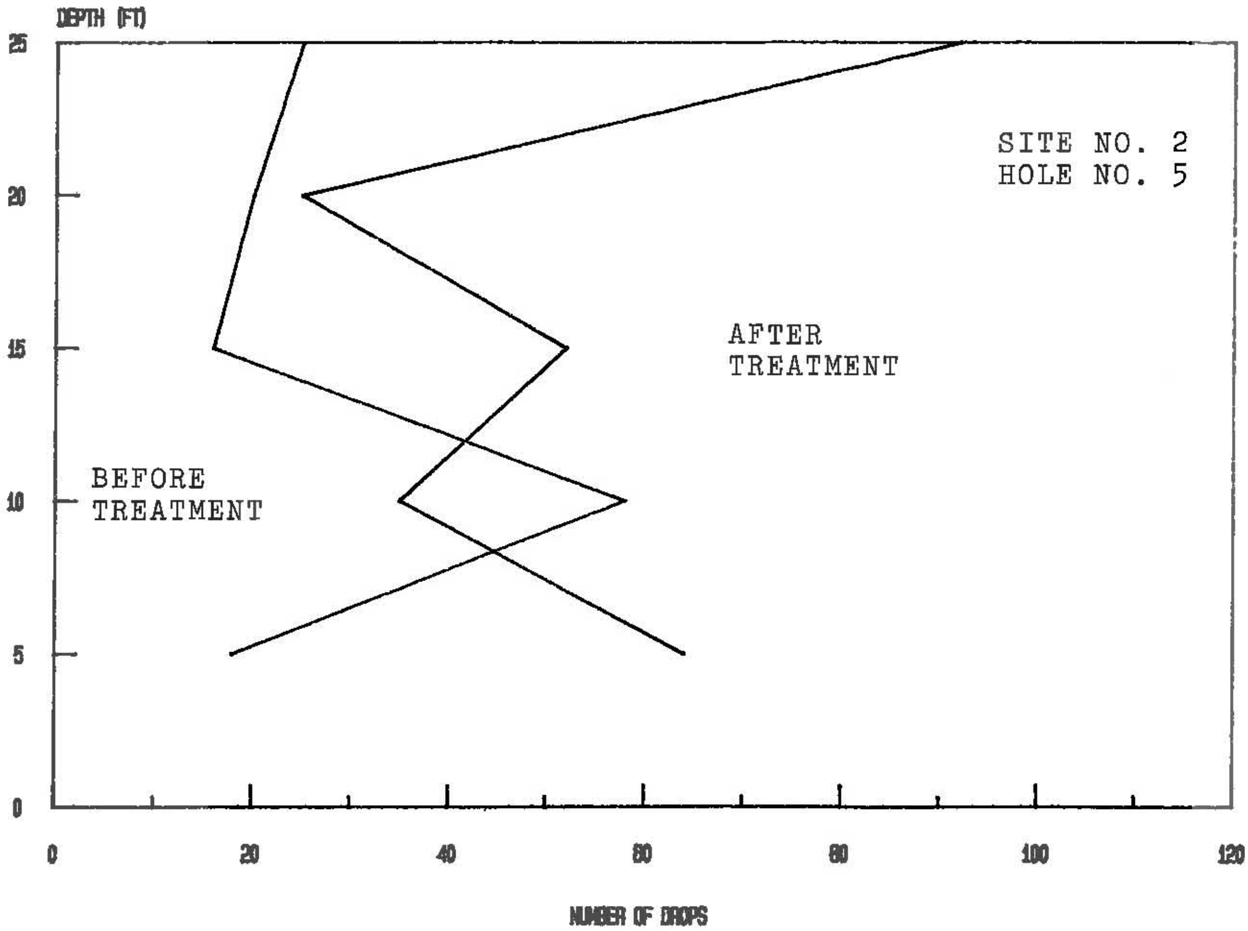


FIGURE 58 - STANDARD PENETRATION TEST



The anchor movements were monitored at the ground surface by means of optical surveying. The results are presented in figures 59 and 60. The results were logical and followed the expected trend. The anchors closer to the ground surface moved deeper into the ground due to absorbing greater share of the impact energy. At Site A, the anchors located at 8, 10, and 16 ft. moved down 3.5, 2.5, and 1.3 ft. At Site B, the anchors located at 9.4, 13.6, and 18.0 ft. moved down 1.25, 1.1, and 0.6 ft. into the ground. In other words, the impact energies decrease as they travel deeper into the ground. The above data suggest that Dynamic Compaction was more effective in Site A. This was definitely related to 5 to 7 ft. of overburden flyash material overlying the trash in Site B. Flyash behaves like cohesive soil which is least affected by Dynamic Compaction treatment.

Results of the Static Load tests are presented in Figures 61 and 62. The results are unsatisfactory and do not represent the actual ground surface settlements. The settlement plates were first placed horizontally on top of the ground surface. But unfortunately, the contractor used a heavy duty conveyor and built the static load from one direction. As a result, after completion of the loading, the extension tubes were bent and no longer represented the true ground surface movements.

The vibrations generated by the Dynamic Compaction treatment were measured to determine their effects and safety on the adjacent structure.

For vibrations generated by blasting, Nicolls (1971) proposed a safe peak particle velocity of 2 inches per second. But for vibrations generated by traffic, Leonard and Whiffin (1971) set a safe level of 0.2 inches per second, an order of magnitude less than for blasting vibrations. This is a reflection of the more continuous and sustained nature of traffic vibrations causing them to be more damaging and annoying.

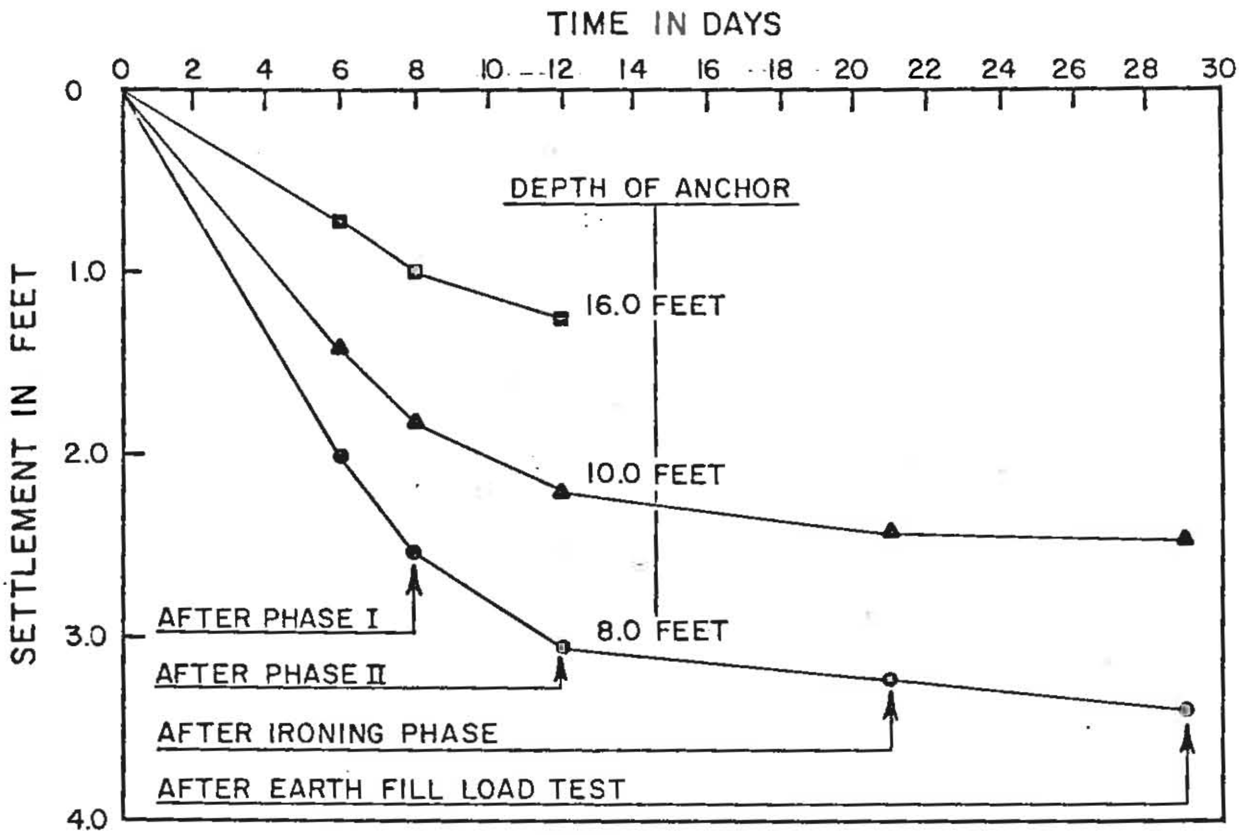


FIGURE 59

SITE A
SETTLEMENT OF ANCHORS

PREPARED FOR

COLORADO STATE DEPARTMENT
OF HIGHWAYS

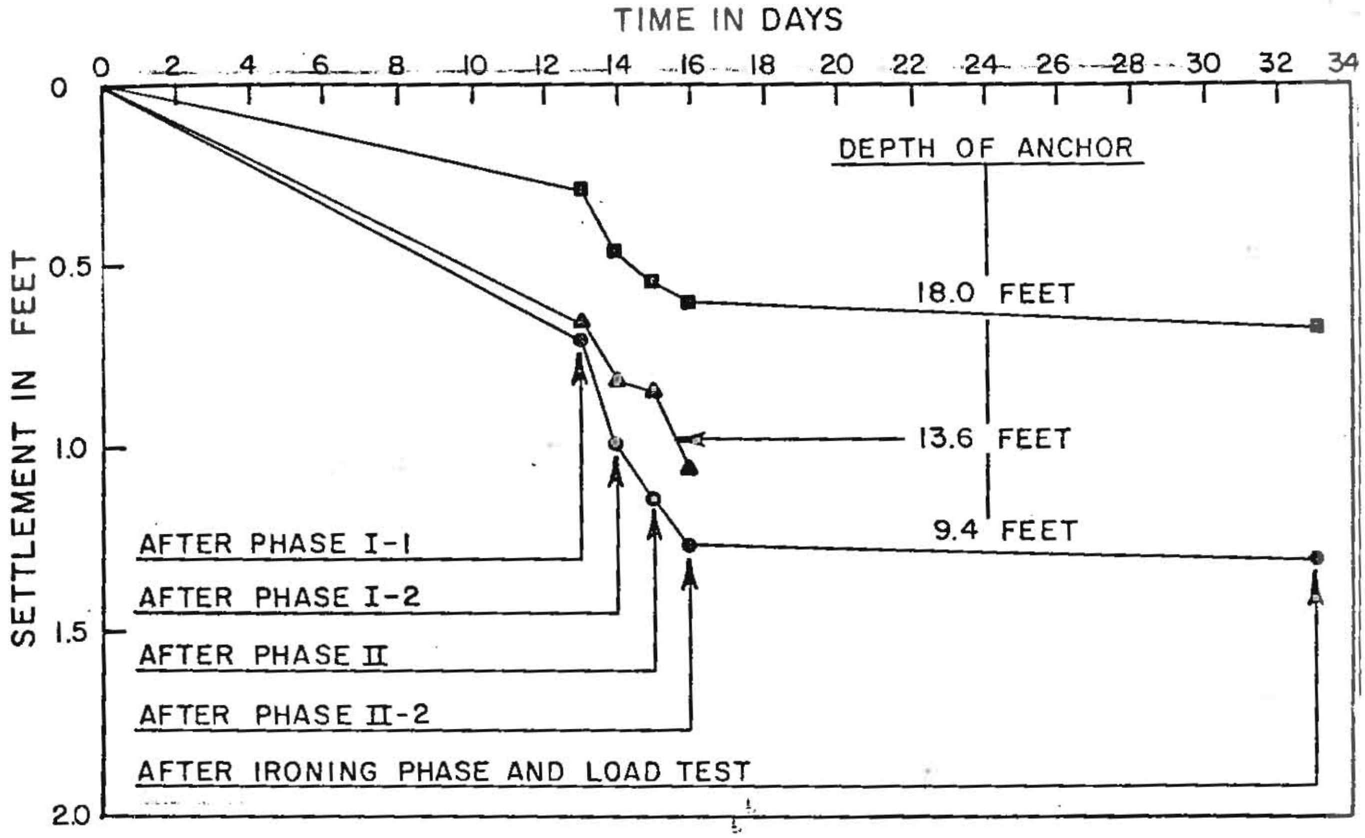


FIGURE 60

SITE B
SETTLEMENT OF ANCHORS

PREPARED FOR

COLORADO STATE DEPARTMENT
OF HIGHWAYS

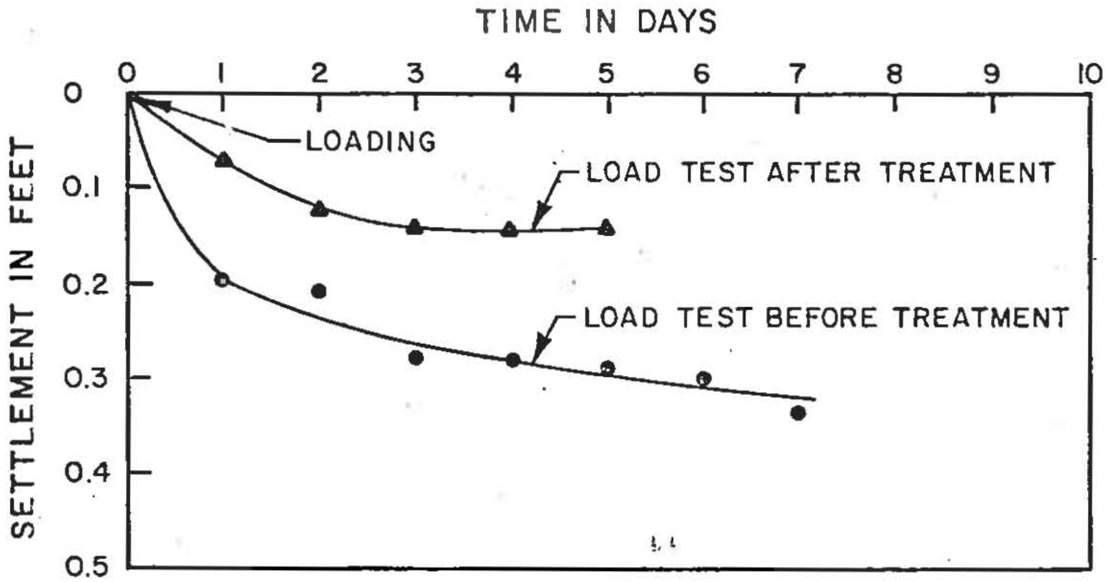


FIGURE 61

SITE B
EARTH FILL LOAD TESTS
BEFORE AND AFTER TREATMENT

PREPARED FOR

COLORADO STATE DEPARTMENT
OF HIGHWAYS

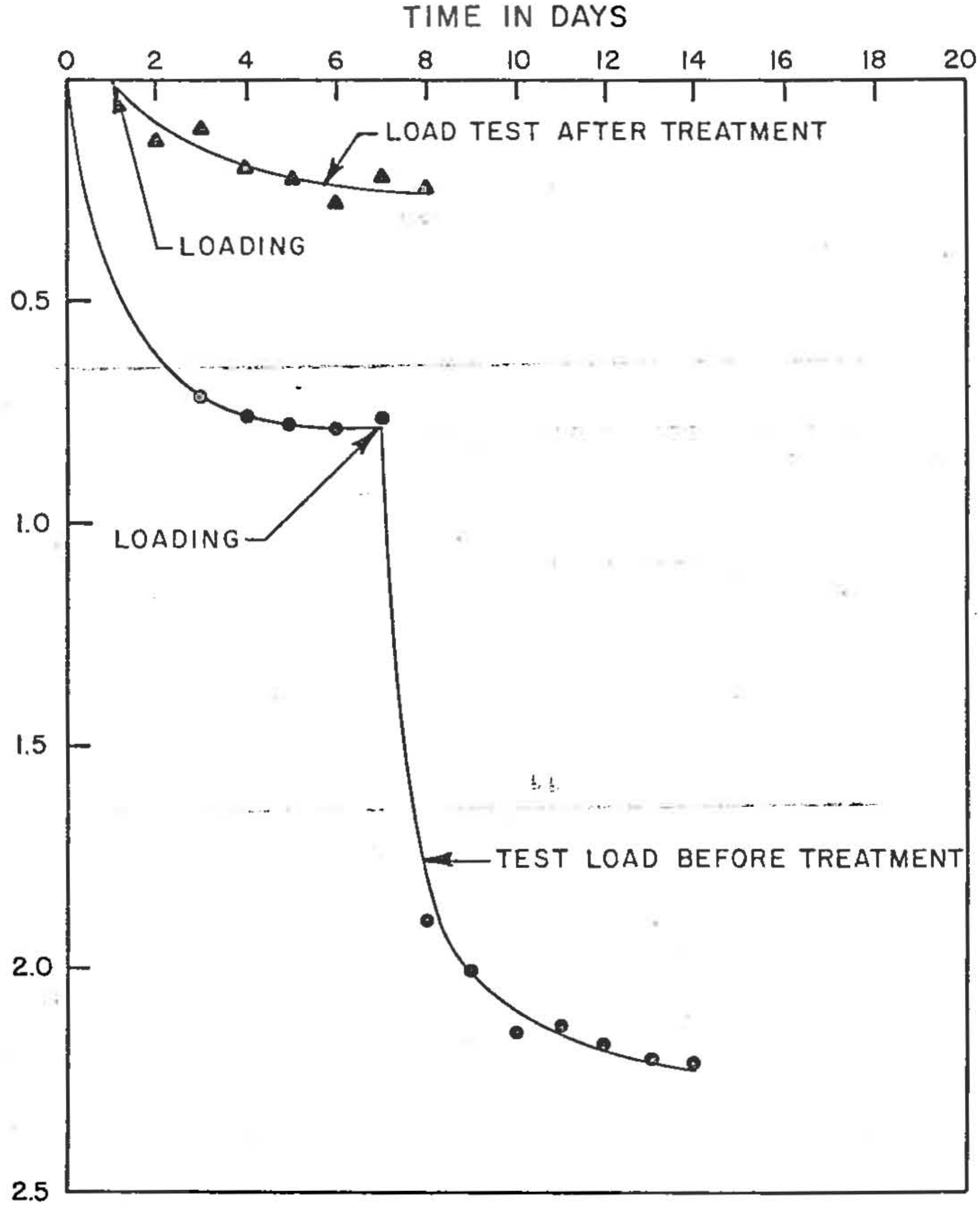


FIGURE 62

SITE A
EARTH FILL LOAD TESTS
BEFORE AND AFTER TREATMENT

PREPARED FOR

COLORADO STATE DEPARTMENT
OF HIGHWAYS

The vibrations caused by Dynamic Compaction are similar to those caused by blasting. Therefore, the safe peak particle velocity of 2 inches per second was used for comparison purposes.

Two seismographs were available for measurement of the shear wave velocities generated by load impact. The results are presented in Table 4. It is evident that the magnitude of wave velocities decreases as the distance from the impact point increases. The maximum velocity was 1.39 inches per second at a distance 42 ft. from the impact point. This is well within the acceptable range, therefore; it is considered safe on the adjacent structures. The last measurement was taken in the backyard of the closest residential house 251 ft. away from the impact point. The measured velocity was .043 inches per second which is considered negligible.

TABLE 4 RESULTS OF PEAK SHEAR VELOCITIES MEASURED DURING THE COMPACTION PROCESS.

	SEISMOGRAPH NO.1 SITE NO.1			
DISTANCE (ft)	42	90	100	175
VELOCITY (IN/SEC)	1.39	0.77	0.56	0.25

A

	SEISMOGRAPH NO.2 SITE NO.1			
DISTANCE (ft)	42	50		
VELOCITY (IN/SEC)	1.1	0.94		

B

	SEISMOGRAPH NO.2 SITE NO.2			
DISTANCE (ft)	50	98	150	251
VELOCITY (IN/SEC)	0.51	0.35	0.08	0.043

C

CHAPTER VI

CONCLUSION AND RECOMMENDATION

6.1 Conclusion

The Dynamic Compaction experiment was performed at two test sections located in sanitary landfills with high groundwater tables in northwest Denver. The experiment was carried out smoothly and the results exceeded expectations.

The main drawback to this experiment was the inability to theoretically calculate the behavior of the subsurface ground material due to the unknown nature of the trash. None of the geotechnical laboratory tests was suitable for trash; and as a result, we had to investigate the problem from a practical point of view with lack of a solid theoretical model.

The results of standard penetration tests (SPT) indicate a two to four time improvement in the driving resistance of both test sections. This means that the ground stiffness is increased two to four times; the settlements will be reduced; and the factor of safety increased accordingly. The results also indicate that effective depth of penetration extends beyond 20 ft. This meant that both sanitary landfills from ground surface down to bedrock were influenced by this method.

Instrumentation was valuable in this experiment. After completion of the experiment, the results of the standard penetration tests, driving anchors, heave and penetration tests produced valuable information to evaluate the effectiveness of Dynamic Compaction treatment. Use of piezometers in landfill areas is strongly discouraged due to the large void volumes in the subsurface material. The results of the seismic tests indicate that the shock waves produced by the impact load have no effect, or minimal effect, on the adjacent structures. The results of the static load test are unreliable due to poor procedures adopted by the contractor. The static load test, if set up properly, can produce valuable information on the change of settlement magnitude

In sanitary landfills, settlements are caused either by compression of the void or decaying of the trash material over long periods of time. Dynamic compaction is effective in reducing the volume of voids and consequently reduces the amount of immediate and primary settlements after construction of the highway embankments. This method is also effective in reducing the decaying problem since collapse of voids means less available oxygen for decaying process. Therefore, future settlements are expected due to secondary consolidation process and some decaying of the trash material.

Based on a literature review performed by University of Colorado, it is estimated that 60 to 70% of the total settlement in sanitary landfills occurs rapidly (immediate and primary consolidations). The remaining 30 to 40% of the total settlement will take place slowly due to secondary settlements and continuous decaying of the trash material. Therefore, it is safe to assume that Dynamic Compaction will be effective in reducing the immediate settlements by considerable amounts. It may also reduce the decaying process, but it will not help to eliminate this process. All estimated numbers at this point are speculative and are subject to change depending on the long-term performance of the pretreated landfills. The long-term performance of treated areas can only prove the effectiveness of Dymanic Compaction. If the post construction settlements are uniform and not excessive, then Dynamic Compaction may be considered an appropriate alternative. On the contrary, if the settlements are excessive and nonuniform, then alterantive methods such as preloading or flyash grouting may be considered.

Presently, the Colorado Department of Highways is conducting a study on preloading of sanitary landfills. It is hoped to compare the results of preloading method with those obtained from Dynamic Compaction to determine the effectiveness of each method in stabilizing sanitary landfills.

For this project, the average cost of Dynamic Compaction was $7.74 per sq. yd. In addition to this, the cost of cushion material must be added to determine the total cost. The cushion material was provided by the prime contractor at the cost of $5.35 per ton.

It is obvious that a substantial amount of money may be necessary to stabilize sanitary landfills. But the end result will be smoother roads with less maintenance work and consequently substantial savings on maintenance costs on a long term basis.

6.2 Recommendation

Dynamic Compaction, if designed and performed properly, can produce immediate and dramatic results. On the other hand, if it is designed poorly and not controlled, then the results may not meet the standard design values, and consequently it may be a total loss of time and money. The following procedure is recommended to complete the design of a Dynamic Compaction operation:

1. Determine the type and quality of the subsurface ground material by means of a quality ground exploration program.
2. Prior to Dynamic Compaction, determine the strength and compressibility properties of the ground material by more than one method.

3. Perform a theoretical settlement analysis based on the engineering properties of subsurface ground material.
4. Design a proper grid pattern with effective grid point spacings. Use the experience of the specialty contractors to overcome any doubts.
5. Specify the type and quantity of the cushion material. The cushion material is the most expensive part of Dynamic Compaction operation. Therefore, it should be minimized to keep the costs down. The most important criteria about the cushion material is that it needs to be granular and contain lots of various size gravels.
6. Specify the weight and the height of the falling object.
7. Specify the number of the individual phases (primary, secondary, tertiary, and ironing) to complete the Dynamic Compaction operation.
8. Prior to beginning of each phase, determine the number of the effective drops using the penetration and heave tests.
9. Repeat step number two to obtain information for comparison purposes.
10. Theoretically evaluate the reduction in future settlements using the new engineering properties of the subsurface ground material.
11. Monitor the long-term performance of the treated sites by means of the surveying techniques.

6.3 Implementation

This method was recently used to stabilize one of the sanitary landfills located along the future path of I-76 in northwest Denver. It is also evident that this method will be used increasingly in future projects, therefore, it is suggested to use this method with quality control to achieve maximum improvement in soft foundation material.

APPENDIX A

DETAILS OF VARIOUS IN-SITU TESTS

DURING DYNAMIC COMPACTION TREATMENT

A-1 Details of Penetration Test

Prior to each phase of Dynamic Compaction, Penetration tests should be performed to determine the effective number of drops for each phase of the treatment. The following procedure is generally followed for each penetration test:

1 - Choose an appropriate location for the test.

2 - Determine the ground surface elevation.

3 - At each two drop intervals, measure the depth and the diameter of the crater.

4 - Plot the number of drops versus penetration depth.

5 - Estimate the point with 80 percent maximum curvature on the curve. Assume the number of drops corresponding to this point is the effective number of drops for the next phase of the compaction.

Table A-1 contains data obtained during a typical penetration test. The volumes are calculated based on the assumption that the craters are cylindrical as shown in Figure A-2. The results are plotted in Figure A-1. Point A seems to be at approximately 80% maximum curvature. Therefore, the effective number of drops should be equal to or less than 14 as illustrated in Figure A-1.

A-2 Details of a Heave Test

The heave test is designed to measure penetration with more precision. In this test, the depth and the diameter of the crater are measured at two

Table A-1

Typical, Data Collected from
a Penetration Test

Number of Drops	Elevation	Height (H) (ft)
0	1255.00	0
2	1253.70	1.25
4	1252.90	2.10
6	1252.10	2.90
8	1251.70	3.30
10	1251.10	3.90
12	1250.90	4.10
14	1250.50	4.50
16	1250.25	4.75
18	1250.00	5.00
20	1249.90	5.10

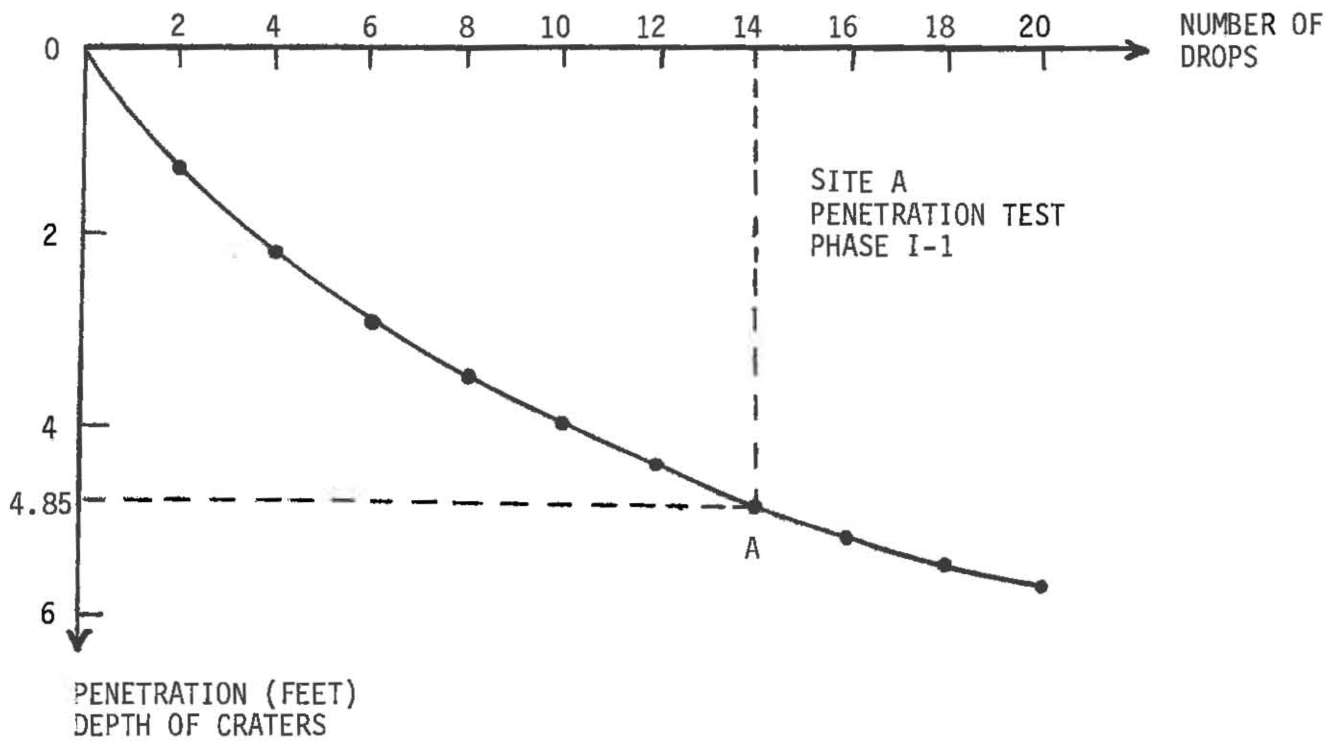


FIGURE A-1 TYPICAL PLOT FOR A PENETRATION TEST

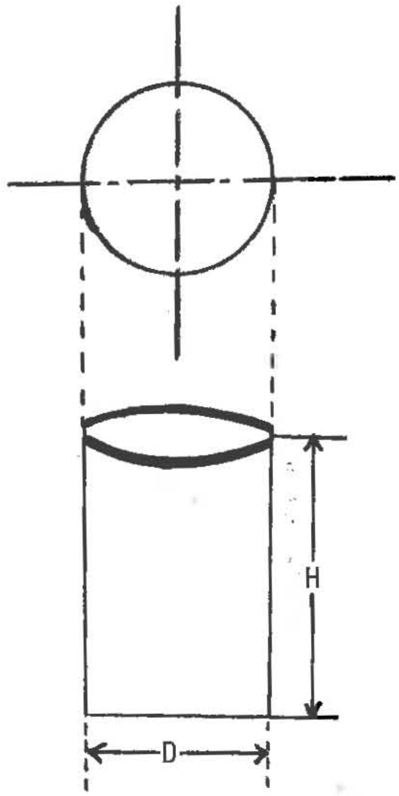


FIGURE A-2 THE CRATERS ARE ASSUMED TO BE CYLINDRICAL FOR THEORETICAL PURPOSES.

drop intervals. In addition, the heave around the crater is measured by obtaining the elevations of the surrounding soil at 7, 10, 13 and 16 feet from the center of the impact in four different directions. Tables A-2 and A-3 show typical heave test data. The following procedures are used to analyze the data:

1. Find the average elevation of the heave, on each direction, using the following formula:

   $h = (D_1 + D_2 + D_3 + D_4)/4$
   h = Average Elevation of Heave
   D_1, D_2, D_3, D_4 = Elevations at 7', 10' 13' and 16' from the center of impact around the crater

2. Assume the cross sectional shape of the heave is rectangular as shown in Figure A-3.

3. For each quarter of the circle around the crater find the surface area of the heave. This is accomplished by using the following formula and the dimensions specified in Figure A-3A.

   $A_Q = 1/4\ (R\ \ - r\ )$ = One quarter of heave surface area
   $R = 16'$
   $r = 7'$

4. Calculate the heave volume for each quarter using the following formula:

   $V_Q = A_Q(h)$
   V_Q = Heave volume for a particular quarter

5. Calculate the total heave volume by adding the volumes of each quarter.

   $V_H = (V_Q)_N + (V_Q)_W + (V_Q)_S + (V_Q)_E$
   V_H = Total heave volume

D_1 Elevation at 7.0 feet from center of crater
D_2 Elevation at 10.0 feet from center of crater
D_3 Elevation at 13.0 feet from center of crater
D_4 Elevation at 16.0 feet from center of crater

	North				West				South				East			
No. of Drops	D_1	D_2	D_3	D_4	D_1	D_2	D_3	D_4	D_1	D_2	D_3	D_4	D_1	D_2	D_3	D_4
0	50.20	50.21	50.24	50.22	50.33	50.30	50.30	50.30	50.30	50.31	50.32	50.31	50.31	50.34	50.33	50.30
2	50.22	50.22	50.26	50.23	50.35	50.32	50.31	50.30	50.32	50.30	50.33	50.31	50.32	50.35	50.33	50.30
4	50.23	50.23	50.26	50.24	50.36	50.33	50.32	50.31	50.33	50.31	50.34	50.31	50.34	50.36	50.33	50.30
6	50.25	50.23	50.24	50.24	60.36	50.34	50.33	50.31	50.34	50.32	50.34	50.31	50.35	50.37	50.34	50.31
8	50.26	50.24	50.27	50.25	50.37	50.35	50.35	50.32	50.36	50.33	50.35	50.32	50.37	50.36	50.36	50.32
10	50.27	50.25	50.29	50.26	50.39	50.37	50.36	50.33	50.37	50.35	50.37	50.33	50.40	50.37	50.36	50.33
12	50.29	50.27	50.31	50.26	50.41	50.39	50.36	50.34	50.39	50.37	50.39	50.33	50.42	50.39	50.37	50.34
14	50.31	50.30	50.31	50.26	50.42	50.42	50.36	50.34	50.43	50.41	50.41	50.35	50.46	50.43	50.39	50.34
16	50.34	50.32	50.31	50.26	50.43	50.43	50.37	50.34	50.43	50.43	50.41	50.35	50.48	50.44	50.39	50.34
18	50.35	50.33	50.31	50.26	50.44	50.43	50.37	50.34	50.44	50.44	50.42	50.35	50.49	50.45	50.39	50.34
20	50.36	50.33	50.31	50.26	50.45	50.44	50.37	50.34	50.44	50.44	40.42	50.35	50.49	50.45	50.39	50.34

Table A-2 – Typical Heave Test Field Data. D_1, d_2, D_3, and D_4 are at 7.0', 10.0', 13.0', and 16.0' from center of the crater at north, west, south, and east directions.

Table A-3

Diameter and Height Measurements for the Above Heave Test

No. of Drops	Diameter (ft)	Bottom of Crater Elevation	Depth of Crater (ft)
0	0	1250.27	0
2	5.70	1251.03	.76
4	6.01	1252.09	1.82
6	6.50	1254.19	3.92
8	7.10	1256.20	5.93
10	7.50	1257.50	7.23
12	7.60	1258.10	7.83
14	7.90	1258.20	7.93
16	8.00	1258.25	7.98
18	8.05	1258.30	8.03
20	8.07	1258.35	8.08

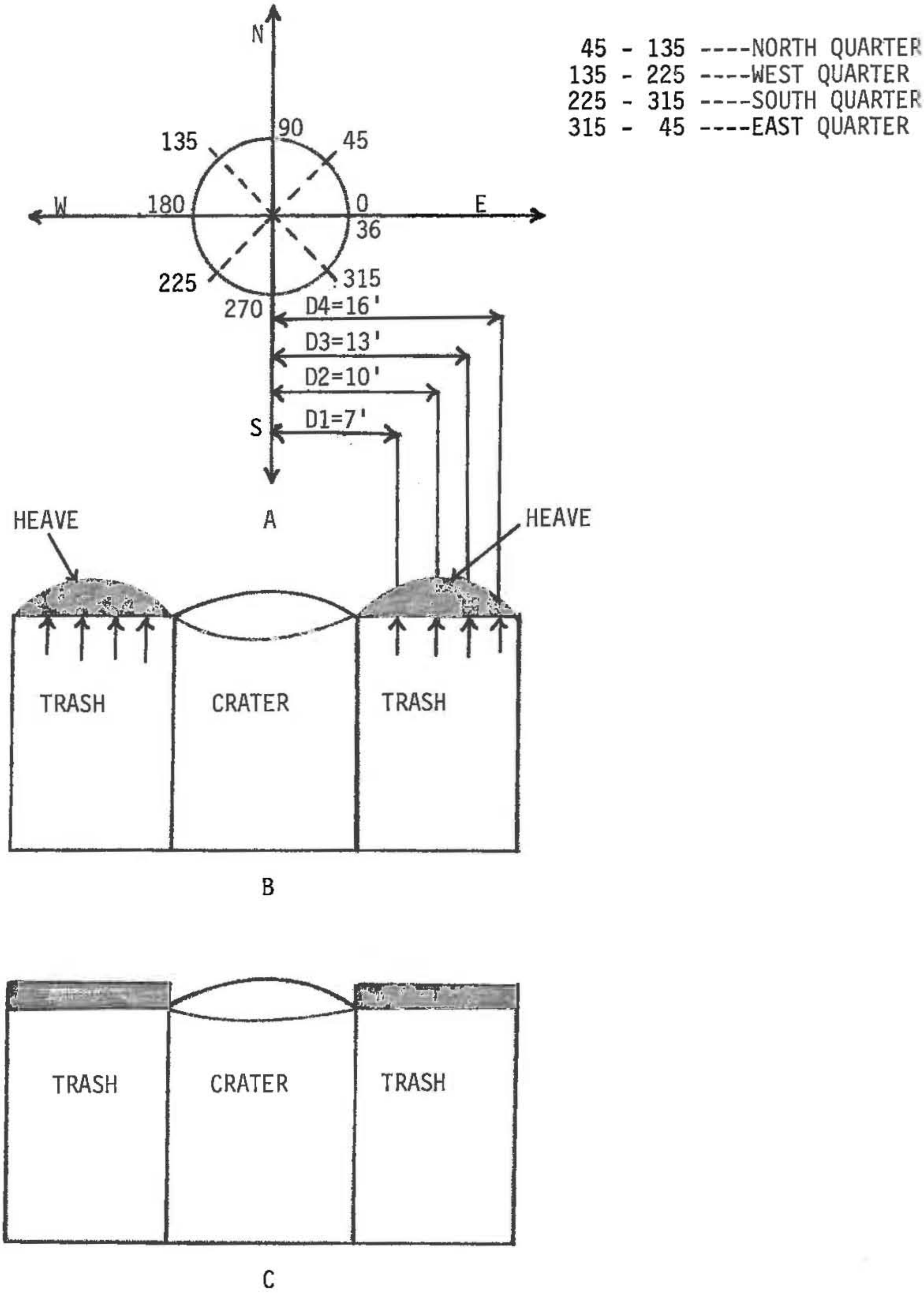


FIGURE A-3 THE TOP THREE FIGURES SHOW THE PLAN AND SIDE VIEWS OF A TYPICAL CRATER IN A DYNAMIC COMPACTION PROCESS. FIGURE C-3C SHOWS THE APPROXIMATED RECTANGULAR CROSS SECTION OF THE HEAVE IN FIGURE C-3B.

6. Calculate the crater volume using the following formula:

$V_H = (Rr_i^2)H$
V_C = Crater volume
H = Depth of crater
r = Radius of crater

7. Subtract heave volume from the crater volume to obtain the net volume (V_N) of the crater.

$V_N = V_C - V_H$

8. Plot the graph of net volume versus number of drops.

9. Estimate the coordinates of the point with 80% maximum curvature. Assume the number of drops for this point is the effective number of drops for the next phase of the Dynamic Compaction.

Figure A-4 shows the plot of the net volume versus number of drops for the data presented in Tables A-2, A-3, and A-4.

A-3 Volume Measurement and Analysis

At the end of each phase of the Dynamic Compaction, the volume of craters are measured and deducted from the heave volumes. There are two advantages for measuring the volumes: (1) the overall results as presented in Figures 43 to 49 show the reduction in crater volumes in various stages of the Dynamic Compaction, and (2) average settlement induced by the Dynamic Compaction process can be calculated by averaging the total measured volume, reduced by the percentage of heave during the heave test and divided by the area concerned. The net enforced settlement is a good approximation of the amount of strain induced in the deep layers.

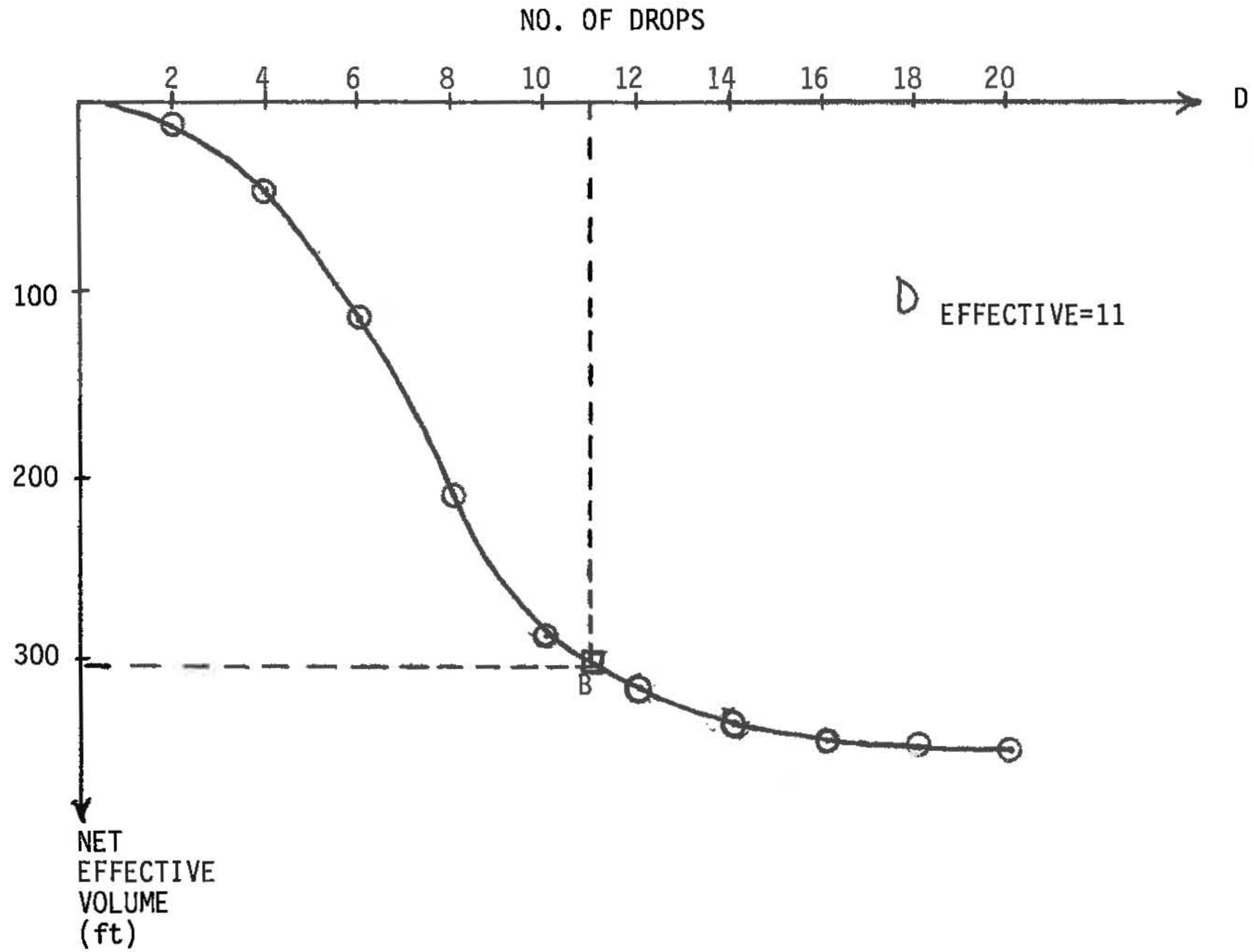


FIGURE A-4 PLOT OF THE FINAL RESULTS FOR A HEAVE TEST. THE X COORDINATE FOR POINT B CONTAINS THE EFFECTIVE NUMBER OF DROPS FOR THE NEXT PHASE OF THE DYNAMIC COMPACTION TREATMENT.

Table A-4

Analysis of Data for a Heave Test

	North			West			South			East			Total Heave Volume	Crater Volume	Volume of Crater
No. of Drops	h (ft)	A_Q (ft)	V_Q (ft)	h (ft)	A_Q (ft)	V_Q (ft)	h (ft)	A_Q (ft)	V_Q (ft)	h (ft)	A_Q (ft)	V_Q (ft)	V_H (ft)	V_C (ft)	V_N (ft)
0	0	162.6	0	0	162.6	0	0	162.6	0	0	162.6	0	0	0	0
2	.015	162.6	2.44	.010	162.6	1.63	.005	162.6	.813	.008	162.6	1.30	6.18	19.39	13.21
4	.022	162.6	3.58	.020	162.6	3.25	.012	162.6	1.95	.012	162.6	1.95	10.73	51.63	40.90
6	.027	162.6	4.39	.025	162.6	4.06	.018	162.6	2.93	.022	162.6	3.58	14.96	130.08	115.12
8	.037	162.6	6.02	.038	162.6	6.18	.030	162.6	4.88	.030	162.6	4.88	21.96	234.78	212.82
10	.050	162.6	8.13	.052	162.6	8.46	.045	162.6	7.32	.045	162.6	7.32	31.23	319.41	288.18
12	.065	162.6	10.57	.060	162.6	9.76	.060	162.6	9.76	.060	162.6	9.76	39.85	355.20	315.35
14	.078	162.6	12.68	.075	162.6	12.20	.090	162.6	14.63	.085	162.6	13.82	53.33	388.70	335.37
16	.090	162.6	14.63	.082	162.6	13.33	.095	162.6	15.45	.092	162.6	14.96	58.37	401.11	342.75
18	.095	162.6	15.48	.085	162.6	13.82	.102	162.6	16.58	.098	162.6	15.93	61.81	408.69	346.88
20	.098	162.6	15.93	.090	162.6	14.63	.102	162.6	16.58	.098	162.6	15.93	63.07	413.28	350.21

To illustrate the above procedures, the data presented in Table A-5 will be analyzed as follows:

Table A-5

Volume and Surface Area Measurements at the End of the Primary Phase in Site A

Crater Number	Volume $(ft)^3$	Surface Area $(ft)^2$
1	304	70.9
2	185	50.3
3	245	56.7
4	280	62.5
5	230	54.5
6	230	54.5
7	230	54.5
8	210	52.5
9	185	50.3
10	160	48.9
11	255	60.9
12	250	60.5
13	290	64.5
14	270	63.1
15	270	63.5
16	210	60.5
17	200	58.5
18	240	59.6
19	320	71.2
20	250	62.0
21	280	64.1
22	200	54.5
23	210	55.0
24	235	57.0
25	255	60.1
26	260	61.1
27	265	62.0
28	305	70.1
29	210	61.0
30	260	62.0
31	280	63.1
32	280	63.9
33	230	56.0
34	310	71.0
35	320	72.0
36	280	62.0
	8994	2174.8

$V_A = 1/36\ (V_1 + V_2 + \ldots\ldots\ldots + V_{36})$

$V_A = 1/36\ (304 + 185 + \ldots\ldots\ldots + 280)$

$V_A = 249.8\ ft^3$

$H_V = 40.1\ ft^3$

$A_A = 1/36\ (A_1 + A_2 + \ldots\ldots\ldots + A_3)$

$A_A = 1/36\ (70.9 + 50.3 + \ldots\ldots\ldots + 62.0)$

$A_A = 60.4$

$V_N = V_A - H_V$

$V_N = 249.8 - 40.1 = 209.7\ ft^3$

$S_A = 209.7 / 60.4 = 3.5\ ft$

V_A = Average volume of craters

H_V = Average heave volume obtained during a heave test

A_A = Average surface area

V_N = Net volume

S_A = Average settlements induced by the Dynamic Compaction

The volume corresponding to the above data are presented in Figure 46.