DYNAMIC COMPACTION FOR HIGHWAY CONSTRUCTION
VOL. I: DESIGN AND CONSTRUCTION GUIDELINES

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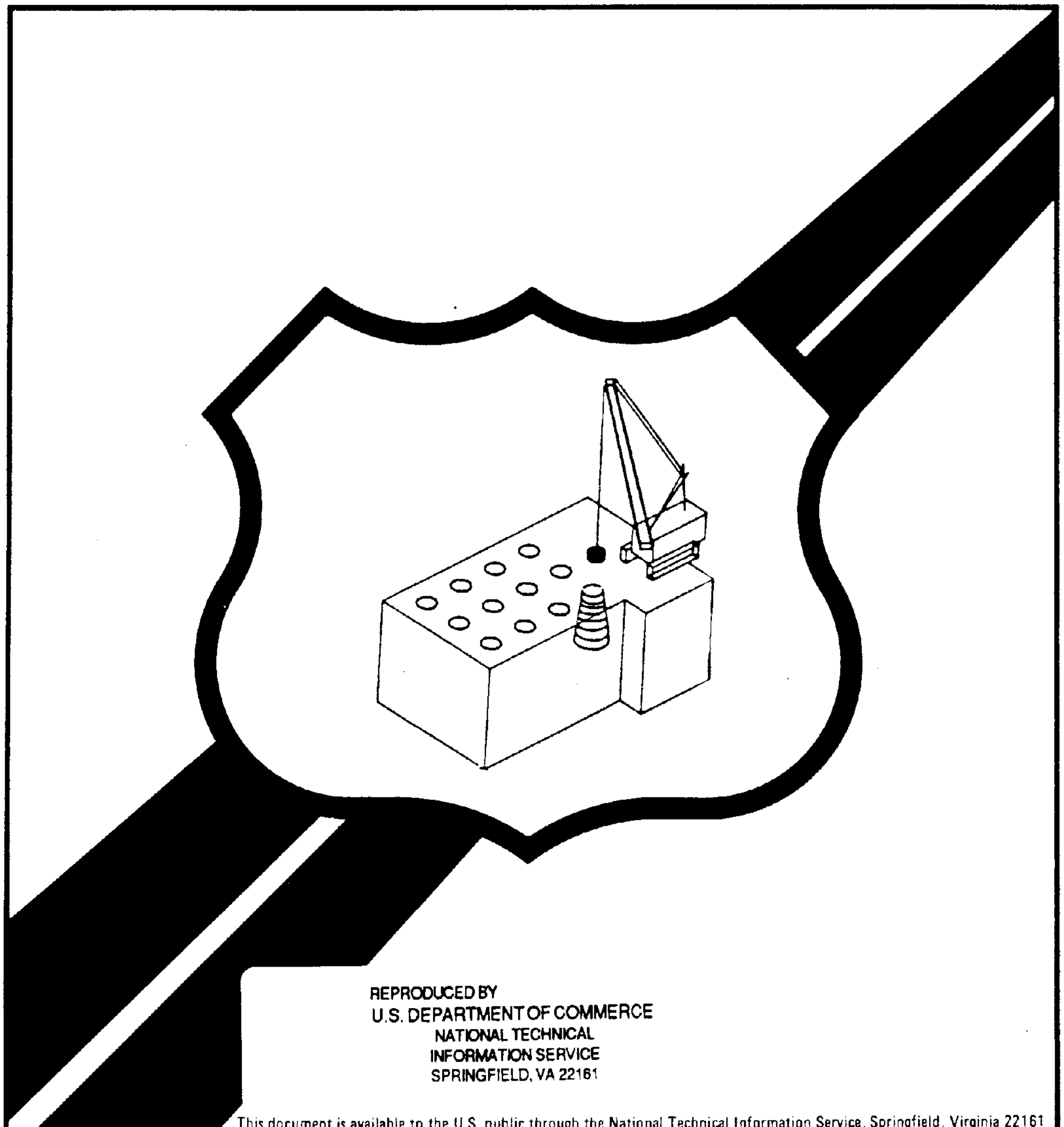

FOREWORD

This report presents the results of a comprehensive investigation [illegible] dynamic compaction to improve embankment and bridge foundation[illegible] construction guidelines for using dynamic compaction as a ground [illegible] technique are presented along with detailed cost data and [illegible] tions. This report will be of interest to bridge engineers, roadway design specialists, construction and geotechnical engineers concerned with [illegible] compaction problems.

Sufficient copies of the report are being distributed by FHWA Bull[illegible] to provide a minimum of two copies to each FHWA regional and division off[illegible] three copies to each State highway agency. Direct distribution is [illegible] made to division offices.

for Richard E. Hay, Director
Office of Engineering [illegible]
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16. Abstract

Dynamic compaction has been found to produce densification in certain natural and fill deposits to depths varying from 10 to 35 ft below grade and has an application for highway construction. This manual, Volume I, presents the "State of the Art" of dynamic compaction. The information contained herein was assembled from published articles, actual job records, interviews with engineers and specialty contractors engaged with dynamic compaction and data obtained from instrumenting three project sites.

Guidelines are presented for: determining the suitability of deposits for dynamic compaction, estimating the depth and degree of improvement, planning the spacing between drop points, estimating the required unit applied energy, monitoring the improvement and predicting offsite ground vibrations. In addition, non-technical aspects of dynamic compaction are discussed including equipment requirements, typical costs, and methods for negotiating a contract including specification guidelines.

Volume 2 contains detailed technical data for the three dynamic compaction projects that were instrumented as part of this study to provide additional information to aid in the preparation of Volume 1.

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CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	by	To Obtain
bars	1.02	tons per square foot
inches	2.54	centimeters
feet	0.305	meters
ton feet per square foot	2.982	tonne meters per square meter
square feet	0.093	square meters
cubic feet	0.028	cubic meters
cubic yards	0.764	cubic meters
tons (2000 pounds)	1.120	tonnes
pounds (mass) per cubic foot	0.016	megagrams per cubic meter
pounds (mass) per cubic yard	0.593	kilograms per cubic meter
pounds (force)	4.448	newtons
tons (force) per square foot	95.8	kilonewtons per square meter

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LIST OF SYMBOLS AND ABBREVIATIONS

H = Drop Height of Tamper

W = Weight of Tamper

V = Velocity of Falling Tamper

D = Depth of Improvement

n = Empirical Coefficient that Reduces the Depth of Improvement As a Result of Soil and Equipment Factors

N = Number of Drops of a Tamper

E_r = Kinetic Energy

m = Mass of Tamper

P_l = Limit Pressure Which is Considered the Failure Pressure Reached During the Performance of a Pressuremeter Test

qc = Cone Tip Resistance

R_u = Ultimate Capacity of the Soil

R_d = Dynamic Component of Soil Resistance

F = Inertia Force

SPT = Standard Penetration Test Performed in Accordance with AASHTO-206

CPT = Cone Penetration Test

PMT = Pressuremeter Test

txm/m^2 = Ton Meters Per Meter Squared

CHAPTER 1 - INTRODUCTION

1. Purpose and Scope of Manual

Weak or poor ground support conditions for roadways and other developments are present throughout many areas of the country. These weak deposits may be natural or man-made. Natural deposits include loose soils deposited in flood plains or wind blown deposits. Man-made deposits consist of deep fills frequently placed in old quarries or clay pits, mine spoil, building rubble deposited in former basements of structures, and landfill, both recent and old, frequently placed in low lying areas. In the past, these sites have been avoided because they have not been economically justified for development. However, in and near urban areas these parcels of land are frequently situated in choice locations or are the only locations where highways must be extended or structures built. In order to keep foundation costs to a minimum, ground stabilization techniques have been developed to improve subgrade support conditions. Dynamic compaction is one of these ground stabilization techniques.

The purpose of this manual is to familiarize the reader with the basic fundamentals of dynamic compaction. The guidelines presented herein were prepared from information presented in published articles, personal interviews with specialists involved with the dynamic compaction process and first hand experience with dynamic compaction projects.

As with any developing technique, not all the principles of the dynamic compaction process or the influences of certain construction techniques are completely understood. In addition, some variables that are believed to affect the degree or depth of densification cannot be directly taken into account. For this reason, the information contained in this manual should be considered as a guideline to aid in planning and implementing dynamic compaction. However, adjustments should be made at each project site to account for local conditions.

This manual discusses not only the technical aspects of dynamic compaction but also addresses the type of equipment required to achieve the goals, the methods of negotiating for services, and the current construction costs. All of these items have a bearing on the decision on whether to use dynamic compaction on any given project.

2. Definition of Dynamic Compaction

Dynamic compaction is defined as the densification of soil deposits by means of repeatedly dropping a heavy weight onto the ground surface. This process has also been called by other names including heavy tamping, impact densification, dynamic consolidation, pounding, and dynamic precompression. Most dynamic compaction is undertaken with weights ranging from 6 to 30 tons (5.4 to 27.2 t) although weights as light as 2 tons (1.8 t) or as heavy as 100 tons (90.7 t) have been used. The drop heights generally range from 30 to 75 ft (9.1 to 22.9 m) but projects have been undertaken with drop heights as low as 15 ft (4.6 m) and as high as 120 ft (36.6 m).

For weights up to 20 tons (18.1 t) and drop heights up to 100 ft (30.5 m), the weight is generally lifted and dropped by a conventional crane using a single cable with a free spool to allow a near free fall drop. Figure 1 shows a 125 ton (113.5 t) capacity crawler crane lifting a 16.5 ton (15 t) weight to a height of 65.6 ft (20 m). Figure 2 shows a 6 ton (5.4 t) weight being raised for a 40 ft (12.2 m) drop. For weights greater than 20 tons (18.1 t), either the conventional equipment is altered to reinforce certain components or specially designed equipment is used to lift and drop the weight.

Densification of the deposits is achieved because sufficient energy is applied to the ground to cause one or more of the following effects to occur:

- Densification of partially saturated soil in a manner similar to that which occurs when performing laboratory impact compaction by the Proctor method.

- Restructuring of the soil grains into a denser packing at a lower water content. In saturated or nearly saturated soils, excess pore water pressures develop on impact, and occasionally the soils liquefy. Following dissipation of the pore water pressures, the properties of the soil improve.

- Collapse of voids within the soil deposit that have been formed as a result of bridging or other mechanisms. In this case the void would be an opening other than the normal void space between soil particles.

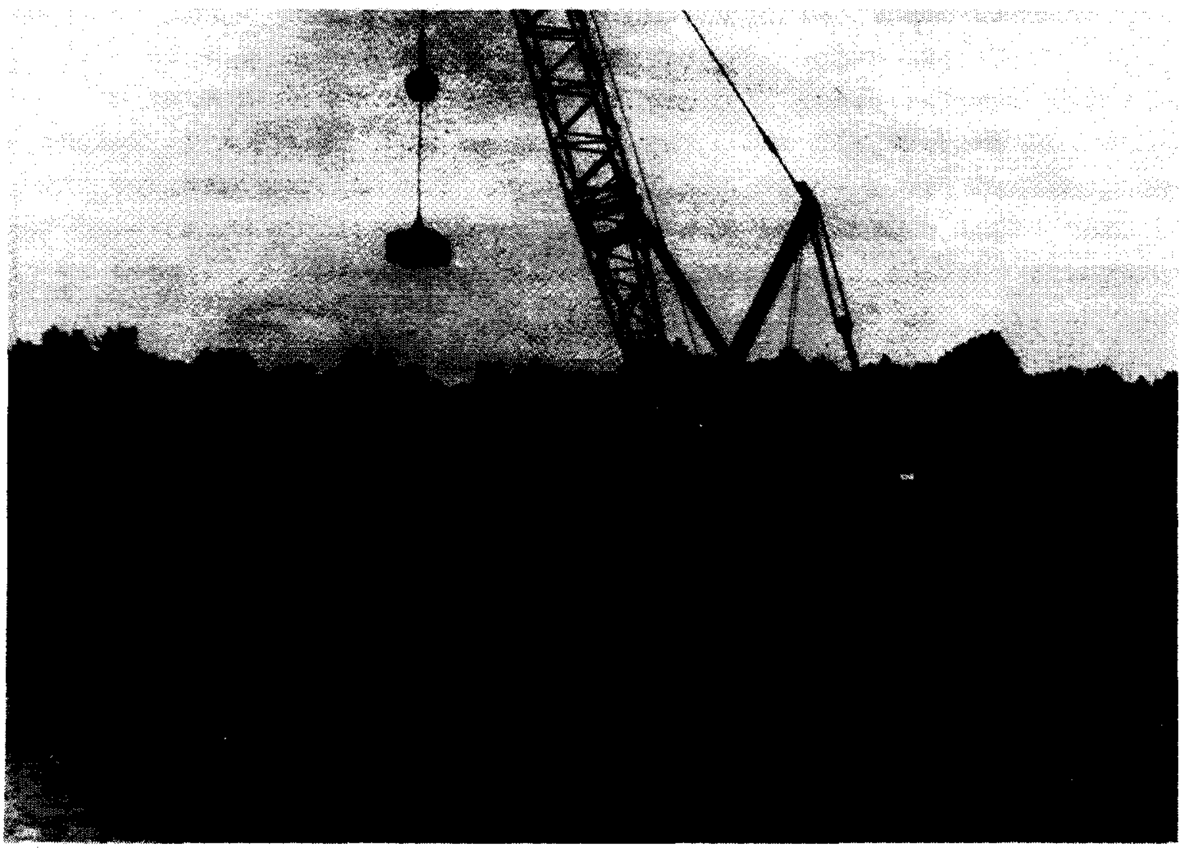


Figure 1 : A 125 ton crawler crane lifting a 15 - tonne weight to a height of 20 meters

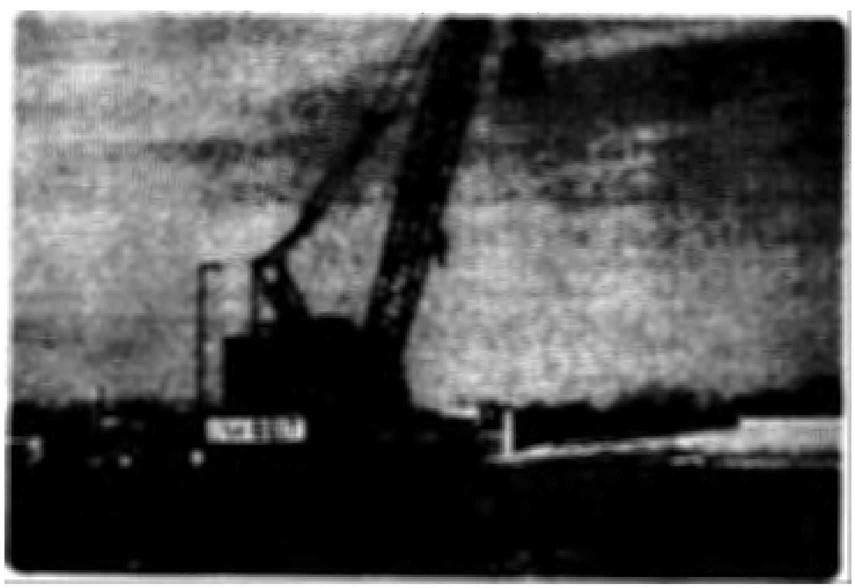


Figure 2 : A 50 ton crawler crane lifting a 6 - ton weight to a height of 40 feet

The type of densification that occurs on any project depends upon the deposit and the degree of saturation of the deposit. In partially saturated soil deposits situated above the water table, densification is due primarily to compaction. The energy imparted into the ground causes the particles to move closer together thereby resulting in an increase in unit weight. This in turn results in an increase in strength and a reduction in compressibility.

In saturated soils, the energy applied to the soil causes an immediate increase in pore water pressure and a reduction in effective stress. As the excess pore pressure dissipates the soil particles move into a denser state of packing under the confining pressures of the overburden. In soils of high permeability such as sand and gravels, the excess pore water pressure usually dissipates within minutes after impact. However, in semi-pervious deposits such as silty soils, the time required for excess pore pressure dissipation can range from days to weeks, depending upon the permeability of the deposit and the length of drainage path. On some projects artificial drains such as wick drains have been installed to decrease the drainage path lengths in the fine grained saturated soil deposits.

In addition to soil property improvements resulting from a decrease in void ratio, large lateral strains are induced in the soil mass adjacent to the impact points resulting in an increase in the coefficient of horizontal earth pressure and soil modulus. In effect, creation of a highly densified upper layer following dynamic compaction acts as a mat to help distribute the stresses transferred to the underlying deposit much the same way as a subbase layer distributed load beneath a pavement. In the case of a dynamically compacted soil, the stiffened upper layer can be as much as 10 to 15 ft (3.0 to 4.6 m) in thickness, as opposed to a normal subbase of only 12 to 24 in. (30.5 to 61.0 cm) in thickness.

There are significant differences between dynamic compaction and conventional fill compaction that are indicated below to help illustrate dynamic compaction.

- In conventional compaction, the soil is placed at a water content near the optimum for compaction as determined by AASHTO T99 or T180 and then placed in thin lifts usually less than 12 in. (30.5 cm) in thickness and compacted to a desired density. In dynamic compaction, the deposits are compacted throughout the entire thickness from ground surface at their prevailing water content.

- Dynamic compaction is effective both above and below the ground water table although to reduce construction difficulties the ground surface should be maintained at least 6.6 ft (2 m) above the water table either by raising the grade or dewatering. In conventional compaction, the work is always undertaken above the water table.

- The depth limitation of conventional compaction is generally be on the order of 2 ft (.6 m), although large vibrating compactors have produced densification to 6 ft (1.8 m) depth in granular soils. With dynamic compaction, improvements to depths of 15 to 30 ft (4.6 to 9.1 m) are common.

- Dynamic compaction has been used on deposits containing large particles such as broken concrete or boulders. Conventional compaction is restricted to particle sizes smaller than about 6 in. (15.2 cm).

- During dynamic compaction in saturated soils, especially fine sands or silts, high excess pore water pressures develop. These may be sufficiently high to cause water to boil or emerge from the ground until the pore pressures decrease. The soil structure rearranges into a denser state of packing after the dissipation of the pore pressures. This type of densification is similar to that which occurs by vibroflotation or blasting rather than by conventional compaction[65].

3. Historical Development of Dynamic Compaction

Densification of loose deposits by means of heavy weight impacting into the soil is not a new development. In an article written in 1812 and reprinted in the early 1830's, in a German book of 5 volumes "Kunst Zu Baun", a Frenchman, Rondelet, describes a soil compaction technique using a falling weight. The soil is compacted until the imprints are less than the maximum settlements Rondelet would accept for building loads of corresponding magnitude. He used height of fall of up to 65.6 ft (20 m) and hammers of 1.1 to 2.2 tons (1 to 2 t). Thus, on the assumption that the imprint on the soil from a falling weight, with a particular impact load per unit area, is equal to the settlement of a foundation with the same static load per unit area, Rondelet presented tables for use in foundation design.

Dr. Wilhelm Loos[49] evaluated the effectiveness of different methods for compacting cohesionless soils in Germany. One of the processes that was evaluated was consolidation by the impact of heavy steel plates. The process is described as follows: "In 1933 when compaction was called for on many jobs, only the so-called universal units were available. A cast iron plate of about 2 tons (1.8 t) was fastened on a rope of a steam shovel and the impact of its fall from 5.0 ft (1.5 m) utilized it for compaction." In the course of the investigation, the effect of different weights were tested. An increase of the weight above 5.0 tons (4.5 t) did not result in increased compaction. Apparently the force of the impact disturbed and loosened the soil.

The Corps of Engineers experimented with heavy tamping at the Franklin Falls Dam construction site in 1936[21]. The drop height was only 18 in. (45.7 cm) and the weight was just over 1 ton (.91 t). The tamping was undertaken to densify 4 ft (1.2 m) thick test embankments. The densification was found to be fairly uniform throughout this depth, but the degree of compaction was less than desired.

The Port of Dublin has been using heavy tamping to compact filled ground since the early 1950's, although little published information is available. The weights being used are reinforced concrete with a steel base plate, weighing 3.3 tons (3 t) and 3.3 ft^2 ($1m^2$) base area. Drop heights vary from 15 to 20 ft (4.5 to 6 m). Tamping continues until no further settlement can be achieved. The fill material consists of dumped domestic refuse and building and industrial waste about 16 ft (5 m) deep. The effectiveness of compaction has been evaluated by penetrometer tests which indicate an average increase in resistance of penetration of over 100%.

Dynamic compaction was used in Durban, South Africa in 1955. Hobbs[43], describes compaction of a loose hydraulic fill to support a 250 ft (76.2 m) diameter crude oil tank. A 6 foot (1.8 m) concrete cube was dropped a distance of 12 to 15 ft (3.7 to 4.6 m) over the entire tank area resulting in 13 in. (30 cm) of immediate ground settlement. Improvements in soil properties were reported and the tank was constructed on the densified ground. The tank reportedly settled just over one inch during water testing.

In 1963, Bobylev[8] reported on 46 experiments carried out on an embankment of silty loam. The weights varied from 0.6 to 2.8 tons (0.5 to 2.5 t), and drop heights from 1.6 to 8.2 ft (0.5 to 2.5 m). Measurements of surface stress showed that for lower energy levels, the height of drop is more influential than the mass of the weight. By increasing the energy input either in terms of energy per blow or number of blows, the surface stress is increased to a maximum.

Pryanik[76] reported on field trials using a 4.4 ton (4 t) weight dropping 11.5 to 13 ft (3.5 to 4 m) on a natural clay 4.9 ft (1.5 m) thick overlying a compressible loess 4.9 ft (1.5 m) thick. Compaction of the clay was achieved with 6 drops if wetted, compared to 15 drops at the natural water content. The clay had a natural water content below the plastic limit. The depth of compaction was found to increase with additional blows.

Zakharenkov and Marchuk[103] also carried out tests on loessial sandy loams using a 2.8 ton (2.5 t) weight dropped from 20 ft (6 m). He concluded that the improvements occurred to a depth of 6.6 ft (2 m) using 6 to 8 drops. At a higher number of drops, further compaction occurred in the 6.6 ft (2 m) layer without any improvement in the soils below 6.6 ft (2 m).

It appears that the Russians had a code of practice for surface compaction by heavy tampers as early as 1958.

While densification of deposits by impacting had been undertaken in the earlier years, the development and promotion of this heavy tamping technique for large areas and a wide range of soil types is attributed to Louis Menard[61]. Beginning in 1969 in Europe, Menard used weights of 8.8 to 11 tons (8 to 10 t) dropped from a height of 26.2 to 39.4 ft (8 to 12 m) to densify fill deposits. This process was called "heavy tamping". It was generally used on good quality fill deposits such as rock waste, rubble, and sand. After a few years, the amount of energy delivered to the ground was increased by using weights ranging from 13 to 220 tons (12 to 200 t) and drop heights ranging from 59 to 121 ft (18 to 37 m). Heavy tamping was introduced into Canada in 1972 and the United Kingdom in 1973. Initially, a licensing arrangement was made between Menard and local contractors for undertaking this work under the supervision of the Menard organization. In 1983, the licensing arrangements were terminated.

In the United States, densification by tamping was undertaken on a continuing basis starting in 1971 by local contractors under the engineering direction of STS Consultants, Ltd. Weights in the range of 2 to 6 tons (1.8 to 5.4 t) were dropped through distances of 20 to 35 ft to densify loose fill deposits to support lightly loaded structures[51, 52]. This technique was called, "pounding". Later, weights up to 15 tons (13.6 t) were used to densify former landfills[87]. The Menard organization opened an office in the United States and completed their first job in 1974. Other specialty contractors began doing dynamic compaction in the United States around 1980.

In 1972, Menard proposed the use of this technique for densifying fine grained saturated soils and renamed this process, "Dynamic Consolidation". A U.S. patent applied for in May, 1973 was obtained in August, 1975 for the specific purpose of consolidating clays, silts, or clays mixed with sand. The Dynamic Consolidation process is described in the patent as "a process which fluidizes the soil while exerting dynamic forces so that the material behaves like a suspension of particles in water. Following pore water dissipation the material is restructured to a denser condition." Since this patent applies specifically to densification of saturated clayey soils, it is not generally considered pertinent for most sites where dynamic compaction is used. Chapter 2 of this document discusses deposits suitable for dynamic compaction and saturated clayey soils are excluded.

At the present time, there are approximately four specialty contractors that do dynamic compaction on a regular basis in the United States. There are also numerous non-specialty contractors who have undertaken dynamic compaction working under the direction of a supervising engineer. All of this work has been completed without consideration of the Menard patent and no patent infringement suits have been filed.

4. Advantages and Disadvantages of Dynamic Compaction

Dynamic compaction is becoming increasingly popular because it offers certain unique technical and economic advantages. These include but are not limited to:

- The equipment required to undertake dynamic compaction is relatively simple and consists primarily of a weight and a standard crane. However, cable and drum wear is higher than normal.

- Impacting of the weight into the soil serves as both a probing and a correcting tool. If weak ground conditions are present in localized areas, the weight will penetrate further into the ground causing large crater depths. This provides the field engineer with immediate feedback on ground response. A decision can then be made regarding further energy application in this area to either correct the poor ground condition or undercut and remove the poor ground if it will not compact.

- The effect of densification can be observed as the work is proceeding. This ground response can be used as a guide for the field personnel. The depth of the initial craters plus decreasing depth with successive passes is an indication of the resistance of the ground. Also, the average ground settlement which takes place following each pass over an area provides an indication of the overall amount of improvement being achieved.

- Dynamic compaction can be applied over a fairly wide range of deposits ranging from large boulders and broken concrete to silt size formations containing a small percentage of clay. Deposits that formerly were thought uncompactable or not controllable, such as building rubble debris or decomposed sanitary landfills, can be compacted by this process.

- Densification usually results in a stratum having a more nearly uniform compressibility, thereby minimizing differential settlements. Weaker zones within the deposit undergo the most improvement thereby eliminating zones of potentially high compressibility.

- The costs of dynamic compaction are generally significantly less than other forms of site improvement or alternate forms of construction such as excavation and replacement using conventional compaction equipment. The cost savings will vary on each job depending upon site conditions and the depth of improvement required.

- Densification can be attained below the water table in pervious and semi-pervious deposits, thereby eliminating costly dewatering and/or lateral bracing systems that would be required for conventional excavation and replacement techniques.

- Dynamic compaction of pervious deposits can be undertaken in rainy weather and can even be done with a limited amount of frost in the ground.

Some of the disadvantages of dynamic compaction are:

- Heavy tamping produces ground vibrations which can travel significant distances from the point of impact. In congested areas this may require limiting the dynamic compaction to areas well within the property lines, reducing the drop heights, or trenching to reduce vibrations.

- The position of the water table has an influence on the construction operations. Ideally, the water table should be greater than 6.6 ft (2 m) below ground surface. It may be necessary to lower the water table or raise the grade prior to dynamic compaction to achieve the 6.6 ft (2 m) distance between the working surface and the water table.

- In very loose deposits such as recent landfills, it is often necessary to place a mat of gravel or crushed stone at the surface to provide a working platform for operation of the equipment to limit penetration of the weight at impact, and to provide confinement for the underlying weak deposits. In addition, large settlements usually occur following dynamic compaction in these deposits thereby requiring additional granular fill to be brought in as the work is underway. The cost of importing the granular fill can be about as much as the cost of the dynamic compaction.

- Lateral ground displacements of 1 to 3 inches (2.5 to 7.6 cm) have been measured at distances of 20 ft (6.1 m) from the point of impact of 16.5 to 33.1 tons (15 to 30 t) tampers. Utilities or buried vessels situated within the zone of influence could be displaced or damaged.

CHAPTER 2 - DEPOSITS SUITABLE FOR DYNAMIC COMPACTION

1. General Comments

Dynamic compaction has been undertaken on many types of deposits ranging from coarse to fine grained soils including natural soils of a relatively homogeneous nature to heterogeneous fills. At some locations the soils were partially cemented such as collapsible soils in the Western United States, while at other sites fill deposits were in such a loose condition that the fill was still consolidating under its own weight when dynamic compaction was utilized. The position of the water table has varied from close to ground surface to relatively great depths below grade for projects undertaken on land. Dynamic compaction has also been undertaken below water for offshore projects using a specially adapted tamper. In most cases, dynamic compaction was considered successful to varying depths although in a few instances, ground improvement was not achieved.

Projects where dynamic compaction has been most successful include sites where coarse grained pervious soils were present, either above or below the ground water level. Previous soils includes sands, gravels, cobbles and combinations of these soils. Projects where dynamic compaction has not been successful or where improvement was so minor that the expenditure may not have been worth the cost were sites comprised of saturated impervious deposits such as clays. Between these two broad limits, there is an intermediate zone in which the soils can be classified as semi-pervious deposits such as silty soils where dynamic compaction has been successful but with some difficulty. Difficulties included the need for applying additional amounts of energy, controlling the ground water, or allowing idle periods between energy application. On a broad basis dynamic compaction works well above the ground water level in most soils ranging from the coarse grained deposits to partially saturated fine grained fill deposits. Below the ground water levels, dynamic compaction works well for coarse grained soils, is not appropriate for the impervious deposits, and works well under controlled conditions in the semi-pervious deposits.

The key site related factors that influence the effectiveness of dynamic compaction are:

- Classification and geologic origin of the soil mass.
- The degree of saturation of the deposit.
- The permeability of the soil mass and length of drainage paths which control the speed at which excess pore pressures can dissipate.

The effects of these factors on the coarse grained pervious deposits, fine grained semi-pervious deposits, and saturated and partially saturated fine grained impervious deposits, is discussed in more detail in the following sections.

a. Coarse Grained Pervious Deposits

The reason dynamic compaction works so well in these deposits is that the energy from the tamper causes the individual soil particles to move almost immediately into a denser state of packing. Above the water table, the soil particles move into a denser state of packing similar to that which occurs from Proctor compaction. Below the water table, densification occurs because the pore pressures generated during tamping dissipate almost immediately or within a short period of time following impact. Thus, good interparticle contact and interlocking is achieved as a result of the densification.

b. Semi-Pervious Deposits

In the semi-pervious deposits that are saturated or nearly saturated, excess pore water pressures are generated following each blow of the tamper. Some period of time, frequently on the order of minutes to days is required for the excess pore water pressures to dissipate. Because of this, the energy that is applied is only partially effective in densifying the soil. If additional blows are applied before the excess pore water pressures dissipate, the energy is transmitted through the soil mass without causing densification. The deposit often behaves in a spongy fashion and ground heave occurs. When the volume of ground heave that occurs with each blow is equal to the volume of the additional induced crater depth, the soils are being displaced in a plastic fashion without densification.

In semi-pervious saturated deposits, the total energy should be applied using multiple passes and a relatively wide grid spacing. On some projects, the excess pore pressures are sufficiently high to cause liquefaction of the soil. Sand or silt boils can then form at the ground surface, Figures 3 and 4. During this time, the deposits are in a loose and unstable condition and mobility of construction equipment on the surface is difficult. However, after a period of time, the excess pore pressures dissipate and the deposits assume a denser condition than the initial condition.

In partially saturated semi-pervious soils, densification can be achieved with less difficulty, depending upon the degree of saturation. Usually, as the soils become more dense, the degree of saturation increases until they become or behave as fully saturated soils. When this occurs, the problems as discussed in the previous paragraph develop.

For saturated or unsaturated semi-pervious deposits, significant improvements can be achieved, but the field operations must be carefully controlled to allow excess pore pressure dissipation

Figure 3 : Sand boil formed from dissipation of pore water pressure in a silty sand

Figure 4 : Numerous sand boils that developed one day after dynamic compaction in fine sand

between tamps. Usually, this requires the installation of pore pressure monitoring devices, multiple passes over the area and wide spacing between impact points.

c. Saturated Impervious Deposits

Dynamic compaction to improve the properties of saturated impervious deposits is generally not effective. Excess pore pressures which develop under impact cannot be rapidly relieved because of the low permeability of these deposits and frequently long drainage paths. Numerous investigators have experimented with dynamic compaction of clayey soils where the water table was near the surface of the clay and the results have been marginal, at best.

Charles and Watts[15] describe a project where a 8.2 ft (2.5 m) thick layer of alluvial silty clay was dynamically compacted for a roadway embankment. The water table was at the surface of the deposit. The alluvial deposit had liquid limits in the range of 40% to 70% with the plasticity indices between 20% and 45%. The average water content at the time of densification was about 35%. To improve the drainage, a 6.6 ft (2 m) thick granular surcharge was placed on top of the clay and granular fill was placed in drainage trenches located on a 19.7 by 49.2 ft (6 by 15 m) grid. Dynamic compaction was applied with a 16.5 ton (15 t) weight and a 65.6 ft (20 m) drop. While water was observed to be expelled from the drainage trenches, excess pore pressures up to 2 meters of head remained two months after tamping. The purpose of the dynamic compaction was to increase the undrained shear strength. However, unconsolidated-undrained triaxial tests on samples taken before and after tamping showed an increase of only 36% in the shear strength. Charles and Watts concluded that this increase in shear strength probably was produced by preloading with the 6.6 ft (2 m) thick granular surcharge and stated that "It was not self evident that dynamic consolidation is an effective method of improving the engineering properties of saturated fine grained soils". They believe that for soft alluvial soils, preloading combined with improved drainage would seem the obvious approach to ground improvement.

Crossley[20] reports on a project where dynamic compaction was used to densify granular fill deposits. During the densification, a large mass of soft, saturated, cohesive material was encountered that did not respond to dynamic compaction. This cohesive deposit had liquid limits of 27% to 28% with plasticity indices of 11% to 12%. The natural water content was close to the liquid limit and under impact, the material liquefied. Eventually, the cohesive soil was excavated and replaced with a well graded granular deposit which was successfully densified by tamping.

Choa, et.al.[18] report on the use of dynamic compaction on a marine clay with the ground water above the surface of the clay. A 21.3 ft (6.5 m) granular fill deposit was placed on top of the clay and tamped with a 38.6 ton (35 t) weight and a 131 ft (40 m) drop. The clay had liquid limits ranging from 60% to 80% and a plasticity index range of 40% to 50%. The natural water contents of the clay were 50% to 60%. Only a slight improvement was observed within the upper portion of the clay. Choa, et.al. concluded that dynamic compaction was not effective for the marine clay.

Some investigators; i.e., Ramaswamy, et.al.[77], Thompson and Herbert[92], and Floss[29], report slight improvements in natural clays below the water table, but factors other than dynamic compaction probably influenced the results. Intermixing of the surface granular layer during tamping, or partial consolidation of the deposit under the weight of granular surcharge may have resulted in the property improvements. The degree of improvement in shear strength was on the order of 150% to 200%, but since the original shear strength was less than 520 psf (25 kN/m^2), the magnitude of improvement was small. Thus, the improved ground still is weak and compressible. Gambin[31], states that improvement in natural clays is generally minor so only flexible structures such as embankments should be constructed over these deposits.

Menard and Broise[61] postulated that pore water dissipation occurs even in fine grained deposits without artificial drainage paths because air bubbles within the voids of the soil form cleavage planes following impact allowing pore water pressures to dissipate. This phenomenon may have occurred at some specialized sites where Menard did the testing, but as reported by the investigators in the previous paragraph, there is sufficient evidence to conclude that this occurrance is not widespread.

d. Partially Saturated Impervious Deposits

Under proper circumstances, modest improvements in shear strength and a reduction in compressibility can be attained in partially saturated impervious soils. Clay fills situated above the water table have been improved by dynamic compaction when the water content of the clay was near or below the plastic limit. Where this is the case, the densification results from moving the clay lumps or particles within the fill mass into a tighter state of packing thereby reducing the void ratio. The densification takes place before significant excess pore pressures develop.

Thompson and Herbert[92] report on three projects where dynamic compaction was used on clay fill deposits. At one site, the improvement consisted mainly of developing a more homogeneous fill. At other sites, significant improvements in the shear strength were achieved. Charles, et.al.[13] report on a 79 ft (24 m) deep cohesive waste deposit which was densified by impacting with a 16.5 ton (15 t) weight and a 66 ft (20 m) drop. The water table was near the bottom of the fill. The natural water content of the fill ranged from 7% to 28% with a mean of 18%. The average liquid limit was 28% and the average plasticity index 11%. Densification was achieved to a depth of 6 meters below the surface.

Moseley and Slocome[66] used dynamic compaction on a 20 ft (6 m) thick clay fill where the water table was below the fill. The liquid limit of the clay was 27% to 37%, and the plasticity index ranged from 11% to 19%. The natural water content was 14% to 18%. Pressuremeter tests indicated an increase in limit pressure by a factor of 2.5 and an increase in pressuremeter modulus by factors ranging from 2 to 4.

Lukas[55] reports on dynamic compaction of a mine spoil fill deposit consisting of residual clay and weathered shale particles. Typically, the liquid limit of the reworked clay and weathered shale was on the order of 30% to 40%, and the plastic limit was on the order of 20% to 25%. The natural water content was found to range from 15% to 20%. The water table was 32 ft (9.75 m) below ground surface. Dynamic compaction was undertaken with an 18 ton (16.3 t) weight dropped from a height of 75 ft (22.9 m). The depth of improvement, as measured by increase in Standard Penetration Resistance Values, pressuremeter tests, and Dutch cone tests was found to be 25 ft (7.7 m).

2. Suitability of Deposits for Dynamic Compaction

The important parameters of the soil that affect the suitability for dynamic compaction have been discussed in the previous section. These parameters include the soil classification, degree of saturation, and permeability plus length of drainage paths.

On typical projects, two of these parameters can be determined by conventional sampling and testing; soil classification plus degree of saturation. The third parameter, permeability and length of drainage path, is rarely determined. On some projects such as sanitary landfills where classification is difficult, only one parameter, the degree of saturation, is established. Even this parameter, is only determined to the extent of establishing the position of the water table from which one can assume the soils are fully saturated below and only partially saturated above.

Fortunately, the permeability of the more conventional soil deposits can be estimated from the soil classification and on most projects, the permeability has been inferred by this means.

Table 1 has been prepared to indicate the suitability of various deposits for dynamic compaction based upon the classification of the materials, the degree of saturation of the deposit, and the inferred or measured permeability and drainage of the deposit. This rating system is somewhat subjective and should be used only as a guide. Local subsurface conditions or techniques could produce results somewhat different than anticipated. In areas where there is doubt as to the usefulness or suitability of dynamic compaction, test sections should be undertaken to further define the degree of improvement that can be achieved. Test sections are discussed in Chapter 4.

The first four categories described on Table 1, fall into the broad category of deposits that can be classified by conventional index tests. These four categories are described in more detail in the following section. This is followed by a discussion of recent landfill and organic soil deposits where conventional index tests are not appropriate for identification.

a. Deposits That Can Be Classified By Conventional Index Tests

Most natural soil or fill deposits can be characterized or identified on the basis of conventional soil mechanics index tests including grain size gradation and/or Atterberg limits. This would include natural or fill deposits consisting of various proportions of cobbles, gravel, sand, silt, or clay. Fill deposits of building rubble, decomposed landfill, mine spoil, crushed concrete or flyash would also be included. On a typical grain size gradation chart, Figure 5, three gradation zones are shown. This figure is a supplement to Table 1. Zone 1 on Figure 5 represents the gradation range where dynamic compaction is the

TABLE 1

Suitability of Deposits for Dynamic Compaction

General Soil Type	Most Likely Fill Class.	Most Likely AASHTO Soil Type	Degree of Saturation	Suitability for D.C.
Pervious deposits in the grain size range of boulders to sand with 0% passing the #200 sieve Coarse Portion of Zone 1*	Building Rubble Boulders Broken Concrete	A-1-a A-1-b A-3	High or Low	Excellent
Pervious deposits containing not more than 35% silt Fine portion of Zone 1*	Decomposed Landfills	A-1-6 A-2-4 A-2-5	High	Good
			Low	Excellent
Semi-Pervious soil deposits, generally silty soils containing some sand but less than 25% clay with Pl<8 Zone 2*	Flyash	A-5	High	Fair
	Mine Spoil		Low	Good
Impervious soil deposits, generallyclayey soils where Pl>8 Zone 3*	Clay Fill Mine Spoil	A-6 A-7-5 A-7-6 A-2-6	High	Not recommended
			Low	Fair - minor improvements - water content should be less than plastic limit
Miscellaneous fill including paper, organic deposits, metal and wood	Recent Municipal Landfill	None	Low	Fair - long term settlement anticipated due to decomposition Limit use to embankments.
Highly organic deposits peat-organic silts		None	High	Not recommended unless sufficient granular fill added and energy applied to mix granular with organic

*These zones are identified on Figure 5

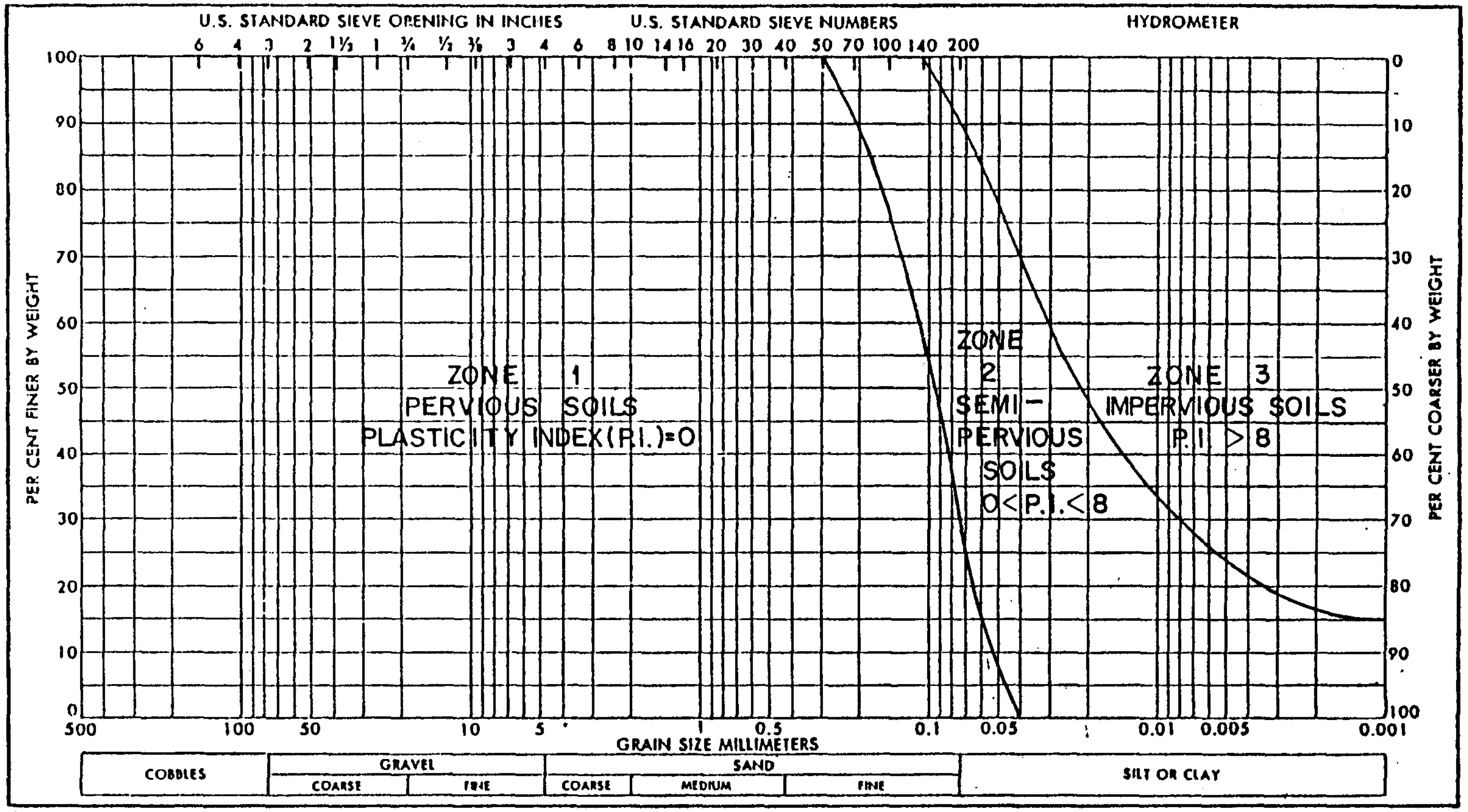


Figure 5 : Grouping of soils for dynamic compaction

most appropriate. Zone 3 is the gradation range where dynamic compaction is not recommended when these deposits are fully or nearly fully saturated. Zone 2 is a transition range where dynamic compaction will work but multiple passes are required to allow excess pore pressures to dissipate before more energy is applied. A more thorough discussion of each of these gradation zones follows.

(1) Zone 1: The most suitable deposits for dynamic compaction are pervious soils ranging from boulders to the sand sizes. Sandy silts or silts with no clay size particles and a plasticity index of 0 are also included in this category. The permeability of these deposits is sufficiently high so that even with a high water table, excess pore water pressures following tamping generally dissipate relatively quickly. Typically, the coefficient of permeability in Zone 1 is above 2×10^{-3} ft/min (1×10^{-3} cm/sec). Within the coarse range of this zone, densification is immediate. In the finer size range of the zone, several hours may be required for excess pore water pressures to dissipate if the deposits are saturated.

(2) Zone 2: The deposits which fall in this zone are classified as semi-pervious soils. These deposits include silts, sandy silts, or clayey silts with a PI equal to or less than 8%. Normally, the coefficient of permeability is in the range of 2×10^{-3} to 2×10^{-6} ft/min (1×10^{-3} to 1×10^{-6} cm/sec). These deposits can be improved by dynamic compaction even if they are fully saturated, but with more difficulty than for a partially saturated deposit. Tamping operations have to be carefully carried out because excess pore water pressures which are developed immediately following impact in saturated or nearly saturated soils may require days or even weeks to dissipate before additional tamping can be

undertaken. Multiple passes of the equipment are required to achieve the improvement. Because permeability is an important factor in determining the rate at which excess pore water pressures can be dissipated, it is important that permeability tests be performed upon soils which fall within this gradation range.

(3) Zone 3: This zone consists of impervious deposits. The soils included in this zone consist of clays or soils where the clay content is typically higher than 25% and the plasticity index is greater than 8%. The coefficient of permeability is generally less than about 2 X 10^{-6} ft/min (1 X 10^{-6} cm/sec). The degree of saturation of these deposits has a major influence on the effectiveness of dynamic compaction. If these deposits are saturated or nearly saturated, dynamic compaction would likely be ineffective. This generally includes natural soil deposits above or below the water table or fills below the water table. Partially saturated impervious fill deposits above the water table can be successfully densified by dynamic compaction. This generally includes clay fills with a high percentage of air voids which allows drainage of surface precipitation to lower levels and dissipation of excess pore water pressures during tamping. For these deposits, densification will occur as the clay lumps are moved together and air voids collapse. The natural water content of these deposits should be near or below the plastic limit so densification similar to Proctor compaction can occur. If the water content is higher than the plastic limit, the soils will deform plastically under impact without densification.

Not all soils will fall entirely within either Zone 1, 2 or 3 on Figure 5. For example, a granular soil which may almost entirely fall within Zone 1 but might have 5% to 15% fines and these fines may govern the behavior such that the deposit really behaves as if in Zone 2. In this case, it is suggested that permeability tests be performed to define the soil behavior since it is really soil permeability that governs the behavior rather than grain size gradation.

b. Deposits Where Conventional Index Tests are Not Appropriate for Characterization

Certain deposits do not lend themselves to classification by the conventional index tests. These include recent sanitary landfill deposits and highly organic deposits such as peat. Other waste products could also fit in to this category. The use of dynamic compaction on either recent landfills or organic deposits is a relatively new development and will require additional experimentation and observation of completed projects before long term performance following densification can be evaluated. Secondary compression in these deposits could be significant. The rating of these two deposits in Table 1 may need revision as additional information becomes available. As a guide, the following discussions are provided.

(1) Recent Sanitary Landfills

Dynamic compaction of recent landfills is feasible provided long-term settlement can be tolerated. The permeability of existing landfills has been measured as being on the order of 2 X 10^{-2} to 2 X 10^{-3} ft/min. (1 X 10^{-2} to 1 X 10^{-3} cm/sec) (see Appendix B). The upper portions of landfills are generally elevated above the surrounding terrain so the drainage of the upper portion is frequently relatively good. The clay cap that is usually placed to cover landfills is frequently cracked from differential settlement and desiccation which renders them semi-porous. Based upon the typical field permeability values, landfills would be classified similar to Zone 1 and Zone 2 soils. As densification takes place, the permeability of the landfill could decrease significantly, and the soils may move into a classification corresponding to Zone 2 which is still compactable.

A distinction must be made between older landfills and more recent landfills when considering the long-term settlement of the landfill following improvement. Older landfills can be defined as those where the highly organic materials have decomposed and the remaining particles are relatively inert. The composition of the older landfill would consist of a dark colored soil of silt and sand size but containing bottles, metal fragments, wood, and miscellaneous materials. The time it takes for a landfill to reach this stage depends greatly upon climatic conditions. Under anaerobic conditions, the estimated time for the majority of the decomposition to be completed is from 15 to 30 years. At this time, the total volume reduction due to decomposition and consolidation under its own weight will be approximately 10% to 30% of the original volume. Aerobic decomposition in a high temperature environment can take place at a rate 4 to 6 times faster than for anaerobic conditions; Chang, et.al.[12].

For deposits where biological decomposition is complete, dynamic compaction will have the greatest benefit. Densification will result in a higher unit weight and resulting reduction in compressibility with very little long term subsidence under load. For young landfills where organic decomposition is still taking place, dynamic compaction will increase the unit weight of the soil mass by collapsing voids and decreasing the void ratio. However, the dynamic compaction will not stop the biological decomposition which will result in a loosening of the soil structure followed by long-term settlements. A more detailed discussion of the factors affecting settling rates and methods to determine the relative age of landfills is presented in Appendix B.

The Arkansas Highway Department[5] used dynamic compaction to densify a recent landfill for a roadway embankment. The municipal landfill was placed between 1973 and 1978, and dynamic compaction was undertaken in 1982. The deposits consisted of miscellaneous materials including organic municipal waste, paper, cans, metal objects and plastics. Welsh[96] indicates that the sanitary landfill was compressed 5.2 to 8.2 ft (1.6 to 2.5 m) or 20% to 25% of its original thickness as a result of tamping. It was necessary to add 6.6 ft (3 m) of granular material at the top of the landfill initially and to add additional granular material as it was forced into the underlying landfill. The tamping densified the landfill and collapsed voids but also formed short stone columns at the crater locations which were connected by a surface raft of granular material to help spread the loads into the underlying deposits. Load tests were performed by placing earth fill such that a soil pressure of 2000 psf (95.8 kN/m^2) was imposed at the surface of the landfill. These tests were performed before and after tamping to evaluate the effectiveness of the dynamic compaction. Surface settlement before dynamic compaction was 11.5 in. (28.8 cm) one week after loading. After dynamic compaction, surface settlement was 0.56 in. (14 cm) one week after loading.

Following construction of the roadway, settlement readings were taken in August, 1985, on permanent observation points situated adjacent to the roadway[4]. This date corresponds to a time interval approximately 3-1/2 years following dynamic compaction. An average settlement of 4 in. (10.2 cm) and a maximum settlement of 10 in. (25.4 cm) was recorded. It is anticipated that additional settlements will occur over many years as biological decomposition within the landfill occurs. However, this may not be detrimental to the pavement sections since the compacted embankment over the landfill will provide some bridging action and minimize differential movements. Releveling of the roadway surface could be undertaken by overlays or mudjacking as settlements within the landfill occur. Methane gas generation is expected to occur for many years. The gas is expected to vent directly to the atmosphere through the embankment.

Charles, et.al.[14] reports on a 19.7 ft (6 m) thick municipal landfill that was 15 years old and not yet totally decomposed. This deposit was compacted with a 16.5 ton (15 t) weight dropping 65.6 ft (20 m). Long term post-construction settlement readings under a 9.8 ft (3 m) high earth embankment indicate movements varying from 1.6 to 2.2 in. (40 to 55 mm) over a period of four years. In spite of these movements, the roadway is reported to be performing satisfactorily.

(2) Peat Deposits

Dynamic compaction has been used on highly organic deposits including peat or peaty clays. Because these soils have such a high void ratio, the main form of stabilization comes from intermixing of these deposits with the blanket of granular material which is placed over the surface prior to dynamic compaction. A secondary beneficial effect occurs from consolidation of these weak deposits because the granular soils provide a drainage path allowing excess pore water pressures to dissipate. Ramaswamy, et.al.[78] states that mechanical mixing is possible if the peat or organic soil of high void ratio is of limited thickness (less than 19.7 ft (6 m) and if the overlying material is granular and does not exceed 16.4 ft (5 m) in thickness. The amount of energy to accomplish this intermixing is not predictable so it appears that some field experimentation is warranted before starting production work at a specific site.

Lee, et.al.[47] reports on a variation of dynamic compaction entitled: "Dynamic Replacement and Mixing". This technique involves driving columns of granular material into peat, thus replacing the peat with a less compressible material as well as mixing granular soil with the peaty soils. This technique offers merit where the organic deposits are close to the ground surface and the energy is sufficient to cause intermixing. For buried organic deposits, it is doubtful whether sufficient energy can be applied to cause intermixing. Some investigators have reported a lack of intermixing of weak and stronger deposits when they are situated at depths of 9.8 to 16.4 ft (3 to 5 m) below the point of impact.

Because of the paucity of data regarding stabilization of peats and organic deposits, dynamic compaction should be preceded by test sections undertaken in advance of construction to determine the amount of improvement that is achieved. The owner should be willing to accept some risk from secondary compression settlement which can be significant.

(3) Waste Fill Deposits

Certain waste fill deposits may degradate or soften during tamping thereby making compaction difficult and the degree of improvement less than desired. Greenwood[35] experienced difficulty with dynamic compaction of soft rock particles, flaky in shape, ranging in size from silts to gravels, left over from mining of oil shales. These materials exfoliated in the presence of water and disintegrated on tamping because the induced stresses exceeded the natural strength of the material. This disintegration resulted in irregular clayey zones within the fill. Soft shale particles when wetted and stressed could produce a similar effect.

CHAPTER 3 - IMPROVEMENTS IN SOIL PROPERTIES

1. Depth of Improvement

The primary goal of dynamic compaction is to improve the geotechnical properties of deposits to a significant depth below the ground surface. The purpose of the improvement could include:

- A reduction in compressibility to reduce settlement,
- An increase in shear strength to increase the factor of safety against bearing failure,
- A collapse of voids in cemented natural soil deposits or in old fills to minimize ground movements upon loading or saturation, or
- A decrease in the liquefaction potential of granular soil.

The necessary depth of improvement will depend upon the performance requirements of the embankment or structure to be constructed over the densified ground. If settlement is to be limited, the compressible zones within the zone stressed by the loading should be rendered less compressible by the densification. Earth structures such as embankments can tolerate more settlement than structures such as bridge foundations or buildings, so the degree and/or depth of improvement could be less. Synthesis 29 of the National Cooperative Highway Research Program[69] indicates that post construction settlements of as much as 1 to 2 ft (0.3 to 0.6 m) are considered tolerable in an embankment provided that 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. In existing deep fill deposits, it is sometimes acceptable to densify only the upper portion of the fill by dynamic compaction to form a mat or crust of densified ground to reduce differential movement.

As a first approximation on the depth of improvement that can be achieved from dynamic compaction, Menard and Broise[61] stated that the energy per blow, WxH, is an essential parameter. They suggested the following relationship:

$$W x H > D^2 \tag{1}$$

or usually written as:

$$D < (WH)^{1/2} \tag{2}$$

where:

D = depth of improvement in meters
W = weight of tamper in metric tons
H = drop height in meters

A modification of this basic formula has been suggested as follows:

$$D = n \ (W \times H)^{1/2} \tag{3}$$

where:

n = an empirical coefficient which is less than 1.0

The coefficient, n, is required to account for factors which can affect the depth of improvement other than the weight of the tamper and the drop height. Some investigators have suggested using n = 0.5 for all soil deposits as shown in Figures 6 and 7[24, 59]. As a first approximation, this value of n=0.5 is

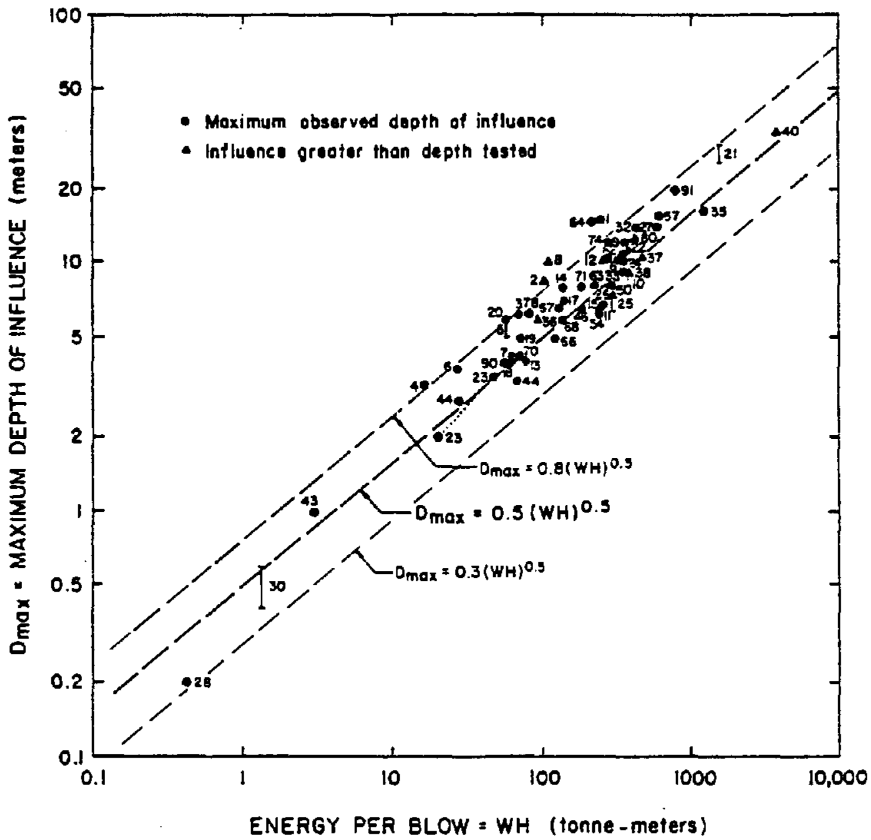


Figure 6 : Trend between apparent maximum depth influence and energy per blow (59)
(See Mayne etal (59) for details of the numbers included in this figure.)

(1m = 3.28ft; 1 tonne = 1.1 ton)

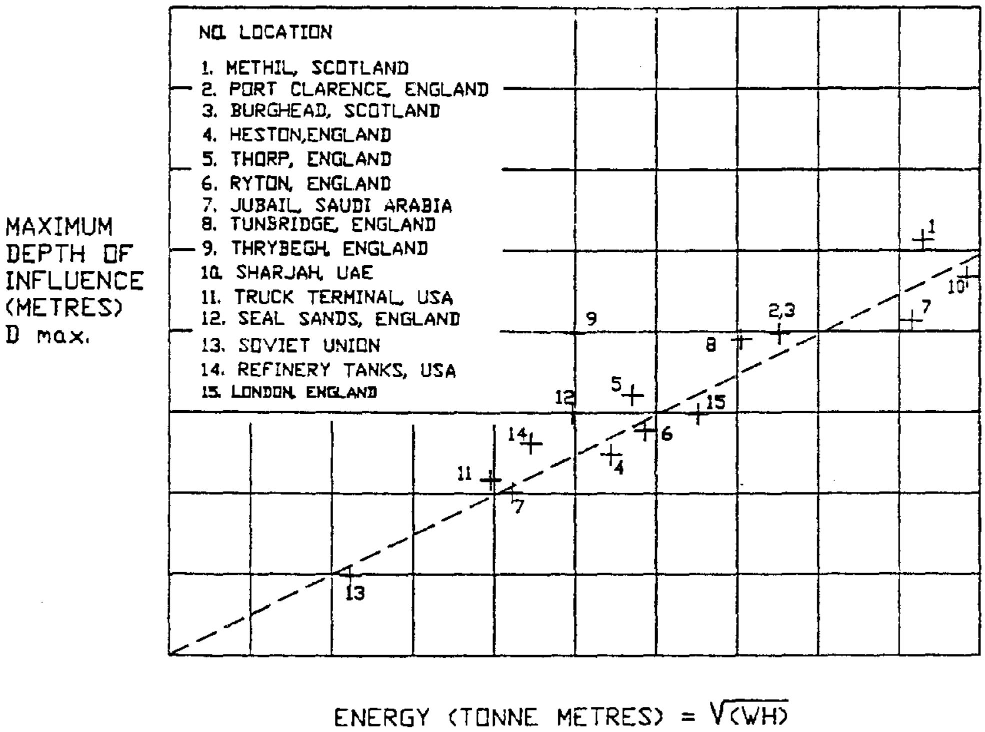


Figure 7 : Plot of energy vs. depth of influence (24)

(1m = 3.28ft; 1 tonne = 1.1 ton)

reasonable. However, there is evidence that n is affected by many variables including:

- The type and characteristics of the deposit being densified.
- The applied energy.
- The contact pressure of the tamper.
- The influence of cable drag.
- The presence of energy absorbing layers.

a. Influence of Soil Type

The depth of improvement appears to be related to soil type. Numerous investigators have reported the depth of improvement attained in widely varying soil types and the results are summarized in Tables 2 to 4 for three categories of soil deposits; ie., pervious granular soils, semi-pervious soils, and partially saturated impervious fill deposits. These three categories were selected to be consistent with the grouping of soils shown on Figure 5.

On these projects, the amount of energy, tamper contact pressure, and type of equipment also varied so the depth of improvement was also affected by these variables. However, there appears to be a definite influence of soil type on depth of improvement. This can be partly attributed to different degrees of damping with different soils and to the development of excess pore water pressures in certain deposits during tamping.

Table 2 summarizes three projects where dynamic compaction was used on mine spoil fill that was essentially cohesive and where the water table was located below the zone of improved soil. This data indicates that the n value should be on the order of 0.35 to 0.40 for partially saturated cohesive fills situated above the water table. The natural water content of the fill was noted to be near the plastic limit at the time of dynamic compaction and this is considered essential to obtain densification. The optimum water content of the fill for dynamic compaction is not known, but it probably is near the optimum water content as determined by Modified Proctor compaction which, in turn, can be estimated as the plastic limit minus 7 percent (70).

TABLE 2

Depth of Improvement in Partially Saturated Impevious Fill Deposits

Classification	Source of Information	W Tonnes	H Meters	D Meters	n	Depth to Water Table meters	Water Content Natural	Water Content Plastic Limit
Heterogeneous Cohesive Fill with Gravel & Rubble	Ground Engineering (36)	15	20	6.5	0.38	24+		
Cohesive Strip Mine Spoil	Thompson and Herbert (87)	15	24	6 to 7	0.38	24	17-22	18
Cohesive Strip Mine Spoil with Shale and Sandstone Fragments	Lukas (55)	16.4	23	7.6	0.39	9.8	15-20	20-25

NOTE

W = Weight of tamper in tonnes

H = Drop height in meters

D = Depth of improvement in meters

$n = \frac{D}{\sqrt{WH}}$

Note: 1 tonne = 1.12 ton
1 meter = 3.28 ft

Table 3 summarizes ten projects where dynamic compaction was used on pervious granular deposits with varying levels of the water table. This data indicates that, n, typically ranges from 0.5 to 0.7 with a few exceptions, regardless of the position of the water table.

Semi-pervious deposits consist primarily of silts, sandy silts and clayey silts. Eight projects where dynamic compaction was used on these deposits is summarized in Table 4. Where the water table is low, the n value ranged from 0.4 to 0.6, but where the water table was high, n, ranged from 0.37 to 0.47. This indicates that the degree of saturation has some influence on the depth of improvement.

b. Influence of Energy Applied

As the compaction energy applied over the entire site increases, ground settlements, as measured by average ground depressions over the entire area, generally increase. This is illustrated by Figures 8, 9 and 10, which were obtained from Pearce[75], Hansbo, et.al.[41], and Mayne, et.al.[59], respectively. Most of this settlement is probably due to continuing densification within the zone where soil property improvements are occurring rather than compression in deeper layers after the surface is compacted. However, Minkov and Donchev[63] found that surface settlements are a good indicator of the depth of the compacted zone. Figure 11, from reference 63, implies that as the surface deflection increases, the thickness of the compacted zone also increases.

Depth of Improvement in Pervious Granular Soil Deposits

Classification	Reference	W tonnes	H meters	D meters	n	Depth to Water Table (meters)
Clean Sand	Gambin (30)	16.4	23.0	15.0	0.77	3
Silty Sand	Gambin (30)	16.4	23.0	12.0	0.62	3
Fine To Medium Sand	Leonards, et.al. (48)	4.1 to 5.9	9.0 to 12.0	3.0 to 5.0	0.50	9 to 10.5
Building Rubble	Lukas (51)	5.5	9.2 to 12.2	4 to 5	0.56 to 0.62	3 to 4
Sand and Silt	Hansbo, et.al. (42)	12.0	12.0	12.0	1.0	2.5
Pervious Mine Spoil	Guyot and Varaksin (37) Dames & Moore (22)	15.0	20.0	10 to 12	0.48 to 0.68	13
Fine to Medium Sand Fill	Lukas (54)	15.0	18.3	9.1	0.5	3
Sand and Silty Sands	Cognan, et.al. (19)	20.0	20.0	12.0	0.6	2
Cinders, Decomposed Refuse and Misc. Fill	Steinberg and Lukas (87)	13.6	18.0	9.2	0.58	2
Rockfill 0.6 to 30 cm	Wightman and Beaton (98)	20.0	30.0	13.0	0.53	1 to 5

NOTE

W = Weight of tamper in tonnes
H = Drop height in meters
D = Depth of improvement in meters

$$n = \frac{D}{\sqrt{WH}}$$

Note: 1 tonne = 1.12 ton
1 meter = 3.28 ft

TABLE 4

Depth of Improvement in Semi-Pervious Soils

Classification	Reference	W tonnes	H meters	D meters	n	Depth to Water Table (meters)
Loess	Minkov & Donchev (63)	8.3 15.0	10.0 10.0	4.0 4.5	0.44 0.37	
Silt to Silty Sand	Lovelace, et.al. (50)	11 to 18	22.0	6.1 to 7.6	0.4 to 0.6	12.0
Micaceous Silty Sand	Lukas (56)	29.0	30.5	11.6	0.39	4.6
Silty Sand With Peat and Clayey Silt Seams	Gambin (30)	16.4	24.4	8.0	0.4	
Silts and Sandy Silts	Gambin (30)	41.0	30.5	16.0	0.45	4.0
Clay, Sand and Ash With Gravel, Brick Fragments, Glass and Wood	Charles, et.al. (14)	14.0	14.0	6.5	0.46	Below 6.5
Silty Fine Sand With Lenses of Fine Sandy Silt Underlain by Silt	O'Brien and Gupton (73)	18.0	25.0	10.0	0.47	3.5
Mine Spoil Consisting of Shale and Siltstone Fragments with a Matrix of Clayey Silt and Sand	Mayne, et.al. (57)	20.9	19.0	10.7	0.54	Below 12 m

NOTE

W = Weight of tamper in tonnes

H = Drop of height in meters

D = Depth of improvement in meters

$n = \frac{D}{\sqrt{WH}}$

Note: 1 tonne = 1.12 ton
1 meter = 3.28 ft

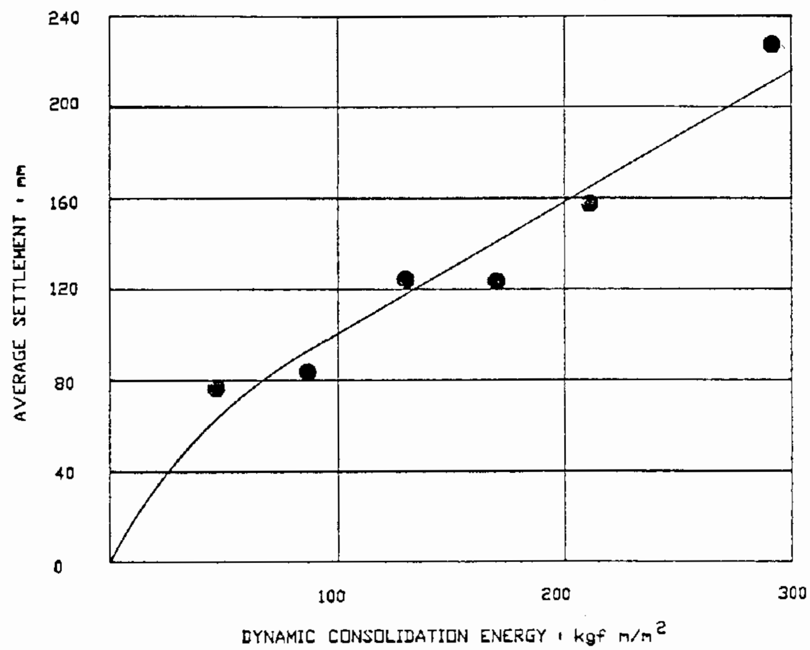
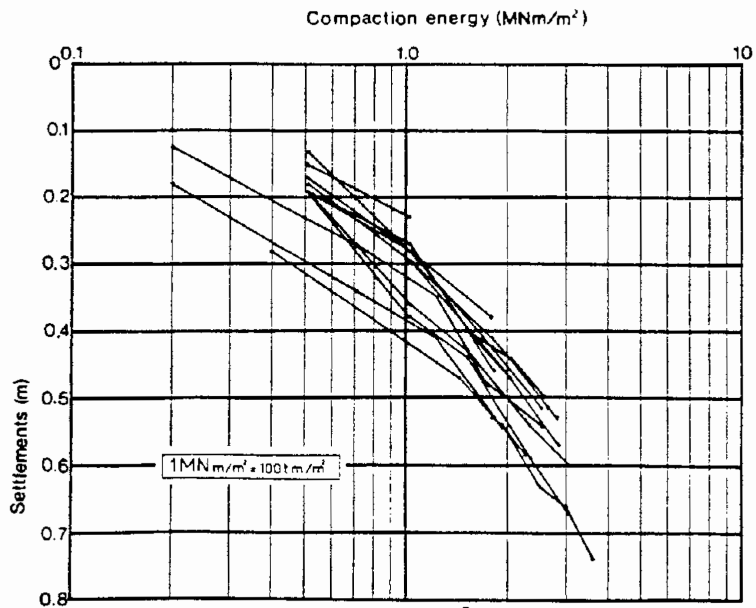


Figure 8 : Induced surface settlement with variation in applied energy at a landfill site (75)

(1 kgf m/m^2 = 0.67 lb-ft/ft^2, 1in = 25.4mm)



(1 tm/m^2 = 0.34 ton ft/ft^2, 1m = 3.28ft)

Figure 9 : Settlement versus logarithm of compacted energy (41)

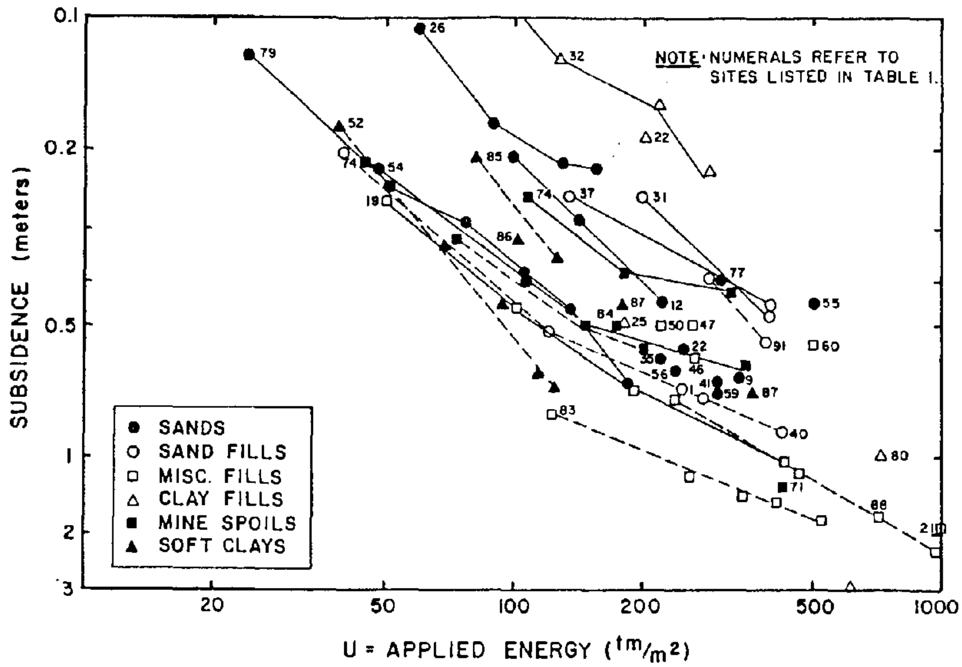
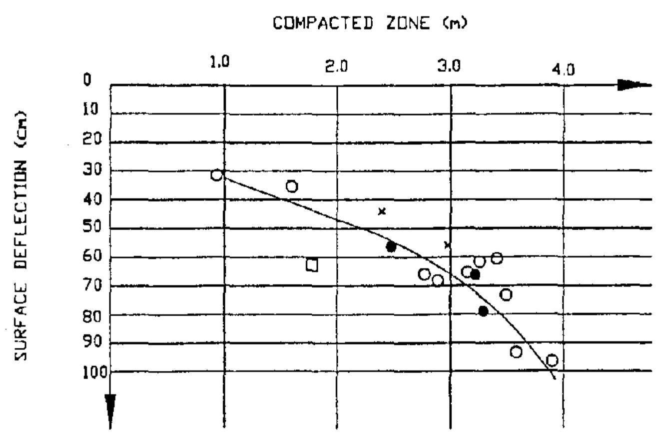


Figure 10 : Observed subsidence with level of applied energy per unit area (59)
(See Mayne etal (59) for details of the numbers included in this figure.)
($1tm/m^2$ = 0.34 ton ft/ft^2; 1m=3.28 ft)



○ = G=15 t (q=30 kPa)
● - G=12 t (q=30 kPa)
× - G=8.3 t (q=17 kPa)
□ - G=3.5 t (q=17 kPa)

NOTE : q = CONTACT PRESSURE AT BASE OF TAMPER
G = WEIGHT OF TAMPER IN TONNES

(1m = 3.28ft; 1cm = 2.54in)

Figure 11 : Thickness of compacted soil as measured by surface deflection for a loess soil (63)

Ground movement measurements were obtained at three instrumented project sites to monitor the depth at which the ground is improved following each blow of the falling weight[54, 55, 56]. Ground displacement readings were obtained by using vertical plates buried at varying distances directly below the point of impact as well as inclinometers located at distances of 10 and 20 ft (3.0 and 6.1 m) laterally from the center line of the point of impact. Unfortunately, many of the vertical indicators below the point of impact were destroyed before the full energy was applied. However, good information was obtained at both the 10 and 20 ft (3.0 and 6.1 m) inclinometer stations through 12 to 14 drops at the same print. The depth of improvement as monitored by the conventional testing of SPT, CPT and PMT tests agreed with the depth of improvement as determined from ground displacements measured by the inclinometers.

The depths at which movements were observed at each of these three sites is plotted in Figures 12 and 13 as a function of the number of drops and in Figures 14 and 15 as a function of the energy applied at the point (calculated by multiplying the tamper weight times the drop height times the number of drops). This data shows that for fine to medium sands and silty sands, approximately 90% of the depth of improvement that is eventually reached occurs with 9038 ft-ton (2500 t-m) of energy. In the silty sand deposit, this occurred after the second drop and in the fine to medium sand after the seventh drop. From that point onward, there is a slight additional depth increase with subsequent blows, but at a rate of only about 0.3 ft (9.1 cm) per drop. At the clayey mine spoil site, 55% of the eventual improved depth occurred after the second drop 1808 ft-ton (500 t-m). There was a steady increase in depth of improvement corresponding to approximately 1 ft (.3 m) of improvement for each additional drop up to the maximum applied 14

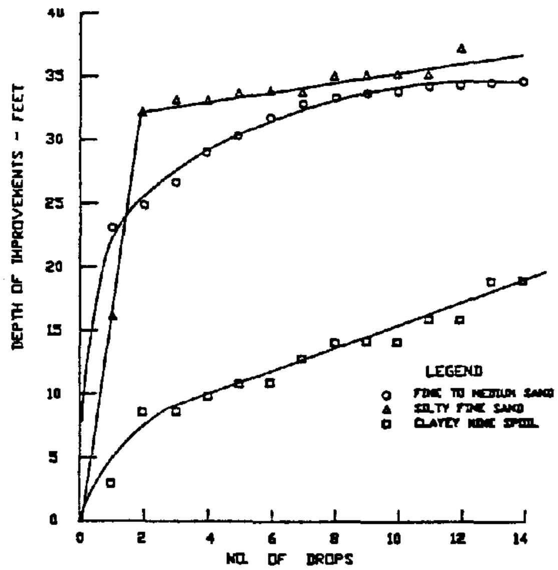


Figure 12 : Depth of improvements as measured by lateral deflection obtained at inclinometer located 10 feet from center of drop point

(1ft = 0.305m)

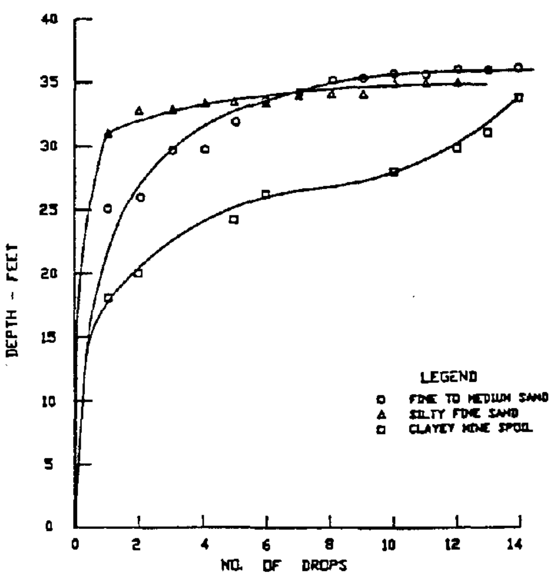


Figure 13 : Depth of improvements as measured by lateral deflection obtained at inclinometer located 20 feet from center of drop point

(1ft = 0.305m)

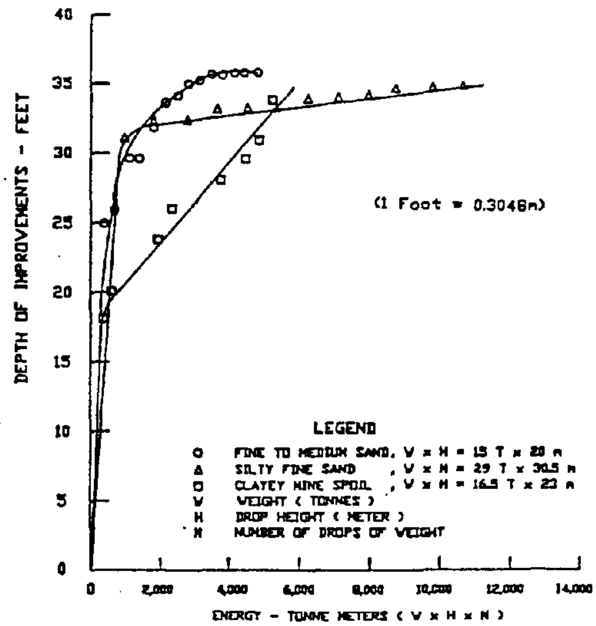


Figure 14 : Depth of improvements as measured by lateral deflection obtained at inclinometer located 10 feet from center of drop point

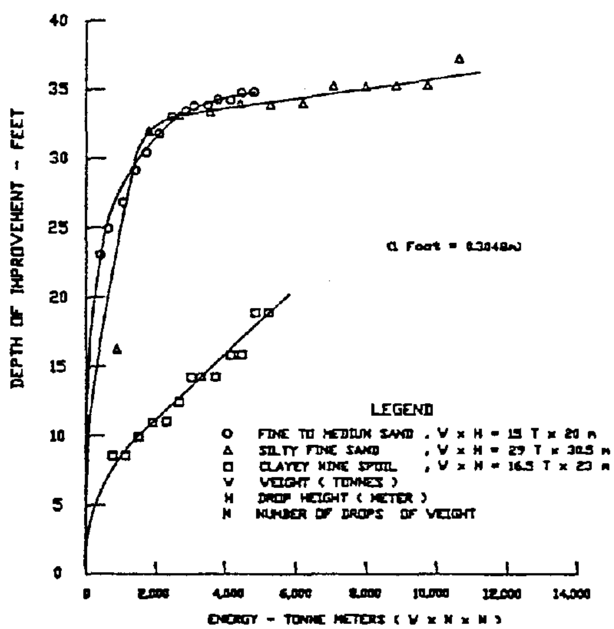


Figure 15 : Depth of improvements as measured by lateral deflection obtained at inclinometer located 20 feet from the center of the drop point

drops. Thus, it appears that the depth of improvement may be affected by additional drops within clayey fill deposits. This contrasts with the sandy and silty sand formations where energy applied after 9038 ft-ton (2500 t-m) (2 to 7 drops) is not significantly effective in extending the depth of improvement.

c. Influence of Contact Pressure

The contact pressure is the weight of the tamper divided by the contact area. Most tampers have a contact pressure on the order of 800 to 1550 psf (40 to 75 kN/m^2). Contact pressures significantly higher than this could result in the weight punching into the ground upon impact, similar to driving a rod into the ground with a sledge hammer. On the other hand, if the contact area of the weight is extremely large and the pressure small, very little improvement at depth would occur under impact. A tamper with a larger contact area is generally used for the final ironing pass when the depth of improvement is to be limited to the upper few meters of the deposit. Unpublished data from two project sites indicates that tampers with contact pressures on the order of 400 psf (20 kN/m^2) developed a densified crust of soil, but only to a depth of D/2, where D was the diameter of the tamper.

d. Influence of Cable Drag

On most projects, the tamper is lifted and dropped with a single cable. A free spool crane is normally used, but there are friction losses as the drum unwinds and as the cable passes over the sheave. There is also some reduction in velocity due to air resistance. The Menard organization has suggested a reduction of the empirical factor, n, from 1.0 to 0.9 to account for cable drag. This is based upon judgment and experience rather than measurements.

Measurements of the velocity of the weight immediately prior to impact were obtained at four sites[54, 55, 56]. The velocity of the falling weight was determined approximately 1 to 2 ft above the point of impact by a device consisting of two laser beams and a timer. As the weight crossed the first laser beam, the timer was started and when it crossed the second laser beam, 1 ft lower in elevation, the timer stopped. Thus, it was possible to calculate the speed immediately prior to impact. At one site, the velocity of the falling weight was also measured using a radar gun. A plot of the change in velocity with time is included in Appendix C. There is a slight difference in velocity of the tamper immediately prior to impact between the radar readings and the laser beam readings. The radar readings are believed to be more accurate since a continuous plot of velocity with time is obtained. This is more accurate than a single velocity reading at one point in time that is obtained with the laser readings.

The theoretical velocity for a freely falling object without resistance can be calculated as follows:

$$v = (2gH)^{1/2} \qquad (4)$$

where: g = acceleration of gravity
H = the drop height

The ratio of the measured velocity to the theoretical velocity is compared in Table 5 for readings which were taken at four different project sites. The weights ranged from 6 tons to 32 tons (5.4 to 29.0 t) and the cranes that were used to raise the weights ranged from 40 tons to 150 tons (36 to 136 t) capacity.

TABLE 5

Velocity of Tamper Prior to Impact

Site	Tamper Weight tons	Drop Height Less 2 ft. feet	Theoretical Velocity ft/sec	Measured Velocity ft/sec	Ratio: $\frac{V\ mea.}{V\ Theor.}$
Becancour Canada	16.5	10	25.4	23.2	.92
		20	35.9	31.5	.88
		58	61.1	55.0	.90
Tulsa Oklahoma	18	18	34.0	30.5	.90
		38	49.5	43.0	.87
		73	68.6	60.0	.88
St. Mary's Georgia	32 Free Fall	98	79.4	78.1	.98
		98	89.4	77.4 (Radar)	.97
Great Lakes Illinois	6	18	34.0	31.1	.91
		33	46.1	42.5	.92

NOTE: The laser device extends 2 ft (.6 m) above grade so the drop heights were adjusted accordingly.

The radar gun data for the St. Mary's, Georgia project is summarized in Appendix C.

(1 m = 3.28 ft, 1 ft/sec = .305 m/sec, 1 ton = .9 t)

The weights at three sites were lifted by the cranes using a single cable and a free spool drum. The weight at the Georgia site was lifted by cables but then allowed to drop free fall. For the three sites where the single cable drop was used, there is a remarkable similarity in the ratio of the measured to theoretical velocity prior to impact indicating that the cable drag, drum resistance and sheave friction is proportionately similar for each of the rigs. Because of the wide variation in rigs and weights dropped in this study, it appears that this ratio will be approximately the same for most types of equipment where a free spool drum is used with a single cable drop assuming the rigs are in good working condition and an experienced person operates the rig.

The amount of energy delivered to the ground is a function of the velocity squared.

$$E_r = 1/2mv^2 \tag{5}$$

where: m = mass of the tamper
v = velocity at the time of impact

For a velocity of 90 percent of the theoretical maximum immediately prior to impact, this means that the amount of energy delivered to the ground is approximately 80 percent of the maximum potential energy, WH. This reduction in efficiency is included in the empirical coefficient, n, that is used in Equation 3.

As indicated in Table 4, the velocity of the free falling weight was found to be approximately 97 percent of the theoretical maximum velocity. This results in energy delivered to the ground of 94 percent of the maximum potential energy. Thus, the free fall delivers approximately 18 percent more energy than for a single cable dropped weight so the n values used in Equation 3 could theoretically be increased by 18 percent if a freely falling weight is used.

e. Influence of Hard or Energy Absorbing Layers

Where hard or cemented layers occur within the upper portions of the deposits being densified, these deposits will distribute the pressures induced by the impact. The energy of tamping will therefore not be transmitted as deep as in a loose deposit. On some projects, where hard layers have been observed at the ground surface, they were either broken up to loosen them or removed and replaced with a different fill. If the hard layer is located near the bottom of the zone of densification, the presence of the hard layer can improve the densification since a portion of the compression and shear waves induced by the tamping are reflected back up from the hard layer causing additional densification. This phenomenon was observed at a site where densification was applied to a loose refuse fill overlying a stiff clay deposit[87].

Saturated clay layers that may be present within an otherwise non-clayey soil deposit also have an effect of reducing the amount or depth of densification achieved. Leonards, et.al.[48] reports on a project where the energy had to be doubled to get adequate depth of improvement because of a clay layer situated within a sand deposit. The saturated clay layers appear to absorb some of the energy.

f. Suggested Method for Predicting Depth of Improvement

The anticipated depth of improvement for various combinations of tamper weight and drop height should be calculated using Equation 3. Suggested values of the empirical factor, n, to be used with Equation 3 are listed in Table 6. These values of n reflect the influence of soil type and cable drag for a tamper dropped with a free spool drum and single cable. They do not account for variations in the applied energy, tamper contact pressure or influence of energy absorbing layers. The n values in Table 6 are for typical conditions where the applied energy ranges from 34 to 100 ton-ft/ft^2 (100 to 300 txm/m^2), the contact pressure of the tamper ranges from 800 to 1500 psf (39 to 82 kN/m^2) and no energy absorbing layers are present that could reduce the depth of improvement.

TABLE 6

Recommended n Value for Different Soil Types

Soil Type	Degree of Saturation	Recommended n Value*
Pervious Soil Deposits - Granular Soils	High	0.5
	Low	0.5 to 0.6
Semi-Pervious Soil Deposits, Primarily Silts with P.I.<8	High	0.35 to 0.4
	Low	0.4 to 0.5
Impervious Deposits Primarily Clayey Soils with P.1>8	High	Not recommended
	Low	.35 to 0.40 Soils should be at a water content less than the plastic limit.

*For an applied energy of 34 to 100 ton-ft/ft^2 (100 to 300 txm/m^2) and for a weight dropped using a single cable with a free spool drum.

Greater or lesser depths of improvement could be reached for energy levels higher or lower than this range.

The depth of improvement calculated using Equation 3 and Table 6 should be considered only as a guideline. The depth of improvement reached at a specific project site will depend not only upon the variables discussed but also upon specific site conditions and the types of measurements that are used to measure the depth of improvement. Some in-situ tests used to measure improvement may indicate greater or lesser depths of improvement than others. Experience with variations in test results is discussed in Chapter 5.

g. Depth Limitation

If dynamic compaction is to be undertaken with readily available cranes, there is a limitation on the depth of improvement that can be achieved. The maximum capacity cranes normally available are rated as 150 to 175 tons (136 to 158 t) and these size cranes can lift weights up to 20 to 22 tons (18.1 to 20 t) with a maximum drop height of about 66 to 98 ft (20 to 30 m). Using an average n value of 0.5, a maximum depth of improvement in the range of 33 to 39 ft (10 to 12 m) can be achieved.

If greater depths of improvement are required, either heavier duty or specialized equipment is required to lift heavier weights to greater heights. Some of this equipment is available through specialty contractors. Some specialty contractors can drop 30 to 32 ton (27.2 to 29.0 t) weights from a height of 100 ft (30.5 m) using multiple cables and pulleys to raise the weight and then allowing it to drop free fall. Cable life is greatly lengthened with this method, but each drop of the weight requires a time of about 5 minutes primarily because of the slow lift and descent of the cable pulley system. A tripod rig developed by the Menard organization is capable of dropping a 36 ton (40 t) weight through a distance of 131 ft (40 m). This equipment has been used on approximately seven projects to date throughout the world. Alternate procedures would be to use different ground improvement techniques at the greater depths. Vibrating probes, blasting, or grouting have been used on some projects for deeper densification[83, 65].

2. Anticipated Degree of Improvement

In order to plan and design a project situated over dynamically compacted ground, it is necessary to anticipate the amount of improvement that will be attained. If the anticipated improvements are not sufficient to increase the bearing capacity or limit settlements to desirable values, some other form of improvement or ground support should be investigated.

a. Factors Affecting Improvement

The primary factor affecting the degree of improvement on a dynamic compaction project is the average energy applied at the ground surface. The greater the amount of energy applied, the greater the improvement in the soil properties, although there is usually a diminishing effect for additional increments of energy applied beyond some reasonable amount. The ground improvement can be likened to the improvement that occurs in a Proctor mold when different energy is applied. The unit weight that is achieved by compacting soil in a Proctor mold will be higher for Modified Proctor energy than for Standard Proctor energy. This increase in unit weight is not directly proportional to the energy but does result in an increase in strength and a reduction in compressibility. A similar phenomenon occurs in the ground during dynamic compaction.

Gambin[32] measured the increase in pressuremeter modulus with applied energy. The results of these measurements are shown in Figure 16 for a granular fill and in Figure 17, for the underlying natural sandy silt. The improvement in the pressuremeter modulus in the natural soil increased linearly with increasing energy up to the maximum applied energy of 100 ton-ft/ft^2 (300 txm/m^2). In the granular fill, it appears that additional energy would result in even greater property improvement.

Mayne, et.al.[59] plotted limit pressures from pressuremeter tests that were measured in coarse grained deposits, Figure 18, and fine grained soils, Figure 19, for ranges in applied energy of 34 to 168 ton-ft/ft^2 (100 to 500 txm/m^2). Typical limit pressures before treatment ranged from 3 to 6 tsf (3 to 6 bars) for the coarse grained soils to 2 to 3 tsf (2 to 3 bars) for the fine grained soils. Improvements in limit pressures were observed with increasing amounts of applied energy but the relative improvements were much greater for the coarse grained soils.

In semi-pervious soil deposits with particle sizes ranging from silty fine sand and smaller, other factors can also affect the degree of improvement. These would include the grid spacing

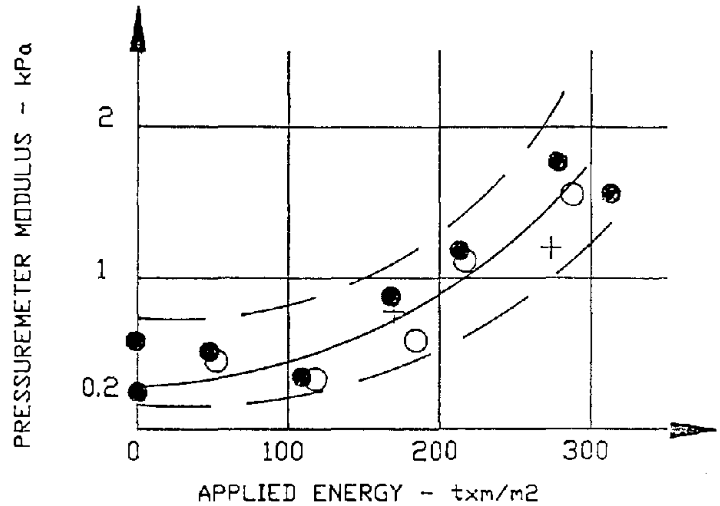


Figure 16 : Soil improvements in sand and gravel fill using 170 tonne tamper and 25 meter drop (32)

(1 txm/m^2 = 0.34 ton-ft/ft^2; 1 tsf = 95.8kpa)

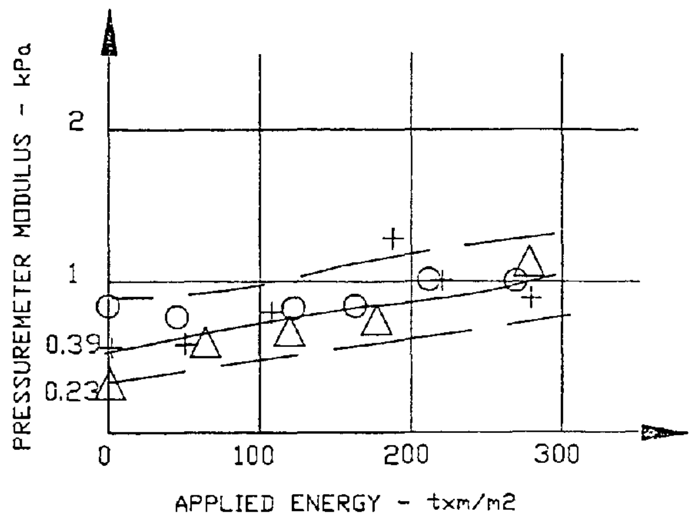


Figure 17 : Soil improvements in natural clayey silts alternating with sand using a 170 tonne tamper and 25 meter drop (32)

(1 txm/m^2 = 0.34 ton-ft/ft^2; 1 tsf = 95.8kpa)

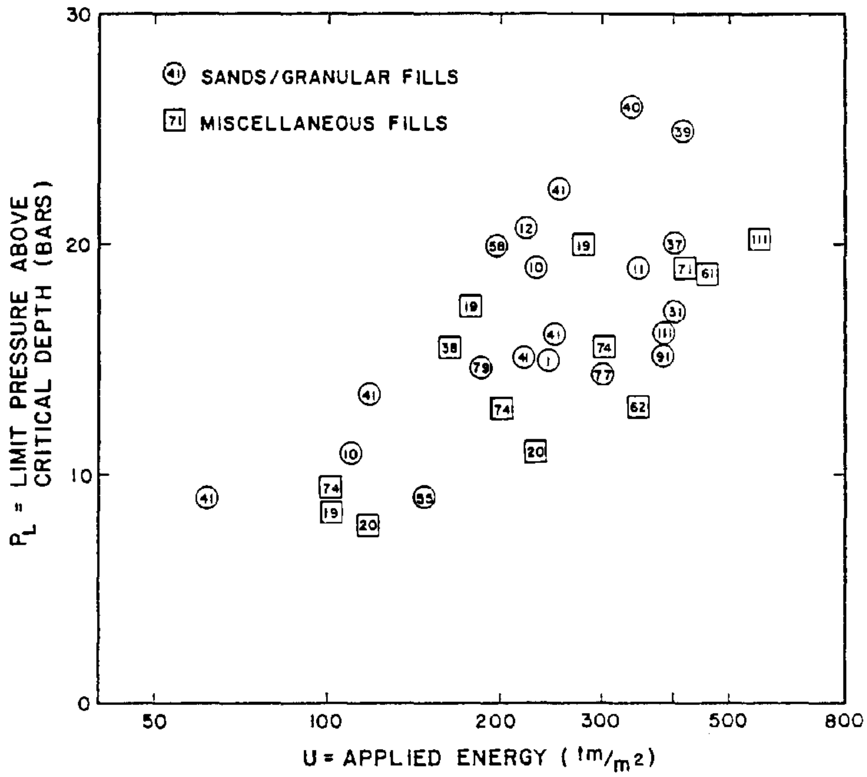


Figure 18 : Observed trend between limit pressure and applied energy for granular soils (59) (See Mayne etal (59) for details of the numbers included in this figure.)

($1 tm/m^2$ = 0.34 $ton\text{-}ft/ft^2$; 1 bar = 1.02 ton/ft^2)

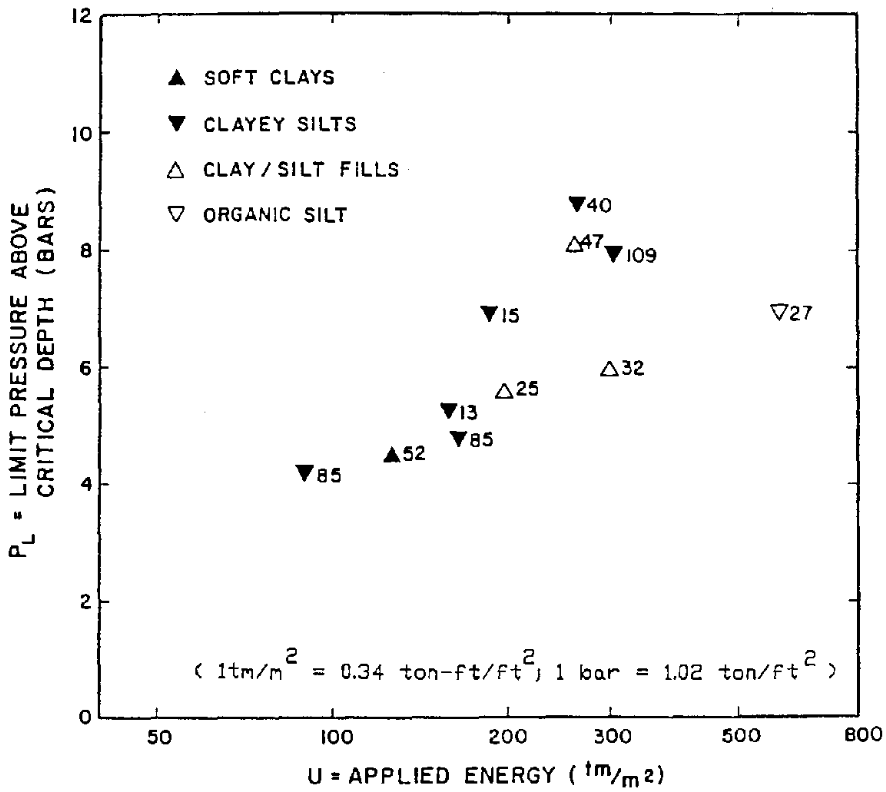


Figure 19 : Observed trend between limit pressure and applied energy for cohesive soils (59) (See Mayne etal (59) for details of the numbers included in this figure.)

between craters, the number of impacts at each individual crater, the position of the water table, and the time delay between passes. In the semi-pervious deposits, excess pore pressures usually develop during dynamic compaction, depending upon the degree of saturation and the position of the water table. Thus, the sequence at which the energy is applied is very important. If the weight is dropped repeatedly at a location and the deposits become liquefied, additional tamping will not be effective until the excess pore pressures are allowed to dissipate. Repeated tamping at full saturation merely produces plastic deformations in the soil. The crater may grow deeper in depth, but heave will occur around the perimeter of the crater. It will be necessary at this time to allow the excess pore pressure to dissipate before additional tamping is undertaken to achieve further densification.

b. Magnitude of Improvement

For a specific soil deposit, the magnitude of the improvement is dependent upon the amount of energy applied at ground surface as well as the initial relative density of the deposit. For energies ranging from about 34 to 100 ton-ft/ft^2 (100 to 300 txm/m^2), improvements in the properties of the soils are generally on the order of a 100 to 400% increase in strength as measured by tests such as a static cone or a standard penetration test or a reduction in compressibility measured by tests such as the pressuremeter. More specifically, the magnitude of the anticipated improvements for representative soil types are shown in Table 7. For each soil, the upper level range of improvement is for deposits that were initially in a loose condition and the lower bound for deposits initially in a more compact condition.

There is an upper bound of improvement that can be achieved. A further discussion of limiting values is presented later in this chapter.

TABLE 7

Anticipated Relative Improvements for Different Soil Types

Soil Type	Anticipated Amount of Improvement*
Pervious Coarse Grained Soils - Sands and Gravels	300 to 400%
Semi Pervious Soils	
A. Silty sands	100 to 400%
B. Silts and partially saturated clayey silts	100 to 250%
Partially Saturated Impervious Soils - Clay Fills and Mine Spoil	200 to 400%
Landfills	200 to 400%
Building rubble	200 to 300%

*For applied energies of 34 to 100 ton-ft/ft^2 (100 to 300 txm/m^2)

On some projects, it may not be necessary to attain the improvement indicated in Table 7. Only a slight increase in bearing capacity or a slight reduction in compressibility may be sufficient to satisfy the project requirements. Where this is the case, the applied energy could be reduced to match the desired improvement.

The magnitude of improvement indicated in Table 7 is usually not uniform throughout the entire predicted depth of improvement. Generally, the maximum improvement is observed within a zone on the order of one-third to one-half the predicted depth of improvement below which the improvement diminishes with depth.

Figures 20a to 20c illustrate the approximate degrees of improvement that are achieved at various stages of the dynamic compaction operations. During the initial stages of tamping, the surface is disturbed and the test values are usually less than the initial. This is due to the loosening and disturbing effect from the surface craters. Following the ironing pass or compaction with conventional equipment, the improvement approaches that shown by Figure 20b. Figure 20c indicates the condition that is achieved where a significant amount of energy is applied during the ironing pass to make the improvement more uniform to a depth of D/3 to D/2, thereby forming a crust of hardened material of substantial thickness.

c. Uniformity of Improvement

One of the advantages of dynamic compaction is that previously undetected weak pockets or loose zones which may be present within the soil mass can be eliminated, thereby resulting in a more uniform soil profile. The dynamic compaction can be adjusted in the field to apply more energy in the loose zones. Lukas (51) reports on an improvement in a sandy soil deposit in Algeria where the Standard Penetration Resistance values were initially on the order of 1 to 2 blows per foot in a localized area. Following dynamic compaction, the Standard Penetration Resistance increased to values on the order of 15 to 20 blows per foot within this stratum. At other sites where voids may actually be present, the dynamic compaction will frequently collapse these voids, thereby eliminating localized sources of settlement. The presence of these weak spots is generally revealed during the site work by larger-than-normal craters or greater-than-normal ground subsidence. By making the subsurface profile more uniform, the risk of differential settlement is greatly reduced.

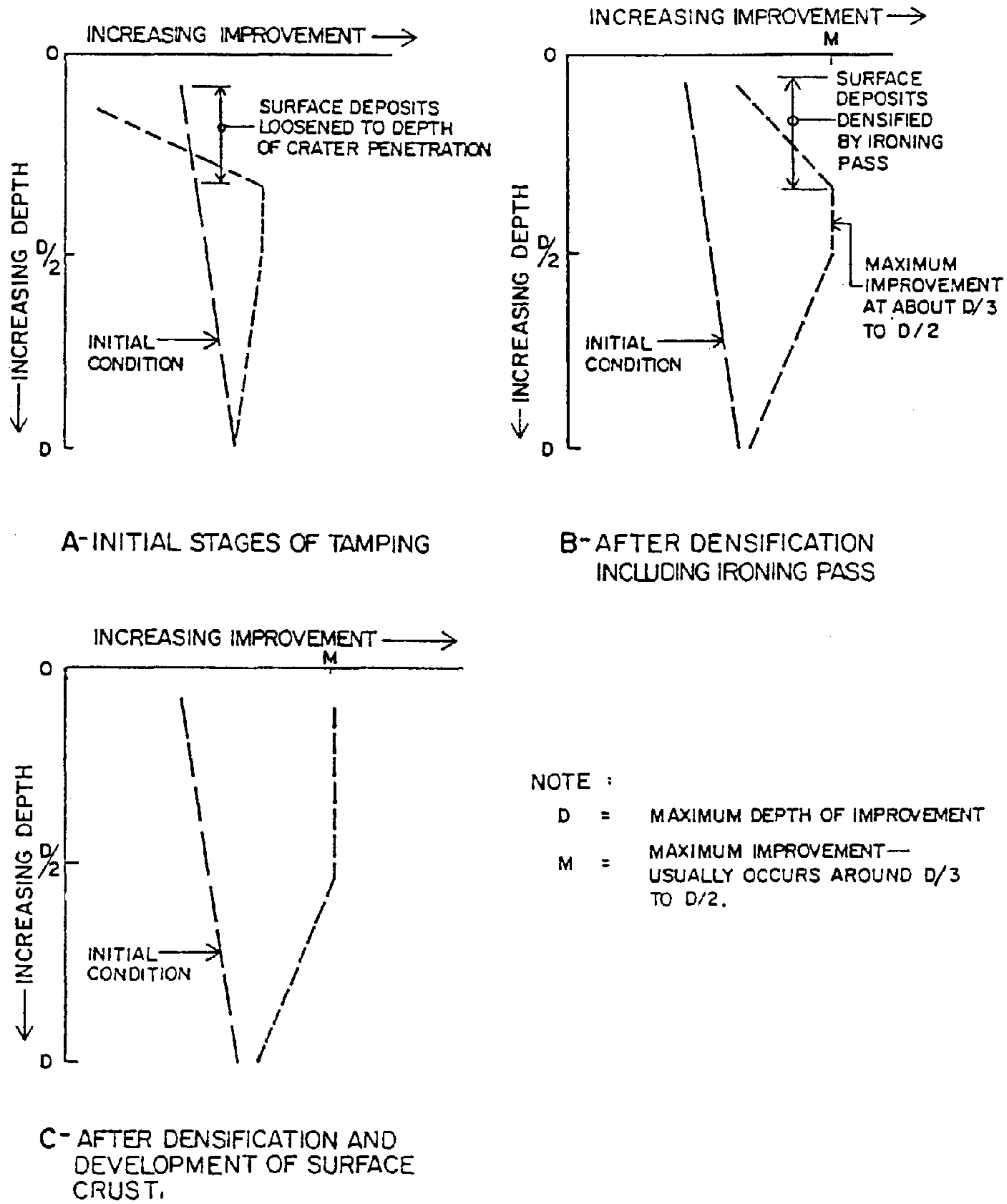
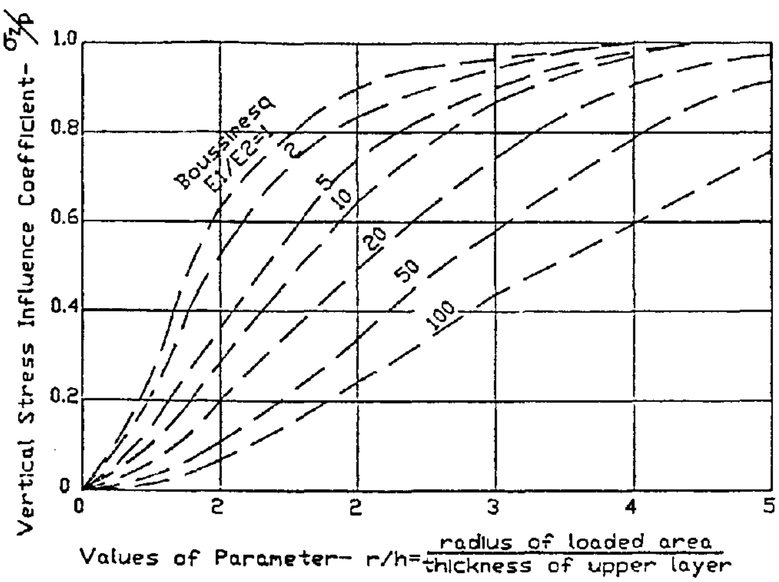


Figure 20 . : Variations in improvements with depth and stages of tamping during compaction

The creation of a relatively uniform layer of densified soil by dynamic compaction is beneficial in distributing stresses imposed by the new loads because the densified layer acts similar to a mat foundation. The soil deposits above a depth of D/2 are precompressed laterally because of the large horizontal stresses imposed in the soil during impacting.

Lukas[54,55,56] measured lateral ground displacements on the order of 6 to 12 in. (15.2 to 30.5 cm) in soils ranging from sands to clayey mine spoil at a distance of 10 ft (3.0 m) from the center of the drop point and 1 to 3 in. (2.5 to 7.6 cm) at a distance of 20 ft (6.1 m) from the center of the drop point. This data is summarized in Chapter 6. These lateral earth movements compress the ground during dynamic compaction in a lateral direction. Burmister[10] has shown mathematically that the pressure imposed on a lower layer of a two layer system is a function of the ratio of the modulus of the upper stiffer layer to the modulus of the lower layer. The vertical pressure induced in the lower layer can be greatly reduced from that normally assumed by Boussinesq. Figure 21 shows the variation in vertical stress coefficient for different values of modulus ratio for a two layer system. It is not unusual following dynamic compaction to develop a well compacted layer over a less compacted layer with a modulus ratio on the order of 3 to 1.

Schmertmann[82] reports on measurements of the horizontal earth pressure coefficient, K, determined by dilatometer tests on a dynamically compacted sand site. At two test sections where the K value was 0.66 and 0.98 before dynamic compaction, these values increased to 1.17 and 1.19, respectively, after dynamic compaction. Figure 22 illustrates the effect of K on a vertical



Basic pattern of two-layer vertical stress curves (σ_z /p vs r/h) at the interface-z=h. $\mu 1=\mu 2=0.5$

Figure 21 : Influence of stiff layer in reducing vertical stress (10)

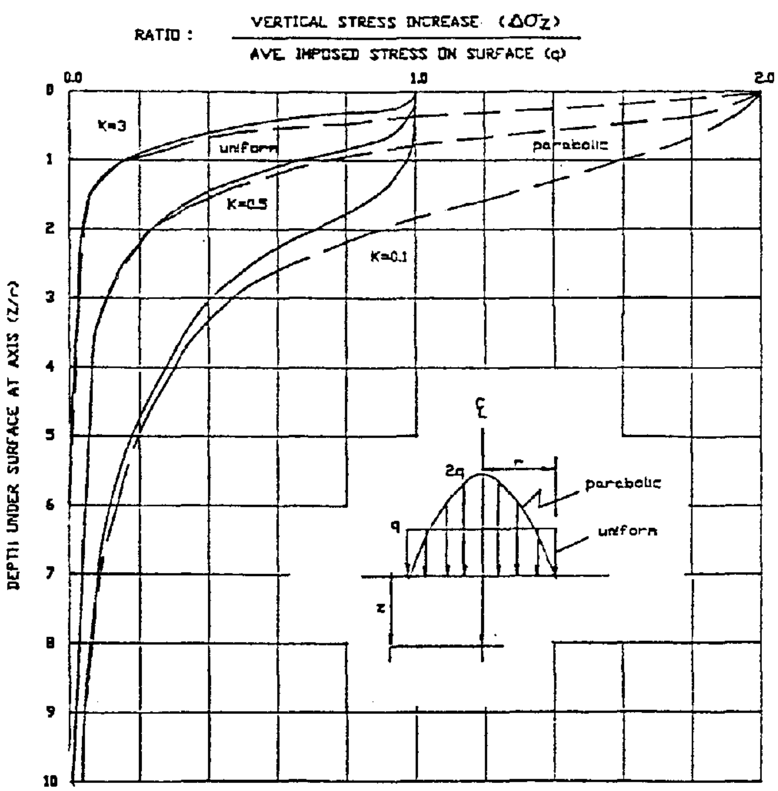


Figure 22 : Vertical stress increase below axis of a circular area load on surface , using probabilistic particulate mechanics , and showing importance of initial K condition(82)

stress increase distribution under the center of uniform and parabolic circular loaded area on the surface of a particulate mass.

d. Limitations on Improvement

Leonards, et.al.[48] indicates that there may be an upper limit to the densification that can be achieved by dynamic compaction. Figure 23 from this paper shows the average cone penetration resistance that was achieved at eight project sites. The cone penetration resistance value was found to increase up to a limiting value of about 154 tons/ft^2 (150 Kg/cm^2) as the amount of energy increased. There appeared to be a good relationship between the cone resistance and the product of the energy per drop times the total energy applied for the entire area.

Mayne, et.al.[59] summarized static cone penetration tests for many of the same sites as Leonards, et. al.[48], but added a few additional sites. This data is shown in Figure 24. The cone resistance increases with applied energy. At an applied energy of 117 ton-ft/ft^2 (350 txm/m^2), which is normally near the high end of energy application for dynamic compaction, the cone resistance is on the order of 185 ton/ft^2 (180 Kg/cm^2).

Based upon a review of a number of projects, typical upper bound test values for SPT, CPT, and PMT tests following dynamic compaction are summarized in Table 8. The highest values occur in the pervious coarse grained soils and the lowest test values in the landfills, typically the weakest of the deposits discussed. A range in the maximum test values is shown because there is some scatter reported in the literature, especially since different amounts of energy were used at different sites. At some projects,

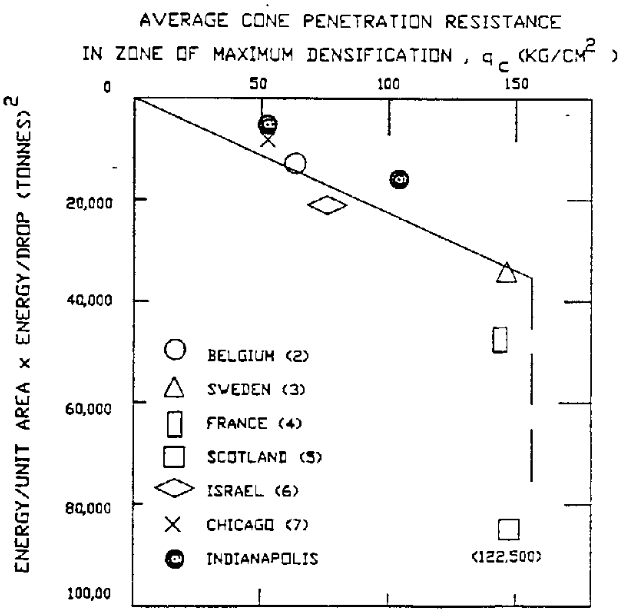


Figure 23 : Cone penetration resistance versus product of energy/unit are times energy/drop (48)

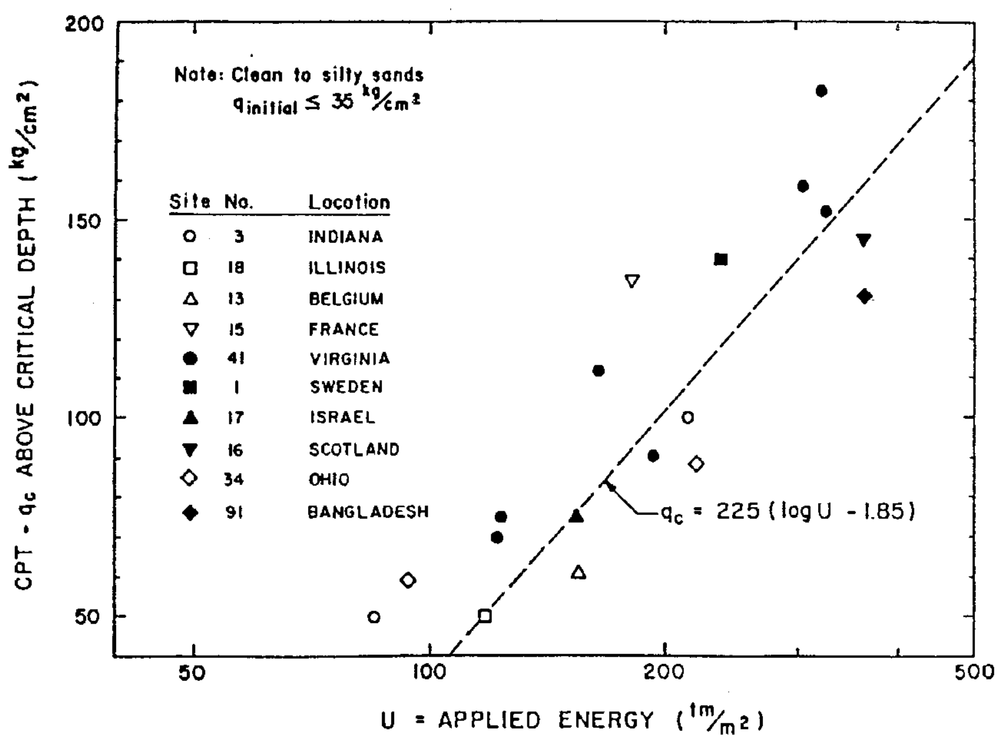


Figure 24 : Increase in cone resistance in granular soils with applied energy(59)

test values higher than the reported maximum values shown in Table 8 were obtained, but only on a localized basis for one or two isolated tests. It is conceivable that these test values were influenced by larger particles within the soil mass or by extremely hard layers which may have been present.

TABLE 8

Upper Bound Test Values After Dynamic Compaction

Soil Type	SPT blows/ft	Maximum Test Value CPT (tsf)**	Limit Pressure PMT (tsf)**
Pervious Coarse Grained Soil - Sands & Gravels	40-50	200-300	20-25
Semi-Pervious:			
Sandy Silts	34-45	140-180	15-20
Silts and Clayey Silts	25-35	100-140	10-15
Partially Saturated Impervious Deposits - Clay Fill - Mine Spoil	30-40*	N/A	15-20
Landfills	20-40*	N/A	5-10

*Higher test values will occur when sampling on large particles present in the soil mass.

**1 ton/ft^2 = 95.8 kN/m^2

The maximum test values shown in Table 8 correspond approximately to point M shown on Figures 20b and 20c. The maximum improvement generally occurs within a zone between D/3 to D/2 with lesser amounts of improvement below this level due to the diminishing effects of the impact energy. Above this zone, the improvement may be less than observed at D/3 to D/2 because of surface disturbance during impacting. The ironing pass may not be adequate to improve the surface soils to the same extent as the deposits within the range of D/3 to D/2.

Table 8 can also be used to determine whether dynamic compaction is necessary at a particular site. If the SPT, CPT or PMT test values are already near the maximum, then the amount of additional improvement might not be enough to justify dynamic compaction. It should be pointed out that the maximum values listed in Table 8 represent improvements from projects where the normal amount of energy of 34 to 100 ton-ft/ft^2 (100 to 300 txm/m^2) have been applied. Some additional improvement could be obtained if greater energies were used.

3. Applied Energy Requirements

In the literature, the applied energy is generally reported in terms of the unit energy applied over the ground surface in tonne meters/meters^2 (txm/m^2). Determination of the precise amount of energy to apply at any given project site is difficult because many factors enter into this decision. These factors include:

- The type of deposit being densified.
- The initial relative density of the deposit.
- The thickness of the deposit being densified.
- The required degree of improvement.

Generally, the greatest amount of applied energy has been used at landfill sites to obtain adequate improvement. This is likely due to the extreme loose nature of the deposit prior to densification as well as to the nature of these deposits which allows numerous voids to exist because of localized bridging within the soil mass. The deposits where the least amount of applied energy is necessary, are the granular pervious deposits. Lukas[54] reports on a loose sand that was dumped into the water to raise the grade approximately 30 ft (9.1 m). The energy required to improve the soils at this site was approximately one-third the energy used to densify landfill sites. Both the loose sand and the landfills could be considered to be in an extremely loose condition prior to densification so the difference in energy requirements is attributed to the difference in soil type. In the case of the sand, some densification must have occurred during placement even though no compactive effort was applied. Furthermore, during dynamic compaction, the ground vibrations induced by the impact probably help densify the sands, whereas, they may not be as effective in landfills.

Depth also enters into the planning for the applied energy. Since some deposits are of limited thickness, less energy is required for densification than for an otherwise equal quality deposit of greater thickness. For this case, the deposit of limited thickness is assumed to be present at and below existing grade rather than buried at some substantial depth below ground surface.

The required degree of improvement that is required for any particular site may be less than the maximum test parameters shown on Table 8 in order for the deposit to behave satisfactorily. As an example, a shallow embankment over a loose deposit may only have to be improved a slight amount to still perform satisfactorily whereas the same deposit would have to be improved much more if spread footings for a bridge were to be placed upon the deposit.

Table 9 presents guidelines for planning the required applied energy taking into account the various factors listed in the previous paragraphs. A distinction is made between the pervious granular soil deposits, the semi-pervious deposits, and landfill deposits. The range in applied energies within each deposit is to compensate for the initial condition of the deposit. More energy is required for those deposits that are initially looser than for those that are initially denser. The depth factor is taken into account by specifying the required energy in tonne meters/meter^3 (txm/m^3). To convert this energy into the more conventional txm/m^2 would require multiplying the suggested values by either the thickness of the compressible zone or the maximum depth of improvement that can be reached for the weight and drop height used, whichever is least. If this amount of energy is applied, then the improvements listed in Tables 7 and 8 are anticipated. If a lesser amount of improvement is acceptable, then the applied energy could be reduced, and the amount of improvement determined by field tests at this lower level of energy.

TABLE 9

Applied Energy Requirements

Type of Deposit	Applied Energy Normally Used
Pervious Coarse Grained Soil - Zone 1 of Figure 5	20 - 25 txm/m^3
Semi-Pervious Fine Grained Soils - Zone 2 and Clay Fills Above the Water Table- Zone 3 of Figure 5	25 - 35 txm/m^3
Landfills	60 - 110 txm/m^3

NOTE: Standard Proctor energy equals 60.5 txm/m^3
1 txm/m^3 = .10225 ton-ft/ft^3

To illustrate the use of this table, an example is presented. Two sites are situated close to each other and both consist of a loose sand deposit that one could classify as a pervious soil formation. At Site A, the deposit is 15 ft (4.6 m) thick and at Site B the deposit is 28 ft (8.5 m) thick. The first step is to compute the required drop height and tamper to compact the thickest deposit which is at Site B. Using a 16.5 ton (15 t) tamper with a 65.6 ft (20 m) drop and an empirical coefficient, n, of 0.5 would result in a predicted depth of improvement of 28 ft (8.5 m), so this tamper and drop height are selected for the project. Table 9 suggests an applied energy of 2.0 to 3.1 ton-ft/ft^3 (20 to 30 txm/m^3). Using an average value of 2.6 ton-ft/ft^3 (25 txm/m^3), the suggested energy for Site A where the deposit is 15 ft thick (4.55 m) would be 37.9 ton-ft/ft^2 (113 txm/m^2). At Site B where the deposit is 28 ft thick (8.55 m) the suggested energy would be 70.8 ton-ft/ft^2 (211 txm/m^2). These energies should be rounded off to some convenient number such as 40 ton-ft/ft^2 (125 txm/m^2) for Site A and 75 ton-ft/ft^2 (225 txm/m^2) for Site B. Using this amount of energy for a sand deposit, one would expect the relative improvement and limiting values shown in Tables 7 and 8. If less than this amount of improvement is required, then the energy could be reduced. In any case, a minimum size tamper and drop height should be used to achieve the predicted depth of improvement for the deposit under consideration. It would not be possible to achieve densification of the 28 ft (8.5 m) thick deposit at Site B if too small of a weight or small drop height were used, even though the required amount of energy were applied by dropping this weight the proper number of times to achieve the average energy requirement.

It should be pointed out that Table 9 is to be used merely as a guide since soil deposits will respond differently to dynamic compaction and adjustments are necessary in the field at specific project sites.

CHAPTER 4 - PLANNING THE FIELD PROCEDURES

1. General Considerations

The manner in which the energy is applied to the ground can affect the depth and degree of improvement that is achieved. The object of the field densification program is to attain the deep seated improvement during the initial tamping followed by the surface improvement during the later stages of tamping. The method in which the energy is applied should therefore be planned with the following considerations:

- The application of too much energy within a short period of time in a limited area should be avoided in deposits where time is required for dissipation of excessive pore water pressures. Additional energy applied before dissipation of excess pore water pressures causes plastic deformation without densification. The energy that is applied at this time is wasted energy. This condition generally occurs in semi-pervious deposits such as silty fine sand and silts. In the coarser granular soils or building rubble, excess pore water pressures either do not occur or generally dissipate very quickly after impact. In these deposits, the energy can usually be applied in one or two passes with close spacing between craters.

- The energy should be applied in such a manner that a crust of hardened material is not formed near the surface before the deeper densification is achieved. This is especially important for granular soils deposits. If the energy is applied at too close spacing or if a lighter weight than required to achieve the deep densification is initially applied to the area, a crust of hardened surface deposits may develop. This crust severely limits deep densification from later tamping. It is also possible that a hardened crust may be present before tamping is initiated, and this would tend to distribute the energy. In this case, it may be necessary to break or loosen the surface layer. Sometimes repeated drops in the same print is sufficient to fracture a cemented layer.

- Very deep craters should be avoided. Deep crater formations can occur in weak deposits such as recent landfills. There is a possibility that the weight can get stuck within the crater or that cables may snap in extracting the weights due to the side friction or suction forces that develop as the weight is extracted. This is more of a construction problem than a design problem, but it must still be addressed. The usual remedy to counteract deep crater penetration is to place a working mat of granular soil on the order of 2 to 4 feet (0.6 to 1.2 m) thickness over the entire area prior to commencing compaction. When the crater depth exceeds the height of the weight plus a few feet, tamping should stop until the crater is filled or the ground leveled.

- The tamping pattern should avoid encirclement of an area of unimproved ground by densified ground or entrapment of unimproved ground between densified ground and impermeable soil. The densified ground is generally less permeable as a result of tamping so when tamping starts in the unimproved areas, pore water pressures cannot easily dissipate, and the ground water may rise in the encircled area. Steinberg and Lukas(87) report on a water problem in a landfill as the tamping approached the edge of the former clay pit.

2. Test Program

In many situations, it is helpful to conduct a test program in advance of the full scale dynamic compaction operations. The purpose is to determine the ground response to the impacts and evaluate the depth and degree of improvement that can be achieved. Test sections are especially helpful in semi-pervious deposits where the rate of excess pore pressure dissipation is important to construction planning and in unusual formations such as landfill sites where it is difficult to predict the amount of improvement.

The test program should be undertaken with the same type of equipment that is planned for the full scale operation. The size of the test section area should be large enough so as to provide meaningful data. As a guide, the minimum width of the densified area should be 1.5 times the thickness of the deposit that is to be improved with a minimum pattern of 16 prints. Usually the test tamping is undertaken on a square or rectangular area. Only the central portion of the square would be representative of what would occur under full scale operations. Sufficient instrumentation should be installed to monitor excess pore pressures, average induced ground settlements, crater settlement with adjacent ground heave measurements at a few drop points, and ground vibrations during the tamping. Borings taken before and after tamping can be used to evaluate the depth and degree of improvement.

Ideally, test programs should be undertaken as far in advance of production operations as possible so as to allow time for excess pore pressures to dissipate and to evaluate the test results. If heavy equipment is required, such as 100 ton (91 t) capacity crawler cranes, for lifting heavy weights, the mobilization charges for the test section can be quite large. In this case, it may be feasible to undertake the test section at the start of the production operations or even to plan for a week or two delay with the equipment at the site before starting the production operations. When dynamic compaction is undertaken with lighter weight equipment such as 40 to 50 ton (36 to 45 t) cranes, the mobilization charges are not large, and it may be economically justifiable to mobilize and demobilize the rig well in advance of production operations for undertaking a test section.

3. Area to Densify

Densification should be applied throughout the entire area to be used for foundation support plus some distance beyond the edges of the loaded area. For an embankment project, this would include the width of the base of the embankment plus a minimum distance equal to approximately one half of the thickness of the layer to be densified on either side of the edge of the embankment. If densification were to be applied to a structurally loaded area such as a building, the densification should be applied to the building area plus a minimum distance beyond each edge equal to about half the thickness of the deposit to be densified.

In areas that are more heavily loaded than others or where it is desired to further reduce the compressibility of the ground, additional energy can be applied at isolated locations after the entire grid area has been densified. For example, on some building projects, additional densification was applied at column locations after the entire area was tamped on the grid basis[51].

4. Position of the Water Table

If the water table is within about 6.5 ft (2 m) of the ground surface, serious construction problems can develop, and the effectiveness of the densification will be reduced. During impacting, water frequently will rise up into the craters as the excess pore pressures developed during tamping dissipate, see Figure 25. Repeated tamping may cause the upper surface deposits to become liquefied. This weakened upper zone will not densify nor transmit energy effectively to the greater depths because a portion of the energy is wasted in shearing and displacing the weakened surface soils. Therefore, it is necessary to maintain the water table no closer than about 6.5 ft (2 m) below the ground surface from which the impacts are applied.

Figure 25 : Ground water rise into a crater following dynamic compaction

The water table can be controlled either by lowering through pumping from pits or by raising the ground surface by the placement of fill prior to the start of dynamic compaction. Both of these techniques have been used with success.

5. Print Spacing

The distance between prints can vary depending upon the type of soil being densified and the position of the water table. In semi-pervious soils where the water table is high, it is essential that the spacing between prints be maintained at a relatively large distance. Usually, distances on the order of 30 ft (10 m) to as much as 50 ft (15 m) have been used for these deposits. A rule of thumb that has been applied on some projects is to use a print spacing equal to the thickness of the deposit being densified. The reason for using a large spacing is to allow the excess pore pressures to dissipate at each individual crater location following the tamping while tamping is taking place in a distant area. This pore pressure dissipation occurs much more quickly if the spacing between prints is large. After the entire area is densified by one complete pass, the tamping can be resumed at intermediate points with successive passes at a similar print spacing.

In granular deposits where the water table is not close to ground surface, the print spacing can be reduced and the energy applied in only one or two passes. For shallow depth treatment, print spacings have been as close as 7 ft (2 m) and effective densification was achieved in one pass.

6. Drops Per Print

Two factors may control the number of blows that can be applied at a print in one pass. These factors are:

- The depth of a crater following repeated impacts should be limited to the height of the tamper plus a few feet so that the tamper does not get stuck within the crater.

If deep craters occur, either the craters should be filled or the ground leveled before tamping is resumed.

- If heave occurs adjacent to the print, and the volume of the heave approaches the volume induced within the crater, then additional tamping does not produce densification. Large heave generally occurs when excess pore water pressures develop in the underlying soil mass and the soil behaves plastically. This upheaval around the crater is called the "moustache effect". A graphical portrayal of the heave is shown in Figure 26.

  The variation in net effective volume with number of drops at a landfill site(23) is shown in Figure 27. The net effective volume for each blow is computed by calculating the crater volume and subtracting the heave volume. The heave volume is based upon measurements taken of displacement stakes embedded about 1 ft (0.3 m) below ground surface. To obtain sufficient data, at least four stakes should be set just beyond the edge of the print to distances of 16 ft (4.9 m) from the centerline of the prints in four quadrants. At this site it was determined that beyond the 11th drop, the heave volume approximated the crater volume, so any additional drops would not be effective in producing densification. A suggested method for calculating crater and heave volumes is presented in Appendix D.

7. Number of Passes

The number of passes that are needed is somewhat dependent on how the ground responds to the impacts. If only a few tamps can be applied before the crater depths become too deep, or if high

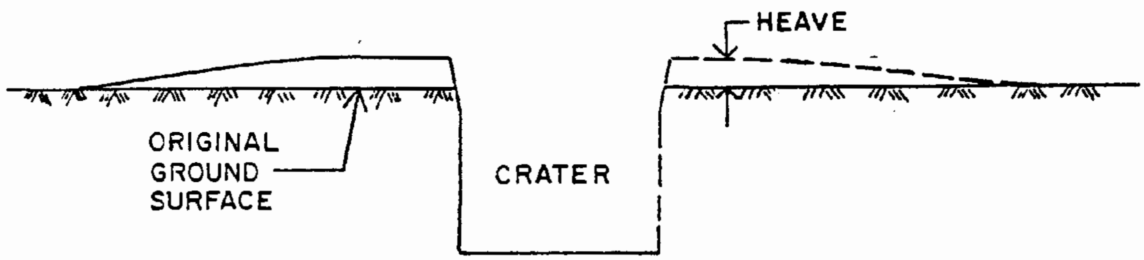


Figure 26 : Typical heave pattern

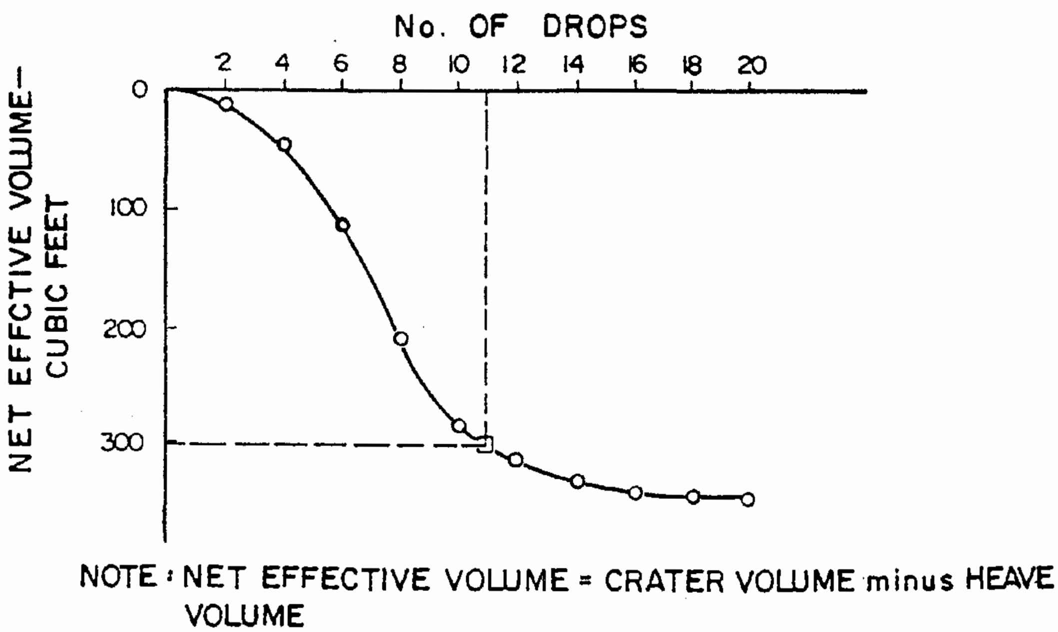


Figure 27 : Determination of effective number of drops from a heave test (23)

excess pore water pressures occur following only a few tamps, then more passes will be required. Generally, pervious deposits can be densified after only 1 to 2 passes, whereas, semi-pervious deposits, especially those with a high water table, may need as many as 6 to 8 passes.

The first pass over the area should be utilized as a trial or exploratory pass. The information gathered following the first pass can be used to help plan the application of energy for the subsequent passes. As it is not possible to predict precisely in advance how deep the crater depths will be or the magnitude of the excess pore pressures following impact, the first pass will reveal much about the ground support conditions. Furthermore, the ground support conditions can vary across the area, and if weak spots are present, they will generally be revealed following the first pass. The weak spots will result in greater than normal crater depths.

8. Ground Leveling and Surface Compaction

After each pass, the upper portion of the soil mass is completely disturbed as a result of crater formations and heave between craters, Figure 28. The ground should be leveled using a dozer to blade the soil from between craters into the craters. The area should be track rolled and elevation readings taken across the area. The leveling provides a smooth surface for access of the rig to apply the next level of tamping. In weak ground such as landfills, it may be better to fill the craters with coarse granular fill rather than level the ground between passes because of the difficulty of traffic mobility on these sites. In addition, relatively deep craters can occur and if surface materials are pushed into craters, this may expose a weaker and possibly wetter underlying layer at locations between points.

Figure 28 : Loosened ground surface following dynamic compaction

After the last pass, the surficial zone will remain uncompacted. Compaction can be accomplished with normal compaction equipment if the loose zone is not too thick. On many jobs, surface compaction is undertaken with a low energy drop of a low contact pressure tamper. This final pass is called the ironing pass. The tamper is dropped at a close interval or overlapping spacing without developing craters.

9. Pore Water Pressure Monitoring

Whenever dynamic compaction is undertaken on semi-pervious deposits, piezometers should be installed and monitored during tamping. Significant excess pore pressures can develop following repeated impacts, and the data obtained from the piezometers can be used to control the tamping sequence so as to allow the excess pore pressures to dissipate before additional energy is applied. Field testing such as pressuremeter testing must be delayed until the excess pore pressures dissipates in order to get an accurate indication of material properties.

Open standpipes respond too slowly and frequently are severed as a result of nearby tamping, so their usefulness is limited. Hydraulic or pneumatic piezometers placed in a sealed borehole are preferred.

In addition to monitoring pore water pressures, other types of field monitoring are frequently undertaken. Measurements of ground vibrations and lateral deformations are discussed in Chapter 6.

CHAPTER 5 - MONITORING THE IMPROVEMENT

1. General Considerations

Ground improvement is generally assessed by indirect methods as dynamic compaction is underway and by soil borings with conventional sampling after the full energy has been applied. The measurements taken as the work is underway are called production control tests and include monitoring of the crater depths and average ground settlement following each pass. After all the passes are completed, more specific information is obtained by the performance of field tests, usually conventional sampling conducted in boreholes, and the results compared with the data obtained prior to commencing dynamic compaction.

2. Production Control Tests

When the weight impacts into the ground, it can be considered both as a probing tool and as a corrective mechanism. Where weak spots are present in the ground, the heavy impacts will cause greater than normal crater depths or greater settlement in this region than in adjacent areas which are in a more dense condition. Information gathered from these crater measurements or ground measurements can be used in planning the second or third passes of the dynamic compaction project as well as determining in an indirect manner if the ground improvements are occurring.

a. Crater Depth Measurements

All the crater depths should be measured and plotted as the work progresses. If the craters that are formed following impact are irregular in shape, diameter and depth readings can be taken of each crater so that crater volumes can be plotted. A contour plot of crater volumes following three passes for a project in Florida is shown in Figure 29[94]. From this plot, two localized areas where greater than normal crater volumes have occurred are evident and these are indicative of weak spots. Additional densification was applied in these areas to correct the weak ground. The decision to monitor only crater depths or to measure crater volumes with more accuracy depends upon the complexity of the job.

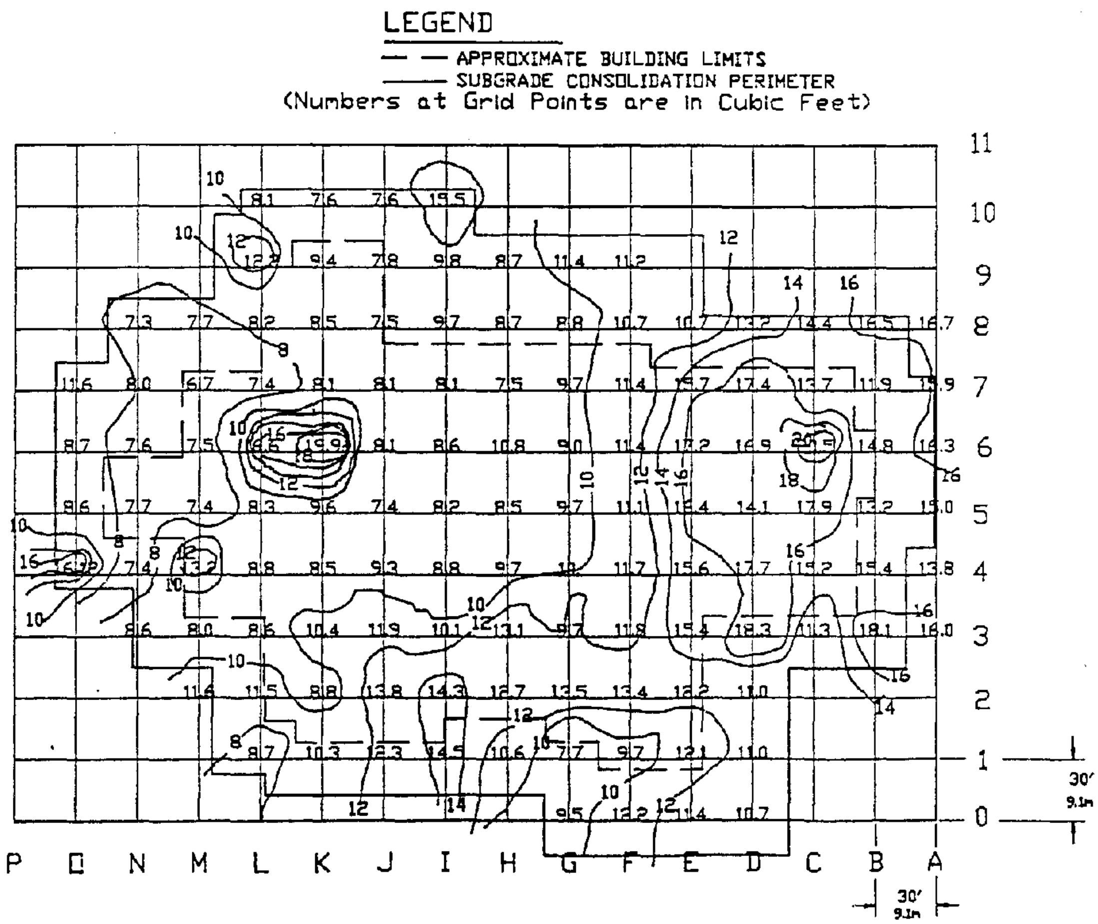


Figure 29 : Dynamic settlement chart. Phases 1,2 & 3(94)

b. Average Ground Settlement

Following a complete pass over an area, the ground surface should be leveled using a dozer to blade the soil from between craters into the craters. The area should be surface compacted with a dozer and elevation readings obtained. The average ground settlement is an indication of the improvement achieved. In very dense formations, average ground settlements may be only on the order of a few inches, but in loose deposits, the average ground depression could be in terms of many feet.

The elevation readings should be obtained on a grid basis throughout the area. In this manner, the average ground depression in one portion of the site can be compared with other portions. This would be a further indication of a weak ground condition in a particular area.

After all the passes are completed, the typical average ground settlement is usually on the order of 5% to 10% of the thickness of the densified zone. The range in ground settlements will be dependent upon the density of the ground before improvement as well as the amount of energy applied during dynamic compaction. In extremely loose deposits, such as recent landfills, average ground depressions of 20 to 25% of the original thickness have been reported[96].

On projects where the ground surface is weak and difficult to support the equipment, fill may be brought in to raise the grade following each pass of dynamic compaction. In this case, the volume of fill brought in should be documented in order to yield information about the amount of ground depression.

3. Field Tests After Dynamic Compaction

The best way to measure ground improvement is to perform field tests before and after dynamic compaction. The amount and depth of improvement can be determined by this means. If the contract has been let based upon achieving a minimum test value, the borings after completion of the work can then be considered verification borings.

a. Conventional Tests

The usual methods include SPT, CPT or PMT tests. If these tests are performed at approximately the same locations before and after dynamic compaction, the amount of improvement can be directly compared. The validity of these tests after dynamic compaction would be the same as the validity beforehand. That is, if the designer predicted a bearing capacity and settlement based upon a SPT value before dynamic compaction, the SPT value after dynamic compaction would be used to calculate the improved bearing capacity and settlement.

The most applicable type of test depends upon the soil type and the preference of the design agency or the consultant. Those who use SPT values for estimating bearing capacity or predicting settlement, comparison SPT values before and after testing would be acceptable. SPT testing is currently the primary method for determining the liquefaction potential of granular soils so it would be useful for these purposes.

Many designers prefer PMT tests for monitoring dynamic compaction. This test has some definite advantages because it stresses a large zone of soil which is helpful in coarse grained deposits or fills. The presence of large particle sizes is averaged out by the stiffness of the matrix. Furthermore, a modulus is obtained which is an indication of the compressibility of the deposit from which settlements can be readily predicted.

CPT tests are frequently used to indicate the degree of improvement by comparing the cone tip resistance values obtained before and after dynamic compaction. Some difficulties have been experienced with obtaining reliable cone values when the deposit contains large particle sizes such as might be present in mine spoil fills or landfills. Similar but less severe problems can occur for SPT and PMT testing.

On many projects, more than one test is used to monitor the improvement. PMT and SPT tests can be performed within the same borehole as it is being extended through the densified ground. The improvement can then be assessed by comparison of both test values before and after dynamic compaction. Generally, when two types of tests are used to monitor the improvement, the relative degree of improvement is similar. The relative degree of improvement refers to the ratio of a test parameter at a specified depth after improvement to the test value before improvement. Ramaswamy, et.al.[79] performed CPT, SPT and PMT tests at three different sites where sandy soils were present. These tests were performed before and after dynamic compaction. The degree of improvement for all three tests was very close at two sites, but only approximately the same at the site where calcareous sand was present. Figure 30 shows the SPT, CPT and PMT tests before and after dynamic compaction at one of the sites.

The variation in degree of improvement with depth as measured by SPT, CPT and PMT tests for three sites are shown in Figures 31a to 31c[54, 55, 56]. The subsoil at site in Figure 31a consists of a loose dumped fine to medium sand. There was excellent agreement in degree of improvement between the SPT and PMT tests with a slightly higher ratio of degree of improvement measured by the CPT tests. The data shown in Figure 31b is for a clayey mine spoil containing chunks of sandstone and shale. The CPT data was

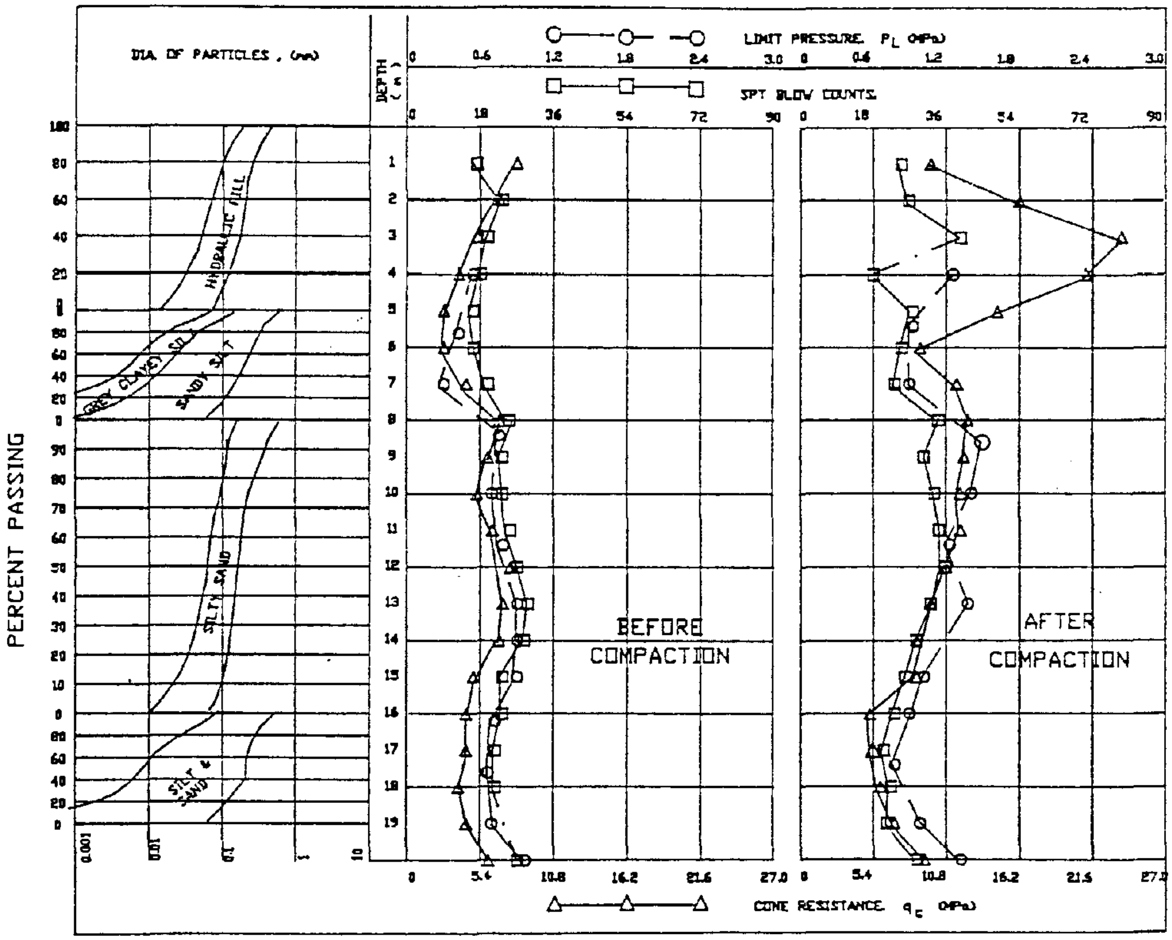
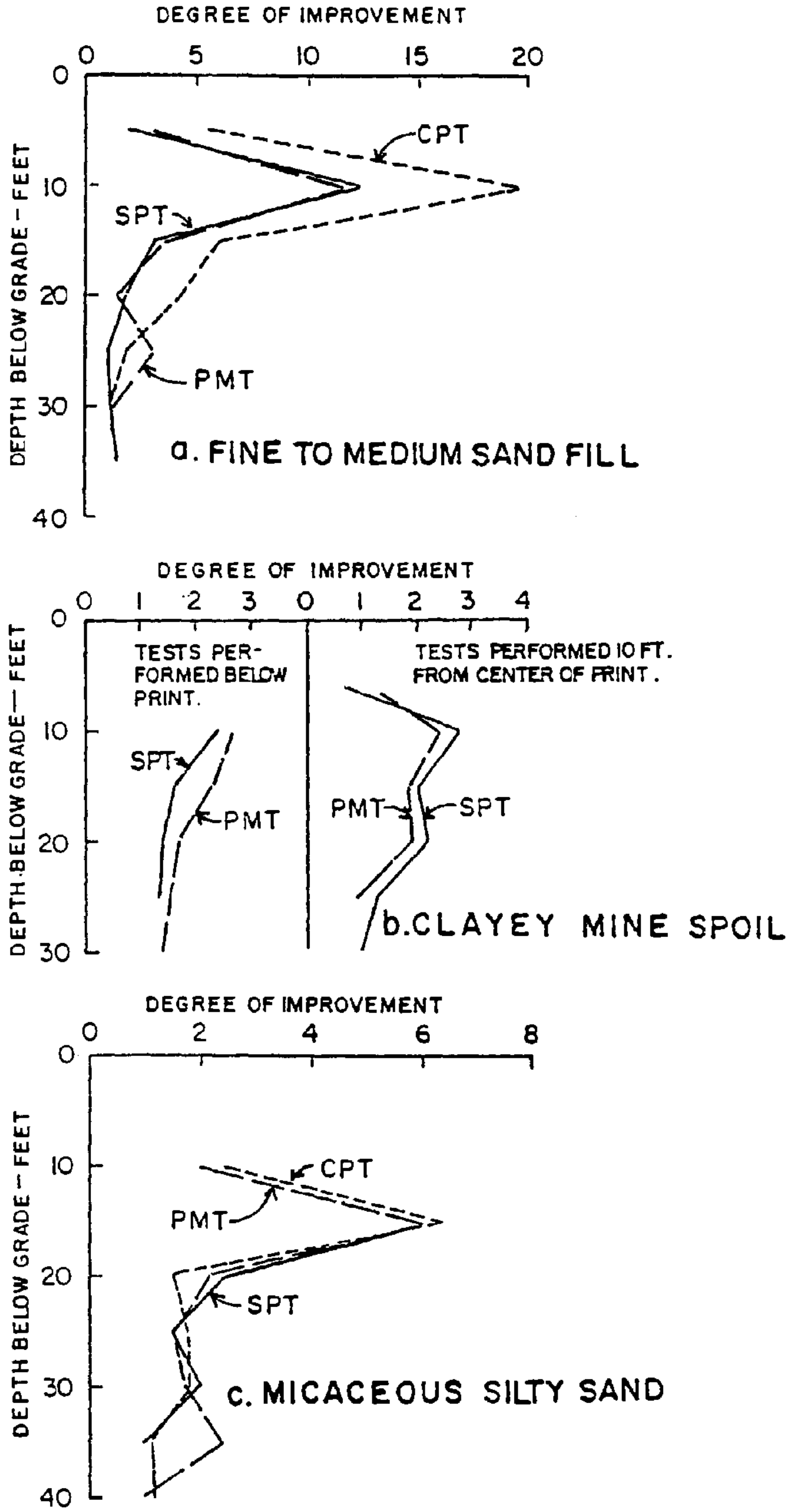


Figure 30 : Correlation of PMT , CPT and SPT.
Ashuganj fertilizer factory , Bangladesh (79)

(1m = 3.28ft; 1 tonne = 1.1 ton)



(1ft = 0.305m)

Figure 31 : Degree of improvements measured by : SPT , CPT and PMT test

greatly affected by the large particle sizes, both before and after dynamic compaction so it is not plotted on this figure. The degree of improvement as measured by the SPT and PMT tests indicates a similar amount of improvement at points directly beneath the drop and at a horizontal distance of 10 ft from the point of impact. The data shown on Figure 31c is for a micaceous silty fine sand and the degree of improvement as measured by the CPT, SPT, and PMT tests are remarkably similar.

It is often helpful to use more than one method of testing for a project site since one method may be determined to be inappropriate, such as the CPT testing at the mine spoil site discussed above.

The number of borings and tests to perform to evaluate the effectiveness of dynamic compaction is difficult to specify. Many factors influence the decision such as:

- Type of Soil Deposit - In coarse grained deposits such as boulders and broken rock, mine spoil and building rubble, it is oftentimes very difficult to advance boreholes and perform field tests. A greater reliance is frequently placed on other measuring techniques such as average ground settlement. Some borings including conventional testing (CPT, SPT, PMT) should be planned to obtain comparison data.

- Uniformity of densified deposit - In erratic or stratified deposits, more borings should be planned than in uniform deposits so as to obtain a significant number of tests in each soil stratum.

- Sensitivity of the Structure - If densification is undertaken for an embankment and settlements of 1 to 2 ft (.3 to .6 m) are acceptable, fewer borings and tests are necessary than would be for a bridge pier or a building where settlements must typically be limited to less than 1 to 1.5 in. (2.5 to 3.8 cm).

The following guidelines are suggested to aid in planning the exploration program following dynamic compaction:

- A minimum of one boring with in-situ testing should be planned for a) every 10,000 square feet (929 m^2) of area that was densified for a structure, such as building or bridge pier, and b) every 40,000 square feet (3716 m^2) of embankment. However, there should be at least two borings with testing regardless of the size of densified area.

- The borings should extend through the deposit to be densified and in-situ testing should be performed at no greater than 5 ft (1.5 m) depth intervals to determine the degree and depth of improvement.

- Additional borings and tests should be performed in areas where anomalies were observed during the densification. This would include areas where the crater depths were significantly greater than average or where the ground settlement was greater than normal. These tests are necessary to confirm that the substandard area has been improved to the same level as the adjacent areas.

- The exploration program should be undertaken only after excess pore water pressures have dissipated and preferably a short period of time after that. In pervious soils, this work could be undertaken within days after densification, but in semi-pervious deposits pore pressure devices should be monitored and the borings undertaken when the readings indicate excess pore pressures have dissipated. Additional comments on time for testing are presented later in this chapter.

b. Alternative Methods of Measuring Improvement

There are methods for measuring ground improvement other than the conventional tests. These methods have not been used extensively and in some cases, interpretation of the results is somewhat difficult. These specialized methods would include:

- A measurement of the increase in unit weight of the deposit. This is usually done by obtaining samples with shelby tubes or with liners placed with a split-barrel sampler. Samples are obtained before and after dynamic compaction, and the increase in unit weight evaluated.

- Deceleration Readings - If an accelerometer is attached to the tamping weight, it is possible to measure the deceleration when the weight strikes the ground. From this information, it is possible to calculate a spring constant of the subgrade soil. Hansbo[42] correlated deceleration readings with actual load test results and was able to demonstrate a similar load and deflection relationship between the load test and interpretation of the deceleration readings. The deceleration technique was used at this site because large rockfill was dumped into the sea to raise the grade. Conventional tests within boreholes could not be undertaken in these deposits. A more thorough discussion of the interpretation of deceleration readings is presented in Appendix C of this report.

- Load Test - On some sites where the deposits are extremely heterogeneous, such as recent landfills, load tests have been used to measure the improvement in the load supporting capability of the subgrade. Settlement plates can be installed and earth berms built on top of the landfill. Readings are then taken as a function of time and compared with a control load test performed on unimproved ground.

- Cross Borehole Seismic Test - It is possible to measure the increase in shear velocity after dynamic compaction by performing cross borehole seismic tests. Unfortunately, these tests are often time consuming and expensive so this procedure has not been used extensively.

- Other Recently Developed Geotechnical Devices - Other geotechnical measuring tools can also be used. This would include some of the more recent devices such as the dilatometer or the screw plate. However, interpretation of these results is sometimes difficult. It is anticipated that as these devices become more widespread, their use in evaluating dynamic compaction may become more commonplace.

4. Improvement With Time

Following dynamic compaction, the properties of the densified ground as measured by SPT, CPT or PMT tests improve with time. The delayed improvement is especially pronounced in the fine grained deposits but also has been observed in sands. At sites where pore water pressure measurements have been taken, improvements have occurred after the excess pore pressures have dissipated.

In the sand and fine grained deposits, the energy imparted by dynamic compaction frequently destroys the fabric and remolds the soil. The strength after tamping is initially lower and the compressibility higher than before. Lukas[51] reports on this phenomenon for a clayey silt deposit, Figure 32. Improvement in pressuremeter parameters were observed 70 days after tamping even

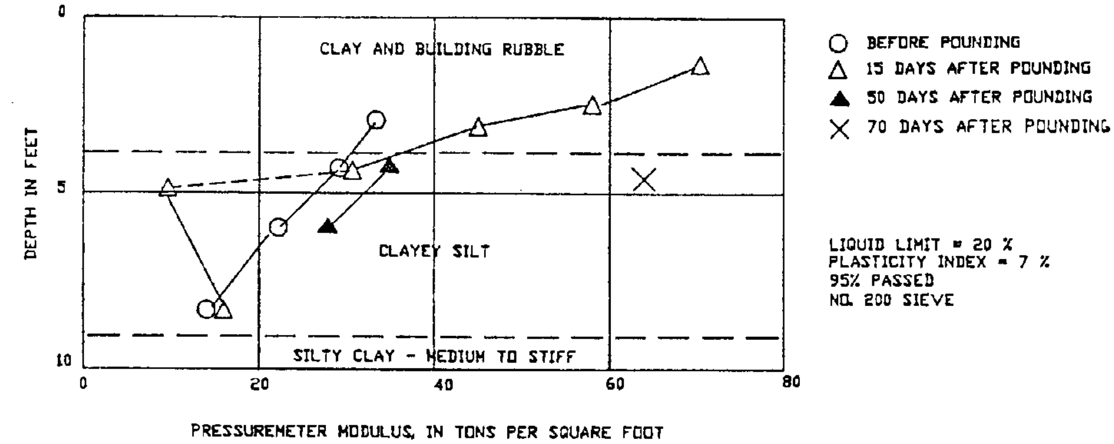


Figure 32 : Pressuremeter tests in clayey silt : Site 8 (51)

(1 ft.=0.305 m : 1 ton/sq. ft. - 95.8 kPa)

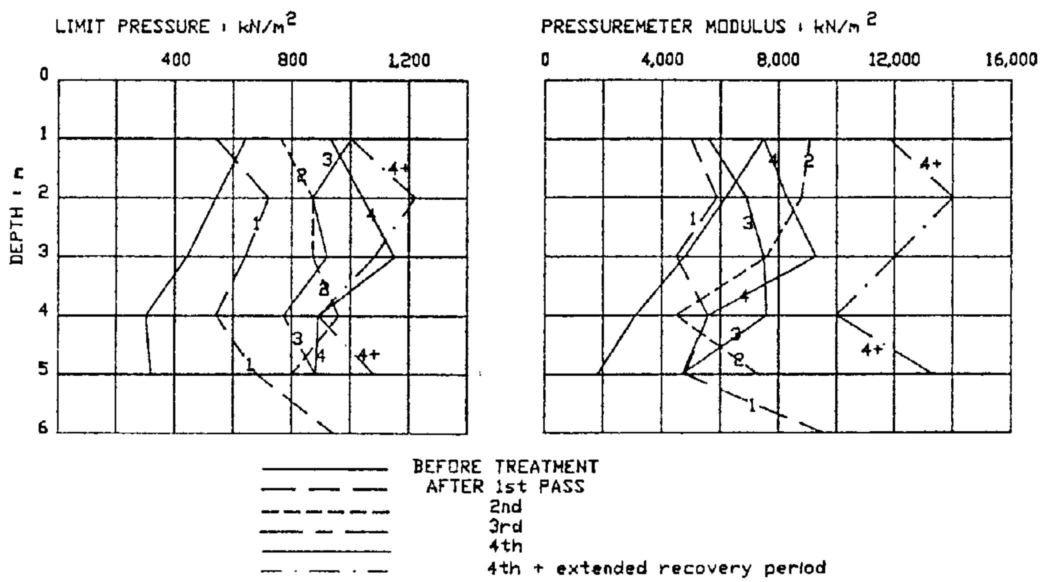


Figure 33 : Housing development ; comparison of improvements to sucessive 1.0 m layers with consecutive tamping passes(66)

(1 tsf = 95.8kN/m^2; 1m = 3.28ft)

though excess pore water pressure dissipated 40 to 45 days after tamping. Moseley and Slocombe[66] measured an increase in pressuremeter modulus, Figure 33, after an extended recovery period in a clay fill.

Mitchell and Solymar[65] report on a delayed increase in strength within an alluvial sand deposit that was densified by means of blasting. The cone penetration test values obtained 9 days after blasting were lower than the initial test values even though there was significant ground subsidence following the blasting. After eleven weeks, the cone penetration resistance of the deposit improved significantly. Mitchell postulates that the strength gain is due to a recementing of the particles from dissolution and precipitation of silica films over an extended period of time.

Schmertmann, et.al.[83] reports on an improvement in CPT values that occurred in a deposit of fine sand with trace of silt. Above a depth of 33 ft (10 m), improvements in CPT values were observed over a period of sixty days even though excess pore pressures dissipated within hours after completion of dynamic compaction. At locations where two drops of the 32 ton (29 t) weight were used, the CPT values after 60 days were approximately 1.35 times higher than CPT tests prior to dynamic compaction. Where four drops were used, the CPT values were approximately 1.8 times higher and where six drops were used, the CPT values were approximately 2.4 times higher. A plot of these test results is shown on Figure 34. The higher ratio of improvement that occurred where more drops were used is attributed to greater initial or primary densification from the higher applied energy rather than from secondary effects. The authors conclude that the long-term CPT test results can be assumed as 1.35 times the test value obtained shortly after dynamic compaction.

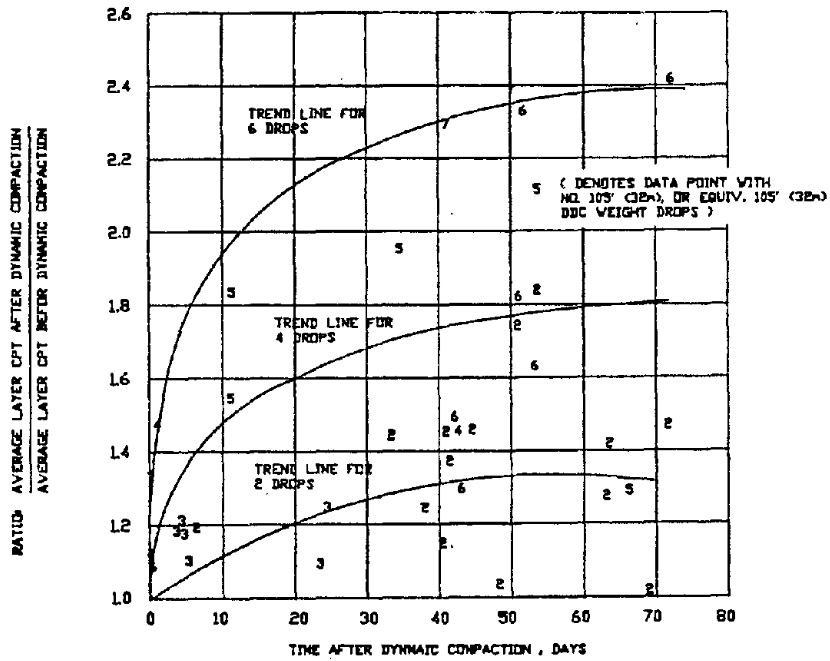


Figure 34 : Effects of time on the relative improvements in CPT test values from soil in depth range of 2 to 8 m (83)

Mesri and Godlewski[62] state that all deposits undergo secondary compression following primary consolidation. This applies for granular soils as well as cohesive soils. Secondary compression is defined as the change in void ratio with the log of time.

Since dynamic compaction results in a reduction in void ratio frequently accompanied by development and dissipation of excess pore water pressure in the soil mass, this densification produces an effect similar to primary consolidation. Thus, when dynamic compaction is complete and excess pore water pressures dissipated, secondary compression should follow.

The magnitude of the improvement during secondary compression is difficult to predict since only the change in void ratio can be calculated with time, and usually, soil property improvements are measured by SPT, CPT, or PMT tests.

Some idea of the anticipated magnitudes of improvement can be obtained by calculating the change in relative density during secondary compression for different types of sands and silty sands. Using values of secondary compression index, suggested by Mesri and Godlewski(62), plus typical values of maximum, minimum and in-situ void ratios, the change in relative density computes to be on the order of 1/2 to 1-1/2% following dynamic compaction. This is not sufficient to explain the magnitude of improvements normally obtained. Thus, while secondary compression results in a decrease in void ratio and a corresponding increase in soil properties, other factors such as particle reorientation following primary consolidation, additional primary consolidation under the overburden pressure accompanied by low excess pore water pressures that are not measured in the field or recementation are the more likely reasons for the significant increase in soil properties following dynamic compaction. This delayed improvement also occurs for conventionally compacted soils where post construction settlement of compacted fills is recognized(71).

Because of the gain in strength with time, it is advisable to delay testing as long as possible, even in sandy soils. If testing must be performed immediately after dynamic compaction, provision should be made for some additional tests after an extended period of time.

CHAPTER 6 - OFF SITE EFFECTS

1. General Comments

When heavy impacts are applied to the ground surface, adjacent areas can be affected. Ground vibrations are transmitted significant distances from the point of impact and may travel off site. Lateral ground displacements adjacent to craters could result in displacement of utilities within the streets or of permanent structures such as retaining walls at the edge of the property. During construction, airborne debris can cause damage or injury unless precautions are taken to minimize the travel.

2. Ground Vibrations

When the weight strikes the ground, vibrations are transmitted to varying distances from the point of impact. When doing the tamping within the interior of a large site, the off site effect of the ground vibrations is usually negligible. However, when tamping is done near the edges of the property in developed areas, ground vibrations can be transmitted into adjacent facilities and in some instances may cause annoyance or damage.

It has been found by U.S. Bureau of Mines studies, Nicholls, et.al.[72], that particle velocity is more closely associated with damage to structures than either displacement or acceleration. Therefore, particle velocity is generally measured at ground surface adjacent to existing facilities and used as an indicator of potential damage. Mayne[60] emphasizes that the particle velocities should be reported in terms of the maximum single component amplitude or the true vector sum, which is the vector summation of the horizontal, transverse, and longitudinal components at the same point in time. It is also important to measure vibration frequencies, because frequency enters into the damage criterion.

Lukas[51, 54, 56] and Steinberg and Lukas[87], have accumulated information regarding the particle velocity measured at ground surface from dynamic compaction operations at varying distances from the point of impact in different soil types. This data is shown in Figure 35. The magnitude of ground vibration increases with the square root of the energy applied and decreases as the distance from the point of impact. This figure can be used for planning purposes.

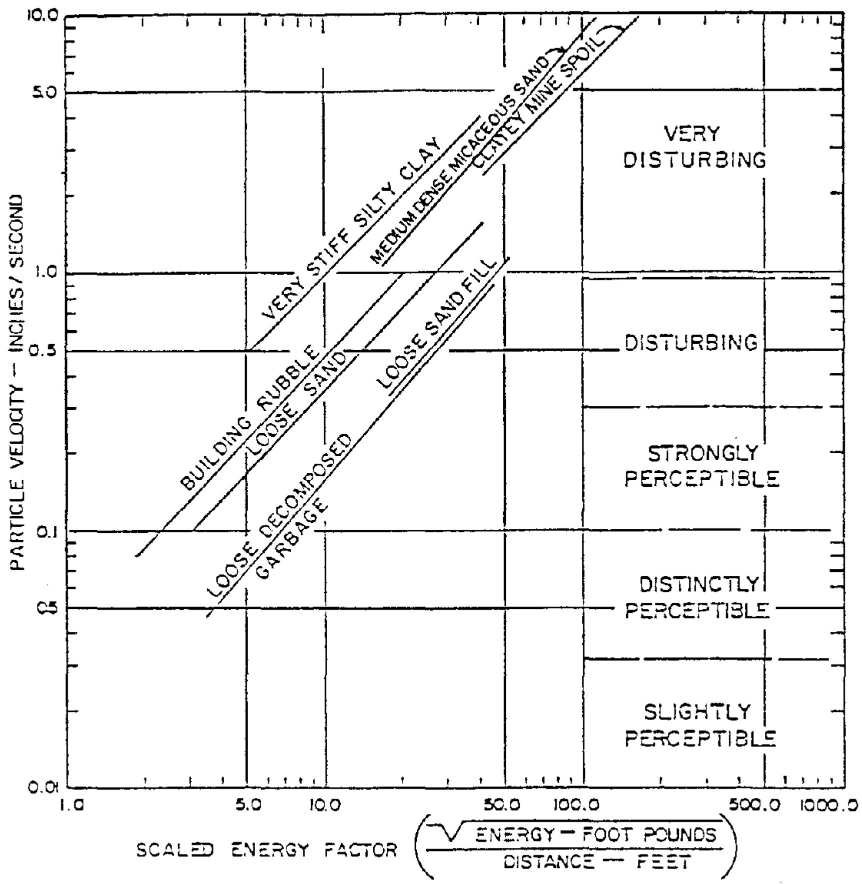


Figure 35 : Scaled energy factor vs particle velocity

(1in/sec = 2.54cm/sec; 1ft-lb = 1.36 Nm)

The values of peak particle velocity predicted from Figure 35 were obtained as densification was being undertaken on a site and with the readings taken in an area that was not yet improved similar to that which might occur for an off-site condition. The peak particle velocity shown on this figure could increase if readings were taken within an area where the densification has been completed. Dumas and Beaton[25] present information showing how the peak particle velocity increases from .16 to .51 in/sec (4 to 13 mm/sec) as the number of drops increases from 1 to 21. These readings were obtained at a point located 66 ft (20 m) from the imprint where a 20 ton (18.1 t) tamper was being dropped over a height of 98 ft (30 m) onto a silty rock fill. Thus, some variation in predicted peak particle velocity can be anticipated, depending upon whether the readings are taken off site where no compactive energy is applied, during the initial stages of dynamic compaction, or during the final stages of dynamic compaction.

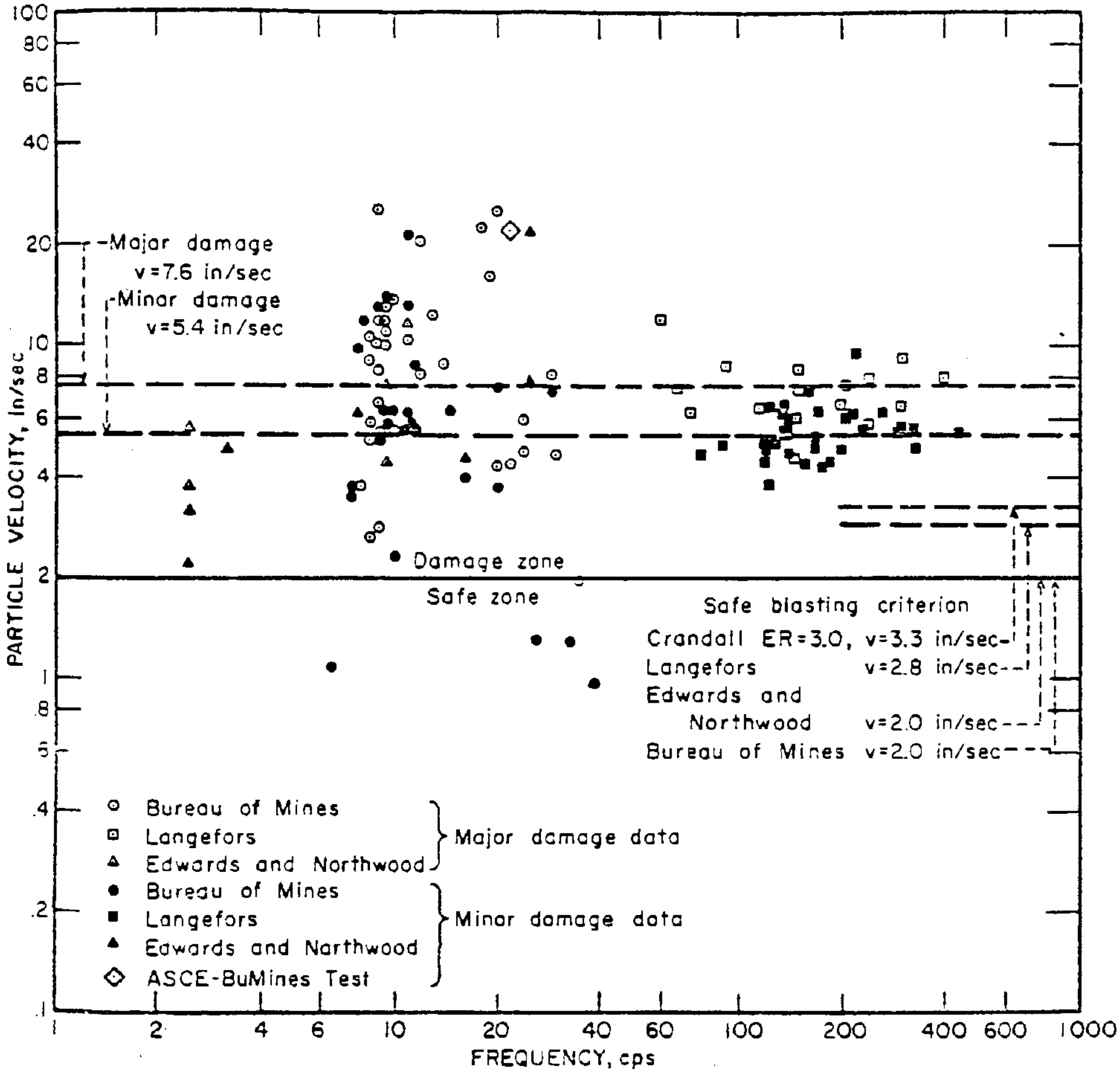
On a particular project, ground vibrations should be measured at locations where the vibrations may be critical to an off site structure. Strata layering and geologic discontinuities such as joints, slickensides or buried pipelines can cause multiple seismic wave reflection and refraction. Ground water also forms an additional boundary, with different wave propagation velocities above and below the water table.

In some circumstances, vibration levels can be reduced if isolation trenches are dug between the point of impact and the area to be protected. Thompson and Herbert[92] found a reduction in vibration level by a factor of 20 when 9.8 ft (3 m) deep trenches were dug between the impact area and a bridge abutment. The tamping was undertaken with a 10.5 ton (9.5 t) weight. Varaskin[95] has accumulated data from various sites indicating that the peak particle velocity can be reduced by a factor of about 2 if a cutoff trench is dug to a depth of 5 to 6 ft (1.5 to 2.5 m) between the point of measurement and point of impact.

Varaskin states that the observed damping is greater than theory would predict. He states that this may occur because theory neglects the effect of lateral displacement caused by penetration of the pounder and the presence of a stiff crust of soil that is typically encountered above the water table at most sites.

The level of particle velocity which can be tolerated off site depends on whether the ground vibrations are critical for building damage or whether the ground vibrations must be kept below a certain level for human response. The Bureau of Mines[72] has concluded, based upon a 10 year study of monitoring blasting operations, that a peak particle velocity limit of 2 in/sec (51 mm/sec) as measured from any of three mutually perpendicular directions in the ground adjacent to a structure should not be exceeded if the probability of damage to the structure is to be small (Probably less than 5%). Figure 36 illustrates the results of the readings taken on different structures over a 10 year period and their relationship to the 2 in/sec (51 mm/sec) damage criteria. Numerous dynamic compaction projects have been undertaken where readings in the range of 1 to 2 in/sec (25 to 51 mm/sec) have occurred at the ground surface adjacent to occupied structures without damage. Figure 37 shows a dynamic compaction operation in an urban area. At this site, a 6 ton (5.4 t) weight was dropped 35 ft (10.7 m) to densify building rubble. No damage occurred even though densification occurred within 20 ft (6 m) of the adjacent building.

In 1980, the Bureau of Mines revised the damage criteria level for blasting[85] to lower levels of particle velocity. The allowable particle velocity was also correlated to frequency of ground vibration as shown in Figure 38. Most of the dynamic compaction vibrations produce frequencies in the range of 3 to 10 Hertz, which would imply that the new criteria for off site ground vibrations would range from 0.5 in/sec (12.7 mm/sec) for old buildings constructed of plaster to 0.75 in/sec (19.1 mm/sec) for



·Particle velocity versus frequency with recommended safe blasting criterion.

(1in/sec = 2.54cm/sec)

Figure 36 : Partical velocity versus frequency with recommended safe blasting criterion(72)

new buildings constructed of drywall. However, the duration of the vibration following impact during dynamic compaction is only for 1 to 3 cycles with decaying intensity. Since the time duration of ground vibration from blasting is usually much longer, than the time duration of dynamic compaction generated waves, the 1980 Bureau of Mines Criteria may be too restrictive for dynamic compaction.

The Bureau of Mines constructed a wood frame test house built in the path of an advancing surface coal mine so that it could investigate the effects of repeated blasting on a residential home[86]. The house was subjected to repeated vibrations with particle velocities ranging from 0.1 to 6.94 inches/sec (2.54 to 176.3 mm/s). Their findings indicated that cosmetic or hairline cracks .0004 to .004 in. (0.01 to 0.10 mm) in width occurred during construction of the home and during periods when no blasts were detonated. The formation of cosmetic cracks increased from 0.3 to 1.0 cracks per week when ground motions exceeded 1.0 in/sec (25.4 mm/sec). Human activity and changes in temperature and humidity caused strains in walls that were equivalent to those produced by ground motions up to 1.2 in/sec (30 mm/sec). When the entire structure was mechanically shaken, the first crack appeared after 56,000 cycles, the equivalent of 28 years of shaking by blast generated ground motions of 0.5 in./sec (12.5 mm/sec) twice a day. During the shaking, the operating frequency ranged from 1.5 to 15.0 hz. Their data also indicates that when ground motions exceeded 1.0 in/sec (25.4 mm/sec), the rate of crack formation was more than 3 times the rate observed when the motions were less than 1.0 in/sec (25.4 mm/sec). On this basis, it appears that the new criteria shown in Figure 38 would be slightly conservative for newer homes.

Konon and Schuring[46] suggest that a lower particle velocity be used as the acceptable criterion when vibrations are induced

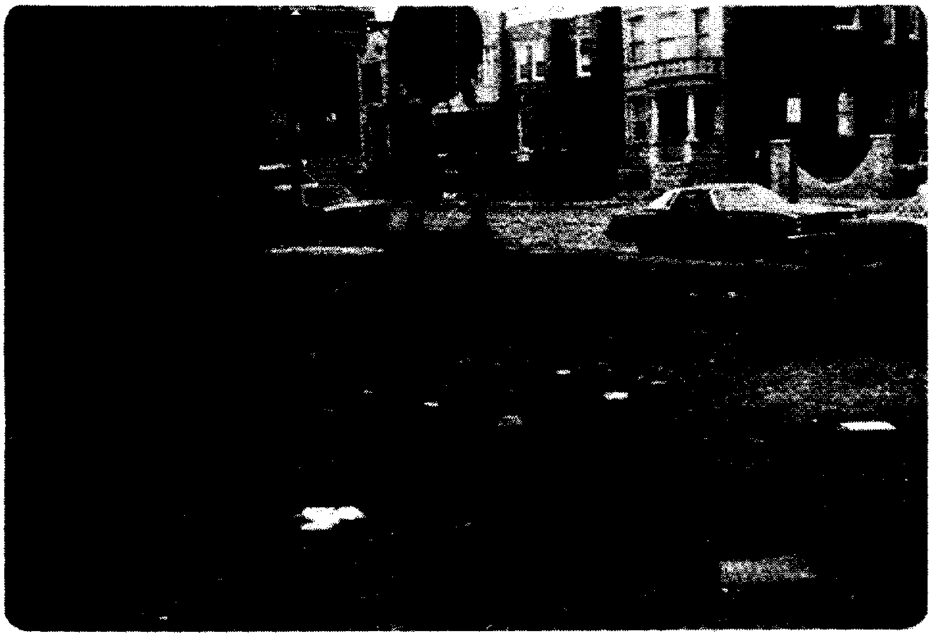


Figure 37 : Dynamic compaction in an urban area in close proximity of existing building

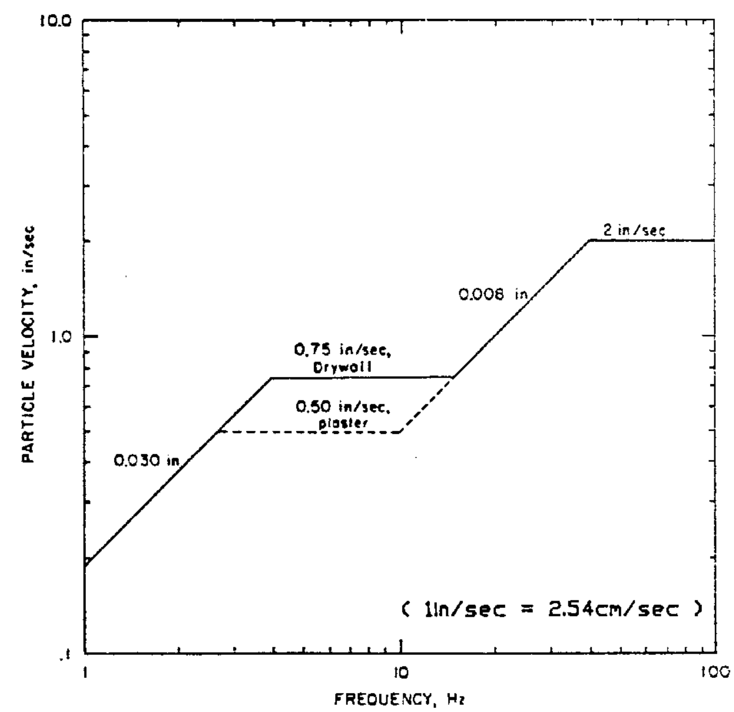


Figure 38 : Safe levels of blasting vibration for houses using a combination of velocity and displacement (85)

adjacent to historic structures. They suggest a maximum particle velocity of 0.25 in/sec (6.35 mm/sec) within the range of 1 to 10 hz and 0.5 in/sec (12.7 mm/sec) above 40 hz and with a linear variation in peak particle velocity between 10 to 40 hz frequency. Konon and Schuring point out that ground vibrations with frequencies close to the natural frequency of the structure are the most damaging. Most residential structures and their components have natural frequencies between 4 and 24 hz and the natural frequencies of historic structures, with their typically more massive construction, tend towards the lower end of this range. Dynamic compaction induces frequencies generally in the range of 3 to 10 hz, which would be close to the natural frequencies of historic structures. Fortunately, the duration of the vibrations induced by dynamic compaction are very short, and Konon and Schuring point out that more damage occurs when steady state vibrations are produced. Steady state vibrations are those which are typically generated by vibratory pile drivers or vibration compaction.

Based upon the current state of knowledge, it appears that no structural damage will occur if the peak particle velocities do not exceed the values shown in Figure 38 for the more modern structures and less than about 0.25 in/sec (6.4 mm/sec) for historic buildings. Higher levels of peak particle velocity have been experienced by numerous structures in the past, and therefore it is possible that these values could be exceeded, at least intermittently, without causing significant damage.

While structural damage can be prevented in adjacent buildings or utilities, it is not always as easy to eliminate human response to vibrations. Many complaints or claims of damage can occur when vibration levels are well below those required to induce damage. Complaints generally start in the distinctly perceptible to perceptible range of general vibrations which ranges from 0.05 to 0.3 in/sec (1.3 to 7.6 mm/sec). Siskind, et.al.[85] report that even though the 2 in/sec (50.8 mm/sec) safe level criterion was used by mine operators while blasting, many mining operators had to design their blasts to keep the velocities as low as 0.25 in/sec (6.4 mm/sec) to minimize complaints. Severe house rattling caused fear of property damage and home owners were attributing all cracks to the blast vibration. Siskind, et.al. also report that Pennsylvania was the first state to adopt the 2 in/sec (50.8 mm/sec) peak particle velocity criterion as a safe standard in 1957. However, in 1974, it was forced to adopt stricter control because of citizen pressure and lawsuits involving both annoyance and alleged damage to residences. Nicholls, et.al.[72] documented the number of complaints as a percentage of the number of families affected by one particular project. This data is shown in Figure 39. The conclusion of the study was that if the number of complaints and claims are to be kept below 8% of the potential number of complainants, a peak particle velocity of 0.4 in/sec (10.2 mm/sec) should be imposed.

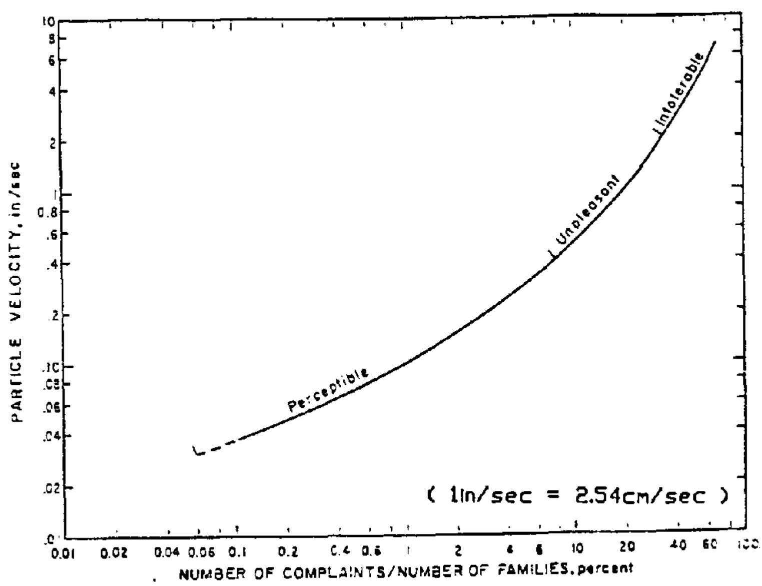


Figure 39 ; Complaint history , Salmon nuclear event with superposed subjective response (72)

If dynamic compaction operations are to be undertaken in or near to a residential area, it would be helpful if the neighbors were alerted as to the ground vibrations that will occur and to indicate that vibration units are being installed to monitor for damages. In this way, vibration occurrence should be a less alarming experience to the residents and it may therefore be possible to allow particle velocities higher than would otherwise have been tolerated.

In some instances, it may be desirable to undertake a preconstruction damage survey of nearby structures. This would include a thorough visual inspection of nearby buildings to log the locations, lengths and widths of cracks, binding of doors, peeling of wall coverings, floor sags, and tilt or lean of vertical members. Distressed areas should be photographed and pointed out to the owners prior to construction. The width of cracks could be monitored at selected locations by setting reference points on each side of the crack. The cracks may open or close due to temperature changes or wind, so readings should be taken at various times prior to construction to record these fluctuations. A seismograph could also be set up inside structures to measure vibrations as people walk the floors, slam doors, as well as from nearby traffic. Frequently, the vibrations from these sources will be greater than from the dynamic compaction operations.

For buried utilities, Wiss[100] indicates that particle velocities of 3 in/sec (76.2 mm/sec) have not damaged pipes and mains. Wiss also indicates that high pressure pipelines have withstood 10 to 20 in/sec (254 to 508 mm/sec) without experiencing any distress as apparent from dynamic strain gage measurements.

3. Permanent Lateral Displacements

Permanent lateral displacements occur in the soil mass within a certain zone of influence from the point of impact. As the tamper strikes the ground, a significant portion of the movement is downward, but lateral displacements occur and have been reported in the literature.

Pearce[75] reports on a project where 16.5 ton (15 t) weights dropping 65.6 ft (20 m) onto a cohesive fill produced lateral movements up to 1.6 in. (40 mm) at distances of 9.8 ft (3 m) from the point of impact. The measurements were obtained with an inclinometer, and the maximum movements occurred at a depth of about 26 to 39 ft (8 to 12 m). Dumas and Beaton[25] report on a project where a 16.5 ton (15 t) weight was dropped through a height of 66 ft (20 m) onto a miscellaneous fill deposit. A permanent deflection of approximately 0.75 in (1.9 cm) was observed in an inclinometer when tamping was undertaken at a distance of approximately 13 ft (4 m). A displacement of only 1/4 in (6.4 mm) was measured in the inclinometer when tamping at a distance of 19.7 ft (6 m). Lukas[53] reports on inclinometer measurements obtained in building rubble where a 6 ton (5.4 t) weight was dropped 35 ft (10.7 m). No permanent deflections were observed in the inclinometer until the centerline of the print was situated 23 ft (7.0 m) from the inclinometer. At this distance, a permanent deflection of 0.1 inch (.3 cm) was measured within a depth range of 2 to 6 ft (.61 to 1.8 m).

The results of inclinometer measurements that were obtained at distances of 10 ft (3 m) and 20 ft (6.1 m) from the print[54, 55, 56] are summarized in Figures 40 and 41. Figure 40 summarizes the results of deflections at an inclinometer located 10 ft (3.0 m) from the center line of the drop point for three sites. Figure 41 summarizes the data from the same three sites but for the inclinometer that was located 20 ft (6.1 m) from the centerline of the drop point.

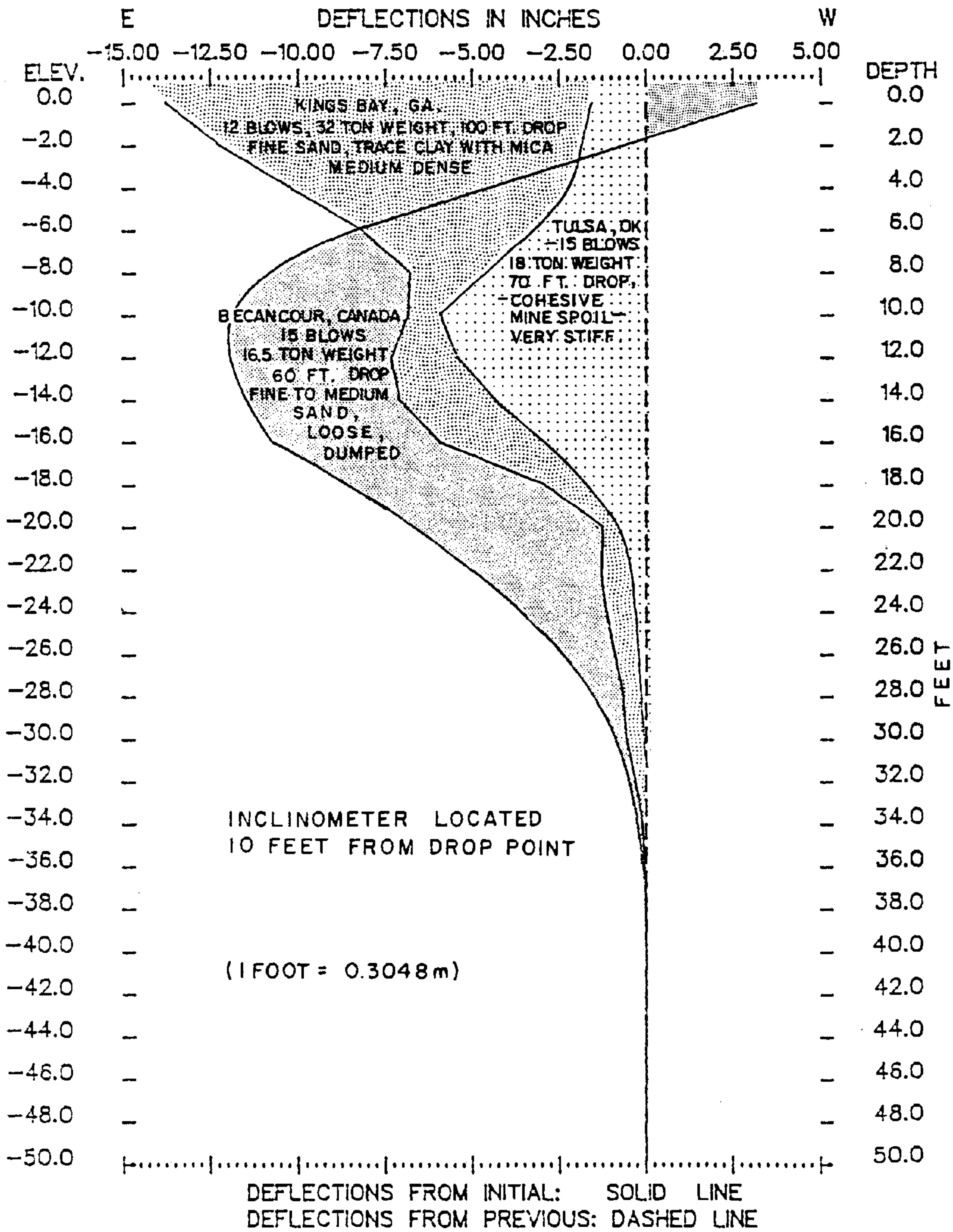


Figure 40 : Inclinometer observations

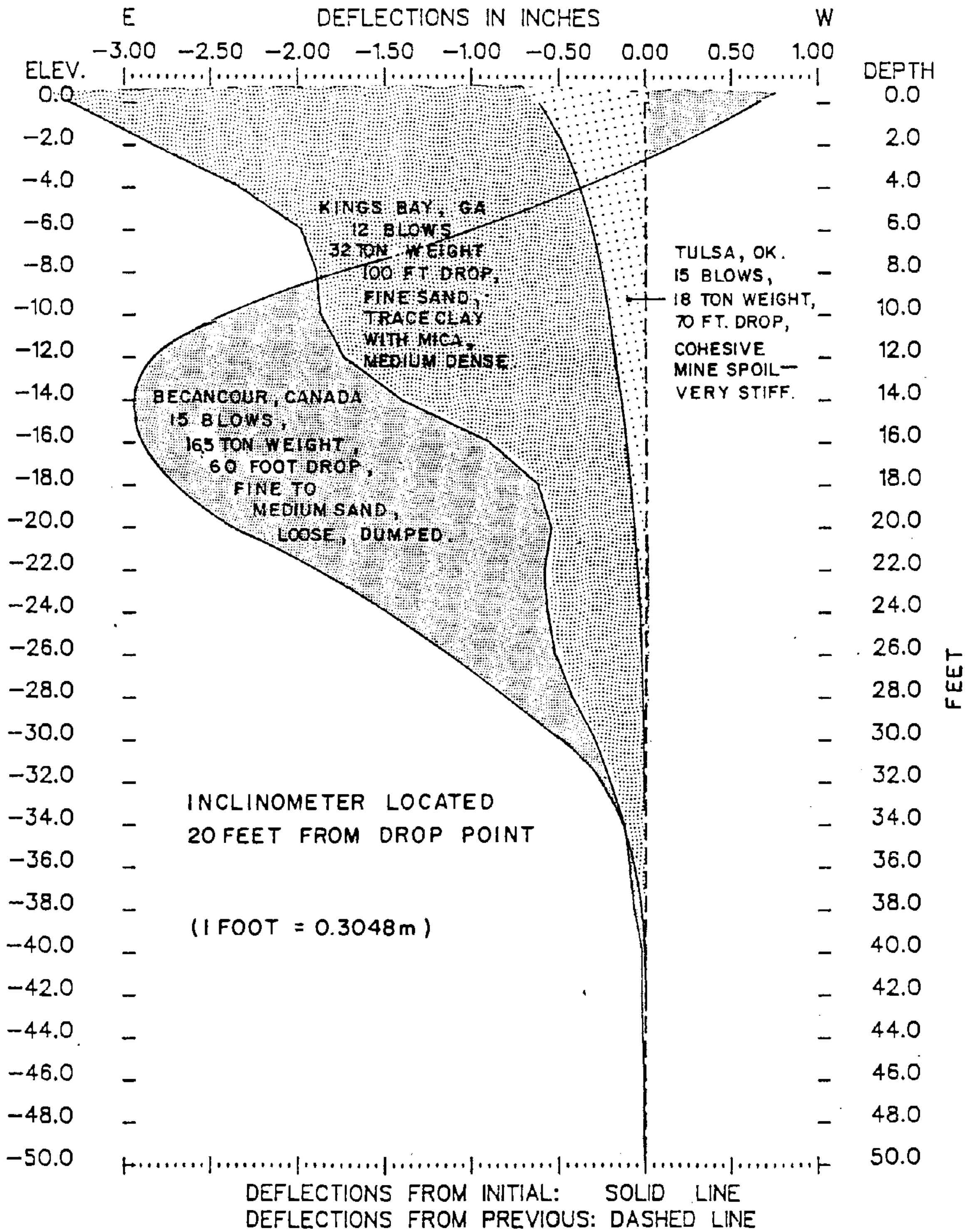


Figure 41 : Inclinometer observations

At 10 ft (3.0 m) distance, the maximum lateral deflection occurred at a depth of about 6 to 16 ft (1.8 to 4.9 m) below ground surface and the magnitude of the deflection ranged from 5 in. (12.7 cm) for a cohesive mine spoil to 12.5 in. (31.8 cm) for sandy soils. In the case of the Georgia site, larger deflections were observed within the upper few feet below ground surface, but these deflections are due to the influence of a widening crater. At the 20 ft (6.1 m) inclinometer, the lateral deflections range from approximately 1/2 in (1.3 cm) for the cohesive mine spoil to 3 in. (7.6 cm) for the sandy deposits.

Because of the lack of available data on lateral ground displacements, it is possible only to draw a few general conclusions. These are:

- o Permanent lateral ground displacements are greatest close to the location of the print and diminish as the distance increases.

- o For tampers in the range of 16.5 to 33 tons (15 to 30 t) and drop heights of 66 to 98 ft (20 to 30 m), maximum permanent displacements at a point situated 10 ft (3.0 m) from the center of the prince can range from approximately 1 to 5 in. (2.5 to 12.7 cm) in cohesive fill deposits and 10 to 14 in. (25.4 to 35.6 cm) in sandy soil deposits. At 20 ft (6.1 m) from the center of the print, maximum deflections will probably be on the order of 1/2 to 3 in. (1.3 to 5.1 cm).

- o For 6 ton (5.4 t) tampers with drop heights of 30 to 40 ft (9.1 to 12.2 m), maximum permanent displacements on the order of 0.1 inch (2.5 mm) are anticipated at a distance of 23 ft (7.0 m) from the centerline of the imprint.

- Close to the print the maximum permanent displacement generally occurs at a distance of around 6 to 16 ft (1.8 to 4.9 m) below ground surface, with lesser amounts of displacement above and below this level. However, at distances of 20 or more feet (6.1 m) from the centerline of the imprint, the maximum displacement can be located close to the ground surface.

Based upon these conclusions, it appears that dynamic compaction operations where the tamper is in the range of 16.5 to 33 tons (15 to 30 t), should not be undertaken within 25 ft (7.6 m) of any buried structure situated within the upper 30 ft (9 m) of the ground mass if movement could cause damage. This would include a utility or perhaps shallow foundation for a structure. There are also some associated vertical movements that take place within the ground mass which could also effect the functioning of these structures.

4. Construction Debris

Occasionally when the weight strikes the ground, debris may become airborne from the impact. Rubble fill and landfill sites can produce significant amounts of flying debris due to the irregular shaped and often times large particles being impacted. The fine grained deposits can result in airborne dust when dry or mud when wet (Figure 42). Granular deposits generally have less of a problem with flying debris. Although debris transmitted off site may present a greater hazard, certain protective safety measures can be instituted for on-site personnel. This would include the wearing of hard hats, maintaining a safe distance from the point of impact and, in certain cases, erecting a protective shield or fencing adjacent to the point of impact. Along the exterior, it is possible to minimize off site effects by placement of a traveling plywood board or boxes adjacent to the point of impact to deflect the debris, see Figure 43.

Figure 42 : Airborne dust and small particles following impact in clayey fill deposit

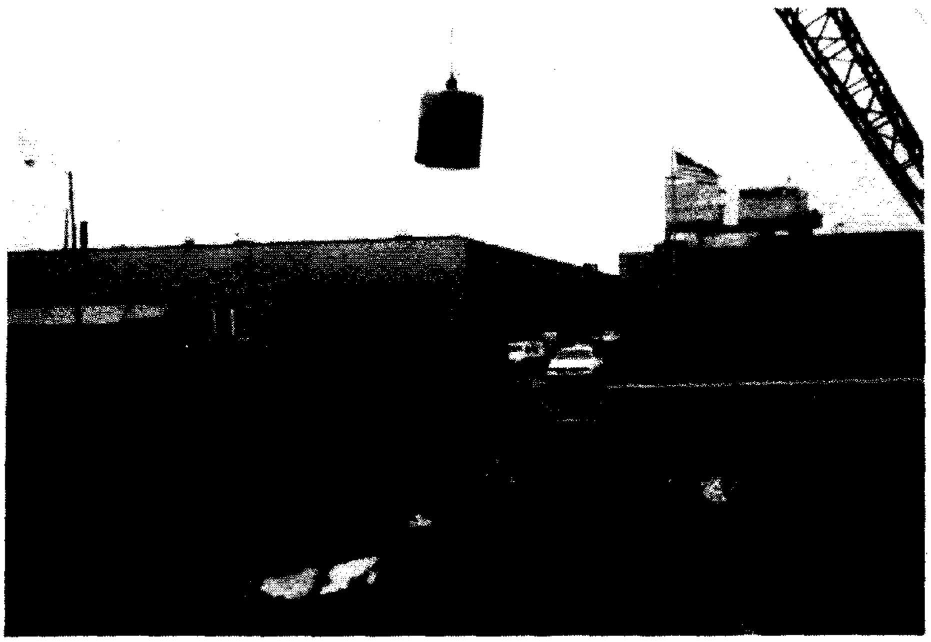


Figure 43 : Cardboard boxes placed between crater location and sidewalk to minimize problem of flying debris

1. Conventional Lifting Equipment

For tampers up to 25 tons (22.5 t), conventional crawler cranes can be used to perform the dynamic compaction operations. The tamping weight should be lifted with a single cable by a drum that is classified as a free spool or mechanical drum. This type of equipment is standard on most large cranes but a free spool is not available on hydraulic cranes. The free spool is necessary to allow the drum to unwind with a minimum of friction.

The size of equipment normally required for different weights is listed in Table 10.

TABLE 10

Equipment Requirements for Different Size Tampers

Tamper Weight	Crawler Crane Size	Cable Size
6 to 8 tons	40 to 50 tons	3/4" to 7/8"
8 to 14 tons	50 to 100 tons	7/8" to 1"
15 to 18 tons	100 to 125 tons	1" to 1-1/8"
18 to 25 tons	150 to 175 tons	1-1/4" to 1-1/2"

NOTE: 1 ton = 0.91 metric tonnes (t)
1 in = 2.54 cm

The reason the rated capacity of the crane is significantly higher than the weight of the tamper is due to a number of factors:

- o Crawler cranes are rated for maximum hoisting using multiple lines plus pulleys. Only a single line is used in dynamic compaction, unless the equipment is rigged to allow for a free fall of the tamper.

- The capacity rating for a crane is made for loading of the boom in a near vertical position, whereas, in dynamic compaction operations, the boom usually is held at an angle of about 55 degrees from the horizontal. This angle has been found to be suitable to: 1) keep the rocking of the crane within acceptable values as the weight is released from its highest position, 2) allow for an impact at some distance from the crane because of flying debris as the weight hits the ground and, 3) allow some horizontal projection of the boom which is necessary to reach the impact points. In general, the more horizontal extension there is to the boom, the smaller a weight the crane can lift.

- The hoisting capacity of the drum as well as the capability of the drum to handle the large cable sizes is an important factor. The cables have been found to cause more problems on dynamic compaction jobs than any other piece of equipment. The 100 to 125 ton (91 to 113 t) cranes generally only handle up to 1-1/8 inch (2.9 cm) cables, and cable wear is significant. A cable life of approximately one week for weights on the order of 15 to 18 tons (14 to 16 t) with a 1-1/8 inch (2.9 cm) cable is not uncommon. Cable wear occurs when the cable strands unwind as the weight is lifted and then rewind when the weight strikes the ground. Swivels are usually provided between the cable and the weight so as to allow this twisting to occur and minimize the weight from twisting. Wherever the cable passes over the top sheave repeatedly or winds on the drum repeatedly, accelerated wear occurs at these locations. To minimize cable drum winding problems due to cable overlap, the cable is generally made just long enough to wrap a few times around the drum when the weight is at the ground surface. Therefore, when a cable breaks, the entire cable is generally wasted and another cable installed.

Some contractors have prolonged the cable life by not using a swivel thereby preventing the cable from repeatedly unwinding and winding. The rotation of the weight is controlled by a tag line, one end of which is attached to the weight and the other end to a spool which is fixed to the crane. The spool allows the tag line to pay out or retrieve as the weight is raised and dropped.

The cable life is sometimes extended by placing rubber tires on top of the tamper, thereby eliminating the slap of the cable on the tamper when the tamper impacts into the ground (Figure 44). Special cables such as non-rotating surface cables, are sometimes used to prolong the life. However,

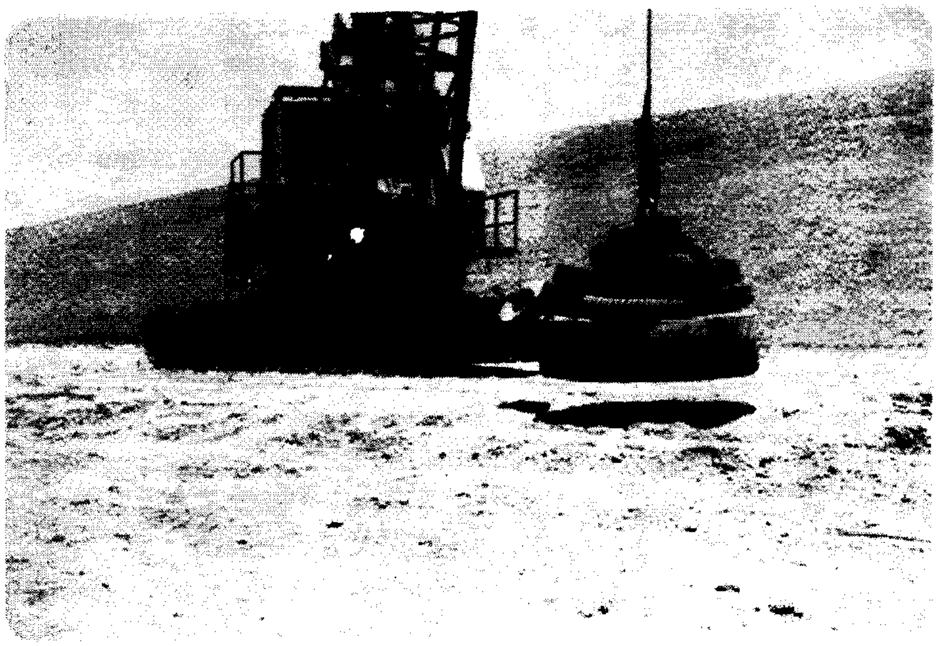


Figure 44 : Rubber tires mounted on top of steel tamper to protect cable and clamping mechanism

the cost of these cables is as much as four times higher than for normal cables.

Occasionally, specialized features are added to the equipment such as sleeves on the drum to extend its life, reinforcement of the rollers, and modification of the sheave, the hoist shaft bearing and the clutches.

The maximum size crane listed in Table 10 is 175 tons (159 t). Actually, crawler cranes are made in capacities up to 500 tons (454 t) but cranes larger in capacity than 175 tons (159 t) require rail shipping because the individual pieces of the crane are so heavy that they could not be transported on trucks. On large projects where a crane would be used for an extended period of time it may be economically practical to bring in a larger crane by rail to lift weights heavier than 25 tons (22.7 t). The cable cost for such rigs could also be quite large.

2. Specialized Lifting Equipment

For weights greater than 25 tons (23 t), specialized lifting equipment is available from specialty contractors. One contractor uses a two part line to raise a 32 ton (29 t) tamper and then allows the tamper to free fall. During lifting, a hydraulically operated clamp grips the tamper. The time required to raise and drop the weight is on the order of 5 minutes. Tower cranes exist which are capable of lifting weights of 44 tons (40 t) to a height of 131 ft (40 m). This equipment has been used on approximately seven jobs throughout the world. There is also a machine called the Giga machine which rolls on 132 tires and can lift a 220 ton (200 t) weight to a height of 82 ft (25 m). This equipment has only been used on one project[32]. The tower crane and the Giga machine are shown in Figures 45 and 46, respectively.

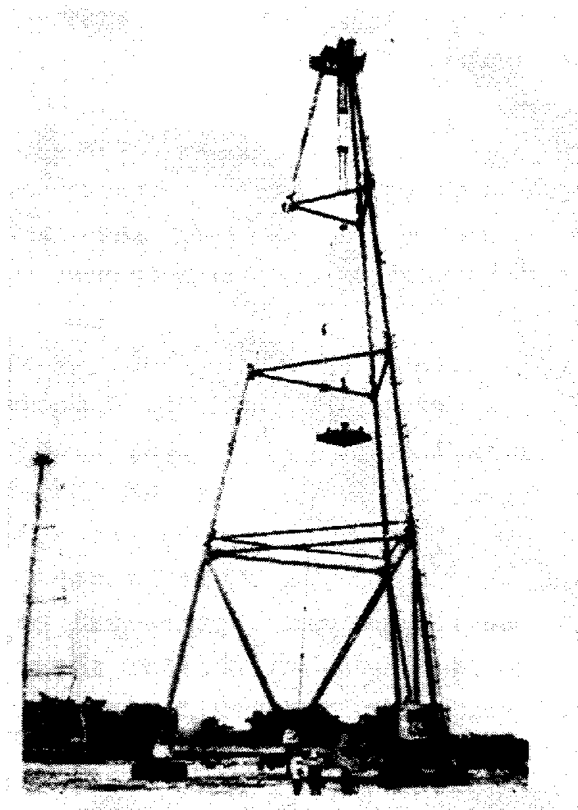


Figure 45 : Tower crane capable of lifting 40 - tonnes with a 40 meter drop

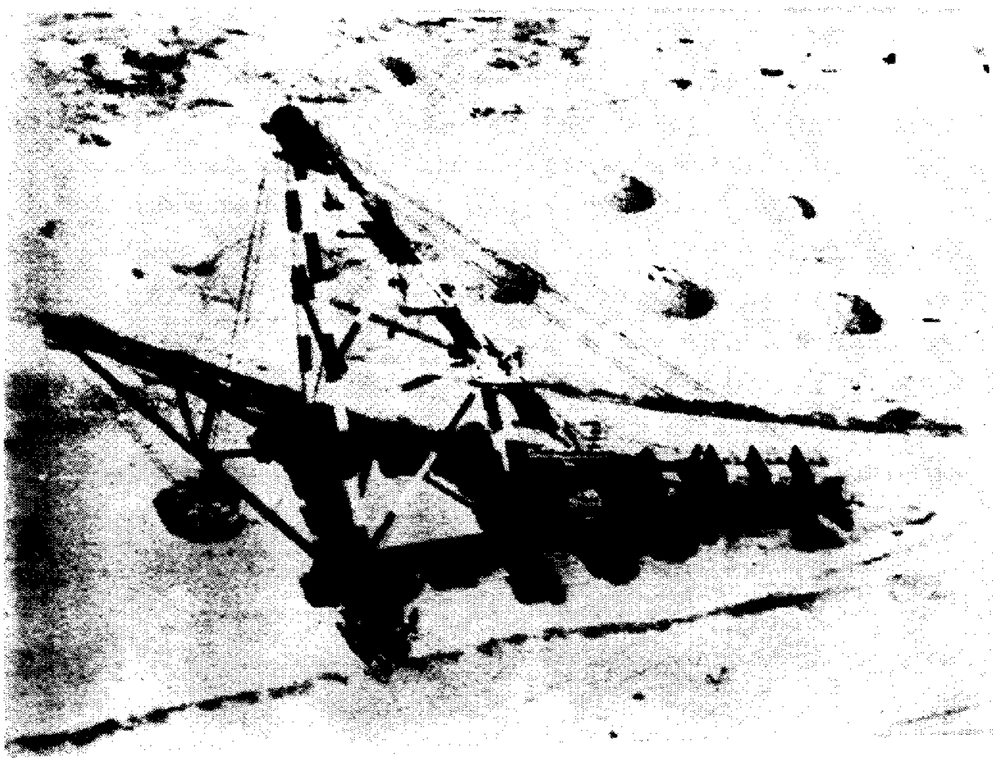


Figure 46 : Giga machine capable of lifting 170 - tonnes with a free fall of 25 meters

One special feature of these cranes is that the weight is allowed to free fall thereby eliminating the cable problem and the drag on the weight which could reduce the effectiveness of the depth of densification.

3. Tamper

The weights that are used for tamping are generally constructed of steel or have an outer steel shell and base of steel which are filled with concrete. A solid steel circular weight of 15 tons (13.6 t) with a 6 inch (15.2 cm) thick steel base plate is shown in Figure 47. Weights have been constructed entirely of concrete with some reinforcing but tend to fall apart after repeated impacts. Square shaped steel weights are also used as shown in Figure 48.

Figure 47 : A circular 15 - ton steel weight

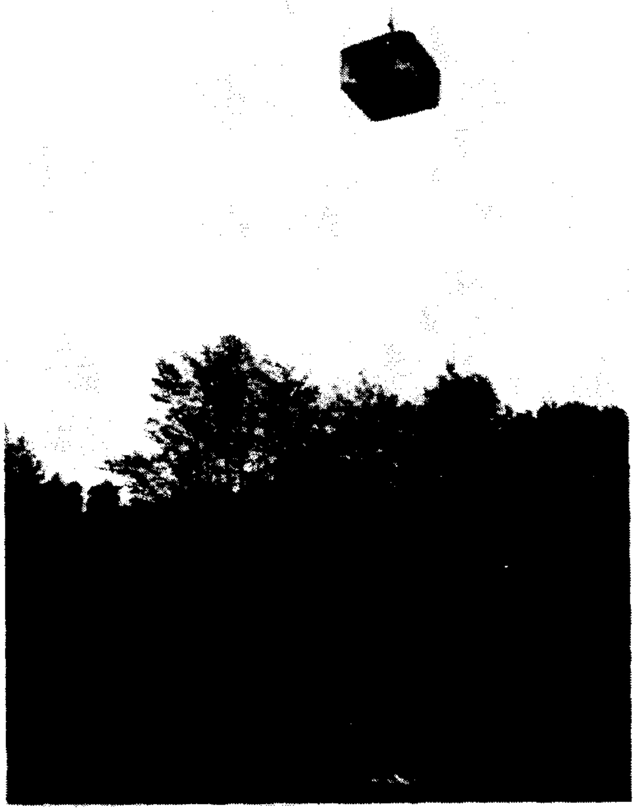


Figure 48 : A square 15 - tonne weight

The preferred shape of the base plate for the weight would be circular or close to circular such as octagonal. These types of weights produce a solid hit at the base of the crater without scraping the sides in the event the weight twists slightly during the lifting or dropping. The operator can control the location in which the weight is dropped by keeping the boom at the same position but there is little control over the rotation of the weight other than by providing a swivel or a tag line.

Square weights are frequently used for the final ironing pass so as to produce a continuous tamped surface. In this case, the weights are usually dropped from a low height, and the weight is usually lighter than used for the deeper densification so the twisting is almost negligible.

The contact pressure at the base of the weight is important, since it is a factor in the depth of improvement that is achieved. The static contact pressure for the most commonly used weights is on the order of 750 to 1500 psf (36 to 72 kN/m^2). Weights having much lower contact pressures have been found to produce a limited depth of improvement. There is no experience with weights of a significantly higher contact pressure but it is anticipated that such weights could penetrate the ground during impact without causing adequate densification.

Typically the weights used have a flat bottom so as to create a high dynamic stress upon impact. Frequently, the upper portion of the weight is smaller in size than the base plate, see Figure 47, so as to reduce the suction forces that could develop in the event there is a slight cave-in of the sides of the crater following impact.

CHAPTER 8 - CONTRACTS

1. General Information

There are two basic ways of contracting for dynamic compaction; i.e., Performance and Method Specifications. Satisfactory jobs have been accomplished with either method so the choice depends on the preference of the agency handling the work. Factors influencing the decision to use either could include the in-house expertise or lack of expertise with dynamic compaction, the complexity of the job, proximity of specialty contractors to the site, time available for test sections and experimentation plus department or agency philosophy. Whatever method of contracting for services is used, there must be independent verification of the work to maintain the proper system of checks and balances.

Guidelines for preparing specifications and sample specifications for each type of contract plus commentaries are included in Appendix A.

2. Performance Specification

One method for contracting of services is to prepare a performance specification leaving the procedures and equipment to be used up to the contractor to achieve the desired goal. The end product must be clearly defined and this requires adequate engineering by the owner or his consultant before preparing the specifications. If the goal is to reduce settlement, the properties of the existing subsoils must be defined, a settlement analysis must be undertaken for existing and anticipated improved conditions following dynamic compaction, and a judgment rendered as to whether the embankment or facility will perform satisfactorily while experiencing settlement. Alternate ground stabilization techniques and alternate methods of construction should be considered. The specifications should then be prepared in such a way to outline the minimum property improvements that are acceptable to achieve the desired goal. Contractors experienced with dynamic compaction would then be requested to bid this work. The work is usually undertaken on a lump sum arrangement. The speciality contractors would have to do a certain amount of pre-engineering work to figure the weights and drops to be used on the project, the total amount of energy to apply, the need for importing fill to form a working mat or dewatering in the event of a high water table, and other factors that could affect the final outcome of the dynamic compaction operations. Contractors who are not familiar with dynamic compaction projects should not be allowed to bid on this type of work because of the expertise necessary to plan and undertake the project. This can be handled by prequalifying contractors such as having the contractor exhibit proof of successful completion of similar projects.

The risk for obtaining the specified improvement within the budget rests entirely with the specialty contractor. The agency awarding the work should provide field monitoring as a check on the contractor's operation as well as testing after the work is underway and completed to be sure that the minimum specified improvements are achieved. The contractor will also perform his own tests to guide him in his work but this should be separate from the testing and monitoring undertaken by the agency for verification.

At the present time, there are about four or five specialty contractors capable of undertaking dynamic compaction in the United States. Distant or remote jobs could be handled by these specialty contractors without too much difficulty since the crane and labor forces that are used on the project sites are generally engaged locally. The specialty contractors will send an experienced superintendent plus the tamper(s) and some equipment to the project site. However, the specialty contractor has to cover the overhead of the office staff for the engineering, planning and implementation of the project plus has to include a contingency factor for assuming the risks associated with applying more energy than originally contemplated if the improvements are not realized within the original scope of the field operations.

For determining if the required degree of improvement has been attained, only one testing method should be specified in the contract for payment purposes. Other testing would be helpful but under certain circumstances, the interpretation of the different test results could result in varying opinions on the degree and depth of improvement.

3. Method Specification

The second method for undertaking dynamic compaction is for the agency or department to do the engineering and planning for the dynamic compaction operations, prepare plans and specifications, and then open the bidding to all contractors who can provide the proper equipment to undertake the work. Most foundation and excavating contractors have various size cranes, and weights could be constructed to match the requirements of the project.

For this situation, the agency or department must have someone knowledgeable with dynamic compaction operations on the staff or engage a consultant knowledgeable in dynamic compaction. The agency would be required to plan the entire operation for the contractor including the size of weight and the drop height, the minimum size crane to use, the crater spacing and number of passes over the area and the delay time, if any, between passes. In addition, the agency would provide the field monitoring forces to observe and adjust the tamping operations. The agency would also determine the need for piezometers, seismograph readings in nearby areas and settlement observation points. Other items should also be considered such as the need for granular fill if the surface ground is poor quality, the need for fill following dynamic compaction as a result of the ground depressions, and the need for removal of poor ground that will not compact under dynamic compaction.

In this type of contract, the contractor assumes very little, if any, risk related to the depth or degree of improvement. Production risks still remain with the contractor, since the time for completion schedules must be maintained. The contractor will also be responsible for having the proper equipment and manpower at the job site, maintaining the rig, providing sufficient cables and swivels so as to keep a continuous operation, and proceeding in a workmanlike manner. The agency or department assumes the risk that the improvement will be achieved. If the desired improvement is not reached after the specified amount of energy has been applied, it will then be necessary to either increase the energy over and above that provided for in the contract or to accept something less than the desired improvement. The agency or department will need to have additional funds within its organization for providing this engineering and planning service for undertaking the dynamic compaction operations and for contingencies related to performance. This work is usually contracted for on a lump sum basis for the scope of work outlined on the plans and specifications, but with unit pay items for extras beyond the anticipated quantities. Such items include additional drops, excavation and replacement of unstable deposits that can't be compacted, or extra granular material placed at the surface to stabilize weak deposits such as landfills.

On relatively simple jobs, a high degree of sophistication or knowledge of dynamic compaction is not essential. This would include densification of granular deposits where the water table is relatively deep and only minor or moderate improvements are necessary to achieve the goal. On the more sophisticated projects such as improvement of water bearing silts, a higher degree of sophistication in the planning of the operations will be required. It may be possible under this type of contract to undertake a test section first, evaluate the results, and then plan for the final dynamic compaction program.

CHAPTER 9 - DYNAMIC COMPACTION COSTS

1. General Comments

Usually, the cost for dynamic compaction is stated in terms of a price per unit area such as dollars per square foot. Costs stated this way can easily be translated to total cost, because the area that needs to be improved is generally known early in the planning. Other methods of ground improvement are also stated in terms of cost per unit area. This would include normal excavation and compaction, stone columns, vibroflotation, and preloading.

2. Factors Affecting Costs

As with all forms of construction, the cost per square foot will vary with each project site depending upon the requirements. In general, the primary factors affecting the costs for dynamic compaction are:

- Size of Area to be Treated - If the size of the area to be treated is large, the initial mobilization and the engineering required to plan the construction operations will be a smaller percentage of the total cost. On the other hand, if the project area is small, the mobilization and engineering cost will be a relatively large factor of the total cost.

- Depth and Degree of Improvement - As the required depth of improvement increases, a larger weight and a greater height of drop is required. This in turn requires a larger, more expensive crane. The degree of improvement also affects the cost. As the degree of improvement increases, the amount of energy required also increases. It may be necessary to use multiple passes to apply the required energy. The time required and therefore, the cost of a project is directly proportional to required energy.

- Complexity of the Site - Ideal subsoil conditions for dynamic compaction would consist of granular highly permeable deposits with a deep water table. It is relatively easy to get improvements in these deposits with a minimum of uncertainty. On the other hand, if the subsoil conditions are more complex such as saturated silty sands or silts, there is always some uncertainty as to the proper energy per blow, the number of drops that can be applied before large excess pore water pressures develop, the time delay between passes and the total energy to apply. At complex sites, it will be necessary to plan for multiple passes with delays between passes and to install piezometers at these sites to monitor pore water pressures to make field adjustments in the timing between passes.

  If the water table needs to be lowered, this will also increase the costs.

- If the deposits are extremely loose such as landfill sites, it will probably be necessary to import granular or other acceptable materials to:

  - Provide a working mat from which the cranes will work.

  - Provide a deposit of sufficient stiffness to minimize crater depths.

  - Provide confining pressure on the underlying weak deposit.

o Compensate for ground settlements.

The cost for imported granular fill could represent a significant portion of the costs for dynamic compaction and should be taken into account in estimating the total cost of the site improvement.

3. Typical Costs

Typical costs for dynamic compaction projects are summarized in Table 11. These cost figures were obtained from discussions with contractors who have completed dynamic compaction jobs and pricing information from project files. These costs represent projects undertaken in the years of 1980 to 1985. The cost figures shown include the equipment and operator costs but do not include costs for importing any material that may be required as a result of ground lowering or for site preparation prior to dynamic compaction. For projects where a specialty contractor is doing the work, the costs include engineering time spent in planning and controlling the field operations as well as borings and field testing to confirm the improvement. For method specification projects where non-specialty contractors compete for the work, the contractors bid item is usually significantly less than shown in Table 11. However, the control agency must do the planning, monitoring, borings, field testing and data analysis and reporting, so total project costs may be comparable to the ranges shown in Table 11.

The largest part of the cost for a dynamic compaction operation is associated with the rental of the crane and the crew. If the unit can be kept productive without breakdowns or significant time lost for cable repair or moving about the site, it should be possible to deliver 400 to 500 tamps per 8 hour working day for a 15 ton (13.6 t) weight dropped 65 to 95 ft (20 to 29.0 m). For a 6 ton (5.4 t) weight dropping 30 to 40 ft (9 to 12 m), it should be possible to deliver 600 to 800 tamps per 8 hour day. A 50 ton (45 t) crane with a crew generally rents for about $1000 per day, a 100 ton (91 t) crane and crew for $1700 per day and a 150 ton (136 t) crane, for about $2100 per day. Mobilization charges also increase as the size of the equipment increases. A 40 to 50 ton (36 to 45 t) crawler crane can usually be transported to the site and erected relatively quickly. However, a 100 ton (91 t) and larger size crane requires hauling portions of the crane on separate trailers and using an additional crane at the site to assemble the sections. Two or three days may be required for erection and demobilization.

TABLE 11

Dynamic Compaction Costs

Size of Weight Required	Unit Cost Dollars/ft^2
4 to 8 ton	.50 to .75
8 to 16 ton	.75 to 1.00
16 to 25 ton	1.00 to 1.50
25 to 35 ton	1.50 to 3.00
35 to 100 ton	Negotiated for each job

NOTE: These prices are based upon projects undertaken during 1980 to 1985.

1 tonne (t) = .91 ton

1 ft^2 = 0.092 m^2

APPENDIX A - SPECIFICATIONS; GUIDELINES FOR PREPARATION AND SAMPLE SPECIFICATIONS WITH COMMENTARIES

1. General Requirements for Method Specifications

If a method specification is to be used, sufficient information should be provided within the specifications so that it is clear to the contractor exactly what must be provided. Some of the important considerations are listed below:

o. General Descriptions - A general description of the new facility to be constructed along with a general description of dynamic compaction should be in the specifications to acquaint contractors with the project. All pertinent information including topographic mapping, surveys, soil boring logs and geotechnical information should be provided.

o. Work Area - The extent of the area to be improved by dynamic compaction should be outlined on a drawing or set of plans. This would include the plan dimensions of the embankment or building plus the additional area that is to be improved beyond the limits of the embankment or the building. Any utilities or other subsurface features should be shown on these drawings since they may affect the dynamic compaction operations. If some areas are designated for one type of weight and drop height and others for different weight and drop height, these areas should be differentiated. The total square footage of area to be dynamically compacted should be shown on the drawings.

o. Equipment Required - The tamper weight and the drop height should be specified for the contractor. This should be calculated in advance to match the depth of required improvement and the soil type and not left up to the contractor. Guidelines are presented in Chapter 3. The range in contact pressure of the tamper should be specified. It should also be pointed out that the weight must be raised and dropped with a single cable with a free spool drum or by free fall methods.

In order to complete the job on a timely schedule, either the number of pieces of equipment should be specified or a starting and completion date should be given so that the contractor can plan the proper number of pieces of equipment to complete the work within the time frame.

o. Energy Application - The amount of energy to apply at ground surface should be specified. The energy is usually given in txm/m^2. If different energy levels are to be specified for different areas, these areas should be clearly delineated on the drawings.

The grid spacing, number of drops per print, and number of passes required should be specified. Guidelines for estimating energy levels, grid spacing, and number of passes, and number of drops per pass have been presented in Chapters 3 and 4 of the Dynamic Compaction Manual. Since it is not always possible to predict how the ground will behave under repeated drops of the weight, it will be necessary to specify a maximum crater depth that can be tolerated for each pass and if this depth is reached before the desired number of blows is reached, it will be necessary to fill the crater before applying additional blows or to apply an additional pass or passes, as required, so that the specified applied energy is imparted.

After the primary energy has been applied, the amount of energy to apply during the ironing pass to compact the surface of the deposit should be specified. If the surface is to be compacted with conventional compaction equipment instead of an ironing pass, this should also be specified.

o. Backfill and Ground Leveling - If a backfill material is required to either raise the grade or to provide a working mat on weak ground, the thickness and type of backfill should be specified. If additional fill is to be brought in to fill craters, this should be stated in the specifications.

On most projects, fill is not required either in advance or during dynamic compaction, and in this case the contract should state that ground leveling should be undertaken after each pass using a dozer to blade the ground from the high areas into the craters and then track rolling the surface. This is necessary to form a smooth surface from which the equipment can work for the next pass as well as to obtain ground surface elevations.

o. Required Testing - As the work is underway, certain tests should be performed to evaluate the effectiveness of dynamic compaction. These tests could include measuring crater depths, measuring heave adjacent to certain craters, determining ground losses from settlement readings following each pass, and borings with conventional soil testing. It should be spelled out in the specifications who will do this testing and how many tests will be performed. If the contractor is made responsible for the soil borings, then the type of rig and type of samples and sampling intervals should also be specified. If the owner is to perform some specialized testing within boreholes such as pressuremeter testing, this should also be spelled out in the specifications so that the contractor can provide the proper equipment and money in the budget to compensate for time lost during this testing.

If dynamic compaction is to take place adjacent to built up areas, it will be necessary to make seismic readings to determine the magnitude of ground vibrations being transmitted off site. It should be made clear in the specifications who will be responsibile for taking the readings and how often they will be taken. It should also be pointed out who will interpret the readings.

o. Recordkeeping - Records should be kept of amounts of fill brought onto the site, the number of drops per day, the number of drops at each print, the number of passes completed to date, plus other general field records. The specification should point out who will keep these records and to whom they will be made available.

o. Payment - An equitable form of payment for a method specification contract would be to have a lump sum for undertaking the dynamic compaction for the energies and area specified and then to have unit rates for additional work. The additional work could consist of additional drops, where needed, or for undercutting and removal of soil that will not compact properly and for replacement with new fill, where required. If it is necessary to place granular fill as a working mat or to fill craters with granular fill, this should be a separate bid item. These unit price items for work incidental to dynamic compaction will take some of the risk out of the total operation and allow the contractor to figure his budgets for the dynamic compaction work in the most economical fashion.

2. General Requirements for Performance Specification

If a performance specification is to be used, the required improvement of dynamic compaction should be clearly stated so the contractors can plan the field densification program to meet this objective. Some of the important considerations to be included in the specifications are listed below:

- Project Description - A general description of the project should be provided in the specifications. Soil boring logs, the geotechnical report, property line surveys, topographic maps, and enough drawings showing the new facility should accompany the specifications to fully acquaint the contractors with the site conditions and the proposed new construction.

- Work Area - The extent of the area to be improved by dynamic compaction should be outlined on a drawing or set of plans. This would include the entire area of the embankment or building plus the extension beyond the limits of the new facility that is also to be improved. Any utilities or subsurface features should be shown on these drawings since they may affect the dynamic compaction operations. If a different amount of improvement is to be achieved in different areas, these areas should be differentiated on the drawings. The total square footage of area to be dynamically compacted should be shown on the drawings.

- Required Improvement - The amount of improvement that is to be achieved at the project site should be presented in the specifications. This means that a certain amount of engineering must be undertaken prior to writing the specifications by the owner or his consultant. First, it must be ascertained whether dynamic compaction is appropriate for the prevailing subsurface conditions. If appropriate, then it is necessary to determine the minimum improvements that are needed so that the new structure or embankment will function satisfactorily. As an example, if the dynamic compaction were undertaken to reduce the potential for liquefaction under a design earthquake, the owner or his consultant could undertake an analysis which will show the minimum required Standard Penetration test value at various depths or the minimum relative density at various depths. The specifications could then reflect the minimum SPT or relative density values that are needed at the depths of concern. The contractor is then free to select the right amount of energy to attain these minimum test values. If after application of all the energy the minimum values are not met in certain areas, additional energy must be applied to improve the soils to the minimum standards. It is important that the owner (designer) not choose a value of soil improvement which cannot be achieved.

In certain cases where the minimum improvement needed at a site is difficult to predetermine such as at a recent landfill site, the specifications could state that the amount of deflection of a test embankment after dynamic compaction shall be less than a certain predetermined value. The test embankment should be made comparable to the pressures imposed by the final embankment. The test embankment may not be a true indicator of the final performance of the embankment since some long-term decomposition of the landfill will take place and settlements will increase over a period of time. However, there is at present no better way of specifying performance of landfill sites unless it can be demonstrated that conventional testing will work.

On projects where conventional soils are being densified, such as natural or fill deposits of sand or silt or mine spoil, conventional soil sampling techniques consisting of Standard Penetration Test (SPT), Cone Penetration Test (CPT), or Pressuremeter Test (PMT) could be used to evaluate the stability and settlement of these deposits with and without dynamic compaction from which minimum values of SPT, CPT or PMT following dynamic compaction could be specified.

The important point to keep in mind is to undertake a sufficient amount of pre-engineering to identify the problem and determine the minimum parameters that are required following densification such that the structure or embankment will perform satisfactorily. This should then be made clear to the contractor in the specifications.

- Prequalification - Since the desired end product is stated in the specifications without the methodology to achieve this end product, only qualified contractors should be allowed to bid on these projects. The contractors will have to rely upon their previous experience, engineering ability, and judgment to determine the right amount of energy to apply, the grid spacing, time delays between passes, drop heights, and size of weights to achieve the final goal. One method of prequalification would be to allow only contractors who have completed some number of successful dynamic compaction projects to bid for the work. The documentation should be presented at the time of the bid. An alternative method of prequalification would be to require the contractor to submit a detailed work plan and an equipment list at the time of bidding for the owner (designer) to evaluate as a condition of accepting the bid. This presumes that the owner (designer) is sufficiently knowledgeable to screen out inexperienced contractors.

- Time Duration for Dynamic Compaction - Some means of assuring that the project will be completed on a timely basis should be discussed in the specifications. This could be accomplished most easily by providing a certain length of time for the work to be accomplished, thereby requiring the contractor to bring in the proper pieces of equipment to meet the schedule. An alternate method would be to specify a minimum number of dynamic compaction rigs, which at an average amount of production per day would also complete the work within a timely fashion. The difficulty with the latter approach is that if there are equipment breakdowns or if there is poor weather, the time to complete the work may be longer than desired. With the first approach, the contractor may choose to work longer hours or on weekends to complete the work within the time schedule.

- Site Preparation - The condition and elevation of the existing site should be discussed in the specifications. If some site preparation is required prior to dynamic compaction, it should be stated who will do this work and whether it is part of the dynamic compaction bid or some other subcontractor. Site preparation could include removal of trees or some surface debris, flattening out of a hilly terrain to a more nearly level surface, or placement of new fill to change the grade. Frequently, this work is undertaken with a different subcontractor but it should be made clear in the specifications as to what the grades and condition of the area will be at the time the dynamic compaction contractor will start his work.

- Required Testing - To confirm that the minimum value of improvement has been achieved, certain tests will need to be performed. This would include SPT, CPT, or PMT tests in boreholes or monitoring of test embankments with settlement plates to determine the amount of ground deformation under load. Whatever method is selected for evaluating the improvement should be clearly spelled out in the specifications. One test method should be selected as the acceptance criteria to avoid confusion in the event that two or three different test methods all showed different degrees of improvement. It should also be clearly stated who will perform these tests. It is suggested that the testing be done under the direction of the owner but if the testing is to be done by the contractor, then a representative of the owner should be present during the testing. The specification should also detail how many tests will be performed at what time intervals during the course of the project.

If dynamic compaction is to take place adjacent to built up areas, it will be necessary to undertake seismic readings to determine the magnitude of ground vibrations being transmitted off site. It should be made clear in the specifications who will take the readings and how often they will be taken. It should also be pointed out who will interpret the readings.

- Record Keeping - Since the contractor will be undertaking his planned field densification program, it will be necessary to specify that the contractor keep records of this operations. This would include, but not be limited to, grid patterns, drop heights, drop weights,, number of blows, depth of crater penetration at each location, number of passes over the entire area, and ground settlement. The types of records that are kept should be agreed upon in advance and should be provided to the field engineer during the duration of the project.

- Payment - Whenever work is done on a performance basis, it is usually undertaken on a lump sum basis. Separating the bid into itemized unit quantities is not possible because only the contractor knows what types of equipment he will provide at the site, how much energy will be applied and whether any special additional work such as dewatering or bringing in more fill will be required. The contractor will include in his bid all the items he feels are necessary to undertake the work plus some engineering time for planning the dynamic compaction and for borings to monitor his work. The contractor must also include in his bid item some additional funds to cover uncertainties and risks which will tend to raise the bid price. On the other hand, an experienced contractor could use ingenuity and past experience to develop an economical field program to accomplish the goal, thereby off-setting some of the costs associated with assuming more risk.

  There are exceptions to undertaking the work totally on a lump sum basis. For instance, when working on a landfill site, it is not possible to determine how much granular material may be required to stabilize the surface of the landfill. The contract should be written for a lump sum for the dynamic compaction work with a unit rate for granular material to be brought in and placed over the area as required to maintain a stable ground surface.

3. Sample Specification Plus Commentaries

a. Example 1

The following specification was prepared by a State Highway Department for densification of a landfill deposit. It is a performance specification. A specified deflection under a maximum pressure of 1 tsf imposed by a fill pile is the determining criteria for acceptance and payment.

The specialty contractor has freedom (within limits) to select tamper weight, grid spacing, drop height, number of passes, and equipment. Only a knowledgeable specialty contractor could bid this work because payment is a lump sum dependent upon the surface area that is improved regardless of the time or equipment required to the job.

The requirement of 1 inch of settlement under a load test with a pressure of 1 tsf following dynamic compaction in all likelihood will not be related to future performance of the embankment. The load test results are also influenced by the location of the test. IF the test after dynamic compaction is run at the same location as before, it could be argued that the reduced settlement is partly attributed to the preloading affect of the first test. The final embankment height probably varies so the pressure imposed on the densified landfill will probably vary significantly. Settlement of the embankment will vary with many factors such as thickness of landfill, width and height of embankment and rate of decomposition. Until more experience is gained with performance of landfills, an engineering acceptance criteria will be difficult to specify. In spite of the difficulty in obtaining good test results, SPT, PMT and CPT tests should be undertaken before and after dynamic compaction to obtain relative improvements in soil properties.

The basis for payment is clear and easy to calculate. The dynamic compaction is based upon the square yards of area densified which are spelled out in advance and by the cubic yards of fill used to fill the crater which cannot be accurately predicted in advance of the work.

Prequalification requirements were stipulated in another section of the specifications as follows: "Only firms that can demonstrate that they have successfully completed five dynamic compaction projects will be allowed to bid".

Because of the difficulties associated with measuring the improvement, determining the amount of granular fill required, and long-term settlement of landfills, it appears that a method specification would be more appropriate for this type of work.

DYNAMIC COMPACTION

119-1 Description

The work under this Section consists of compacting a landfill site utilizing a high energy compaction technique referred to herein as Dynamic Compaction. All work shall be in accordance with these specifications and in conformity with the alignment, grades and details shown in the plans.

119-2 Equipment

The Contractor may use machines or combinations of machines and equipment that are in good, safe working condition and will produce the results required by these specifications.

119-3 Preparation

The contractor shall establish, in conjunction with the Department's Foundation Engineer, a format by which appropriate record keeping data may be accumulated by the Contractor during the prosecution of the project. This format shall include, but not be limited to: impact grid pattern, free fall distance, tonnage and size of weight, number of blows and depth of penetration at each location and the number of passes over the entire area. One copy of the approved format shall be provided to the Project Engineer at least two weeks prior to beginning Dynamic Compaction operations.

During Prosecution of the project, the Contractor shall accumulate data utilizing the approved format and provide the Department with one reproducible copy of such data upon project completion.

Prior to beginning compaction operations, the Contractor shall level the terrain. Three feet of granular material from the A-1 or A-3 soil groups shall then be placed over the landfill area; this material shall be graded and substantially compacted in order to support the construction equipment during the Dynamic Compaction. This work platform shall be graded to drain and shall be suitable for movement of large crawler cranes and other equipment.

119-4 Construction Methods

The Dynamic Compaction technique involves the dropping of a heavy weight (10-20 tons) free fall from a height of 50 to 100 feet. The high energy levels which are developed produce a deep compaction of the underlying materials.

Two or more passes shall be made over the designated area according to a predetermined impact grid pattern. The impact grid pattern, free fall distance, tonnage and size of weight shall be determined by the Contractor.

During the final compaction pass, as much of the surrounding loose granular material as possible shall be utilized to level out the impact craters. After the final compaction pass is completed, the entire area shall be graded, scarified to a depth of eight inches and compacted to a density of not less than 100 percent of the maximum density obtained by AASHTO T99, with correction for varying amounts of coarse particles in accordance with AASHTO T224.

The completed, compacted landfill shall be capable of sustaining a minimum uniform pressure of one ton per square foot over an area large enough to induce an additional vertical stress at the bottom of the landfill stratum of not less than 0.2 tons per square foot. The resulting immediate settlement under this field loading shall be less than one inch. This Static Load Test operation shall be conducted upon completion of Dynamic Compaction. The Contractor shall provide and construct select or suitable embankment material for the Static Load Test at no additional cost to the Department. This material will remain in place upon project completion.

The contractor's attention is directed to the fact that the Department will be monitoring certain aspects of the construction operations which may delay the normal progression of the Contractors operations. This monitoring will include, but not be limited to, counting the number of blows and measuring the depth of craters at selected drop locations. The Contractor shall coordinate his construction operation as necessary.

119-5 Method of Measurement

The quantity to be paid for under this Section shall be the plan quantity in square yards of Dynamic Compaction regardless of the number of passes. The area shall be as shown at the top of the select fill for the Test Site subject to the provisions of 9-3.2.

119-6 Basis of Payment

The quantity of Dynamic Compaction, determined as provided above, shall be full compensation for all work and materials specified under this Section including the materials and equipment for the Static Load Test, but shall not include those items which are to be paid for separately under other items of the Contract.

Payment shall be made under:

Item No. 119-70 - Dynamic Compaction - per square yard
Item No. 120-2 - Borrow Material - per cubic yard

15. <u>Borrow Excavation - Value Engineering Incentive</u> (8-83)

SUBARTICLE 120-2.2.2 (Page 124) is expanded as follows:

A value engineering cost proposal submittal based on the use of borrow material from within the project limits will not be considered.

16. <u>Borrow Areas</u> (8-83)

ARTICLE 120-6.2 (Pages 127 and 128) is modified as follows:

No borrow pits will be required by the Contractor from private sources that will fall within the corridor of any interstate highway without written permission of the Department.

b. Example 2

Method Specification for dynamic compaction at an industrial building site to be constructed at a site where the subsoils consist of a silty fine sand.

The geotechnical consultant determined the tamper weight, contact pressure, drop height, grid spacing, and number of tamps at each crater prior to preparing the specification. This allowed specialty contractors and non-specialty contractors to price the job.

The specifications clearly delineate the equipment the contractor shall supply and the submittals required. Some flexibility is allowed in the contract by the insertion of Section 3.6. However, this also introduces some uncertainty in the contractor's bid if the scope of work were to be greatly altered. It may be better to state that additional drops may be required above the number specified and a pay item provided for this.

It would be better to specify the required average applied energy per unit area instead of the number of drops and distances between prints. In this way, there is some flexibility in adjusting the field work while maintaining the proper average energy. The number of drops can be altered or the drop height adjusted to obtain the required energy level.

Section 1.6 is a poor attempt at a disclaimer. First, the contractor is warned that the information on the reports may not be accurate and must familiarize himself with the site conditions (but not specified how). This is followed by a warning that the contractor shall be thoroughly familiar with the recommendations in the report.

Section 1.5.4.1 refers to "good" drops but this is not defined.

DYNAMIC SOIL COMPACTION

1.0 GENERAL

1.1 This specification applies to dynamic soil compaction and appurtenant work. Contractor shall furnish adequate equipment, materials, and labor as necessary to comply with the requirements as specified herein.

1.2 The Contractor shall be responsible for compliance with all federal, state, county, and local laws, regulations and ordinances during the progress of his operations.

1.3 Related Work Specified Elsewhere

Site Preparation and Earthwork, Specification 2.100.

1.4 Definition

Dynamic compaction, also called "Pounding", is a process wherein a large weight is raised above the ground and allowed to free fall, impacting with high compactive energy. The depth of compaction influence depends upon the size of the weight and the height of the free fall. The weight is to be raised and dropped a specified height and shall hereinafter be called the "Pounder".

1.5 Submittals:

The Contractor shall submit the following to the Owner's Representative for review:

1.5.1 Drawing of pounder showing size, connections, and material.

1.5.2 Proposed grid pattern of area to be pounded showing location and sequence of drops. Pattern shall be such that clear distance between craters shall not exceed 18 inches.

1.5.3 The methods of operation and equipment to be used.

1.5.4 A daily log shall be submitted each day showing the following for that day:

1.5.4.1 The number of impacts or good drops made.

1.5.4.2 The equipment used.

1.5.4.3 A statement of any unusual happening that occurred.

1.5.4.4 The number of men that were employed.

The Soils Engineer shall submit the following:

1.5.5 Date, grid point designation, and number of pounder impacts at each grid point.

1.5.6 Location of soil borings and test results.

1.5.7 Record of monitoring vibration during pounding including a statement as to whether or not vibrations at the property line or nearby structures is less or more than acceptable limits for causing structural damage.

1.6 The following attachments pertaining to soil conditions and site investigation are included for information only. The accuracy of this data is not guaranteed, as conditions at the present time and at other locations may vary from those disclosed in these reports. Contractor shall familiarize himself with the soils condition on the site, whether or not covered in these reports, and shall thoroughly understand all recommendations made by the Soils Consultant.

1.6.1 Subsurface Investigation

1.6.2 Supplementary Exploration and Analysis

1.6.3 Letter, addendum to supplementary Exploration and Analysis

2.0 <u>PRODUCTS</u>

Contractor shall furnish the following equipment subject to the approval of the Owner's Representative.

2.1 Pounder shall have a minimum weight of 15 tons with a contact surface having a minimum contact pressure of 1,900 pounds per square foot under its own weight. Pounder shall be sturdily constructed so that it will last throughout the job and leave a rounded footprint.

2.2 Grading equipment as required for compliance with this Specification

2.3 Crane capable of lifting a 22.5-ton weight to a height of at least 60 ft. Crane shall have a recommended capacity of 100 tons and shall be capable of dropping a weight in a free fall without restraint from the crane mechanism, except for natural friction, such as between the line and drum.

2.4 Self-propelled vibratory roller having a minimum dry weight of 27,000 pounds. Roller shall be Raygo or equal as approved by the Owner's Representative.

3.0 <u>EXECUTION</u>

3.1 Prior to commencing dynamic soil compaction, Contractor shall visit site and examine area delineated on the drawings to be compacted by dynamic soil compaction and report to the Owner's Representative in writing of any unacceptable conditions. Such defects shall be corrected in compliance with the appropriate specifications. The compaction shall not begin until such corrections have been made. Commencement of compaction shall be construed as the site being acceptable for satisfying the requirements of this specification.

3.2 After clearing, grubbing, and stripping of topsoil by others, Contractor shall grade and proofroll compaction area. Soft spots and existing in-place material within the compaction area which cannot be densified to 95% of Modified Proctor at optimum dry density shall be removed. The extent and depth of removal shall be as determined by the Soils Engineer. Following this, the area shall again be graded and proofrolled.

3.3 This method of soil compaction shall consist of the lifting and dropping of a 15-ton pounder from a distance of 40 ft from the bottom face of the pounder to the ground.

3.4 Nine drops per location shall be made on the approved grid spacing. Spacing shall be such that clear distance between craters shall not exceed 18 inches.

3.5 When a crater of more than 4 ft in depth is reached before nine good drops have been made, the crater shall be releveled and pounded again until a total of nine good drops have been made. A good drop is one in which the energy of the pounder is not dissipated by hitting the side of the crater or by being otherwise dampened and the allowable distance from the geometric center of the impacted pounder to the grid point does not exceed 12 inches.

3.6 The height and number of drops may be modified by the Soils Consultant during the first stage of pounding.

3.7 After 1/3 of the area has been pounded, the Contractor shall grade and proofroll the area. Following proofrolling, the Soils Consultant shall take soil borings to verify the compaction of the area. It may be required that the Contractor shall repound, grade, and proofroll the area as direacted by the Soils Consultant.

3.8 The same procedure shall be followed when 2/3 of the area has been pounded, graded, and proofrolled, and again when the entire area has been pounded, graded, and proofrolled.

3.9 The Soils Consultant shall determine when completion of dynamic compaction at a grid point is accepted.

c. Example 3

This method specification was prepared by a municipal authority for densifying building rubble which extended to a depth of 8 to 10 ft below grade in order to support an industrial building.

The municipal authority prepared detailed specifications so local contractors could bid the dynamic compaction. The municipal authority planned the dynamic compaction in conjunction with a geotechnical consultant, provided the tamper weights, and had engineering personnel in the field to monitor and direct the work. The general contractor provided the rig and personnel to operate the equipment.

The specifications are very detailed as to the equipment, number of drops, grid pattern (etc.). The contractor has no flexibility to modify or alter the dynamic compaction procedure so accepts no risk in so far as performance of the structure. All field adjustments are to be undertaken under the direction of the field engineer, but there are pay items to cover additional work. Thus, any kind of contractor with the proper equipment could bid this work.

The cost for this project can vary depending upon the field adjustments specified by the engineer leading to some uncertainty of the scope of the project making it somewhat difficult for a contractor to bid.

SECTION 2B
DYNAMIC COMPACTION

PART 1 - GENERAL

1.01 REQUIREMENTS

A. The Contractor shall raise and drop a weight (herein-after referred to as the "Pounder") on the grid points shown on the Contract Drawings. Two (2) six-ton Pounders will be furnished by the Authority as provided in Section 3.02 hereof.

B. The Contractor shall grade the compacted area after dynamic compaction in compliance with other provisions of these Specifications.

1.02 COORDINATION AND STAGING

The dynamic compaction shall commence at the test sections A, B, C and proceed to zones denoted 1 and 2 on the Contract Drawings in that order.

1.03 SUBMITTALS

A. The Contractor shall submit for approval his methods for operation and equipment to be used.

B. The Contractor shall furnish to the Engineer at the end of each day, a memorandum showing for that day (a) the number of impacts made, (b) the equipment used, (c) a statement of any unusual happening that occurred, and (d) the number of men that were employed. Such memorandum shall not be deemed to be a substitute for the notices, time slips, memoranda, or other data required under the clauses of the Form of Contract relating to Compensation for Extra Work.

1.04 INSPECTION AND TESTING

A. The Engineer shall evaluate the Pounder's penetration into the ground. If, in the opinion of the Engineer, the Pounder is not penetrating the ground satisfactorily at a particular grid point, the Contractor shall at the Engineer's direction take action as specified in 3.03D hereof.

B. Lifts and drops which do not in the sole opinion of the Engineer meet all the requirements of these specifications shall not be included in calculating the number of drops for which compensation will be paid under Pay Item No. 1 of the Schedule of Unit Prices.

PART 2 - PRODUCTS

DELETED

PART 3 - EXECUTION

3.01 DYNAMIC COMPACTION METHOD

This method of soil compaction shall be accomplished by the lifting and dropping of a six-ton "Pounder" a height between 20 and 40 ft, as directed by the Engineer.

3.02 PREPARATION

A. The Authority will furnish to the Contractor for use by him in the execution of this Contract two (2) six-ton Pounders. From the date the foregoing materials are furnished to the Contractor, they shall form a part of the materials included in the risks assumed by the Contractor as provided in subparagraph (c) of the clause of the Form of Contract entitled "Risks Assumed by the Contractor".

It is presently expected, but not guaranteed, that the Pounders will be furnished to the Contractor by the times required. They will be delivered tailboard of truck at the construction site. The Contractor shall unload same for use under this Contract.

Upon the completion of the Work, or when there is not longer any need for the Pounders, whichever may occur first, the Contractor shall return the Pounders to the Authority at a location at the construction site, designated by the Engineer.

B. The Contractor shall supply and operate two (2) free spool type single cable cranes, fifty (50) ton minimum capacity, adequate to permit the dropping of a six-ton Pounder a distance of forty (40) feet from the bottom face of the Pounder to the ground. The second crane will not be required for Work in Test Sections A, B, or C.

C. The contractor shall connect the Pounder to the crane cable, and may use an energy absorbing connection.

D. The Contractor shall control the limits of crane "boom and swing" to allow for the required lift height and grid point impact.

3.03 APPLICATION

A. Dynamic compaction impact locations shall be on a 7-ft X 7-ft square pattern unless modified in accordance with these Specifications under the direction of the Engineer.

B. The Pounder shall be dropped to the ground from a predetermined height under its own weight, in a free fall without restraint from the crane mechanism, except for natural friction, such as between the line and drum.

C. The Engineer will determine when dynamic compaction at a grid point has been satisfactorily completed.

D. Existing Obstructions

1. Exploratory excavations by the Authority have indicated the presence of foundation walls at the location of demolished buildings as shown on the Contract Drawings. The Authority makes no representation that the information indicated is typical or complete or that other obstructions do or do not exist at these or other locations.

2. Based on the Engineer's evaluation, the Engineer may direct the Contractor to:

a. pound on the existing obstruction, or

b. pound on either side of the existing obstruction, or

c. excavate the existing obstruction and backfill in accordance with the Specifications.

E. If the Pounder penetrates to a depth exceeding eighteen (18) inches, the Contractor shall, at the Engineer's direction, backfill the crater with adjacent suitable on-site material and resume pounding at the grid points.

F. The allowable distance from the geometric center of the impacted pounder to the grid point shall not exceeding 12 inches.

G. The Engineer will evaluate the Pounder's penetration into the ground. If, in the opinion of the Engineer, the Pounder is not penetrating the ground satisfactorily at a particular grid point, the Contractor shall at the Engineer's direction, take action as specified in paragraph 3.03D of this Section of the Specification.

3.04 RECORDS

The Contractor shall assist the Engineer who will record certain information relating to the dynamic compaction process including, but not limited to the following, by supplying information requested by the Engineer:

A. Date

B. Grid Point Designation

C. Number of Pounder Impacts

3.05 TEST SECTIONS

A. When and as directed by the Engineer, the Contractor shall perform Dynamic Compaction and various operations within the areas designated as Test Sections "A", "B", and "C". These operations shall include, but not be limited to, excavation of Test Section "C" to the elevation shown on the Contract Drawings and cooperation and assisting the Engineer to the extent necessary to permit the Engineer to perform all required testing in Test Sections "A", "B", and "C".

B. Suitable material excavated from Test Section C to the elevation shown on the Contract Drawings, shall be temporarily stockpiled outside the limits of the Test Section until the testing in Section C is completed. Such material shall then be backfilled and dynamic compaction repeated at all grid points in Section C.

SECTION 2C
DYNAMIC COMPACTION

PART 1 - GENERAL

1.01 REQUIREMENTS

A. The Contractor shall raise and drop a weight (herein called the Pounder) on the grid points shown on the Contract Drawings. Two (2) six-ton Pounders will be furnished by the Authority as provided in 3.02 inches.

B. The Contractor shall grade the compacted area after dynamic compaction in compliance with other provisions of these Specifications.

1.02 COORDINATION AND STAGING

Comply with the requirements of the clause of the Specifications entitled "Sequence of Construction".

1.03 SUBMITTALS

A. Prior to commencing any Work at the construction site, the Contractor shall submit to the Engineer for approval the methods of operation and equipment he proposes to use.

B. The Contractor shall furnish to the Engineer at the end of each day, a memorandum showing for that day (a) the number of impacts made, (b) the equipment used, (c) a statement of any unusual happening that occurred, and (d) the number of men that were employed. Such memorandum shall not be deemed to be a substitute for the notices, time slips, memoranda, or other data required under the clauses of the Form of Contract relating to Compensation for Extra Work.

1.04 INSPECTION AND TESTING

A. The Engineer shall evaluate the Pounder's penetration into the ground. If, in the opinion of the Engineer, the Pounder is not penetrating the ground satisfactorily at a particular grid point, the Contractor shall at the Engineer's direction take action as specified in 3.03D hereof.

B. Lifts and drops which do not in the sole opinion of the Engineer meet all the requirements of these specifications shall not be included in calculating the number of drops for which compensation will be paid under Pay Item No. 4 of the Schedule of Unit Prices.

PART 2 - PRODUCTS

NOT APPLICABLE

PART 3 - EXECUTION

3.01 DYNAMIC COMPACTION METHOD

This method of soil compaction shall be accomplished by the lifting and dropping of a six-ton "Pounder" a height between 20 and 40 ft, as directed by the Engineer.

3.02 PREPARATION

A. The Authority will furnish to the Contractor for use by him in the execution of this Contract two (2) six-ton Pounders. From the date the foregoing materials are furnished to the Contractor, they shall form a part of the materials included in the risks assumed by the Contractor as provided in subparagraph (c) of the clause of the Form of Contract entitled "Risks Assumed by the Contractor".

   Pounders will be furnished to the Contractor by the times required.

   Upon the completion of the Work, or when there is not longer any need for the Pounders, whichever may occur first, the Contractor shall return the Pounders to the Authority at a location at the construction site, designated by the Engineer.

B. The Contractor shall supply and operate free spool type single cable cranes, fifty (50) ton minimum capacity, adequate to permit the dropping of a six-ton Pounder a distance of forty (40) feet from the bottom face of the Pounder to the ground.

C. The contractor shall connect the Pounder to the crane cable, and may use an energy absorbing connection.

D. The Contractor shall control the limits of crane "boom and swing" to allow for the required lift height and grid point impact.

3.03 APPLICATION

A. Dynamic compaction impact locations shall be on a 7-ft X 7-ft square pattern unless modified in accordance with these Specifications under the direction of the Engineer.

B. The Pounder shall be dropped to the ground from a predetermined height under its own weight, in a free fall without restraint from the crane mechanism, except for natural friction, such as between the line and drum.

C. The Engineer will determine when dynamic compaction at a grid point has been satisfactorily completed.

D. Existing Obstructions

1. Exploratory excavations by the Authority have indicated the presence of foundation walls at the location of demolished buildings as shown on the Contract Drawings. The Authority makes no representation that the information indicated is typical or complete or that other obstructions do or do not exist at these or other locations.

2. Based on the Engineer's evaluation, the Engineer may direct the Contractor to:

a. pound on the existing obstruction, or

b. pound on either side of the existing obstruction, or

c. excavate the existing obstruction and backfill in accordance with the Specifications.

E. If the Pounder penetrates to a depth exceeding thirty (30) inches, the Contractor shall, at the Engineer's direction, backfill the crater with adjacent suitable on-site material and resume pounding at the grid points.

F. The allowable distance from the geometric center of the impacted Pounder to the grid point shall not exceed 12 inches.

G. The Engineer will evaluate the Pounder's penetration into the ground. If, in the opinion of the Engineer, the Pounder is not penetrating the ground satisfactorily at a particular grid point, the Contractor shall at the Engineer's direction, take action as specified in paragraph 3.03D of this Section of the Specification.

3.04 RECORDS

The Contractor shall assist the Engineer who will record certain information relating to the dynamic compaction process including, but not limited to the following, by supplying information requested by the Engineer:

A. Date
B. Grid Point Designation
C. Number of Pounder Impacts
D. Total Depth of the Crater

3.05 TEST SECTIONS

A. When and as directed by the Engineer, the Contractor shall perform Dynamic Compaction and various operations within the areas designated as Test Sections A, B1, B2, C, D, E and F. Cooperate with and assist the Engineer to the extent necessary to permit the Engineer to perform all required testing.

CHAPTER II

PRICES AND PAYMENTS

7. UNIT PRICES

The following Schedule of Unit Prices does not constitute an outline of the Work required by the Contract Drawings and Specifications in their present form but merely a list of the items of Classified Work to be used in computing the Contractor's compensation. It contains all such items. The compensation computed therefrom is full compensation for all Work whatsoever required by the Contract Drawings and Specifications in their present form.

In the case of each item of Classified Work, the Work performed will be measured and the Contractor's compensation will be computed as hereinafter provided in this numbered clause. In case of discrepancy between the prices quoted in writing and those quoted in figures, the writing shall control.

The Estimated Total Contract Price is solely for the purpose of fixing the amount in which security is to be maintained by the Contractor for the faithful performance of the Work. Prior to the signature of the comparison of Proposals and of computing damages in the event of a default by the successful bidder in the agreement created by the acceptance of his Proposal. The estimated quantities are given solely as a basis for the computation of the Estimated Total Contract Price. The Authority makes no representation as to what the actual quantities will be and shall not be held responsible even though the estimated quantities are not even approximately correct. Insofar as the Contractor's compensation is based upon Classified Work, it will be computed from actual quantities of Work performed, whether greater or less than estimated quantities.

SCHEDULE OF UNIT PRICES				
Item No.	Estimated Quantities	Items of Classified Work with Unit Prices Written	Figures	
			Unit Prices	Amounts(1)
1	19,000 S.Y.	SITE LEVELING, PER SQUARE YARD. ____________DOLLARS ____________CENTS		
2	40 HOURS	EXCAVATION AND BACKFILL FOR TEST PITS, PER HOUR. ____________DOLLARS ____________CENTS		
3	88 HOURS	PLANT, EQUIPMENT AND LABOR TO PERFORM DYNAMIC COMPACTION IN TEST SECTIONS, PER HOUR. ____________DOLLARS ____________CENTS		
4	29,500 DROPS	LIFTING AND DROPPING OF POUNDER, PER DROP ____________DOLLARS ____________CENTS		
5	19,000 S.Y.	ROLLING, PER SQUARE YARD. ____________DOLLARS ____________CENTS		

(1) The amount for each item shall be computed by multiplying the estimated quantity of that item by the unit price for the item.

SCHEDULE OF UNIT PRICES				
			Figures	
Item No.	Estimated Quantities	Items of Classified Work with Unit Prices Written	Unit Prices	Amounts(1)
6	2000 C.Y.	EXCAVATION, REMOVAL AND DISPOSAL OF EXISTING UNSUITABLE MATERIALS, PER CUBIC YARD. __________ DOLLARS __________ CENTS		
7	12000 C.Y.	EXCAVATING, HAULING AND STOCKPIPING OF EXISTING UNSUITABLE MATERIAL, PER CUBIC YARD. __________ DOLLARS __________ CENTS		
ESTIMATED TOTAL CONTRACT PRICE (2)				

(1) The amount for each item shall be computed by multiplying the estimated quantity of that item by the unit price for the item.

(2) The Estimated Total Contract Price shall be computed by totaling the amounts inserted in the "Amounts" column, computed as described in (1) above.

d. Example 4

The following documents were prepared by a State Highway Department for densifying an existing landfill. The documents are divided into two portions:

1. A general performance specification (initial four pages), outlining the extent and basic requirements of dynamic compaction from which specialty contractors could prepare a proposal.

2. A contract (last 5 pages) with the specialty contractor that submitted the best proposal to the Highway Department.

This method of engaging a contractor is rarely used. This document was prepared for one of the first projects in the U.S. using dynamic compaction on a recent landfill. The project was somewhat of an experiment in that the amount of energy to apply, tamper weight, and drop height, evaluation of the effectiveness of dynamic compaction, and likelihood of success were difficult to predetermine. Thus, the first document requests a "design and build" proposal to evaluate from specialty contractors solicited by the Highway Department. The method for prequalifying contractors was not specified. The best proposal was then accepted and the second document prepared to put the proposal in contract form.

The contractor assumed some risks with this project. If long-term settlement is deemed excessive by the department, the contractor will be required to share in mud-jacking the roadway. This seems somewhat harsh since the "state of the art" and long-term performance of embankments in recent dynamically compacted landfills has not been established. Furthermore, long-term settlement of organic deposits due to decomposition will occur and the department should be aware of this and assume the risks.

The work was done on a lump sum basis but a large part of the cost was associated with placement of a granular fill in the landfill to facilitate dynamic compaction. This item should have been separated from dynamic compaction.

COORDINATION AND SUPPORT OF DYNAMIC COMPACTION OPERATION

DESCRIPTION: This item shall consist of specifications relating to the coordination of work during construction operations and shall be supplementary to Section 105, Control of Work, of the Standard Specifications, Edition of 1978.

Coordination of work will be necessary with a Specialty Contractor, to be designated by the Department, who will be responsible for the Dynamic Compaction of the sanitary landfill contained within the limits of this Job. The Specialty Contractor will provide operators and equipment necessary for the Dynamic Compaction of materials necessary for the continuous support of the Dynamic Compaction operations.

Each Contractor involved shall assume all liability, financial or otherwise, in connection with his contract and shall protect and save harmless the Department from any and all damages or claims that may arise because of inconvenience, delay, or loss experienced by him because of the presence and operations of the other contractor working within the limits of this project.

CONSTRUCTION: The Dynamic Compaction technique involves the dropping of a heavy weight (10-20 tons) free fall from a height of 50-100 feet. The high energy levels developed produce a deep compaction of the in-situ materials. Two or more passes are made over the site dropping the weight according to a predetermined impact grid pattern. Granular material is used as a working platform for the heavy crane used to mobilize the weight and as material for filling the impact craters.

The Contactor shall schedule and perform the support operations required by the Dynamic Compaction of the landfill site located between Station 172+00 and Station 180+00.

Support of the Specialty Contractor shall include, but not be limited to, the following items:

1. The Contractor will be required to prepare the sanitary landfill site in order to accomplish the Dynamic Compaction as soon as possible after the work order for the project is given. The Department and the Specialty Contractor will require a minimum of two weeks' notice in order to mobilize the pre-engineering and testing equipment and a minimum of three weeks to mobilize the compaction equipment.

2. Access to the entire site for equipment and personnel. Special attention must be paid to access to the south slope; this access shall be coordinated with the Specialty Contractor.

3. Fences and barricades which are a part of normal site security.

4. A static load test shall be conducted before and after the Dynamic Compaction treatment. The Contractor shall construct a pile of material approximately 40 feet in diameter and 20 feet high, creating a load of approximately 150 tons, exerting a pressure of one ton/sq. ft. on a steel plate placed on the surface with a 3 to 4-inch diameter PVC pipe extending up through the center. The Specialty Contractor shall supply and utilize this pipe and plate for testing over a period of seven days. After completion of the individual load test, the loading material must be removed by the Contractor. The Contractor will be allowed to utilize Granular Material used in Dynamic Compaction of Sanitary Landfill in the initial test loading and Unclassified Excavation in the final test loading; however, no direct payment will be allowed for the construction or removal of the test load, and payment for the materials will be allowed only if these materials are used in the final completed roadway structure.

5. An acceptable bulldozer shall be required throughout the Dynamic Compaction operations.

6. Five (5) feet of Granular Material used in Dynamic Compaction of Sanitary Landfill must be placed over the landfill area to be compacted. The granular material will serve as a construction platform and shall be graded with substantial compaction in order to support the construction equipment during the Dynamic Compaction. The platform shall be graded to drain and shall be suitable for movement of a large crawler crane and other equipment.

7. During the final compaction pass, as much surrounding loose granular material as possible will be utilized to level out the prints. After the final compaction pass is made, the entire area will be uniformly graded, scarified to a depth of 8 inches and compacted to a density, as

determined by AASHTO T191 or T238, of not less than 95% of the maximum density obtained by AASTHO T99, with correction for varying amounts of coarse particles in accordance with AASHTO T224.

The Contractor shall have the necessary equipment, operators, and materials available at the beginning and/or end, on any intermediate point of the Dynamic Compaction operation, to insure that the sequence of work on the project will progress in an expeditious manner.

A representative of the Specialty Contractor will be in attendance at a pre-bid conference at the Department's Central Office Auditorium at 10:00 a.m. on September 9, 1981, to answer questions concerning the required support for the operation. The representative will be available the entire day for consultation with prospective bidders.

The Contractor will coordinate with the Department prior to placement of roadway embankment on the landfill site following Dynamic Compaction operations to allow for the installation of long-term monitoring equipment by the Department.

MEASUREMENT AND PAYMENT: This item will be measured as a complete unit and will be paid for at the contract lump sum price bid for Coordination And Support Of Dynamic Compaction Operation, according to the following schedule:

PARTIAL PAYMENT SCHEDULE

a. 15% of the total lump sum cost upon completion of mobilization by the Contractor;

b. 70% of the total lump sum cost in separate payments upon 100 percent of work completion, as agreed upon by the Department and the Contractor; and

c. 15% of the total lump sum cost upon acceptance of the Dynamic Compaction operation.

The items of Granular Material used in Dynamic Compaction of Sanitary Landfill, Borrow Material, and Unclassified Excavation will be measured and paid for under their respective bid items.

Work performed and paid for under this item will be paid for at the contract bid price, which shall be full compensation for furnishing all equipment, tools, labor, and incidentals necessary to complete the work.

Payment will be made under:

Pay Item	Pay Unit
Coordination and Support of Dynamic Compaction Operations	Lump Sum

AGREEMENT FOR PROJECT SERVICES

This Agreement, entered into and executed this __________ day of __________, 19___, by and between the State Highway and Transportation Department, acting by and through its Director of Highways and Transportation who is so authorized to act by the State Highway Commission, hereinafter called the "DEPARTMENT", and Company, hereinafter called the "CONTRACTOR".

WITNESSETH:

WHEREAS, the Department is undertaking the Dynamic Compaction of a sanitary landfill, and

WHEREAS, funds are available for the project; and

WHEREAS, the Contractor's staff is adequate and well qualified;

NOW THEREFORE, it is considered to be in the best public interest for the Department to obtain the services of the Contractor's organization in connection with said construction and engineering project. In consideration of the faithful performance of each Party of the mutual covenants and agreements set forth hereinafter, it is mutually agreed as follows:

SECTION I -- EMPLOYMENT OF CONTRACTOR

The Department agrees to employ the Contractor to perform, and the Contractor agrees to perform the Dynamic Compaction of a landfill site, hereinafter designated Project, as set forth in the Sections to follow: and the Department agrees to pay, and the Contractor agrees to accept, a lump sum fee, as specified in the Sections to follow, as full and final compensation for work accomplished in the specified time.

SECTION II -- DESCRIPTION OF THE PROJECT

The Project consists of the Dynamic Compaction of a sanitary landfill site in accordance with the Request for Proposal and the Contractor's Proposal which are hereby included by reference.

The landfill was permitted in February, 1977, and officially closed in November, 1978. The site was permitted to serve the area and/or nearby towns with a total population of approximately 65,000. The estimated sources of landfill materials were:

Residential	40%
Commercial	20%
Industrial	40%

No industrial and/or hazardous wastes were to be disposed of at the site except by special permission by the Department of Pollution Control and Ecology. Total area of the landfill and the area to be treated is approximately one hundred eighty five thousand (185,000) square feet; maximum depth of the refuse in the landfill is approximately thirty-five (35) feet below the present grade.

The Project shall consist of developing a method of Dynamic Compaction for treating the landfill and employing this method in compacting the landfill to provide adequate soil support for the intended purpose of highway construction.

SECTION III -- CONTRACTOR'S RESPONSIBILITIES

1) The Contractor shall have a representative in attendance at the Department's Central Office Auditorium to answer questions concerning the support required of the Prime Contractor of the entire roadway construction.

2) The Contractor shall mobilize pre-engineering and testing equipment within two (2) weeks upon receipt of a Work Order; and the Contractor shall mobilize the compaction equipment within three (3) weeks upon receipt of this same Work Order. The Contractor shall complete the Project within one hundred forty (140) calendar days upon receipt of the Work Order.

3) The Contractor shall provide, at his expense, for the testing as specified in the Contractor's Proposal. This testing is to assure that adequate soil support values are achieved during and upon completion of the Project. The Contractor shall be responsible for monitoring of equipment related to this testing and recording of test data.

4) The Contractor shall coordinate with the contractor responsible for the entire roadway project to assure the expeditious completion of the project.

5) The Contractor shall maintain daily records of time and progress on the Project.

6) The Contractor shall provide the equipment, tools, labor, and incidentals necessary to Dynamically Compact the entire sanitary landfill site, located between Station 172+00 and Station 180+00 and encompassing approximately 185,000 square feet.

7) The Contractor shall be responsible for all surveying and layout of impact areas required for the Project except as noted in Section IV.

8) If the differential settlement is deemed excessive by the Department within five (5) years, the Contractor will share the costs to perform deep slab jacking by "compaction grouting" in an effort to level out effected areas. This work will be performed on a 50-50 basis with the Department, with the Contractor supplying the supervision and special equipment at 50% of their then current rates and the Department supplying the remainder of the labor and materials.

9) The Contractor will furnish to the Department a draft final report within thirty (30) days after the completion of the Project describing overall success of the Project, a narrative of the Project's operations, initial and final test results and any additional significant information. This draft final report will be reviewed and commented upon by the Department and the Federal Highway Administration within forty-five (45) days. These comments will be considered for incorporation into the final report. This final report shall be received by the Department within thirty (30) days of the submittal of comments by the Department. This final report shall be considered the property of the Department; the Department and the Federal Highway Administration may make use of any and all material contained in this final report as they deem appropriate.

10) At the execution of the contract, the Contractor shall furnish a surety bond or bonds in a sum equal to the full amount of the contract. The form of the bonds and the security shall be acceptable to the Department and shall be signed or countersigned by an Agency of the surety with Power of Attorney to support his signing authority.

11) If the Contractor shall fail to complete the work within the time limit herein specified, he shall pay to the Commission, as liquidated damages, and not in the nature of a penalty, the sum of One Hundred Fifty Dollars ($150.00) for each day delayed, it being understood and agreed between the parties hereto that the said sum fixed as liquidated damages is a reasonable sum, considering the

damages that the Commission will sustain in the event of any such delay, and said amount is herein agreed upon and fixed as liquidated damages, because of the difficulty of ascertaining the exact amount of damages that may be sustained by such delay. The said sum shall be deducted from the final amount of estimate due the Contractor.

SECTION IV -- DEPARTMENT'S RESPONSIBILITIES

The Department will gather and provide any and all relevant data on hand and provide a reasonable effort toward finding other information as may be requested.

The Department will set construction stakes establishing the centerline and right-of-way limits prior to the beginning of Dynamic Compaction operations and as deemed necessary by the Engineer.

Work control for the project shall be in accordance with Section 105, Control of Work, of the Standard Specifications, Edition 1978. The Department shall assign a Resident Engineer as its direct representative for the Project; the Resident Engineer shall have immediate charge of the engineering details of the Project.

The Department shall install and monitor equipment it deems necessary for long-term monitoring of the Project following completion of the Dynamic Compaction operations.

SECTION V -- FEES AND PAYMENT

For and in consideration of the services rendered by the Contractor, the Department shall pay the Contractor ____________. Payment under this contract will be paid for at the contract lump sum price and will be paid by the Department to the Contractor according to the following schedule:

a) 15% of the total lump sum cost upon completion of mobilization by the Contractor;

b) 75% of the total lump sum cost in separate payments based upon percent of work complete, as agreed upon by the Department and the Contractor; and

c) 10% of the total lump sum cost upon acceptance of the Project and receipt of the final report.

Work performed and paid for will be paid for at the contract price and shall be full compensation for furnishing all equipment. tools, labor, and incidentals to complete the Project.

SECTION VI -- STANDARD SPECIFICATIONS AND SPECIAL PROVISIONS

Applicable to this Contract, but not bound herein, are the State Highway Commission Standard Specifications for Highway Construction, Edition of 1978.

CONTRACTOR

____

By____________________________

STATE HIGHWAY COMMISSION

____

By____________________________

e. Example 5

Method Specifications for 5-story building in an urban area where the thickness of loose sand and rubble fill was less than 10 ft, underlain by clay soils. The building was designed for footings bearing on the fill and with the slab bearing on the fill.

The geotechnical consultant provided sufficient information on the tamper weight, drop heights, grid spacing, and areas to densify so that local contractors could bid the work. A wrecking contractor who owned the proper size weight was awarded the project.

These specifications are very brief and can be used only on simple, small projects. It is adequate to specify the tamper weight, drop height and grid spacing rather than energy because very few field adjustments in energy application are foreseen or even desired other than altering the number of blows. The weight size and drop has been predetermined for compacting the full thickness of the loose zone so altering the tamper weight or drop height is not necessary and could even disturb the underlying clayey soils if too much energy were applied during a single drop. The sand and rubble fill compacts well without the need for multiple passes so adjustments in this regard are not necessary.

While not shown, this project was bid on the basis of a lump sum for the dynamic compaction based upon a given area for dynamic compaction shown on the Phase I and II drawings plus a unit price for additional drops and for removal of unsuitable soil and replacement with new fill.

POUNDING SPECIFICATIONS

1a. Clear and grub all plants, trees and topsoil from the surface of the area that will be pounded. Remove the brick road, concrete paved alleys, and sidewalks from the proposed building area.

1b. The existing 9 inch sewer line and 4 inch water main will be removed from the building area by others under a separate pay item prior to pounding. All other utilities in the building area will be terminated prior to pounding. Refer to site preparation section for discussion on utility removal and/or termination.

2. All pounding should be performed prior to installing the building foundations.

3. The ground surface in the building area shall be pounded by dropping a weight of 6 tons through a distance of 30 to 40 ft. A free spool-type crane with a single cable (40 to 50 ton capacity) shall be used to minimize loss of energy of the falling weight. The weight must have a contact pressure of 750 psf or greater. A 6 ton weight, 4.5 ft in diameter, has a contact pressure of 750 psf.

4. The weight shall be dropped nine (9) times at each grid point. The pounding shall take place in two phases.

5. Phase I pounding shall consist of pounding the building area in a grid pattern with a grid spacing of 8 ft. The pounding area shall extend approximately 10 ft beyond the building lines (see Phase I Pounding Diagram). After the pounding is completed, the ground surface should be leveled and compacted to a minimum of 95% of the maximum density obtained in accordance with ASTM Specification D-1557.

6. Phase II pounding shall consist of pounding the footing areas in a grid pattern with a grid spacing of 8 ft (see Phase II Pounding Diagram).

7. In the event that soft or unsuitable areas are noted during pounding or during check soil borings after pounding, they shall be removed to the extent indicated by the Engineer. The excavated soils shall be replaced with approved, well-graded granular backfill free of organic matter and debris. The excavation and fill replacement in those areas specified by the Engineer will be done on a time and materials basis.

8. Ground surface elevations will be taken at representative points both before and after the pounding by the Engineer. No material should be removed or added to the pounded area during this period unless directed by the Engineer.

9. Seismic analysis for vibration monitoring will be performed during pounding by the Engineer to determine a safe distance between buildings, sewer lines, etc. and impact areas.

10. All fill material used to achieve the final floor slab grade shall consist of approved, well-graded granular backfill, free of organic matter and debris. This fill should be placed in lifts not exceeding 12 inches in loose thickness and compacted to 95% of the maximum density in accordance with ASTM Specification D-1557, Modified Proctor Method. In areas where floor loads are anticipated to be less than 500 psf, the fill may be compacted to 90% of the aforementioned recommendations. A minimum compacted thickness of 6 inches of such fill shall be placed under the slab.

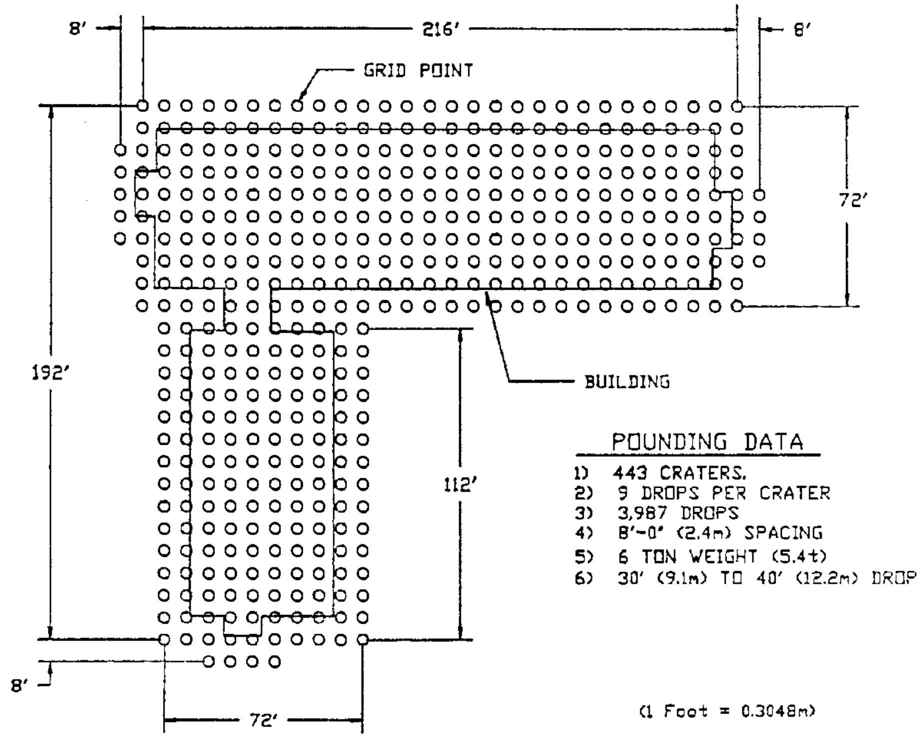


Figure 49 : Phase I pounding diagram

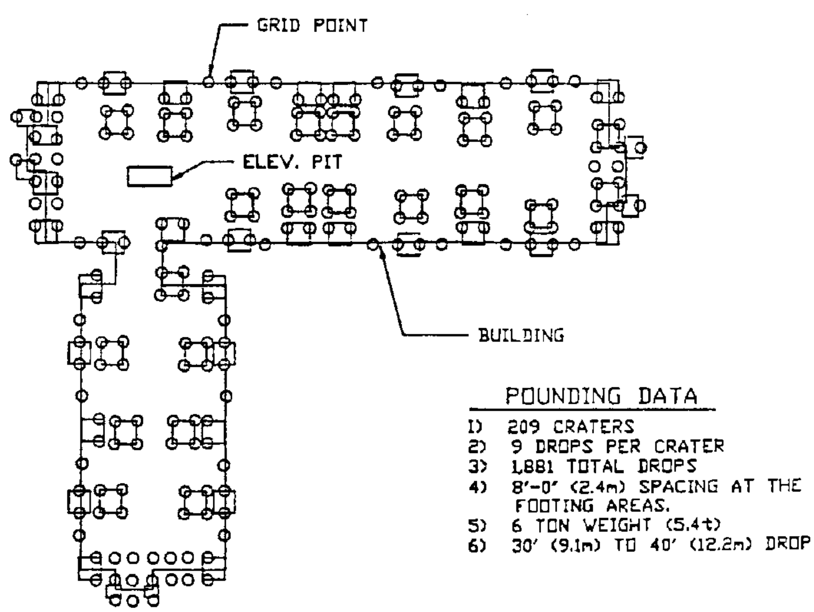


Figure 50 : Phase II pounding diagram

f. Example 6

The contractor is required to densify the soils until a minimum relative density is obtained using cone penetration testing. A wide range in tamper weight and drop height is specified allowing the contractor freedom to adjust the energy per drop. The contractor is also free to select the applied energy, number of passes, and number of drops per pass to attain the final goal. This is an example of a performance specification.

The minimum requirements of improvement following densification are clearly spelled out in Section 3.2.2 and methods for improving deficient areas is contained in Section 3.3, so there should be no misunderstanding of what is required.

To accomplish this work requires an experienced dynamic compaction contractor and there is no mention of prequalification in this section.

Certification testing is required to be performed by the contractor and this is typical of many projects. However, it would be in the owner's best interest if this testing were done by the owner or his consultant. This would avoid complications of acceptance of the contractor's work by the contractor's tests.

Requiring the contractor to submit a test report (Section 4.2.1) following completion of the test program appears to place great reliance on the contractor's judgment and assessment of the improvement. Submittal of all the test data obtained by the contractor should suffice and the interpretation left up to the department or their consultant.

In spite of the comments presented above, this is a good performance specification.

CONTENTS

1.0 SCOPE

1.1 General

1.1.1 This specification covers the technical and other requirements for subsurface densification by the Dynamic Consolidation method.

1.1.2 It is not the intent of this specification to outline all the technical requirements nor to set forth those requirements adequately covered by applicable codes, specifications and standards. Contractor shall perform high quality construction work meeting the requirements of this specification and industry standards.

1.2 Work to be Provided by Contractor

Contractor's work shall include, but not necessarily be limited to, providing all items of labor, material and equipment necessary to perform the work densifying the foundation and soil by Dynamic Consolidation.

1.2.1 Contractor shall establish and maintain horizontal and vertical ground control during execution of the work.

1.2.2 The zone of densification shall extend from elevation of bottom of footings, as shown on the drawings, to the cemented sand stratum. The approximate depth to the cemented sand stratum is shown on the boring profile drawings.

1.2.3 Contractor shall perform all earthwork required to provide a working surface after each compaction pass.

1.2.4 Contractor shall provide for the diversion and removal of all surface water, construction water, rainfall and/or ground water by means of ditching, damming, pumping or other suitable means deemed acceptable to Engineer. All water shall discharge into the Runoff Sediment Control Pond located on the site. All procedures and drawings shall be submitted for Engineer's review and comment.

1.2.5 Contractor shall establish and maintain a quality program in accordance with the Contract "Special Conditions".

1.3 Work to Be Provided by Owner

Owner's work will include the providing of design drawings indicating the exact limits of the Dynamic Consolidation work areas. Owner will also install baselines and benchmarks and perform field tests required to audit Contractor's quality program.

2.0 CODES, SPECIFICATIONS AND STANDARDS

2.1 General

2.1.1. Material and services furnished in accordance with this specification shall comply with the codes, specifications and standards listed in Paragraphs 2.2 and 7.1. Later editions may be used by mutual consent in writing between Contractor and Engineer.

2.1.2 Any conflict between this specification and the referenced codes, specifications and standards shall be immediately brought to Engineer's attention for written resolution.

2.2 Listing

ASTM - American Society for Testing and Materials

- D1586-76 Method for Penetration Test and Split-Barrel Sampling of Soils
- D3441-79 Standard Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil

OSHA - Occupational Safety and Health Administration

Regulation 29 CPR Part 1926 - Occupational Safety and Health

Regulation for Construction (October 1, 1979)

3.0 TECHNICAL REQUIREMENTS

3.1 Subsurface Densification

The subsurface foundation soils shall be densified by the Dynamic Consolidation method. The method shall consist of providing large energy impacts at the ground surface by dropping a pounder, 16 to 60 tons in weight, from a height of 60 to 100 ft. The weight shall be dropped repeatedly (at locations called prints) on a grid pattern covering the entire work area. A pass shall consist of a grid of prints over a specific area to be treated. Additional passes shall be made as necessary, either using the same print locations or split spacing, until the required results are achieved. After each pass, the Contractor shall re-level the work area.

3.1.1. A work program including the pounder weight, drop height and grid pattern to be used shall be established by the Contractor and approved by the Engineer prior to start of work. The final program shall be established by a test section discussed in Section 4.2.

3.2 Densification Criteria

3.2.1 It is required that the soils be densified to provide a bearing capacity of 8 kips per square foot for live and dead loads and 10 kips per square foot for live, dead and wind or seismic loading.

3.2.2. The average relative density for each test location through the zone of densification, as defined in Paragraph 1.2.2., shall be at least 85 percent. Within this zone, the average of any three consecutive relative densities shall be at least 75 percent. No single relative density value shall be less than 85 percent within the top 15 feet of the zone of densification and no less than 65 percent in the remaining portion of this zone. Relative density values less than 75 percent shall be brought to Engineer's attention for review and acceptance.

3.2.3 Certain existing soil borings indicate the presence of localized pockets of soft cohesive soil or loose material at a depth of approximately 55 feet corresponding to the contact interface with the underlying dense to very dense cemented sands. If and when such pockets are encountered at any elevation within the zone of densification, Engineer may direct Contractor to apply the Vibroflotation method of densification.

3.2.4 In local areas where the relative density criteria presented in Paragraph 3.2.2 are not satisfied, Contractor may, with acceptance of Engineer, perform additional testing to verify the compressibility of the foundation. These tests may be pressuremeter type together with calculations that verify that the differential settlements due to the local conditions will be within tolerable limits. The maximum differential settlements tolerated shall be 1/4 inch between structural columns as determined by Engineer.

3.3. Re-Compaction

If the after-densification tests indicate that in an area of work the densification criteria have not been achieved, Contractor shall re-compact the area within 25 feet of each side of the test location and the area shall be re-tested. This procedure shall be repeated until the densification criteria are satisfied.

3.4 Field Control

Contractor shall accurately lay out and maintain the compaction points of the compaction pattern using numbered wooded stakes.

3.5 Daily Log

Contractor shall log and record the energy exerted at each compaction point on a Contractor's daily log form. Engineer shall be permitted to witness the recording of, and make comments on, the log form. A copy of the form shall be submitted to Engineer at the end of each work shift.

3.5.1 Information presented on the daily log shall include, but not limit to, the following:

a - Compaction point number and pass number
b - Ground surface elevation before compaction
c - Start and finish time at each compaction point
d - Pounder weight and height of free fall
e - Number of unusual circumstances encountered during performance of the compaction

3.6 Certificate of Compliance

Contractor shall furnish a Certificate of Compliance stating that all work and materials furnished comply with this specification and any accepted deviations that may arise and are agreed upon during construction.

4.0 INSTALLATION

4.1 General

Contractor's construction services shall be in accordance with the applicable provisions of OSHA Regulations 29 CFR Part 1926.

4.2 Test Program

The work program required in Section 3.1.1 shall be verified or modified as required based on the result of a test program. The test program shall involve an area of one acre or larger as required to define the work program. Parameters such as the optimum number of consecutive drops in a single location, the weight of pounder, the height of drops, the pint spacing, and the time required for pore pressure dissipation, shall be determined by the evaluation of the test program. These parameters shall be included in the production work program which shall be subject to the review and acceptance of Engineer.

4.2.1 A test report shall be prepared and submitted to Engineer following completion of the test program.

4.2.2 The test program shall be performed within the limits of densification and will be accepted as production work upon satisfactory compliance with the requirements of this specification.

5.0 TESTS

5.1 Quality Program

Contractor shall perform field and laboratory control tests of the foundation soil densification as specified for the static cone penetration tests. The testing shall be performed in accordance with Contractor's quality program. Engineer shall be notified of the time and location of testing at least five (5) days prior to testing. All testing shall be performed in the presence of Engineer unless authorized in writing by Engineer.

5.2 Densification Testing

At a minimum, before and after densification testing shall be performed by Contractor using static cone penetration tests (CPT). Evaluation of test results will be performed by Contractor to verify achievement of the specified densification criteria. Frequency of before and after densification testing shall be at a minimum rate of one (1) CPT per 2,500 square feet and located on an approximate 50 foot grid pattern. Where appropriate, test locations shall be offset from the grid pattern to a proposed footing foundation area.

5.2.1 Densification will evaluated by relative density (D_r) determination utilizing correlation of the q_c values obtained from static cone penetration tests (CPT), ASTM D3441.

5.2.2. The relative density will be calculated from q_c values obtained at 7.9 inch intervals and averaged over a 39.4 inch depth.

5.3 Audit

Contractor's quality program will be subject to evaluation and audit by Engineer. Audit will include field and laboratory testing and documentation review.

6.0 CONTRACTOR'S DATA SUBMISSION AND SCHEDULE

6.1 Data Submission

Contractor shall submit the following information and data in accordance with the Contract "Special Conditions".

6.1.1 An overall schedule of work to demonstrate Contractor's ability to perform work within the specified time.

6.1.2 A quality program including written policies, procedures and instructions for the inspection, testing and documentation of the work performed.

6.2 Data Submission Schedule

Contractor shall provide the listed information in accordance with the following schedule:

Item Description	Specification Paragraph	Required Submittal Time (weeks)	Engineer's Review Time (weeks)
Overall Schedule	6.1.1.	With Proposal	Not Applicable
Technical Data	7.3	With Proposal	Not Applicable
Quality Program	6.1.2	4 After Contract Award	4
Water Diversion and Removal Procedure	1.2.4	4 After Contract Award	4
Work Program	3.1.1.	4 After Contract Award	4

Item Description	Specification Paragraph	Required Submittal Time (weeks)	Engineer's Review Time (weeks)

Test Program	4.2	4 After Contract Award	2
Test Report	4.2.1	3 After Completion of Test	Not Applicable
Certificate of Compliance	3.6	2 After Completion	Not Applicable

7.0 TECHNICAL DATA

7.1 General Requirements

State or Local Ordinances ____Where Applicable____

7.2 Technical Data by Engineer

Subsurface information including foundation studies, boring logs, laboratory test results and vibroflotation test reports are available at Engineer's office for Contractor's inspection if desired.

7.3 Technical Data by Contractor

7.3.1 Type of Equipment to be Used:

Tripod(s) ____________________

Crane(s) ____________________

Pounder(s) ____________________

Earthmoving Equipment ____________________

APPENDIX B - LANDFILL SETTLEMENT

1. Factors Affecting Settling Rates in Landfills

There are several factors which affect the future rate of settling of refuse in a landfill. These are:

- The current state of compaction of the refuse.
- The moisture content of the refuse.
- The composition of the in-place refuse.
- The landfill environment.

a. Compaction of the Refuse

The degree of future refuse settlement is to a large extent based on how much a landfill is already compacted[80]. The degree of compaction is effected primarily by the following parameters:

(1) The Use of Compactors on Site

Various claims are made for different types of compactors. The density of the refuse in hauling trucks is typically 300 to 500 lbs/yd^3 (.18 to .3 Mg/m^3). As a rule of thumb, refuse densities are typically 700 to 900 lbs/yd^3 (.42 to .53 Mg/m^3) when compactors are not in use at a landfill and from 1000 to 1500 lbs/yd^3 (.6 to .89 Mg/m^3) when they are in use.

(2) The Topography of the Landfill

The topography of the landfill affects compaction in two ways. First, the depth of the overburden effects the density of the refuse. Typically, refuse that is less than 30 ft (9 m) deep has a density of 700 to 800 lbs/yd^3 (.42 to .53 Mg/m^3), refuse that is 30 to 60 ft (9 to 18 m) deep has a density of 1000 to 1300 lbs/yd^3 (.6 to .77 Mg/m^3), and refuse greater than 60 ft (18 m) deep has a density of 1400 to 1500 lb/yd^3 (.83 to .89 Mg/m^3). Second, the topography can limit the effectiveness of compactors. That is, it is easier to gain compaction on level areas than on slopes.

(3) The Moisture Content of the Landfill

Wet refuse has a higher bulk density than dry refuse[39,67]. Water in itself is more dense than refuse. Also, the wetting of particles increases their adhesion to each other and so increases the density. Wetting of upper layers would increase the overburden and so the compaction.

(4) The Cover Used on the Site

Because soil has four times the density of refuse, the higher the percentage of cover materials, the higher the density of the trash. Additionally, overburden loading tends to collapse the pore spaces in the refuse. Conversely, the more extensive the cover, the lower the percentage of moisture, and the lower the density.

(5) Refuse Materials

It is difficult to judge the precise effect differences in refuse composition would have on compaction. For example, the failure to segregate large objects (such as tree stumps and appliances) may impede the performance of compactors. Also, while demolition debris may be, in itself, dense, only poor compaction may be achievable because of large particle size. Refuse materials with a high water content (such as putrescible wastes) would increase refuse density.

(6) Pretreatment of Waste

Refuse may be treated in several ways before decomposition. Commonly, refuse is deposited "as is" as it comes off the truck. Apart from incineration, the two major pretreatment methods for conserving space in landfills are shedding and baling. Shredding refuse,in itself, decreases refuse density. However, by creating a more uniform particle size, it may allow greater compaction to be achieved[38], and, as will be discussed later, increases the rate of decomposition. Baling creates a greater initial density, typically 50 to 70 lbs/ft^3 (.8 to 1.1 Mg/m^3). Because it prevents the percolation of water through the refuse, it causes a slowing of the decomposition rate.[33] There is evidence that it does not stop the decomposition completely.[90].

(7) Time of Deposition

There is a slow reorientation of landfill materials as they adjust to loading and as they weaken and disintegrate which eliminates void spaces.[2] Landfills can settle as much as 20 percent of the initial fill depth the first year and may continue to settle at reduced rates logarithmically.[99] It has been found that 90 percent of the ultimate settlement of a landfill

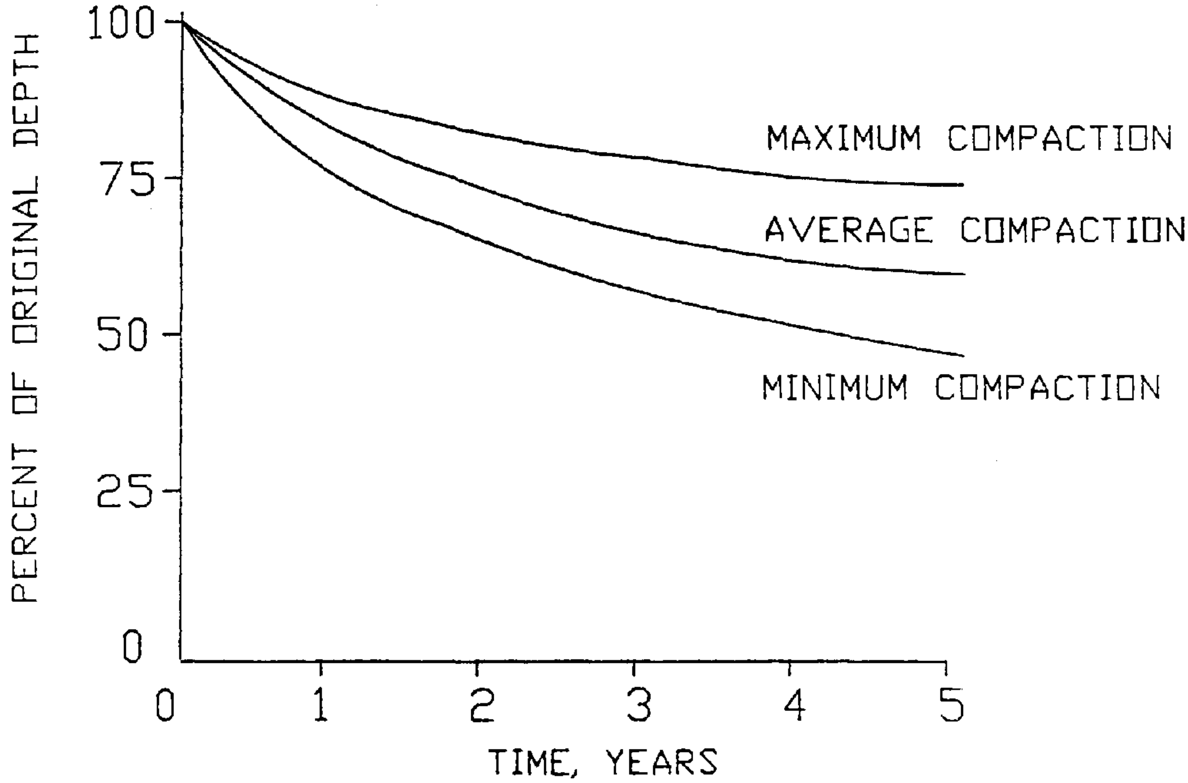
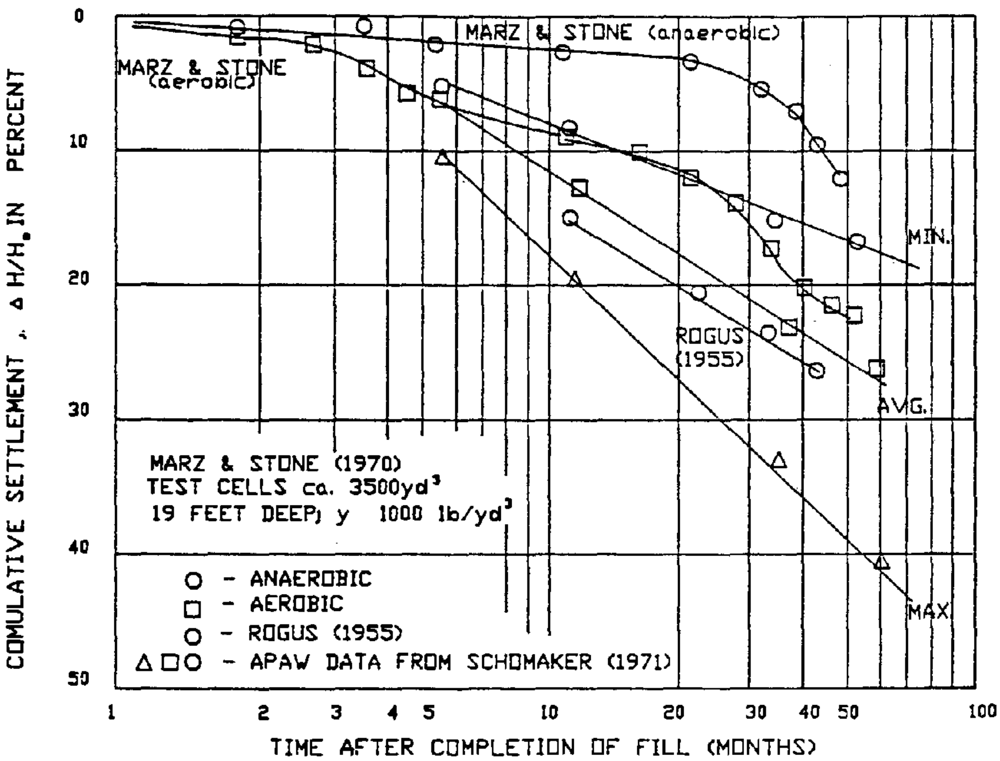


Figure 51 : Surface settlement of compacted landfill (91)



(1 Foot = 0.3048m , 1 Yard = 0.9144m)

Figure 52 : Settlement of landfills no external load (9)

occurs within the first five years (Figure 51). The time rate of settlement of sanitary landfills under self weight in different environmental conditions is shown on Figure 52, which was obtained from Reference 9.

(8) Overburden

The greater the overburden, the greater the compaction of the underlying layers.

b. Contents of Refuse

The amount of moisture in a landfill is the primary factor affecting the rate of decay of the refuse. Settlement occurs as a result of this decay of solids to gases. There are several ways moisture can enter a landfill. They are:

(1) Climate

The amount and pattern of rainfall an area receives is one of the primary factors effecting the amount of moisture that enters a landfill.

(2) Cover Materials

The amount and type of cover used at a site can drastically affect the amount of water present in a landfill. Table 12 shows the theoretical volume of water that could enter 1 ft^2 (.1 m^2) of cover material in one day. Sites with excellent clay cover in areas with high rainfall have been found to have dry refuse even several years after initial deposition.

TABLE 12

Theoretical Volume of Water That Could Enter a Completed Landfill Through 1 Square Foot of Various Cover Materials in One Day

From Reference 91

Cover Material	Volume of Water, Gal.
Uniform Course Sand	9,970
Uniform Medium Sand	2,490
Clean, Well-Graded Sand & Gravel	2,490
Uniform Fine Sand	100
Well-Graded Silty Sand & Gravel	9.7
Silty Sand	2.2
Uniform Silt	1.2
Sandy Clay	0.12
Silty Clay	0.022
Clay (30 to 50 percent clay sizes)	0.0022
Colloidal Clay	0.000022

NOTE: gallon X 0.003785 = m^3
$1\ ft^2 = .1\ m^2$

(3) Landfill Topography and Construction

The landfill topography effects the amount of water falling on a landfill that will actually penetrate the site. Factors which help prevent water infiltration are uniform grading, which creates an absence of low points in which water can pool, a high percentage of sloped areas, and a high percentage of above ground level areas. The presence of impermeable soil layers within the landfill also raises the height of perched water. The irrigation of landscape vegetation in closed landfills can also lead to increased water levels, if the amount of water applied exceeds the evapo-transpiration rate. The presence of a well designed drainage system is one of the most important factors in reducing the moisture content of refuse.

(4) Site Geology

Site geology can effect the moisture content of a landfill in several ways. Most obviously, if a landfill is built on a marsh or stream bed it will have a high water content. Site geology also effects the type and availability of cover materials, which, as previously discussed, determine the degree of water infiltration. Canyon fills in dry areas often have high perched water levels because the rainwater which drains down the center of the canyon is absorbed by the refuse. Quarry fills may have high perched water levels because water cannot percolate out at appreciable rates through the quarry walls. Likewise, a landfill with a natural clay bottom will have a high perched water table.

(5) Added Liquids

The addition of liquids that are not toxic to methane producing bacteria may add substantially to the generation rate of a site.

(6) Density

Tests performed on baled refuse by Northwestern University under contract to the American Public Works Association showed the following percolation rates:

Sample No.	Wet Density lb/yd^3	Wet Density Mg/m^3	Coefficient of Permeability (ft/day)	Coefficient of Permeability (cm/sec)
1	965.2	0.57	42.6	21.3
2	1,323.0	0.79	13.6	6.8
3	1,409.4	0.84	10.0	5.0
4	1,917.0	1.14	2.0	1.0

Sample 1 is typical of a conventional landfill with good compaction. Sample 4 is typical of the more dense bales produced by a high-pressure baler. The American Public Works Association concluded that the 20:1 difference in permeability should result in an equivalent lower flow of water through the refuse for a given hydraulic gradient.[33] This low flow would result in a decrease in decay rates.

c. Refuse Composition

The compaction of refuse present in a landfill can have considerable effect on the degree and rate of refuse decay and compaction. Inorganic substances such as glass, metals and masonry may not decompose, but can be crushed or destroyed by corrosion and leaching. Organic components differ as to their readiness to decay.

Some researchers[84] divide organics into three categories: readily decomposible (food and grass), moderately decomposable (paper, wood, and textiles), and refractory (plastics and rubber). These terms are relative and in actuality decay rates vary several fold between landfills and within a given landfill. Practically, however, typical municipal refuse does not vary considerably in composition and efforts should be made to determine the presence of large percentages of unusual deposits (e.g., a very high percentage of demolition debris) rather than try to document the vast array of items that a particular landfill may accept. Also, some materials, such as soft organic sludges, do not compact readily.

Landfills are typically deficient in nitrogen and phosphorus, a situation which inhibits bacterial growth and refuse decay. For this reason, the deposition of nutrient rich items such as wastewater treatment plant sludges or some types of agricultural wastes will cause substantial increases in refuse decay rates.

d. Environmental Factors

Other factors effecting refuse decay rate are:

(1) Ph

Refuse decays most rapidly at neutral pH. However, the volatile acids formed as a result of biological processes can corrode metals.

(2) Temperature

Generally, the higher the temperature, the higher the gas production. Although methane producing bacteria are reported to be predominately mesophyllic with an optimum temperature range of 20° to 40° C (68° to 104° F), thermophyllic species have been identified, and temperature does not appear to be an inhibiting factor.

(3) Presence of Toxics

Chemical dumping may inhibit bacterial decay of refuse.

It should be noted that pH and temperature are indicative of the stage of the decay cycle of refuse within a landfill. A thorough discussion of refuse decay in landfills can be found in Schumacher[84]. Also, temperature is a function of refuse depth. Since refuse is an effective insulator of heat, temperatures typically increase with depth. Below the first 10 ft (3 m), refuse temperatures are not generally effected by climatic fluctuations.

2. Effects of Dynamic Compaction on Settling Rates

There is little written on the long-term effects of dynamic compaction on landfill settling and decomposition rates. Some studies are in progress. The Arkansas State Highway and Transportation Department[3] used dynamic compaction on a sanitary landfill near Springdale, Arkansas. This landfill operated from 1977 to 1978 and was approximately 35 ft (10 m) deep. Welsh[96] describes the dynamic compaction of this landfill. The compaction created a stiff surface mat 10 ft (3 m) thick, and resulted in a 20 to 25 percent reduction in total landfill volume. Since the density of the landfill prior to compaction was 826 lbs/yd^3 (.5 Mg/m^3) the average density of the landfill after compaction was 1032 to 1101 lbs/yd^3 (.61 to .65 Mg/m^3). These densities are well within the range that is commonly found in actively generating landfills. Most of the increase in density, however, was in the upper 3 meters. This dense mat would probably act as a moisture barrier preventing liquids from infiltrating the landfill. Therefore, refuse decomposition would be slowed both below and within the mat.

The roadway was constructed and opened to traffic in December, 1984. Since then monitoring for differential settlement of this area has continued by the Research Section of the Springdale Residency. In June of 1985, a 150 ft (46 m) long depression was noticed in the pavement. A check of the settlement plates revealed that some areas of the buried landfill has settled as much as 10 in (25 cm). The reason for the difference in the amount of surface and subsurface settlement has not been determined. The settlement area is located along the steep slope of the compacted landfill where the landfill was largely unconfined. Other areas in the landfill have settled very little. Overall, the settlement is not generally noticeable at highway speeds and has not resulted in any loss of ride quality. No corrective measures are planned now, but monitoring will continue.

Charles, Burford, and Watts[14] describe the results of dynamic compaction in five U.K. landfills, three of which contained refuse. At a site with refuse over 40 years old, dynamic compaction of about 9 percent vertical compression (0.58m) was achieved. After 100 days, the refuse had settled an additional 0.06 to 0.08 m. At a site with 15 year old refuse, the tamping settled the refuse 0.5 m (about a 10 percent volume reduction).

Clearly, there is a need for the compilation of historical data on settling rates in landfills where dynamic compaction has occurred. However, the general principles of landfill behavior can be applied to predict what the effect may be.

3. Testing of Landfills

Because of the large number of factors effecting the decay rate of a landfill, it is desirable to perform some type of on-site testing to determine the present state of decay, the potential for further decay, and the rate at which this decay will occur. Three stages are required to accomplish this. These are:

- Initial site visit and test plan development.
- Sample collection and testing.
- Laboratory analysis and data review.

a. Preliminary Landfill Evaluation

A preliminary landfill evaluation is required in order to plan the test. The preliminary landfill evaluation should include information derived from:

- o Site Walk
- o Discussions With Landfill Operations and Owners
- o Topographic Maps
- o Other Documents

During the site walk, the area geology, landfill topography, and cover type and extent should be noted, particularly as they could effect moisture levels and decay rate within the landfill. Leachate weeps should be noted, as they are evidence that moisture has infiltrated the landfill. Site personnel should be questioned as to the dumping of liquids, and the methods used for their disposal.

Although many actively generating landfills show little evidence of gas, dead vegetation, odors, and gas bubbling through ponded water are all good indicators of active generation.

Discussions with landfill operators and owners are necessary to determine the quantities and types of wastes that enter the site, the site construction, and the dates that the various portions of the landfill were filled.

Topographic maps should be analyzed to determine in-place volumes of refuse as well as the potential for water infiltration into the site.

Other documents that may prove useful are the original soils report of the site, records of gate receipts, and monitoring data from existing vents and wells on the site.

b. Procedures

Based on the preliminary evaluation, a test can be designed to determine the state of decay of the refuse, and the potential for further decay. Many test methods are in use in the landfill gas recovery and monitoring industry ranging from superficial monitoring with methane detectors to well tests with active withdrawl. The following techniques, however, utilizing monitoring probes should give the most usable results at a reasonable cost. A monitoring probe is a partially perforated piece of pipe which is inserted in a hole in a landfill (Figure 53). The probe is used to measure landfill gases and pressures.

- o A sufficient number of probe locations should be chosen to adequately categorize the site.

- o Probe holes should be drilled with a 6 in (15 cm) auger. The condition of the refuse should be logged, along with any dates that are discernable. Although it is desirable to drill to the bottom of the refuse, this may not be possible due to economic constraints.

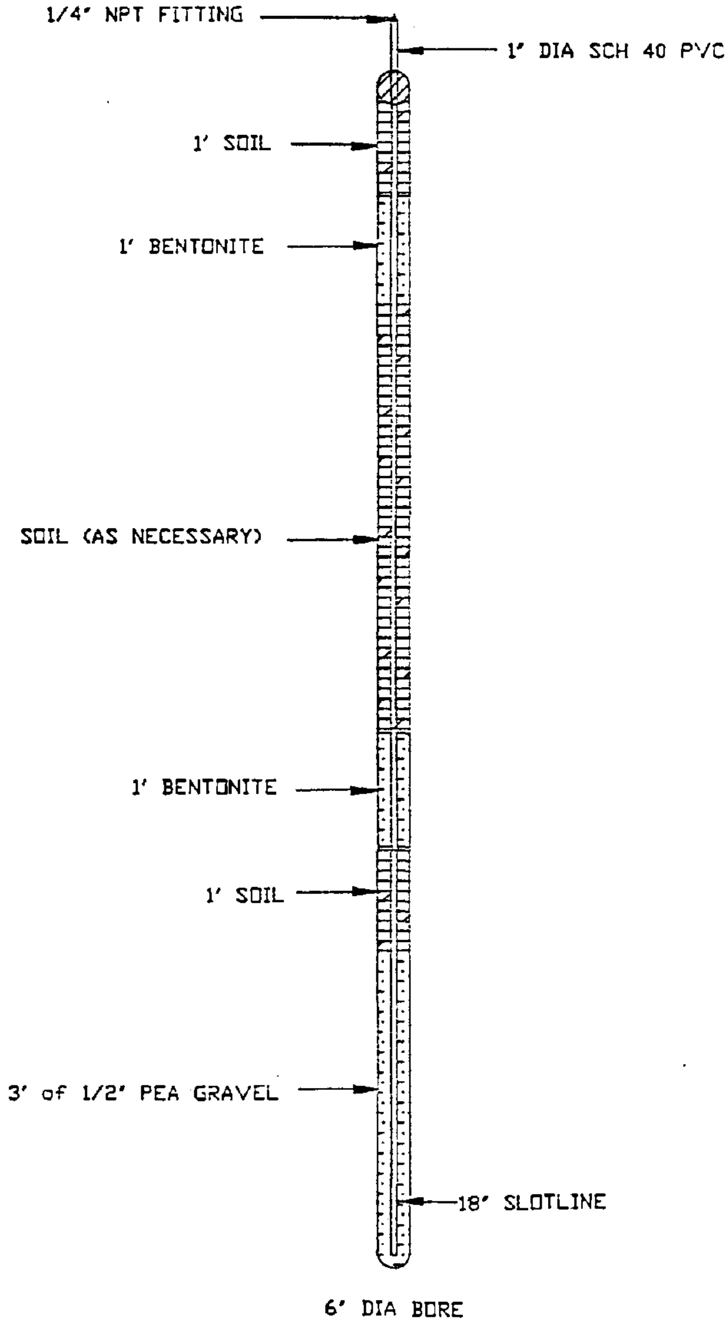


Figure 53 : Diagram of probe installation

- For a more precise analysis, representative refuse samples of at least 1 pound weight (0.5 kg) should be obtained, and stored in moisture roof plastic bags. The in-situ temperature and pH of the refuse should also be noted.

- Probes should be installed as indicated in Figure 53.

c. Monitoring

Probes should be monitored for pressure and gas composition. A magnehelix or water manometer with a range of 0 to 13 in (32 cm) of water can be used to measure pressure. Detailed gas analysis requires the use of a gas chromatograph. However, a flame ionization detector can be used to measure CH_4 and CO_2 in the field. A properly installed probe in an actively generating landfill will emit gas with a composition of 53 to 60 percent CH_4, 38 to 46 percent CO_2, 0 to 2 percent H_2, and no 0_2 or N_2. Lower CH_4/CO_2 ratios indicate that a landfill either has not yet reached optimum generation levels (Figure 54), or is in a die off phase. Appreciable quantities of 0_2 or N_2 indicate either low generation or a poorly installed probe.

The methane present in the landfill is a function of both the landfill permeability and the generation rate. Even in landfills with low gas generation, landfill gas can lodge in soil or refuse pore spaces. For this reason, it is necessary to measure pressure as well as

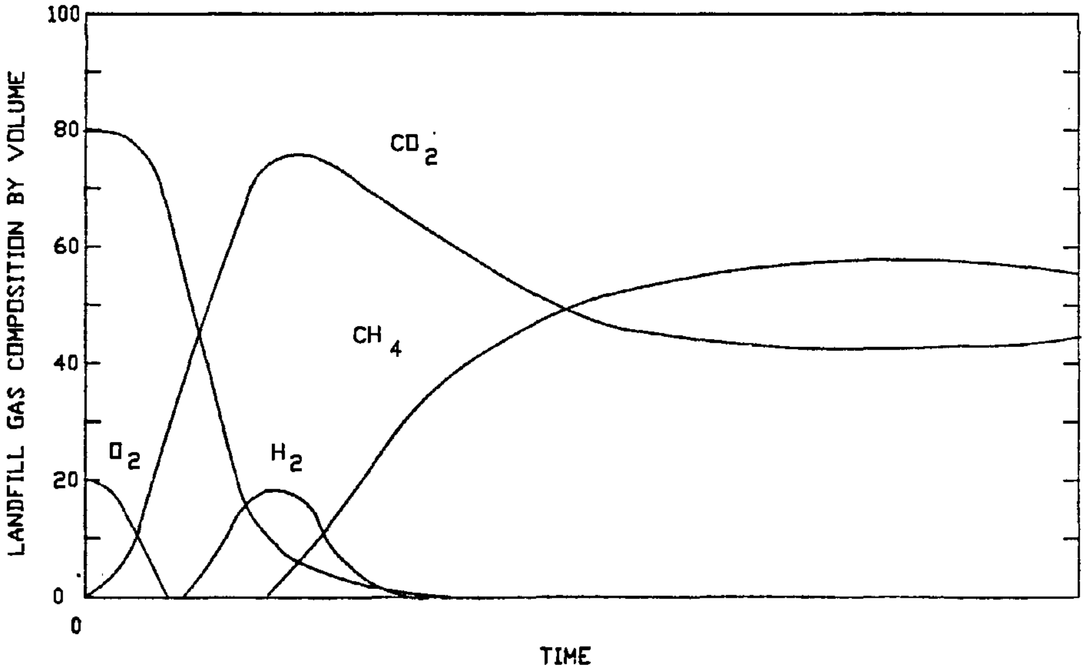


Figure 54 : Landfill gas composition transition (84)

generation rate. Gas pressure is a function of both gas flow and substrate permeability. Without measuring permeability, it is not possible to calculate generation rate. However, as a general case, landfills with probe pressures of less than 0.1 inches of water are low gas generators.

As landfill pressures fluctuate over the course of the day, with readings usually being highest mid-day, a diurnal monitoring program is recommended.

d. Data Analysis

Refuse can be visually inspected for degree of decomposition. Highly decomposed refuse has a dark, soil-like appearance. Other than certain non-decomposable fractions, (e.g., treated wood, glass, cinders, metal fragments, and rubber products), individual objects cannot be discerned. A more precise way is to analyze the refuse for percent volatiles.

Incoming refuse typically has about a 60 to 75 percent volatile content (Table 13). By measuring the volatile content of the refuse, it is possible to determine how much raw material is left to be decayed.

TABLE 13

Percent Volatiles in Solid Waste

Source	*Percent Volatiles	Percent Moisture
Golucke and McGauhey[34]	75	20
Woodyard, et.al. [101]	60 to 71	22 to 27
Dynatech R/D[26]	69	28
Bell[6]	67	21

*Dry weight basis includes combustible plastics.

The moisture content of the refuse can also be determined in the laboratory. Incoming refuse typically has a moisture content of 20 to 28 percent (Table 13). In-place refuse that has less than 20 percent moisture is considered dry, while refuse with more than 40 percent moisture is near saturated. The moisture content of the refuse is indicative of the decay rate of the landfill. Refuse that is dry can have a half-life (the time it takes half refuse to decay) of over 25 years, while wet refuse may have a half-life of less than 2 years. Since refuse decay may follow first order kinetics, dry landfills will be decomposing at low rates for possibly up to 100 years after closing.

Temperature is an indicator of the gas generation rate, although it has not to date been precisely correlated. In general, the gas from low generating refuse will be less than 80° F (27° C), from actively generating refuse and from 80 to 100° F (27 to 38° C), and from extremely high generating refuse over 105° F (41° C). Since refuse is a good insulator of heat, lower layers of refuse may remain hot even after being mostly decayed. Because of the complexity of the various factors effecting gas generation rates, it is recommended that computer based modeling techniques be used for making long term projections of decay rates.

4. Prediction Models

Several models are available which describe settlement rates in refuse and shreaded refuse, some which may be adaptable to dynamically compacted refuse. Due to the complexity of these models, they will only be discussed briefly here; the individual references should be obtained for further details.

Zimmerman, et.al.[104] developed a mathematical model that considered two separate time related settlement phenomena: pore pressure and creep behavior. The time-settlement behavior of solid waste materials was modelled using two simultaneous equations, one of which is nonlinear. The time-pore pressure dissipation relationship is represented by a general equation of continuity based on the theory of mixtures. This portion includes the effects of finite strain, biological and chemical activities, and the time variation of saturation. Creep behavior is modelled using a rate process function, the parameters of which vary with displacement to account for large strains.

Rao, et.al.[80] related refuse settlement to initial density and pressure as applied by loading for both flyash grouted and ungrouted refuse. Both field and laboratory studies were performed. According to the laboratory studies, stress history, initial density, load increment ratio, and the degree of pressure effect settling rates. In the field, settlement was also effected by temperature and rainfall, which impact biological decomposition.

Chen and Zimmerman[17] studied time settlement characteritsics of milled refuse and found that creep deformation and creep strain rate increased with the load increment ratio and the extent of biochemical decomposition. The experimental results were based on raw and aged milled refuse with a previously developed theoretical model.

Chen, Franklin and Quon[16] refined some of the earlier work done on the settlement of milled refuse. Modifications, by various investigators, to the theory of one dimensional consolidation developed in the 1920's by Terazaghi are discussed. Accoring to Terazaghi, settlement of soils occurs in two stages, a primary stage where pore pressure is dissipated and a secondary stage where continous deformation occurs. Phase one is controlled by the rate of hydraulic flow, while phase two is creep under effective stress. Chen, Franklin, and Quon believe that the general equation of continuity based on the theory of mixture and the creep equation based on the micropore structure of cellulose give a good and complete description of the consolidation process of milled refuse. A computer model, including code, which models the consolidation process is contained in the document.

APPENDIX C - INSTRUMENTATION OF DYNAMIC COMPACTION

1. Introduction

Normally, the effectiveness of dynamic compaction is measured by conventional soil tests undertaken in borings that are performed before and after dynamic compaction. These soil tests include standard penetration resistance tests, cone penetrometer tests, and pressuremeter tests. A comparison of the test results after dynamic compaction can be made with the test results obtained before dynamic compaction and by this comparison, the depth and degree of improvement can be established.

Alternate methods of measuring improvement have been tried on a few occasions. One method involves the placement of accelerometers which are attached to the tamper with leads connected to the measuring and recording instruments. When the tamper strikes the ground, it subjects the soil strata to high pressure for a short time duration. From measurements of forces in motions, an average soil strength and compressibility value can then be predicted for the strata which has been stressed by the tamper impact.

The response of the soil to the impacting load is dependent on the magnitude of the force and how fast that force is being applied. This principle has long been recognized and indeed has been applied to measure the dynamic behavior of soils during pile driving, for example. Thus, if it were possible to measure the forces and motions occurring in the soil during the compaction process, then an average soil strength and compressibility value could be obtained. These results would be representative for the soil's properties within the zone affected by the impacting weight and at the time of testing.

As it applies to dynamic soil compaction, the idea of taking and evaluating force and motion measurements has already been exploited by Andreasson and Hansbo[1], Calomino and Rausche[11], Mayne[58], and Jessberger and Beine.[44] Recent test results will be presented in the following discussion after an introduction to the requirements and the potential of measurements conducted during dynamic compaction.

2. Basic Relationships and Assumptions

There are two basic principles which may be used to compute the bearing capacity of the soil during the penetration of the tamper into the ground. The first one assumes that the energy contained in the tamper immediately before impact equals the work done on the soil. The second principle equates the force exerted by the soil against the tamper and its sides to the ultimate soil bearing capacity.

a. Energy Approach

The tamper is assumed to be rigid (as compared to the soil) with a mass, m. If the tamper falls freely from a height, H, then its velocity at the time it touches the soil is

$$v_i = \sqrt{2gH} \tag{6}$$

where g is the earth gravitational acceleration.

For applications where the tamper does not fall freely, i.e. when it has to unwind the hoist cable and thereby rotate the drum in the crane, the impact velocity is, of course, reduced. This is usually accounted for by introducing an efficiency factor, e, into Equation 6 which then becomes:

$$v_i = \sqrt{2gHe} \tag{7}$$

The impact velocity of the falling weight is important because it allows for a direct computation of its kinetic energy, E_r, at the time when it contacts the soil.

$$E_r = 1/2\ m\ v_i^2 \qquad (8)$$

Suppose that the soil responds to the impacting energy with a resistance force, R, then the work done by the soil on the impacting mass may be computed from

$$W = Rs \qquad (9)$$

where s represents the distance that the mass penetrated into the soil, i.e. the crater depth. This is really only true if the soil resistance does not vary from the time the mass starts to compress the soil until it reaches the deepest point of penetration. At that point, the mass velocity is zero and all energy has been delivered to the soil.

If the mass compresses the soil sufficiently, then it may be assumed that R is equal to the ultimate capacity of the soil, R_u. Substituting R_u for R and E_r for W in Equation 9 and solving for R_u, one obtains

$$R_u = E_r/s \qquad (10)$$

which is the desired result. Of course, the assumption of constant resistance limits the applicability of the above relationship. Comparisons with conventional testing methods can, however, be used to develop correlation or adjustment factors to be applied to Equation 10 for a given site. The basic required dynamic measurements are E_r or v_i and s. Since there is usually little rebound in dynamic soil compaction, s may be set equal to the easily measured final crater depth.

It should be mentioned that for both Energy and Force Methods, the resulting R_u value does not contain the weight of the tamper itself. In some instances, it may be necessary to add this weight for better accuracy.

b. Force Approach

After impact, and during penetration into the soil, the motion of the mass is slowed down and eventually comes to rest. The rate of change of velocity is the deceleration, a, of the mass. The decelerating mass exerts an inertia force, F, onto the soil.

$$F = ma \tag{11}$$

This inertia force must be equal to the soil resistance, R. Therefore, the soil resistance can be computed from

$$R = F = ma \tag{12}$$

if the deceleration is known. R varies in the same way in which a varies. Conceivably, the deceleration is small at the beginning of impact and increases to large values (a) while the velocity of the ram is still high, (b) while a large volume of soil is displaced, and (c) after the compressed soil assumes a high stiffness. When the maximum penetration is reached, the mass has attained a state of zero velocity which is similar to static applications. Thus, it may be postulated that

$$R_u = ma, \text{ when the velocity of the mass is zero} \tag{13}$$

Before the mass velocity has become zero, the soil resistance will have a static and a dynamic component

$$R = R_u + R_d \tag{14}$$

However, in general, the dynamic component, R_d, is of no interest except for the fact that it does absorb energy. It therefore does not need to be calculated.

For the proposed force approach, the deceleration and the time of zero velocity must be known. If the deceleration is measured, then the velocity can be computed from it by integration. However, this integration of deceleration to velocity is not necessarily a simple task. In general, it is necessary to divide the deceleration record into little time steps. Then, starting with a known or assumed impact velocity, v_i, deceleration values multiplied by the time steps are subtracted. At the end of the event, i.e. when the deceleration has become zero, the velocity must be equal to zero. If this is not the case, an adjustment may need to be made to either the assumed impact velocity or the deceleration zero line, which is usually not well known.

Furthermore, the integration process can be repeated with the computed velocity. Then the final penetration can also be checked. This value must equal the crater depth. Again, adjustments may need to be made if there is a disagreement.

Basically, however, the deceleration is all that is needed to produce a mass velocity and a mass penetration vs time curve. In order to compute the R_u value, only the deceleration at the time when the computed velocity becomes zero needs to be determined and multiplied with the mass of the falling weight.

An additional result is a soil resistance vs mass penetration plot. This plot may be obtained by simply choosing resistance and penetration values at the same time. An example for this procedure is given in a later section.

In summary, there are two dynamic measurements which are desirable for the evaluation of drop weight performance and soil response. The first quantity to be measured is the impact velocity and the second one, the deceleration, of the impacting weight.

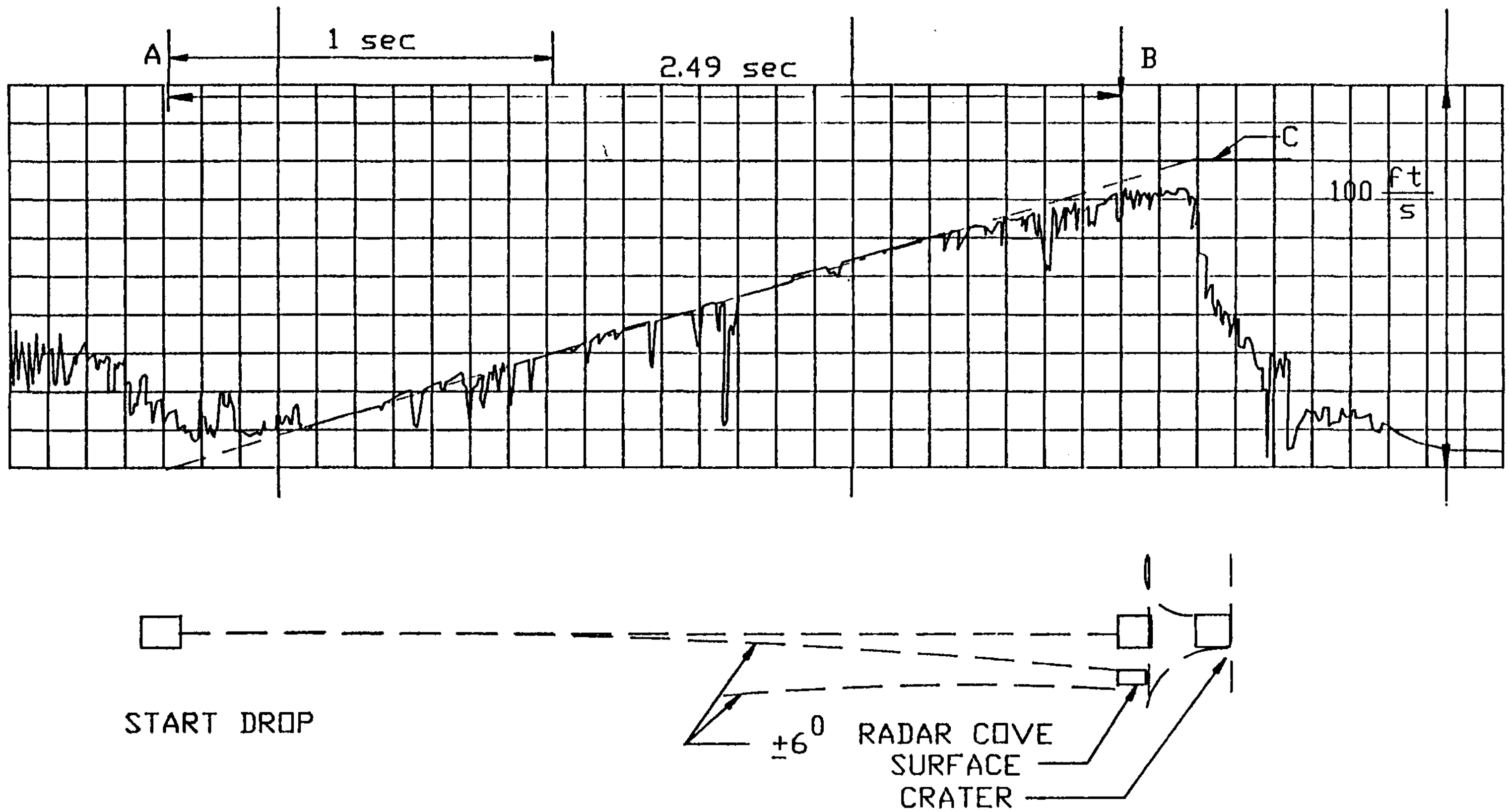
3. Details of Measurement Apparatus

a. Impact Velocity

There are numerous means for finding impact velocity. First, a highly sensitive accelerometer may be placed on the falling mass. It was demonstrated by Rausche et al.[81] that such an acceleration reading may be used to determine the mass impact velocity, v_i. The disadvantage of this method is the need for a very precise zero acceleration adjustment, since only small shifts of this signal integrated over the time of the mass fall will lead to large errors.

Another method utilizes two light beams which are interrupted by the falling mass shortly before contact with the ground surface. This is a very simple method which may yield satisfactory results, depending on the accuracy of the electronic timing device.

In recent years, radar has been successfully used for the measurement of the speed of pile driving rams. The mass of a dynamic compacting device displays significantly higher velocities which requires only a very simple modification of that device. The radar results are usually displayed in terms of a continuous velocity trace on a strip chart. Such a trace shown for the example of a freely dropping mass from a height of 100 ft (30 m) is shown in Figure 55. For conditions including a loss due to cable and drum friction, the slope of the velocity time curve would clearly indicate the actual acceleration and velocity of the falling weight.



A: Theoretical Time of Mass Release
B: Theoretical Time of Mass Impact for 100 ft. Drop
C: Theoretical Impact Velocity

Figure 55 : Example of a radar measured mass velocity vs. time strip chart

Although the results from radar are rather accurate and very instructive, the mounting of the radar antenna usually poses problems. Attempts to mount it on the tip of the crane boom often fail because of the motion of the crane boom (the radar antenna needs a fixed reference position for accurate readings) and the difficulty of directing the antenna onto the mass. (The antenna usually has a $\pm$ 6° range). Also, for great drop heights, the intensity of the radar reflection may be insufficient, particularly if the top of the mass is covered with soil.

The record of Figure 55 was obtained with the radar antenna pointing upwards to the underside of the mass. Of course, it was necessary to move it away from the immediate target area and for that reason, the last few feet of mass travel before impact were not recorded. Extrapolation shows, however, that the impact velocity was 78 ft/s (24 m/s). The mass had a weight of 64 kips (285 kN) and, therefore, the energy of the mass immediately preceding impact was ($g = 32.2$ ft/s^2):

$$E_r = 0.5(78)^2(64/32.2) = 6046 \text{ kip-ft } (8250\text{kN-m}) \qquad (15)$$

Since the drop height was 100 ft (30 m), the theoretically available energy was 100 ft times 64 kips = 6400 kip-ft (8750 kN-m) and the efficiency of the drop was therefore 94%. The losses could be caused by an inaccurate height, h, and air resistance.

Three drops were applied to the same crater. The successive crater depths readings were 42, 59 and 64 in (1.1, 1.5, and 1.6 m). The corresponding penetrations were therefore 3.5, 1.42 and .42 ft (1.1, 0.4 and 0.13 m). For the first drop, the average soil resistance was therefore 6046/3.5 = 1727 kips (7750 kN).

The cross sectional area of the tamper was 5 X 7 = 35 ft^2 (3.3 m^2). Thus, an average ultimate pressure of 49 ksf (2.35 mPa) was measured.

Since dynamic resistance components were probably high, this limit pressure should be used with a large safety factor, e.g. for an allowable pressure of about 4.9 ksf (235 kN/m^2).

For the second and third drop, the measured average pressures were 122 and 411 ksf (5.9 mPa and 18.4 mPa). Since pore water pressure may have significantly changed during the first and second blows, the later results are probably not representative of the soil's long-term bearing capacity.

b. Deceleration

Radar measurements can only give partial velocity records. For example, low velocities and the time of zero velocities are not resolved. Deceleration measurements may be made by employing a variety of accelerometers. The maximum deceleration level depends on the stiffness of the compacted soil strata. However, it is not expected to exceed 100 g's. Piezoelectric accelerometers are relatively inexpensive, rugged and easy to use. They measure the electric charge in a quartz crystal which is subjected to the inertia forces of a small mass. However, piezoelectric accelerometers are dynamic instruments, which means that they leak-off or lose the constant portions of the acceleration signal.

If it is intended to determine the impact velocity by integration over relatively long periods of time (several seconds), then the piezoresistive accelerometer type may be better suited since it can maintain a DC signal. Piezoresistive units are basically small, stiff cantilever beams with a mass attached to their free end and strain gages picking up the bending force imposed in the beam by a change of velocity of the cantilever mass.

Both types of accelerometers are satisfactory as far as the dynamic components of the records are concerned, i.e. during impact. The maximum frequency of the mass deceleration may, however, reach 1000 Hz or more, again depending on the stiffness of the soil.

The accelerometer should be mounted near the center of gravity of the mass or more than one accelerometer has to be used for the cancellation of the effects of a rocking motion and a better representation of the motion of the object. In general, the signals will be transmitted through cable. However, telemetric transmission is also possible. In tests conducted thus far, cable attachments have not been a problem.

Measurements of deceleration should always be recorded for later reanalysis or for record keeping purposes. Either digital disks or analog tapes are satisfactory recording media. It is recommended to perform all calculations automatically in digital form. For this reason, the digital disk would be the preferable medium for data storage.

Figure 56 is an example of the deceleration record of a mass with 6 ton (54 kN) weight and 4 ft (1.2 m) diameter. It had been dropped from a height of 40 ft (12.2 m) onto debris of a demolished building. The crater depth after the drop was approximately 5-1/2 inches (140 mm). A maximum deceleration of 37 g's was recorded with piezoelectric accelerometers. The deceleration record was shifted by a small amount such that after integration the

- o final set equaled the crater depth and the
- o final velocity equaled zero.

These calculations produced an impact velocity of 23.3 ft/s (7.3 m/s), which corresponds to 47% of the theoretical impact velocity. Using Equation 8, the energy of the mass was calculated yielding 21% of the potential ram energy. While relatively high energy losses must be expected for a fall restricted by a cable and winch, the 21% number seems to be unusually low. On the other hand, the maximum deceleration and therefore the maximum resistance force will be little affected by the accuracy of the computed mass impact velocity, since it is only slightly affected by the shift of the deceleration zero level.

Figure 56 shows the resistance force (proportional to deceleration), velocity and displacement traces, all as a function of time. Figure 57 shows the relation between resistance and mass displacement into the soil. R_u as defined in Equation 13 is shown at maximum displacement. Note the large difference between maximum and static resistance value. Also, the unloading slope is indicated, which relates the soil resistance during rebound to the mass upward motion. This rebound behavior yields the best estimate of static load deflection behavior of the 4 ft (1.2 m) diameter load area.

4. Conclusions and Recommendations

It is believed that the deceleration measurements hold great promise for an instantaneous evaluation of the compaction effects. Some work needs to be done, however, if the method is to be generally useful.

- The proper accelerometers have to be selected.

- The impact velocity of the ram should be independently measured and used for a check on the deceleration results. This impact velocity may be either measured by light beam or radar technology.

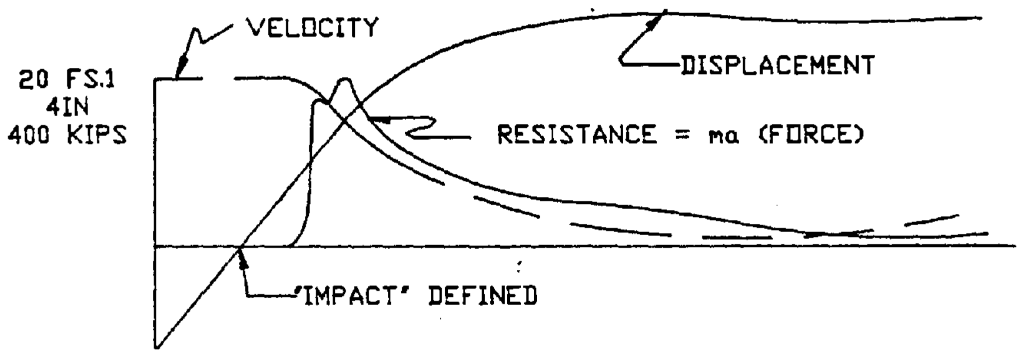


Figure 56 : Top force (soil resistance from measured deceleration times mass), velocity and displacement of mass as a function of time

(1in/sec = 2.54cm/sec)

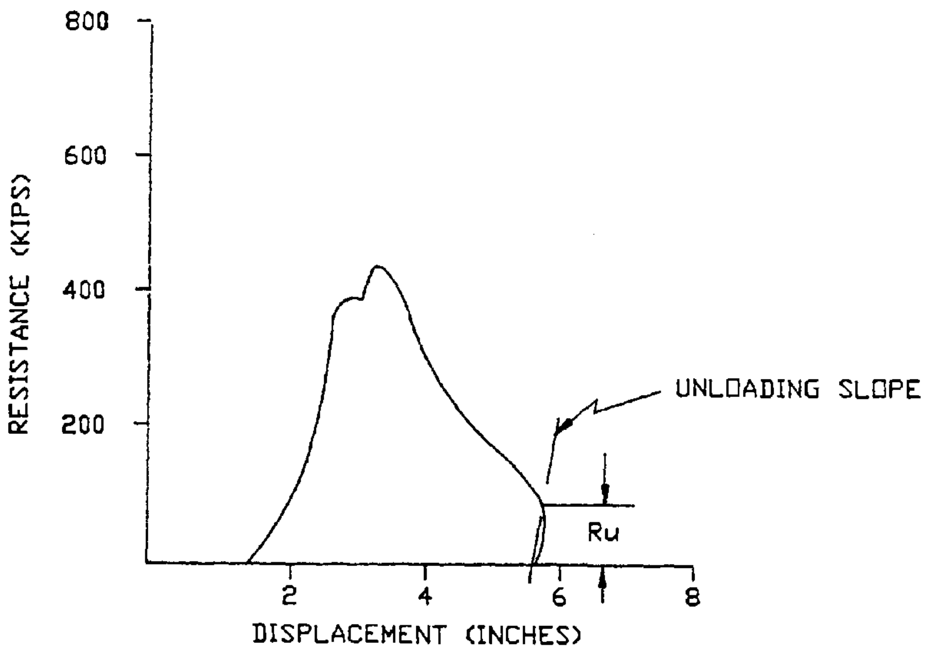


Figure 57 : Resistance versus mass displacement

(1in/sec = 2.54cm/sec)

- The information from all sensors has to be immediately conditioned and processed using a microprocessor based machine. Output should include plots for an immediate interpretation by an experienced engineer.

- Correlations with other in-situ measurement methods of soil strength and stiffness should be collected for an improved precision of prediction.

The purpose of performing this test and interpretation of the calculations is discussed in Chapter 4.

Two volumetric quantities are calculated from measurements taken during this test.

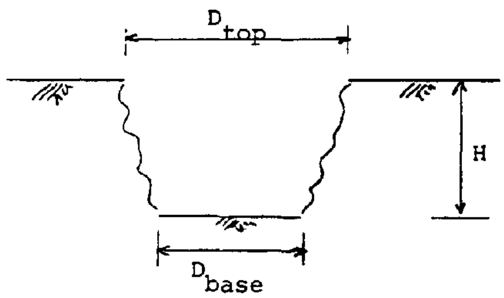
A. Volume of the crater.

B. Volume of ground heave (if any) adjacent to the crater.

Measurements are generally taken following two drop intervals of the tamper at the same point.

A. Volume of the Crater

Elevation readings should be taken at grade before the first drop and at two drop intervals. For a circular weight, the diameter of the base of the crater will be the same as the diameter of the tamper. The top diameter may be larger due to caving of the soil and should be measured with a tape.



D_{base} = Diameter of base of crater for a circular weight which is the same diameter as the tamper.

D_{top} = Diameter of top of crater which is obtained by tape measurement.

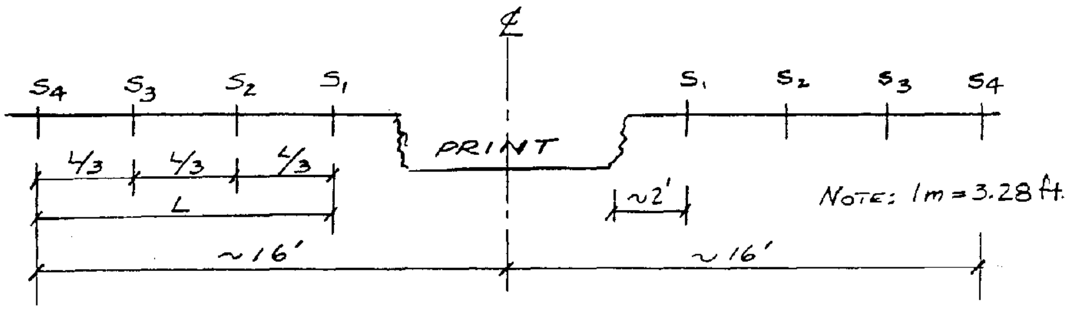
$$\text{Volume of Circular Crater} = \left[\frac{D_{base} + D_{top}}{2}\right]^2 (0.785)\ (H) \qquad (16)$$

If the tamper has a different cross sectional shape, use an appropriate volumetric calculation formula.

B. Volume of Ground Heave

The suggested procedures for measuring the ground heave (if any) adjacent to the crater is as follows:

1. Within each quadrant, set four displacement stakes or long nails to a depth of about 1 ft below grade around the print. The closest stake should be set about 2 ft from the edge of the print and the furthest about 16 ft from the centerline of the print. Space the other two stakes at equally spaced intermediate points. (See Diagram)



S_1 = Stake closest to print

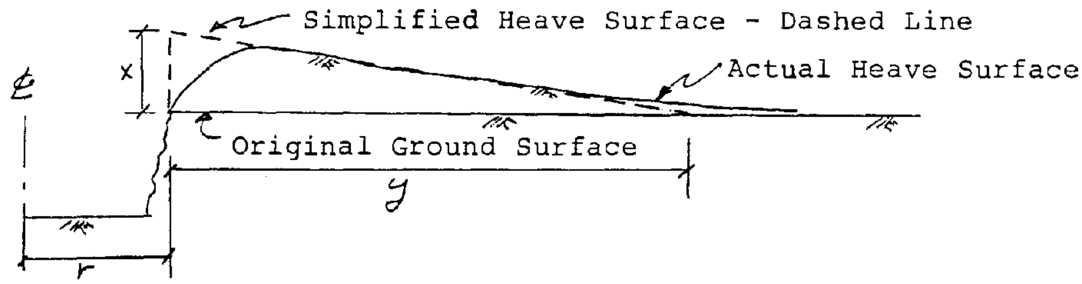
S_4 = Stake farthest from print

2. Determine elevations of all sixteen stakes (nails) before first drop and at two drop intervals.

3. Determine average heave at locations S_1 to S_4 by averaging corresponding readings from each quadrant:

$$S_1 \text{ average} = \frac{S_1 \text{ quadrant}_1 + S_1 \text{ quadrant}_2 + S_1 \text{ quadrant}_3 + S_1 \text{quadrant}_4}{4}$$

4. The heave is generally greatest near the edge of the print and diminishes to small amounts at greater distances from the print. The heave calculations can be simplified by assuming a linear heave pattern as shown on the diagram.



5. Calculate the volume of heave from the following simplified expression:

$$\text{Volume of Heave} = xy\pi\,[\,y/3 + r\,] \qquad (17)$$

6. Determine the net effective volume at any drop point as the crater volume minus the heave volume.

GLOSSARY

Within the text, some terms have a special meaning when applied to dynamic compaction so these terms are described below. In addition, soil deposits are categorized within this manual into classifications such as impervious, semi-pervious or pervious, and these deposits are defined in the context in which they are used within this manual.

APPLIED ENERGY The amount of energy per unit area imparted into the ground from the dynamic compaction operations. It is computed on the basis of the total amount of energy applied divided by the area densified. It is expressed in tonne meters per meter squared which is abbreviated txm/m^2. Example:
For a project where a 15 tonne weight is dropped 20 meters at a 10 meter spacing and 10 tamps are applied at each print, the average applied energy for this pass is:

$$\frac{15 \text{ tonnes X } 20 \text{ meters X } 10 \text{ tamps}}{10\text{m X } 10\text{m}} = 30 \text{ txm/m2}$$

If four passes are applied during the dynamic compaction operations, the applied energy would be 120 txm/m^2.

COARSE-GRAINED DEPOSITS This refers to granular soils usually in the sand to gravel range but it could include cobbles. The particle sizes are coarser than the 200 sieve.

CRATER The depression formed in the ground following single or multiple impacts.

FINE-GRAINED DEPOSITS This refers to soil deposits which are primarily in the silt or clay size range. The majority of the particles are finer than the No. 200 sieve.

IMPERVIOUS DEPOSITS This refers to soils which are in the clay or silty clay size range. More than 25% of the deposit consists of clay size particles and the plasticity index is greater than 8%. The coefficient of permeability is typically less than $1 \text{ X } 10^{-6}$ cm/sec.

IRONING PASS The application of energy to compact and smooth the surface of the densified deposit. Following deeper densification, the surface deposits are disturbed by crater formations and general disturbance. The ironing pass is applied last.

MOUSTACHE EFFECT This refers to the shape of the ground surface adjacent to the crater when upheaval takes place. The heave is greatest at a short distance from the edge of the crater and diminishes to zero some distance away. This effect generally occurs when the applied energy is no longer effective or only partially effective in producing densification, thereby resulting in the ground heave.

PASS The coverage of the entire area to be densified with a portion of the planned energy. Multiple passes may be required to apply the total amount of energy. The term phase is also used to designate the same meaning.

PERVIOUS DEPOSIT This refers to granular soil deposits which have a high permeability. The coefficient of permeability is greater than 1×10^{-3} cm/sec. The gradation range would extend from boulder sizes to sandy deposits with not more than 30% passing the No. 200 sieve. These deposits are non-plastic.

PRINT The location where the weight is dropped. This word is used interchangeably with crater location.

TAMPER The weight used in dynamic compaction. The word pounder is often substituted for tamper.

TAMPING The process of repeatedly raising and dropping the weight.

SEMI-PERVIOUS DEPOSIT This refers to deposits which are in the silt size range including sandy silts or clayey silts. The coefficient of permeability is typically in the range of 1×10^{-3} to 1×10^{-6} cm/sec. There should be no more than 25% of clay size particles which is defined as .005 mm. The plasticity indices of these deposits range from 0 to 8%.

TONNE This is a metric ton which is 1.1 times US tons.

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