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16. Abstract

Compaction and construction control of certain fills used for the structural cross-section of mechanically stabilized earth (MSE) retaining walls can present significant problems in design and quality assurance. Locally available soil types, in particular fine sands, are difficult to compact easily with standard equipment and often give unrealistically low densities in standard compaction tests. This results in inappropriate specifications that do not produce adequate material properties in the field. The resulting variability of material properties can result in a number of problems, including undesirable differential settlements.

A significant program of laboratory testing was undertaken to investigate these problems, including performing three different types of laboratory compaction methods on a total of 62 cohesionless sands. The standard Proctor test method gave low values of maximum dry density in almost all cases, while the modified Proctor test did so mainly for sands of coefficient of uniformity below about 3.5. The vibrating hammer test method gave significantly higher values of dry density over the complete range of tests, which it is believed agree more closely with field values that can be achieved with the fine sands in question. The grain size distribution and particle shape were also found to have an effect on the dry densities achieved. Researchers recommend that serious consideration be given to the adoption of vibratory test procedures for fine sands.

Revisions are suggested to the material specifications for MSE wall backfill, among other things, specifying the fill in terms of gradation, plasticity, soundness, and corrosion properties. A revised material testing manual has been proposed for the testing of backfill for MSE retaining walls. A new field inspector's manual for the construction and supervision of MSE retaining wall systems has also been prepared.

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SPECIFICATIONS FOR BACKFILL OF REINFORCED-EARTH RETAINING WALLS

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IMPLEMENTATION RECOMMENDATIONS

The use of reinforced-earth type retaining walls is sufficiently widespread that improved specifications and field control of backfilling operations are liable to affect almost every district in the TxDOT system.

(1) It is recommended that a field inspector's manual for mechanically stabilized earth (MSE) retaining wall systems be made available to districts across the state, to assist them not only in the details of desirable field inspection details, but also to assist less experienced inspectors in understanding the overall principles of MSE design. A suitable such manual has been produced as part of this project.

(2) It is advised that the material specifications for backfill for MSE wall systems be rigorously enforced. Some revisions to the current specifications are given, specifying the fill in terms of gradation, plasticity, soundness, and corrosion properties. It is also suggested that a distinction be made between compaction specifications for fill greater than 1 m (3 ft) from the face of the wall and for operations less than 1 m (3 ft) from the face of the wall, for which lighter (preferably hand-operated) equipment is desirable.

(3) It is recommended that serious consideration be given to the adoption of vibratory compaction standards for laboratory testing of soil designated for use as backfill, (although this should not exclude conventional impact-type testing). The results of this study show that such vibratory laboratory testing produces results that agree much more closely with the typical field behavior of certain soils, notably the fine sands that were the main subject of this study.

The recommendations developed on the project should provide a systematic method for the department to apply the research results in a manner suitable for inclusion of results in new wall construction. New retaining wall construction is ongoing across the state for TxDOT, and it is anticipated that implementation could be possible directly on conclusion of the study. TTI will be happy to coordinate with department personnel in ensuring that results and recommendations will be incorporated into TxDOT practice as expeditiously and effectively as possible.

1. INTRODUCTION

1.1 General

For as long as civil engineers have been constructing highways, bridges, embankments, buildings and retaining walls, they have been searching for ways to improve the soil beneath these structures to ensure their long term stability. One means of improving the soil is through compaction, which is the densification of the soil using mechanical means. While the principles of soil compaction have been known since Proctor's (1933) early work, there is evidence for cohesionless granular soils that the existing impact laboratory tests may produce results that are significantly low compared to what can be achieved during construction with existing compaction equipment (Felt, 1958; Parsons, 1992). In fact, this discrepancy has led to the development of new laboratory test procedures (Felt, 1958; Youd, 1973; Parsons, 1992) and investigations into the factors that affect the compaction of cohesionless soils (Burmister, 1948; Johnson and Sallberg, 1962; Tiedemann, 1973; Youd, 1973; Dickin, 1973; Poulos and Hed, 1973; Reitz, 1973; Semmelink and Visser, 1994).

To date, the results from these studies are often difficult to compare and interpret, because of two primary reasons. First, the majority of the new laboratory compaction methods have focused on determining the maximum possible dry unit weight of the soil (i.e., vibrating table compaction test, modified vibrating table compaction test, and variations of these), which has not found widespread use among practicing engineers (see discussion in Poulos, 1988). Secondly, no standardized methods have been put forth or used to investigate the various factors influencing the compaction characteristics of cohesionless granular soils, which has made it difficult to compare results from various authors. In the literature, there is still a debate over which laboratory compaction tests should be used and which factors are important in the compaction of cohesionless granular soils (see discussions by Poulos, 1988; 1989; and Bowles, 1989). Consequently, there is a need for an investigation of the various factors influencing the compaction characteristics of cohesionless granular soils, which utilizes standard laboratory methods and provides useful results to practicing engineers.

1.2 Objectives

The objectives of this project were to examine and determine the various factors which affect the compaction of granular soils, specifically, the compaction characteristics of fine cohesionless sands, as this is the type of soil commonly used in Texas for mechanically stabilized earth (MSE) retaining wall systems, but for which standard compaction procedures have been found to be inadequate.

Specific objectives included:

(a) determining the current problems with MSE retaining wall systems, and specifically to determine what effect the backfill type and the compaction of the backfill may have on the stability of MSE retaining wall systems.

(b) investigating the current Texas Department of Transportation (TxDOT) procedures and specifications for use of structural backfill soils to determine if they are sufficient or if recommendations for changes are justified.

(c) investigating the importance of the water content, grain size distribution of the particles, grading of the soil, grain shape of the particles, and grain crushing during testing on the effectiveness of laboratory compaction.

(d) determining the effect of three different laboratory compaction procedures (i.e., Standard Proctor, Modified Proctor, and the Vibrating Hammer tests) on the compaction of cohesionless sands.

(e) correlating the various factors with the different compaction tests.

(f) comparing the results of this study with results in the literature and other sands to determine the applicability of the results obtained.

(g) estimating the maximum possible settlement of a compacted soil from the results of various compaction tests.

(h) revising TxDOT specifications for backfill and the associated field compaction specifications.

1.3 Outline of the Research

This study is reported in a number of sections. After extensive documentation in Section 2 of the problems that have been encountered with MSE retaining wall systems, previous work on the compaction of cohesionless sands is reviewed in Section 3. The effect of different laboratory test methods are then investigated in Section 4. Section 5 then presents the results of specific compaction tests carried out for this project on 62 separate cohesionless sands using three different compaction techniques. A detailed analysis of these test results is presented in Section 6, and this is followed by the conclusions and recommendations of this study.

Appendix A contains recommendations for revised specifications for backfill material for MSE walls, followed in Appendix B by revised material test procedures for such backfill. A new field inspector's manual for construction and supervision of MSE retaining wall systems is contained in Appendix C.

2. PROBLEMS ENCOUNTERED WITH MSE RETAINING WALL SYSTEMS

2.1 General

During the last 20 years MSE retaining wall systems have gained widespread popularity because of their flexibility, ease of installation, and economic advantages. Much of the economy of these walls is derived from their ability to utilize readily available, low cost backfill. While these walls have, in general, performed satisfactorily, various problems have been identified which affect the constructability and long term performance of these walls.

2.2 Problems with MSE Retaining Walls

After surveying a number of districts in the state of Texas, a list of the various problems, their effects on the retaining walls, and the possible causes of the problems was compiled. Table 1 is a brief representation of the survey. From Table 1, the most prominent problems arise from the backfill properties and the compaction of the backfill. Each of these has a direct influence on the construction and long term performance of the MSE retaining walls and will be individually investigated below. (For a more in-depth outline of the problems and methods to correct the problems, see Appendix C, Mechanically Stabilized Earth Retaining Wall System Field Inspector's Manual.)

2.3 Backfill Properties

The problems encountered with the backfill properties stem from three different sources: a) backfill specifications, b) laboratory tests on the backfill, and c) quality control of the backfill properties during construction.

<u>Backfill Specifications</u>

The backfill specifications for TxDOT were compared with various states to determine if there were any similarities or differences, most notably with the specifications from the Federal Highway Administartion (FHWA), the state of Missouri, and the state of California. The TxDOT specifications are in accord with the FHWA's and California's specifications for backfill, which are broad specifications. However, Missouri's specifications for backfill are more restricting by limiting the percent passing six different sieve sizes rather than only three sieve sizes. This essentially means that the Missouri specifications for backfill place bounds on the overall gradation of the backfill.

TABLE 1. Problems Encountered with MSE Retaining Walls.

PROBLEM	EFFECT ON RETAINING WALL	POSSIBLE CAUSE OF THE PROBLEM
I. COMPACTION	A. Wall Leaning Out	Compaction of the backfill within 3 feet of the wall
		Overcompaction or excessive compactive effort
		Excessive vibratory compaction of uniform fine sands
		Backfill material placed wet of optimum water content
	B. Wall Leaning In	Inadequate compaction of backfill
		Collapse of voids in the backfill
	C. Differential Settlement of Wall	Backfill material not uniformly compacted
		Collapse of voids in the backfill
	D. Damage to the Reinforcing Strips	Excessive compactive effort used on the backfill
		Lift thickness not thick enough above the reinforcing strips
II. BACKFILL PROPERTIES	A. Wall Leaning Out	Backfill material contains excessive fines
		Backfill material saturated by heavy rain
		Backfill material is not uniform
	B. Wall Leaning In	Backfill material is not uniform
	C. Differential Settlement of Wall	Poor quality of backfill material
		Backfill material not uniform
		Free draining backfill allows subsoil to undergo consolidation

TABLE 1. Problems Encountered With MSE Retaining Walls (continued).

PROBLEM	EFFECT ON RETAINING WALL	POSSIBLE CAUSE OF THE PROBLEM
III. WATER OR DRAINAGE	A. Wall Leaning Out	Excessive pore pressure in the backfill acting on the wall
		Consolidation settlement of the backfill upon saturation
		Improper grading of the backfill
		Leaving the backfill exposed for long periods of time prior to completion of the wall
	B. Wall Leaning In	Washout of the backfill creating voids behind the wall
		Voids left by consolidation settlement of the backfill upon saturation
		Improper grading of the backfill
	C. Differential Settlement of Wall	Local washout zones in the backfill creating voids behind the wall
		Local consolidation settlement of the backfill upon saturation creating voids
		Improper grading of the backfill
		Leaving the backfill exposed for long periods of time prior to completion of the wall
	D. Backfill material is not uniform	Intermixing of the clay from the embankment with the sand backfill
		Leaving the backfill and the clay embankment exposed for long periods of time prior to completion of the wall

TABLE 1. Problems Encountered with MSE Retaining Walls (continued).

PROBLEM	EFFECT ON RETAINING WALL	POSSIBLE CAUSE OF THE PROBLEM
IV. FACING CONSTRUCTION PROBLEMS	A. Gaps between the panels (could lead to washout problems and voids)	Improper propping of the panels
		Improper spacing between the panels
	B. Torn or Damaged Filter Fabric (could lead to washout zones)	Improper installation of the filter fabric
		Improper propping of the panels
		Improper backfilling procedures
	C. Distortion of the Wall	Improper propping of the panels
		Damage to the reinforcing strips during compaction
		Improper connecting of the reinforcing strips to the panels
	D. Cracked or Chipped Panels (could lead to washout zones)	Improper handling of the panels
		Improper spacing between the panels

The present TxDOT specifications for backfill for MSE retaining walls are based on a limited number of sieves to determine the gradation of the soil, but this allows for a very wide range of soil types. In general this is favorable because it allows the walls to be built at a low cost, but it has also caused problems for the walls. For example, the most common type backfill causing problems for MSE walls in Texas is fine, uniform sands, also known as "sugar" or "blow" sands. This material is encountered both along the coast and inland, and is the most inexpensive and widely available backfill meeting the specifications for backfill for MSE walls. However, fine, uniform sands are difficult to compact and are subject to settlement (both during construction and post construction), which leads to serious problems with MSE walls.

Therefore, two possible means of restricting TxDOT specifications for backfill for MSE walls are to specify that the gradation be based on a wider number of sieves (like Missouri's for example) or to specify ranges for the coefficient of uniformity for the soil.

Laboratory Tests on the Backfill

Grain size analysis and pH and resistivity tests are performed on the backfill samples to determine if they meet the required specifications. The results of the grain size analysis, along with the pH and resistivity tests, are dependent upon the soil sampled being representative of the backfill used behind the walls.

Soil Sampling. Commonly, a single soil is taken from a stockpile designated by the contractor, up to a year in advance of the soil being used for backfill. The soil is then tested and determined as to whether or not it meets the required specifications. This practice often leads to problems because the soil tested only represents a minuscule fraction of the soil used for backfill. The soil could change gradation and properties, thereby changing its mechanical behavior. To avoid this problem, the engineer should require that the soil being tested is in fact the soil to be used for backfill, and numerous samples should be taken to determine the uniformity of the soil (see Appendix B).

Grain Size Analysis. The sampled soil is tested for grain size analysis to determine if it is suitable for use as a backfill material. These tests are fairly routine and reliable. However, at present not all districts record the entire grain size distribution; in fact, a common procedure is to record only the percent passing the number 40 and 200 sieves. This procedure leads to problems because the engineer cannot determine if the soil tested is the same as that used in the field, and more importantly, often changes in gradation lead to changes in mechanical behavior of the soil. To eliminate this problem, the entire grain size distribution of the soil should be recorded and kept for future reference.

Quality Control of the Backfill During Construction

In light of the possibility of changes in gradation of the backfill, which could lead to problems with the long term performance of the wall, the backfill should sampled and tested in the field to determine its properties. This could be done at various increments of wall height during construction. At present, this is rarely done by any of the districts. If this field testing were done, it could eliminate some of the problems with backfill soil - namely poor quality backfill and improper compaction specifications.

2.4 Compaction

The problems encountered with compaction of the backfill stem from three different sources: 1) laboratory tests on the backfill, 2) compaction specifications, and 3) quality control of the compaction during construction.

Laboratory Tests on the Backfill

Laboratory compaction tests on the soil designated for use as backfill are performed because they are incorporated into the specifications for field compaction of the backfill. Therefore, it is important that the laboratory tests for compaction be performed correctly and on various samples of soil to determine a relationship between gradational changes and compaction characteristics.

The TxDOT laboratory compaction test for granular soils is Test 113-E. This test is an impact compaction test, which utilizes a disk to cover the top of the soil in an attempt to achieve more energy being imparted into the soil. However, the energy used to compact the soil is not known, and therefore, the test cannot be used as a guide for method specifications for field compaction. In spite of this drawback, it appears from our laboratory results that Test 113-E is essentially equivalent to the Standard Proctor compaction test.

A problem with this test and the Standard Proctor compaction test is the relatively low values of maximum dry density or dry unit weight achieved during these tests on fine, uniform sands. These values are often so low that contractors claim that they can pour the sand out of a dump truck and obtain nearly 80 percent of the value obtained by these two tests. This means that the contractors have to minimally compact the sand to achieve the required specifications. Due to the low dry density values obtained with Test 113-E for the fine, uniform-grained sands, other compaction tests have been investigated such as the Modified Proctor compaction test and the Vibrating Hammer compaction test.

Compaction Specifications

The compaction specifications for TxDOT were compared with various states to determine if there were any similarities or differences. A brief comparison of TxDOT's specifications with FHWA's, Reinforced Earth's, Missouri's and California's specifications. indicate that TxDOT's specifications are in general accord with all of the specifications. However, California's specifications for compaction of the backfill allow for ponding or jetting of water. This method of compacting fine, uniform sands usually works very well as long as it is controlled. In addition, this method of compacting the soil is in accord with collapsing soil tests (see Lawton, 1995).

Control of Compaction during Construction

Two different areas of quality control need to be addressed. These are the measurement of the field dry unit weight or density and the field determination of the gradation of the soil used as backfill.

Measurement of the Field Dry Density. Various methods for determining the field or insitu dry density of the backfill have been proposed, such as the sand cone test, balloon test, nuclear probe test, etc. These tests all are done on the surface layer of the compacted fill and may not yield accurate results because they all involve disturbing the soil. As known from deformation tests on sand, sands which are loose will densify when deformed, and sands that are dense will dilate or loosen when deformed. What this means is that loose sands may give higher densities when tested, whereas dense sands may yield lower density values when tested. As a consequence, the true dry density of the sand insitu may not be known. Therefore, other test methods have been suggested in the literature, such as ground penetrating radar, and should be investigated.

In addition to the test method for determining the field compacted state of the soil, the spacing and frequency of the measurements should also be considered. The spacing and frequency of the measurements are important because they determine the uniformity of the compacted soil. As indicated in Table 1, nonuniformity of the compacted backfill can lead to serious problems for MSE walls.

Field Determination of the Gradation of the Backfill Soil. Compaction of the backfill depends upon the water content, gradation of the soil, grain shape of the soil, energy used, and method of compaction used. As such, to ensure uniformity of the compacted backfill soil, the gradation characteristics of the soil should be determined in the field because the compaction specifications for the backfill may have to be changed with a gradation change in the soil.

3. LITERATURE REVIEW

3.1 Fundamentals of Compaction

Proctor (1933) first presented the fundamentals of soil compaction. Soils are three-phase systems consisting of a solid phase, a liquid phase, and a gas phase. Prior to compaction soil occupies a certain volume (Figure 1). After compaction, the volume of soil is decreased as the volume of the soil voids are decreased. The decrease in volume of the soil causes an increase in the dry unit weight of the soil. Consequently, compaction is the densification of the soil through a reduction in volume using mechanical means.

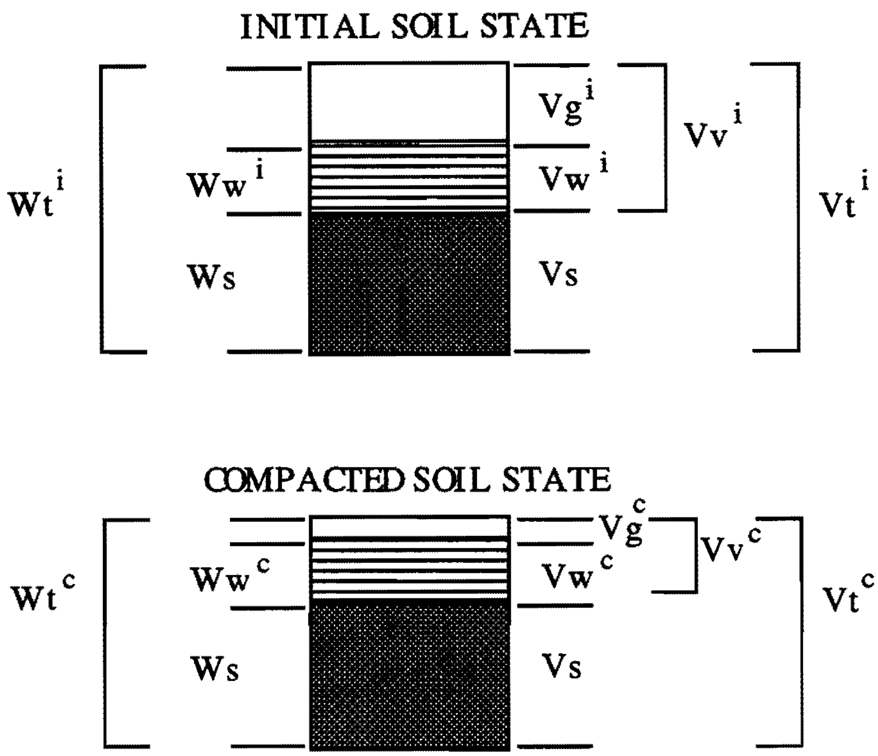
Numerous methods and types of equipment are used to compact soils. While full-scale field compaction testing is preferable, it is rarely used in practice because the cost is extremely high (Lawton, 1996). Laboratory compaction tests commonly cost less than field tests and are used to specify the field compaction of the soil.

3.2 Factors Influencing the Compaction of Cohesionless Sands

Burmister(1948), Johnson and Sallberg (1962) and Semmelink and Visser (1994) stated the most important factors controlling the compaction of granular fill are the water content, grain size distribution of the particles, grading of the soil, shape of the particles, and the laboratory test method used. These factors will be briefly investigated individually to illustrate their relative importance in the compaction of granular soils.

Water Content

Felt (1958) and Johnson and Sallberg (1962) indicated that the water content within a granular soil could actually resist the compactive effort and yield low dry densities, especially in impact types of compaction tests. The reason for this behavior is that at low water contents, capillary stresses exist within the soil which resist the densification of the soil. Consequently, compaction curves for granular soils can be oddly shaped, with the maximum dry density or unit weight occurring for either a dry or saturated soil (Figure 2). This behavior has led some engineers to specify that the field compaction of the soil be done either at water contents that are either dry or saturated. These are impracticable in most areas because it is difficult to dry the soil in the field, and the amount of water needed to saturate the soil can be quite high so as to make this recommendation expensive (note: cohesionless sands are basically free draining so that water must be continuously added to keep them saturated). Therefore, it is necessary to document which soil types and laboratory tests might yield this behavior.



$Vt^i > Vt^c$

<u>Compacted Dry Unit Weight</u>

$\gamma_{dry} = Ws / Vt^c$

<u>Definitions</u>

Ws = weight of solids
Ww = weight of liquid
Wt = total weight

Vs = volume of solids
Vw = volume of liquid
Vg= volume of gas
Vv = volume of voids
Vt = total volume

Figure 1. Basic Explanation of Soil Compaction.

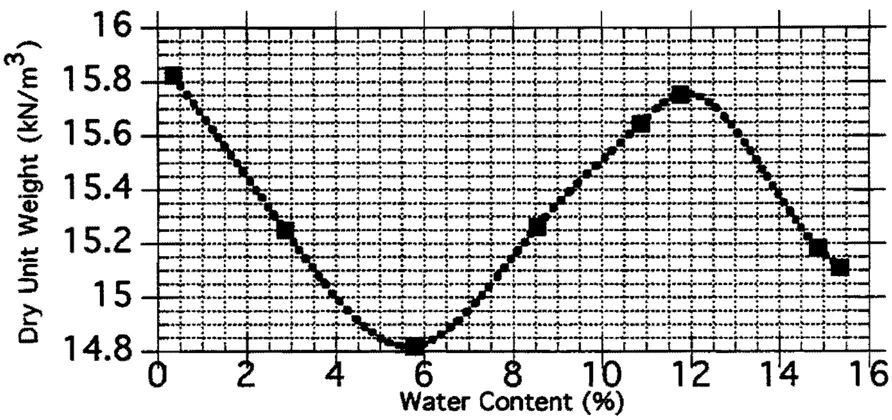


Figure 2. Standard Proctor Compaction Curve for a Uniform Fine Grained Sand.

The maximum dry unit weight is 15.82 kN/m^3, which occurs for a near zero water content. Notice that for a water content of 12%, the maximum dry unit weight is 15.75 kN/m^3.

Grain Size Distribution

The grain size distribution refers to the range of particle sizes within a granular soil. This important parameter can be estimated using the coefficient of uniformity, which is defined as:

C_U = Coefficient of Uniformity = D_{60}/D_{10}

where: D_{60} = Grain Size for 60% Passing

and D_{10} = Grain Size for 10% Passing.

The coefficient of uniformity would be 1 for a uniformly grain sized soil and would be greater than 1 for soils with a wide range of grain sizes. In principle, the greater the coefficient of uniformity, the more wide spread the grain sizes and the higher dry unit weight which should be obtained by the soil. In fact, Kolbuszewski and Frederick (1963) demonstrated that the maximum dry density increases with an increase in the range of particle sizes for granular soils. Consequently, the results of Kolbuszewski and Frederick (1963), Youd (1973), Poulos and Hed (1973), and Johnston (1973) indicate that the dry density or dry unit weight of granular soils should increase with an increasing value of the coefficient of uniformity. However, a problem with considering only the coefficient of uniformity is that it does not account for the shape of the grain size distribution curve. For example, Figure 3 indicates two grain size distribution curves which have the same coefficients of uniformity. However, the two curves are different and could yield different compaction properties.

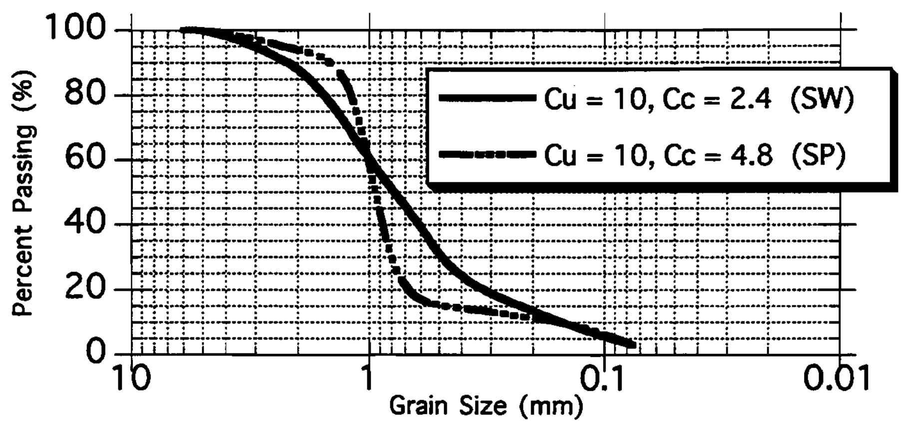


Figure 3. Grain Size Distribution Curves For Well Graded and Poorly Graded Sands.

The two sands have the same coefficients of uniformity but have different coefficients of curvature.

Grading of the Soil

Grading of the soil refers to the shape of the grain size distribution curve, which is quantified by the coefficient of uniformity and the coefficient of curvature. The coefficient of curvature is defined as:

C_c = Coefficient of Curvature = $(D_{30})^2/(D_{10}D_{60})$

where: D_{60} = Grain Size for 60% Passing

D_{30} = Grain Size for 30% Passing

and D_{10} = Grain Size for 10% Passing.

The coefficient of curvature and coefficient of uniformity are used in defining the type of soil according to the Unified Soil Classification System. The Unified Soil Classification System groups granular soils with similar grain size distribution curves, which also implies that they have similar mechanical behavior. The grading of granular soils is important because it is an indicator of strength, compressibility, and compaction. For example Burmister (1948) showed that granular soils that are well graded compact to a denser state than poorly graded soils.

The primary reason for this behavior is that smaller grain sizes can fill in the voids left by larger grains for well-graded soils to produce a denser, more solid like material. Poorly graded soils have either a smaller range of grain sizes or have an insufficient amount of certain grain sizes. Thus, poorly graded soils have voids left within the soil that cannot be filled. This generic type of behavior is illustrated by Figure 3. At present, research efforts have focused on the coefficient of uniformity in connection with the compaction of granular soils (Youd, 1973; Poulos and Hed, 1973; Reitz, 1973; Semmelink and Visser, 1994). Burmister (1948) stated that the grading of the soil is not as important as the grain size distribution. It is important to determine if this conclusion is true.

Shape of the Particles

The shape of the particles is important in the compaction of granular soils because it provides a means of estimating the ease to which the particles may be arranged. For example, round grains can be forced together fairly easily because they can roll and twist into place. In contrast, angular grains are fairly difficult to force together because their pointed corners tend to prevent rolling of the grains, and the grains can only be forced together by sliding. A conclusion of the effect of the particle's grain shape on compaction is that the dry density or dry unit weight should decrease with increasing angularity of the particles. This is exactly what Kolbuszewski and Frederick (1963), Youd (1973), and Dickin (1973) found in their studies.

Grain Crushing During Testing

The crushing of individual grains during testing is important to document because it can change the behavior of the soil (Poulos, 1988; Semmelink and Visser, 1994). For example, the reduction in grain size and a change in grading due to grain crushing during testing could change the behavior of the soil from a cohesionless granular soil to a cohesive granular soil (Poulos, 1988). It is expected that grain crushing will be important for materials like calcareous sands (which has weak particles) and angular sands (the angular points of the grains could break off), especially during impact compaction laboratory tests. However, to document its effect, the grading of soil will have to be tested both before and after each test.

Laboratory Testing Methods

Numerous methods have been used to compact granular soils (see Felt, 1958; Johnson and Sallberg, 1962; and Parsons, 1992). These typically fall into one of two categories: 1) impact tests; and 2) vibration tests. Impact compaction tests include standard Proctor, modified Proctor, and variations of them. Vibration tests include the vibrating table test and the vibrating hammer test. If compaction was independent of the laboratory test type, then each method should ideally yield a single maximum dry density or dry unit weight and water content for a given granular soil, but they do not. This is because the conditions under which each of the tests are performed are different.

Table 2 indicates the differences in methods and operators for a known standard granular soil, Ottawa sand. Consequently, it is difficult for the practicing engineer to select which method should be used to determine the dry density or dry unit weight to be used in specifying field compaction of the soil. Criteria for determining which method(s) should be used are the ease of running the tests, ease of interpreting the data, and reproducibility of the results. Impact tests satisfy this criteria and have been widely used; however, low values of maximum dry unit weight have been achieved with these tests on fine sands compared to what contractors can achieve in the field (Felt, 1958; Parsons, 1992). In contrast, the vibrating hammer test also satisfies the above criteria and yields higher values of the dry unit weight in accordance with that which contractors can achieve in field compaction (Parsons, 1992). In this study, impact and vibrating hammer tests were used.

Table 2. Comparison of Various Methods for Determining the Maximum Dry Unit Weight for Ottawa Sand

SOIL TYPE	SOURCE	R*	γ_{dry} Max. (kN/M^3)	γ_{dry} Max. METHOD
OTTAWA SAND 109#	YOUD (1972)	3.5	17.6	VIBRATING TABLE
OTTAWA SAND 109#	YOUD (1973)	3.5	18.4	CYCLIC SIMPLE SHEAR
OTTAWA SAND 109#	HOLUBEC & D'APPOLONIA (1973)	3.5	17.54 17.55	-VIBRATION BY TAPPING -MODIFIED PROCTOR
OTTAWA SAND 109#	VAID & NEGUSSEY (1988)		17.33	-VIBRATING TABLE -PLUVIATIOIN
OTTAWA SAND 109#	this report	3.5	18.2	-VIBRATING HAMMER
FINE OTTAWA SAND#	VAID & NEGUSSEY (1988)		16.66	-VIBRATING TABLE -PLUVIATIOIN
OTTAWA SAND 190#	YOUD (1973)	3.5	18.4	CYCLIC SIMPLE SHEAR

* R = Roundness expressed in terms of Power's Chart for Estimating Roundness (AGI Data Sheets, 1982).
0.5 (very angular) $\leq\leq$ R $\leq$ 5.5 (well rounded) {3.5 = subrounded}

Properties of the Soils:

Ottawa Sand 109# - D50 = 0.36 mm, Cu = 1.8 - 1.9
Fine Ottawa Sand# - D50 = 0.16 mm, Cu = 1.8
Ottawa Sand 190# - D50 = 0.68 mm, Cu = 1.3

3.3 Interaction of the Various Parameters

The above discussion assumed that each parameter was independent of the others. This is not true, as the parameters all interact with one another during both laboratory and field compaction of cohesionless soils. While the literature contains numerous articles on compaction of granular soils, only Burmister (1948) and Youd (1973) have studied the interaction among some of the various parameters that affect the compaction of cohesionless sands.

Burmister (1948) studied the effects that grain size distribution and soil grading have on the compaction of granular soils. Burmister (1948) performed compaction tests on a wide range of granular soils using an early version of the vibrating hammer. The results of the study indicated that the grain size distribution was the most important characteristic influencing compaction of sands.

Youd (1973) studied the effects of grain size distribution, grading of the soil, and particle shape on the compaction of cohesionless sands. Youd (1973) performed compaction tests on 22 natural and commercially graded sands using a cyclic shear apparatus. The results supported Burmister's (1948) conclusion but also indicated that the particle shape was equally as important. Youd (1948) presented curves of void ratio versus coefficient of uniformity for various particle shapes to support his conclusions. Figure 4 illustrates one of Youd's (1973) curves for a subround particle shape, assuming that the specific gravity of solids is 2.67.

Figure 4 also illustrates the results of Poulos and Hed (1973) for Fill II[1] and Johnston (1973). The results shown in the figure support Burmister's (1948) conclusion that the grain size distribution is an important parameter for cohesionless sands. Additionally, the figure indicates that the laboratory compaction test method used may also have a significant effect on the compaction of cohesionless sands. Therefore, it is important to determine if these conclusions are valid so that a better understanding of the compaction of cohesionless sand can be obtained.

[1] Fill I is not represented in Figure 4, as it included some sections of gravel, and the particle shape ranged from subround to subangular. It is unclear as to the actual particle shape of the soils, as this was not reported. Fill II was a homogeneous fill, composed of subround sands.

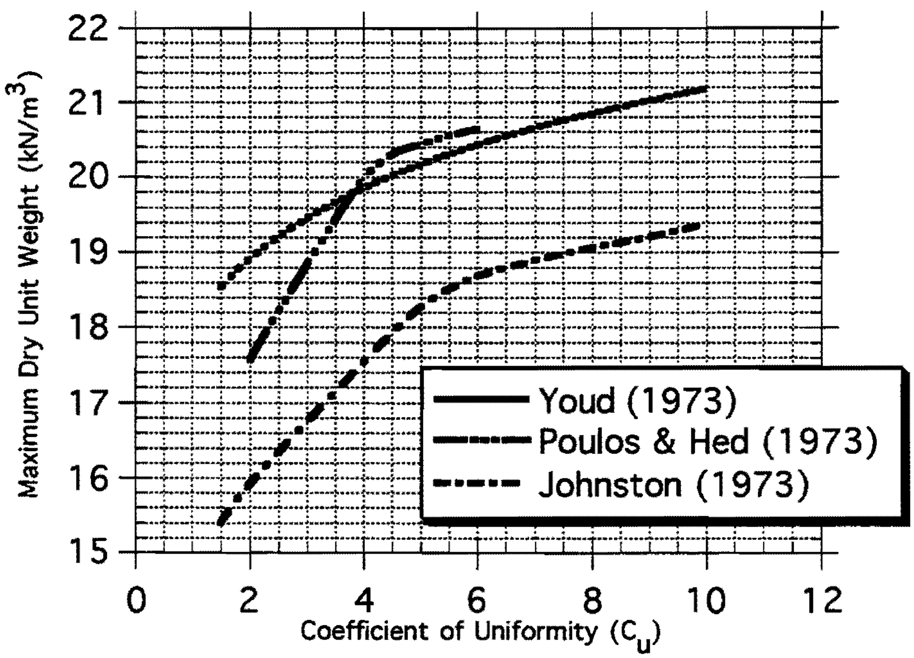


Figure 4. Maximum Dry Unit Weight vs Coefficient of Uniformity for Various Clean Subrounded Sands.

Different compaction methods were used by the various authors. Youd (1973) used a cyclic simple shear device. Poulos and Hed (1973) used the modified Proctor impact compaction test. Johnston (1973) used a modified vibrating table test.

4. LABORATORY TESTING PROCEDURES

4.1 General

To adequately determine the importance of the various factors that influence the compaction of cohesionless sands, a comprehensive series of laboratory tests was required. This included the following test procedures, defined as follows: (a) sample preparation to prepare the sample for sieve analyses and compaction testing; (b) sieve analysis for determining the grain size distribution and grading of the soils; (c) particle shape analysis for determining the degree of roundness of the soils; and (d) different laboratory compaction tests to determine the influence of the test method and provide a basis for evaluating the various factors. A description of each of the laboratory testing methods used in this study is provided as follows.

4.2 Sample Preparation

Sample preparation was used to prepare the soil for particle size analysis and compaction testing. The sample preparation procedure followed that specified by ASTM (1998), designation D 421-85.

4.3 Sieve Analysis

The grain size distribution of the soil samples was determined using a sieve analysis. The sieve analysis procedure followed that specified by ASTM (1998), designation D 422-90. The results of the sieve analysis were analyzed graphically on a semilogarithmic plot by graphing the percent passing a given sieve versus the grain diameter, plotted logarithmically to base 10, as is the normal custom.

4.4 Particle Shape Analysis

The shape of the grains was determined using the procedure outlined by Youd (1973). The procedure is as follows:

a) A sieve analysis is done on each soil sample.

b) For each sieve fraction, at least 100 grains of sand were examined under a microscope to visually determine the particle shape for that fraction.

c) The shapes of the particles were compared with the roundness chart by Powers, presented by AGI (1982).

d) Each of the 100 grains within a given fraction was assigned a roundness value (Table 3) and the average roundness value was determined as follows:

$R_i =$ roundness value for each individual grain within a sieve fraction.

Then average roundness value for the sieve fraction j is $R_j = \left[\sum_{i=1}^{100} R_i \right] \Big/ 100$

e) Calculate the total roundness value for the entire sand sample.

$$R = \left[\sum_j P_j R_j \right] \Big/ 100$$

where $P_j = (W_j / W_{total}) \times 100\%$

W_j = weight retained on the j^{th} sieve

W_{total} = total weight of soil sample

Table 3. Roundness Criteria and Values*

Roundness Class	Description	Mean Roundness Value
Very Angular	Particles with unworn fractured surfaces and multiple sharp corners and edges	0.5
Angular	Particles with sharp corners and approximately prismoidal or tetrahedral shapes	1.5
Subangular	Particles with distinct but blunted or slightly rounded corners and edges	2.5
Subround	Particles with distinct but well rounded edges and corners	3.5
Round	Irregularly shaped rounded particles with no distinct corners or edges	4.5
Well Rounded	Smooth nearly spherical or ellipsoidal particles	5.5

* - This table is modified from Youd (1973), with the mean roundness values being taken from AGI (1982) Data Sheet 18.1.

4.5 Standard Proctor Compaction Test

The standard Proctor compaction test procedure followed that specified by ASTM (1998), Designation D-698-91. Since this test method is essential to the results of this study, the procedure will be briefly summarized. The procedure is as follows:

> A soil at a select water content is compacted in three equal layers within a 9.44×10^{-4} m^3 (1/30 ft^3) cylindrical steel mold. The compaction of each layer is performed by applying 25 blows from a 2.49 kg (5.5 lb) hammer that drops from a height of 0.305 m (12 in). The total soil is subjected to a total compactive effort of 600 kN-m/m^3 (12,400 lbf-ft/ft^3). The soil within the mold is trimmed so that the volume is exactly 9.44×10^{4} m^3. The soil is then weighed, and the unit weight is calculated. A water content test is performed on the compacted soil, and the dry unit weight is determined. This process is repeated for different water contents, and the results are plotted to establish a relationship between the dry unit weight and the water content for the soil. The maximum dry unit weight and its associated water content are then recorded. Figure 5 illustrates this process.

4.6 Modified Proctor Compaction Test

The modified Proctor compaction test procedure followed that specified by ASTM (1998), Designation D-1557-91. Since this test method is essential to the results of this study the procedure will be briefly summarized. The procedure is as follows:

> A soil at a select water content is compacted in five equal layers within a 9.44×10^{-4} m^3 (1/30 ft^3) cylindrical steel mold. The compaction of each layer is performed by applying 25 blows from a 4.54 kg (10.0 lb) hammer that drops from a height of 0.457 m (18 in). The total soil is subjected to a total compactive effort of 2,700 kN-m/m^3 (56,000 lbf-ft/ft^3). The soil within the mold is trimmed so that the volume is exactly 9.44×10^{4} m^3. The soil is then weighed and the unit weight is calculated. A water content test is performed on the compacted soil and the dry unit weight is determined. This process is repeated for different water contents and the results are plotted to establish a relationship between the dry unit weight and the water content for the soil. The maximum dry unit weight and its associated water content are then recorded. Figure 5 illustrates this process.

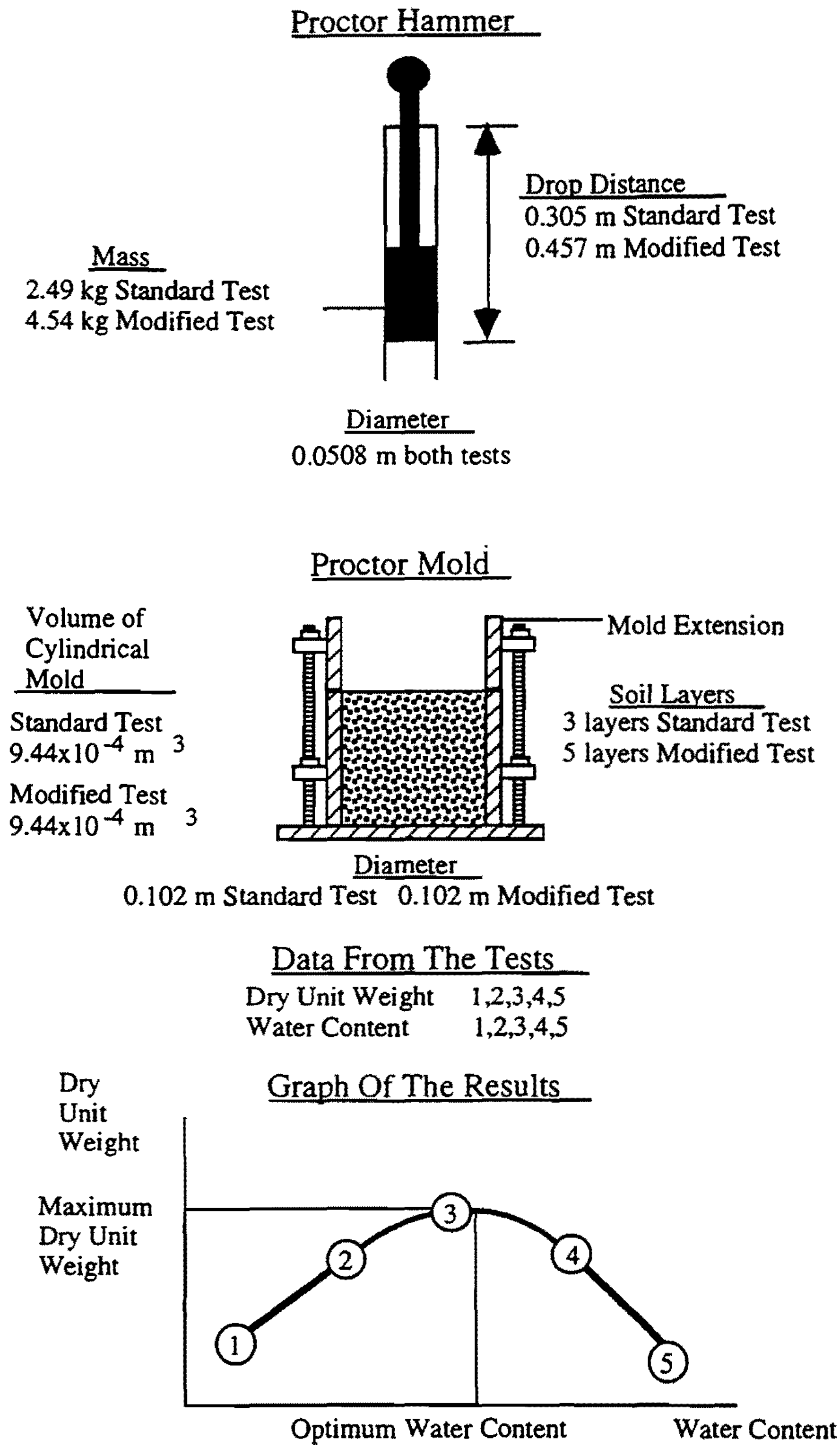


Figure 5. Basic Principles of the Proctor Compaction Tests.

4.7 Vibrating Hammer Compaction Test

The vibrating hammer compaction test procedure followed that specified by the British Standards Institute (1990), British Standard BS-1377. Since this test method is not well known and is essential to the results of this study, the procedure will be briefly summarized. The procedure is as follows:

> A soil at a select water content is compacted in three equal layers within a 32.26×10^{-4} m^3 (1/8.73 ft^3) cylindrical steel mold. The compaction of each layer is performed with a vibrating hammer, with one minute of vibration per layer. The vibrating hammer has a power consumption of 600 - 750 watts, an operating frequency of 1500 - 2500 cycles/min., a circular tamping foot of 0.146 m (5.75 in), and weights attached to it so that the total static load on the tamping foot is between 300 N (70 lbf) and 400 N (90 lbf). The soil within the mold is compacted to a thickness between 0.127 m (5 in) and 0.133 m (5.25 in). The depth of the compacted soil is determined by measuring from a datum bar across the collar of the mold to the top of the soil using a caliper. The unit weight of the soil is determined using the weight and the calculated volume. A water content test is performed on the compacted soil, and the dry unit weight is determined. This process is repeated for different water contents, and the results are plotted to establish a relationship between the dry unit weight and the water content for the soil. The maximum dry unit weight and its associated water content are then recorded. Figure 6 illustrates this process.

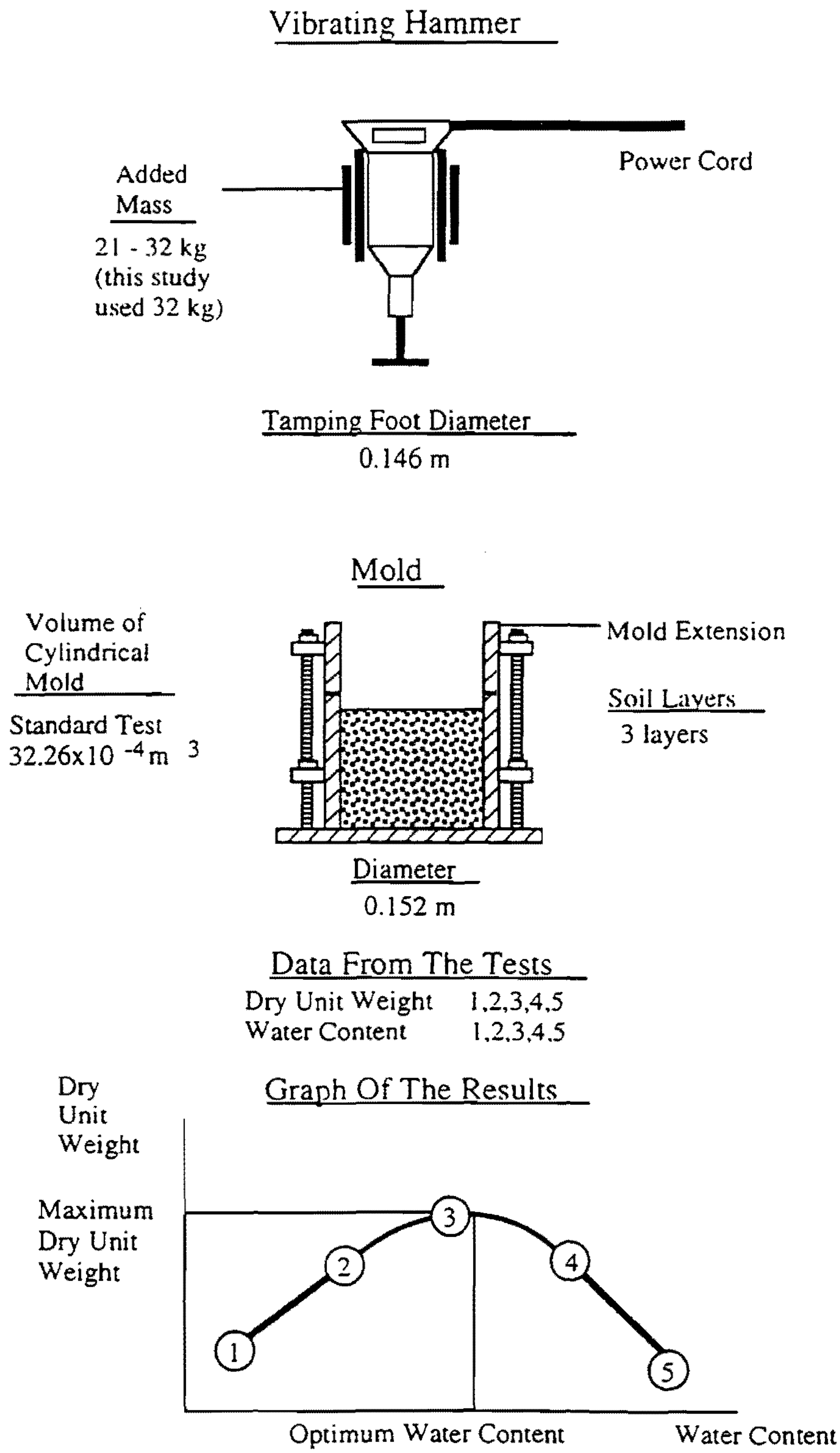


Figure 6. Basic Principles of the Vibrating Hammer Compaction Test.

5. MATERIALS TESTED AND TEST RESULTS

5.1 Materials Tested

A total of 62 cohesionless sands were used in this study. The sands included sandbox sands, concrete aggregate, and naturally occurring sands (the origin for most of the naturally occurring sands is not known). The sands and aggregate were chosen because of their wide range of grain sizes and particle shapes.

5.2 Sieve and Particle Shape Analyses

The results of the grain size distribution, soil grading, and particle shape analyses are given in Table 4. The soil classifications are given in Table 5 and are based on the Unified Soil Classification System. Figure 7 illustrates some representative grain size distribution curves for sands.

5.3 Compaction Test Results

The results of the standard Proctor, modified Proctor, and vibrating hammer tests are given in Table 6. Figures 8 and 9 illustrate some representative dry unit weight versus water content curves for the soils tested. No oddly shaped compaction curves were obtained, nor was the optimum water content found to be zero or near the saturated state for the soil.

5.4 Sieve Analyses after Compaction Testing

Table 7 indicates the change in percent passing the number 200 sieve for the 62 soils tested using the modified Proctor compaction test. The change in percent passing the number 200 sieve is an indicator in the change in gradation of the soil. Soils 2, 22, 30, 31, 39, and 40 in Table 7 all had changes in gradation. This is not surprising since these soils are angular/subangular, where breakage of grain points could occur. The values listed did not significantly change the grading of the soils, nor did it affect the compaction results. For the standard Proctor test and the vibrating hammer compaction test, there was no significant change in the percent passing the number 200 sieve (i.e., the change was less than 1 percent). Grain crushing is not important for the soils or tests conducted, presumably because the soils were all quartz rich.

Table 4. Sieve Analysis and Particle Shape Results.

Soil #	Percent Passing Sieve #4	Sieve #10	Sieve #20	Sieve #40	Sieve #80	Sieve #200	Soil Grading Cu	Cc	R
1	100	74	35	15	5	2	6.00	1.36	4.6
2	100	87	58	25	6	1	3.90	1.30	2.1
3	100	92	63	37	7	4	4.00	0.77	3.4
4	100	95	60	31	8	3	4.20	0.95	2.6
5	100	82	57	36	8	2	5.00	0.86	3.6
6	100	97	44	13	6	2	2.30	0.80	2.6
7	100	85	65	34	7	2	3.70	1.10	2.5
8	97	85	65	23	7	3	3.40	1.12	2.5
9	100	100	93	62	17	4	2.40	1.2	2.6
10	100	99	90	60	7	1	3.60	0.78	2.8
11	100	100	97	84	13	1	2.00	1.05	3.2
12	100	100	78	33	4	1	2.00	0.88	3.0
13	100	100	98	80	10	2	1.90	0.91	2.6
14	100	100	92	35	7	1	2.00	0.74	3.6
15	100	100	97	77	33	4	1.90	0.80	3.3
16	100	85	58	38	12	1	5.35	0.80	3.7
17	100	94	83	60	20	3	2.80	0.79	3.5
18	100	100	93	65	15	2	3.25	1.30	4.4
19	100	96	85	62	25	5	3.50	0.88	4.7
20	100	93	67	49	27	3	6.20	0.65	5.0
21	100	96	77	55	27	4	6.00	0.60	4.8
22	100	99	95	60	10	2	2.20	1.00	2.6
23	100	100	90	62	5	1	2.00	0.98	3.4
24	100	100	76	5	1	1	1.40	1.03	3.4
25	100	100	97	77	33	4	2.50	0.80	3.3
26	100	87	57	32	6	2	4.30	0.85	3.5
27	84	60	40	18	7	1	8.00	0.92	3.0
28	100	84	54	18	6	2	4.30	1.30	3.0
29	100	100	97	67	4	2	1.40	0.80	2.7
30	100	100	90	57	10	1	2.50	0.92	2.2
31	100	100	85	56	28	3	4.30	0.75	2.3

$C_u = D_{60}/D_{10}$ and $C_c = D_{30}^2/(D_{60}D_{10})$ R = roundness

where D_{10} = Grain size for 10% passing

D_{30} = Grain size for 30% passing

D_{60} = Grain size for 60% passing

Table 4. Sieve Analysis and Particle Shape Results (continued).

Soil #	Percent Passing Sieve #4	Sieve #10	Sieve #20	Sieve #40	Sieve #80	Sieve #200	Soil Grading Cu	Cc	R
32	68	33	15	7	3	1	8.00	1.45	3.7
33	100	100	92	7	0	0	1.30	0.90	4.0
34	100	100	97	63	7	1	1.80	1.01	3.4
35	100	97	75	33	7	1	2.90	1.00	4.5
36	95	72	44	13	6	2	3.90	0.97	4.5
37	100	100	98	78	12	3	1.90	0.85	3.5
38	100	100	100	97	23	2	1.70	1.10	2.6
39	97	82	53	37	7	1	4.50	0.88	2.0
40	100	97	63	35	6	2	3.10	0.90	2.5
41	100	100	85	27	4	1	2.00	0.89	2.0
42	96	91	78	43	8	4	2.40	0.97	2.0
43	96	72	27	5	3	1	2.80	0.50	4.4
44	100	100	95	68	18	4	3.40	1.0	2.5
45	98	70	58	37	8	2	5.60	0.80	3.4
46	100	77	27	13	5	3	4.80	1.8	3.5
47	98	90	57	34	7	4	4.60	0.65	3.3
48	95	73	52	28	8	2	5.20	0.81	3.0
49	92	68	43	19	6	3	5.60	1.1	3.8
50	100	93	73	43	17	5	6.20	1.36	3.2
51	90	66	43	26	8	1	6.80	0.76	3.2
52	100	70	45	25	10	4	7.20	0.80	3.2
53	93	67	42	20	4	1	4.70	0.80	3.0
54	100	95	62	32	12	1	5.40	1.31	3.0
55	100	97	77	45	18	4	6.40	1.40	3.4
56	100	90	67	44	22	4	6.50	0.68	2.8
57	100	100	98	82	15	2	2.00	0.98	2.5
58	100	95	80	57	15	4	3.00	0.64	2.6
59	100	100	62	25	8	2	4.00	1.56	3.8
60	100	83	54	33	6	1	5.00	0.76	3.7
61	73	48	25	6	4	2	6.00	0.67	4.0
62	75	70	58	29	8	4	7.0	1.10	3.7

$C_u = D_{60}/D_{10}$ and $C_c = D_{30}^2/(D_{60}D_{10})$ R = roundness

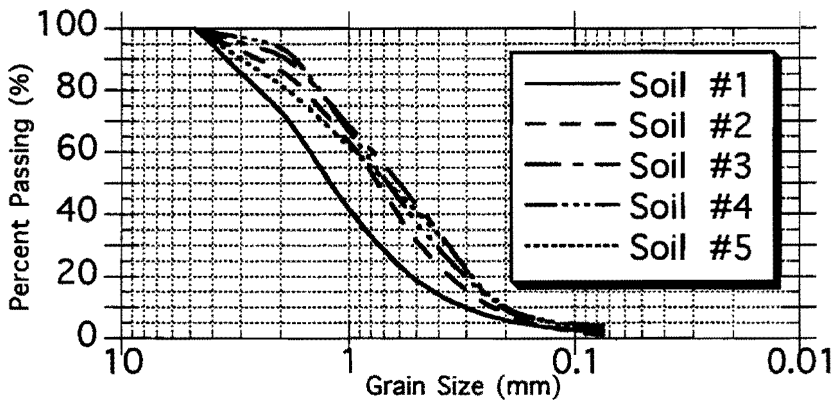
where D_{60} = Grain size for 60% passing.

D_{30} = Grain size for 30% passing.

D_{10} = Grain size for 10% passing.

Table 5. Unified Soil Classification System Classification of the Sands.

Soil #	Soil Type	Description of Soil	Soil #	Soil Type	Description of Soil
1	SW	Medium, well graded	32	SW	Coarse, well graded
2	SP	Medium, poorly graded	33	SP	Medium, poorly graded
3	SP	Medium, poorly graded	34	SP	Fine, poorly graded
4	SP	Medium, poorly graded	35	SP	Medium, poorly graded
5	SP	Medium, poorly graded	36	SP	Medium, poorly graded
6	SP	Medium, poorly graded	37	SP	Fine, poorly graded
7	SP	Medium, poorly graded	38	SP	Fine, poorly graded
8	SP	Medium, poorly graded	39	SP	Medium, poorly graded
9	SP	Fine, poorly graded	40	SP	Medium, poorly graded
10	SP	Fine, poorly graded	41	SP	Medium, poorly graded
11	SP	Fine, poorly graded	42	SP	Medium, poorly graded
12	SP	Medium, poorly graded	43	SP	Medium, poorly graded
13	SP	Fine, poorly graded	44	SP	Fine, poorly graded
14	SP	Medium, poorly graded	45	SP	Fine, poorly graded
15	SP	Fine, poorly graded	46	SP	Medium, poorly graded
16	SP	Medium, poorly graded	47	SP	Medium, poorly graded
17	SP	Medium, poorly graded	48	SP	Medium, poorly graded
18	SP	Fine, poorly graded	49	SP	Medium, poorly graded
19	SP	Fine, poorly graded	50	SW	Medium, well graded
20	SP	Fine, poorly graded	51	SP	Medium, poorly graded
21	SP	Fine, poorly graded	52	SP	Medium, poorly graded
22	SP	Fine, poorly graded	53	SP	Medium, poorly graded
23	SP	Fine, poorly graded	54	SP	Medium, poorly graded
24	SP	Medium, poorly graded	55	SW	Medium, well graded
25	SP	Fine, poorly graded	56	SP	Medium, poorly graded
26	SP	Medium, poorly graded	57	SP	Fine, poorly graded
27	SP	Medium, poorly graded	58	SP	Fine, poorly graded
28	SP	Medium, poorly graded	59	SP	Medium, poorly graded
29	SP	Fine, poorly graded	60	SP	Medium, poorly graded
30	SP	Fine, poorly graded	61	SP	Medium, poorly graded
31	SP	Fine, poorly graded	62	SW	Medium, well graded



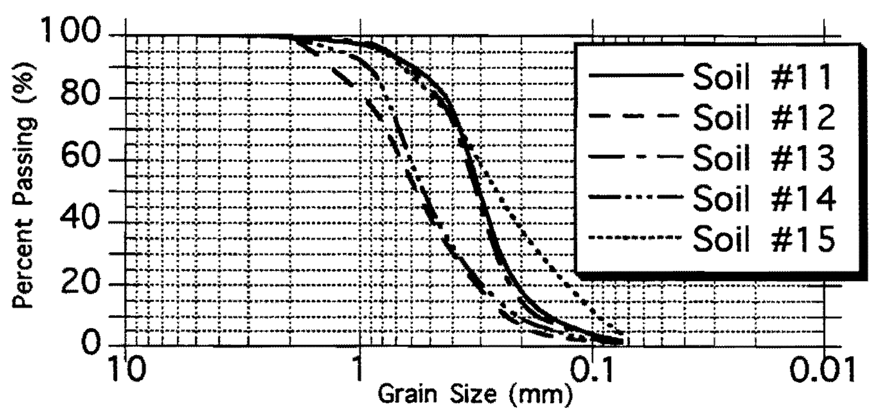


Figure 7. Sample Graphs of the Grain Size Analysis for the Various Soils.

Table 6. Compaction Test Results.

	Standard Proctor		Modified Proctor		Vibrating Hammer	
Soil #	Max. Dry Unit Weight (kN/m^3)	Optimum Water Content (%)	Max. Dry Unit Weight (kN/m^3)	Optimum Water Content (%)	Max. Dry Unit Weight (kN/m^3)	Optimum Water Content (%)
1	19.929	9.0	21.181	7.0	21.125	7.1
2	17.201	13.5	18.597	10.9	18.577	11.6
3	18.094	11.8	19.647	9.1	19.541	9.8
4	17.690	12.3	19.299	10.0	19.106	10.1
5	19.012	10.4	20.406	8.9	20.202	8.8
6	16.474	15.2	17.523	12.8	18.121	12.0
7	17.459	13.0	18.669	10.3	18.986	10.4
8	17.156	13.7	18.300	11.5	18.700	11.4
9	16.605	14.7	17.550	12.5	18.183	12.8
10	17.579	12.6	18.798	9.8	18.856	10.7
11	16.658	14.5	17.450	12.4	18.324	11.8
12	16.321	15.2	17.253	13.8	18.116	12.0
13	16.165	15.6	16.780	13.8	17.620	13.8
14	16.900	13.8	17.646	11.3	18.746	10.5
15	16.318	15.3	17.407	12.5	18.277	11.9
16	19.265	10.4	20.627	8.1	20.421	8.1
17	17.600	12.7	18.610	11.3	19.717	9.4
18	18.244	11.1	19.350	9.3	19.795	9.1
19	18.806	10.8	20.131	8.9	20.311	8.6
20	19.807	9.2	21.346	7.5	21.250	7.1
21	19.782	8.9	21.456	7.1	20.969	7.5
22	16.359	15.2	17.138	13.0	17.995	12.8
23	16.444	15.4	17.586	12.5	18.465	11.0
24	15.788	16.3	16.840	13.3	17.682	13.5
25	17.071	14.1	18.125	12.0	18.778	10.8
26	18.435	10.8	20.278	9.2	19.873	9.0
27	19.374	9.7	20.558	7.9	20.343	8.9
28	18.000	12.0	19.500	9.0	19.404	10.3
29	15.346	17.3	16.350	14.5	17.213	14.5
30	15.579	17.0	16.920	13.0	17.526	13.8
31	17.419	12.5	18.966	9.7	18.778	10.2

Table 6. Compaction Test Results (continued).

Soil #	Standard Proctor		Modified Proctor		Vibrating Hammer	
	Max. Dry Unit Weight (kN/m^3)	Optimum Water Content (%)	Max. Dry Unit Weight (kN/m^3)	Optimum Water Content (%)	Max. Dry Unit Weight (kN/m^3)	Optimum Water Content (%)
32	19.999	9.1	21.266	7.5	20.969	9.5
33	16.494	15.2	17.436	13.1	18.308	11.4
34	16.644	14.4	17.403	13.5	18.308	11.6
35	18.232	11.8	19.407	9.1	19.873	9.8
36	18.691	10.5	19.986	8.5	20.186	8.4
37	16.800	9.8	18.308	11.3	18.308	11.9
38	15.789	16	16.540	15.0	17.526	13.3
39	17.403	12.8	18.715	10.5	18.621	11.7
40	17.162	13.9	18.193	11.8	18.621	11.0
41	15.364	18.1	16.095	15.3	16.900	14.8
42	15.648	16.3	16.148	15.0	17.213	13.8
43	18.012	11.5	19.032	10.2	19.169	9.8
44	17.739	12.3	18.869	9.8	19.247	9.4
45	18.601	10.4	19.916	9.3	19.717	9.5
46	18.661	10.0	20.162	8.3	20.061	8.8
47	18.341	11.2	19.858	9.4	19.779	9.1
48	18.689	10.1	20.065	8.2	19.904	8.9
49	18.973	10.0	20.751	8.4	20.186	8.1
50	19.007	9.9	20.350	8.8	21.250	7.1
51	19.221	10.2	20.789	8.1	20.374	8.8
52	19.523	9.5	20.606	8.5	20.499	8.5
53	18.280	11.5	19.579	9.3	19.560	10.0
54	18.719	10.4	20.212	8.2	20.030	11.0
55	19.192	9.8	20.466	8.0	20.343	8.8
56	18.822	10.8	20.358	8.3	19.951	8.6
57	16.004	15.7	16.766	15.3	17.604	13.5
58	17.084	14.1	17.974	11.8	18.621	9.1
59	18.401	11.1	19.873	9.3	19.873	8.9
60	19.012	10.3	20.598	8.4	20.343	8.2
61	19.487	9.5	20.718	8.2	20.656	7.8
62	19.821	9.3	21.002	7.9	20.812	7.4

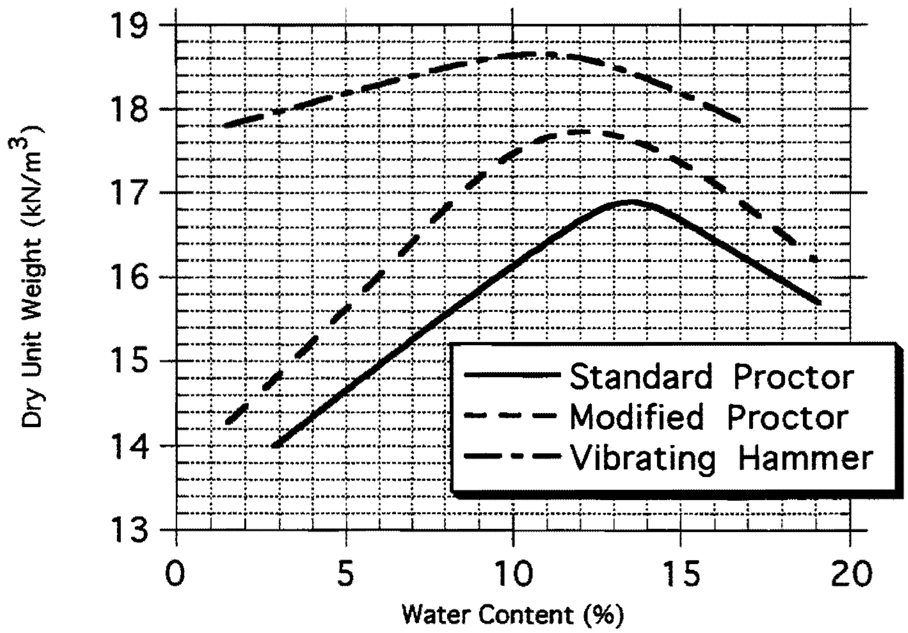


Figure 8. Compaction Curves for a Fine, Poorly Graded Sand - Soil #14.

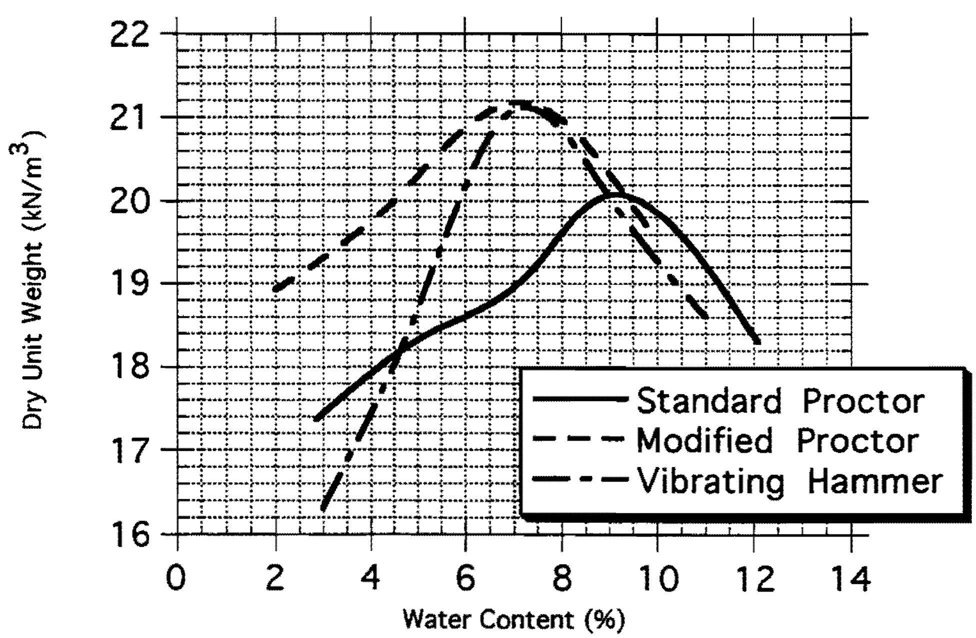


Figure 9. Compaction Curves for a Well Graded Sand - Soil #1.

Table 7. Change in Gradation after Modified Proctor Compaction Testing.

Soil #	Change in % Passing #200 Sieve	Soil #	Change in % Passing #200 Sieve	Soil #	Change in % Passing #200 Sieve
1	0.0	22	1.1	43	0.0
2	2.2	23	0.5	44	0.2
3	0.5	24	0.4	45	0.5
4	1.2	25	0.6	46	0.2
5	0.2	26	0.6	47	0.6
6	0.8	27	0.5	48	0.5
7	0.4	28	0.9	49	0.1
8	0.6	29	0.8	50	0.4
9	0.5	30	2.4	51	0.2
10	0.6	31	2.1	52	0.3
11	0.2	32	0.6	53	0.3
12	0.9	33	0.0	54	0.3
13	0.3	34	0.4	55	0.4
14	0.5	35	0.0	56	0.2
15	0.4	36	0.0	57	0.8
16	0.6	37	0.1	58	0.9
17	0.2	38	0.9	59	0.2
18	0.0	39	3.1	60	0.1
19	0.0	40	1.1	61	0.3
20	0.0	41	2.9	62	0.1
21	0.0	42	2.3		

6. ANALYSIS OF THE TEST RESULTS

6.1 General

The most important factors controlling the compaction of granular fill are the grain size distribution of the particles, the shape of the particles, and the laboratory test method used (Burmister, 1948; Dickin, 1973; Holubec and D'Appolonia, 1973; Poulos and Hed, 1973; Youd, 1973; Johnson and Sallberg, 1962; and Semmelink and Visser, 1994). Water content and the grading of the soil play a limited role in the compaction of soils (Burmister, 1948; Johnson and Sallberg, 1962; and Parsons, 1992). All of these factors are analyzed in this section to gain a better understanding of their relative importance in the compaction of granular soils.

The results of the compaction tests (as shown in Table 6, Compaction Test Results) demonstrate that the laboratory compaction method is important. This is illustrated by the different maximum dry unit weights reported by the various compaction tests. Consequently, the analysis of the test results was performed according to the individual laboratory compaction method.

6.2 Individual Assessment of the Various Factors That Affect the Laboratory Compaction of Cohesionless Sands

Water Content

Figures 10, 11, and 12 illustrate a linear relationship between the maximum dry unit weight and the optimum water content for the different compaction tests. The equation of the best fit line for the various compaction tests is given as follows:

Standard Proctor

Maximum Dry Unit Weight = 24.19 kN/m^3 - 0.5148 kN/m^3 $\times$ Optimum Water Content (%)

Modified Proctor

Maximum Dry Unit Weight = 25.78 kN/m^3 - 0.6461 kN/m^3 $\times$ Optimum Water Content (%)

Vibrating Hammer

Maximum Dry Unit Weight = 24.92 kN/m^3 - 0.552 kN/m^3 $\times$ Optimum Water Content (%)

These equations indicate that the effect of optimum water content on the maximum dry unit weight is not that significant and is relatively insensitive to the laboratory compaction method. For example, for an optimum water content of 10 percent, the maximum dry unit weights obtained from the above equations are 19.04 kN/m^3, 19.32 kN/m^3, and 19.4 kN/m^3, respectively.

Grain Size Distribution of the Soil

Figures 13, 14 and 15 illustrate the effects of grain size distribution of the soil on the maximum dry unit weight for the different compaction tests. The coefficient of uniformity represents the grain size distribution of the soils. These figures indicate the importance of the coefficient of uniformity on the compaction of granular soils and that there is a curvilinear relationship between the maximum dry unit weight and the grain size distribution of the soil.

Particle Shape

Figures 16, 17, and 18 illustrate the relationship between the maximum dry unit weight and the roundness of the soil particles. From these figures, the trend of the results indicates that the maximum dry unit weight increases with increasing roundness and that particle shape is important in the compaction of cohesionless soils. However, there is sufficient scatter in the results to preclude trying to fit a curve through it.

6.3 Interaction between the Various Factors

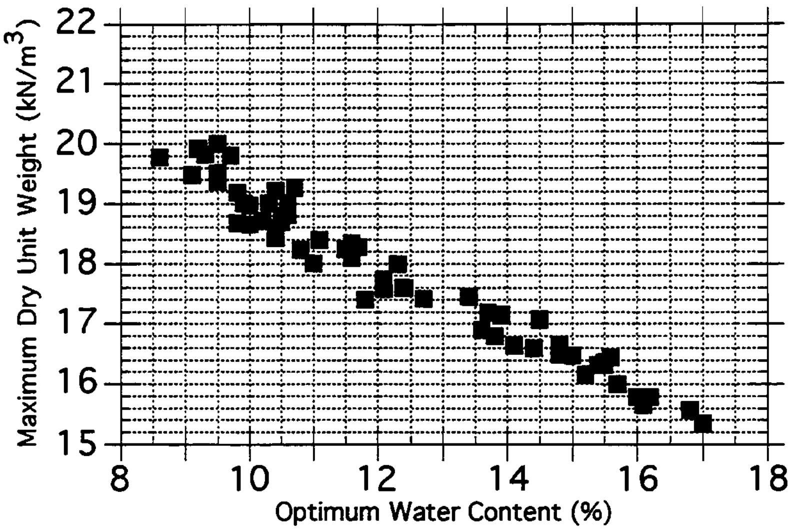
Interaction between the Maximum Dry Unit Weight, Grain Size Distribution and Particle Shape for the Different Compaction Methods

Figures 19, 20 and 21 illustrate the relationship between the grain size distribution, particle shape, and the maximum dry unit weight. These figures clearly illustrate that the maximum dry unit weight increases with increasing grain size distribution (coefficient of uniformity) and with increasing roundness. The results support Youd's (1973) conclusion that the grain size distribution and the particle shape of the soil are very important in the compaction of cohesionless sands.

Interaction between the Maximum Dry Unit Weight, Grading of the Soil, and Particle Shape for the Different Compaction Methods

Figures 22, 23 and 24 illustrate the relationship between the grading of the soil (given by both the coefficient of uniformity, C_u, and the coefficient of curvature, C_c), particle shape, and maximum dry unit weight. These figures indicate that the grading of the soil plays a minor role in determining the compaction of cohesionless soil.

a)



b)

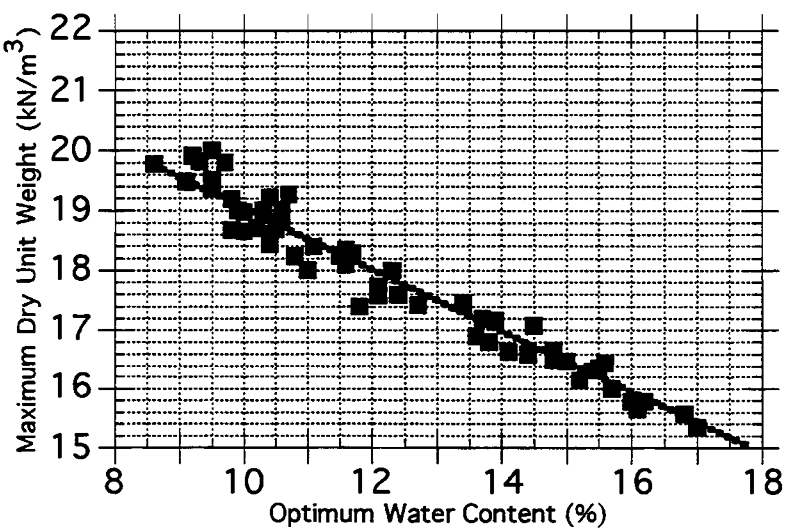
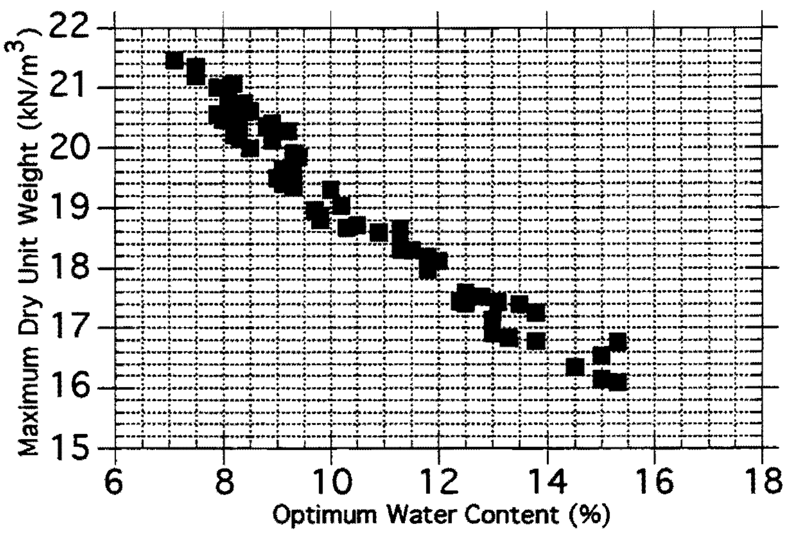


Figure 10. Effect of Optimum Water Content on the Maximum Dry Unit Weight Determined from the Standard Proctor Compaction Test.

a) Plot of the data. b) Best fit linear curve to the data. Maximum Dry Unit Weight = 24.19 kN/m^3 - 0.5148 kN/m^3 $\times$ Optimum Water Content (%).

a)



b)

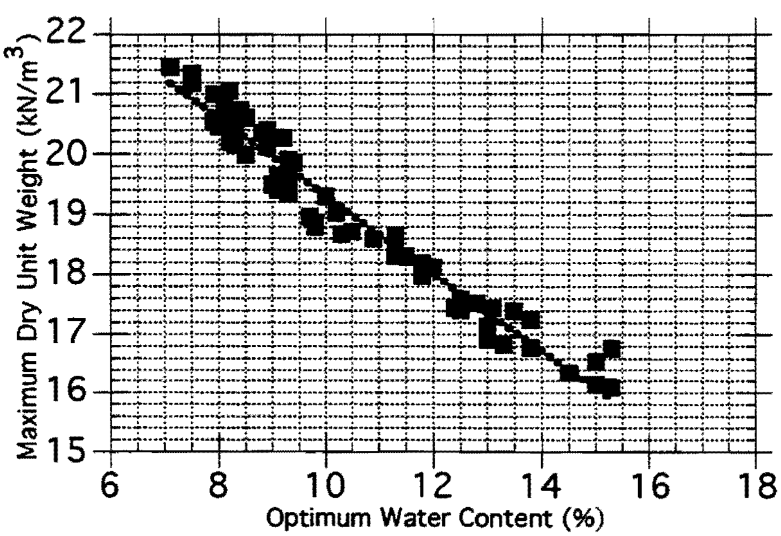
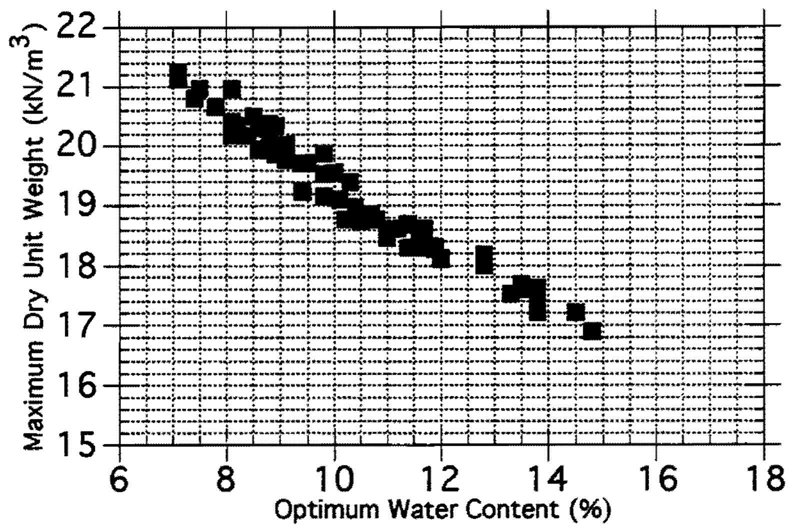


Figure 11. Effect of Optimum Water Content on the Maximum Dry Unit Weight Determined from the Modified Proctor Compaction Test.

a) Plot of the data. b) Best fit linear curve to the data. Maximum Dry Unit Weight = 25.778 kN/m^3 - 0.6461 kN/m^3 $\times$ Optimum Water Content (%).

a)



b)

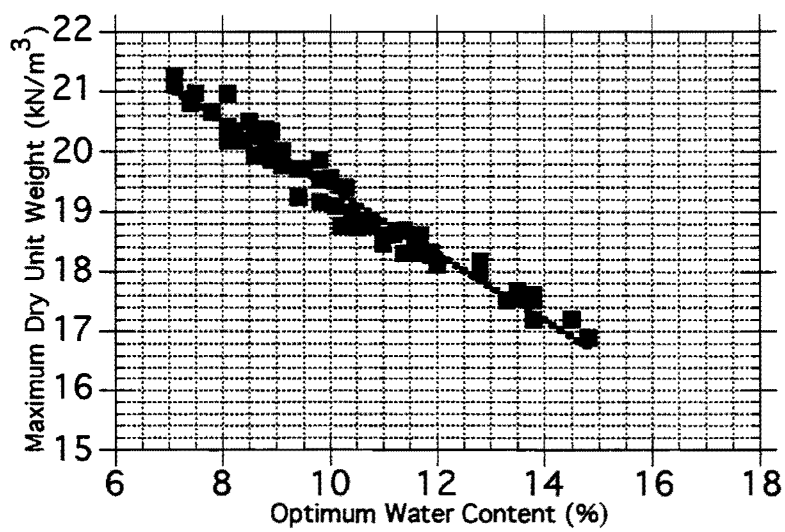


Figure 12. Effect of Optimum Water Content On The Maximum Dry Unit Weight Determined from the Vibrating Hammer Compaction Test.

a) Plot of the data. b) Best fit linear curve to the data. Maximum Dry Unit Weight = 25.778 kN/m^3 - 0.6461 kN/m^3 $\times$ Optimum Water Content (%).

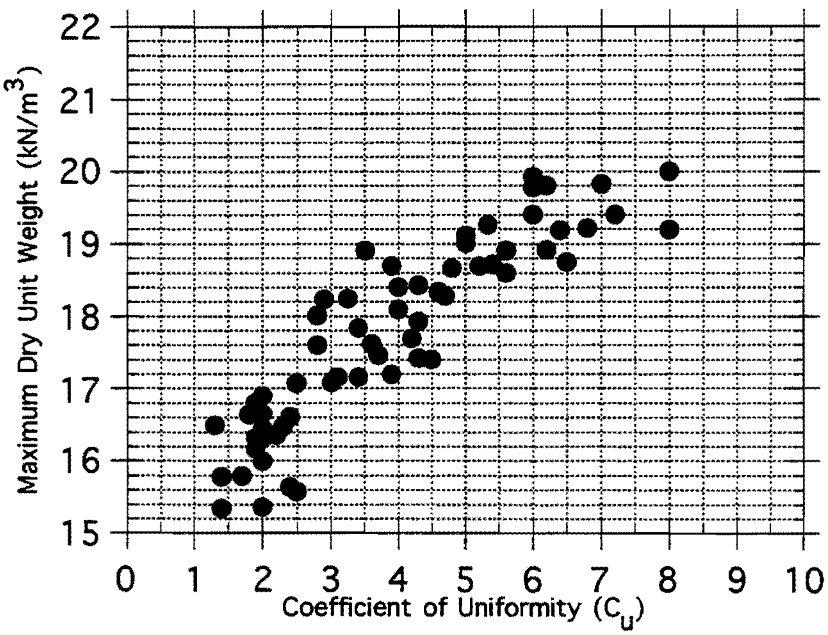


Figure 13. Maximum Dry Unit Weight Variation as a Function of Coefficient of Uniformity for the Standard Proctor Compaction Test.

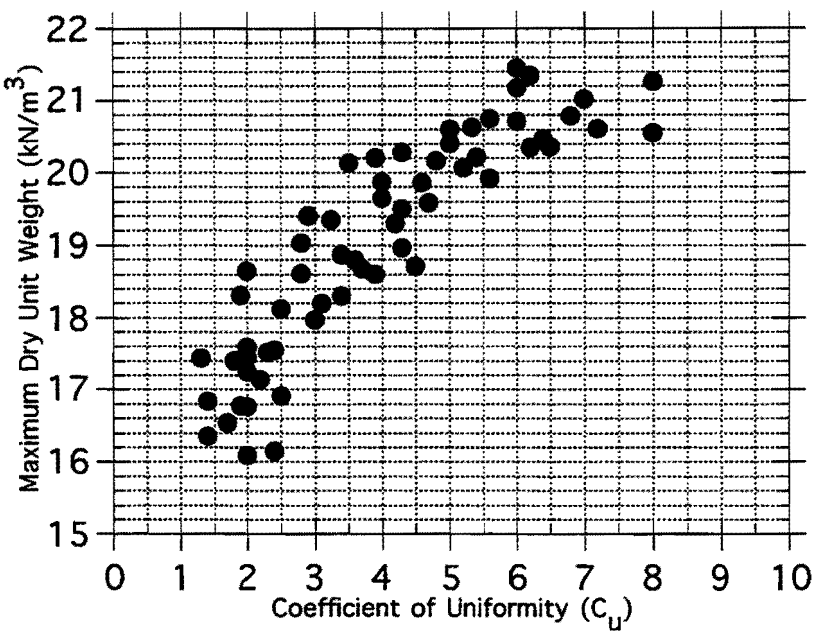


Figure 14. Maximum Dry Unit Weight Variation as a Function of Coefficient of Uniformity for the Modified Proctor Compaction Test.

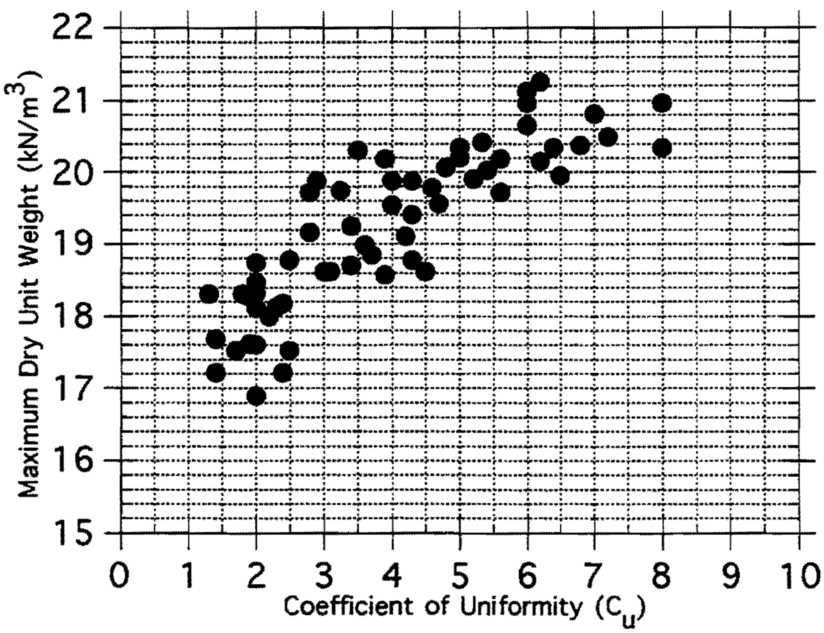


Figure 15. Maximum Dry Unit Weight Variation as a Function of Coefficient of Uniformity for the Vibrating Hammer Compaction Test.

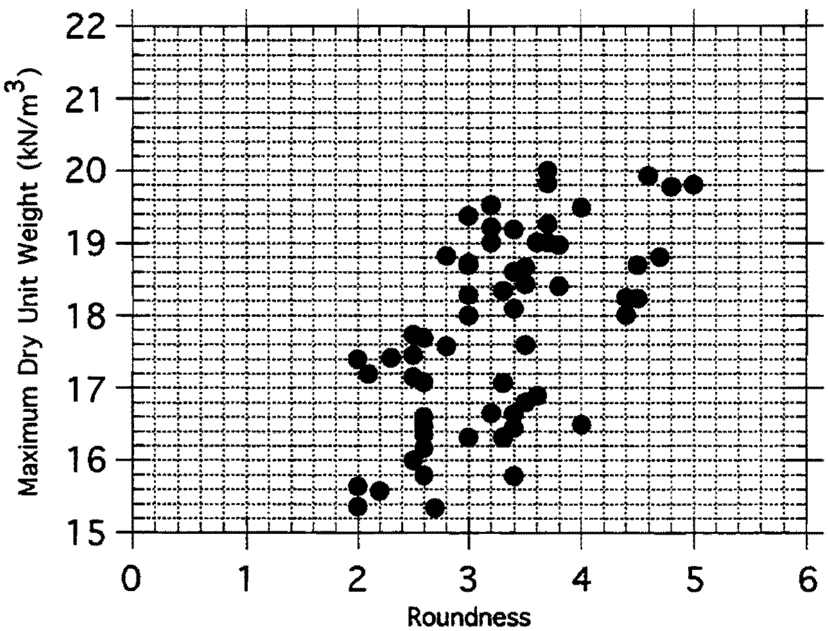


Figure 16. Maximum Dry Unit Weight Variation as a Function of Roundness or Particle Shape for the Standard Proctor Compaction Test.

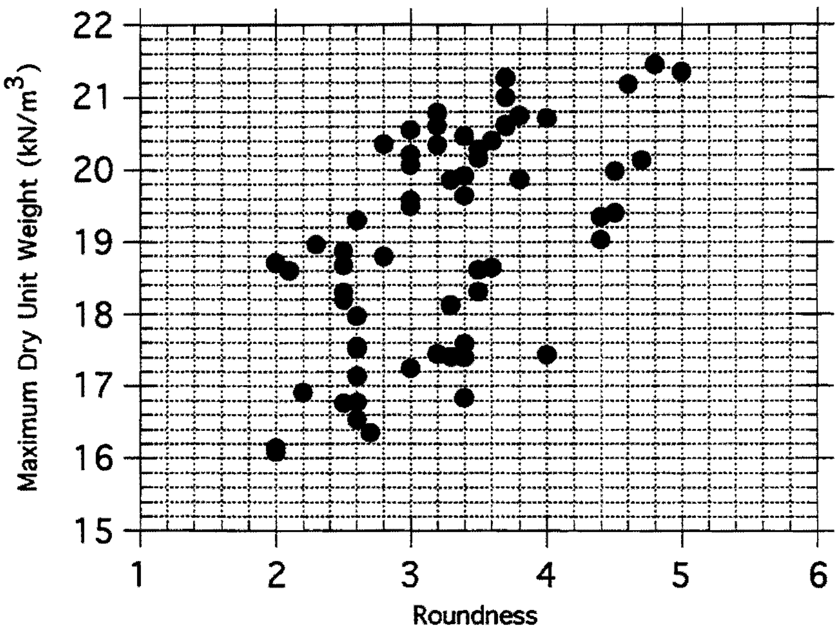


Figure 17. Maximum Dry Unit Weight Variation as a Function of Roundness or Particle Shape for the Modified Proctor Compaction Test.

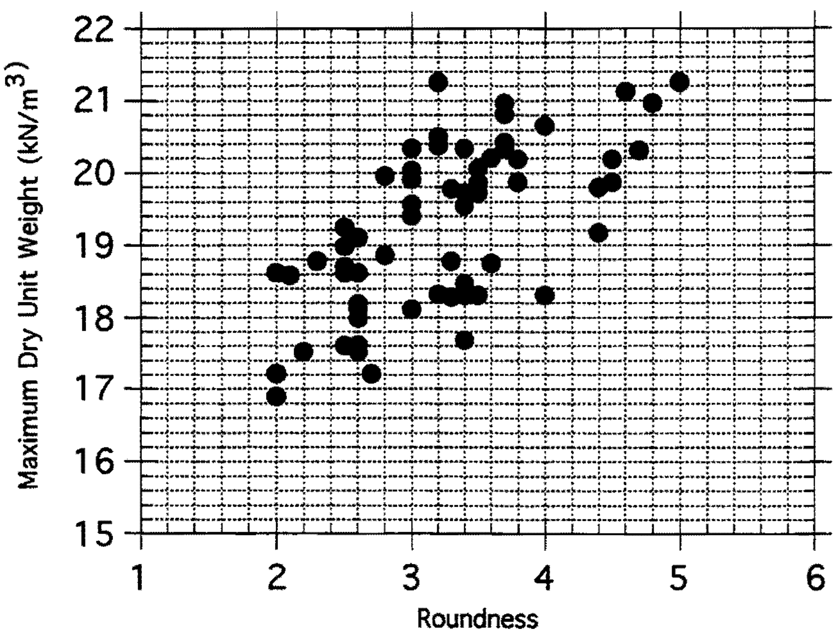
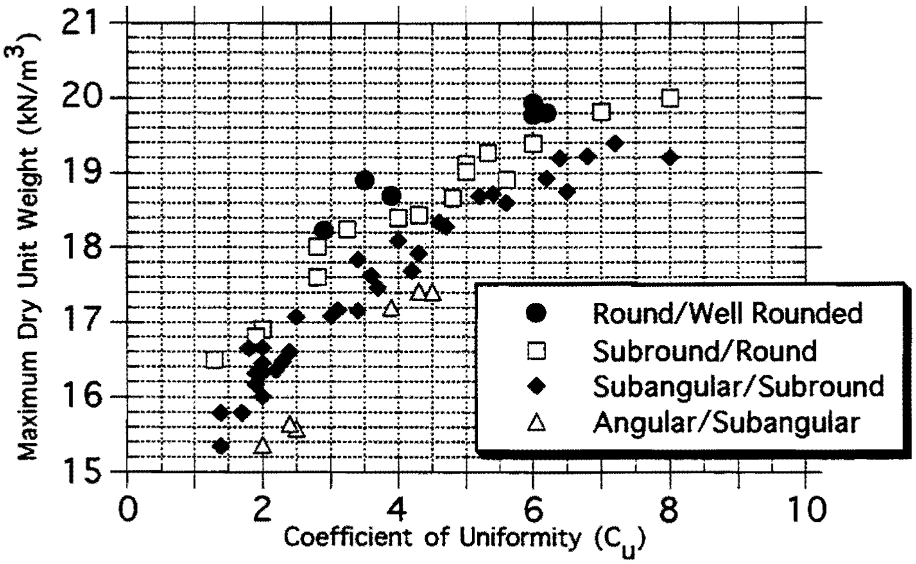


Figure 18. Maximum Dry Unit Weight Variation as a Function of Roundness or Particle Shape for the Vibrating Hammer Compaction Test.

a)



b)

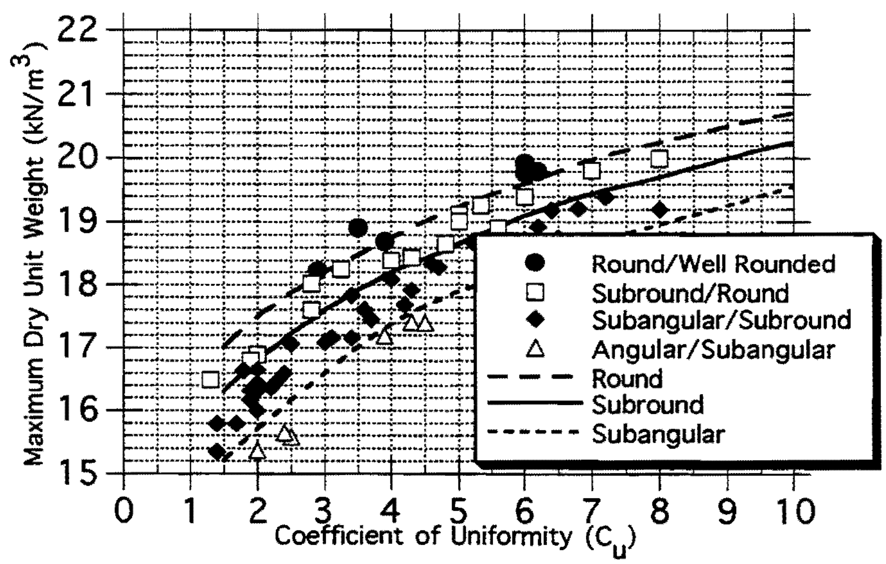
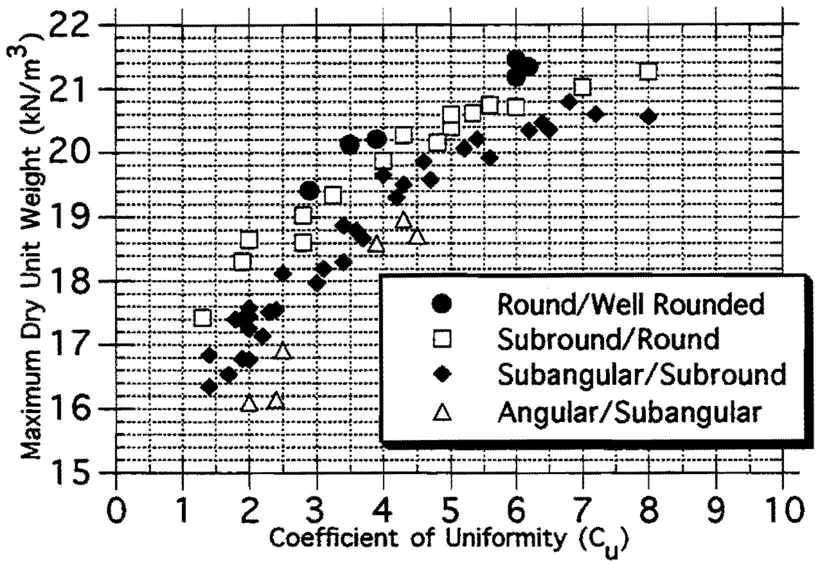


Figure 19. Standard Proctor Maximum Dry Unit Weight as a Function of: a) Grain Size Distribution and Particle Shape; and b) Grain Size Distribution and Particle Shape Including the Particle Shape Curves.

a)



b)

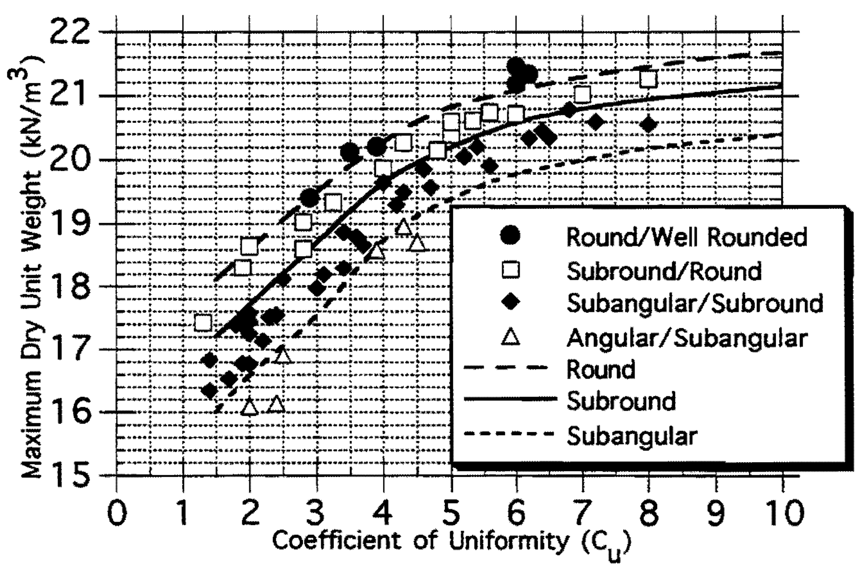
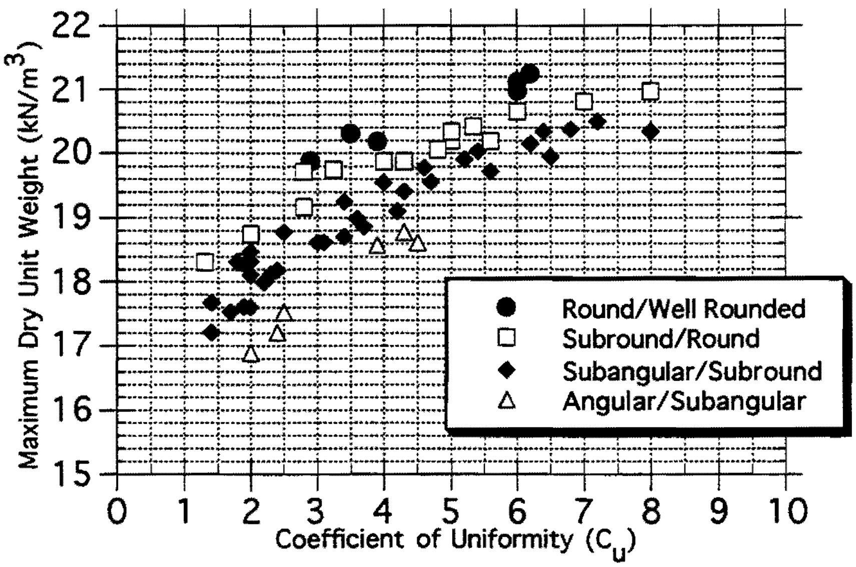


Figure 20. Modified Proctor Maximum Dry Unit Weight as a Function of: a) Grain Size Distribution and Particle Shape; and b) Grain Size Distribution and Particle Shape Including the Particle Shape Curves.

a)



b)

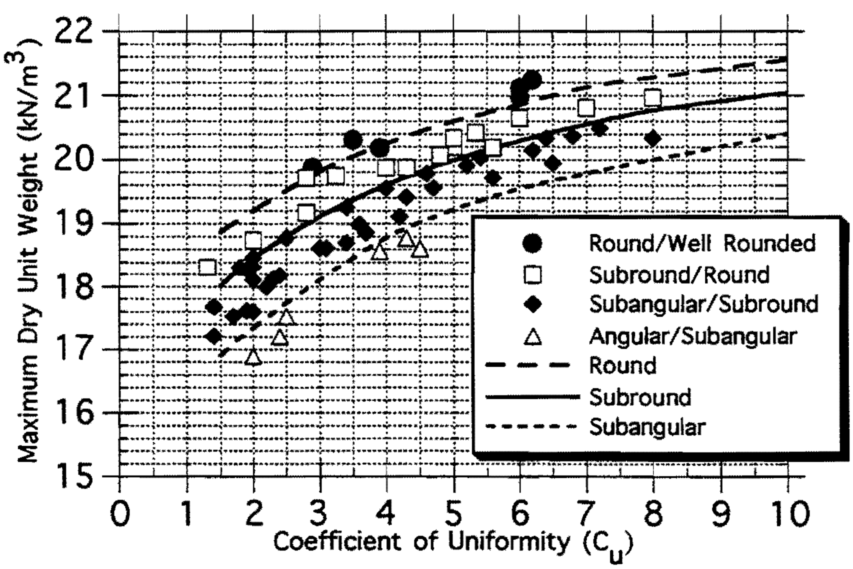
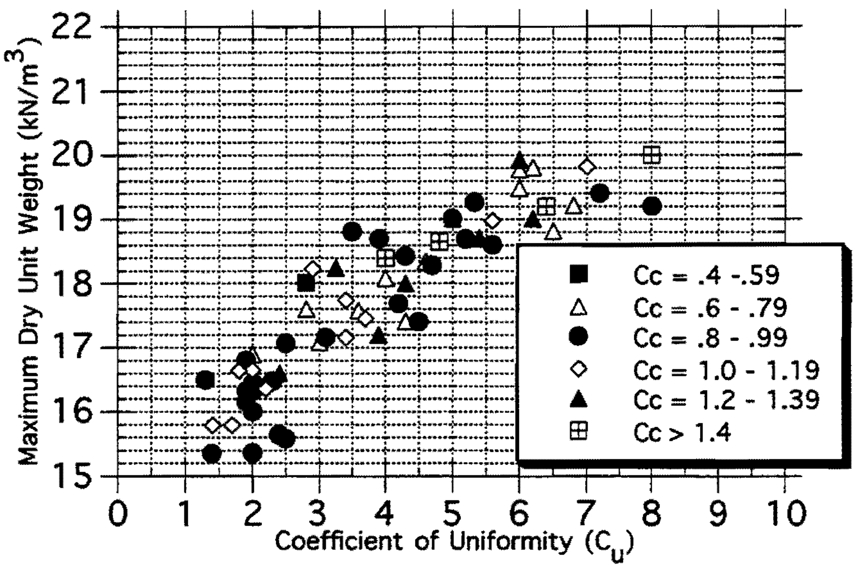


Figure 21. Vibrating Hammer Maximum Dry Unit Weight as a Function of: a) Grain Size Distribution and Particle Shape; and b) Grain Size Distribution and Particle Shape Including the Particle Shape Curves.

a)



b)

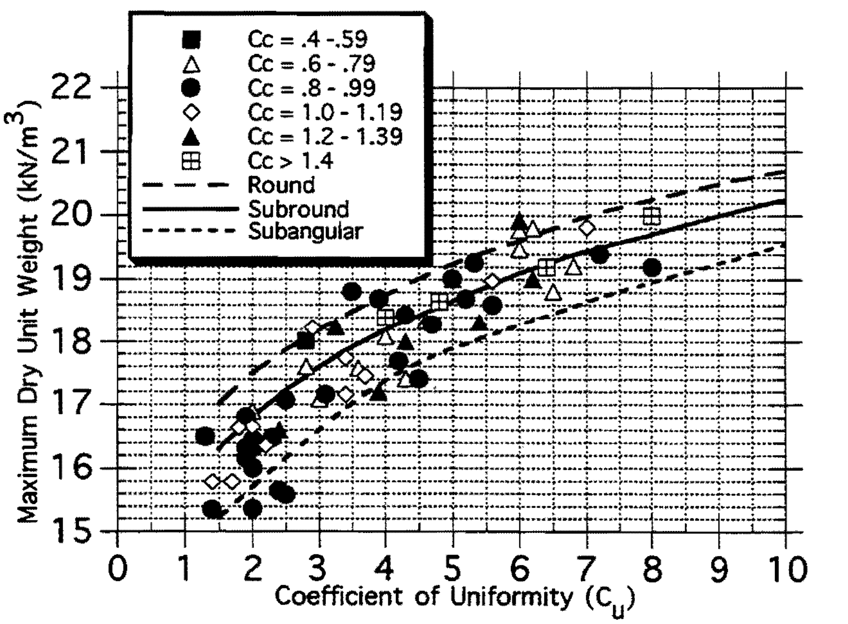
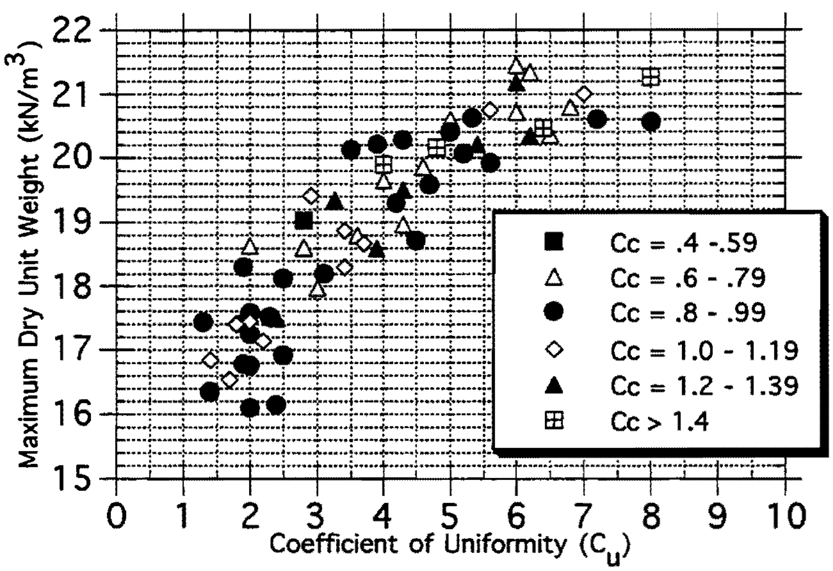


Figure 22. Standard Proctor Maximum Dry Unit Weight as a Function of: a) Grading of the Soil; and b) Grading of the Soil and Particle Shape.

a)



b)

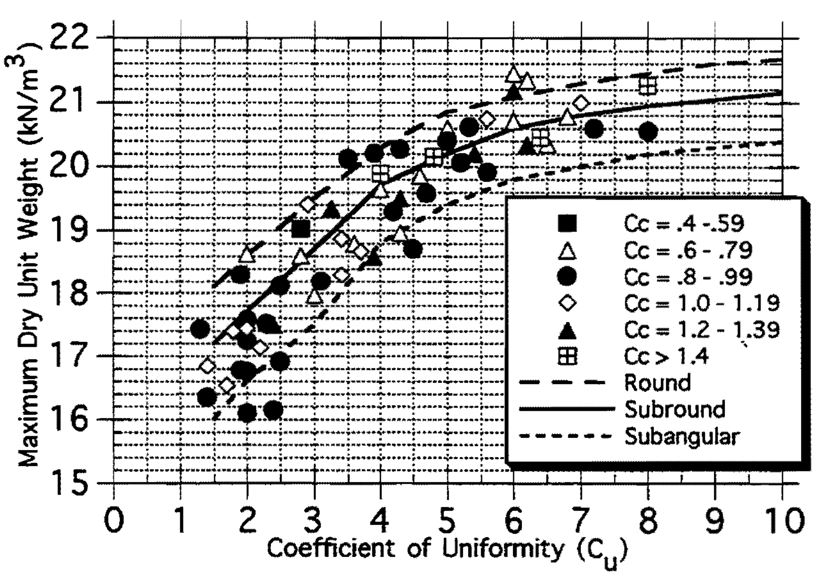
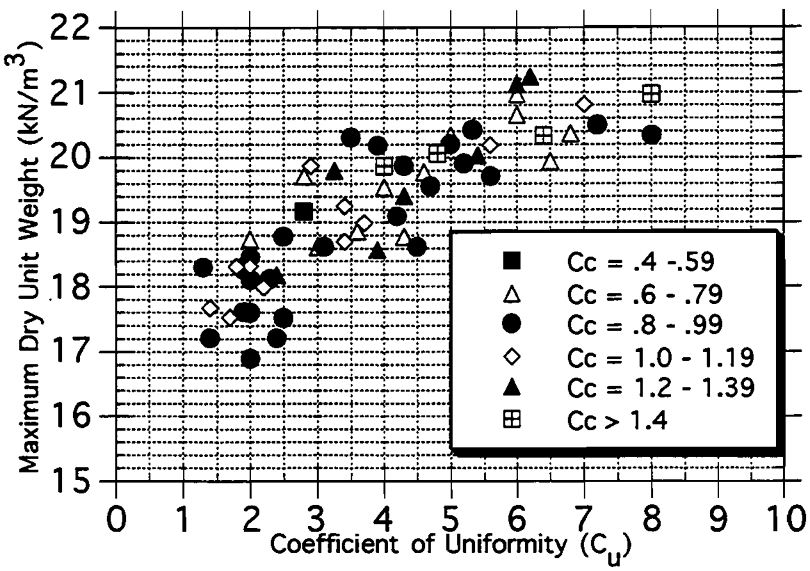


Figure 23. Modified Proctor Maximum Dry Unit Weight as a Function of: a) Grading of the Soil; and b) Grading of the Soil and Particle Shape.

a)



b)

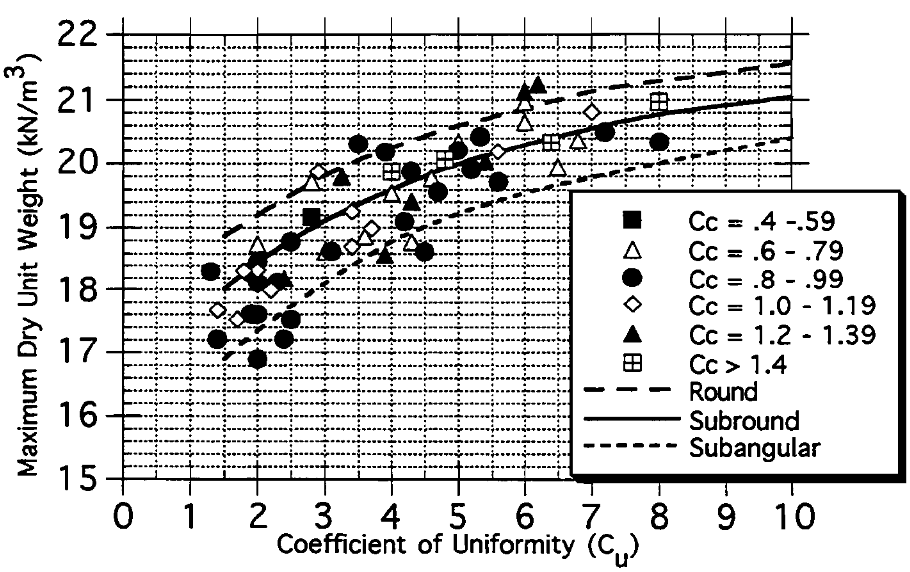


Figure 24. Vibrating Hammer Maximum Dry Unit Weight as a Function of: a) Grading of the Soil; and b) Grading of the Soil and Particle Shape.

Generally, within a particle shape zone (i.e., between the particle shape boundary lines), the maximum dry unit weight increases slightly with increasing values of the coefficient of curvature, while holding the coefficient of uniformity constant. The results support Burmister's (1948) conclusion that the grain size distribution is more important in the compaction of cohesionless sands than the grading of the soils.

6.4 Laboratory Compaction Graphs for the Different Compaction Methods

The preceding results documented the effects of the water content, grain size distribution, grading of the soil, particle shape, and laboratory compaction method. Of these factors, only the grain size distribution, particle shape, and laboratory compaction method strongly influence the compaction of cohesionless sands. The grading of the soil and the optimum water content are secondary contributors.

For practicing engineers to be able to utilize the information contained in this section, they would need graphs or charts summarizing the above information. Figures 25, 26 and 27 summarize the above information in individual figures for each laboratory compaction test method. These figures allow an engineer to estimate the compaction maximum dry unit weight and optimum water content, knowing only the coefficient of uniformity of the soil and the roundness of the grains.

6.5 Summary of Results

The results in this section indicate that the grain size distribution, particle shape, and laboratory compaction test method are important contributors to the compaction of cohesionless sands. However, the effects of these parameters are not constant over the range of tests discussed. Some parameters are important only over certain ranges for certain types of compaction tests. Table 8 shows a summary of the range of importance of the different parameters.

Table 8. Range of Importance of Different Parameters.

Parameter	Method	Range of Importance
Grain Size Distribution	Standard	Entire C_u range
	Modified	Range of $C_u < 5$
	Vibrating Hammer	Entire C_u range
Particle Shape	Standard	Entire C_u range
	Modified	Entire C_u range
	Vibrating Hammer	Entire C_u range
Laboratory Test Method	Standard	Entire C_u range (low)
	Modified	Entire C_u range (low for $C_u < 3.5$)
	Vibrating Hammer	Entire C_u range

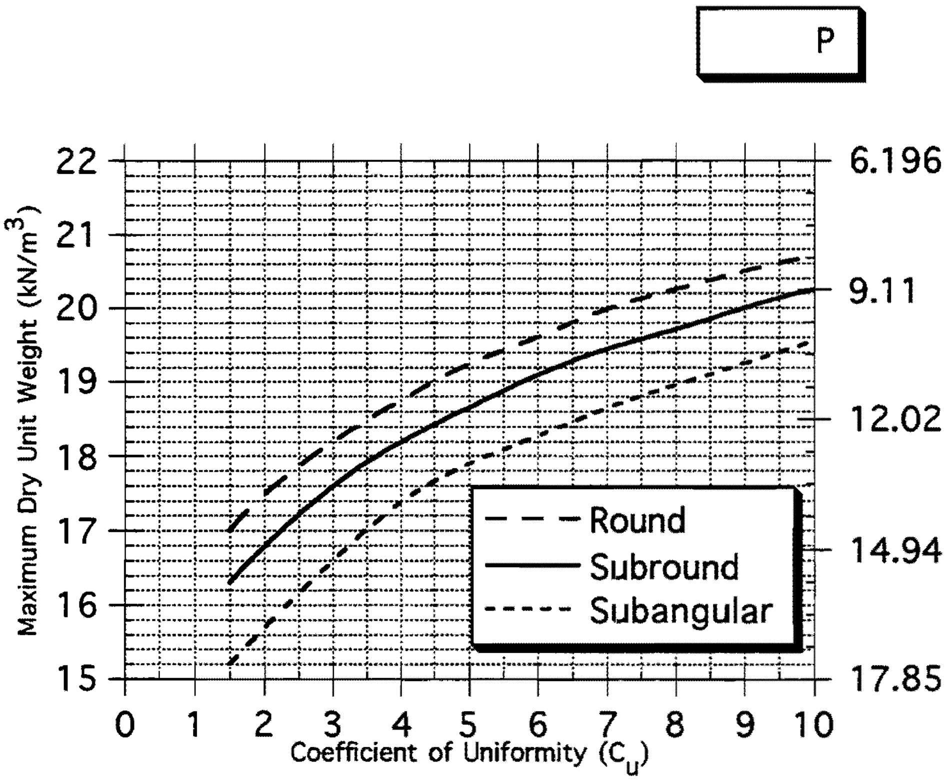


Figure 25. Laboratory Compaction Graph for the Standard Proctor Compaction Test.

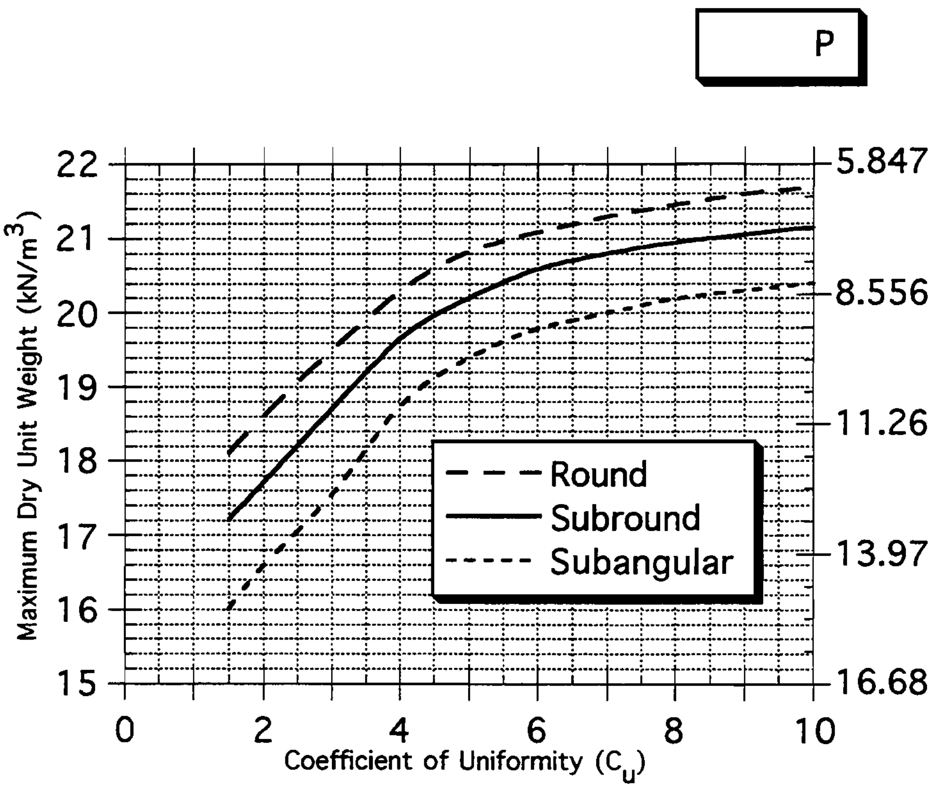


Figure 26. Laboratory Compaction Graph for the Modified Proctor Compaction Test.

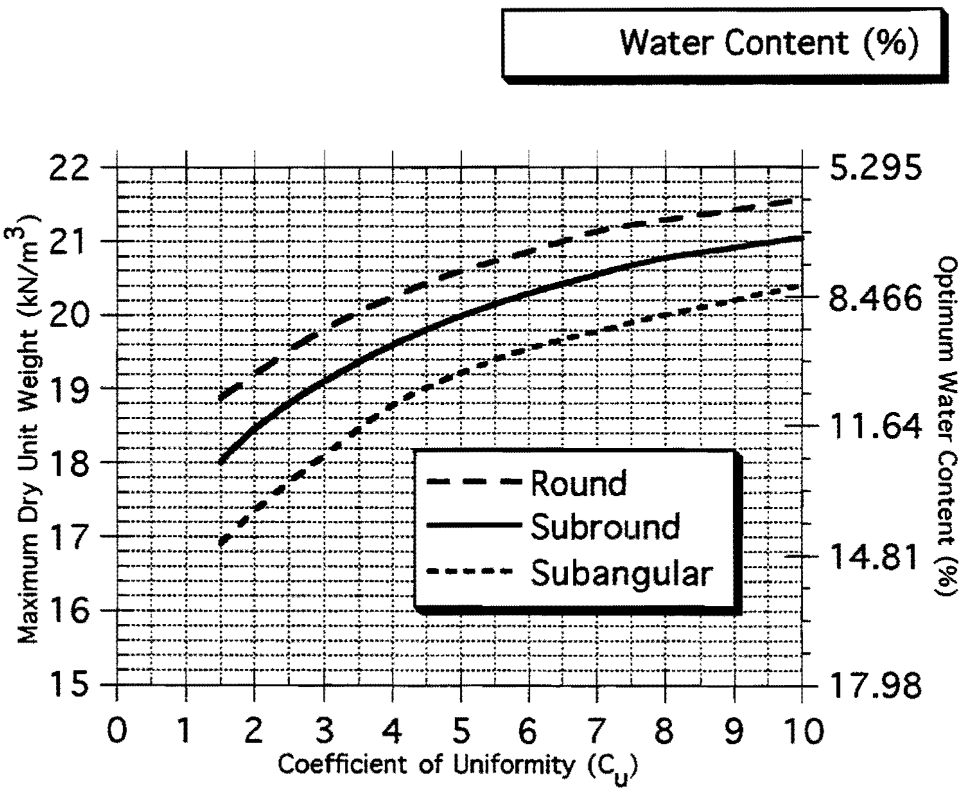


Figure 27. Laboratory Compaction Graph for the Vibrating Hammer Compaction Test.

6.6 Comparison of Results with the Literature

Figure 28 is a comparison of the results shown in Figures 25, 26 and 27 with Figure 4. Figure 28 indicates that the results for the modified Proctor compaction test compare very well with that of Poulos and Hed (1973), while the results for the vibrating hammer compaction test compare favorably with that of Youd (1973). However, neither the modified Proctor, standard Proctor, nor the vibrating hammer results compare with that of Johnston (1973). While the trends are the same, the maximum dry unit weights obtained by Johnston (1973) appear to be very low. A possible reason for this could be that Johnston (1973) used a modified version of the vibrating table compaction test without a surcharge load being applied to the surface of the soil. The surcharge load on the top of the soil acts as confinement on the soil to aid with the compaction process, as without it the soil may not properly densify (Felt, 1958).

6.7 Discussion of the Results

The analyses of the various factors affecting compaction and the laboratory compaction graphs for the different methods should be used by practicing engineers as a guide only and should not replace the actual laboratory compaction testing of the soil. For example, while grain crushing was not found to be important for the materials tested, it presumably would be important for sands comprised of soft calcareous grains. Consequently, the gradation of the material could change during the compaction testing and yield a different maximum dry unit weight than expected.

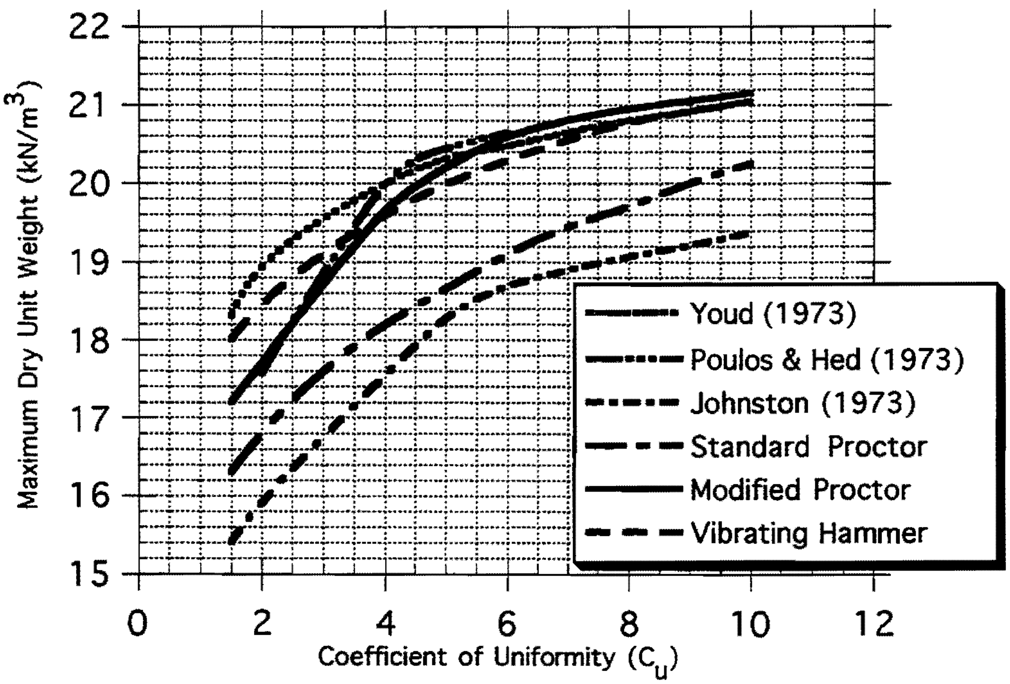


Figure 28. Comparison of Different Compaction Methods for Subround Cohesionless Sands.

For the methods used by Youd (1973), Poulos and Hed (1973), and Johnston (1973), see Figure 4.

7. ESTIMATING SETTLEMENT FROM LABORATORY COMPACTION TESTS

7.1 General

The laboratory graphs presented in Chapter 6 are important because they represent obtainable values of the maximum dry unit weight and optimum water content using the various laboratory compaction methods. Consequently, they can be used to estimate the possible settlement of the compacted soil.

7.2 Estimating Settlement of Cohesionless Fill

Figure 29 illustrates the maximum vertical strain that can be achieved from a one dimensional settlement analysis for various values of the initial relative compaction. The figure can be used to estimate the maximum vertical settlement as follows.

For example a typical specification for field compaction might call for a relative compaction of 95 percent of the modified Proctor dry unit weight. Two cohesionless sands with subround sand grains are available, one with a coefficient of uniformity of two and one with a coefficient of uniformity of four, and it is desirable to estimate the maximum possible settlement of a 2 m (6.56 feet) thick layer of sand after compaction. Figure 30 is a synthesis of Figures 13, 14 and 15 for subround cohesionless sands and shows (for coefficients of uniformity of two and six using the subround curve) maximum dry unit weights of 17.8 kN/m^3 and 20.6 kN/m^3, respectively.

To use Figure 29, the modified Proctor compaction test is identified with the Index Maximum Dry Unit Weight (IDUW). For both sands, the initial relative compaction is 95 percent; however, the final relative compaction for the two sands are different. For the sand with a coefficient of uniformity of two, the highest obtainable maximum dry unit weight is obtained from the vibrating hammer compaction test (i.e., 18.7 kN/m^3), and the final relative compaction is 105 percent (i.e., 18.7 kN/m^3 / 17.8 kN/m^3). For the sand with a coefficient of uniformity of six, the highest obtainable maximum dry unit weight is obtained from the modified Proctor compaction test (i.e., 20.6 kN/m^3), and the final relative compaction is 100 percent (i.e., 20.6 kN/m^3 / 20.6 kN/m^3).

Using these values for the curves represented in Figure 29, the maximum vertical strain is obtained by starting at 95 percent initial relative compaction and proceeding vertically on the graph until 100 percent of IDUW and 105 percent of IDUW is intersected and then reading the value off from the vertical scale. For a sand with C_u = 2, the maximum vertical strain is 9.5

percent, while for a sand with $C_u = 6$, the maximum vertical strain is 5 percent (note: the maximum vertical strain = {1 - (Initial Relative Compaction/Final Relative Compaction)} × 100%).

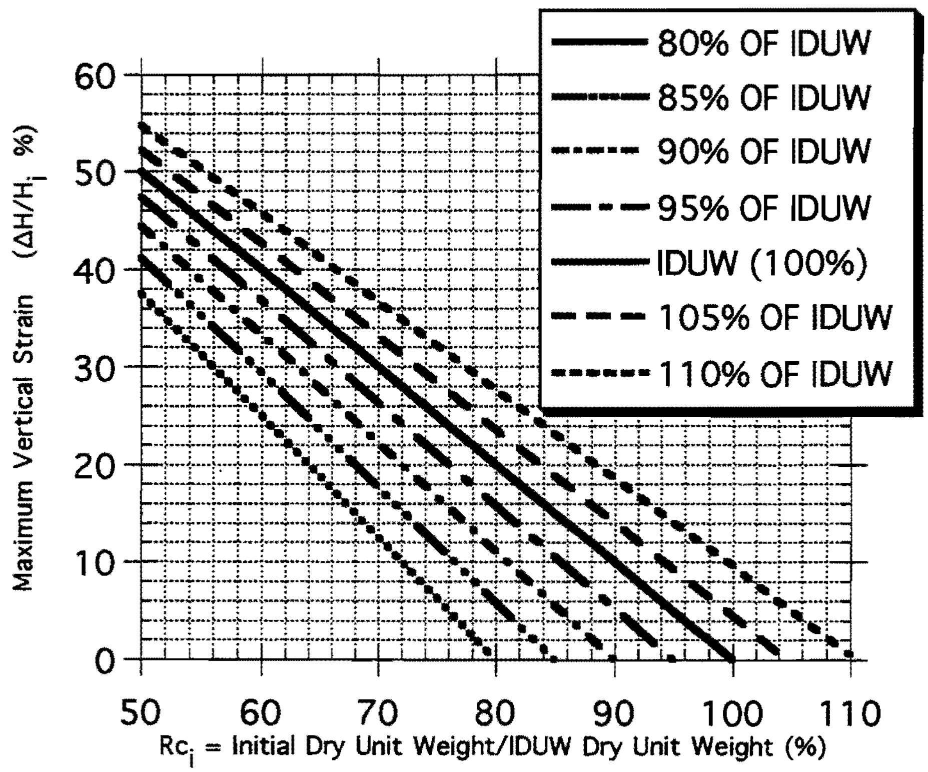


Figure 29. Graph of the Maximum Vertical Settlement Strain as a Function of the Initial Relative Compaction of the Soil.

The curves represent the final relative compaction state.

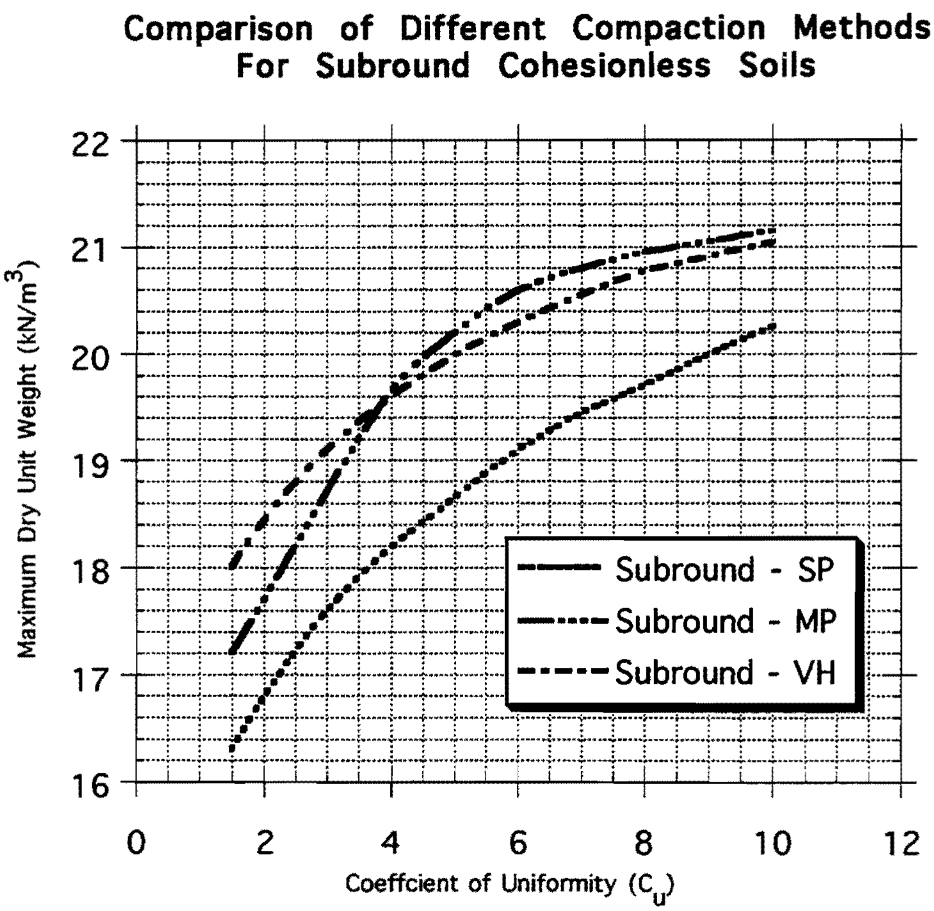


Figure 30. Comparison of Different Compaction Methods Used in This Study for Subround Cohesionless Sands.

SP = standard Proctor, MP = modified Proctor, and VH = vibrating hammer.

Consequently, the sand with $C_u = 2$ could settle 0.19 m (19 cm or 7.48 in), whereas the sand with $C_u = 6$ could settle 0.1 m (10 cm or 3.94 in). These values are upper bounds on the possible settlement that could occur due to vibrations, consolidation upon wetting, etc.

In most cases these values might not be achieved. However, for some cases, these values can be approached. For example, the backfill for a 7.62 m (25 ft) high MSE retaining wall on Highway 358 in Corpus Christi, Texas, was a fine, uniform, subround sand with a coefficient of uniformity of two. This sand was field compacted to 90 percent of the standard Proctor compaction value. After six years, the retaining wall suffered severe distress and had to be repaired. Upon repairing the wall a large void was found behind the uppermost panels (Figure 31) on the order of 0.92 to 1.07 m (3 to 3.5 feet) in size (Dan Stacks, personal communication).

Using this chart, this size of void could have been predicted. For a sand with a coefficient of uniformity of two, the maximum dry unit weight for the standard Proctor compaction test is 16.8 kN/m^3, whereas the highest obtainable value of the maximum dry unit weight obtained from the vibrating hammer compaction test is 18.4 kN/m^3. Identifying the standard Proctor compaction test with the Index Maximum Dry Unit Weight (IDUW), the final relative compaction is 110 percent. Using these values the maximum vertical strain is obtained from Figure 4 as 18.2 percent. The settlement of a 7.62 m (25 ft) high layer of backfill soil is 1.39 m (4.55 ft). This value compares favorably with that observed when the retaining wall was repaired.

The maximum vertical settlement ratio predicted in Figure 29 can be used for any compaction project ranging from highways to foundations for buildings. However, one drawback of the figure is that it may give an engineer the false impression that one should always try to achieve the highest possible dry unit weight for the soil to prevent settlement problems. This would be true if the engineer did not have to worry about grain breakage or lateral constraints on the soil (i.e., retaining walls, foundations, etc.). Compacting the soil to higher and higher dry unit weights can cause significant breakage of grains, changing the grain size distribution of the soil, which would affect other properties such as shear strength and permeability. Additionally, the compaction to higher dry unit weights causes an increase in lateral stresses within the soil, which can cause unwanted movements in retaining walls or foundations.

Consequently, there is a limit to how high the dry unit weight should be, and from the literature, it appears as though the vibrating hammer compaction test results are the most suitable over a wide range of grain size distributions (see references in Parsons, 1992). In fact, the use of the vibrating hammer compaction test results has limited the settlement on some projects (Cross, 1970). It should be noted that for soils with a coefficient of uniformity of greater than about 3.5 to 4, the modified Proctor test is equally applicable.

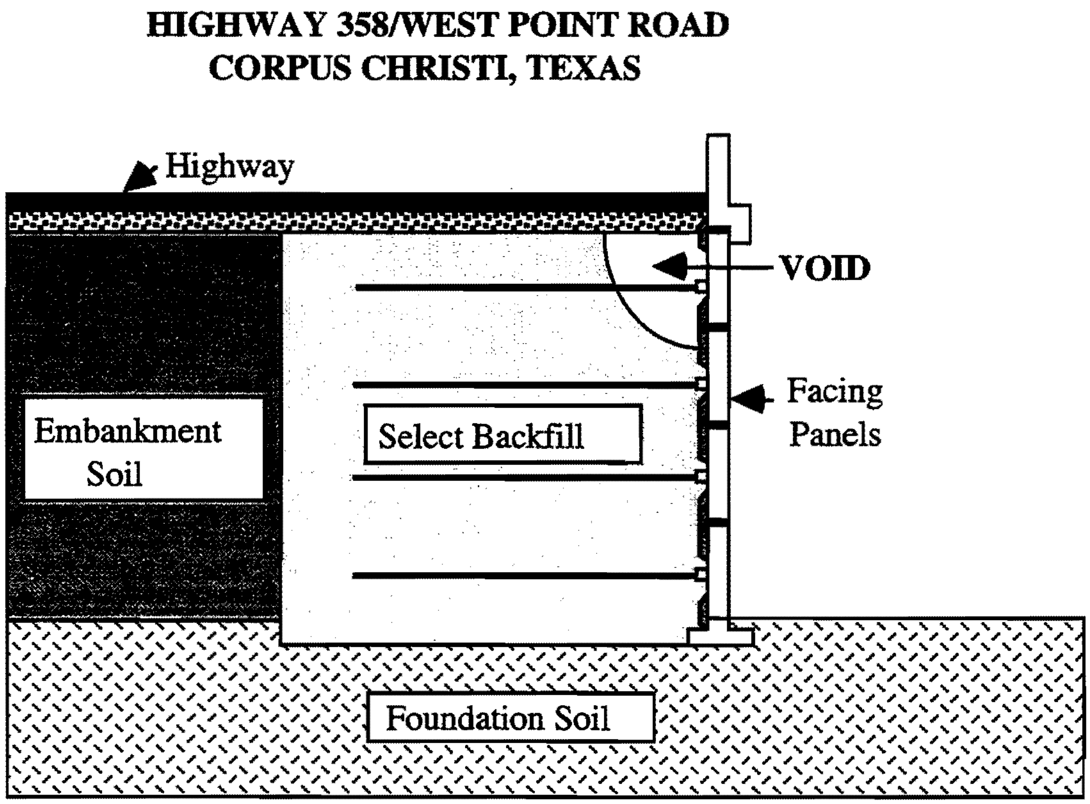


Figure 31. MSE Retaining Wall Cross Section for Corpus Christi, Texas.

The height of the select backfill or cohesionless sand is 7.62 m (25 ft) for some projects (Cross, 1970). It should be noted that for soils with a coefficient of uniformity greater than 3.5 to 4, the modified Proctor compaction test is equally applicable.

7.3 Discussion

Compaction specifications for cohesionless sands usually include the relative compaction and a specified range for the water content. The engineer typically regards these specifications as being sufficient to reduce unwanted settlement. However, the results of this section indicate that this may not be true and have provided a means for estimating the maximum vertical settlement of cohesionless sands using laboratory compaction graphs from different methods. Calculating the maximum vertical settlement prior to construction can provide an engineer with insight into potential problems that could occur and provides a basis for re-evaluating the compaction specifications on the project. This process of evaluating the compaction specifications and estimating the maximum vertical settlement is essential if the long term stability of structures which use cohesionless sands is to be achieved.

8. CONCLUSIONS AND RECOMMENDATIONS

Several conclusions can be drawn from the results presented in this study. They are:

(1) The compaction of cohesionless soils is dependent upon the grain size distribution, particle shape, and laboratory method used for compacting the soil. The results support the previous work of Burmister (1948), Johnson and Sallberg (1962), Dickin (1973), and Youd (1973).

(2) Grain crushing during testing was not important in the compaction of cohesionless sands, presumably because the grains were predominantly quartz.

(3) The grading of the soil was not an important factor in the compaction of the cohesionless sands. This supports the results of Burmister (1948).

(4) The laboratory method used for compacting the soil was important for standard Proctor over the entire range of grain size distributions studied. In fact, the results from the standard Proctor compaction test yield unreasonably low values of maximum dry unit weight, as noted by Felt (1958) and Parsons (1992). The modified Proctor test yields low values of maximum dry unit weight for sands with the coefficient of uniformity less than 3.5. For a coefficient of uniformity greater than 3.5, the modified Proctor test yields results approximately equal to that of the vibrating hammer test.

(5) The most consistent results for a given test method over the entire range of grain size distributions and particle shapes tested was that of the vibrating hammer. In fact, Parsons (1992) has shown that the results of the vibrating hammer compare favorably with that which can be obtained during field compaction.

(6) The graphical representation of the results indicates the relationship between the maximum dry unit weight, laboratory test method, grain size distribution, particle shape, and optimum water content for a given laboratory compaction test method. In general, the maximum dry unit weight of a cohesionless soil increases with increasing coefficient of uniformity, increasing roundness, and decreasing water content. While the graphs display this information, they are not intended to replace actual laboratory compaction testing of cohesionless sands.

(7) A comparison of the results with the laboratory compaction results of naturally occurring Texas sands indicated the graphs are reliable.

(8) The degree of settlement after field compaction was estimated using a one dimensional calculation and relying on the initial and final relative compaction of the soil. The amount of settlement for sands can be quite high if the sand is not compacted properly.

The results of this study suggest that further questions still remain, which include:

a) Determining what effect increasing the percentage of fine material would have on the compaction of sands. For example, would the results of this study change if the percentage of fines was increased from 5 percent to 10 percent or even to 15 percent ?

b) Investigating the cause of the settlement of sands to determine what are some of the most common processes. For example, ground vibrations can lead to settlement as well as saturating the soil with water (i.e., consolidation settlement due to saturation or collapse). Additionally, determine what percentage of the maximum vertical strain each process contributes.

c) Trying to develop a better means of describing the roundness of the grains. Using a microscope and counting grains is time consuming and subject to operator interpretation. A better means might be to develop a laboratory test which would provide an estimate. One may have to search other disciplines to see if any tests exist. This would be useful since it would standardize the process.

d) Investigating the structure of the soil or the soil fabric imparted by the different compaction methods to see what effect it has on the maximum dry unit weight of cohesionless sands

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APPENDIX A

RECOMMENDATIONS FOR REVISED SPECIFICATION FOR BACKFILL MATERIAL FOR MECHANICALLY STABILIZED EARTH RETAINING WALL SYSTEMS

Recommendations for Revised Specifications for Backfill Material for MSE Retaining Wall Systems

Backfill material for MSE retaining wall systems shall be free from organic or otherwise deleterious materials, and shall conform to the following gradation limits as determined by Test Method Tex-110-E:

Type A:

Sieve Size	Percent Passing
3 inches	100
No. 40	0 - 35
No. 100	0 - 10
No. 200	0 - 5

The coefficient of uniformity[1] for Type A backfill shall be greater than or equal to 4. ($C_u \geq 4.$)

Type B:

Sieve Size	Percent Passing
3 inches	100
No. 40	0-50
No. 100	0-20
No. 200	0-10

Type A backfill shall be used unless otherwise specified on the plans.

The backfill shall conform to the following additional requirements:

(1) The plasticity index (P.I.) as determined by Test Method Tex-106-E shall not exceed 6.

(2) Soundness - The material shall be substantially free of shale or other soft, poor durability particles.

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[1] The coefficient of uniformity, C_U, is the ratio of the grain size of 60% passing of the sample to the grain size of 10% passing: $C_U = D_{60}/ D_{10}$

(3) Electrochemical Requirements - The backfill material shall meet the following requirements:

	Requirements	Test Method
a.	pH between 5.5 - 10	Tex-128-E
b.	Resistivity > 3000 ohms.cm	Tex-129-E
c.	If resistivity is 1500-3000 ohms.cm, then the chloride content < 100 ppm and sulphate content <200 ppm	Tex-620-J

(4) MSE wall systems using nonmetallic or epoxy coated metallic reinforcements may use backfill which does not comply with the pH and resistivity measurements. Epoxy coated metallic reinforcements may be used only when shown on the plans or approved by the engineer. All connection hardware used with nonmetallic or epoxy coated reinforcements shall likewise be nonmetallic or epoxy coated.

When nonmetallic or epoxy coated reinforcements are used, the maximum allowable backfill particle size shall be 3/4 inch.

Compaction of the backfill material behind the wall shall conform to the following specifications:

Zone Greater Than 1 m (3 ft) From the face of the Wall

(1) The compaction of the backfill material shall be accomplished in lifts, 20 cm (8 in) thick of loose soil.

(2) The acceptance of the compaction will be based on 9 out of every 10 measurements of in-situ dry density and water content of the backfill meeting the following:
The compacted soil shall be within 2% of the optimum water content on the dry side and 95% of the dry density determined in the laboratory using the vibratory hammer.

(3) The compaction of the backfill material within this zone shall be accomplished without damage or distortion of the reinforcement or the wall facing panels.

Zone Less Than 1 m. (3 ft.) From the Face of the Wall

(1) The compaction of the backfill material shall be accomplished with hand operated vibrating plate compactors or walk behind compaction equipment.

(2) The compaction of the backfill material shall be accomplished in lifts, 20 cm (8 in) thick of loose soil.

(3) The acceptance of the compaction will be based on 9 out of every 10 measurements of in-situ dry density and water content of the backfill meeting the following:

The compacted soil shall be within 2% of the optimum water content on the dry side and 85% of the dry density determined in the laboratory using the vibratory hammer.

(4) The compaction of the backfill material within this zone shall be accomplished without damage or distortion of the reinforcement or the wall facing panels.

APPENDIX B

RECOMMENDATIONS FOR MATERIAL TESTING OF BACKFILL FOR MSE RETAINING WALL SYSTEMS

The backfill material used in Mechanically Stabilized Earth retaining walls is one of the key components in the stability of the walls. Therefore, proper identification and testing of the backfill is very important. The following is a list of the tests to be performed on the backfill material. These tests are considered to be standard tests for all backfill materials. Where possible the procedures for the tests have been referenced to the Texas Department of Transportation (TxDOT) Tests and where modifications are needed reference has been made to the American Society for Testing and Materials (ASTM) manual, Section 4, Volume 4.08, on Soil and Rock; Dimension Stone; Geosynthetics.

The following are required tests for the backfill soil:

1) Soil Sampling
2) Determination of Particle Size Analysis of Soils
3) Determination of Liquid Limit, Plastic Limit and Plasticity Index of Soils
4) Classification of Soils
5) Determination of Soil pH and Resistivity
6) Laboratory Compaction Testing
7) Determination of the In-Place Density of Soils

The following are optional tests to aid in identifying problem backfill soils:

8) Determination of the Shear Strength of Soils
9) Determination of Permeability of Soils
10) Determination of Collapse Potential of Soils
11) Determination of Specific Gravity of Soils

REQUIRED TESTING OF BACKFILL FOR MECHANICALLY STABILIZED EARTH RETAINING WALLS

1) SOIL SAMPLING

The results of the various tests performed are dependent upon the soil sampled and the sampling method being used. Commonly a single soil sample is taken from a stockpile designated by the contractor, up to a year in advance of the soil being used for backfill. This practice often leads to the following problems: 1) Sampling form a stockpile might lead to a misrepresentation of the gradation of the soil, because segregation of the soil occurs in stockpiles; 2) The soil tested only represents a small fraction of the soil used for backfill. The soil could change gradation and properties, thereby changing its mechanical behavior, altering the stability of the wall. To avoid these problems, the engineer should require. that the soil being tested is taken from a test pit rather than from a stockpile. This would then give the engineer a chance to get a feel for the extent of the soil and any possible gradational changes in the soil within the test pit. Consequently then, more than one sample would probably have to be taken from the test pit to obtain a proper representation of the soil and to determine the uniformity of the soil.

TEST METHOD: Tex-100-E Surveying and Sampling Soils for Highways

MODIFICATION:

1) Samples should be taken from a test pit rather than from a stockpile, as discussed above.

2) If sampling has to be done from a stockpile, then follow the guidelines put forth in the ASTM test method D 75 - Appendix X1 - section X1.2, page 70, Standard Practice for Sampling Aggregates (see following page).

The following is taken from:
American Society for Testing and Materials (ASTM) Manual, Section 4, Volume 4.08, on Soil and Rock; Dimension Stone; Geosynthetics. ASTM D 75 - Appendix X1 - section X1.2, page 70, Standard Practice for Sampling Aggregates (1996).

Section X1.2.1 "In sampling material from stockpiles it is very difficult to ensure unbiased samples, due to the segregation which often occurs when material is stockpiled, with coarser particles rolling to the outside base of the pile. For coarse or mixed coarse and fine aggregate, every effort should be made to develop a separate, small sampling pile composed of materials drawn from various levels and locations in the main pile after which several increments may be combined to compose the field sample. If necessary to indicate the degree of variability existing within the main pile, separate samples should be drawn from separate areas of the pile."

Section X1.2.2 "Where power equipment is not available, samples from stockpiles should be made up of at least three increments taken from the top third, at the mid point, and at the bottom third of the volume of the pile. A board shoved vertically into the pile just above the sampling point aids in preventing further segregation. In sampling stockpiles of fine aggregate, the outer layer, which may have become segregated, should be removed and the sample taken from the beneath material. Sampling tubes approximately 30 mm (1 1/4 in.) min by 2 m (6 ft) min in length may be inserted into the pile at random locations to extract a minimum of five increments of material to form the sample."

2) DETERMINATION OF PARTICLE SIZE ANALYSIS OF SOILS

The sampled soil is tested for particle size analysis to determine if it is suitable for use as a backfill material. These tests are fairly routine and reliable. However, because the present backfill requirements rely only on the percent passing the following sieves: namely the 3 in. & the numbers 40 and 200, not all districts record the entire grain size distribution. In fact, a common procedure is to record only the percent passing the number 40 and 200 sieves. This procedure leads to problems, because the engineer can not determine if the soil tested is the same as that used in the field and changes in gradation lead to changes in mechanical behavior of the soil. To eliminate this problem, the specifications for the backfill material have been altered to include a wide range of sieves, rather than a limited number. Consequently, the entire grain size distribution of the soil should be recorded and kept for future reference.

TEST METHOD: Tex-101-E, Part I-A Preparation of Soil and Flexible Base Materials for Testing

TEST METHOD: Tex-110-E Determination of Particle Size Analysis of Soils

MODIFICATION:

1) Include all of the following sieves: 3 in, 1 1/2 in, 3/4 in, 3/8 in, No. 4, No. 10, No. 20, No. 40, No. 80, No. 200.
2) Record and report percent passing for all of the sieves specified.
3) Plot the grain size distribution curve on Form 481.
4) Calculate and record the coefficient of uniformity for the soil, defined as: $C_u = D_{60} / D_{10}$

where:

C_u = Coefficient of Uniformity

D_{60} = Particle size diameter corresponding to 60% passing the cumulative particle size distribution curve

D_{10} = Particle size diameter corresponding to 10% passing the cumulative particle size distribution curve.

Note: Definition of coefficient of uniformity is from ASTM method D 2487, Section 12.3, Classification of Soils for Engineering Purposes

3) DETERMINATION OF LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX OF SOILS

The liquid limit, plastic limit and plasticity index of soils are used to help identify soils and to correlate soils with similar engineering behavior, such as permeability, shear strength and compressibility.

TEST METHOD: Tex-101-E, Part I-B Preparation of Soil and Flexible Base Materials for Testing

TEST METHOD: Tex-103-E Determination of Moisture Content in Soil Materials

TEST METHOD: Tex-104-E, Part I-A Determination of Liquid Limit of Soils

TEST METHOD: Tex-105-E Determination of Plastic Limit of Soils

TEST METHOD: Tex-106-E Method of Calculating the Plasticity Index of Soils

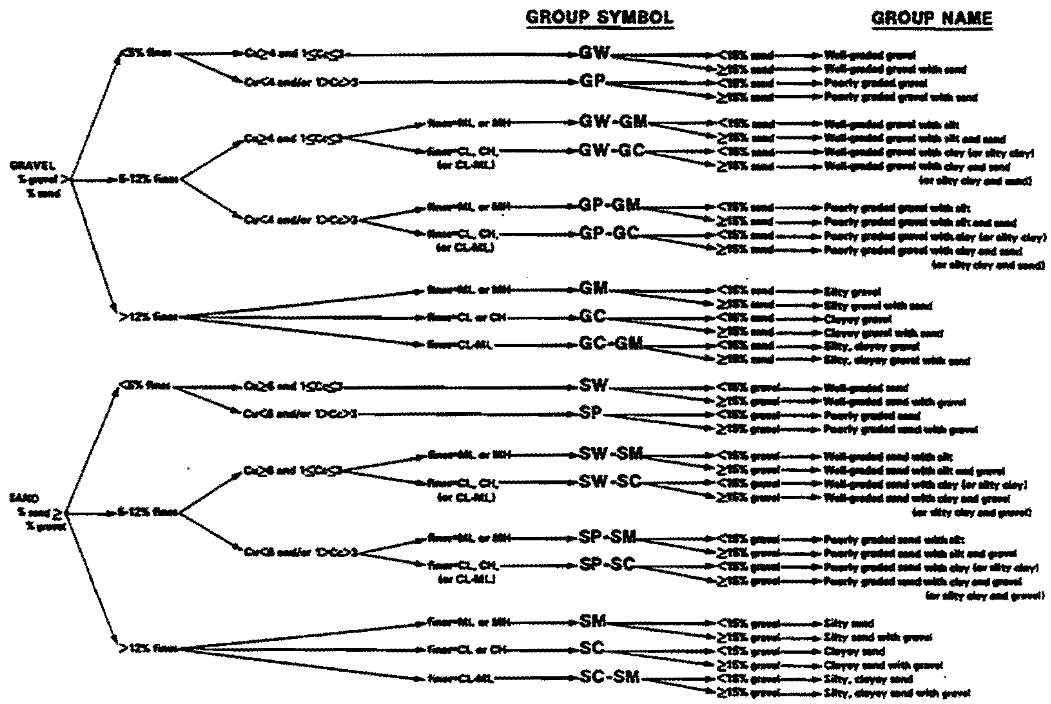
4) CLASSIFICATION OF SOILS

Through the use of the particle size analysis, liquid limit and plasticity index, the classification of the soil can be obtained, which correlates and identifies the engineering behavior of the soil.

TEST METHOD: ASTM D 2487 Classification of Soils for Engineering Purposes

TEST METHOD: ASTM D 3282 Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes

The following is adapted from:
American Society for Testing and Materials (ASTM) Manual, 1996,
Section 4, Volume 4.08, on Soil and Rock; Dimension Stone; Geosynthetics.
ASTM method D 2487, Section 4, Figure 2, Classification of Soils for Engineering Purposes.



The following is taken from:
American Society for Testing and Materials (ASTM) Manual, 1996,
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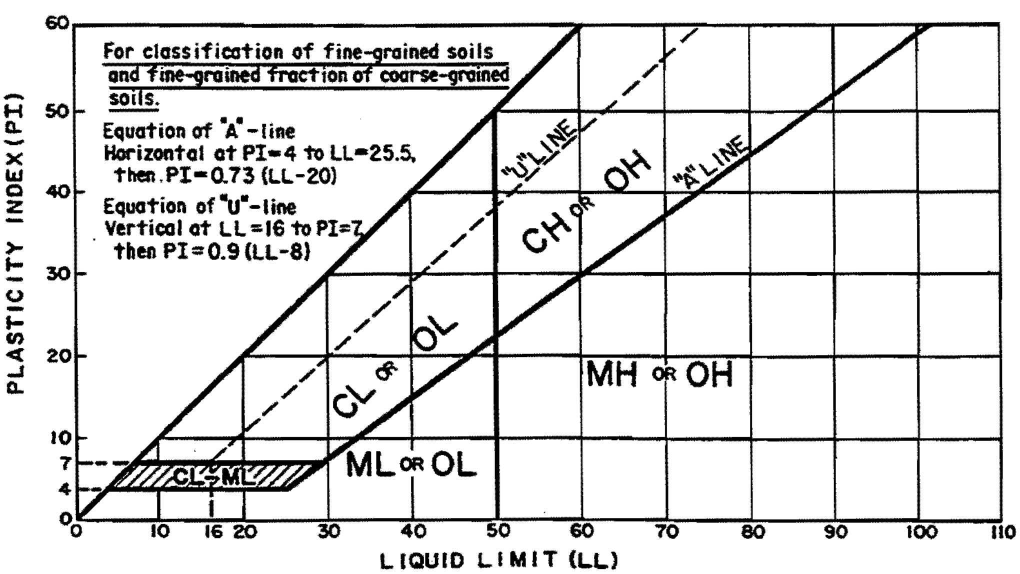
ASTM method D 2487, Section 11, Figure 3, Classification of Soils for Engineering Purposes.



The following is taken from:
American Society for Testing and Materials (ASTM) Manual, 1996, Section 4, Volume 4.08, on Soil and Rock; Dimension Stone; Geosynthetics. ASTM method D 3282, Section 9, Tables 1 and 2, Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes.

General Classification	Granular Materials (35 % or less passing No. 200)			Silt-Clay Materials (More than 35 % passing No. 200)			
Group Classification	A-1	A-3[A]	A-2	A-4	A-5	A-6	A-7
Sieve analysis, % passing:							
No. 10 (2.00 mm)	...	...	...	...	...	...	...
No. 40 (425 μm)	50 max	51 min	...	...	...	...	...
No. 200 (75 μm)	25 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 (425 μm):							
Liquid limit	...	...	[B]	40 max	41 min	40 max	41 min
Plasticity index	6 max	N.P.	[B]	10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to Good			Fair to Poor			

[A] The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.
[B] See Table 2 for values.
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TABLE 2 Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 % or less passing No. 200)							Silt-Clay Materials (More than 35 % passing No. 200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, % passing:											
No. 10 (2.00 mm)	50 max	...	...	...	...	...	...	...	...	...	...
No. 40 (425 μm)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
No. 200 (75 μm)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 (425 μm):											
Liquid limit	...		...	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[A]
Usual types of significant constituent materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General rating as subgrade	Excellent to Good							Fair to Poor			

[A] Plasticity index of A-7-5 subgroup is equal to or less than *LL* minus 30. Plasticity index of A-7-6 subgroup is greater than *LL* minus 30 (see Fig. 1).
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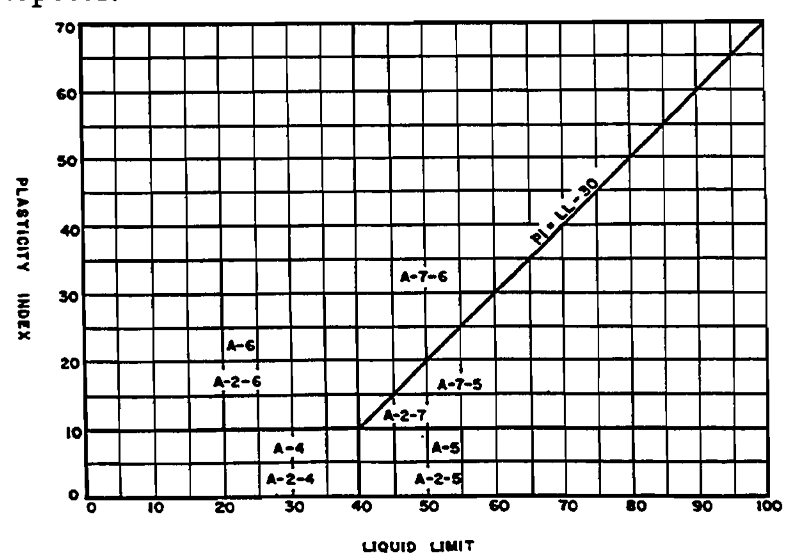
The following is taken from:
American Society for Testing and Materials (ASTM) Manual, 1996, Section 4, Volume 4.08, on Soil and Rock; Dimension Stone; Geosynthetics. ASTM method D 3282, Section 9, Figure 1, Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes.



NOTE—A-2 soils contain less than 35 % finer than 200 sieve.

5) DETERMINATION OF SOIL pH and RESISTIVITY

The choice of backfill soils for use with mechanically stabilized earth retaining walls must not alter the long term stability of the wall. Therefore, important properties of the soil is that it be relatively non-corrosive with respect to the reinforcing elements of the wall. Consequently, it is imperative to determine the pH and resistivity of the soil.

TEST METHOD: Tex-128-E Determination of Soil pH

TEST METHOD: Tex-129-E Method of Test for the Resistivity of Soils Material

6) LABORATORY COMPACTION TESTING

Laboratory compaction tests on the soil designated for use as backfill are performed, because they are incorporated into the specifications for field compaction of the backfill. Therefore, it is important that the laboratory tests for compaction be performed correctly and on various samples of soil to determine a relationship between gradational changes and compaction characteristics.

The TxDOT laboratory compaction test for granular soils is Test 113-E. This test is an impact compaction test, which utilizes a disk to cover the top of the soil in an attempt to achieve more energy being imparted into the soil. However, problems with this test occur, which are as follows: 1) The energy used to compact the soil in Tex-113-E is not known and therefore, the test can not be used as a guide for method specifications for field compaction; 2) Tex-113-E is essentially equivalent to the Standard Proctor compaction test, both of which yield relatively low values of maximum dry density or dry unit weight, especially on fine, uniform sands; 3) These values are often so low that contractors claim that they can pour the soil out of a dump truck and obtain nearly 80% of the value obtained by these two tests; and 4) This means that the contractors have to minimally compact the soil to achieve the required specifications based on Tex-113-E and Standard Proctor tests. Due to these problems with Test 113-E, the Vibrating Hammer Compaction Test will be used to determine the laboratory dry density - moisture content relationship of the soil. It is worth noting that the Vibrating Hammer compaction test is used as the British Standard for compacting granular soils and was developed because impact compaction test methods could not simulate the field compaction of the soil.

TEST METHOD: Tex-101-E, Part II (Preparation of Soil and Flexible Base Materials for Testing)

TEST METHOD: Tex-103-E (Determination of Moisture Content in Soil Materials)

TEST METHOD: British Standard 1377: 1990, British Standards Institute, Gr 10, Part 4 - Soil Compaction Tests, Section 3 - Determination of the Dry Density/Moisture Content Relationship for Granular Soils, Subsection 3.7 - Method Using the Vibrating Hammer.
(details of this follow)

The following is taken from:
British Standard 1377: 1990, British Standards Institute, Gr 10, Part 4 - Soil Compaction Tests, Section 3 - Determination of the Dry Density/Moisture Content Relationship for Granular Soils, Subsection 3.7 - Method Using the Vibrating Hammer.

3.7 Method using vibrating hammer

3.7.1 *General.* This test covers the determination of the dry density of soil, which may contain some particles up to coarse gravel size, when it is compacted by vibration in a specified manner over a range of moisture contents. The range includes the optimum moisture content at which the maximum dry density for the specified degree of compaction is obtained. In this test the soil is compacted into a CBR mould using an electrically operated vibrating hammer.

The test is suitable for certain soils containing no more than 30 % by mass of material retained on the 20 mm test sieve, which may include some particles retained on the 37.5 mm test sieve. It is not generally suitable for cohesive soils.

The requirements of Part 1 of this standard, where appropriate, shall apply to this test method.

3.7.2 *Apparatus*

3.7.2.1 *A cylindrical, corrosion-resistant metal mould,* i.e. the CBR mould, as described in 7.2.2.2.

3.7.2.2 *An electric vibrating hammer* having a power consumption between 600 W and 750 W and operating at a frequency between 25 Hz to 45 Hz.

NOTE. For safety reasons the vibrating hammer should operate on 110 V, and an earth leakage circuit breaker should be included between the hammer and the mains supply.

3.7.2.3 *A steel tamper* for attachment to the vibrating hammer. Essential dimensions are shown in figure 7(b), which also indicates one suitable design of tamper.

3.7.2.4 *Supporting guide frame* for vibrating hammer (optional).

3.7.2.5 *A depth gauge or steel rule,* or other device which enables the sample depth to be measured to an accuracy of 0.5 mm.

3.7.2.6 *A balance* readable to 5 g.

3.7.2.7 *A straightedge,* e.g. a steel strip about 300 mm long, 25 mm wide, and 3 mm thick, with one bevelled edge.

3.7.2.8 *Test sieves,* of aperture sizes 37.5 mm and 20 mm, and receiver.

3.7.2.9 *A corrosion-resistant metal or plastics tray* with sides, e.g. about 80 mm deep, of a size suitable for the quantity of material to be used.

3.7.2.10 *A scoop.*

3.7.2.11 *Apparatus for the determination of moisture content* as described in 3.2 of BS 1377 : Part 2 : 1990.

3.7.2.12 *A stopclock* readable to 1 s.

3.7.2.13 *Apparatus for extracting compacted specimens from the mould* (optional).

3.7.3 *Calibration of vibrating hammer*

3.7.3.1 *General.* The vibrating hammer shall be maintained in accordance with the manufacturer's instructions. Its working parts shall not be badly worn.

The calibration test described in 3.7.3.3 shall be carried out to determine whether the vibrating hammer is in satisfactory working order, and able to comply with the requirements of the test described in 3.7.5.

The pressure check described in 3.7.3.4 shall be made by the operator carrying out the calibration test.

3.7.3.2 *Material.* Clean, dry, silica sand, from the Woburn Beds of the Lower Greensand in the Leighton Buzzard district*. The grading shall be such that 100 % passes a 600 μm test sieve and 100 % is retained on a 63 μm test sieve. The sand shall be free from flaky particles, silt, clay and organic matter.

3.7.3.3 *Calibration test*

3.7.3.3.1 Take a 5 $\pm$ 0.1 kg sample of the sand specified in 3.7.3.2, which has not been used previously, and mix it with water in order to raise its moisture content to 2.5 $\pm$ 0.5 %.

3.7.3.3.2 Compact the wet sand in a cylindrical metal mould of 152 mm diameter and 127 mm depth, using the vibrating hammer as specified in 3.7.5.1.

NOTE. The operator can usually judge the required pressure to apply with sufficient accuracy after first carrying out the check described in 3.7.3.4.

3.7.3.3.3 Carry out a total of three tests, all on the same sample of sand, and determine the mean dry density. Determine the dry density values to the nearest 0.002 Mg/m^3.

3.7.3.3.4 If the range of values in the three tests exceeds 0.01 Mg/m^3, repeat the procedure. Consider the vibrating hammer suitable for use in the vibrating compaction test if the mean dry density of the sand exceeds 1.74 Mg/m^3.

3.7.3.4 *Pressure check.* Apply pressure combined with vibration to ensure the required degree of compaction. A downward force on the sample surface of 300 N to 400 N shall be applied, this being greater than the force needed to prevent the hammer bouncing on the soil.

The required pressure shall be assessed by applying the vibrating hammer, without vibration, to a platform scale. The required force is applied when a mass of 30 kg to 40 kg is indicated.

3.7.4 *Preparation of sample.* Prepare the test sample as described in 3.2.5.1, 3.2.5.2, 3.2.5.3, 3.2.7.1, 3.2.7.2 or 3.2.7.3 as appropriate.

3.7.5 *Procedure*

3.7.5.1 *Compaction procedure for soil particles not susceptible to crushing*

3.7.5.1.1 Weigh the mould, with baseplate attached, to 5 g (m_1).

Measure the internal dimensions to 0.5 mm.

3.7.5.1.2 Attach the extension to the mould and place the mould assembly on a solid base, e.g. a concrete floor or plinth.

3.7.5.1.3 Place a quantity of moist soil in the mould such that when compacted it occupies a little over one-third of the height of the mould body.

3.7.5.1.4 Place the circular tamper on the soil and compact with the vibrating hammer for 60 ± 2 s. During this period apply a steady downward force on the hammer so that the total downward force on the sample, including that from the mass of the hammer, is between 300 N and 400 N. (See note to 3.7.3.3.2).

NOTE. A disc of polyethylene sheet may be placed immediately beneath the tamper plate to prevent sand particles moving up through the annular gap.

3.7.5.1.5 Repeat 3.7.5.1.3 and 3.7.5.1.4 twice more.

3.7.5.1.6 Remove any loose material lying on the surface of the sample around the sides of the mould.

3.7.5.1.7 Lay a straightedge across the top of the extension collar and measure down to the surface of the sample to an accuracy of 0.5 mm. Take readings at four points spaced evenly over the surface of the sample, all at least 15 mm from the side of the mould. Calculate the mean height, h (in mm), of the sample. If the sample is less than 127 mm or more than 133 mm in height, reject it and repeat the test from 3.7.5.1.3 until a sample of the required height is obtained.

3.7.5.1.8 Weigh the soil and mould with baseplate to 5 g (in m_2).

3.7.5.1.9 Remove the compacted soil from the mould and place it on the metal tray. Take a representative sample of the soil for determination of its moisture content as described in 3.2 of BS 1377 : Part 2 : 1990.

3.7.5.1.10 Break up the remainder of the soil, rub it through the 20 mm or the 37.5 mm test sieve and mix with the remainder of the prepared test sample.

3.7.5.1.11 Add a suitable increment of water and mix thoroughly into the soil.

NOTE. The water added for each stage of the test should be such that a range of moisture contents is obtained which includes the optimum moisture content. In general, increments of 1 % to 2 % are suitable for sandy and gravelly soils. To increase the accuracy of the test it is often advisable to reduce the increments of water in the region of the optimum moisture content.

3.7.5.1.12 Repeat 3.7.5.1.3 to 3.7.5.1.11 to give a total of at least five determinations. The moisture contents shall be such that the optimum moisture content, at which the maximum dry density occurs, lies near the middle of the range.

3.7.5.2 *Compaction procedure for soil particle susceptible to crushing*

NOTE. The soil should be considered susceptible to crushing during compaction if the sample contains granular material of a soft nature, e.g. soft limestone, sandstone, etc., which is reduced in size by the action of the vibrating hammer. The procedure described in 3.7.5.2 for soils susceptible to crushing during compaction can be applied to all granular soils if it is convenient to do so.

3.7.5.2.1 Weigh, measure and prepare the CBR mould as described in 3.7.5.1.1 and 3.7.5.1.2.

3.7.5.2.2 Carry out a compaction test on each of the prepared samples in turn as described in 3.7.5.1.3 to 3.7.5.1.9.

3.7.5.2.3 Discard the remainder of each compacted sample.

3.7.6 *Calculations, plotting and expression of results* (see form 4.B, appendix A)

3.7.6.1 Calculate the bulk density, ρ (in Mg/m^3), of each compacted specimen from the equation

$$\rho = \left(\frac{m_2 - m_1}{Ah}\right) 1000$$

where

m_1 is the mass of mould and baseplate (in g);

m_2 is the mass of mould, baseplate and compacted soil (in g);

h is the height of the compacted sample (in mm);

A is the circular area of the mould (in mm^2).

3.7.6.2 Calculate the dry density, ρ_d (in Mg/m^3), of each compacted specimen from the equation

$$\rho_d = \frac{100\rho}{100 + w}$$

where

w is the moisture content of the soil (in %).

3.7.6.3 Plot the dry densities obtained from a series of determinations as ordinates against the corresponding moisture contents as abscissae. Draw a curve of best fit to the plotted points and identify the position of the maximum on this curve. Read off the values of dry density and moisture content, to three significant figures, corresponding to that point. (See figure 6.)

NOTE. The maximum may lie between two observed points but, when drawing the curve, care should be taken not to exaggerate its peak.

3.7.6.4 On the same graph, plot the curves corresponding to 0 %, 5 % and 10 % air voids, calculated from the equation

$$\rho_d = \frac{1 - \dfrac{V_a}{100}}{\dfrac{1}{\rho_s} + \dfrac{w}{100\,\rho_w}}$$

where

ρ_d is the dry density (in Mg/m^3);

ρ_s is the particle density (in Mg/m^3);

ρ_w is the density of water (in Mg/m^3), assumed equal to 1;

V_a is the volume of air voids in the soil, expressed as a percentage of the total volume of the soil (equal to 0 %, 5 %, 10 % for the purpose of this plot);

w is the moisture content (in %);

(See figure 6.)

3.7.7 *Test report.* The test report shall affirm that the test was carried out in accordance with this Part of this standard and shall contain the following information:

(a) the method of test used;

(b) the sample preparation procedure, and whether a single sample or separate samples were used;

(c) the experimental points and the smooth curve drawn through them showing the relationship between moisture content and dry density;

(d) the dry density corresponding to the maximum dry density on the moisture content/dry density curve reported as the maximum dry density to the nearest 0.01 (in Mg/m^3);

(e) the percentage moisture content corresponding to the maximum dry density on the moisture content/dry density curve reported as the optimum moisture content to two significant figures;

(f) the amount of stone retained on the 37.5 mm test sieve reported to the nearest 1 % by dry mass;

(g) the particle density and whether measured (and if so the method used) or assumed;

(h) the information required by 9.1 of BS 1377 : Part 1 : 1990.

7) DETERMINATION OF THE IN-PLACE DENSITY OF SOILS

Most specifications for field compaction of backfill are based on the relative compaction of the soil, which is the ratio of the in-place dry density to the laboratory determined dry density. Consequently, it is imperative to determine the in-place dry density accurately from the various methods that are available.

TEST METHOD: Tex-115-E Field Method For Determination of In-Place Density of Soils and Base Materials

MODIFICATION:

1) Regardless of which method for determining the in-place density is used, each method should be calibrated to a known standard prior to use.

2) Recalling that cohesionless soils are sensitive to volume changes due to shearing, the method to be used should then be tested on a laboratory compacted soil samples to generate a calibration curve for that soil. This is done by plotting the test method dry density versus the actual laboratory measured dry density for various water contents.

3) The soil calibration curve can then be an aid in determining the relative compaction of the soil backfill.

OPTIONAL TESTING OF BACKFILL FOR MECHANICALLY STABILIZED EARTH RETAINING WALLS

The following tests aid in identifying problem backfill soils.

8) DETERMINATION OF THE SHEAR STRENGTH OF SOILS

The shear strength of the backfill is important because it plays an key role in the stability of the retaining wall. However, when a cohesionless soil is compacted to a sufficiently high dry density the shear strength of the soil is also high. Consequently, the shear strength of the backfill soil is commonly not measured, but it can indicate problem soils.

TEST METHOD: Tex-117-E Triaxial Compression Tests for Disturbed Soils and Base Materials

9) DETERMINATION OF PERMEABILITY OF SOILS

The permeability of the backfill soil is not necessary for free draining cohesionless soils, but becomes important as the percentage of fines increases. It can indicate if pore water pressures acting on the retaining wall will be important.

TEST METHOD: ASTM D 2434 Permeability of Granular Soils (Constant Head)

MODIFICATION: Calculate and record the permeability of the soil as follows:

$$\text{Permeability} = D_f / A$$

where

D_f = Drainage Factor

and

A = Cross-sectional area of the specimen

10) DETERMINATION OF COLLAPSE POTENTIAL OF SOILS

Soils with low values of dry density and water contents can undergo significant settlement upon wetting or being saturated, which is known as collapsing soils. Collapsing backfill soils can lead to differential settlement problems behind the retaining wall, which can lead to stability problems. Consequently, it is important to determine which soils will settle upon saturation from laboratory tests.

TEST METHOD: ASTM D 5333 Measurement of Collapse Potential of Soils

11) SPECIFIC GRAVITY TEST OF SOILS

The specific gravity of the soil is important in determining the weight-volume relationships of the soil and various engineering parameters, such as the void ratio, porosity and the percentage of the air voids.

TEST METHOD: Tex-101-E Part I Preparation of Soil and Flexible Base Materials for Testing

TEST METHOD: Tex-108-E Determination of Specific Gravity of Soils

MODIFICATION:
1) Weigh out 100 - 200 g of air dried soil from Tex-101-E
2) The soil should be representative of the total soil sample.
3) The specific gravity can be used to calculate the appropriate parameters through the weight-volume relationships of the soil.

APPENDIX C

FIELD INSPECTOR'S MANUAL
FOR
MSE RETAINING WALL SYSTEMS

PART I. PRECONSTRUCTION PREPARATION

Chapter 1. Definition and Highway Applications of Mechanically Stabilized Earth Retaining Wall Systems

Introduction

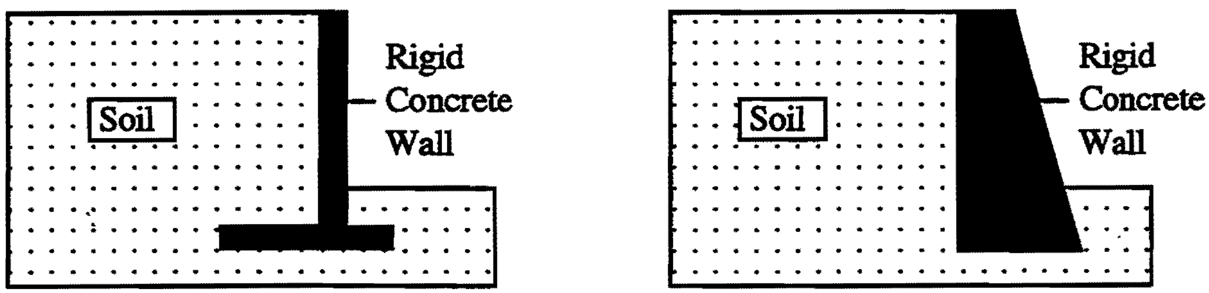
During the last 20 years, there has been significant advances in earth retention systems throughout the world. Consequently, in practice and in the literature different terminology and definitions exist for the different earth retention systems. Therefore, the purpose of this chapter is to provide a basic understanding of what a Mechanically Stabilized Earth Retaining Wall System is, along with discussing its advantages and disadvantages, as well as some of its uses for highways applications .

Concept of Mechanically Stabilizing Soil

The most abundant and typically the least expensive construction material available is the local soil. However, many soils are inherently weak, which limits their use for structural applications. For example, walking across a dry sandy beach illustrates that the sand is weak. Consequently, in order for soils to be used for structural applications in retaining wall systems a method of increasing the strength of the soil would have to be found. Two possible means of increasing the strength of the soil are to laterally confine the soil and to add inclusions to the soil. Lateral confinement is the principle used in conventional retaining wall systems, whereas adding inclusions to the soil in the form of reinforcements is the principle used in Mechanically Stabilizing Earth Retaining Wall System (Figure 1.1). The addition of reinforcements to the soil produces a composite material, like reinforced concrete, which combines the best load carrying features of both components.

The concept of mechanically stabilized soil is best illustrated in Figures 1.2a and 1.2b, which are from Mitchell and Villet (1987). Figure 1.2a shows the maximum slope that can be obtained by dry sand, which is approximately 32^0. Figure 1.2b shows the same dry sand, reinforced horizontally by strips of paper which are used as reinforcements. In this case a vertical slope of the sand could be obtained. It should be noted that the paper facing indicated in Figure 1.2b is only required to keep the sand confined between the reinforcement strips from running out. Several important principles are gleaned from this simple example of mechanically

Conventional Retaining Wall System

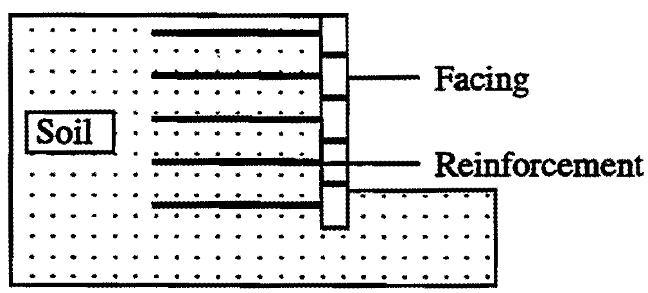


Cantilever Retaining Wall

Gravity Retaining Wall

Note: The rigid concrete wall acts to confine the soil and resists the forces acting on it.

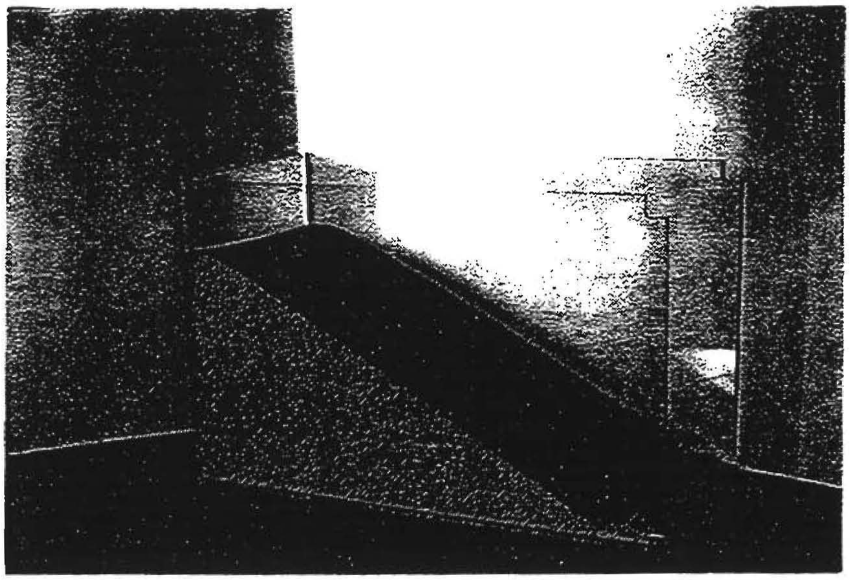
Mechanically Stabilized Earth Retaining Wall System



Note: The reinforcements within the soil strengthen the soil and reduces the forces acting on the wall face.

Figure 1.1. Differences between Conventional and Mechanically Stabilized Earth Retaining Wall Systems.

a)



b)

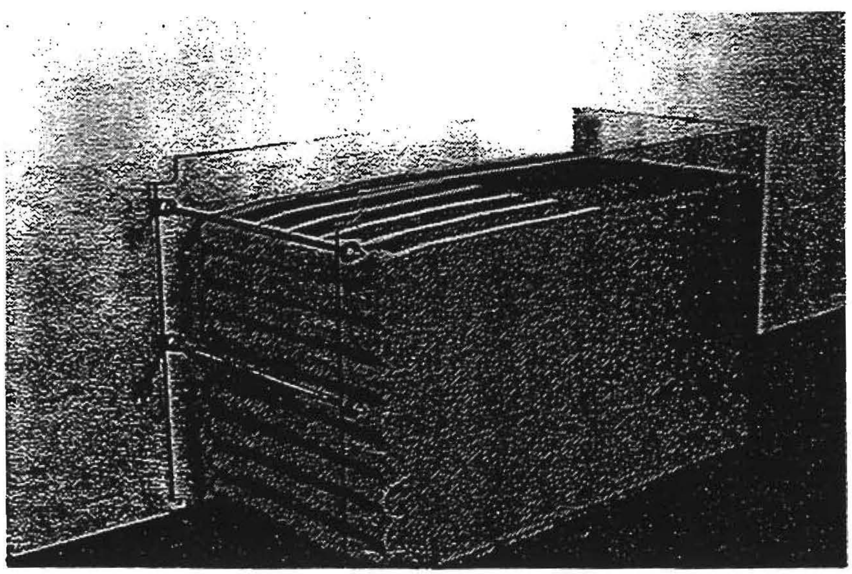


Figure 1.2 - Concept of mechanically stabilized soil from Mitchell and Villet (1987). a) Unreinforced sand mound. b) Mechanically stabilized sand using paper reinforcements.

Table 1 - 1

EARTH RETENTION SYSTEMS FOR RETAINING WALLS

EXTERNALLY STABILIZED SYSTEMS

- **In-Situ Walls**
  - timber
  - precast concrete
  - sheet piles
  - soldier piles
  - cast in-situ
    - -slurry wall
    - -secant pile
    - -tangent pile
  - bored in-place
  - soil cement
  - **Braced**
    - cross-lot
    - takers
  - **Tied-Back**
    - augered
    - belled
      pressure
      injected
- **Gravity Walls**
  - masonry
  - concrete
  - catilever
  - counterfort
  - gabion
  - crib
  - bin
  - cellular
    cofferdam

INTERNALLY STABILIZED SYSTEMS

- **Reinforced Soils**
  - **mechanically stabilized earth**
  - anchored earth
- **In-situ Reinforcement**
  - soil nailing
  - reticulated
    micro-piles
  - soil dowelling

Hybrid Systems

- tailed gibbons
- tailed masonry

Special Materials

- polymer impregnated soil
- low density fills
  - -expanded low density concrete and cement
  - -expanded polystyrene

(Modified from O'Rourke and Jones, 1990) stabilized soil: 1) The reinforcements add strength to the soil; 2) The wall facing does not assume a major load carrying capacity and is there to retain the soil between the reinforcements; and 3) The wall facing would, in principle, not be required if the reinforcements could be placed between each layer of soil grains, however, this is impractical. Therefore, mechanically stabilized earth represents a means of utilizing readily available soil, using reinforcements to increase the soils strength and reducing the amount of concrete needed for the retaining wall.

Definition of a Mechanically Stabilized Earth Retaining Wall System

Mechanical stabilization of soil is defined as the inclusion of reinforcing elements in a soil mass to improve its mechanical properties, whereas a retaining wall is a structure which provides vertical, or near vertical, grade separation at the ground level. However, this does not truly define what a Mechanically Stabilized Earth Retaining Wall System is. Consequently throughout the literature and across the world many different definitions exist.

The classification scheme for earth retention systems for retaining walls in Table 1-1 provides a method to help distinguish and define what a Mechanically Stabilized Earth Retaining Wall System is. The table separates the earth retention systems into two categories of externally or internally stabilized systems. The externally stabilized system utilizes a structural barrier or wall, which provides both weight and structural support for the stabilization of the forces acting on them. The internally stabilized system involves the use of horizontal reinforcements within the soil mass behind the wall to a distance beyond the potential failure plane(s) to stabilize the soil. Within this system two different types of reinforcements within the soil mass can be used: 1) A reinforced soil (Figure 1.3a), where the reinforcement is in the form of metallic or non-metallic grids or strips. The reinforcements and the backfill soil are installed incrementally behind the wall face, starting at the bottom and proceeding to the top. The reinforcements are attached to the wall face and may be anchored on the end away from the wall; and 2) In-situ reinforcement (Figure 1.3b), where soil nails, micro-piles and soil doweling are used for reinforcements. The reinforcements are installed incrementally behind the wall face into the native soil as construction proceeds starting from the top of the wall and working downward. In both cases, the facing is only required to prevent local raveling and

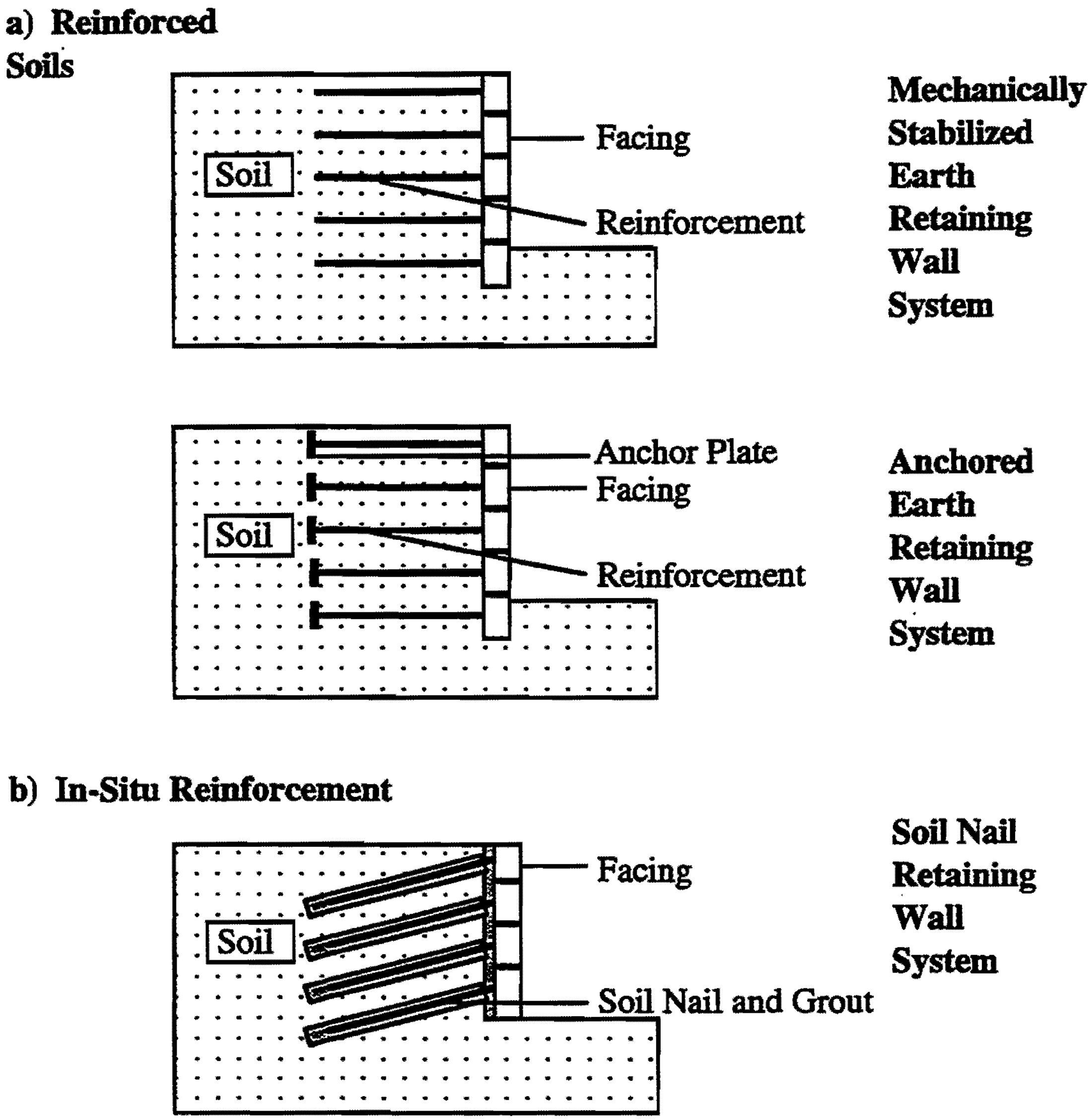


Figure 1.3. Examples of Reinforced Soil and In-Situ Reinforcement Retaining Wall Systems.

deterioration rather than to provide primary structural support, as is the case for externally stabilized soils.

A direct consequence of the classification scheme presented in Table 1-1 is that a fundamental definition of what a Mechanically Stabilized Earth Retaining Wall System is can be defined.

DEFINITION*: A Mechanically Stabilized Earth Retaining Wall System is a structural composite system, consisting of a wall face and a reinforced soil mass behind the wall, which provides vertical, or near vertical, grade separation at the ground surface. The reinforced soil mass utilizes horizontal reinforcements, which are attached to the wall face and are in the form of metallic or non-metallic grids or strips, to stabilize and strengthen the soil. The wall facing is not used for primary structural support, rather it is used to prevent local raveling and deterioration of the soil. Both the wall face and the reinforced soil mass are built incrementally, starting at the bottom and proceeding to the top.

Advantages and Disadvantages of Using Mechanically Stabilized Earth Retaining Wall Systems

Mechanically Stabilized Earth Retaining Wall Systems offer the following advantages over the conventional wall systems (i.e. the gravity and in-situ wall systems - see Table 1-1) used in highway applications:

* **Considerable Savings in Cost**
  Cost savings are on the order of 20 - 60 % versus conventional walls, because locally available soil can be used for backfill. The walls also reduce the right-of-way acquisition, because of their incremental and vertical construction. Construction is also simplified.
* **Reduction in Manufactured Materials**
  The walls use precast concrete facing elements and reinforcements, which are relatively simple to use and are fairly inexpensive. These manufactured materials can also be installed under a wide range of conditions (except during rain), whereas under the same conditions the pouring of concrete can be difficult at best.

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* The definition of a Mechanically Stabilized Earth Retaining Wall System could equally aply to an Anchored Earth Retaining Wall System. In fact, some people (Mitchell and Villet, 1987) have considered them as equals, but this manual considers the two as separate because of the two different roles the reinforcements play in each system (see Figure 1.3 and discussion in Mitchell and Villet, 1987).

* **Ease of Construction**
  The use of the prefabricated facing, reinforcing elements and backfill soil make these walls easy to construct and construction proceeds fairly rapidly. These walls do not require any large or specialized equipment, which results in a reduced working space in front of the wall. The workers do not need any specialized skills for construction. Additionally, the walls are well suited to modern day construction conditions and methods.
* **High Load Carrying Capacity**
  Since the backfill soil is reinforced to increase its strength, the walls are capable of sustaining higher static and dynamic loads.
* **Flexibility**
  Requires little site preparation, because the walls are capable of absorbing deformations due to compressible foundation soils.
* **Aesthetically Pleasing Appearance**
  The precast concrete facing elements can be made into various shapes and with various textures, thereby making the walls architecturally pleasing.

Mechanically Stabilized Earth Retaining Wall Systems have a few disadvantages, which include:

* **Requires a Large Space Behind the Wall**
  A large space is required behind the wall face, due to the placement of the reinforcements in the backfill.
* **The Backfill Soil Must be a Granular Soil**
  The backfill soil must be a granular soil to achieve adequate drainage behind the wall and for the optimum interaction between the backfill and the reinforcements to achieve the increased strength. In areas where obtaining a granular soil is difficult, importing a suitable granular soil may significantly reduce the competitive cost of these walls.
* **Newer Reinforcements Have Not Been Adequately Tested**
  As technology advances, so does the material being used for the reinforcements. At present, some of the newer plastic reinforcements that are being put into practice have not been adequately tested and features like long term creep could affect the stability of the wall. Until such time as these newer materials are tested, both in the lab and in actual field use, they must be used with caution.

Highway Applications of Mechanically Stabilized Earth Retaining Wall Systems

Mechanically Stabilized Earth Retaining Wall Systems are a cost effective alternative to conventional walls for nearly all applications, especially for highway applications. Some common highway applications of Mechanically Stabilized Earth Retaining Wall Systems include:

1) Construction of roadways in steep sided terrain, where slope stability might be a problem.
2) Construction of roadways in areas where the foundation soil(s) is poor.
3) Construction of roadways in areas where the right-of-way is restricted.
4) Construction of approach ramps and bridge abutments. Different types of bridge abutments include: a) Beam seat abutments; b) Exterior pier abutments; and c) Interior pier abutments.

Examples of the above applications are illustrated in Figures 1.4 through 1.6.

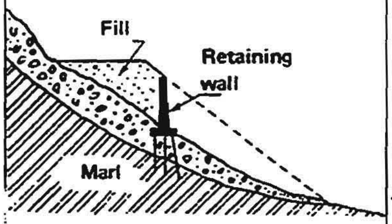
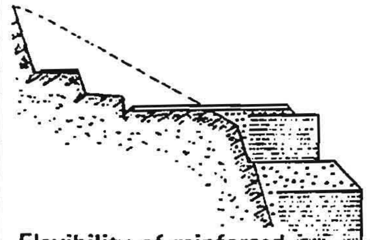
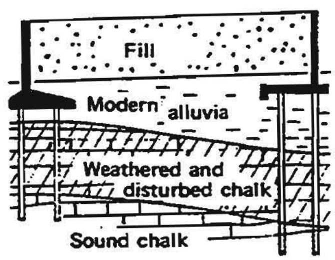
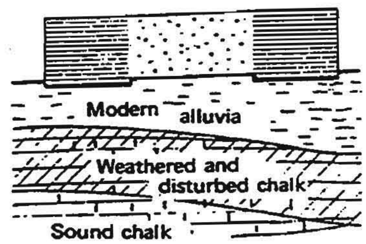
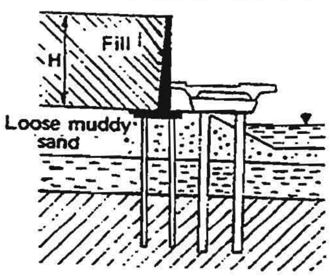
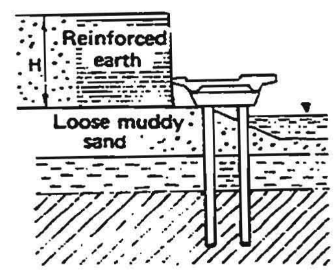
TYPE	Situation of structure	Conventional solution	Reinforced earth solution	Photograph of structure
RETAINING WALLS IN MOUNTAINOUS COUNTRY	Highway A 8 (Roquebrune-Vigna II · 1968 Menton)	 - Unstable scree - Deep foundations necessary brut risk of shearing of piles	 - Flexibility of reinforced earth in relation to movement of scree - No deep foundations Economy about 30 %	
URBAN RETAINING WALLS	RN 187 - F 18 Connection with Pont de Sèvres	 - Subgrade of weak bearing capacity - Deep foundations necessary	 - No deep foundations - Economy 45 %	
	Sète Interchange (CD 2)	 - Subgrade of weak bearing capacity - Deep foundations necessary	 - No deep foundations - Economy about 50 %	

Figure 1.4 Examples of the use of Mechanically Stabilized Earth Retaining Wall Systems for highway applications in mountainous and urban areas (from Terra Armada, 1976).

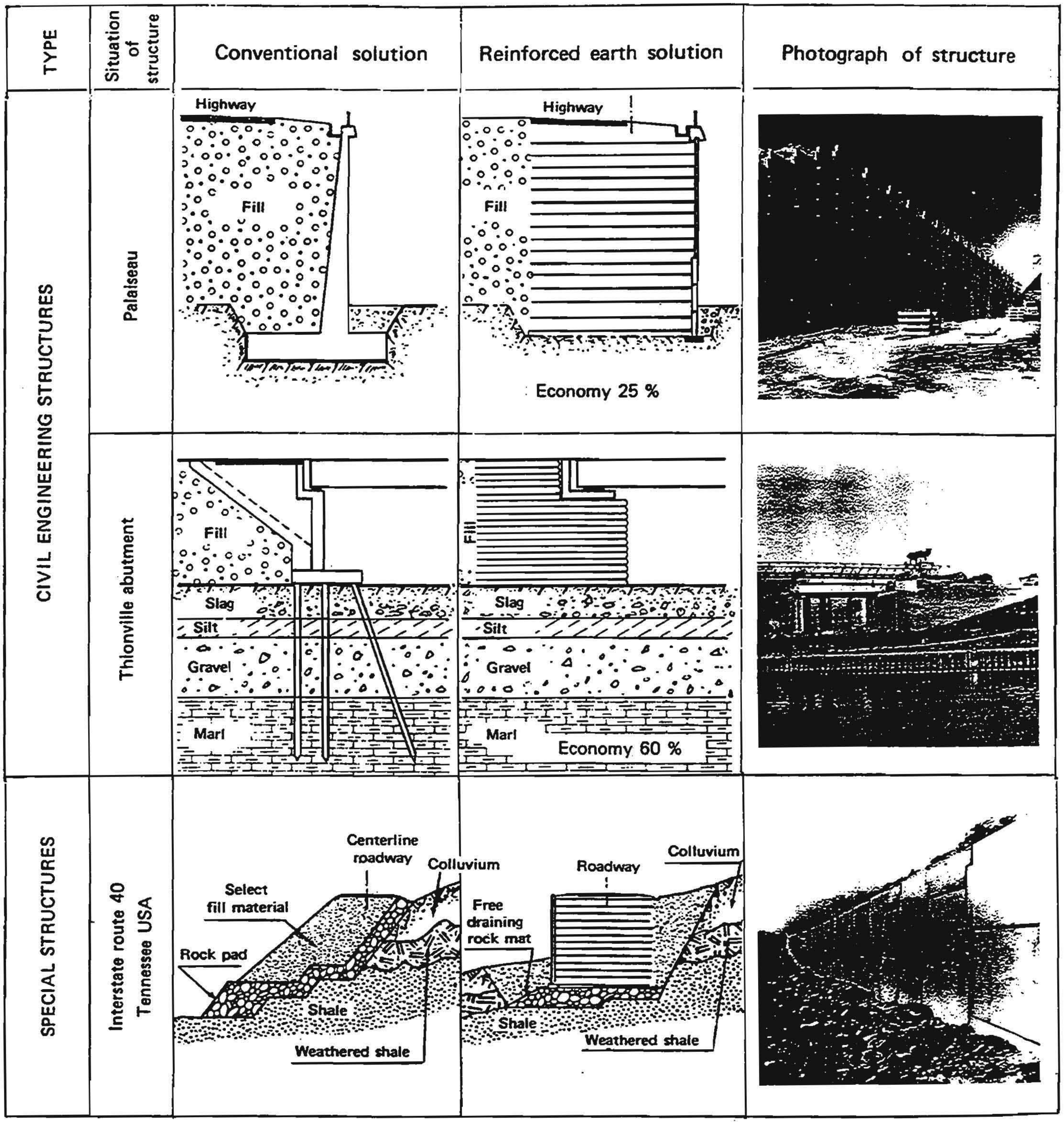


Figure 1.5 Examples of the use of Mechanically Stabilized Earth Retaining Wall Systems for highway applications in civil engineering structures and difficult areas (from Terra Armada, 1976).

a)

b)

c)

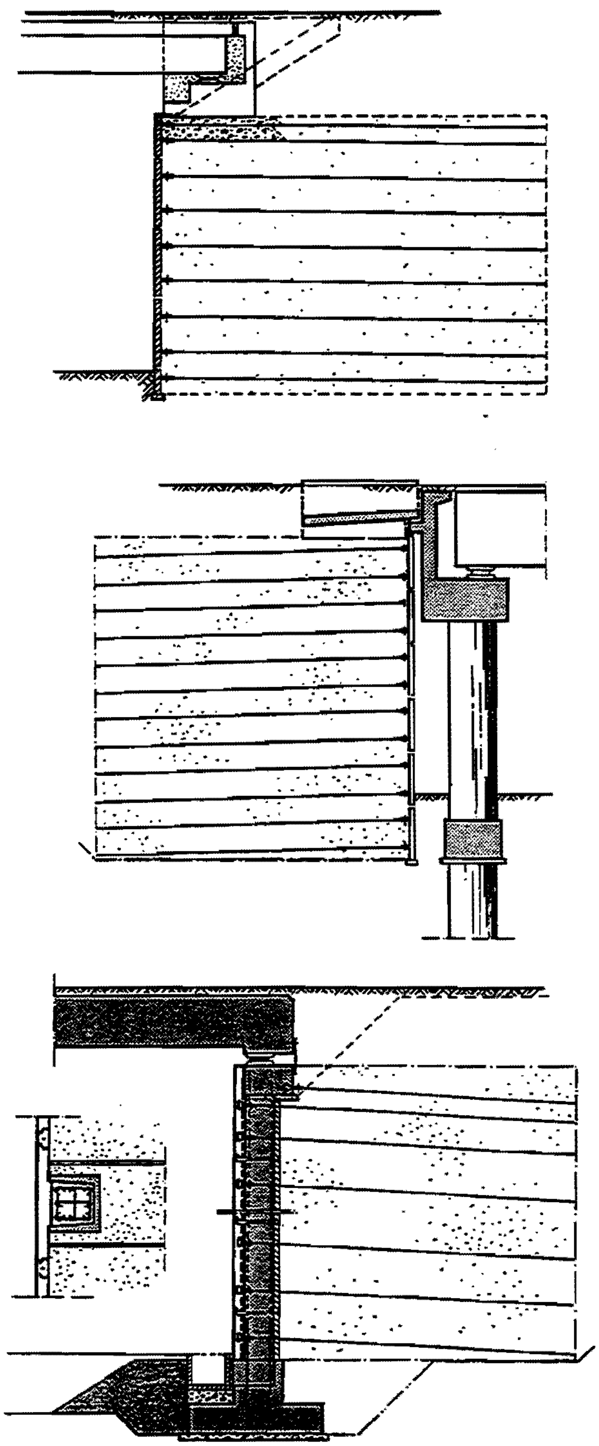


Figure 1.6 Examples of the use of Mechanically Stabilized Earth Retaining Wall Systems for bridge abutments in highway applications (adapted from Jones, 1996). a) Bridge abutment. b) Bridge abutment with an exterior pier supporting the deck. c) Bridge abutment with an interior pier supporting the deck.

PART I. PRECONSTRUCTION PREPARATION

Chapter 2. Understand the Elements of Mechanically Stabilized Earth Retaining Wall Systems

Introduction

There are five essential elements of a Mechanically Stabilized Earth Retaining Wall System. These are the foundation soil* , backfill soil, soil reinforcement, reinforcement connections and facing elements or panels. Figure 2.1 illustrates these elements and their relative proportions within a mechanically stabilized earth retaining wall. Consequently, a knowledge of these elements provides the inspector with important information on the function of each element, which will aid in understanding the construction and stability of the walls. This chapter discusses each of the elements individually.

Foundation Soil

The foundation soil has the primary role of providing support for the retaining wall structure, such that shear failure of the soil (i.e. plastic flow and/or lateral movement of the soil from beneath the wall and backfill) and excessive settlements do not occur under the loads imposed by the retaining wall, backfill and associated structures (i.e. a bridge and/or highway). To achieve these requirements, the foundation soil should meet the following: 1) The soil should be free of deleterious materials, which could break down over time or with the addition of water; 2) The bearing capacity of the soil should support the total load imposed by the retaining wall and associated structures; and 3) The compressibility of the soil should be low to moderate, to limit or reduce excessive settlement of the retaining wall structure. To determine if the foundation soil meets these requirements the inspector should check the geotechnical site investigation report and test the soil. The inspector should consult with a geotechnical engineer if they have any questions (See Part II, Chapter 1).

While most foundation soils meet these requirements, however, some soils do not and soil improvement measures need to undertaken. The

* While the foundation soil is not strictly part of a Mechanically Stabilized Earth Retaining Wall System, it is included here as such because of its importance in both the construction and behavior of the retaining wall.

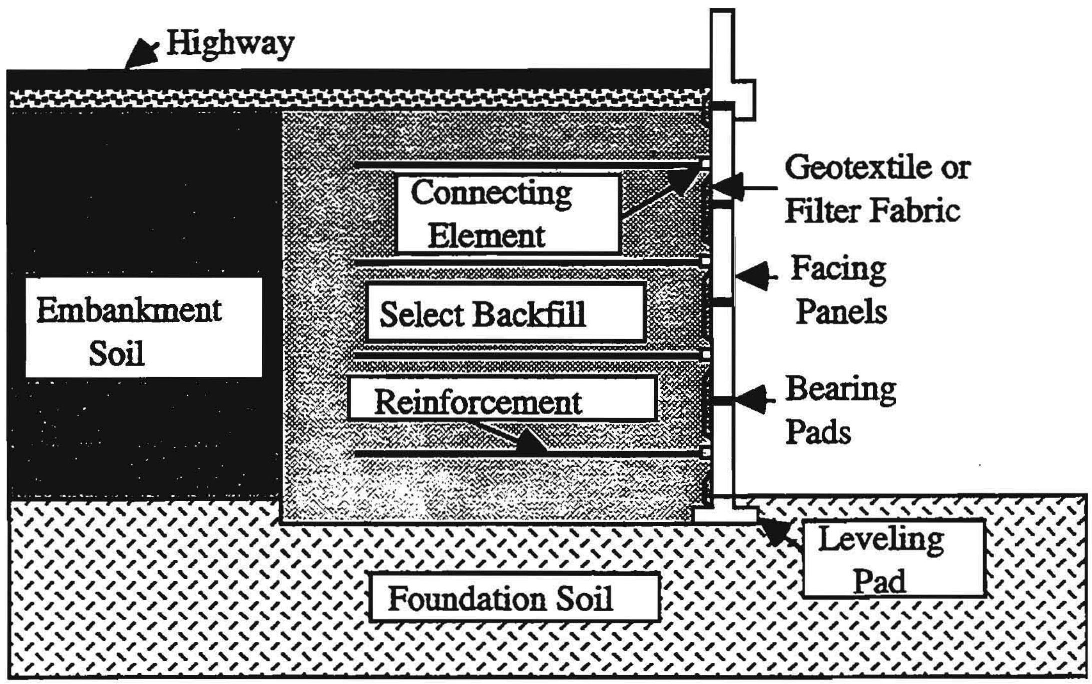


Figure 2.1. Basic elements of a Mechanically Stabilized
Earth Retaining Wall System.

three most often used soil improvement methods are: 1) Substitution -- This involves the removal and replacing of the foundation soil, usually to a depth of a few meters (i.e. 12 - 16 feet), with a good, compacted fill (Figure 2.2a); 2) Pilings are Placed in the Foundation Soil -- This method utilizes either concrete or stone piles in the foundation soil to help distribute the load of the retaining wall structure through the inadequate soil to a deeper and better quality soil (Figure 2.2b); and 3) Preloading of the foundation soil -- This involves the addition of a load onto the soil, in the area where the retaining wall structure is to be built, to aid in the reduction or elimination of settlements and to increase the shear strength of the foundation soil. Two primary methods of preloading are used, which are preloading with an ordinary fill and combined preloading. Preloading with an ordinary fill involves loading the site with a temporary ordinary fill prior to the retaining wall structure being built to help improve the foundation soil (Figure 2.3a), whereas combined preloading involves two types of loading, one from the construction of the Mechanically Stabilized Earth Retaining Wall System and one from the addition of fill on top of the retaining wall structure to help improve the foundation soil (Figure 2.3b). Of the two methods, preloading with an ordinary fill is the most often used method, because not all retaining wall structures can undergo moderately large settlements without causing damage to the structure.

When a Mechanically Stabilized Earth Retaining Wall System is constructed, the stiffness (i.e. elastic stiffness) of the foundation soil plays an important role in determining the behavior of the structure. For example, the stiffness of the backfill soil and the reinforcements behind the wall face is fairly high, due to compaction of the backfill and the interaction of the two elements. However, the foundation soil could be of equal, greater or less stiffness. The stiffness of the foundation soil relative to the stiffness of the retaining wall structure determines the elastic settlements and a portion of the wall rotation. For example, in the ideal case where the foundation soil and the retaining wall structure were of the same stiffness wall rotation would be minimized and elastic settlements would be small. This is not the case for most retaining wall structures, since the foundation soil is typically different from that of the retaining wall structure. For a foundation soil which is stiffer than that of the retaining wall structure the face of the wall will tilt outwards and the elastic settlements will be negligible, whereas if the foundation soil is softer than that of the retaining wall structure the face of the wall will tilt inwards and the elastic settlements may not be small. Figure 2.4 summarizes these affects. While most Mechanically Stabilized Earth Retaining Wall Systems are designed for the case where the foundation soil is stiffer than the retaining wall structure, which is the same as for

a)

b)

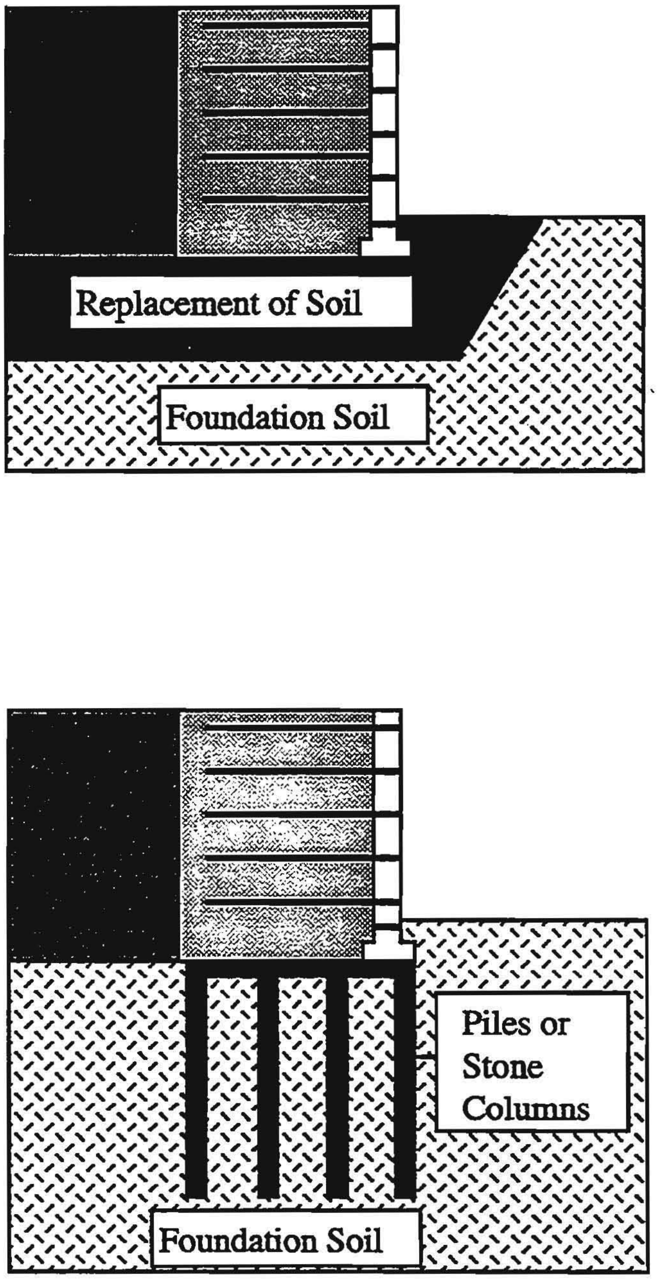
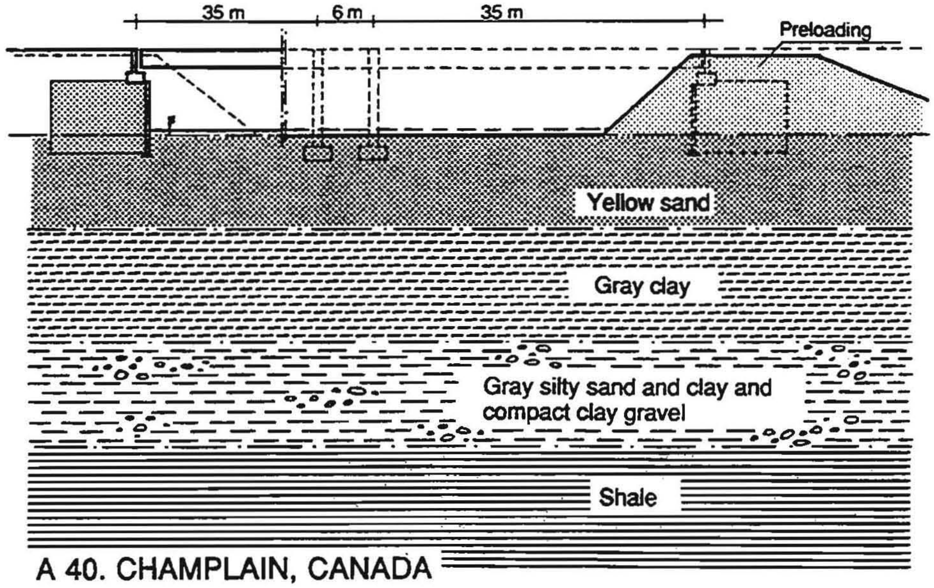


Figure 2.2. Two methods of improving the foundation soil: a) Soil Replacement; and b) Placing Pilings in the Foundation Soil. Both methods help reduce settlement and increase the shear strength of the foundation soil.

a)



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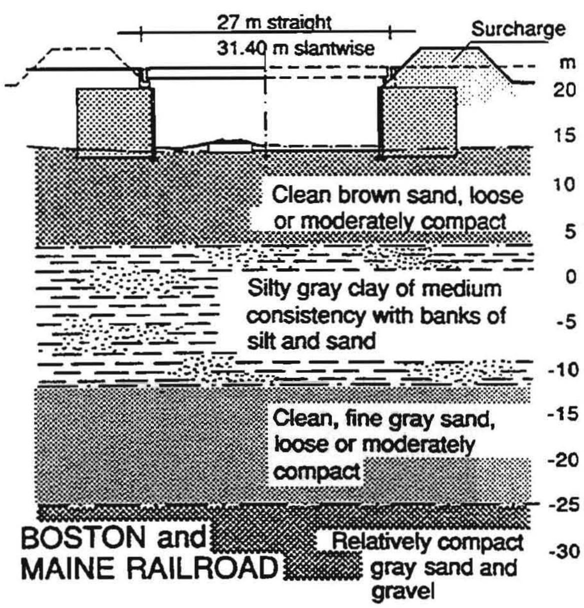


Figure 2.3. Improving the foundation soil through the use of preloading (from Reinforced Earth, 1990). a) Preloading the foundation soil with an ordinary fill prior to retaining wall structure being built. b) Combined preloading of the foundation soil, utilizing both the constructed retaining wall structure and an ordinary fill placed. Both methods help reduce settlement and increase the shear strength of the foundation soil.

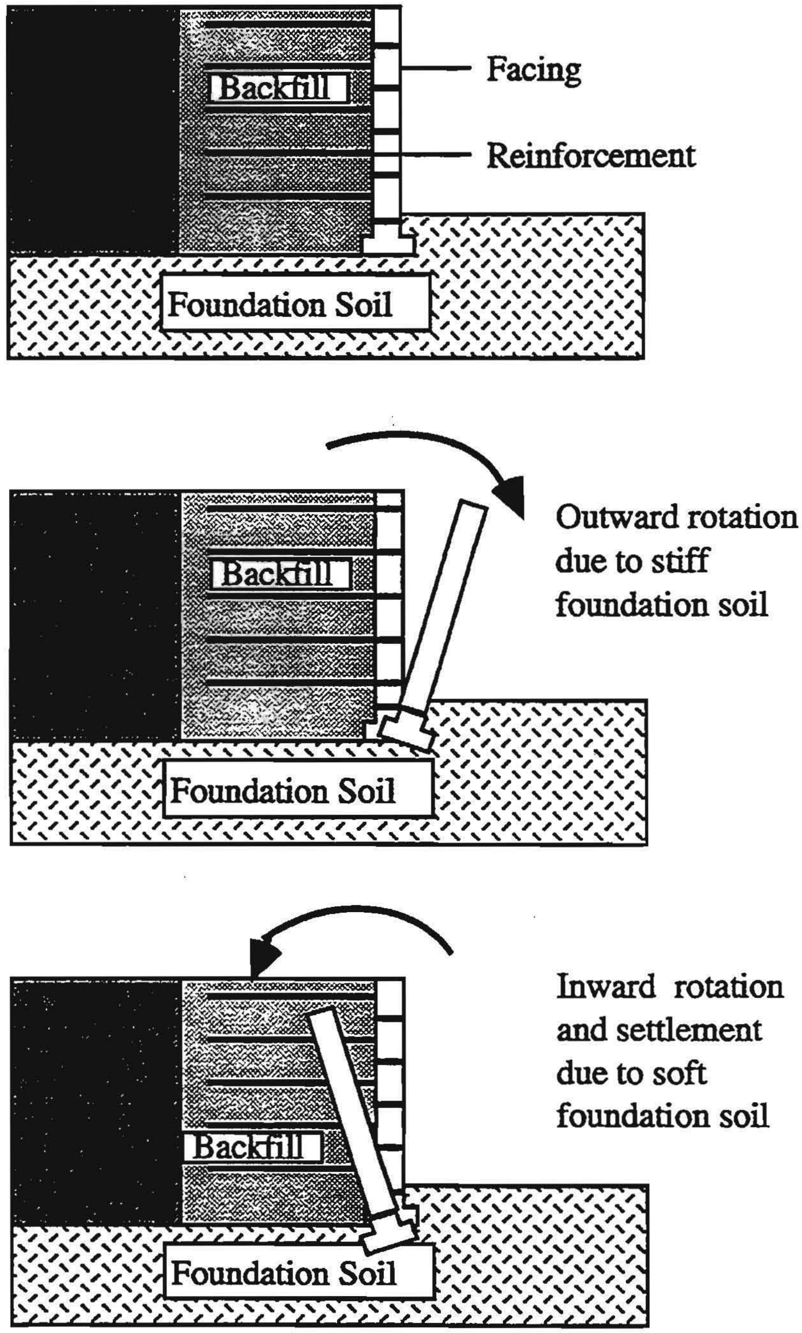


Figure 2.4 Behavior of Mechanically Stbilized Earth Retaining Wall Systems due to differing foundation soil stiffnesses. a) Stiff Foundation Soil. b) Soft Foundation Soil. Note: The depicted movement only illustrates the wall face, but it is meant to imply that the wall face, backfill and reinforcements all move together.

conventional retaining walls, it is important to understand how the overall behavior of the retaining wall structure is changed by a softer foundation soil, because it can aid the inspector in interpreting the retaining wall behavior after construction.

Backfill Soil

The backfill soil provides the bulk, the weight, the compression resistance and the shearing strength to ensure the stability of the retaining wall structure. To accomplish these, the backfill should meet the following requirements: 1) Be Easily Compacted -- Compaction of the backfill is required to achieve the weight and compressive resistance necessary for stability; 2) Be Free Draining -- A free draining backfill will not allow pore water to build up behind the wall face, which would lead to additionally forces on the wall face and potential stability problems; 3) Have a Moderately High Frictional Strength -- The frictional strength of the backfill, when compacted, should be high enough to ensure stability within the backfill alone and to achieve the required interaction with the reinforcements; 4) Low Creep Susceptibility -- The backfill should not creep over time, which could lead to excessive damage of the retaining wall structure; and 5) Low Corrosiveness -- The interaction of the backfill and the metallic reinforcements should not promote or enhance the corrosion of the reinforcements. Only good quality granular soils are capable of meeting the above requirements and should be used for backfill. (For the specifics about the requirements and characteristics for grain size, compaction and shear strength, consult the TxDOT Standard Specifications for Construction of Highways, Streets and Bridges manual.)

Soil Reinforcement

The addition of horizontal reinforcements to the backfill soil produces a composite material, like reinforced concrete, which combines the best load carrying features of both components. The reinforcements drastically reduce the lateral strain within the backfill because of the shear resistance between the backfill soil and the reinforcements. The behavior of the composite material will be as if an additional lateral confinement was applied. A direct consequence of the interaction is that as the vertical stress increases on the composite material the horizontal confining stress also increases proportionally, such that the strength of the backfill soil will be increased. Figure 2.5 illustrates schematically how the strength of the backfill material is increased by the addition of reinforcements.

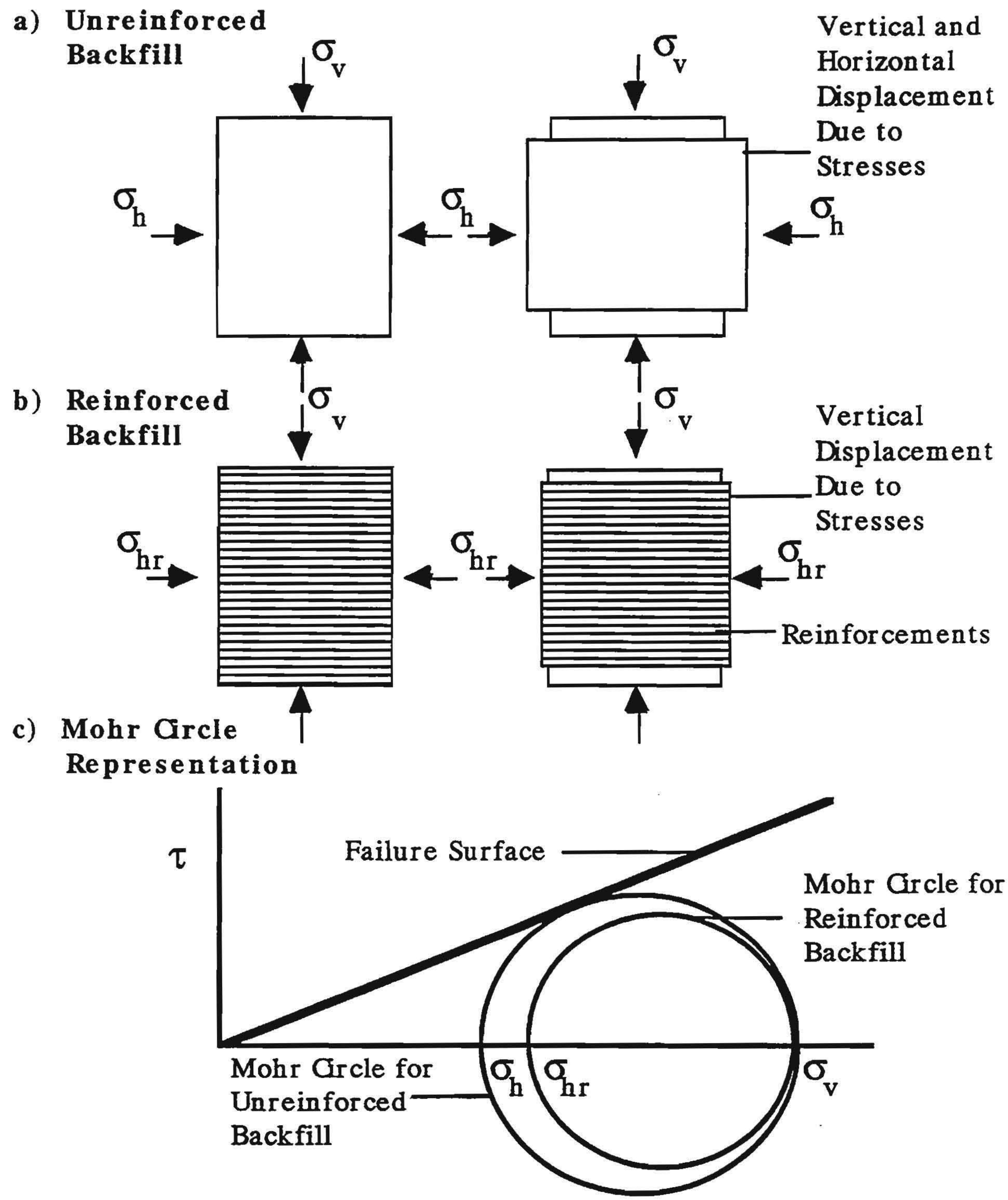


Figure 2.5 The behavior of unreinforced and reinforced backfill under stress. a) Unreinforced backfill loaded by stresses to failure undergoes vertical and horizontal displacement. b) Reinforced backfill loaded by stresses undergoes mostly vertical displacemnt. c) Mohr circle representation of the state of stresses for the Unreinforced and Reinforced backfill, showing the increases in the lateral stress and strength for the Reinforced backfill.

The importance of the reinforcements in strengthening the backfill soil places restrictions on their characteristics, which should include the following: 1) High tensile strength; 2) A failure mode which is not brittle; 3) A high resistance to cre The importance of the reinforcements in strengthening the backfill soil places restrictions on their characteristics, which should include the following: 1) High tensile strength; 2) A failure mode which is not brittle; 3) A high resistance to creep; 4) A moderate amount of flexibility to conform with the deformability associated with Mechanically Stabilized Earth Retaining Walls; 5) Be economical; 6) High durability; and 7) Compatible with the backfill soil to develop a high shear resistance along the interface between the two. While various materials meet the above characteristics, the reinforcements are typically of two different types: strips and grids. Additionally, the reinforcements are commonly made from two different materials: metal and polymers, which can be either extensible (i.e. the deformation of the reinforcement at failure is comparable or greater than that of the soil) or inextensible (i.e. the deformation of the reinforcement at failure is much less than that of the soil) Figure 2.6 illustrates the different types and materials used for reinforcements. (For a listing of which companies uses the various reinforcement types, see Part I, Chapter 3.)

For the reinforcements to be effective within the backfill soil, the interaction between the backfill and the reinforcement must be such that a high shear strength is produced. To accomplish this there are two different operative forces working, frictional forces and passive resistance forces, and the magnitude of each depends upon the type of reinforcement used. Figure 2.7 illustrates how frictional forces are developed along the interface, whereas Figure 2.8 illustrates how passive forces are developed along the interface in addition to the frictional forces. Frictional forces result form the transfer of stresses from the soil to the reinforcement by shear along the interface and is operative along any flat section of the reinforcement. As a consequence, frictional forces are present in almost all types of reinforcement. Passive forces result from the transfer of stresses from the soil to the reinforcement by the bearing capacity of the soil at transverse steps or bumps in the reinforcement. Passive forces are present in grid type and ribbed strip type reinforcements. (For a listing of which companies uses the various reinforcement types, along with the various stress transfer mechanisms, see Part I, Chapter 3.)

Facing Elements or Panels

The vertical wall facing is composed of individual units of varying size and is properly termed a facing system. The facing system used most

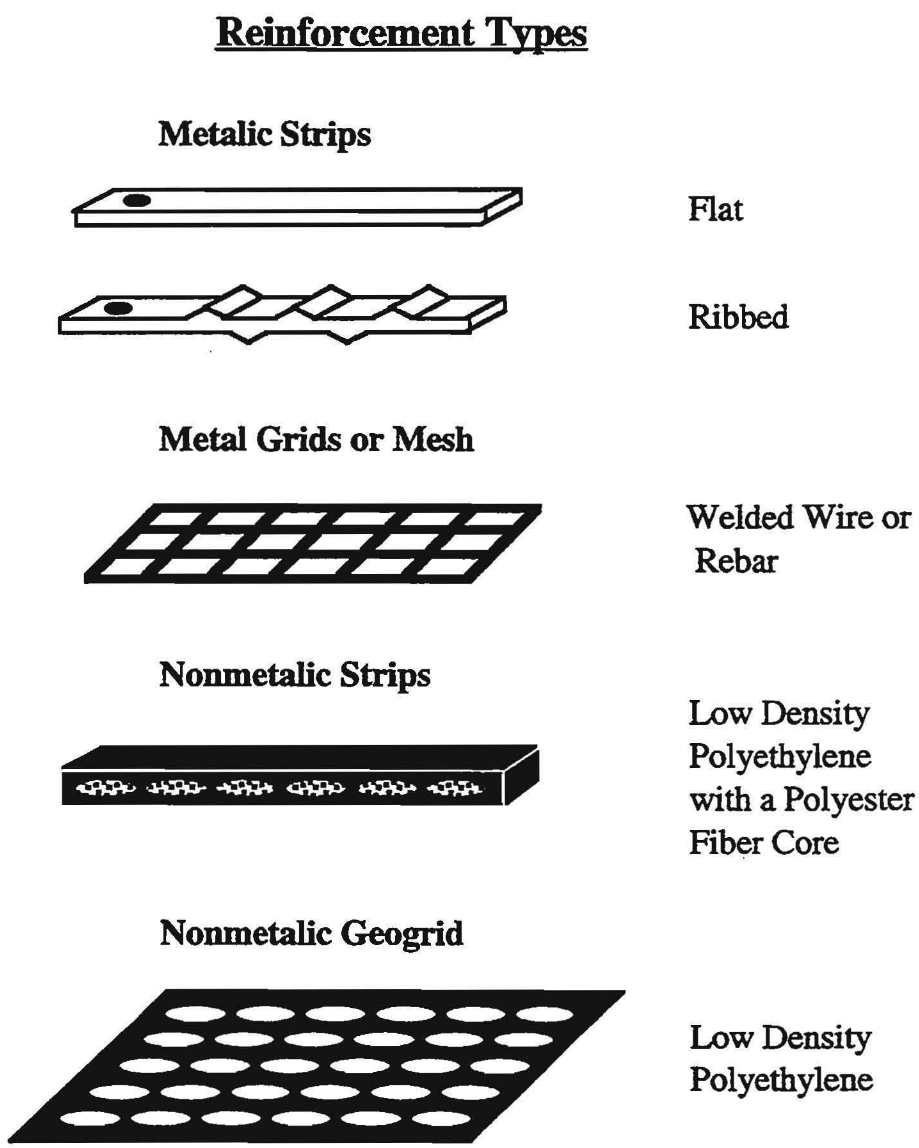
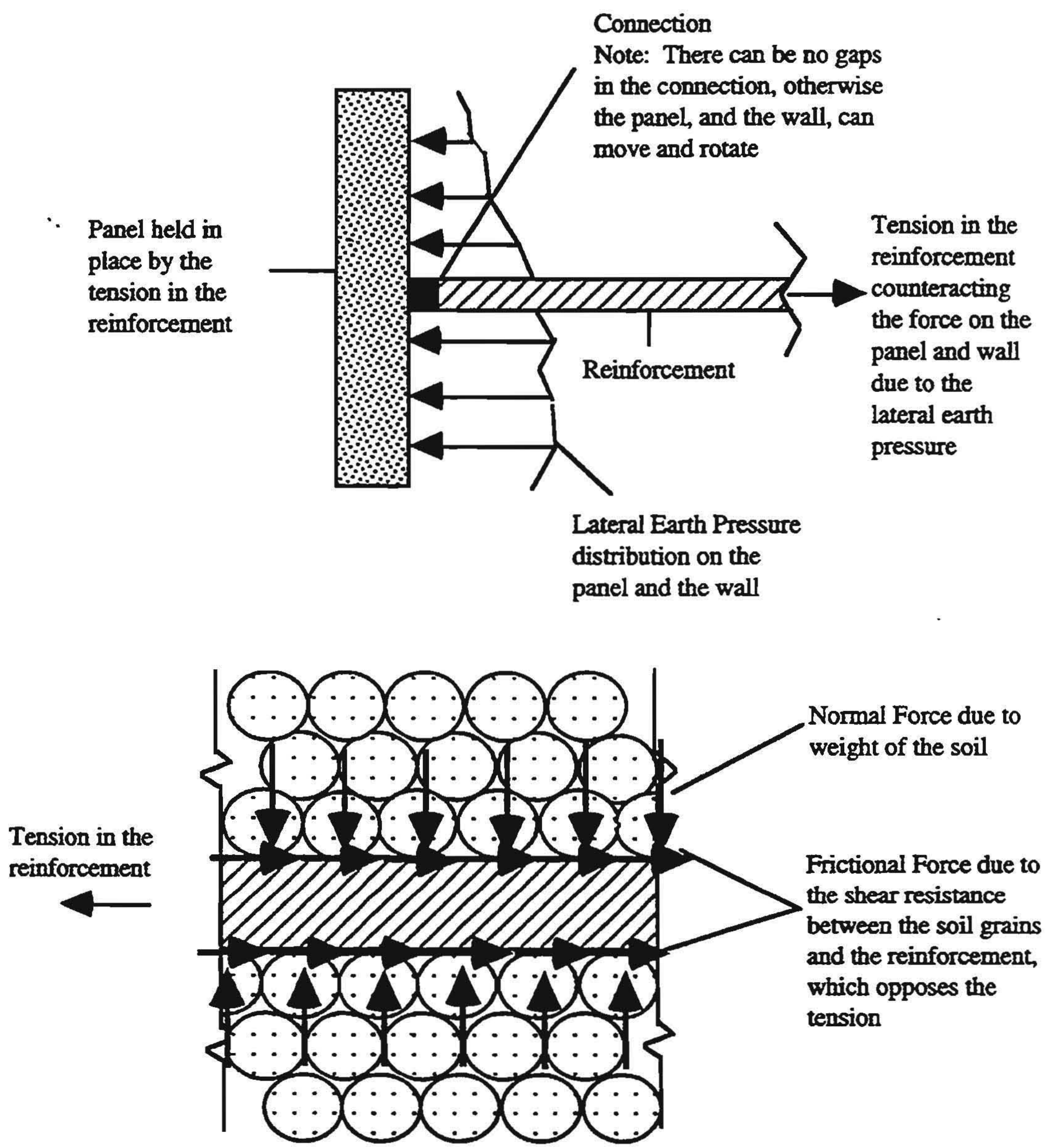


Figure 2.6 Different Types of Reinforcements.

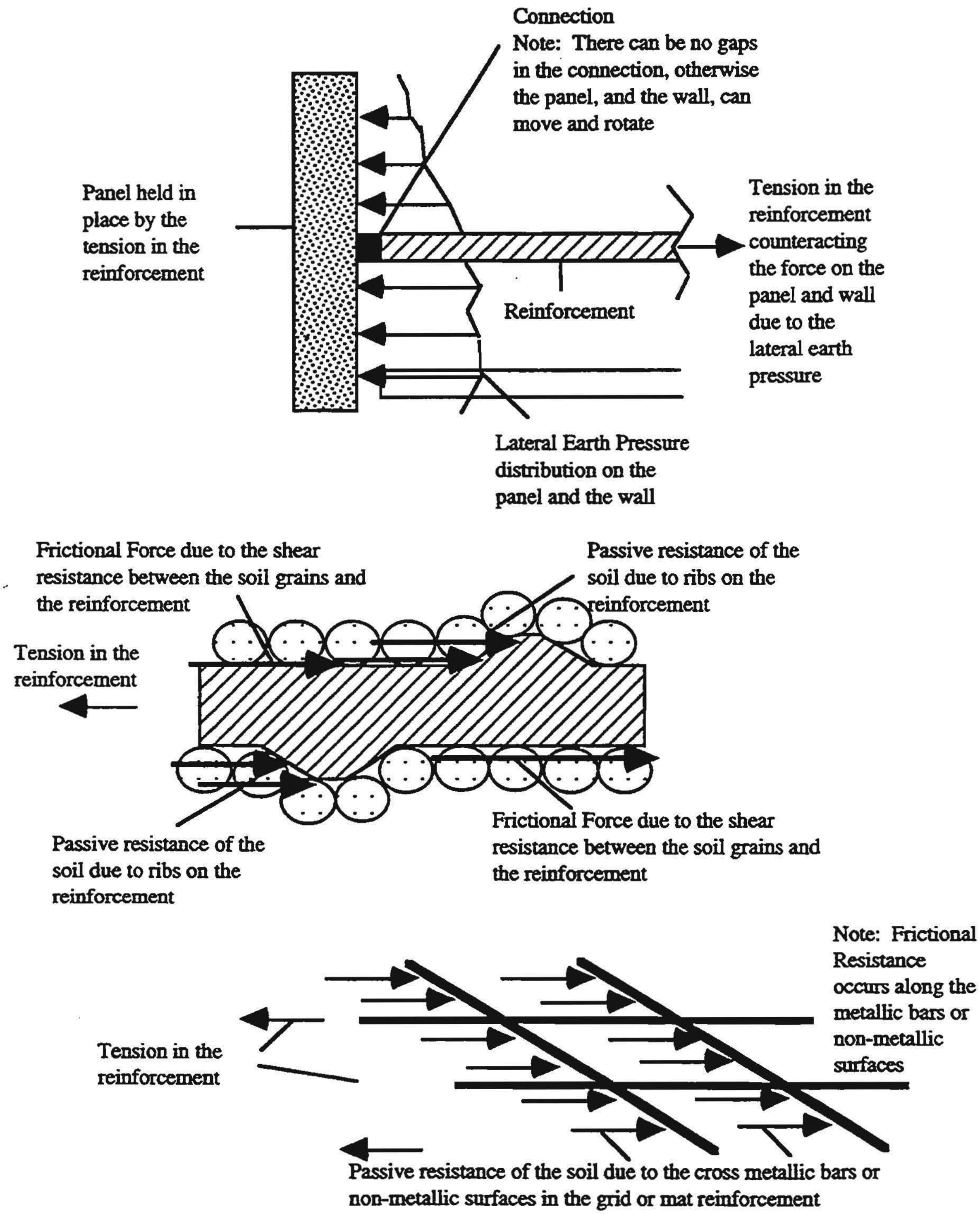
FRICTIONAL STRESS TRANSFER BETWEEN
THE SOIL AND THE REINFORCEMENT



Tension (Pull-Out Force) = Frictional Resistance

Figure 2.7 Stress Transfer Mechanism for Flat or Smooth Metallic Strip Reinforcements

STRESS TRANSFER BETWEEN THE SOIL AND THE REINFORCEMENT
DUE TO FRICTION AND PASSIVE SOIL RESISTANCE



Tension (Pull Out Force) = Frictional Resistance + Passive Resistance

Figure 2.8 Stress Transfer Mechanism for Metallic Ribbed Strips and Metallic and Non-Metallic Grids or Mats commonly in highway applications uses thin, stacked precast concrete panels, which are interlocked to form a continuous but flexible facing. Since the facing panels are used to prevent local raveling and deterioration of the backfill soil and are not used for primary structural support, the facing panels can be thin and flexible. The main advantage of the segmented concrete panel facing system is that it is durable and the external face is aesthetically pleasing and uniform. Many different panel sizes and shapes are available (see Part I, Chapter 3 for a description of some of the panel sizes and shapes used by the various companies), which all have thicknesses on the order of 15.3 to 22.9 cm (i.e. 6 to 9 in). Between the precast concrete panels, compressible or bearing pads are often placed along the joints. Additionally, geotextiles or filter fabrics are placed over all of the joints along the backside of the panels to prevent washout of the backfill soil.

Reinforcement Connection

For Mechanically Stabilized Earth Retaining Wall Systems to perform satisfactorily, the reinforcements have to be connected to the facing panels. The connection type depends upon the reinforcement used. Figure 2.9 illustrates the common reinforcement connection types used for Mechanically Stabilized Earth Retaining Wall Systems. Basically there are three different connection types: 1) Bolted Connection (figure 2.9 Type A and B); 2) Pinned Connection (Figure 2.9 Type C); and 3) Anchor or T Connection (Figure 2.9 Type D). The bolted connection is commonly used for metallic reinforcing strips and provides a good connection, with few problems. The pinned connection is commonly used with grid or mat reinforcements. While this type of connection allows the reinforcement to deform and rotate, it also can cause some problems. For example, the rod or pin that fits into the connection can form a loose connection, because the rod or pin does not fill the opening. A consequence of this is that the facing panel would not be properly supported. The facing panel could rotate and/or displace, because the reinforcement tension would not be properly transferred to the facing panel due to the loose connection. Therefore, it is important when using this type of reinforcement connection to place the pin in the connection, then pull, by hand, the reinforcement taught and place a wood spacer in the opening of the connection. This will allow the reinforcement to properly support the wall facing. The anchor or T connection is a special type of connection, commonly used with welded wire grid or mesh reinforcement. This type of connection attempts to overcome the difficulties with the pin connection. However, one must be very careful when placing the panels together so as

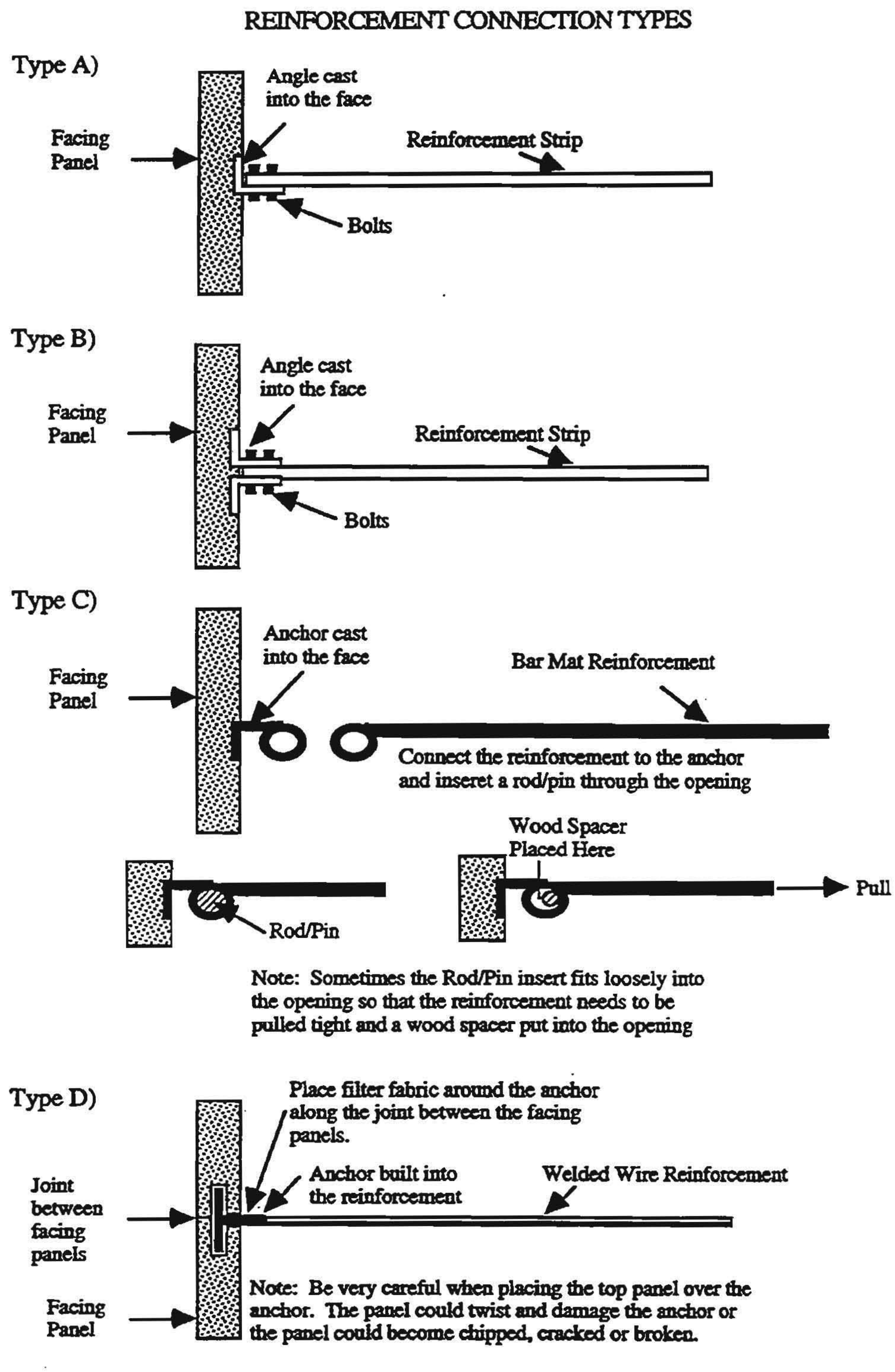


Figure 2.9 Reinforcement connection types. Type A - Bolted Connection, Type B - Bolted Connection, Type C - Pinned Connection and Type D - Anchor or T Connection.

not to damage the anchor or the panels. (For a listing of which companies uses the various reinforcement types, see Part I, Chapter 3.)

PART I. PRECONSTRUCTION PREPARATION

Chapter 3. Review the Specific Mechanically Stabilized Earth Retaining Wall System

Introduction

The introduction, development and refinement of Mechanically Stabilized Earth Retaining Wall Systems in recent years has made it possible for the highway designer to choose from a wide variety of proprietary systems. Consequently, the field inspector should have a working knowledge of the particular proprietary system that is being used. In particular, the field inspector should know what type of reinforcement is being used, the connection type for the reinforcement, and the type of backfill soil that can be used, as well as the type and shape of the wall panels. This chapter presents a brief overview of the different types of proprietary systems commonly used in Texas. For specific details about any of the specific proprietary systems the inspector should refer to the manuals provided by the individual companies.

Proprietary Systems Commonly Used in Texas

Table 3-1 presents a summary of the commonly used proprietary systems for Mechanically Stabilized Earth Retaining Wall Systems throughout the United States. However, only the following proprietary systems are commonly used in Texas: 1) Reinforced Earth; 2) VSL Retained Earth; and 3) Hilfiker Reinforced Soil Embankment (Retaining Walls in Texas, 1987). Figures 3.1, 3.2 and 3.3 show schematically the Reinforced Earth, VSL Retained Earth and Hilfiker Reinforced Soil Embankment Mechanically Stabilized Earth Retaining Walls Systems. While there are similarities among the different systems, Table 3-1 indicates that their differences are primarily in the reinforcement types used, the reinforcement connection types and the wall facing panels. Table 3-2 highlights the different reinforcement types used, the stress transfer mechanism and also indicates the general range of soil type that can be used with the different systems. (The inspector should refer back to Part I, Chapter 2 for reference to the various elements.) The inspector should have a working knowledge of the proprietary system that will be used on a specific project prior to the start of construction, so that they will know and understand the various elements and what type of problems could occur (see Part II, Chapter 7).

Table 3 - 1

SUMMARY OF VARIOUS MSE RETAINING WALL SYSTEMS, REINFORCEMENT AND PANEL DETAILS

SYSTEM NAME*	REINFORCEMENT DETAIL	TYPICAL FACE PANEL DETAIL**
Reinforced Earth The Reinforced Earth Company 1700 N Moore St. Arlington, VA 22209-19601	Galvanized ribbed and non-ribbed steel strips: 4 mm (0.16 in) thick; 50 mm (2 in) wide. Also has Epoxy coated strips to reduce corrosion affects. The connections are Type A or B.	Facing panels are cruciform shaped precast concrete 1.5 m x 1.5 m x 14 cm (4.9 ft x 4.9 ft x 5.5 in). Half sized panels are used at both the top and bottom of the wall.
VSL Retained Earth VSL Corporation 101 Albright Way Los Gatos, CA 95030	Rectangular grid, 61 cm x 15 cm (24 in x 6 in), of W11 or W20 plain steel bars. Each mesh can contain 4, 5, or 6 longitudinal bars. Epoxy coated grids are also available. The connections are Type C.	Precast concrete panel. Hexagon shaped, 1.5 m high x 1.75 m wide x 16.5 cm thick (59 1/2 in high x 68 3/8 in wide x 6 1/2 in thick).
Hilfiker Retaining Wall The Hilfiker Company 3900 Broadway PO Box 2012 Eureka, CA 95502-2012	Welded wire mesh, 5 cm x 15 cm (2 in x 6 in) grid of W4.5 x W3.5, W7 x W3.5, W9.5 x W4 and W12 x W5 in 2.44 m (8 ft) wide mats.	Welded wire mesh, wrap around style with an additional backing mat and 6.35 cm (1.4 in) wire screen at the soil face. Geotextile or shotcrete can also be used at the soil face.
Reinforced Soil Embankment The Hilfiker Company 3900 Broadway PO Box 2012 Eureka, CA 95502-2012	15 cm x 61 cm (6 in x 24 in) welded wire mesh of W9.5 to W20. The connections are Type D.	Precast concrete panel 3.8 m long x 61 cm high (12 1/2 ft x 2 ft) with spaces for anchors on the wire mesh between the panels to lock them together.
Tensar Geogrid System The Tensar Corporation 1210 Citizens Parkway Morrow, GA 30260	Non-metallic polymeric grid mat made from high density polyethylene or polypropylene. The connections are Type D.	Precast concrete facing panels are available in full-height panels or as segmental panels.
Mechanically Stabilized Embankment Dept. of Transportation State of California Division of Engineering 5900 Folsom Blvd PO Box 19128 Sacramento, CA 95819	Rectangular grid, nine 9.5 mm (3/8 in) diameter plain steel bars on a 61 cm x 15 cm (24 in x 6 in) grid. Two bar mats per panel. The connections are Type C.	Precast concrete panels. Rectangular in shape 3.81 m long x 61 cm high x 20 cm thick (12.5 ft x 2 ft x 8 in).
Georgia Stabilized Embankment Dept. of Transportation State of Georgia No. 2 Capitol Square Atlanta, GA 30334-1002	Rectangular grid of five 9.5 mm (3/8 in) diameter plain steel bars on a 61 cm x 15 cm (24 in x 6 in) grid, with four bar mats per panel. The connections are Type C.	Precast concrete panels. Rectangular in shape 1.83 m long x 1.22 cm high (6 ft x 4 ft) with offsets for interlocking.
Websol Soil Structures International, Ltd. 58 Highgate High St. London, N65HX England	135 mm x (5.3 in) wide Paraweb: made from high tenacity polyester fibers and polyethylene. The connections are Type D.	T-shaped precast concrete panel with an area of 3.2 m^2 (34.4 ft^2) and 16 cm (6.3 in) thick.

* The locations given for the System Names is for the main offices in the United States. ** Other facing types are possible within any specific system, check with the manufacturers.

(Modified after Mitchell and Christopher, 1990)

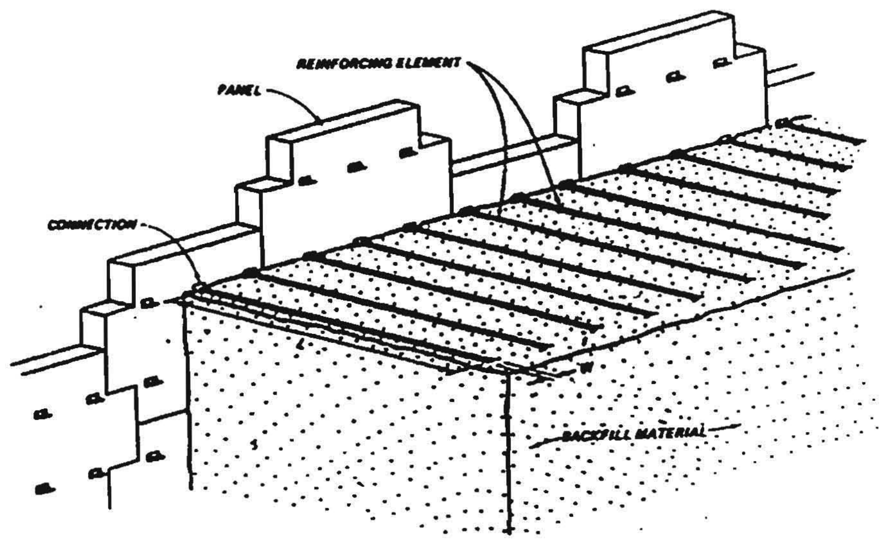


Figure 3.1 Schematic diagram of a Reinforced Earth® Mechanically Stabilized Earth Retaining Wall System.

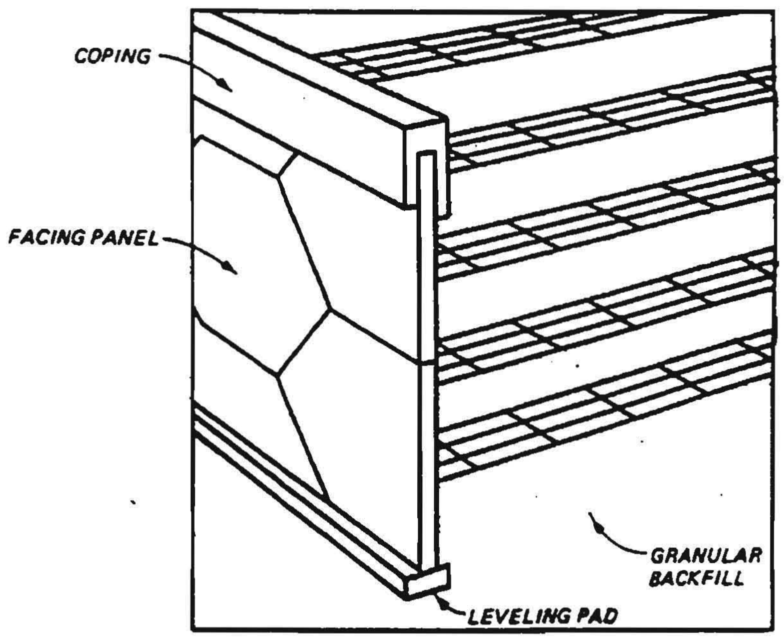


Figure 3.2 Schematic diagram of a VSL Retained Earth® Mechanically Stabilized Earth Retaining Wall System.

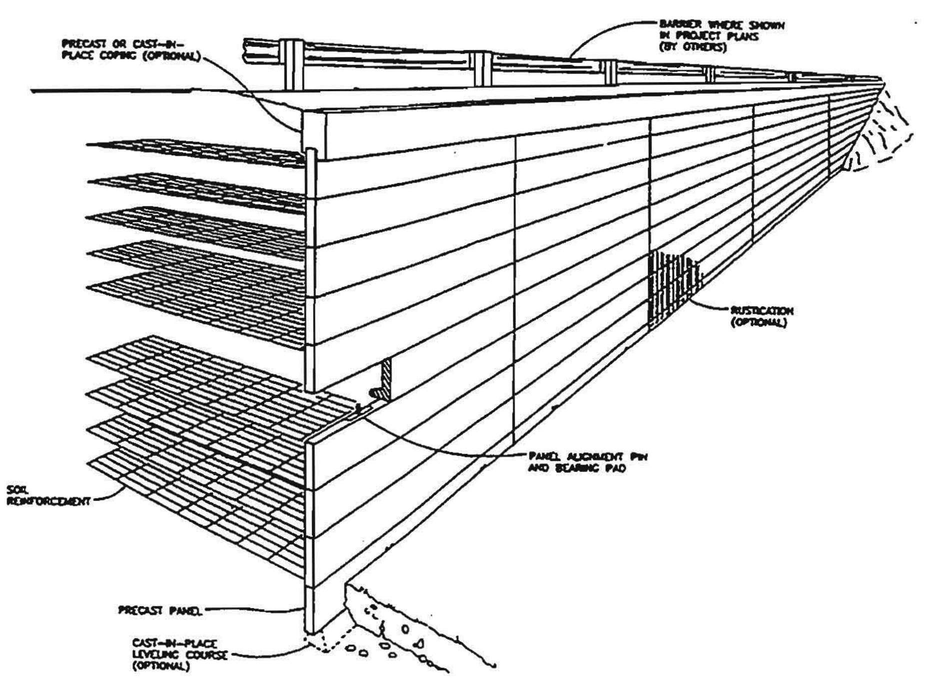
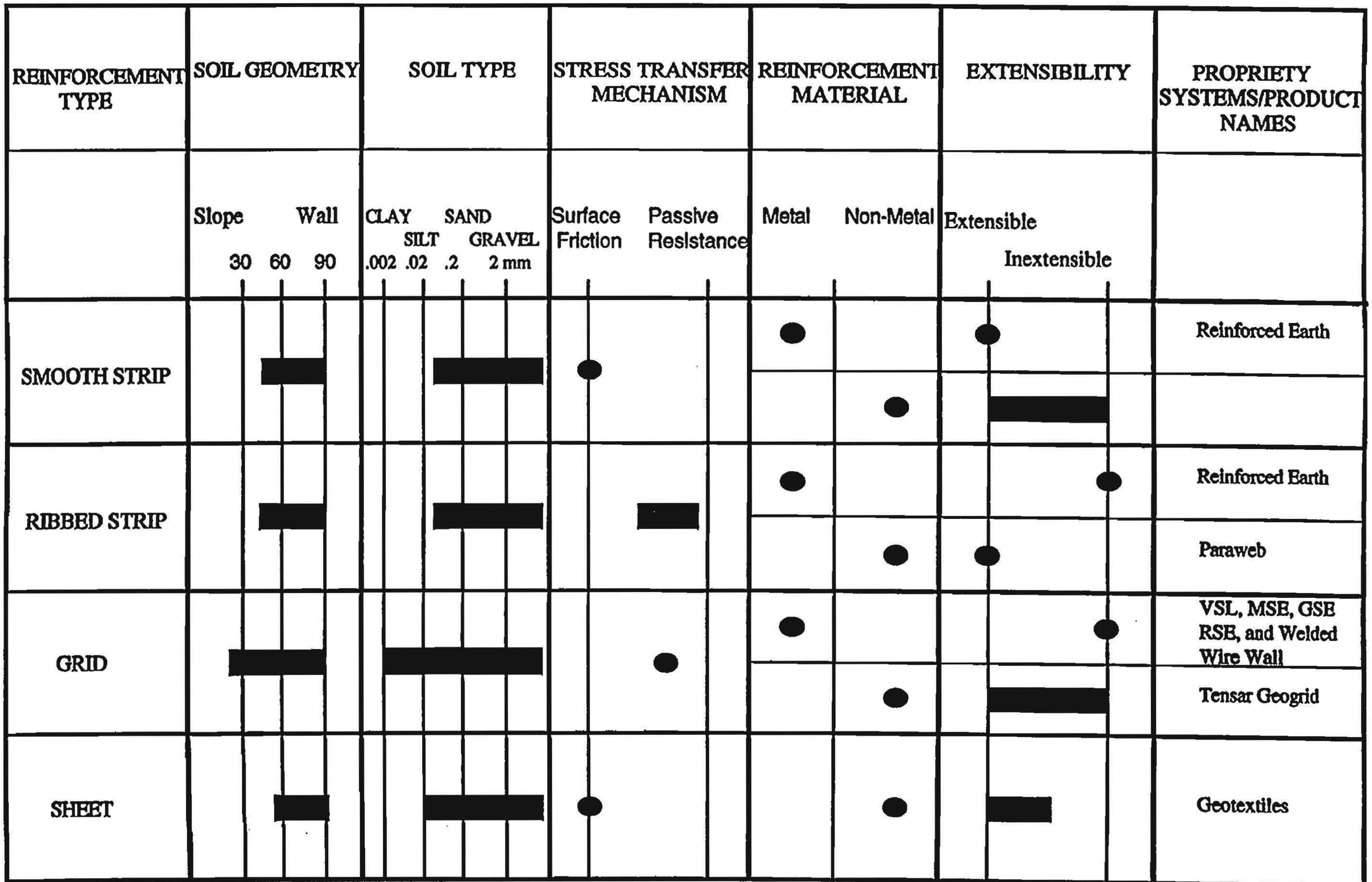


Figure 3.3 Schematic diagram of a Hilfiker Reinforced Soil Embankment® Mechanically Stabilized Earth Retaining Wall System.

Table 3-2



(after Jewell, 1984)

PART I. PRECONSTRUCTION PREPARATION

Chapter 4. Construction of Mechanically Stabilized Earth Retaining Wall Systems

Introduction

An advantage of Mechanically Stabilized Earth Retaining Wall Systems is that the use of the prefabricated facing panels, reinforcing elements and backfill soil make these walls easy to construct and construction proceeds fairly rapidly. Additionally, these walls are well suited to modern day construction conditions and methods. However, the final appearance of a Mechanically Stabilized Earth Retaining Wall System depends upon the construction sequence and the quality control in place during the construction. Consequently, close attention to detail during construction aids in ensuring the long term stability of the retaining wall system. This chapter outlines the general construction sequence for a Mechanically Stabilized Earth Retaining Wall System and provides helpful hints (highlighted in bold letters) to ensure quality construction of the wall system. For specific details of the construction sequence for a specific type of Mechanically Stabilized Earth Retaining Wall System the inspector should refer to the information manuals supplied by the specific company.

The initial phase of constructing a Mechanically Stabilized Earth Retaining Wall System involves the excavation and preparation of the foundation soil, pouring of the leveling pad and preparing the leveling pad for the placement of the first row of panels.

Preparation of the Site

Excavation of the Foundation Soil

- Excavate the foundation soil in the area where the wall is to be built to a depth and width in accordance with the plans and specifications.
- **At this time make sure the foundation soil conforms to that reported in the site investigation report.**

Preparation of the Foundation Soil

- Make sure the foundation soil is suitable for constructing the retaining wall system on. If it is not, then soil improvement in the form of removal of the soil or preloading may be called for.

- Compact the foundation soil in accordance with the plans and specifications.
- **Make sure the foundation soil beneath the proposed retaining wall system is to grade and is in line with the plans and specifications.**

Leveling Pad Construction

Pouring of the Leveling Pad

- Along the line where the face of the retaining wall is going to be placed the concrete leveling pad is poured. The concrete should be allow to cure for ~ 12 hours.
- **Note: During the course of preparing the foundation soil and pouring the leveling pad, care must be taken to ensure that inclement weather (i.e. rain) does not soften the foundation soil, both under the retaining wall and in the area where the leveling pad is to be poured. To ensure that rain water does not pool or pond on the foundation soil adequate drainage must be provided by the contractor or the foundation soil could be covered with plastic. In the event the foundation soil does get wet, the soil should be removed or allowed to dry and recompacted.**

Preparing the Leveling Pad for Placement of the First Row of Panels

- Make sure the finish on the top of the leveling pad is flat and smooth.
- A chalk line placed along the top of the leveling pad will help align the first row of panels.

The second phase of construction of the retaining wall system involves the placement of the first or initial wall facing panels, including the bracing and setting of the batter for the panels, placement and compaction of the backfill soil, installation of the reinforcements, and placing and compacting the backfill soil on top of the reinforcements.

Installation of the Initial Wall Facing Panels

Placement of the First or Initial Wall Facing Panels

- The placement of the first or initial wall facing panels begins by placing alternating half and full panels on top of the leveling pad, along the chalk line (Figure 4.1).
- The batter of the panels is set during this stage and the panels are clamped together and externally braced to hold them in place (Figure 4.2).
- **Each successive level of panels depends upon the level below them, so it is of critical importance to ensure that the first level of panels be placed carefully. Extra time should be taken to ensure that the first level is placed properly. Otherwise, the wall has no chance of being installed properly and the stability of the wall may be greatly affected.**
- The joints between the panels on the backfill side of the wall must be covered with a geotextile or filter fabric to prevent the backfill from getting into the joints or washing out between the panels (Figure 4.3).
- **Make sure during construction the geotextile or filter fabric does not becomes damaged by buckling, formation of laps and backfilling. If it does, then it should be replaced.**

Placement and Compaction of the Backfill for the First Layer of Wall Panels to the Height of the Reinforcement Connection

Placement of the Backfill Soil

- The approved backfill is placed in lifts behind the panels in the direction of panel placement. The backfill should not dumped directly against the panels, as this would cause them to move.
- **The backfill should be dumped at a distance greater than 1.52 m (5 ft) away from the wall panels. The backfill should be spread both parallel to the wall panels by earthmoving equipment and towards the panels by hand.**

Compaction of the Backfill Soil

- Each lift of the backfill is compacted by rollers to within 0.91 m to 1.52 m (3 to 5 ft) of the wall panels. A small hand operated compactor must be used within 0.91 m to 1.52 m (3 to 5 ft), to prevent panel movement.

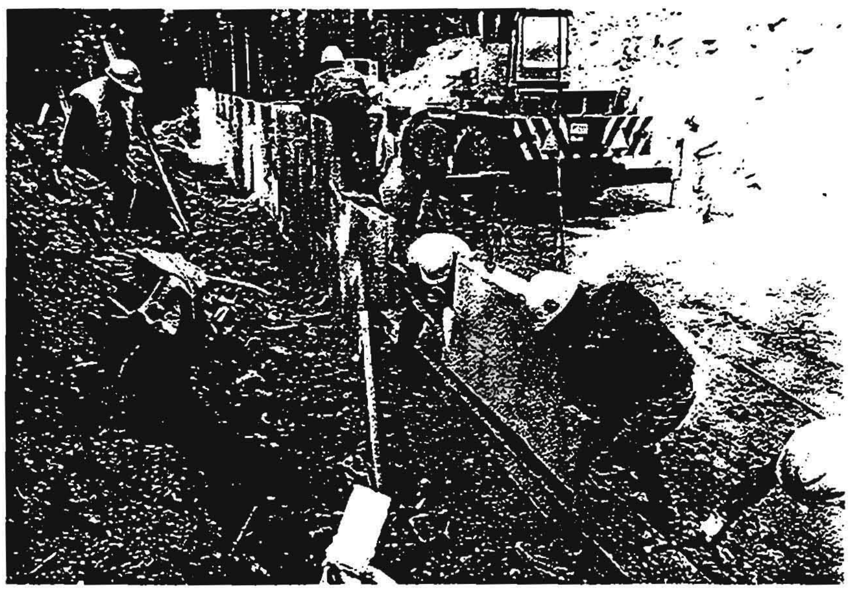


Figure 4.1 Placement of the initial wall facing panels on top of the leveling pad (from Vidal, 1979).

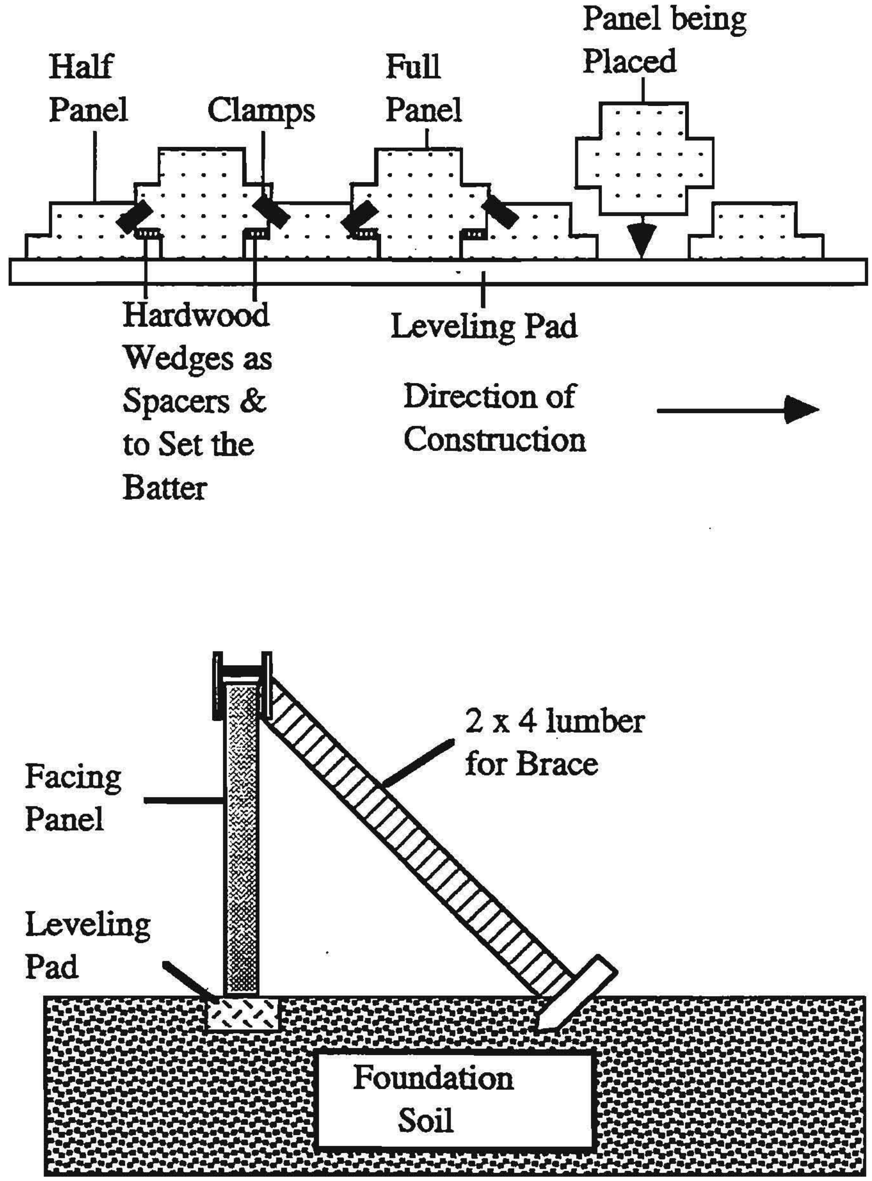


Figure 4.2 Placement of the first or initial wall facing panels and the associated bracing for the panels.

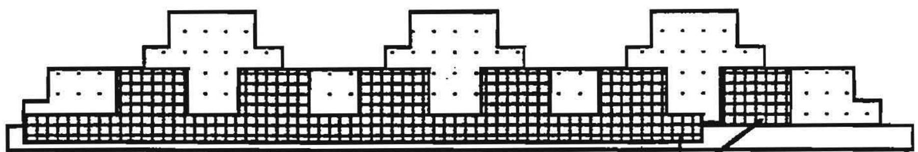


Figure 4.3 Placement of the geotextile or filter fabric over all of the joints on the backfill side of the wall.

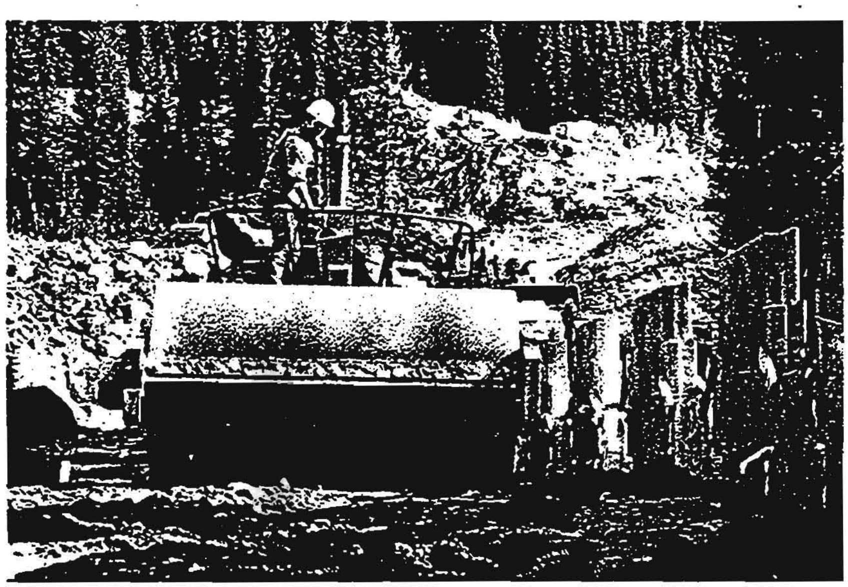


Figure 4.4 The compaction of the backfill soil occurs through the use of rollers. The rollers are supposed to stay 1.52 m (5 ft) back from the wall panels to ensure that the panels do not move or rotate. **Notice in the figure the roller is too close to the wall face.**

- **Compaction must adhere to the specifications and be tested. If the compaction does not meet the specifications or heavy rollers are used close to the face (Figure 4.4), then the wall panels could move and alter the stability of the wall (See Part II, Chapter 7).**
- **The wall alignment must be checked, both the batter and the horizontal alignment, to ensure the panels have not moved significantly.**
- The placement of the backfill and the compaction of it, proceeds until the first reinforcement connection is reached. Make sure to leave room in the backfill around the connection, so that tools can be used to connect the wall face panel to the reinforcement (Figure 4.5).
- Note: If required, the embankment should be constructed at the same time and at the same rate as the backfill. The boundary between the embankment and the backfill soils should be interfingered, to try and minimize any movements between the two that could case problems to the completed structure.
- **At the end of each day, the backfill and the embankment should be graded in a manner that keep water from flowing towards the wall face panels.**
- **In the event that it rains, the alignment of the wall panels should be rechecked to ensure that no movement has occurred.**

Installation of the Reinforcements

Placement of the Reinforcements

- Place the reinforcements on the compacted backfill perpendicular to wall panels.

Connecting of the Reinforcements

- Connect the reinforcements to the wall panels (Figure 4.6).

Tensioning of the Reinforcements

- Then pull or stretch the reinforcements perpendicular to the wall face, laying them on the compacted backfill (Figure 4.7), making sure that the connection is not loose (see discussion in Part I, Chapter 2 - Reinforcement Connections). This process is continued for all the reinforcements for the first layer (Figure 4.8).

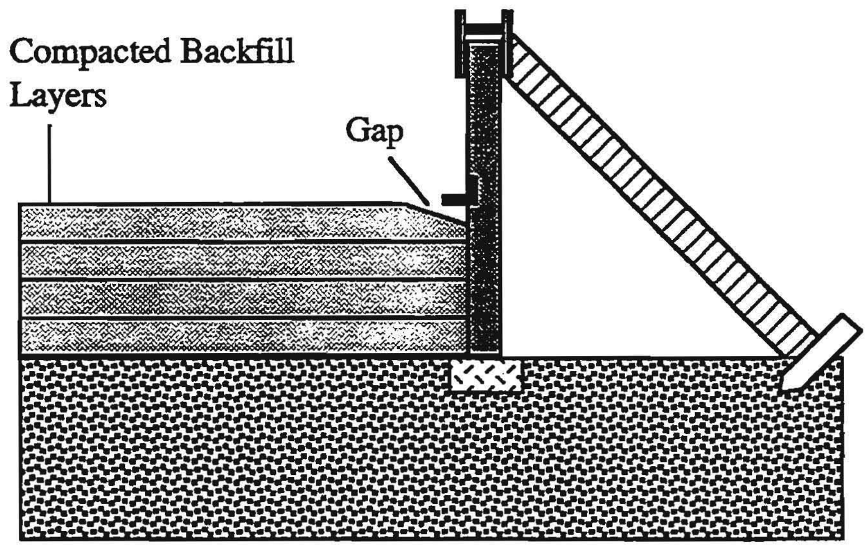


Figure 4.5 Placement and compaction of the backfill behind the wall panel to the first reinforcement connection. The gap or void left adjacent to the connection is there to allow tools to be operated while connecting the reinforcement.

Figure 4.6 Connecting the reinforcement to the wall facing panels (from Vidal, 1979). Note the space or gap beneath the connection to allow tools or a hand to fit under it to aid in the connecting process.

Figure 4.7 Pulling or stretching the reinforcement away from the wall panels to provide a tight connection and to maximize the interaction between the reinforcement and the backfill (from Vidal, 1979).

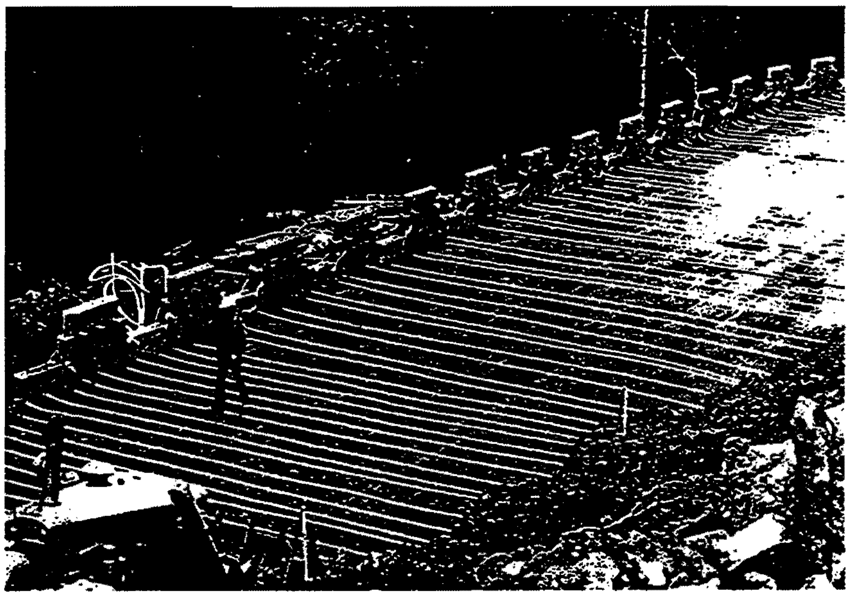


Figure 4.8 The reinforcements for the first layer of wall panels have been connected and pulled tight. The placement and compaction of the backfill over the reinforcements is the next step (From Vidal, 1979).

Placing and Compacting the Backfill Soil on top of the Reinforcements

Placement of the Backfill Soil on top of the Reinforcements

- The approved backfill is placed in lifts behind the panels on top of the reinforcements in the direction of panel placement. The backfill should not dumped directly against the panels, as this would cause them to move.
- **The backfill should be dumped at a distance greater than 1.52 m (5 ft) away from the wall panels (Figure 4.9). The backfill should be spread both parallel to the wall panels by earthmoving equipment (Figure 4.10), and towards the panels by hand.**
- **Make sure that heavy earthmoving equipment with metal tracks does not drive over the exposed reinforcement, because they could become damaged. Rubber-tired earthmoving equipment can drive on the reinforcement, provided care is taken in doing so. If the reinforcing strips become damaged during construction they should be replaced by the contractor.**

Compaction of the Backfill Soil on top of the Reinforcements

- Each lift of the backfill is compacted to within 0.91 m to 1.52 m (3 to 5 ft) of the wall panels. A small hand operated compactor must be used within 0.91 m to 1.52 m (3 to 5 ft), to prevent panel movement.
- **Compaction must adhere to the specifications and be tested.**
- **The wall alignment must be checked, both the batter and the horizontal alignment, to ensure the panels have not moved.**
- The placement of the backfill and the compaction of it, proceeds until the top of the first row of half panels is reached (Figure 4.11).
- If required, the embankment should be constructed at the same time and at the same rate as the backfill. The boundary between the embankment and the backfill soils should be interfingered, to try and minimize any movements between the two that could case problems to the completed structure.
- **At the end of each day, the backfill and the embankment should be graded in a manner that keeps water from flowing towards the wall face panels.**

Figure 4.9 Commonly backfill placement and spreading occur simultaneously. The backfill is placed by a dump truck and spread by a bulldozer (from Vidal, 1979).

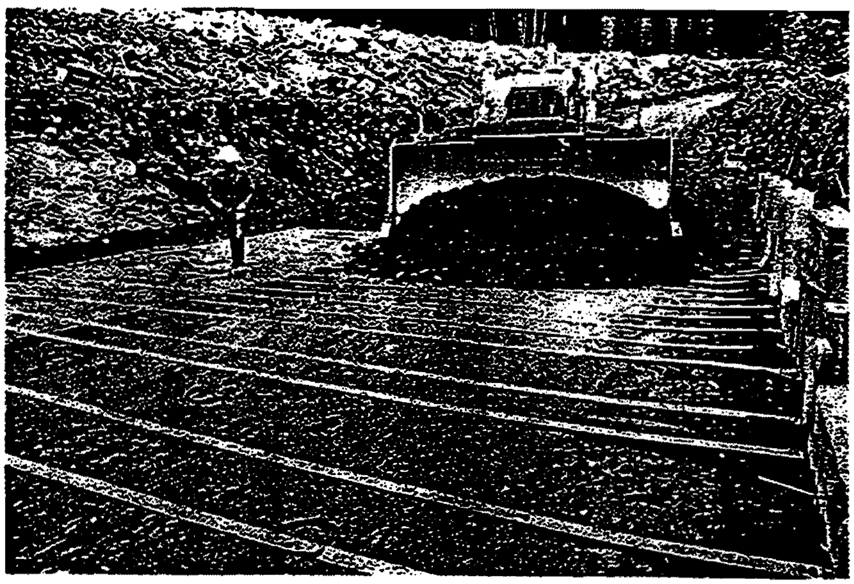


Figure 4.10 The spreading of the backfill soil on top of the reinforcements parallel to the panels with a bulldozer. The bulldozer is approximately 1.52 m (5 ft) back from the wall panels (compare distance with the height of the man on the left). The man walking is making sure that the bulldozer does not damage the reinforcements (from Vidal, 1979).

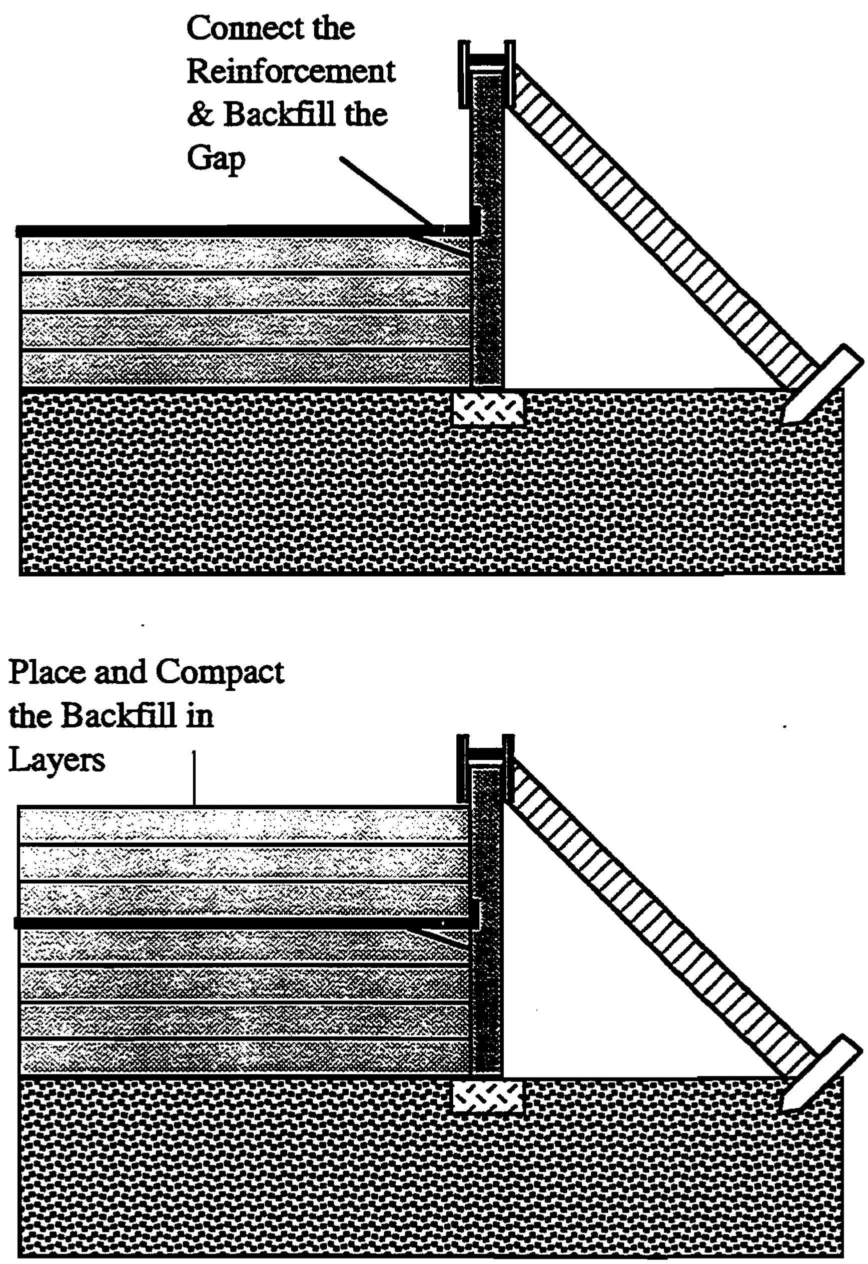


Figure 4.11 a) Connect the reinforcement to the wall panel and fill in with backfill the gap or void left adjacent to the connection. b) Placement and compaction of the backfill behind the wall panel to the top of the first row of wall panels.

Construction of Subsequent Wall Facing Panels, Reinforcements and Backfill

This phase of construction of the retaining wall system involves the placement of the second and subsequent wall facing panels, including the bracing and setting of the batter for the panels, placement and compaction of the backfill soil, installation of the reinforcements, and placing and compacting the backfill soil on top of the reinforcements.

Installation of the Second and Subsequent Layers of Wall Facing Panels

Placement of the Second Wall Facing Panels

- Place bearing pads along the joints on top of the first row of half and full panels.
- The placement of the second and subsequent wall facing panels begins at the location where construction began for the first row of wall panels and proceeds in the same direction as that done for the first row of panels.
- The batter of the panels is set and the panels are clamped together and to hold them in place (Figure 4.12).
- **Check the wall alignment and batter to ensure that the panels are being placed properly and in accordance with specifications.**
- The joints between the panels on the backfill side of the wall must be covered with a geotextile or filter fabric (Figure 4.13).
- **Make sure during construction the geotextile or filter fabric does not becomes damaged by buckling, formation of laps and backfilling. If it does, then it should be replaced.**

Placement and Compaction of the Backfill for the Second and Subsequent Rows of Wall Panels to the Height of the Reinforcement Connection

Placement of the Backfill Soil

- The approved backfill is placed in lifts behind the panels in the direction of panel placement. The backfill should be placed in the same manner as described for the first layer of wall panels.

Compaction of the Backfill Soil

- Each lift of the backfill is compacted by rollers to within 0.91 m to 1.52 m (3 to 5 ft) of the wall panels. A small hand operated compactor must be used within 0.91 m to 1.52 m (3 to 5 ft), to prevent panel movement.

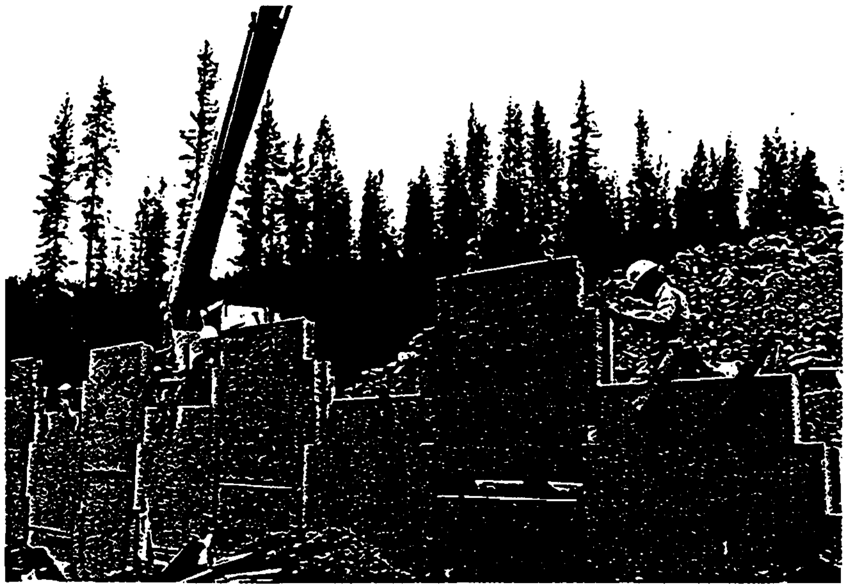


Figure 4.12 Placement of the second row of wall panels and the associated support or bracing for the panels.

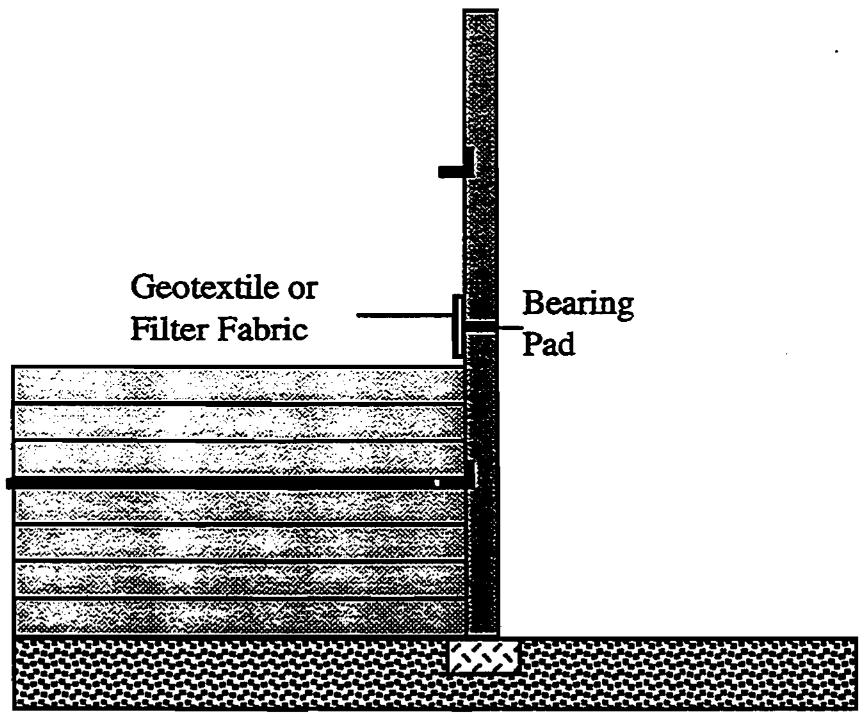


Figure 4.13 Placement of the geotextile or filter fabric over all of the joints on the backside or backfill side of the retaining wall.

- **Compaction must adhere to the specifications and be tested. If the compaction does not meet the specifications or heavy rollers are used close to the face, then the wall panels could move and alter the stability of the wall (See Part II, Chapter 7).**
- **The wall alignment must be checked during compaction, both the batter and the horizontal alignment, to ensure the panels have not moved significantly.**
- The placement of the backfill and the compaction of it, proceeds until the first reinforcement connection is reached. Make sure to leave room in the backfill around the connection, so that tools can be used to connect the wall face panel to the reinforcement (Figure 4.14a).
- Note: If required, the embankment should be constructed at the same time and at the same rate as the backfill, as discussed previously.
- **At the end of each day, the backfill and the embankment should be graded in a manner that keeps water from flowing towards the wall face panels.**
- **In the event that it rains, the alignment of the wall panels should be rechecked to ensure that no movement has occurred.**

The installation of the reinforcements, as well as the placement and compaction of the backfill on top of the reinforcements proceeds as discussed previously for the first layer of wall panels. However, the backfill soil placement and compaction progresses until the top of the previous row of full wall panels is reached (Figure 4.14b). The batter and horizontal alignment of the wall panels should be checked after the completion of the new row of wall panels. The wall panels should be near vertical from compaction. If they are not then changes in the batter should be made for the next row of wall panels, so that they will be near vertical after compaction of the backfill soil. This process of adding a new row of wall panels, placing and compacting the backfill soil, installing the reinforcements and placing and compacting the backfill soil on top of the reinforcements is repeated until the specified wall height is achieved.

Completion of the Wall

After the top wall panels have been installed and backfilling is complete the wedges between the panels can be removed, a coping can

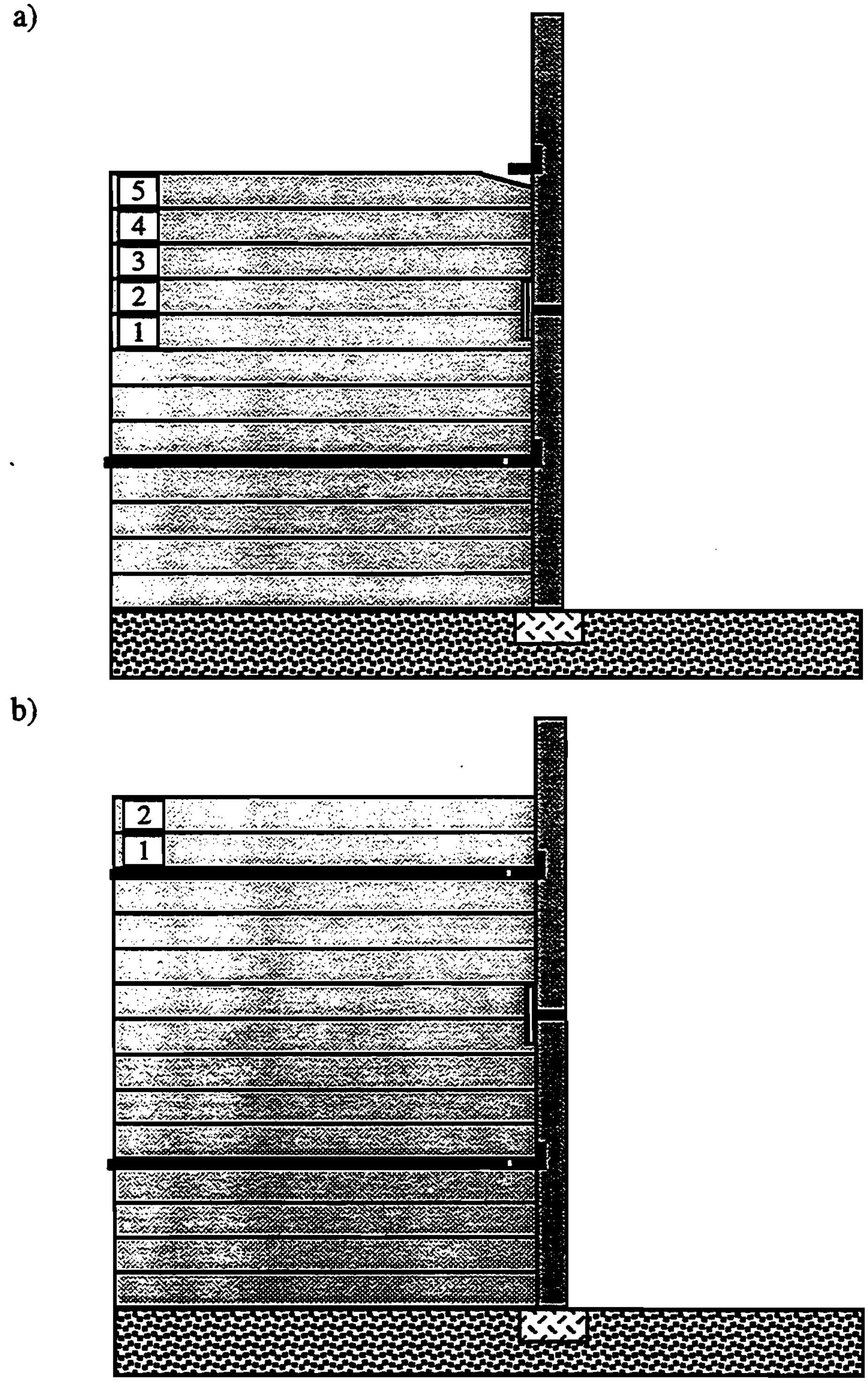


Figure 4.14 a) Placement and compaction of the backfill soil to the reinforcement connection, leaving a small gap or void for the connection. b) Placement and compaction of the backfill over the reinforcement to the top of the previous row of full wall panels. Numbers indicate the compacted backfill layers.

installed and the sub-base and highway structure completed. In addition, for retaining walls with bridge abutements the drainage system should also be installed (Figure 4.15), because most retaining wall systems problems in bridge abutment regions stem from poor or improper water drainage.

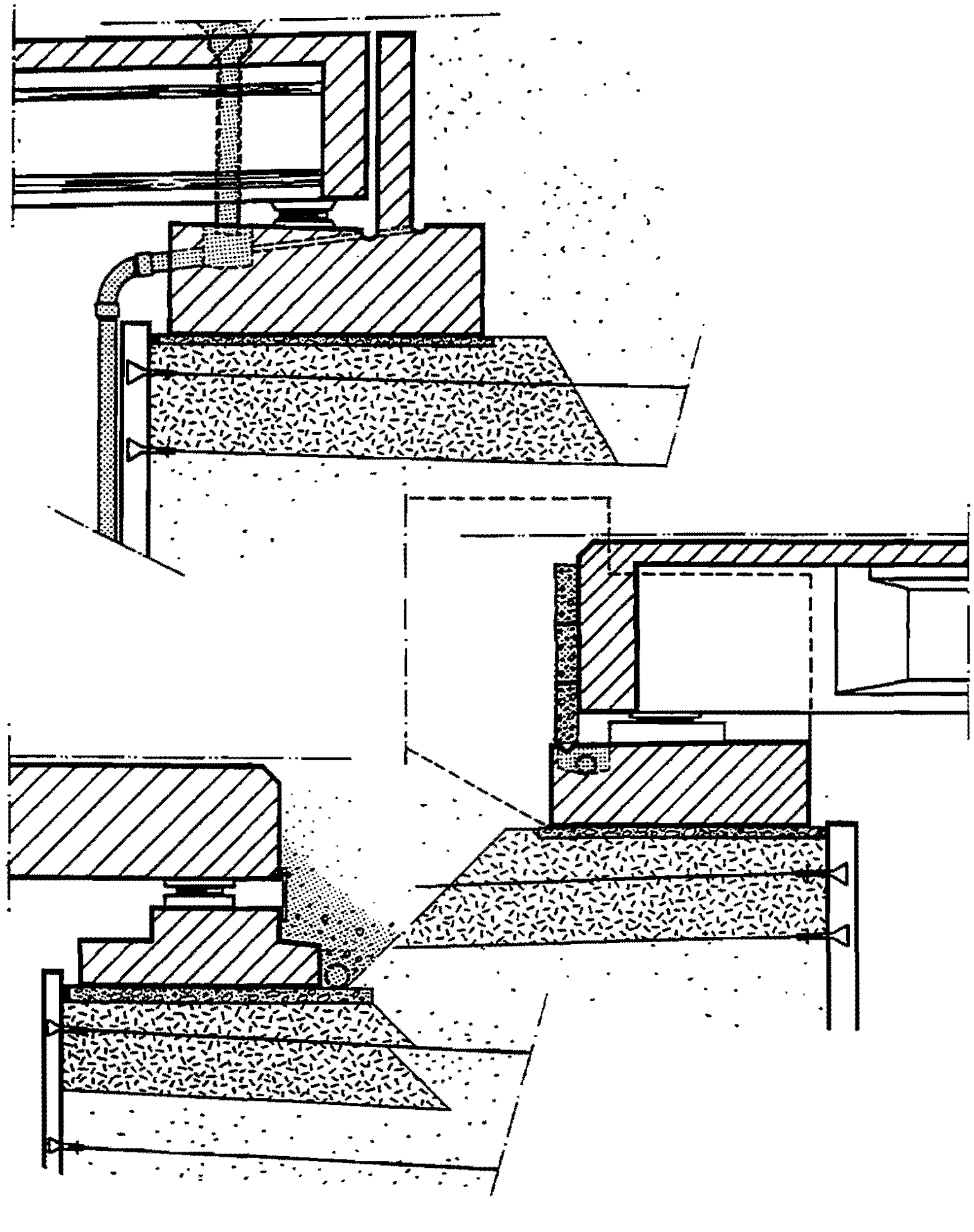


Figure 4.15 Drainage details for bridge abutments.

PART II. CONSTRUCTION INSPECTION

Chapter 5. Material Inspection

Introduction

At the start of construction all of the materials to be used should be inspected at the site. This will elliminate the possibility of poor quality materials contributing to construction problems. The materials to be inspected include the following: 1) The precast wall facing panels; 2) The reinforcing elements; 3) The wall facing joint materials; 4) The backfill soil; and 5) The foundation soil. This chapter is a guideline for the field inspectors on the quality control of the construction materials.

Field Inspection of the Materials

The quality of all the materials used for construction of Mechanically Stabilized Earth Retaining Wall Systems should be done in the field. The acceptance of the various materials should be be based on a combination of the following:

- Visual Observations
  Visual observations typically include examination for damaged or defective materials, measurements of physical dimensions, examination for consistency of materials and examination of the material storage methods. A record of all the inspectors visual observations should be kept.
- Certification by the Manufacturer or Supplier
  Material certification from the manufacturer or supplier should be checked for conformance to the plans and specifications. Additionally, the product labels should be checked, as well as the specific material properties (i.e. diameter, modulus, type etc.). The inspector should keep a record of the materials that have been accepted on the basis of the manufacturer's certification.
- Laboratory Material Testing
  Certain materials can not be adequately tested or verified at the site. For example, the backfill soil and foundation soil would have to be tested in the laboratory. The field inspector would be responsible for the procuring of proper field samples for labarotory testing and transporting them to the laboratory office. The acceptance or denial of the material should be recorded and the contractor should be notified.

Precast Wall Facing Panels

Wall facing panels delivered to the site should be examined prior to use. The examination should include checking the certification by the manufacturer and visual observations. The manufacturer's certification should be examined to ensure that they are consistent with the plans and specifications. Visual observations should include the checking for the following:

- Defects in the form of cracks, chips or imperfect molding.
- Color or finish variations of the front of the panels.
- Out-of-tolerance dimensions.
- Misalignment of or damaged connections.

Wall facing panels that are not consistent with either the manufacturer's specifications or those on the particular projects plans and specifications should be rejected. In certain cases, the contractor may be able to make minor repairs to the wall facing panels to allow them to be accepted, however, this should be done to the inspectors satisfaction and a record should be kept.

On the site the wall facing panels should be properly stored. The inspector should consult the manufacurer's construction manual for details on the storage of the wall facing panels for the specific retaining wall system being used. The inspector should visually observe the manner in which the panels are being stored, to ensure that it is done properly.

Reinforcing Elements

Reinforcing elements come in a variety of materials, configurations and sizes for the different proprietary retaining wall systems. The inspector should be knowledgeable about the specific reinforcing elements and should consult the manufacturer's construction manual for details on storage. The inspector should examine the manufacturer's certification to make sure the material is properly identified, as well as check the specified designation (i.e. AASHTO, ASTM etc.) and properties (i.e. strength, modulus, creep properties etc.). Additionally, the reinforcing elements should be visually inspected for damage, defects and out-of-tolerance dimensions. Nonmetallic reinforcing elements should be sent to the laboratory for material verification testing. If the reinforcements are not to specification they should be rejected.

Wall Facing Joint Materials

Bearing pads, joint filler and geotextiles or filter fabrics are usually considered to be miscelaneous items and are not considered as critical elements of a Mechanically Stabilized Earth Retaining Wall System. However, incorrect placement or the use of a wrong material can cause signifcant distress to the retaining wall system. The inspector should examine each of the materials to determine if they are properly identified, meet the required specifications and are in good condition. Additionally, the inspector should also visually inspect the storage of these materials, because long term exposure to sunlight can cause damage to them.

Backfill Soil

The backfill soil is the key element in the satisfactory performance of Mechanically Stabilized Earth Retaining Wall Systems. It is required to meet gradation, plasticity, soundness, and electrochemical limits, which are given by the manufacturer and are listed in TxDOT's Standard Specifications for Construction of Highways, Streets and Bridges manual. The inspector should sample the backfill for laboratory testing to determine if it meets the required specifications. If the backfill does not meet the specifications then it should be replaced with a different backfill soil which does. In addition to testing the backfill at the onset of construction, the backfill should be tested periodically throughout the construction of the retaining wall system to ensure that it consistently meets the specifications. This testing should be done at least once for every 1,530 m^3 (2,000 yd^3) of backfill soil placed behind the wall facing panels, whenever the material appears to have changed properties or if excessive wall facing panel movement occurs during construction.

Foundation Soil

The foundation soil provides support for the entire Mechanically Stabilized Earth Retaining Wall System structure. It is required to be of sufficient strength to support the structure, free of deleterious materials and be of low to moderate compressibility. The inspector should visually inspect the foundation soil for weaknesses, water spots and uniformity. This can be done quite easily by walking on the soil and looking at it. Additionally, a sample of the foundation soil should be taken back to the laboratory for liquid and plastic limit testing to confirm that it meets the requirements on the plans and specifications. If the foundation soil does not meet the requirements or appears to be weak, the inspector should consult with a geotechnical engineer as to what should be done. A number of options are available (i.e. see Part I, Chapter 2), but which one is the best and cost effective should be left for the geotechnical engineer to decide.

The embankment soil also is required to meet similar specifications as those for the foundation soil and should also be sampled and tested. The testing should involve both the liquid and plastic limit tests to verify the soil meets the specifications on the plans. If it does not a geotechnical engineer should be consulted as to what to do.

Materials Inspection Check List

* Check the manufacurer's certification to verify the wall facing panels are in compliance with the plans and specifications.

* Visually inspect the wall facing panels for defects, damage, color or finish variations and out-of-tolerance dimensions.

* Ensure that the wall facing panels are stored in a manner consistent with the manufaturer's storage procedures.

* Examine the manufacturer's certification for the reinforcing elements to verify their identification, specified designation and structural properties.

* Visuallly inspect the reinforcing panels for damage, defects and out-of-tolerance dimensions.

* For nonmetallic reinforcing elements samples need to taken and laboratory tested for material verification.

* Visually inspect the wall facing joint materials for proper identification and damage.

* Ensure the storage of the wall facing joint materials is in compliance with the manufacturer's specification.

* Visually inspect the backfill soil for uniformity and consistency throughout construction.

* Sample the backfill soil for laboratory testing both at the onset of construction and as follows during construction: 1) At least once for 11,530 m^3 (2,000 yd^3) of backfill placed; 2) Whenever there is a change in properties of the backfill soil; and 3) If excessive wall facing panel movement occurs.

* Verify that the backfill soil meets the required specifications. If it does not then it should be replaced and the contractor should be notified.

* Visually examine the foundation soil for weaknesses, water spots and uniformity.

* Sample and laboratory test the foundation soil to confirm it meets the specifications. If it does not a geotechnical engineer should be consulted.

* Sample and laboratory test the embankment soil to confirm it meets the specifications. If it does not a geotechnical engineer should be consulted.

PART II. CONSTRUCTION INSPECTION

Chapter 6. Construction Monitoring

Introduction

Mechanically Stabilized Earth Retaining Wall Systems are flexible, capable of withstanding moderate deformations without distress, and easy to construct. In fact, they do not require any specialized mechanical equipment and the workers do not need any specialized skills for construction. Consequently, construction proceeds fairly rapidly. These features are what make the retaining wall system both competitive and well suited for modern day construction. However, these features also give the contractor a false impression that not complying with all of the manufacturer's and TxDOT's specifications during construction will not affect the performance and stability of the retaining wall systems. Nothing could be further from the truth. In fact, this impression when put into practice is one of the leading causes in contributing to instability and distress within the system. Therefore, construction monitoring, quality control and quality assurance during construction are a critical factor in determining the long term stability of the retaining wall system. This chapter provides an outline of construction monitoring and quality control measures to ensure that the retaining wall system is built correctly and will be stable.

Materials Useful to Aid in Monitoring Mechanically Stabilized Earth Retaining Wall Systems

Construction monitoring includes the following: 1) Verifying that the construction is performed according to the plans and specifications; 2) Verifying the materials are in compliance with the specifications throughout construction; 3) Measuring the batter of the wall face panels; 4) Measuring the horizontal movements of the wall face panels; 5) Measuring the vertical movements of the wall face panels; 6) Measuring local movements or differential movements of the structure; and 7) Verify that proper grading and drainage are provided for the structure at all stages of construction.

To successfully implement the monitoring during construction a few simple pieces of equipment which are useful for the contractor to have at the site are listed below.

- Tape Measure (at least 3 m or 10 ft in length)
- 2 x 4 that is 3m (10 ft) in length, with one planed edge
- Plum Bob
- 1.22 m (4 ft) Level (see Figure 6.1)
- Compass
- Surveying Equipment (optional)

The above instruments can be used in the following manner to aid with measurements and monitoring.

- Measure the horizontal level of the wall facing panels (Figure 6-2).
  - Use the modified level horizontally along a single wall facing panel and measure the slope (Figure 6.2a).
  - Lay the 2 x 4, with the planed side towards the panels, across the wall panels and use the modified level to determine the slope (Figure 6.2 b).

- Measure the batter of the wall facing panels.
  - Use the modified level vertically along a single wall facing panel and measure the slope.
  - Use the plum bob and the tape measure to determine the slope of a series of wall facing panels.

- Measure the horizontal alignment of the wall facing panels.
  - Visually observe the wall facing panels to see if they are aligned.
  - Lay the 2 x 4, with the planed side towards the panels, along the wall panels and use a compass to determine the direction, compare this with the plans.
  - Use a transit to monitor and measure this.

- Monitor and Measure the Local Movements and/or Deterioration of the wall facing elements.
  - Visual observation.
  - Use the tape measure to monitor joint closings, crack openings and distances between different wall facing panels.

- Measure the horizontal and vertical rotation of the wall facing panels.
  - Use the modified level vertically to monitor the slope change over time.
  - Use the 2 x 4 vertically against the wall facing panels and the modified level to monitor the slope change over time of two wall facing panels.

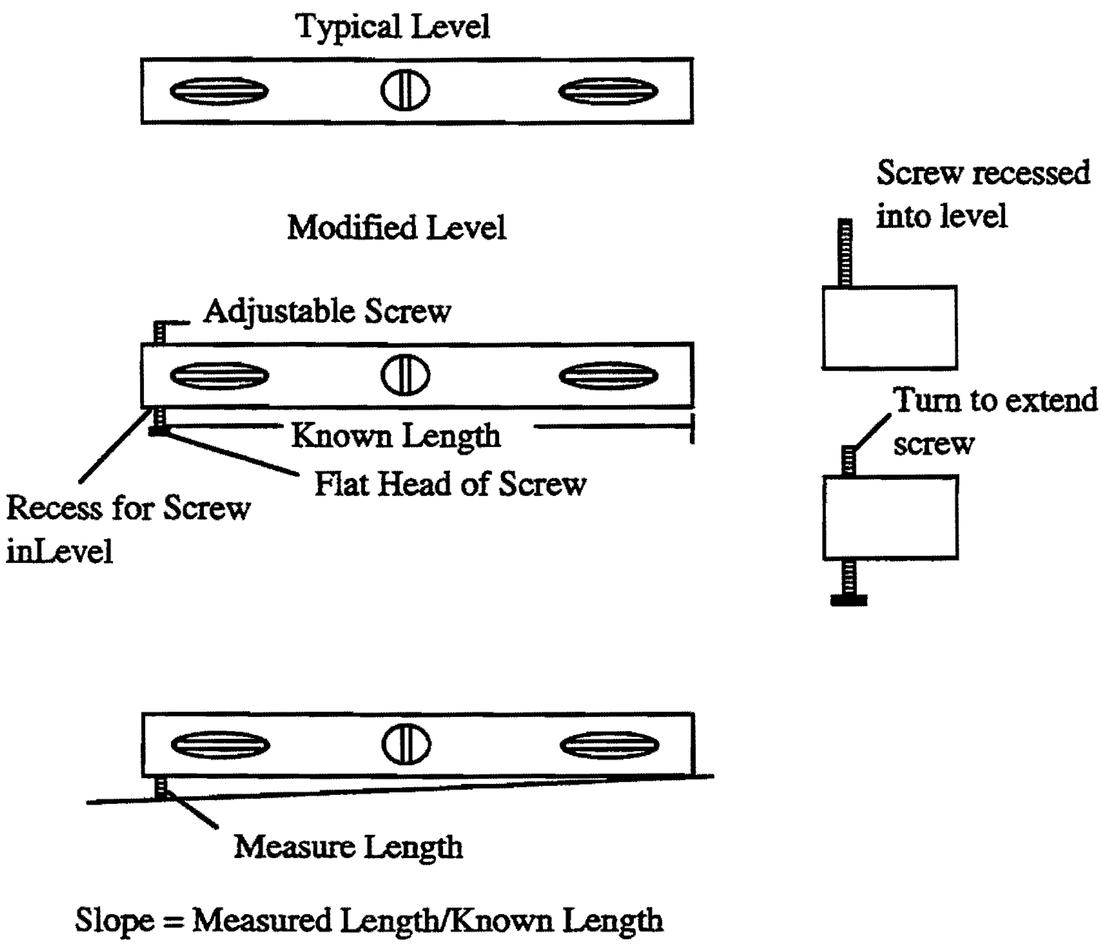


Figure 6.1 a) Standard 1.22 m (4 ft) level. b) Modified level. An adjustable screw has been added to the level. c) The purpose of the screw is to allow the level to measure the slope of any surface.

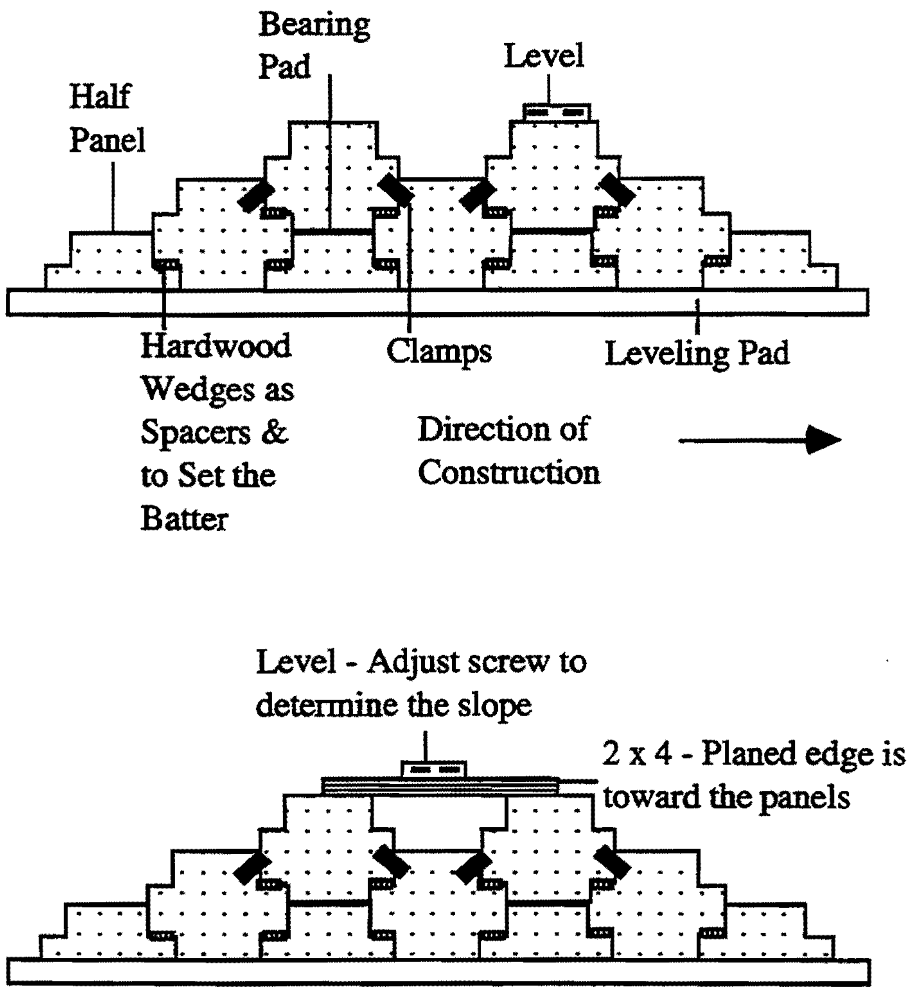


Figure 6.2 Measurement of the horizontal level of: a) individual panels and b) across two panels.

- Use the compass to monitor the azimuth change of a wall facing panel over time.
- Use the 2 x 4 along the wall panels and measure the azimuth change of the panels over time.
- Use a transit to monitor the horizontal rotation over time.

- Measure the vertical movements of the wall facing panels.
  - Visual observation.
  - Use the tape measure to monitor joint closings over time.
  - Use surveying methods to monitor this.

- Measure the horizontal movements of the wall facing panels.
  - Use the compass to monitor the azimuth change of a wall facing panel over time.
  - Use the 2 x 4 along the wall panels and measure the azimuth change of the panels over time.
  - Use a transit to monitor the horizontal movements over time.

While the above outline provides the inspector with a substantial list of measurements which could be obtained, it by no means implies that all of them should be performed all the time. For example, visual observations of the wall facing panels may indicate that a certain problem has occurred, like local vertical movements of the panels. This would call for a more detailed analysis involving measurements. In short, the above list is intended to provide the inspector with some suggestions on how various measurements could be taken to monitor the retaining wall system over time. The specific measurements which need to be taken are indicated in the sections below, which closely follows the construction sequence (see Part I, Chapter 4).

Foundation Soil

Material Property Evaluation

- See Part II, Chapter 5.

Excavation and Compaction of the Foundation Soil

- Monitor the excavation of the foundation soil to ensure that the proper depth is achieved.
- If the foundation soil is over-excavated, then a replacement material should be added and compacted until the proper grade is achieved.

- Density measurements should be taken of the foundation soil after it has been compacted to verify that the proper bearing capacity is achieved.
- **The evaluation of the foundation soil and control of the site preparation are critical to the stability of the retaining wall system**

Leveling Pad Construction

Pouring of the Leveling Pad

- Make sure the concrete is to specification and is uniform.
- Make sure the leveling pad is allowed to cure for ~ 12 hours.
- Make sure the leveling pad is placed along the correct line and is of the dimensions indicated on the plans.
- **Care taken during the construction of the leveling pad will help prevent misaligned panels as the retaining wall structure is built.**
- **Note: During the course of preparing the foundation soil and pouring the leveling pad, care must be taken to ensure that inclement weather (i.e. rain) does not soften the foundation soil, both under the retaining wall and in the area where the leveling pad is to be poured. To ensure that rain water does not pool or pond on the foundation soil adequate drainage must be provided by the contractor or the foundation soil could be covered with plastic. In the event the foundation soil does get wet, the soil should be removed or allowed to dry and recompacted.**

Inspection of Materials

- See Part II, Chapter 5.

Wall Facing Panel Installation

First Row of Panels

- Check to verify that the first row of panels are placed directly onto the leveling pad.
- Make sure that the panels are properly propped into place and are not loose.

- The props or braces should not be removed until the backfill soil has been installed behind the wall facing panel.
- Check the batter of the (see discussion above for a method of measurement), it should be 1:40 - 1:100 (horizontal:vertical). The actual value is obtained from experience and depends upon the backfill soil and the compaction. After compaction the panels should be near vertical.
- Verify the horizontal alignment and horizontal level of the panels (see discussion above for a method of measurement).
- Verify that the panels are installed as specified by the manufacturer and on the plans.
- Check the installation of the geotextile or filter fabric over the joints to make sure it is to specification, covers the joint and is not damaged.
- **Extra care should be taken to ensure the first row of wall facing panels are installed to specification. Otherwise, the stability of the wall may be greatly affected.**
- **In the event that it rains, the alignment of the wall panels should be rechecked to ensure that no movement has occurred.**

Placement and Compaction of the Backfill Soil

Placement of the Backfill Soil

- **Verify that the backfill soil conforms with the specifications (See Part II, Chapter 5).**
- **The backfill should be dumped at a distance greater than 1.52 m (5 ft) away from the wall panels. The backfill should be spread both parallel to the wall panels by earthmoving equipment and towards the panels by hand.**
- **Make sure the embankment is constructed at the same rate as the backfill soil and is interfingered with the backfill soil along the boundary between the two.**
- Check the batter and horizontal alignment of the wall face panels after the backfill soil has been placed to ensure that the panels did not move.

Compaction of the Backfill Soil

- **Compaction Must Adhere to the Specifications for both density and moisture content.**
- **Compaction tests should be taken throughout the backfill and should be performed regularly. The minimum testing frequency should be 10 tests for every two lifts.**
- **Do not use a vibratory roller near the wall panel face or anywhere close to it. It will cause the panels to move.**
- **During compaction the batter and alignment of the panels should be checked.**
- **At the end of each day, the backfill soil and the embankment should be graded in a manner that keeps water from flowing towards the wall face panels.**

Installation of Reinforcing Strips

- Verify that the reinforcing strips are as specified (see Part II, Chapter 5).
- **Make sure the reinforcement connections are tight and not loose. Also make sure the reinforcements are pulled tight and laid flat on top of the compacted backfill soil.**
- **Make sure that heavy earthmoving equipment with metal tracks does not drive over the exposed reinforcement, because they could become damaged. Rubber-tired earthmoving equipment can drive on the reinforcement, provided care is taken in doing so. If the reinforcing strips become damaged during construction they should be replaced by the contractor.**

Installation of the Second and Subsequent Wall Facing Panels

- Placing a wall facing panel onto one that has not been completely backfilled is prohibited.
- The vertically of the wall, batter and wall alignment should be checked after a new row of wall facing panels have been placed.

- **The quality control measures for the backfill placement, compaction, geotextile installation, reinforcement installation and connection are the same as outlined previously.**

Erection Tolerances

- Make sure the construction adheres to the erection tolerances listed in Table 6-1.

Table 6-1

Batter	1:40 to 1:100 (horizontal:vertical)
Horizontal Alignment	± 1:200 (outward:horizontal)
Verticality	± 1:250 (horizontal:vertical)
Bulging and Bowing	± 1:250 (horizontal:vertical) or (outward:horizontal)
Steps at joints	± 10 mm (± 0.4 in)
Alignment along top of panels	± 15 mm (± 0.6 in) from reference alignment

(adapted from Jones, 1996)

Serviceability Limits

- Make sure the construction adheres to the serviceability limits listed in Table 6-2.

Table 6-2

Limits on the post construction vertical internal strain	
Structure	Strain (%)
Bridge Abutments	0.5
Wall Systems	1.0

(adapted from Jones, 1996)

Vertical Internal Strain = {(final height - initial height)/initial height} x 100%.
Thus, the allowable vertical displacement is given by:
Allowable Vertical Displacement = {(Strain %)/100} x initial wall height.

Examle:

Wall Height = 7.62 m (25 ft)

Allowable Vertical Displacement:
Bridge Abutments = 3.81 cm (1.5 in)
Wall Systems = 7.62 cm (3 in)

Notice how restrictive the allowable vertical displacements are, especially for bridge abutments. This emphasizes the importance of qualitiy control, particularly in the region near bridge abutments.

Construction Monitoring Check List

* Monitor the excavation of the foundation soil, to ensure it is to the proper depth and is suitable for construction on.

* Make sure the concrete for the leveling pad is to specification, uniform and allowed to cure for ~ 12 hours.

* Make sure the leveling pad is placed along the correct line and is of the dimensions indicated on the plans.

* Make sure the first row of panels are properly supported and placed directly onto the leveling pad.

* Verify the batter and horizontal alignment of the first row of panels.

* Check the installation of the geotextile or filter fabric to ensure it conforms to specification and is not damaged.

* **Verify that the backfill soil conforms to the specifications.**

* Observe that the backfill soil is placed away from the wall and is spread by hand towards the wall face.

* **Compaction must adhere to the specifications for density and moisture content.**

* **Verify the grading of the backfill and the embankment at the end of the day, which should be such that water flows away from the wall facing panels.**

* Check the batter and alignment of the panels during compaction to make sure they are within tolerance.

* **Make sure the embankment is constructed at the same rate as the backfill soil and is interfingered with the backfill soil along the boundary between the two.**

* Verify that the reinforcement strips are as specified.

* **Make sure the connections are tight and the reinforcements are pulled taught over the compacted backfill soil.**

* **Make sure that heavy construction equipment does not drive over the reinforcements, because they could damage them.**

* Make sure the construction adheres to the erection tolerances and serviceability limits.

PART II. CONSTRUCTION INSPECTION

Chapter 7. Problem Solving on Construction

Introduction

The most prominent problems encountered during construction of Mechanically Stabilized Earth Retaining Wall Systems arise from the foundation soil, facing panel installation, backfill properties and installation, compaction of the backfill and controlling water and drainage. Each of these directly influences the long term performance of the Mechanically Stabilized Earth Retaining Wall System. Therefore, it is important to understand and know how to identify these problems before they occur. This chapter describes each of the various problems, their effects on the retaining walls, the possible causes of the problems and recommendations for avoiding the problems. The discussion is general in nature, since the conditions on a construction site depends upon the retaining wall system used, the contractor, the backfill soil and the local soil conditions.

I. **General Problem - Foundation Soil**

Specific Problem - Soft, wet, unstable subsoil.

Effect on the Retaining Wall - Difficulty in keeping the wall near vertical during construction and distortion of the wall after construction.

Possible Cause - Consolidation of the subsoil, both during and after construction.

Suggested Remedy of the Cause - Remove or replace the unsuitable soil and/or preload the subsoil prior to construction of the wall.

Subsoil conditions are important in all retaining wall constructions. Mechanically Stabilized Earth Retaining Wall Systems are better suited than others for accommodating differential settlements. However, there are limits to what the wall can withstand without encountering significant problems. When soft, wet and unstable soil conditions are encountered consolidation is a problem. If possible, it is important to follow TxDOT specifications which state that "any foundation soil found to be unsuitable shall be removed and replaced". This is not always possible, since the thickness of the unsuitable material may be such that removal is not an option. In this case it is important to preload the area where the wall is to be built by placing a temporary fill on it and leaving it for period of time so the subsoil can consolidate prior to the wall being constructed.

II. **General Problem - Facing Panel Installation**

Specific Problem - Out of specification panels.

Effect on the Retaining Wall - Difficulty in keeping the wall near vertical during construction and assuring proper joint spacing between the panels.

Possible Causes - A lack of inspection of the panels by the contractor and/or engineer.

Suggested Remedy of the Causes - The contractor should inspect every panel to ensure that it meets all of the specifications.

Mechanically Stabilized Earth Retaining Wall Systems are built to be flexible and be able to undergo various types of movements, both during and after construction. This assumes that a proper spacing between the panels has been maintained. The spacing between the panels is a function of the panels and the proper shimming between the panels (see below). Incorrect sized panels or damaged panels (see below), especially the bottom panels, make it difficult to maintain the proper spacing required to keep the wall near vertical and stable. All panels should be inspected for out of specification panels, which should be rejected by the contractor and/or the engineer, should be rejected.

Specific Problem - Improper alignment of the panels.

Effect on the Retaining Wall - Tilting of the wall

Possible Causes - Unsuitable propping of the panels or a lack of time spent aligning the panels.

Suggested Remedy of the Causes - The engineer and contractor should assure that the propping of the panels is done properly such that the panel alignment can be maintained. Additionally, the alignment of the panels, both horizontally and vertically, should be checked and monitored throughout the construction of the wall.

The alignment of a Mechanically Stabilized Earth Retaining Wall System is dependent upon the proper propping of the panels, the proper care in setting each panel in place and the joint spacing between the panels. The panels must be aligned both vertically and horizontally. Each successive level of panels depends upon the level below them, so it is of critical importance to ensure that the first level of panels be placed carefully. Extra time should be taken to ensure that the first level is placed properly. Otherwise, the wall has no chance of being installed properly and the stability of the wall may be greatly affected. The contractor and the engineer should check to make sure the panels are properly propped in place and also measure the alignment of the wall. The alignment of the wall, both the vertical and horizontal, can be checked easily using simple equipment (see Part II, Chapter 6).

Specific Problem - Failure to account for and monitor the batter of the panels.

Effect on the Retaining Wall - The wall may lean out or lean in excessively.

Possible Causes - Contractor neglecting the batter of the panels and a lack of inspection.

Suggested Remedy of the Causes - The batter of the panels should be measured at every level of panel installation by the contractor and should be checked by the engineer.

The construction of Mechanically Stabilized Earth Retaining Wall System involves placing the facing panels on the wall and propping them into place until the backfill soil has been placed and compacted. As the backfill is placed and compacted an outward force is placed on the panels from the backfill soil causing the panels to move or rotate outward away from the backfill. If this movement is not accounted for the wall may lean out excessively, up to 31 - 38 cm (12 to 15 in), causing both stability and aesthetics problems. Therefore, the contractor should account for this by setting the panels with an initial batter, such that when the wall is completed the face panels are vertical. The amount of the initial batter depends upon the Mechanically Stabilized Earth Retaining Wall System used, backfill used, compaction method used, etc. To overcome the effects of tilt from backfilling and compaction it is suggested that the batter be set between 1 and 40 and 1 and 100 (i.e. 1 inch of inward tilt for every 40 inches or 100 inches of vertical measurement). As the panels are backfilled and compaction proceeds, the vertical alignment of the wall should be monitored. Measuring the batter of the wall can be done using simple equipment (see Part II, Chapter 6).

Specific Problem - Large gaps between the panels.

Effect on the Retaining Wall - Alignment problems with the wall and potential zones where washout can occur (see Water and Drainage below).

Possible Causes - Improper propping of the panels and/or improper spacing between the panels.

Suggested Remedy of the Causes - The panels should be propped sufficiently so as to not allow any rotation or large scale movement during backfilling and compaction. Shims should be used to achieve the proper joint spacing of the panels until the backfilling is completed.

During construction of the Mechanically Stabilized Earth Retaining Wall System the panels are propped up and shims are used to provide the proper spacing between the panels to accommodate movement of the wall both during and after construction. Improper spacing of the panels can lead to either large gaps between the panels or damaged panels (see below). Both of which can alter the stability of the wall. During construction it is important to use the wall manufacturers shims to ensure proper spacing between the panels and to properly support the wall using props. Otherwise, large gaps between the panels could occur due to movement and/or the use of improper shims. These gaps can then become zones where the backfill soil can be washed out due to water movement through the backfill soil, which would lead to the creation of voids behind the wall and stability problems.

Specific Problem - Cracked or chipped panels.

Effect on the Retaining Wall - Alignment problems and zones of potential washout.

Possible Causes - Mishandling of the panels, improper use of shims and improper propping of the panels.

Suggested Remedy of the Causes - The panels should be handled carefully and inspected prior to installation. During installation of the panels the wall manufacturer shims should be used to ensure proper spacing between the panels and the propping should be such that large movements of the panels does not occur.

As stated above proper propping and shimming of the panels is important not only for stability of the wall, but is also important for keeping the panels from becoming damaged due to direct contact with one another from movement during and after construction. In addition, the panels should be handled carefully so as not to damage them during lifting by a crane and installation. It is the contractors job to follow the wall manufacturers suggestion for handling and installing the panels. Otherwise, wall alignment and stability could be compromised, which could lead to the zones of washout in the backfill (see Water or Drainage Control below).

Specific Problem - **Torn or damaged filter fabric along the joints between the panels.**

Effect on the Retaining Wall - Wall alignment problems due to potential washout zones within the backfill (see Water or Drainage Control below).

Possible Causes - Movement and rotation of the panels, and improper backfilling.

Suggested Remedy of the Causes - The panels must be properly spaced and battered to reduce the amount of movement and rotation. Backfilling should not be done such that the soil is dumped against the back of the facing panels, it should be spread into that zone by hand.

A geotextile or filter fabric is used to cover the joints along the wall on the backside of the panels. This is done to try and keep the backfill soil from washing out through the joints between the panels. During construction the geotextile or filter fabric often becomes damaged by buckling, formation of laps and backfilling. The movement and rotations of the panels is one cause, so proper propping and spacing of the panels is a must. An additional cause is the backfilling of the select fill. Often the select fill is dumped into place with heavy machinery and is done such that the soil actually impacts the backside of the panels in the wall. This causes the geotextile or filter fabric to become damaged and possibly torn. Damaged geotextile or filter fabric can weaken over time leading to areas where the select backfill can actually work its way through the area and produce a washout zone and a void behind the wall. It is important to properly install the geotextile or filter fabric and to monitor both the alignment and spacing between the panels, as well as the propping of the panels and the backfilling procedure of the select fill.

Specific Problem - Excessive slack in the earth reinforcement.
Effect on the Retaining Wall - Outward leaning of the wall.
Possible Cause - Loose connection between the reinforcement and the facing wall panel.
Suggested Remedy of the Cause - Use a small wedge shaped wooden shim to keep the connection tight during construction.

Mechanically Stabilized Earth Retaining Wall Systems are a composite soil structure, utilizing reinforcement to enhance the stability of the backfill and to support the concrete facing panels, which retain the backfill. The connection between the reinforcement and the facing panels is critical in keeping the facing panels in position. However, some times care is not taken in the manner in which the connections are made, slack can occur which allows the panels to rotate and move excessively during backfilling and compaction. To prevent this from happening a small wedge shaped wooden shim can be used to keep the connection tight during construction and minimize the movement of the wall during construction.

Table 7-1 summarizes of the problems listed above, their effect on the retaining wall and the possible cause(s) of the problem. This table provides the inspector with an overview of this section at a quick glance.

III. General Problem - Backfill Soil Installation
Specific Problem - Non-uniform backfill material.
Effect on the Retaining Wall - Wall movement (either leaning out or leaning in), wall rotation and differential movement along the wall.
Possible Causes - Switching of the backfill material by the contractor, a change in soil type within the borrow location and mixing of the backfill soil with the fine grained soil of the embankment.
Suggested Remedy of the Causes - The backfill soil should periodically tested to ensure the uniformity and quality of the backfill.

TABLE 7-1

GENERAL PROBLEM	EFFECT ON RETAINING WALL	POSSIBLE CAUSE(S) OF THE GENERAL PROBLEM
II. FACING CONSTRUCTION PROBLEMS	A. Gaps between the	- Improper propping of the panels
	panels (could lead to washout	- Improper spacing between the panels
	problems and	- Out of specification panels
	voids)	- Improper alignment of the panels
	B. Torn or Damaged Filter Fabric	- Improper installation of the filter fabric
	(could lead to	- Improper propping of the panels
	washout zones)	- Improper backfilling procedures
	C. Distortion and Leaning of the	- Improper propping of the panels
	Wall	- Damage to the reinforcing strips during compaction
		- Improper connecting of the reinforcing strips to the panels
		- Out of specification panels
		- Improper alignment of the panels
		- Failure to account for and monitor the batter of the panels
		- Excessive slack in the reinforcement due to loose connections
	D. Cracked or Chipped Panels	- Improper handling of the panels
	(could lead to washout zones)	- Improper spacing between the panels
		- Improper use of shims
		- Improper propping of the panels

Specific Problem - Poor quality of the backfill material.

Effect on the Retaining Wall - Wall movement (either leaning out or leaning in), wall rotation and differential movement along the wall.

Possible Causes - Switching of the backfill material by the contractor, a change in soil type within the borrow location and mixing of the backfill soil with the fine grained soil of the embankment.

Suggested Remedy of the Causes - The backfill soil should periodically tested to ensure the uniformity and quality of the backfill.

During construction and after construction the stability and movement of the wall face is dependent upon the backfill soil. It is the backfill soil that is at the key to the simplicity and effectiveness of the Mechanically Stabilized Earth Retaining Wall System. Therefore, it is important to assure that the backfill material is uniform and of proper quality during construction of the wall. Otherwise, the compaction specifications may be incorrect for the soil, which could lead to overcompaction and the wall leaning outward or undercompaction and the wall leaning inward. The backfill should be tested periodically throughout the construction of the wall, especially if a change in texture, consistency or color is noted by the engineer. An increase in the amount of fines in the backfill, which could be washed into the backfill from the embankment (see Water and Drainage below), can cause a variety of problems.

Table 7-2 summarizes of the problems listed above, their effect on the retaining wall and the possible cause(s) of the problem. This table provides the inspector with an overview of this section at a quick glance.

IV. General Problem - Compaction of the Backfill

Specific Problem - Damage to the reinforcing strips.

Effect on the Retaining Wall - Wall leaning inward.

Possible Causes - Overcompaction of the backfill soil, improper lifts used during compaction and the driving of heavy machinery on top of the reinforcement.

Suggested Remedy of the Causes - Ensure that the proper lift thicknesses are used prior to compaction and that the contractor does not overcompact the soil or drive heavy machinery on top of the reinforcement.

TABLE 7-2

GENERAL PROBLEM	EFFECT ON RETAINING WALL	POSSIBLE CAUSE(S) OF THE GENERAL PROBLEM
III. BACKFILL	A. Wall Leaning Out	- Backfill material contains
PROPERTIES		excessive fines
		- Backfill material saturated by
		heavy rain
		- Backfill material is not uniform
		- Poor quality of backfill material
	B. Wall Leaning In	- Backfill material is not uniform
		- Poor quality of backfill material
	C. Differential	- Poor quality of backfill material
	Settlement of Wall	- Backfill material not uniform
		- Free draining backfill allows
		subsoil to undergo
		consolidation
		- Poor quality of backfill material
	D. Rotation of the	- Poor quality of backfill material
	Wall	Backfill material is not uniform

During compaction of the backfill, the reinforcement is placed within the backfill and is covered with a lift of backfill soil prior to compaction. To accomplish this often the contractor drives heavy machinery on top of the reinforcements, which damages them. Additionally, once the reinforcements are covered and compaction is progressing overcompaction and the use of improper lift thicknesses prior to compaction can cause damage to the reinforcement. Damage to the reinforcement causes the an excessive inward pull to the panels and causes the wall to lean inwards. To effectively spread and compact the backfill on top of the reinforcements the contractor should adhere to the TxDOT specification for 20.32 cm (8 in) lift thickness and should spread the backfill soil with either out driving on them or using very light weight machinery. During compaction, the density of the backfill soil should be monitored to ensure that overcompaction does not occur.

Specific Problem - **Excessive or overcompaction of the backfill soil.**

Effect on the Retaining Wall - Wall leaning outward.

Possible Causes - Use of compaction equipment to close to the wall face, use of vibratory rollers for fine sands and improper lift thickness.

Suggested Remedy of the Causes - Wall alignment and backfill density should be measured and monitored.

Contractors are often free to choose which compaction machinery they want to use to compact the backfill soil. This often leads to problems of overcompaction, because the machinery used and/or the number of times a roller goes over the soil causes the density to exceed the required density. A consequence of this is that high lateral stress are then transmitted to the face of the wall and causes the wall to lean or bow outward away from the backfill. A case in point, is the use of vibratory rollers on fine sands. It is easy with vibratory rollers to overcompact the soil and cause outward leaning of the wall. A direct consequence of this is that the specifications from some wall manufacturers do not allow the use of vibratory rollers on fine sands. Therefore, it is important to monitor both the wall alignment and the backfill density throughout the construction of the Mechanically Stabilized Earth Retaining Wall System.

Specific Problem - **Inadequate compaction of the backfill soil.**

Effect on the Retaining Wall - Wall leaning inward.

Possible Cause - Contractor can not get heavy machinery into an area to compact the soil.

Suggested Remedy of the Cause - Wall alignment and backfill density should be measured and monitored. Hand operated compaction machinery should be used.

Specific Problem - **Non-uniform compaction of the backfill soil.**

Effect on the Retaining Wall - Differential movement of the wall.

Possible Cause - Contractor can not get heavy machinery into an area to compact the soil.

Suggested Remedy of the Cause - Wall alignment and backfill density should be measured and monitored. Hand operated compaction machinery should be used.

A common problem that contractors have in compacting the backfill soil is that quite often they either can not use or can not get the compaction machinery into an area to compact the soil. For example, the zone within 0.92 m (3 feet) of the wall face is supposed to compacted with hand operated machinery and the zone around bridge abutments is often too tight to get heavy machinery into. In these areas, the soil is very often undercompacted, if compacted at all, and voids develop causing the wall panels to move or differential movement to occur (i.e. around bridge abutments). TxDOT specifications for compaction require that within 0.92 m (3 feet) of the wall face that hand operated compaction equipment be used. This should be monitored to ensure proper compaction of the soil in that zone. Similarly, the same should hold true for the area around bridge abutments. To ensure that the compaction process is not excessive from the use of the hand operated machinery the wall alignment and backfill density should be monitored and measured throughout the construction of the wall.

Specific Problem - The driving of heavy machinery within 1.52 m (5 feet) of the face of the wall.

Effect on the Retaining Wall - Wall leaning outward.

Possible Cause - Improper care by the contractor

Suggested Remedy of the Cause - Machinery should not be operated within 1.52 m (5 ft) to 3.04 m (10 feet) of the face of the wall.

This is related to the damage to the reinforcement and damage to the geotextile (filter) fabric discussed above. During backfill of the soil it is not uncommon for heavy machinery to be driven within 1.52 m (5 feet) of the face of the wall. Since the panels are subject to movement, because tension in the reinforcement does not occur until the backfill soil is compacted, it is imperative to keep the heavy machinery away from the wall face at all times during construction, because outward movements of 5.08 cm (2 in) to 25.4 cm(10 in) of the walls have taken place. Often, the very top panels are subjected to extreme movement as the road surface is being prepared.

Specific Problem - Excessive water within the backfill soil.

Effect on the Retaining Wall - Wall leaning outward.

Possible Causes - The backfill soil being compacted too wet or rain water ponding on the backfill soil.

Suggested Remedy of the Causes - Monitoring of the moisture content of the backfill soil and proper grading of the backfill (see Water or Drainage Control below).

Excessive water within the backfill can lead to greater lateral forces on the wall due to the water pressure and lateral displacements of the soil during compaction. For example, sand backfill soils can undergo large lateral displacements when they are compacted in a near saturated state. This is especially true of fine sands. The large lateral deformations induce outward displacements of the panels and cause the wall to lean outward. To avoid this from happening the moisture content of the backfill soil should be monitored during compaction and rain water should be kept from ponding and running into the backfill (see Water or Drainage Control below). After a heavy rain the contractor should wait some time before working on the backfill to allow the water to flow out of the backfill soil.

Table 7-3 summarizes of the problems listed above, their effect on the retaining wall and the possible cause(s) of the problem. This table provides the inspector with an overview of this section at a quick glance.

V. **General Problem - Water or Drainage Control**

Specific Problem - Improper grading of the backfill soil and embankment.

Effects on the Retaining Wall - Wall movement, leaning outward, leaning inward or differential wall movement

Possible Causes - Allowing water to flow into the backfill and towards the face of the wall.

Suggested Remedy of the Causes - The contractor should properly grade the backfill and the embankment at the end of each day and if needed should cover the backfill soil with plastic to keep the rain water out.

Wall Leaning Out - Various factors could be responsible for the wall leaning outward, such as: 1) excessive pore pressures in the backfill acting on the wall; 2) consolidation settlement of the backfill soil upon saturation due to inadequate compaction; and 3) the washing of fine grained material form the embankment into the backfill creating zones of inhomogeneity within the backfill. Each of these could be reduced and/or avoided if the TxDOT specifications were rigidly adhered to, which state that the contractor shall be responsible for preventing surface water or rainwater from damaging the retaining walls during construction. The TxDOT specifications also state, at the end of each days operation the contractor shall shape the last level of backfill to permit runoff of rainwater away from the face of the wall. In addition, the TxDOT specifications also state that the contractor shall be responsible for maintaining the stability of the interface area between the embankment and the backfill. These are probably the most overlooked statements in the specifications, yet have some of the most significant influence on the behavior of the wall.

Wall Leaning In -- Various factors could be responsible for the wall leaning in, such as: 1) voids left behind from consolidation settlement of the backfill upon saturation and 2) washout of the backfill forming voids behind the wall. Both of these could be prevented with proper compaction, proper placement of the filter fabric, proper panel spacing and keeping water from running into the backfill towards the face of the wall (see discussion above).

TABLE 7-3

GENERAL PROBLEM	EFFECT ON RETAINING WALL	POSSIBLE CAUSE(S) OF THE GENERAL PROBLEM
IV. COMPACTION	A. Wall Leaning Out	- Compaction of the backfill
		within 3 feet of the wall
		- Overcompaction or excessive
		compactive effort
		- Excessive vibratory compaction
		of uniform fine sands
		- Backfill material placed wet of
		optimum water content
		- Driving of heavy machinery within 1.52 m (5 ft) of wall face
	B. Wall Leaning In	- Inadequate compaction of
		backfill
		- Collapse of voids in the backfill
		- Damage to the reinforcement
	C. Differential	- Backfill material not uniformly
	Settlement of Wall	compacted
		- Collapse of voids in the backfill
	D. Damage to the	- Excessive compactive effort
	Reinforcement	used on the backfill
		- Lift thickness not thick enough
		above the reinforcing strips

Differential Wall Movement -- The most common cause of differential wall movement is the inhomogeneity of the backfill material caused by fine grained material from the embankment being washed into the backfill soil during a rain storm This intermixing of soils is usually caused by the fact that the contractor constructs the embankment first and then the wall. So what happens is you have a high area of fine grained material with the backfill soil being built up to meet it. Consequently, when it rains the fine grained material washes straight into the backfill causing it to be non-uniform, especially in those areas where the backfill is lowest. This should not be allowed to happen and could be prevented if the embankment was built at the same rate as the backfill and proper grading of the embankment and backfill material were performed. Additional causes of differential wall movement include local consolidation settlement zones, which produce local voids behind the wall, and local wash out zones.

Table 7-4 summarizes of the problems listed above, their effect on the retaining wall and the possible cause(s) of the problem. This table provides the inspector with an overview of this section at a quick glance.

TABLE 7-4

GENERAL PROBLEM	EFFECT ON RETAINING WALL	POSSIBLE CAUSE(S) OF THE GENERAL PROBLEM
V. WATER OR	A. Wall Leaning Out	- Excessive pore pressure in the
DRAINAGE		backfill acting on the wall
		- Consolidation settlement of the
		backfill upon saturation
		- Improper grading of the
		backfill
		- Intermixing of embankment
		soil with the sand backfill
	B. Wall Leaning In	- Washout of the backfill creating
		voids behind the wall
		- Voids left by consolidation
		settlement of the backfill
		upon saturation
		- Improper grading of the
		backfill
	C. Differential	- Intermixing of embankment
	Settlement of Wall	soil with the sand backfill
		- Improper grading of the
		embankment and backfill
		- Local washout zones in the
		backfill creating voids
		behind the wall
		- Local consolidation settlement
		of the backfill upon
		saturation creating voids
	D. Backfill material	- Intermixing of embankment
	is not uniform	soil with the sand backfill
		- Improper grading of the
		embankment and backfill

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GLOSSARY OF TERMS

Backfill is soil that is placed behind the wall facing elements of a Mechanically Stabilized Earth Retaining Wall System. Normally the backfill will be placed and compacted to a minimum specified density and at a specific water content.

Bearing Pad is a compressible pad made out of rubber, foam or cork and is used along the joints between the wall facing panels to prevent them from touching.

Coarse Grained Soil/Granular Soil is a soil with more than 50 percent of the material larger than the Number 200 sieve size or 0.0075 mm.

Composite Material is a material composed of soil and horizontal reinforcements.

Creep is the time dependent strain under constant stress that develops by the viscous resistance of the soil.

Earth Reinforcement/Soil Reinforcement is the process of increasing the strength of a soil mass by inserting horizontal reinforcements into the soil mass.

Elastic Movement is the recoverable movement of the soil due to an applied load.

Extensible Reinforcement is a reinforcement within the soil that has a deformation at failure equal to or greater than that of the soil.

Facing Element/Wall Facing Element/Wall Facing Panel/Facing Panel/Panel is the component of the retaining wall system to prevent local ravelling and deterioration of the soil. These are usually precast concrete panels of various shapes and sizes, with different types of architecual face styles.

Filter Fabric/Geotextile is a synthetic fabric used as a filter material over joints to prevent the movement of soil through the wall.

Foundation Soil is the insitu soil on which the retaining wall system is to be built.

Grid/Mesh Reinforcement is composed of longitudinal members running perpendicular to wall facing panels and is transversed by members which are parallel to the wall facing panels, both of which are connected together to form a rectangular grid or mesh.

Inextensible Reinforcement is a reinforcement within the soil that has a deformation at failure much less than that of the soil.

Mechanically Stabilized Earth Retaining Wall System is a structural composite system, consisting of a wall face and a reinforced soil mass behind the wall, which provides vertical, or near vertical, grade separation at the ground surface. The reinforced soil mass utilizes horizontal reinforcements, which are attached to the wall face and are in the form of metallic or non-metallic grids or strips, to stabilize and strengthen the soil. The wall facing is not used for primary structural support, rather it is used to prevent local raveling and deterioration of the soil. Both the wall face and the reinforced soil mass are built incrementally, starting at the bottom and proceeding to the top.

Modulus refers to the elastic stiffness of the material.

Reinforcing Element is an inclusion within the soil, with the purpose of increasing the shear strength of the soil through friction and/or passive resistance. These come in various material types, shapes and sizes.

Select Backfill is a granular soil of high quality and with a smal amount of fines that is placed behind the wall facing elements of a Mechanically Stabilized Earth Retaining Wall System. Normally the backfill will be placed and compacted to a minimum specified density and at a specific water content.

Sheet Reinforcement is horizontal thin planar reinforcements.

Stiffness refers to the static stress-strain elastic modulus, which is calculated by dividing the elastic stress by the elastic strain.

Strip Reinforcement is thin, narrow linear reinforcing elements of metal or plastic and may be either ribbed or planar.