Constructing and Controlling Compaction of Earth Fills

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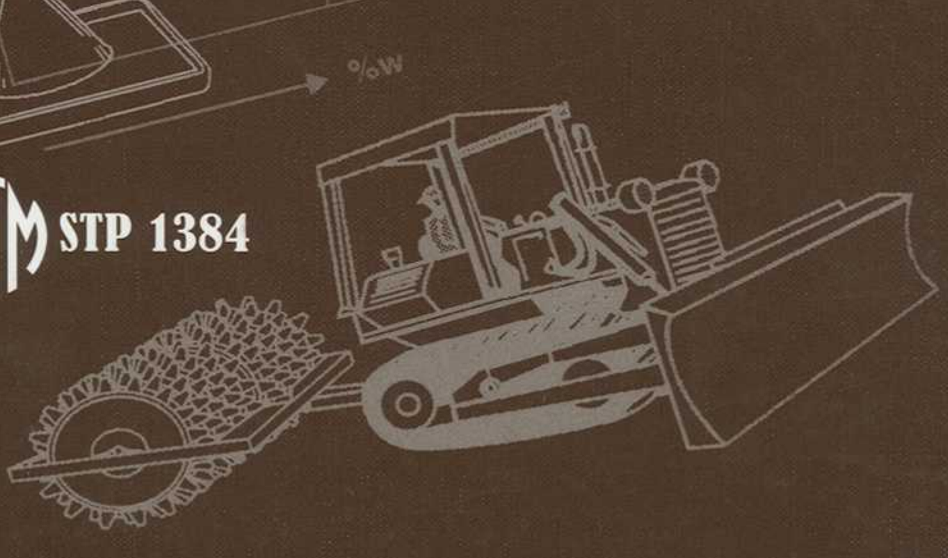

ASTM STP 1384

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Donald W. Shanklin, Keith R. Rademacher, and James R. Talbot, editors

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Each paper published in this volume was evaluated by two peer reviewers and at least one of the editors. The authors addressed all of the reviewers' comments to the satisfaction of both the technical editor(s) and the ASTM Committee on Publications.

The quality of the papers in this publication reflects not only the obvious efforts of the authors and the technical editor(s), but also the work of these peer reviewers. The ASTM Committee on Publications acknowledges with appreciation their dedication and contribution to time and effort on behalf of ASTM.

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Foreword

This publication, *Constructing and Controlling Compaction of Earth Fills,* contains papers presented at the symposium of the same name held in Seattle, Washington, on 1–2, July, 1999. The symposium was sponsored by ASTM committee D18 on Soil and Rock. Donald W. Shanklin, James R. Talbot, and Keith Rademacher presided as symposium chairmen.

Contents

Overview

This book represents the efforts of a number of authors who presented papers at the Symposium titled, *Constructing and Controlling Compaction of Earth Fills*, held in Seattle, Washington, on July 1 and 2, 1999. The book is devoted to papers written on the use of various standardized methods for specifying and controlling the compaction of soils for engineered constructed earth fills. In at least one case, a paper was accepted and written, but the author was unable to be present. The introduction to the symposium, as contained in the symposium program, offered the following information as a focus for the presentations:

> Soil is compacted to improve its performance as a structural building material. The degree of compaction, method of compaction, moisture content, and gradation of the soil materials all have an impact on the final product achieved by the process involved. ASTM has numerous test methods that address different aspects of the compaction process.
>
> It is the objective of this symposium to look at soil compaction control in construction activities from a number of perspectives. These perspectives include the historical background, current state-of-the-art practices, case histories of challenging situations, new concerns regarding appropriate design parameters for compaction control, and new methods to evaluate soil compaction and other related qualities.
>
> The final session of the symposium will feature a review and discussion of a manual currently being developed by ASTM Committee D18 on Soil and Rock, the symposium sponsor. This manual is titled, "Testing Compaction of Earth Fills Using ASTM Standards".

The symposium papers were grouped into three categories for the purpose of presentation at the symposium. These papers covered all of the topics referred to in the program introduction. The history of the development and use of the nuclear gage in the quality control of constructed earth fills was covered in the keynote address by W.F. Troxler of Troxler Electronic Laboratories, Inc. His presentation on "Development and Industry Acceptance of the Nuclear Gauge" was accompanied by a written paper. This paper has been included in this publication. The nuclear gage has had the single largest impact of any technology in the last 30 years in the field of compaction control. Many papers that followed in the symposium used the nuclear gage as a basis for comparison of the results of field density and water content measurements.

A review of the three sessions follows.

Overview of Compaction Control Technology and Comparison of Current Methods

The intent of this session was to feature state-of-the-art practices along with some general historical background. The papers presented provided good insight to both of these areas. Some of the more important aspects of compaction control and testing as practiced by the Bureau of Reclamation are presented in one paper. The Bureau has long been a leader in the field of earthwork construction. Their "Earth Manual" has been a primary reference for engineers and others involved in earthwork construction.

Other papers present comparisons of the results of some of the more commonly used methods of determining in-place densities and water content in the field. One study is a laboratory simulation comparing results of the nuclear gage, sand cone, and calibrated cylinder. Another paper presents the results of extensive actual field testing from various construction sites and a variety of locations. This field testing included comparisons of results between the nuclear gage, sand cone, calibrated cylinder, and the rubber balloon. Both the laboratory and field studies present conclusions on the inherent variability of the various types of density and water content measurement standard test methods.

These discussions should help engineers to better understand test results from the nuclear gage and the sand cone in particular.

Several other papers in this session presented valuable ways to evaluate and develop reference compaction curves for field use. One paper presented an equation for the development of a compaction curve for fine-grained soil. The other paper compiled data from several sources to produce trend curves for standard maximum density and optimum water content for some of the more common Unified Soil Classification soil types. Evaluation of the compactive effort of some of the common hand-operated equipment was also presented in this session. The final paper in this session appealed to the industry in general to take a more professional and passionate interest in quality earth fill work. This appeal was combined with a case history to illustrate some of the concern.

A highlighted emphasis of several papers in this session and following sessions of the symposium was the need to correct the water content measurements made using the nuclear gage. Some of the papers detailed the errors that can arise from using uncorrected water content measurements directly from nuclear gage readings. A few papers seemed to ignore this correction in making comparisons between methods. The need to standardize water content measurements to the oven-dry procedure was reiterated by several authors and needs to be well-understood by all users.

Applications and Lessons Learned in the Field

This session was the largest with most of the papers providing case histories with the primary emphasis on the compaction of coarse-grained materials. Different methods were used to control and verify results of the construction of fills and embankments composed of coarse-grained material. One of the concerns that surfaced in this session was the lack of guidance in ASTM and elsewhere when the percentage of coarse material exceeds 35 to 40% maximum around which the ASTM Standards are developed. Papers discussed new methods for evaluating compaction quality of fills constructed with a significant amount of the materials exceeding the 3/4-inch maximum size. An overview of some guidance provided by a federal agency in the evaluation and use of coarse-grained materials was also presented as both historical and as state-of-the-art.

Another area of concern highligted in this session was the importance of moisture control and especially the percent saturation of soil during compaction. Hydrocompression of certian plastic soils was a feature of several papers. Some similar problems were reported in a paper not presented at the symposium, but the paper is included in this STP. This paper explains problems with settlement of moderately plastic loess soil and the development of new compaction criteria to address the problem.

The understanding and use of the degree of saturation in both specification writing and construction control can lead to greatly improved quality of constructed earth fills. This principal was further emphasized in the next session by the papers concerned with soils being compacted for low permeability liners.

Both of these areas are challenging with regard to guidance in the control of construction and for the development of appropriate design practices and standards. This is a particularly challenging area for the development of new methods and standards by ASTM. The needs are clearly there and many innovative approaches are being used to satisfy those needs.

Soil Liner Construction and New Compaction Technology

The advent of the construction of safer and more sophisticated waste containment facilities has brought on a great deal of interest in the proper construction of clay liners and the impacts of various materials on the constructed liners. Several papers in this session deal with these topics. The compaction control for permeability reduction rather than merely structural strength is a different approach and needs to be recognized. This approach to compaction control emphasizes the control of water content on the wet side, and the thorough processing of soil to remove clods to minimize the size of the voids in the resulting compacted clay liner. Reduced hydraulic conductivity is the primary goal rather than strength parameters. The impacts of the various soil parameters and chemical elements and their relation to hydraulic conductivity were the topic of several papers in this session.

The other main concerns reported on in this session centered on new ways to evaluate the quality of earth fills. The evaluation of modulus as a design and construction parameter, for highways in particular, is presented along with several new techniques and new equipment to test for this parameter and the standard parameters. One of the methods involves the measure of soil stiffness to arrive at values of modulus. Another technique used magnetic waves to provide quality control data for earth fills. A final method explored the use of seismic testing devices in both the laboratory and in-field situations to measure compacted soil qualities.

The development of standards for these new testing methods will be part of future ASTM committee work.

Compaction Manual—Testing Compaction of Earth Fills Using ASTM Standards

This manual has been in the works for some time in Committee D18. It is being reviewed for final publication, but will not be available at the time that this STP is published. The intent of the committee writing this manual is to provide guidance in the overall process of designing, specifying, and constructing earth fills. The focus of the manual will be on the proper application of ASTM Standards in this earth fill process. Various factors have led many experts to believe that the current practice of earth fill design and construction is not as clearly understood as it once was. The practice has strayed from the basics established by Proctor, Terzaghi, Peck, and others, to an exercise that lacks understanding and effective quality control.

Various authors from government agencies and private industry have contributed to the manual. The manual will be an appropriate and important companion to this STP. It should produce renewed interest in achieving quality earthfill work that meets the parameters most appropriate to the designed use of the final product.

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Keynote Address

William F. Troxler Sr.[1]

Development and Industry Acceptance of Nuclear Gauges

Reference: Troxler, W. F., **"Development and Industry Acceptance of Nuclear Gauges,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D.W. Shanklin, K.R. Rademacher, and J.R. Talbot, Eds, American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Mr. William F. Troxler delivered the keynote speech at the ASTM SYMPOSIUM ON *CONSTRUCTION AND CONTROLLING COMPACTION OF EARTH FILLS,* presented on July 1, 1999 in Seattle, Washington. As an original contributor to in-situ moisture and density measurement, Mr. Troxler detailed the history of how these devices were developed and accepted in the construction industry.

Beginning with his early work for the US Department of Agriculture and NC State University School of Agriculture to develop a device to measure the water content, Mr. Troxler describes how this device was combined with a similar device that measures the density of soils to create the surface moisture/density gauges used in the highway construction industry. The first models of this device were unacceptable because of the errors created by the chemical composition and other factors. Mr. Troxler explains how the development of calibration blocks with permanent density and moisture content values improved the gauge precision to the extent that the devices were more precise than current methods employed in the industry. Additional points of interest are included to explain how the nuclear moisture/density gauge became a standard test method in the industry.

Keywords: nuclear moisture density gauge, water content, moisture content, in-situ density measurement, Troxler nuclear gauge, nuclear gauge calibration.

The "Development and Industry Acceptance of Nuclear Gauges," is a story that begins -- or at least my part in it begins -- forty years ago in the Research Triangle area of North Carolina. But before delving into the past, let me set the stage of the present.

We are living and working in a world that moves faster than we once ever imagined, a world that has become dependent on technology in almost every field. And every day, that technology becomes more advanced and complicated. Some household gadgets that are supposed to make life easier for us prove the "complicated" part for us everyday, unfortunately! In industry, what we find is that simple solutions to technical challenges are hard to come by, although almost everyone is, naturally, seeking just such solutions.

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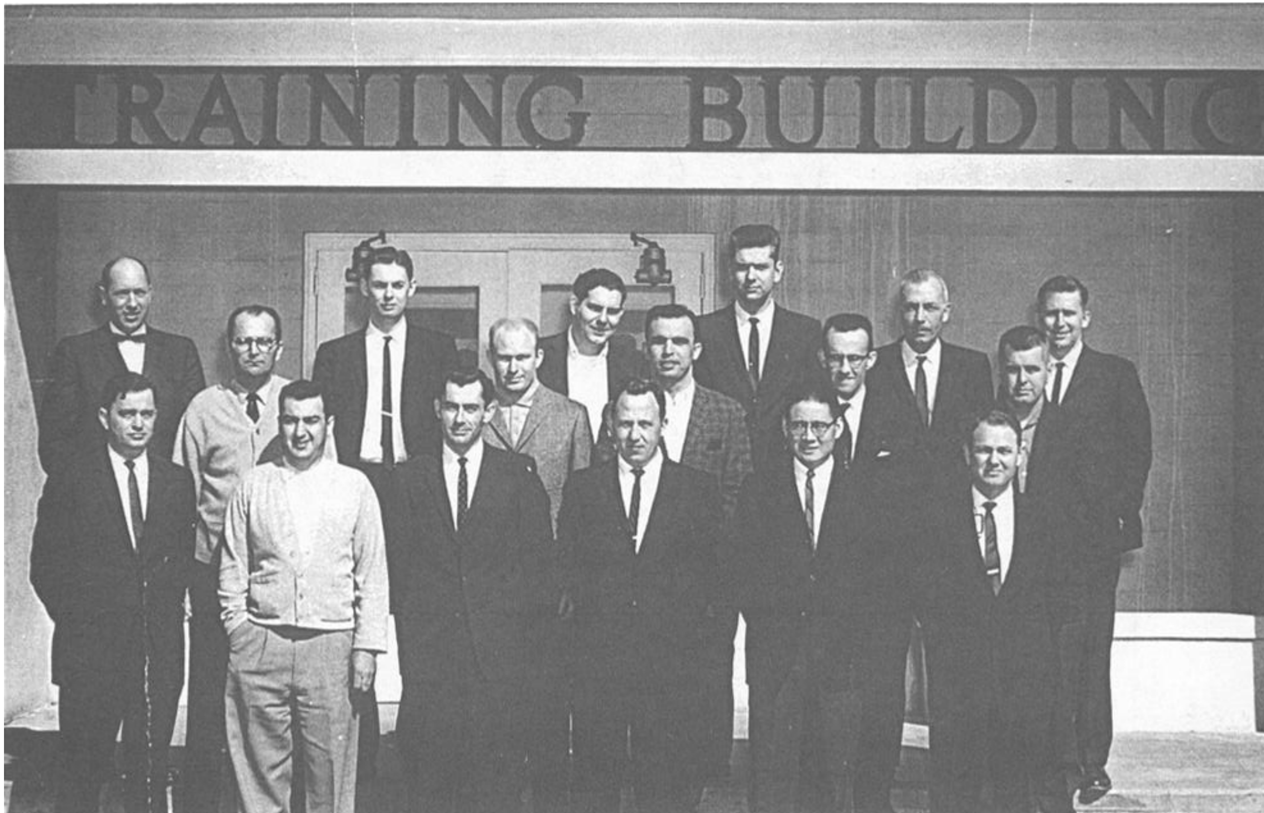

FHWA Highway Materials Engineering Course
Oak Ridge Tennessee • March 9 - 27, 1964

Rear: Lamb, Rizenbergs, Robert, Gumbert, Garner, Long (WV)
Middle: Ibert, Ehorn (IL), Worona (PA), Chastain (IL), Forsyth
Front: Gunderman (TRB), DePoe, Reynolds, Stiles, Chou, Akers

It's been fascinating over a lifetime to watch all sorts of engineering innovations arise, some that have staying power and some that don't. What underlies the innovations that succeed and eventually set standards is fundamental science. In most ways the story of Troxler Electronics and the development of nuclear gauges is a story about a commitment to fundamentals.

Wide acceptance of nuclear gauges came about in the early 1970s, when we succeeded in developing industrywide calibration standards. But, the development of these gauges started more than a decade earlier, so let's step back to the 1950s and early '60s.

At that time, highway engineers used traditional methods such as sand cones to determine the density of the soil on which they were going to build their roads. Imagine them taking their soil samples to the portable burner on the backs of their trucks, cooking the soil to get rid of its water content, and then making their mechanical measurements. It's an almost comforting scene from a less complicated, low-tech era. But as you know, it's a time-consuming process -- two hours as opposed to the five minutes it takes to use a nuclear gauge. For this reason, many engineers would use the heels of their boots to measure compaction after making their initial check. Some were quite good at it, but not all!

ASTM President James Thomas has said that there are "two basic values that should be inherent in every standard - quality and relevance." I agree with him, and I think the back-of-the-boot method certainly shows us the relevance of, in fact the need for, a new technology at the time. And if it were going to succeed, that technology would have to be of high quality.

A number of engineering firms were trying to develop quicker and better ways to test soil in the '50s and '60s. My involvement began in 1959, when I was asked by several faculty members at the NC State University School of Agriculture to develop a neutron probe to measure the water content of soils. This was a project they had been kicking around for some time. I was an alumnus of the University's engineering school and had my own small firm there in Raleigh, so I happened to be in the right place at the right time.

This was not a project that had entered my mind until the University called me. In fact, I had been toying with the idea of trying to develop some devices for the emerging field of satellite communications. I never got around to those. I had a small operation and supported myself and a couple of employees with modest government and industry contracts. When I received the call from NC State, in fact, it had been just a few years since I'd moved the company out of my basement and stopped doing television repairs on the side to feed my family.

Back in the 1950s, nuclear science was still new, and there were not many publications on the subject of non-destructive testing. I did know, however, that radiation intensity decreased with an increase in the density of a material and knew there must be ways to put this knowledge to use. The agricultural researchers gave me their prototype probe with radium-beryllium and asked me to see what I could do with it. I took the prototype back to the lab, outfitted it with a transistorized preamplifier, and it worked like a charm. We were able to measure the rate at which neutrons were thermalized by hydrogen atoms in soil samples, and thus determine water content.

The researchers at NC State encouraged me to market this device, which we called the depth moisture gauge, so I began promoting it to the U.S. Department of Agriculture's Agricultural Research Stations around the country and to other likely groups. This, of course, resulted in it being tested and written about by others, and publicity started to build without a great deal of marketing on my part. We had simply developed a better way to do things those engineers were already doing, so we really had a built-in market.

Soon we used similar technology to build a device that would measure the density of soils. For this device, as you know, we use gamma rays. Later, of course, we combined the two gauges in one device. Our research proved to us that great improvements in field-test results with soils and aggregates were made possible by the direct-transmission test mode that we pioneered.

This technology was a breakthrough, although the early density gauges, especially, needed some refinement. At first we were able to measure density accurately to plus-or-minus 15 pounds per cubic foot, which is not accurate enough for most roads I'd like to drive on. Word about these devices and other work in the field got around quickly, nonetheless, and in 1964 the Federal Highway Administration called a meeting in Oak Ridge, Tennessee, to talk about this technology, among other issues. We extended this meeting for a couple of days so both private and public researchers could meet in nearby Gatlinburg to discuss nuclear gauges at length. The accompanying photo shows industry leaders attending the conference.

We all agreed that these devices -- some developed by us, others by Nuclear-Chicago, Cornell University and NUMEC -- must become more accurate. They needed a precision of plus-or-minus 1% rather than being within the 15 pounds per cubic foot, which was more than 10%. We also needed to establish industrywide calibration standards if these devices were to become universally accepted.

Before beginning part two of this three-part story, I should say a word about patents. We did not patent any of our early work, but a couple of limited patents did apply to the work of others. Cornell University had received a patent in 1957 for a flat-bottom box using the principle of backscatter geometry, but the patent did not cover direct transmission geometry. Nuclear-Chicago would receive a 1966 patent that covered the shielding of the radioactive isotope in its gauge. The direct transmission test mode that we pioneered was not covered by either of these. Sometimes I look back and wonder why we didn't patent our ideas, but I suppose we did well enough.

Now let's move on to part two.

We had to come up with a way to calibrate gauges that was easily understandable to the users, could be replicated easily and would stand the test of time. We had to come up with a way for engineers to adjust their devices to account for nuclear degradation on an annual basis, and we had to set a standard that could be used on devices from various manufacturers. Otherwise we would have the equivalent of some engineers using "Apples" and others using "IBMs," with all the translation problems associated with the early days of personal computers.

We first set out to establish a standard for density. We knew that most highway material has a density of 120 to 164 pounds per cubic foot, and we knew that we had to establish three known quantities for the density equation $R_d=Ae^{-BD}-C$, in which A, B and C are constants. We needed to find materials with known densities that would "bracket" the density range of the highway material, so that, in effect, we could "tune" our gauges to these known factors.

To set a standard at the upper end, we turned to extruded aluminum blocks about 41 inches long, 24 inches wide, and 30 inches deep, with a density of 164 pounds per cubic foot. It should be a closed-cell material, so barring some sort of unforeseen catastrophe; the density is not going to change.

For the bracket at the other end of the continuum, we chose magnesium, a closed-cell material with a known density of 110 pounds per cubic foot. We made similar blocks of this.

All that remained to do was combine these materials for a mid-range density, and we would have the three constants needed to solve the equation.

Now, all of you know where this is going. Aluminum and magnesium will not mix. You might ask why we hadn't thought of this before. We had, of course, but these materials happened to be the perfect ones to go at either end of our continuum. After much scratching of our heads, we actually came up with a solution, which you already know. We took sheets of each material 20/1000ths of an inch thick, alternating aluminum with magnesium, and built up a layered block the same size as the others, bolted together at the corners and secured at the ends with 2-inch aluminum plates. It worked great!

Thus we had our third constant, the density of which was easily determined mathematically. We could calibrate any gauge against these blocks.

Calibrating the moisture portion was a little easier but still presented some challenges. We needed to create only one standard block for water, because the density of water is linear, from 0 to 62.4 pounds per cubic foot in construction materials. The problem was how do you build a block of water?

Actually, we tried several things. For example, we tried a plaster of paris mixture and also cadmium chloride, which worked; but is a very hazardous material. We settled on precision-milled polyethylene sheets, which are almost all hydrogen. We layered them with sheets of magnesium and built a block that became the standard against which to test moisture gauges, because we knew exactly how much hydrogen was in this block. In an industrial environment, clean water will become contaminated with impurities, not to mention evaporate! However, polyethylene is very stable and does not change chemically over time, so this standard can be used for many years to come.

We presented these standards to the industry at a winter 1970 meeting in a drafty building heated by tobacco burners at the North Carolina State Fairgrounds. We put five sets of these blocks on tables and told everyone to bring their gauges, no matter the manufacturer, and calibrate them using our system. Between 100 and 150 people attended. Several years later, ASTM members voted on industrywide calibration standards based largely on the work presented at this conference.

There was at least one other important discovery resulting from that meeting. Many of the attendees had never experienced Southern barbecued pork before, and all weekend long we hauled people back and forth from a barbecue restaurant to the Pine State Dairy plant, where they loaded up on dry ice to pack their food in for the ride home.

At any rate, people were free to try and prove our calibration method wrong. In fact, we wanted people to run this system through as many tests as they could invent. It has stood up to all tests so far. After the conference, people from around the country and then from around the world began sending us their gauges for an annual calibration. We also built and shipped many of these blocks so engineers could calibrate their devices at their own sites. And thus the era of standardized calibration began.

The third and latest chapter of this story opened just this year.

Since 1970, we have been tracking the performance of many thousands of gauges that come into our labs for calibration. As a result of this tracking, we have developed a way to empirically calibrate a device. We use what we call, obviously enough, the Tracker, which is a small computer that you hook directly to your gauge. It monitors the degradation of the radioactive material and offers multiple density measurements that can mimic the standard blocks. Large blocks will always be the standard.

We presented the Tracker at a conference in February of this year. We patented this invention, something we didn't do with our original gauges or with our original block calibration system.

This is the story to date. So far, there is no effective alternative to nuclear gauges in this industry, and the gauges have served the industry extremely well. I think one of the main reasons they have been so successful has been our commitment to fundamentals, which I mentioned at the beginning of this talk, and I'd like to say just a few more words about this.

I believe that testing standards should be based on the fundamental physics of measurement. From an engineering point of view, fundamental means that the response of a system is directly related to the property or constitutive parameters of interest. In other words, it is related in a first order manner. The reason that nuclear techniques have been so successful for contractors is that the response of the system is directly related to the mass per unit volume of the material in question. It's a direct, hands-on form of measurement.

Now as some of you know, there are second and third order effects associated with the nuclear determination of density. We've explored a number of options, and will continue to do so, but because the response of nuclear gauges is fundamentally related to the density of material, they are built -- if you'll excuse the play on words -- on a rock solid foundation. The technology is complex, but its basis is fundamental, and this has served the industry well.

In our own studies of alternative testing methods, we have come across some interesting ideas but have not been satisfied enough with them to bring them to market. Believe me, we would have if we thought they were a genuine improvement. I've seen several new alternative density gauges come to market recently. I don't believe they are a true improvement on nuclear gauges, and I believe there are some real problems associated with their response, but I understand why they've come to market. So often in this fast-paced business world, if you wait you lose. Still, I think time has proven that the best technology rests on the best fundamental science. We will keep striving for innovation, as always, but we will always stay grounded in the fundamentals.

And what have those fundamentals done for the industry?

There are about 8.3 million roadway lane miles in the United States and Puerto Rico. We can reasonably assume that 80% of this mileage was newly built or resurfaced during the past 40 years, and at least 5 million miles of this construction were controlled by nuclear gauges.

We know that there is an estimated savings of $38 per nuclear test as opposed to sand cone and oven drying. In addition, there are about a $200 savings over coring methods. I estimate the savings from the use of nuclear devices on these highway projects at well over $1 billion. Add to that the use of these gauges in the construction of embankments, dams, foundations, airports, utility projects and so forth in this country alone, and we are talking about savings in the 10s of billions. Expand this scenario worldwide, and the savings just explode. These devices certainly have served the industry well.

With them, we now control thin asphalt overlays, quickly and accurately determine the asphalt content of mixes, and measure the density of sediments in harbors and shipping channels. There is great promise for the measurement of cement and the water content of fresh concrete.

At Troxler, we have an entire lab full of research projects that I can't discuss with you today. But, I guarantee that these will make as great a contribution to construction in the next century, as nuclear devices have made in the past 40 years.

Personally, I'm proud that Troxler has had the resources, the expertise and the commitment to make so much of this happen. Not bad for a basement start-up, don't you think?

To summarize, there are five good reasons why nuclear gauges were accepted and used worldwide and these reasons are as follows:

1. Their measurements are fundamentally and directly related to what ASTM and the industry wanted to know: mass per unit volume.
2. The gauges were accurate. We were able to calibrate gauges on an annual basis to prove their continued accuracy and precision (good calibration system).
3. The gauges are not expensive and they are portable.
4. The system is primarily dependent on the mass per unit volume.
5. The technology was wholeheartedly supported by ASTM, FHWA, AASHTO and many others.

I agree with Dr. James Thomas, who said that there are two basic values that should be inherent in every standard—quality and relevance.

The important message here is to stick with the fundamentals. Don't be pushed to standardize a technology or procedure before it is ready and proven. Let us keep ASTM's fine record of achievement a standard in itself.

Appendix I

The Cornell University Patent, 1957

The 1957 patent assigned to Cornell University was a primer on nuclear gauges. A close reading showed that the art of a flat-bottom box using the principle of backscatter geometry was the only patentable item. The patent did not cover direct transmission geometry or the art of shielding the Geiger Mueller detector tubes.

The later use of the direct transmission method overcame many of the inherent problems associated with the backscatter method that the patent covered, such as surface roughness and soil chemical composition.

United States Patent Office 2,781,453 Patented Feb. 12, 1957

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2,781,453

METHODS AND APPARATUS FOR DETERMINATION OF CHARACTERISTICS OF MATTER IN A SURFACE LAYER

Donald J. Beicher, Trevor R. Cuykendall, and Henri S. Sack, Ithaca, N.Y., assignors to Cornell University.

Application February 11, 1953, Serial No. 336,232

6 Claims. (Cl. 250-83.6)

This invention relates to methods and apparatus for the determination of certain characteristics of a surface layer of material, without the need of removing material from this layer, of making a hole into this layer, or otherwise disturbing significantly the layer and, in particular, relates to methods and apparatus for determining the concentration of hydrogenous substance, such as water, in a surface layer and the density of such layer.

The rapid, precise, and easy determination of density and hydrogen content in the top few inches of a material, in particular of natural or artificial soil layers, is of utmost importance in certain fields such as civil engineering and agronomy. In civil engineering, for example, during the building of earth dams, roads, airfields, etc., the density and moisture content is checked regularly during construction.

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In agronomy the recording of density and moisture in the surface layer is important from the point of view of drainage and root growth. The here-mentioned fields are only a few examples and are not exclusive of other possible applications of the here-proposed methods and apparatus, as for instance checking the curing of concrete, the wetness of paper, the moisture and surface density of materials such as grain in storage bins, concrete mixes, plastic materials, etc.

The conventional method for determining the moisture content of soil, for example, is to remove some of the material and to dry it in an oven and then to determine the loss of weight. Conventional methods for determining the density of a soil, for example, involve the removal and weighing of a portion of the soil, and the measurement of the volume the removed portion had occupied by pouring sand in the hole, or by other means. These methods are slow, the results are not immediately available, and the sample may be influenced by local inhomogeneties.

Accordingly, an object of this invention is to provide methods and apparatus for measuring the characteristics of a surface layer which will eliminate the aforementioned difficulties.

Another object of this invention is to provide an improved method and apparatus for measuring surface density.

Another object of this invention is to provide an improved method and apparatus for measuring the content of hydrogenous matter in a surface layer.

Another object of this invention

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is to provide portable instruments of the type described that can be rolled or carried over the surface and which will give an immediate indication of the desired characteristics of the layer of material underlying the instrument.

Another object of this invention is to provide methods and apparatus of the type described for quickly and accurately measuring density or content of hydrogenous material of a surface layer without removing material from the layer, making a hold in the layer, or otherwise disturbing the layer.

These and other objects and advantages of the invention will be made clear by reference to the following description and accompanying drawings in which:

Fig. 1 is a side elevation view of a device for measuring surface density in accordance with the principles of this invention.

Fig. 2 is a plan view of the device shown in Fig. 1, without the recording instrument attached and with top cover plate removed.

Fig. 3 is a graph showing a typical calibration curve for an instrument of the type shown in Fig. 1.

Fig. 4 is a side elevation view of a device for measuring the content of hydrogenous matter in a surface layer.

Fig. 5 is a plan view of the device shown in Fig. 4, and

Fig. 6 is a graph showing a typical calibration curve for an instrument of the type shown in Fig. 4.

The method for determining the characteristics of a surface layer of material according to this invention comprises exposing said layer to direct radiation from a radioactive source outside of said layer, and measuring back-scattered radiation from said layer

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at a position outside of the layer and shielded from the direct radiation from said radioactive source. For determining the surface density the radioactive source used is one capable of emitting gamma radiation and the measuring means includes a detector for the back-scattered gamma rays. For determining the content of hydrogenous material in the layer, the radioactive source is one capable of emitting fast neutrons and the measuring apparatus includes a detector for back-scattered slow neutrons and gamma rays.

The method for density determination is based on the fact that in passing through matter gamma rays are scattered by the electrons of the substance or substances encountered. The equipment consists of an assembly containing principally a gamma ray source, a detector for gamma rays connected to conventional measuring equipment, and a strong gamma ray absorber, such as lead or tungsten placed between the gamma ray source and the detector so as to very greatly reduce the intensity of the gamma ray beam which could reach the detector directly (in a straight line) from the source. When the assembly is placed in contact with some substance, such as soil, gamma rays from the source penetrate the soil mass, interact with the electrons of the material, and are scattered in all directions. The number of rays which are scattered toward the detector and reach it are determined by the measuring equipment. The number so determined is a measure of the density of the surface layer of the substance and for a given assembly a calibration curve can be determined. Such a calibration curve is given in Fig. 3 where the ratio of counting rates is defined as the ratio of

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the number of gamma rays recorded by the detector when the assembly is in contact with the substance whose density is being determined, to the number recorded when the assembly is in contact with a "standard" containing a well determined and fixed density.

Referring now to Figs. 1 and 2, which illustrate one embodiment of the invention, a gamma ray source is shown at 11. This may be a suitable amount of radium or of cobalt 60 or other gamma ray emitter placed in a small sealed metal capsule. For example, a 1 millicurie cobalt-60 capsule may be used. At12 is placed a gamma ray detector, in this case a Geiger counter tube, arranged so that its position may be secured in the framework 13. The height of the tube 12 may be adjusted by means of adjusting screw 14. A triangular lead block 15 truncated at its apex, secured within the framework 13, separates the source 11 and detector 12. To the whole assembly is attached a sheet aluminum base plate 16 which makes contact with the surface of the substance whose density is to be measured. A plug-in type connector 17 permits connecting the Geiger counter tube 12 by means of a coaxial cable 18 to suitable counting equipment 19 such as a count-rate meter or scaler, well known to one skilled in the arts. Finally a handle 20 is provided for convenience in carrying the instrument. Cover plates as 21 and 22, Fig. 1, may be provided for the radiation and detector compartments. Other methods of moving the equipment, especially when in contact with the soil surface, such as rollers or skids, may be carried by a truck, trailer, or the like, and the detecting instrument placed on the ground or the surface.

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Instead of the Geiger counter, other types of detectors such as a scintillation crystal and photomultiplier might be mounted perpendicular to the base plate. The shape of the lead block separating the source and detector is shown as triangular merely as an example of one possible shape. The block will, for example, be rectangular in plan, and a further modification consists of two triangular blocks base to base with the gamma ray detector between them and a source at each apex, in an arrangement similar to that shown in Figs. 4 and 5 described hereinafter.

Since it is known that detector and counting equipment may drift over long periods of time, it is advisable to have means of checking and standardizing the equipment. This is done by placing the instrument on the surface of a block of concrete of sufficient size and to take the reading obtained with the instrument on this concrete block as a normalizing reading. Other material than concrete may be used for this purpose, the principal condition being its permanence as regards density and flatness of surface.

The method for determining hydrogenous matter is based essentially on the fact that fast neutrons are scattered and slowed down more strongly by hydrogenous substances than by substances containing only heavy atoms. The means for carrying out this embodiment of the invention comprise a fast neutron source and a detector for slow neutrons connected to a conventional nuclear measuring instrument. The number of slow neutrons detected by the detector is a measure of the hydrogen content, and for a given assembly a calibration curve can be determined. Such a calibration curve

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is given in Fig. 6 where the ratio is given between the number of slow neutrons indicated by the detector divided by the number of slow neutrons by the same instrument when brought in contact with a "standard" containing a well-determined and fixed amount of water or other hydrogen-containing substance.

Referring now to the drawings and to Figs. 3 and 4 in particular, sources giving off fast neutrons, in this particular case a mixture of radium D and beryllium are shown at 24, 25, and 26. Any other fast neutron source may be used, such as polonium-beryllium, radium-beryllium, and the like. Between these sources is placed and rigidly connected to a frame 27 a slow neutron detector. In the model illustrated this detector consists of a commercial thin-wall Geiger Mueller counter tube 28 surrounded by a silver foil 29 which, as is well-known, transforms absorbed slow neutrons into beta rays which are detected by the GM tube. These elements are placed within a tube 30 of brass, or the like. At 31 is shown a plug-in type connector which permits the connecting of the counter tube 28 by means of a coaxial cable 32 to a suitable counting equipment shown diagrammatically at 33, fig. 4, such as a count-rate meter or a scaler, well-known to one skilled in the arts. This equipment may be carried by a truck, trailer or other conveyance (not shown).

Instead of silver, materials such as rhodium, indium, and others, may be used to convert slow neutrons into beta rays. Instead of the combination of GM tube and metallic foil, other slow neutron detectors may be used such as scintillation counters, boron-filled GM tubes; GM tubes having silver in the inside of the tube, and many other forms.

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Between the neutron sources 24-25 and 26 and the detector 28 are lead blocks 34 and 35 which absorb some of the gamma radiation emitted simultaneously with the neutrons from the sources. The whole assembly is mounted on a flat plane 36, of sheet aluminum or other material which will not block the flow of neutrons appreciably, which makes contact with the surface of the material whose hydrogen content is to be determined. The sources, the counter tube, and the lead are surrounded by paraffin 37 (or some other substance containing hydrogen atoms) within outer frame 38 so as to increase the sensitivity of the instrument. For convenience of illustration Fig. 5 shows the device partially in section and prior to filling it with paraffin. Finally a handle 39 is attached to the fame 27 permitting easy carrying of the apparatus. It is of course possible to mount the apparatus on rollers or other means of easy motion so as to permit rolling the instrument over the surface.

Checking and standardizing this equipment is done by occasionally putting the apparatus on a block of paraffin of sufficient size and to take the reading obtained with the instrument on this paraffin block as a normalizing reading. Other material than paraffin may be used for this purpose, the principal condition being its permanence as regards hydrogenous content and flatness of surface.

Since fast neutrons react with the hydrogen atoms in hydrogen-containing substances in such a way that a certain number of neutrons are captured, and in this process a gamma ray is emitted, the presence of hydrogenous material will not only produce slow neutrons at the

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place of the detector but also gamma rays, both or each of which then can be used as indicators for the hydrogen content. In the model described above, the detector is such that it measures both neutrons and gamma rays. For certain special applications, it may be desirable by changing the kind of detector or by means of appropriate shields to separate the two agents and to measure slow neutrons and the secondary gamma radiation separately.

While in the foregoing for the sake of an example, the application to the determination of moisture has been discussed, the present method and apparatus can be applied equally well to the determination of concentration of any substance containing hydrogen atoms, such as for instance hydrocarbons, etc.

Obviously a very considerable number of modifications may be made to the apparatus and the general method by anyone skilled in the arts and still come under the scope of the present invention. Some of these possible modifications have been mentioned already. For example, several radiation sources may be arranged along the circumference of a circle, the radiation detector being at the center of the circle. Or this arrangement may be reversed by placing a source at the center of the circle and a curved ionization chamber or other suitable detector along the circumference. The distance between source or sources and the detector may be varied, to give optimum sensitivity for the particular range of densities to be measured. The material which shields the detector from the direct gamma ray beam may be made from material other than lead, depending upon which provides the best shielding for a particular shape and weight.

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It is also possible to replace the counting equipment by an automatic recording equipment thus enabling a continuous record of density both in time, and in space by moving simultaneously the apparatus over the surface. The "normalization" of the measurements may be performed in other ways than described above, for example, by bringing a constant radioactive source near the instrument in a well-determined and reproducible position with respect to the detector.

What is claimed is:

1. An apparatus for measuring the characteristics of a surface layer of material comprising a container having a flat bottom adapted to permit intimate contact with the surface of the layer to be measured, a source of radioactivity within said container in close proximity to and adapted to radiate through said container bottom, a detector for radioactivity positioned within said container in close proximity to the bottom thereof, a shield of substantial thickness in said container between said radioactive source and said detector for preventing direct radiation from said source from reaching said detector, and means for connecting the output of said detector to measuring and recording equipment.

2. An apparatus for measuring the density of material contained in a surface layer that comprises a container, the bottom side of which is shaped in such a way as to permit intimate contact with the surface of the layer to be measured, a source of gamma rays positioned in said container in close proximity to the bottom thereof, a detector of gamma rays in said container in close proximity to the bottom thereof and spaced from said source, a shield of

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substantial thickness between said source and said detector for preventing the direct gamma radiation from reaching said detector, and means of transmitting the output of the detector to recording and measuring equipment.

3. An apparatus for measuring the content in hydrogenous material contained in a surface layer comprising a container, the bottom of which is shaped in such a way as to permit intimate contact with the surface of the layer to be measured, a source of fast neutrons in said container in close proximity to the bottom thereof, a detector for slow neutrons in said container in close proximity to the bottom thereof, and spaced from said source, a shield of appropriate thickness between said neutron source and said detecting instrument for preventing the direct gamma radiation from said source from reaching said detecting means, hydrogenous material surrounding said neutron source, shield, and detecting means on the top and sides thereof, and means for transmitting the output of the detector to separate recording instruments.

4. An apparatus for measuring the content in hydrogenous material contained in a surface layer comprising a container, having a flat bottom to permit intimate contact with the surface of the layer to be measured, a source of fast neutrons in said container in close proximity to the bottom thereof, a detector for gamma rays in said container in close proximity to the bottom thereof and spaced from said source, a shield of substantial thickness between said neutron source and said detecting instrument for preventing the direct gamma radiation from said source from reaching said detecting means,

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hydrogenous material surrounding said neutron source, shield, and detecting means on the top and sides thereof, and means for transmitting the output of the detector to separate recording instruments.

5. An apparatus for measuring the content in hydrogenous material contained in a surface layer that comprises a container, the bottom of which is shaped in such a way as to permit intimate contact with the surface of the layer to be measured, a source of fast neutrons in said container in close proximity to the bottom thereof, a detector for the simultaneous counting of gamma rays and slow neutrons in said container spaced from said source and adjacent to the bottom thereof, a shield of appropriate thickness between said neutron source and said detecting instrument for preventing the direct gamma radiating from said source from reaching said detecting means on the top and sides thereof, and means for transmitting the output of the detector to separate recording instruments.

6. An apparatus for measuring the content of hydrogenous material contained in a surface layer that comprises a container, having a flat bottom to permit intimate contact with the surface of the layer to be measured, a source of fast neutrons adjacent opposite ends of said container and in close proximity to the bottom thereof, a detector for slow neutrons positioned between said radiation sources and space therefrom, a lead shield of substantial thickness separating said radiation sources from said detector, hydrogenous material surrounding said shield, said neutron source, and detecting means on the top and sides thereof, and means of transmitting the output of the detector to

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a recording instrument placed separately from the container.

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Appendix II

The Nuclear-Chicago Patent, 1966

A Close reading of the 1996 Nuclear-Chicago patent showed that it covers only a shielding method for radioactive isotope. ASTM realized that something more was needed as the basis for an industry standard, which eventually was written with shielding as a function of gauge design.

United States Patent Office

3,256,434
Patented June 14, 1966

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3,256,434

RADIOACTIVITY APPARATUS FOR INDICATING PROPERTIES OF MATERIALS

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Filed Nov. 20, 1963, Ser. No. 325,186
22 Claims. (Cl. 250-83.1)

This invention relates to an improved form of portable device for measurement of characteristics of materials, and more specifically to a source-and-detector probe for measurement of moisture content in soils. The present application is a continuation-in-part of the application of the same inventors filed June 17, 1960, serial No. 36,945 now abandoned.

It has long been known that certain properties of materials may be measured by observing the effect of their presence on the response of a radioactivity detector to a radioactive source. In the copending application of Phillip Shevick, Serial No. 741,421, filed June 11, 1958, now Patent 3,103,588, there is described a source-and-detector probe for measurement of moisture in soils, designed to be inserted in a suitable borehole. In many applications, such a probe is not practical, both for the reason that the necessity of drilling boreholes makes the making of measurements over a large area extremely difficult, and the further reason that in many instances only the

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moisture at the surface of the soil is of direct interest. Thus in order to make this type of instrument practical for many applications, such as road building and agriculture, it is necessary that the probe containing the source and the detector be placed upon the surface under measurement, rather than inserted into a hole made for this purpose. In principle, it would appear that a probe generally similar to that used in depth measurements should also be suitable for surface measurements. In practice, however, this is not true. In the first place, the "geometry" is vastly superior in the borehole measurement to the surface measurement. In the borehole, the probe is completely surrounded by the medium under measurement so that the emissions in all directions (except for the small solid angle longitudinal of the borehole) contribute to the measurement. Further, in the case of such a depth probe, there is little hazard to personnel during use because the only path for exposure to radiation is directly up the borehole, the soil itself serving as a shield in all other directions.

It is found that the employment of a construction analogous to that employed in the depth probes is incapable of producing a fully practical instrument for surface measurements of moisture. In both types of measurements, of course, the lower limit of intensity of the neutron radiation which may be employed is fixed by the necessity of obtaining counting rates which will produce reasonably low statistical errors without excessive measurement times. It is of course

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possible to compensate for the loss of the favorable geometry obtained in using a depth-type probe in a surface application by increasing the size of the source. However, such a modification is found to be highly impractical; because of the loss of the shielding inherent in borehole measurements, the surface measurement must in general be performed with a source less, rather than more, intense than the borehole measurement. The problem is particularly acute where the source, in addition to the neutrons actually employed, emits a high proportion of gamma rays, as in the case of radium-beryllium sources.

A further problem in design of a satisfactory surface moisture probe is the matter of linearity of calibration, i.e., linear variation of detector counting rate with moisture content over the range of interest, which is, in the case of said measurements, from zero to 50% moisture in the soil.

It has thus been found that although the prior art suggests the possibility of moisture measurements on soil surfaces by neutron emission and detection, and indeed presents at least one probe as suitably designed for the purpose, the reaching of a probe design which will, as a practical matter, be accepted for routine use in making such measurements in the field, in substitution of r other methods and apparatus for such measurements, requires highly specialized design features in order that the measurements may be made safely in a reasonably short time and without impairing linearity of calibration or making the device so cumbersome that it becomes impractical as a portable instrument.

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These features of construction, which have been found by the present applicants to produce a commercially practical instrument, may be understood from the description of a particular embodiment of the invention which is illustrated in the attached drawing.

In the drawing:

FIGURE 1 is a vertical sectional view of a source-and-detector probe for soil moisture measurements made in accordance with the invention;

FIGURE 2 is a fragmentary sectional view corresponding to a portion of FIGURE 1, but illustrating the parts in a different position.

FIGURE 3 is an elevational view partially in section taken along the line 3-3 of FIGURE 1 in the direction indicated by arrows, of a shield cup or sleeve constituting a portion of the device of the invention;

FIGURE 4 is a horizontal sectional view taken along the offset line 4-4 of FIGURE 1 in the direction indicated by arrows;

FIGURE 5 is a more or less diagrammatic plot or graph illustrating certain aspects of the operation of the device of the invention; and

FIGURE 6 is a more or less fragmentary schematic illustration of the electrical connection of detectors employed in the probe.

The illustrated source-and-detector probe construction is enclosed in a housing having a bottom 10, sidewalls 12, and a top 14. On the bottom 10, which is transparent to neutrons of all energies, being made of, for example, 11 gauge aluminum, there is mounted an array of detectors generally designated by the numeral 16. The array is formed of two groups of detectors 18 and 20, parallel with each

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other and spaced by a small central gap 22. Each group of detectors has a header box or cover 24, from which extend five parallel tubular portions 26. Each group 15 and 20 is internally parallel-connected and the two groups are likewise connected in parallel (FIGURE 6). The detectors are, for example, cylindrical boron trifluoride proportional counters having the well-known inverse proportionality between sensitivity and neutron velocity. (The construction of the individual detectors is not herein described in detail, constituting no portion of the invention herein disclosed and claimed. For the purposes of the present invention in its broadest aspects, any type of slow neutron detector may be employed to form the detector array.) In the present embodiment, one end of the tubular counters 26 terminates in the corresponding header 24, which is secured to the bottom 10, and the opposite end rests on a resilient support 28, all of the outer ends being positioned by a clamp bar 30 secured by screws 32. Spaced from the bottom 10 by posts 34 of sufficient height to clear the detector array 16 is a support plate 36. A shield cup 38 is secured to the under side of the support plate 36 at the center by bolts 40, the shield cup being disposed in the gap 22 between the detector groups 18 and 20 forming the array. The shield cup 38, of lead, has a sleeve portion 42 absorbing horizontal gamma radiation and a bottom portion 44 absorbing downward gamma radiation (but transparent to neutrons), extension portions 46 being provided to facilitate the bolted fastening. The dimensioning and purpose of the shielding portions of the cup 38 will be discussed further hereinafter.

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A fast neutron reflector 48, in the form of a disc of copper, and having a flange at 50, rests on the support plate 36. On the reflector 48 is a more or less hemispherical gamma ray shield 52. A cover or sheath 54 provided with a flange 56 mating with the flange 50 covers the shield and reflector and holds them in position, the entire assembly being secured in place by bolts 58 and 60. As more fully described in the copending application of Raymond L. Meeder and Charles E. Mielke, filed January 5, 1960, Serial No. 617, now U.S. Patent No. 3,126,484, the under side of the shield 52 has a diametric channel 62 in which are slidable shield blocks 64 with upwardly facing cam surfaces 66, the slide blocks 64 being biased inwardly by springs 68 to the positions shown in FIGURE 2, wherein they meet. The shield 52 has an axial aperture 70 in register with a central aperture 72 in the reflector. A source capsule 74 containing a radium-beryllium source and a rod 76 returning the capsule 74 in position by means of a spring 73 are all encased in a hollow rod or tube 80, the vertical manipulation of which selectively raises the source into the shield with the slide blocks closed as in FIGURE 2, or drops the source into the cup 38 for the making of a measurement, as in FIGURE 1. The tube 80 is provided with camming and locking surfaces cooperating with a locking hub structure 82 which is not fully described herein, being fully described in the copending application last mentioned.

The general structure of the device of the invention having been described, the manner of operation and the criticality of the selection of materials and dimensions may be now

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understood. The practicality of such a device for field use, as may be seen from the earlier discussion, has four essential requirements, any one of which is fairly easily obtainable, but the combination of which has been found to be obtainable over only a relatively small range of variables in construction. These four factors may be summarized as (1) neutron utilization (2) linearity of calibration over the range of interest in soil measurement (3) light weight and portability and (4) low external radiation at exterior points other than the bottom. As may readily be seen, although these four factors would be of importance even with a pure neutron emitter, such as a polonium-beryllium neutron source, the latter two become even more critical when using a radium-beryllium neutron source, because the presence of gamma rays both lowers the maximum neutron source strength which can be used and lowers the permissible weight of the components actually contributing to the measurement, because of the necessity for the presence of shielding for gamma rays in both the storage and operating conditions.

The fast neutron reflector must have a high ratio of macroscopic scattering cross-section to weight. It is found that copper demonstrates the most desirable properties as the reflector for use in the device of the present invention. The thickness of the copper should be between one-half and one inch, three-quarters of an inch being found highly satisfactory. The optimum diameter (i.e., neglecting the thin flange) is found to be from 4 to 4 ½ inches, further increase of diameter producing no adequate increase in neutron utilization to justify this added weight.

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The overall size of the housing is approximately 10" x 12" in the horizontal plane, with a 6" height. However, the actual effective area covered by the neutron detector array is approximately 7 ¾" square (the sensitive portion of the length of the detector illustrated). It is found that with horizontal dimensions smaller than approximately 6" the neutron utilization is adversely affected to an excessive degree, while with horizontal dimensions greater than approximately 8" square, the linearity of calibration in the range of interest in soil measurements is adversely affected, the reasons for these experimentally observed limitations being discussed later. It will of course be understood that the reflector dimensions referred to previously, as regards horizontal extension, are for use with an array of neutron detectors of the dimensions just stated, the horizontal extension dimensions of the reflector over the central portion of the detector array being half to three quarters the dimensions of the array.

The shield cup 38 serves a multiple purpose. The sleeve portion 42 absorbs sufficient gamma radiation to prevent the slow neutron responsive proportional counters constituting the detectors of the array from producing spurious counts due to high-intensity gammas. The bottom portion 44 acts to attenuate the downward gamma radiation. It will be noted that the housing is so shaped that the distance from the source to the exterior of the housing (except in the downward direction) when the source is in the lowered position is at least equal to the distance of the source from the closest sidewall, thus reducing gamma flux to

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safe levels without the necessity of shielding of the magnitude provided in the withdrawn or storage position of the source. It is also to be noted that the copper neutron reflector provides a substantial degree of shielding of low upward angle gamma radiation because of the thickness of the copper which must be traversed by gamma rays leaving the source at the small upward angles exterior to the solid angle subtended by the gamma-ray shield when the source is in the lowered position. With the present construction (see FIGURE 1) the reflector 48 presents a very large thickness to the gamma rays from the source, and the only direction (other than the safe downward direction) in which the gamma attenuation is limited to the thickness of the shielding sleeve 42 (about ½" of lead) is substantially horizontal, at which there exists negligible hazard of radiation damage to personnel when the probe is in working position on the surface of the ground.

The device illustrated in the drawing and described above presents a safe and practical source-and-detector probe for surface measurements of characteristics of materials, particularly for the measurement of moisture in surface soils. Using a 4 to 5 milligram radium-beryllium source, the illustrated construction is extremely linear in counting rate as a function of moisture content of soil from 2% to 40% with an increment of approximately 300 counts per minute per percent volume of moisture, small departures from linearity occurring in the infrequently-encountered extremely low and extremely high soil moisture contents at the bottom and top of the overall range. With the probe in the operating

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condition resting on the ground, maximum radiation at the outer surfaces of the housing at upward angles more than a few degrees in 10 mr. per hour or less.

It will of course be understood that the description of materials herein used follows the practice customarily used in the art. Thus, for example, lead is considered gamma-ray shielding material and copper or iron a fast-neutron-reflecting material despite the fact that the former will have some degree of efficiency in reflecting neutrons and the latter in absorbing gamma rays. For purposes of the present invention, a fast neutron reflector is a material having negligible neutron absorption and moderation and having a macroscopic scattering cross-section of at least 0.1/cm., for neutrons at the energy of emission from the source and gamma ray shielding material is a material having gamma ray absorption properties at least substantially equivalent to those of lead.

As was previously observed, the detecting area is selected to produce high counting rates with any given intensity of the neutron source while providing substantial linearity of the moisture response curve over the range to be measured. The manner in which this object is achieved can best be understood by consideration of the counting rates of individual counters, and their aggregate.

As shown in the Shevick patent previously mentioned, it is possible to obtain substantial linearity of response over a wide range in soil moisture measurements of the depth type by placing the source directly adjacent to the counter at its longitudinal center, with proper selection of the length of the

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counter. It will be observed that either of the innermost counters 26 of the present device is in a relationship to the source which is approximately analogous to the same type of source-and-detector geometry as is used in a depth probe. In a surface measurement, however, there are a number of factors which make the desirable constructional features substantially different in producing high source utilization with good linearity of calibration. There has already been discussed the use of the reflector to utilize, at least in part, the neutrons emitted by the source in upward directions, thus reducing this type of loss, which is not encountered in depth measurements. In addition to the fast neutrons emitted directly from the source, the reflector also helps to conserve partially moderated neutrons which escape from the medium under measurement (the device itself of course being substantially free of moderating material) before they have reached sufficiently low energy to be detected by the thermal neutron detectors. Nevertheless, the overall geometry is found, whether with or without a reflector, such as to make the use of a source and single detector (or detectors on each side of the source) as used in depth measurements completely unsatisfactory for surface measurements, both as regards source utilization and as regards linearity of the moisture calibration curve. By spacing a counter (or counters) substantially from the source, rather than closely adjacent, it is found to be possible to achieve a fair degree of linearity of calibration, but at the expense of still further loss of counting rates for any given source size.

FIGURE 5 illustrates, in slightly idealized and simplified form, the

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general principle of the manner in which linearity is achieved in the large-area detector geometry of the invention along with great gain in source utilization. For purposes of simplicity, this figure shows only the individual counting rates of two of the counters on one side of the source, the responses being idealized to some degree in order to illustrate the principle most clearly.

The shape of the detector response curve, in any source-and-detector radioactivity measuring device for properties of materials of the present general type, will be greatly affected by the distance between the source and the detector, where the variable under measurement has both an effect on the scattering and an effect on absorption, as is normally the case. It has long been known that a fast neutron source and a slow neutron detector will produce a calibration curve for variations in moisture which has a shape highly dependent upon the spacing used and the particular range of moisture content (or similar variable) under measurement. Typically, departures from linearity are characterized by a second derivative which is positive oat low values of moisture and negative at high values of moisture, even though the counting-rate is an increasing function of moisture over the entire range.

In the surface measurement here involved, with extremely small spacings between source and detector, the first region of the curve extends to the highest moisture concentration values of interest in soil measurements, so that the entire portion of the moisture response curve which is used in concave upward, as shown in FIGURE 5 for the case of the inner counter, the response curve of which is indicated at 90. At much larger

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spacings, this first portion of the curve is not measurable, occurring over a very small range of moisture concentrations close to zero, the gross calibration curve accordingly demonstrating only the negative second derivative, such a curve being illustrated at 92. Such curves are found to be dependent not only upon the perpendicular spacing from the source, but also, in the case of a straight elongated counter, upon the length.

By proper selection of parameters, the counters and differing distances may be so selected that the non-linearities of the characteristic curves are in opposite and compensatory directions, so that when the two are connected in parallel, their total counting rate has much greater linearity than the individual counting rate of one of the counters. This is graphically illustrated in FIGURE 5, which is drawn to idealize the operation in illustration of the principle, the curves 90 and 92 producing, when totalled, the straight-line characteristic or response curve 94. There is also shown in the drawing the dotted "normalized" curve or line 96, corresponding to half the ordinate values of the straight-line plot 94.

As will be obvious, the idealized graph of FIGURE 5 represents the results obtained when there are added the responses of counters which are respectively too near to, and too far from, the source to give the best linearity. When these are added to a counter of intermediate spacing from the source, selected for best linearity over the range of moistures to be measured as was done prior to the present invention, it is seen that both the linearity and the source utilization have been greatly benefited. Such compensatory "response shaping" represents in the

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ultimate the overall principle of the present construction, which closely approximates an entire thermal-neutron-detecting area, with the fast neutron source at its center.

It will of course be understood that the achievement of exact linearity with the accuracy portrayed in FIGURE 5 is not practical. Compensation or cancellation of the opposite non-linearities in measured curves cannot be practically achieved without leaving minor "ripples" in the overall curve. However, the present construction not only does not sacrifice linearity as the price of increased utilization of the source by adding further counters to a single counter at optimum distance, but indeed produces, over a broad range of moisture values, a lower deviation from exact linearity than can be achieved with a single counter.

Although herein illustrated in a particular application, the principle of mutual compensation or self-cancellation of non-linearities of response, while at the same time multiplying the utilization of a source of any given intensity, can be adapted to many other source-and-detector geometries for measurements of properties of materials. In general, even where the highest degree of source utilization is not a requirement, the response curve of any given detector may be linearized by adding at lease one other detector connected in parallel, and having in itself a detector output having an opposite direction of curvature to the non-linear curvature of the first. Since the general, overall theory is not necessarily limited to neutrons, but extends to other measurements in which oppositely-curved non-linear responses can be obtained by adjustment of detector spacing, length, etc., there are many other applications particularly of an industrial or semi-industrial nature, in which linearity of response can advantageously be achieved in this general manner. Obviously utilization of this construction is not limited to identical counters, as here used. Nor is it necessary that the addition of counter outputs be made by simple connection in parallel, so long as the outputs vary in the same direction, but with opposite curvatures (i.e., have first derivatives of the same sign, but second derivatives of opposite sign) over the portion of the range to be linearized.

It will also be understood that particular features of the invention may b employed independently of other, depending upon the requirements and purposes of the particular radioactivity device being designed, many of the features described being adaptable to radioactivity devices for purposes other than that herein described, particularly to surface probes for measuring the properties of materials other than the moisture contents of soils. It will likewise be understood that a substantial amount of variation in the particular construction described and illustrated is permissible without departing from the teachings of the invention. Accordingly, the scope of the invention is not necessarily limited to the particular embodiment herein shown, but is to be determined from the appended claims.

What is claimed is:

1. A radioactivity source-and-detector device for measurement of properties of materials comprising a radioactivity source and radioactivity detectors, having means for supporting said source and detectors closely adjacent to a material under measurement, variation in

the measured property producing variation in the response of the detectors produced by the net effect of change in the same direction of absorption and scattering characteristics of the material, and having:

(a) a plurality of detectors each having response curves varying in the same direction with variation of the property under measurement over a least a portion of the range under measurement,

(b) at least two of the respective detectors being constructed and located to have the relative effects of absorption changes and scattering changes thereon sufficiently different to produce oppositely curved non-linear response characteristic curves over the range to be measured, and

(c) such detectors being electrically connected in parallel to produce an overall output characteristic more linear than that of either of such detector.

2. The device of claim 1 wherein the source is a fast neutron source and the detectors are thermal neutron detectors.

3. The device of claim 1 wherein all the detectors are elongated and closely spaced in side by side relation to form a substantially continuous sensitive detecting area;

4. The device of claim 3 wherein the source is centered in the detecting area.

5. The device of claim 1 wherein the detectors are substantially identical

in all respects other than distance from the source.

6. A neutron source-and-detector device for measurement of moisture content and analogous properties of materials comprising:

(a) a fast neutron source,
(b) at least two thermal neutron detectors at differing distances from the source,
(c) both of said neutron detectors having response curves varying in the same direction with variation of the property under measurement over at least a portion of the range of measurement,
(d) one of said detectors being sufficiently close to the source to produce a response curve concave upwardly and the other being sufficiently far from the source to produce a response curve concave downwardly, said response curves thus being of opposite non-linear curvature in said portion of said range so that the non-linearity of the total detector output response in said portion of the range is smaller than the non-linearity of the output response of either detector.

7. In a portable device for measurement of the characteristics of materials, a plane array of detectors responsive primarily to low-energy neutrons, said array having substantially equal dimensions in its plane, a plane fast neutron reflector closely adjacent and parallel to the central portion of the array and having dimensions in its plane from half to three quarters the corresponding dimensions of the array, and a fast neutron source in the center of the array.

8. The device of claim 7 wherein the reflector is of copper.

9. A portable device for measurement of moisture content in soil comprising the device of claim 7 wherein the detectors form and area from 6 to 8 inches in each direction, the energy characteristic of the neutron source being substantially that of radium-beryllium.

10. The device of claim 7 having a neutron-transparent housing, the array being on the bottom thereof and the housing having all exterior portions other than the bottom spaced from the source by a distance at least equal to half the dimension of the array.

11. In a device for measurement of the characteristics of materials, a flat-bottomed housing, an array of detectors responsive primarily to low-energy neutrons substantially covering the bottom of the housing, a centrally apertured fast neutron reflector above the detector array, a gamma-ray shield above the reflector having a passage in the bottom in alignment with the aperture in the reflector, a gamma-emitting fast neutron source in the shield, means to move the source out of the shield through the passage and aperture and into the central portion of the array for the making of a measurement, and a gamma ray attenuator substantially transparent to neutrons surrounding the source in said central portion of the array.

12. In a device for measurement of the characteristics of materials, a flat-bottomed housing, an array of detectors responsive only to low-energy neutrons substantially covering the bottom of the housing, a centrally apertured fast

neutron reflector immediately above the detector array, a gamma-ray shield above the reflector having an exit passage in alignment with the aperture in the reflector, the detector array having a gap in alignment with said passage and aperture, a gamma-emitting neutron source in the shield, and means to move the source out of the shield through the passage and aperture and into the gap for the making of a measurement.

13. A portable radioactivity device for the measurement of surface characteristics of materials comprising a housing having a neutron-transparent bottom, a lead shield in the central portion of the housing, a plurality of slow neutron detectors on the bottom of the housing defining a detecting area, a copper neutron reflector immediately above the central portion of the detecting area and below the shield and having an aperture therein, a gamma-emitting fast neutron source, a gamma-ray shielding sleeve adapted to receive the source in the central portion of the detecting area, and means for moving the source from the shield through the aperture in the reflector into the shielding sleeve for the making of a measurement on a surface upon which the housing is placed.

14. A portable radioactivity device for the measurement of surface characteristics of materials comprising a housing having a neutron-transparent bottom, a lead shield in the central portion of the housing, a plurality of slow neutron detectors on the bottom of the housing defining a detecting area, a copper neutron reflector immediately above the portion of the detecting area extending from one-half to three-quarters the transverse dimension thereof and having a central aperture therein, a gamma-emitting fast neutron source, a

gamma-ray shielding sleeve adapted to receive the source in the central portion of the detecting area, and means for moving the source from the shielding sleeve for the making of a measurement on a surface upon which the housing is placed.

15. A portable radioactivity device for the measurement of surface characteristics of materials comprising a lead shield, a plurality of slow neutron detectors in a plane defining a detecting area, a fast neutron reflector between and immediately adjacent to the central portion of the detecting area and the lead shield, a gamma-emitting fast neutron source, and a lead shielding sleeve containing the source in the central portion of the detecting area.

16. A radioactivity device for the measurement of surface characteristics of materials comprising a housing having a neutron-transparent bottom, a lead shield in the central portion of the housing, a plurality of slow neutron detectors on the bottom of the housing defining a detecting area, a copper neutron reflector immediately above the central portion of the detecting area and having an aperture therein, gamma-emitting fast neutron source, a gamma-ray shielding sleeve adapted to receive the source in the central portion of the detecting area, and means for moving the source from the shield through the aperture in the reflector into the shielding sleeve for the making of a measurement on a surface upon which the housing is placed.

17. A radioactivity device for measurement of surface characteristics of materials comprising detector means defining a thin detecting area sensitive to slow neutrons and of lateral dimensions large compared to its thickness, a fast

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neutron reflector closely adjacent to one side of said area and also of lateral dimensions large compared to its thickness, a gamma-ray shield on the side of the neutron reflector opposite the detecting area, and a gamma-emitting fast neutron source in the center of the detecting area and closely adjacent to the reflector, the gamma-ray shield subtending a substantial solid angle of radiation from the source, whereby the reflector increases the utilization of neutrons in measurements on a surface while constituting an absorber of effective thickness much greater than its actual thickness for shielding backwardly emitted gamma-rays exterior to the solid angle subtended by the gamma-ray shield.

18. A source and shield assembly for unidirectional neutron exposure comprising a gamma-emitting neutron source, a fast neutron reflector of lateral dimensions large compared to its thickness having its center adjacent to the source, and a gamma-ray shield on the opposite side of at least the center of the neutron reflector and subtending a substantial solid angle of radiation from the source, whereby neutrons are reflected forward, gamma rays emitted at large backward angles are adsorbed by the shield, and gamma rays emitted at small backward angles are absorbed by the large effective thickness of neutron reflector thus encountered.

19. A radioactivity device for the measurement of moisture content of materials comprising a moderator free plane array of closely spaced side-by-side neutron detectors of sensitivity substantially inversely proportional to neutron velocity forming a detecting area of dimensions of from 6 inches to 8 inches in each direction in the plane, and

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a fast neutron source at the center of the array.

20. The device of claim 19 wherein the neutron source is alpha-activated beryllium and the detectors are boron-trifluoride counters.

21. A radioactivity device for the measurement of moisture content of soils and the like comprising detecting means forming a moderator-free plane neutron detection area of sensitivity substantially inversely proportional to neutron velocity, the effective dimensions of the area in both directions being from 6 inches to 8 inches, and a fast neutron source at the center of the detection area, the overall response of the detecting means being a substantially linear function of moisture content over the range from 2% to 40% in soils and like materials against which the source and detecting means are placed.

22. A radioactivity source-and-detector device for measurement of properties of materials comprising a radioactivity source and radioactivity detectors, having means for supporting said sources and detectors closely adjacent to a material under measurement, variation in the measured property producing variations in the response of the detectors produced by the net effect of the change in the same direction of absorption and scattering characteristics of the material, and having:

(a) a plurality of detectors each having response curves varying in the same direction with variation of the property under measurement over at least a portion of the range under measurement,

(b) at least two of the respective detectors being constructed

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and located to have the relative effects of absorption changes and scattering changes thereon sufficiently different to produce oppositely curved non-linear response characteristic curves over the range to be measured, and

(c) means for adding the outputs of the detectors to produce an overall output characteristic which is more linear than that of either of such detectors.

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UNITED STATES PATENTS

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2,556,768 6/1951 McKibben-----250-83.1
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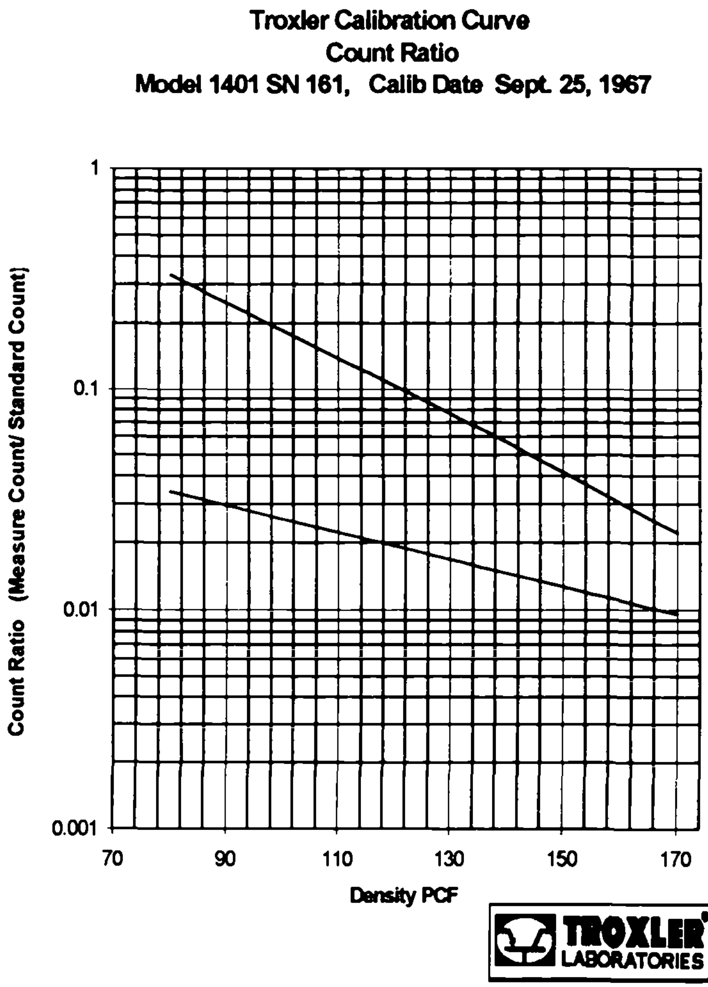
RALPH G. NILSON, Primary Examiner.
A. R. Borchelt, Assistant Examiner.

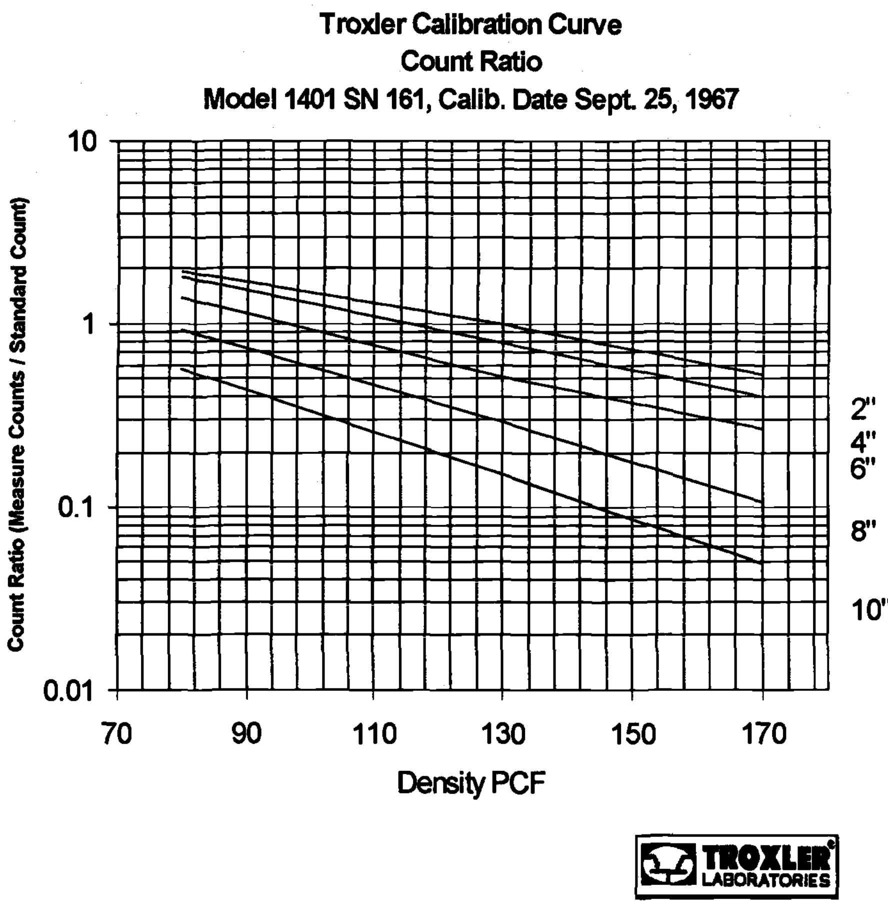
Appendix III

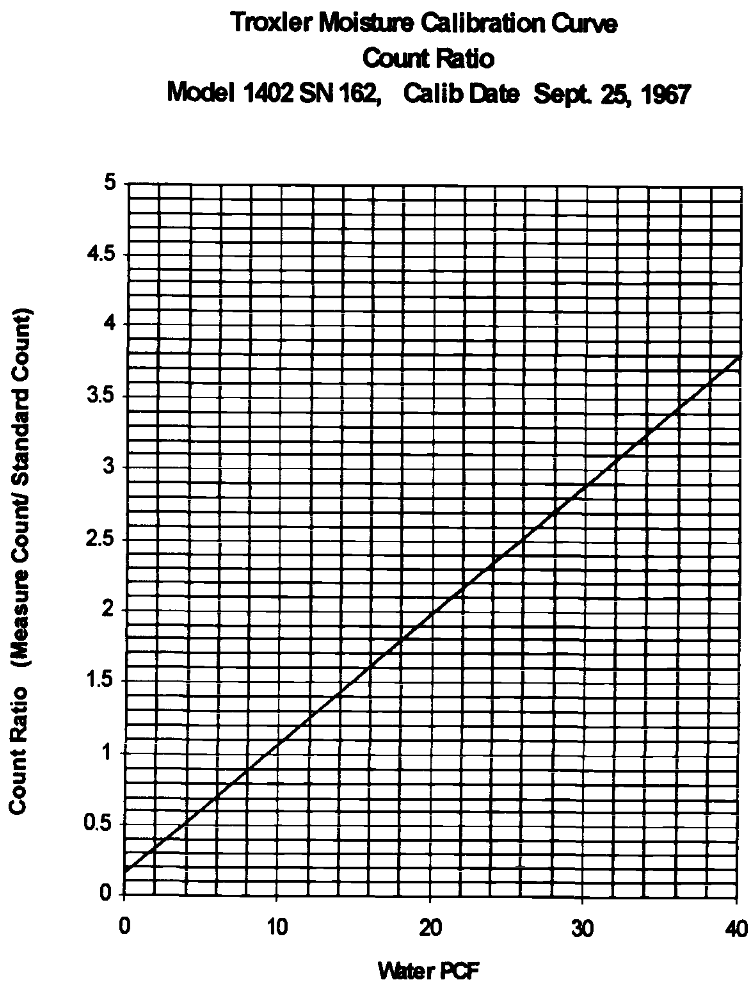
Troxler Calibration Curves, 1967

Troxler models 1401 and 1402 were the original moisture and density gauges, respectively. They were connected by a cable to the scaler. Engineers took two measurements – one for moisture and one for density – and then subtracted the moisture in pounds-per-cubic-foot from density to determine dry bulk density. The calibration curves were hand-drawn on semi-log graph paper, laminated and supplied with the gauge. The graphs were drawn for a density vs. count ratio. This ratio was determined by the operator.

You know the rest of the story. The modern day gauge contains its own computer, small, but programmed for this specific use.


Troxler Calibration Curve
Count Ratio
Model 1401 SN 161, Calib Date Sept. 25, 1967
Count Ratio (Measure Count/ Standard Count)
1
0.1
0.01
0.001
70
90
110
130
150
170
Density PCF
12"
BS
TROXLER
LABORATORIES


Troxler Calibration Curve
Count Ratio
Model 1401 SN 161, Calib. Date Sept. 25, 1967
Count Ratio (Measure Counts / Standard Count)
10
1
0.1
0.01
70
90
110
130
150
170
Density PCF
2"
4"
6"
8"
10"
TROXLER LABORATORIES



TROXLER LABORATORIES

WILLIAM F. TROXLER

BIOGRAPHY

William F. Troxler is a native North Carolinian who attended North Carolina State University on the GI Bill after serving in the U.S. Army Air Force during WWII. He received a B.S. degree in Electrical Engineering in 1952.

Following his graduation, he worked for the U.S. Army Research Command in Ft. Belvoir, Virginia. It was here that he realized there was a great need for sophisticated testing equipment in the post war economic boom. He returned to Raleigh and after doing some post graduate work in electrical engineering, he started his own company. Beginning in the basement of his home in Raleigh, he pioneered the development and production of the nuclear testing and measuring devices used in the construction and agricultural industries. He also designed special devices for NASA, which were used in the first scientific satellites launched in the early 1960's.

Under his direction, the company, Troxler Electronic Laboratories, Inc. grew from a one-man basement operation to its present size of 110,000 square feet and 160 employees. In addition to this facility, which is located in the Research Triangle Park, Troxler has offices in Germany, Canada and seven other U.S. cities.

Since the inception of his company in 1956, Mr. Troxler has been involved in innumerable activities and organizations in his industry and his community.

His present affiliations include: The Transportation Research Board, the American Society of Testing Materials, International Road Federation Board of Directors, Construction Industry Manufacturers Association, General Chairman International Conference on Industrial Radiation and Radioisotope Measurement Applications, National Research Council, Civil Engineering Research Foundation, Senior member of the Institute of Electrical and Electronics Engineering.

His past affiliations include: Governor's Task Force on Science and Technology, Highway Innovation Steering Committee, Chairman of the North Carolina State University Engineering Advisory Council, President of the North Carolina World Trade Association – 1971-1972.

His awards include: 1985 Distinguished Engineering Alumnus Award for North Carolina State University, 1972 President's "E" Award for Outstanding Contribution to Export Expansion, 1981 Governor's Award for Excellence in Exporting, Honor society of Phi Kappa Phi for outstanding engineering accomplishments.

Overview of Compaction Control Technology and Comparison of Current Methods

Jeffrey A. Farrar [1]

Bureau of Reclamation Experience with Construction and Control of Earth Materials for Hydraulic Structures

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Reference: Farrar, J. A., "**Bureau of Reclamation Experience with Construction and Control of Earth Materials for Hydraulic Structures,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384*, D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: The United States Department of the Interior, Bureau of Reclamation has a long history of development of large water supply projects in the western United States. Reclamation developed unique earthwork control techniques such as the Rapid Method of Construction Control. The Rapid Method is a three-point impact compaction test based on standard impact compaction energy level, and is performed on an adjusted wet density basis thus alleviating the need for water content determinations. Each soil sample from the in-place density test is tested using the Rapid Method to assure accurate determination of degree of compaction. Reclamation also uses the relative density for cohesionless soil and has developed a new vibratory hammer test for maximum density. Technology transfer has been accomplished through publications such as the *Earth Manual*, and by active participation with ASTM, resulting in standardization of many test methods. This paper will review Reclamation procedures and discuss current trends in earthwork construction control.

Keywords: earthwork construction control, soils, compaction, in-place density, relative density

Introduction

The purpose of this paper is to present the reader with proven construction control methods developed by the Bureau of Reclamation. The Bureau of Reclamation was created to develop water resources in the western United States. As part of this mission, Reclamation developed many earthen dams, pumping plants, and many miles of canals and pipelines. As part of these efforts, Reclamation's engineers developed proven

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[1]Technical Specialist, Earth Sciences Laboratory, Bureau of Reclamation, PO Box 25007, D-8340, Denver, Colorado, 80225, Chairman, Subcommittee D-18.02 on Sampling and Related Field Testing.

control methods for earth construction. In order to assure quality of this construction Reclamation does their own control testing through a network of field laboratories operating in area offices. This paper will review proven methods of control which are required when the public safety is concerned. These methods can be employed on critical projects such as earth dams and hazardous waste liners and covers.

The experience of the Bureau of Reclamation is well chronicled in the literature. Reclamation has published manuals such as the *Earth Manual*, and *Small Dams Manual* [1, 2, 3]. Numerous training manuals and design standards have been issued [4, 5]. Training manuals were developed on earthwork construction control and use of the rapid method of construction control [6]. These training manuals may be of help for those learning our testing techniques. The manuals are written in an effort to transmit technology to the public.

Early Development of Compaction Control

The Bureau of Reclamation began to build dams and water projects in the early 1900's. Many of these dams were hydraulic fill or dump fill structures. Rolled earthfills began in 1902 with the introduction of the "Petrolithic" (sheeps-foot) roller in 1906. These rollers were used in California to compact storage reservoirs.

The papers by R. R. Proctor of the Los Angeles Water and Power Authority started a method of control known as the "Proctor" or impact compaction test [7, 8, 9, 10]. This method, for soils containing fines, recognized the importance of water content to the compaction process. Reclamation adopted these procedures in parallel with other major water development and transportation agencies. Proctor advocated the penetrometer needle to determine the penetration resistance of the fill. The laboratory penetration resistance could be compared to the foot contact pressures of the roller. Proctor advocated a fill pressure of 300 psi. While this idea helped in the design of rollers, needle pressure testing lost favor and is now not commonly used.

The fundamental principle of construction control consists of measuring in-place dry density, and comparing the in-place density to a laboratory reference test. The proctor test was adopted as the method of choice for soils containing fines. The degree of compaction, or "D" ratio is the ratio of in-place dry density to the proctor maximum dry density.

By the 1940's another compaction test with higher energy was developed. This method is called the "Modified" proctor compaction test ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (D 1557). It is not clear when this higher energy was proposed, but Proctor wrote about it at the Second International Conference on Soil Mechanics and Foundation Engineering [11]. Proctor did not advocate the higher energy due to errors in determining penetration resistance. The author believes the higher energy was developed for airfield and road construction, where higher levels of compaction were desired.

Reclamation Control Philosophy and Practice -- Silty or Clayey Soils

In the area of dam construction there was debate concerning energy and water content for compaction. Reclamation tended to compact soils slightly dry of optimum, while others felt wet of optimum was better. Reclamation performed intensive studies of dam construction by measuring pore water pressures and consolidation which developed in dams during construction [12, 13]. Our data began to show that for tall dams and soils wet of optimum, undesirable pore pressures could build up during construction. Therefore, we stayed slightly dry. Reclamation also developed methods to measure pore air and pore water pressures in soils during shear and consolidation so we could predict when the soils would become saturated during construction compression and predict the resulting shear strength [14]. By performing these detailed observations in performance and laboratory studies we confirmed the engineering properties equaled or exceeded the design assumptions -- indicating compaction slight dry of optimum was preferred.

Dam designers don't always agree on the level of compactive energy that should be applied. Reclamation stayed with the "standard" effort of compaction. We specified a D ratio of 100% for dam cores, which is roughly equivalent to 95% of modified effort. We developed a heavy standard roller that was used from the 1940's through the 1970's. Today, these rollers are hard to find. The rollers were designed to obtain a D ratio of 100% for a wide range of soils. Soil was compacted in 6-inch lifts in 6 to 12 passes, and the ballast could be adjusted. Hilf wrote about these designs and the compaction properties of the fill in the 1950's for 39 dams [15]. The rollers were designed with 9 to 11-inch long (225 to 275mm) teeth. One interesting point given by Hilf was that the rollers didn't always "walk out" of the lift in over half the projects. But even though they didn't walk out compaction was found to be sufficient. Over 80 dams were eventually built using standard effort of compaction. Additional tabulations of compaction data for earth dams constructed by Reclamation can be found in the chapter on earth construction in the *Foundation Engineering Handbook* [16].

Today it is difficult to find the standard rollers, and new designs are available. Rollers today are not necessarily heavier than those used in the 1950's to 1960's. The sheepsfoot designs patterned after Bureau and Corps rollers work well in more plastic clay soils. In silty soils, the standard Bureau roller often fails to walk out. Tooth design has improved and the chevron type foot compacts silty lower plasticity soils better and clay soils sufficiently. For silty soils we have also used heavy vibrating pad foot rollers successfully and they work well in clay wet of optimum.

Reclamation has found that the "standard" effort has been sufficient for most of our projects. Table 1 summarizes our compacted fill requirements for embankment construction. Reclamation normally uses a D ratio of 100% for dam cores and fills over 50 ft. Backfill beneath pumping plants is also compacted to this level. A D ratio of 95% is required for miscellaneous fills. A D ratio of less than 95% should never be specified for earth construction, unless there is high gravel content or the properties of the fill do not need to be controlled. A D ratio of 95% using standard effort is comparable to 90% of modified effort, that is, there is about a 5% difference in D ratio between "standard" and "modified" efforts.

Reclamation control of silty or clayey soils is based on compaction tests performed on the minus #4 fraction. We call this the "Control Faction." Studies were performed to determine the effect of gravel content on the D ratio of the minus #4 fraction. Figure 1 shows the results of these tests. As can be seen, for 100% of standard effort applied to the total material, the D ratio of the control fraction reduces after gravel content reaches 20 to 30%. At 50% gravel, the maximum D ratio of the control fraction is only 50%. Our specifications allow for a reduction in the D ratio of the control fraction according to this figure. This allows for control of most soils containing fines which are controlled by proctor compaction. If the gravel content exceeds 50%, we use reliance on inspection, or use the relative density test if the fines content is less than 10 to15%.

Figure 2 shows roller curves and the density results of the total material and the control fraction for three projects. Roller curves are statistical accumulations of in-place moisture and density data from construction of a single dam. The roller curve for the total material is from all of the in-place density data. The fill density of the control fraction is computed from measurements on the gravel to be discussed later. For the Trenton Dam with no gravel, the D ratio for the control fraction exceeds the laboratory value. For Anderson Ranch with an average of 11% oversize the fill values are still higher than the laboratory. And for Cachuma Dam, with an average of 37% gravel, the D ratio of the control fraction is lower than the lab. Notice, however, that the roller curve for the total material has much higher density, and with the gravel and sand content, has satisfactory engineering properties.

Compaction Testing of Silty and Clayey Soils -- The Rapid Method

The proctor compaction test requires that the dry density of the soil be determined. Determination of water content took overnight oven drying. Reclamation was posed with a problem when constructing large dams. The materials were moved so fast, and in large amounts, that use of typical compaction curves for the maximum dry density control might not be reliable.

Dr. Jack Hilf developed the rapid method of construction control which alleviated the need to determine water content [17]. The method consists of performing a three point compaction test where the density is converted on an adjusted wet density basis. A water content correction is applied and this correction is based on compaction data for many soils. By using a three point test, the true maximum density can be easily determined.

Reclamation decided to use a larger mold than the original 4-inch proctor mold. We use a 1/20th cubic foot mold, while the proctor mold is 1/30th. The energy we use is equivalent to ASTM D698, using a 5.5 lb rammer dropped from 18 inches. All testing of silty and clayey soils are based on the minus #4 sieve size control fraction.

Figure 3 illustrates the Rapid Compaction test data sheet. After the plus #4 material is screened, a cylinder is compacted at field water content. This cylinder is called the "Fill" cylinder (box A, upper right corner Figure 3). After the wet density is obtained, there is a direct measurement of compactive effort. We call the ratio of laboratory fill

Table 1 -- *Criteria for Control of Compacted Dam Embankments*

		Percentages based on minus 4.75 mm (-No. 4) fraction					
		15 m (50 ft) or less in height			15 m (50 ft) or greater in height		
Type of material	Percentage of plus 4.75 mm (+No.4) fraction by dry mass of total material	Minimum acceptable density	Desired average density	Water content limits, w_o-w_f	Minimum acceptable density	Desired average density	Water content limits, w_o-w_f
Cohesive soil: Soils controlled by the laboratory compaction test	0 to 25 percent	$D = 95$	$D = 98$	-2 to +2	$D = 98$	$D = 100$	2 to 0
	26 to 50 percent	$D = 92.5$	$D = 95$	-2 to +2	$D = 95$	$D = 98$	Note[2]
	More than 50 percent[1]	$D = 90$	$D = 93$	-2 to +2	$D = 93$	$D = 95$	
Cohesionless soil: Soils controlled by the relative density test	Fine sands with 0 to 25%	$D_d = 75$	$D_d = 90$	Soils should be very wet	$D_d = 75$	$D_d = 90$	Soils should be very wet
	Medium sands with 0 to 25%	$D_d = 70$	$D_d = 85$		$D_d = 70$	$D_d = 85$	
	Coarse sands and gravels with 0 to 100%	$D_d = 65$	$D_d = 80$		$D_d = 65$	$D_d = 80$	

The difference between optimum water content and fill water content of dry mass of soil is w_o-w_f, in percent.
D is fill dry density divided by laboratory maximum dry density, in percent.
D_d is relative density as defined in chapter I.

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[1] Cohesive soils containing more than 50 percent gravel sizes should be tested for permeability of the total material if used as a water barrier.

[2] For high embankment dams, special instructions on placement water content limits will ordinarily be prepared.

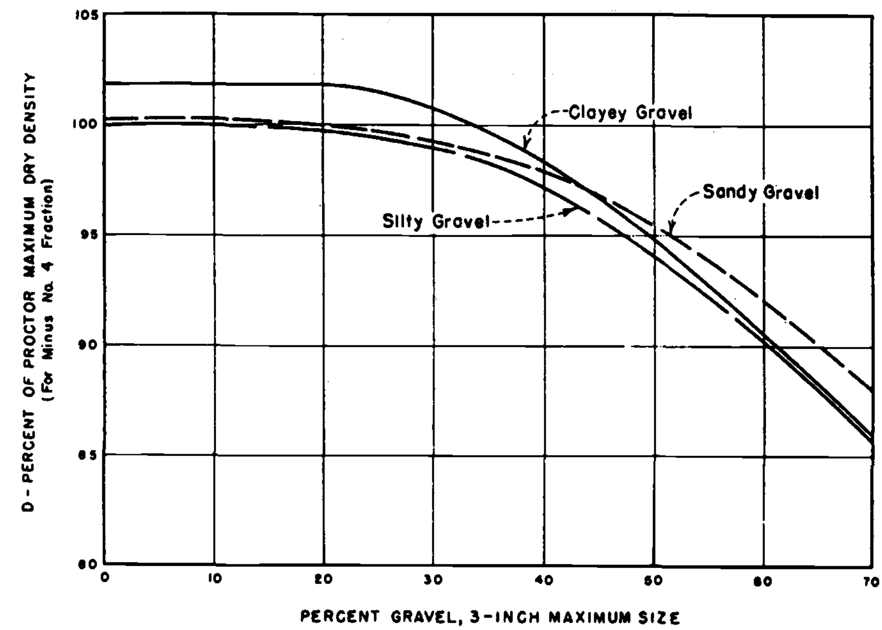

Figure 1 - *Reduction of D ratio of the minus #4 control fraction with gravel content.*

density to the field density the "C" ratio (line 30, Figure 1). If the C ratio does not exceed the required degree of compaction, the degree of compaction is not sufficient and the field inspector can be notified of the failure promptly.

Next, since most of our soils were slight dry of optimum, approximately 2% water was added and another compaction point is run. The wet density is adjusted by the 2% factor (boxes B and C, Figure 1). Another point is run at 4% added water. Normally the curve "breaks" with this addition of water. The approximate peak for optimum is determined by assuming the compaction curve is the shape of a parabola.

In Figure 3, a chart called Y2/Y4 is used to solve the shape of the parabola for the maximum density. The factor Z_m determines the offset to optimum water content. Finally, the D ratio of the fill can be computed as the wet density of the control fraction, divided by the adjusted maximum wet density from the rapid test.

For many soils, the addition of approximately 2% and 4% water is sufficient to "break" the compaction curve. For soils wet of optimum, a point may require drying, and this can be accomplished by the use of portable hair dryers with continuous stirring. For soils close to optimum condition another method can be used. Upon the first addition of method is called the Y1/Y2 method.

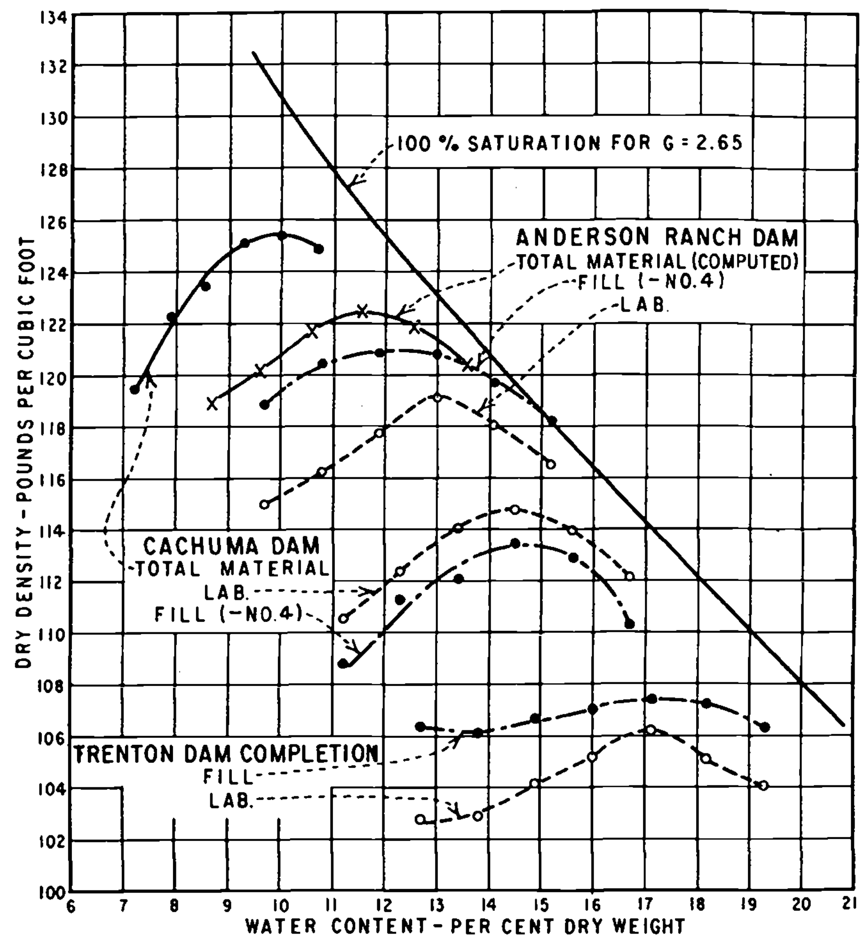

Figure 2 - *Field roller curves and laboratory curves for three dams.*

The development and theory for the test are complicated, yet temporary workers who do not have training in laboratory testing are easily taught. The procedure is straightforward, and the operators learn the behavior of the fill from dry to wet of optimum rapidly. The laboratory chief must understand the background of the method and know how to solve occasional unusual curves, which may require graphic solutions -- and now are solved by handheld calculators.

USBR 7240

7-2092A (12-86)
Bureau of Reclamation

RAPID METHOD TEST RECORD
IN-PLACE UNIT WEIGHT

Designation USBR 7240- 89

PROJECT Example FEATURE Example NO. 14 FILL Zone IA NO. 2

TESTED BY ____ DATE ____ COMPUTED BY ____ DATE ____ CHECKED BY ____ DATE ____

① MO.	DAY	SHIFT A-DAY B-SWING C-GRAVE	TEST NO.	TYPE R-REG. C-CONG D-DOUBT	STATUS A ACC R REJ	BORR. AREA NO.	STD. COMP. METHOD	SOIL CLASS	STATION	OFFSET	ELEV.
4	8	C	2	R	R	3	SR	CL-SC	2710+10	107L	2717

IN-PLACE UNIT WEIGHT DATA

(1) MASS-SAND & CAN NO. ____ lbm
(2) MASS-SAND RESIDUE & CAN ____ lbm
(3) MASS-SAND USED, (1)-(2) ____ lbm
(4) MASS-SAND IN TEMPLATE & CONE, NO. ____ lbm
SEE BACK SIDE FOR FIELD DETERMINATION - USBR 7220
(5) MASS-SAND IN HOLE, (3)-(4) ____ lbm
(6) DENSITY OF CALIBRATED SAND ____ lbm/ft³
(7) VOL. OF HOLE, (5)/(6) ____ ft³
(8) TOTAL MASS WET MATERIAL & CAN ____ lbm
(9) MASS-CAN, NO. ____ lbm

②

(10) TOTAL MASS WET MATERIAL (8)-(9) ____ lbm
* (11) WET UNIT WEIGHT, TOTAL MATERIAL (10)/(7) ____ lbf/ft³
(14) MASS-WET + NO. 4 ____ lbm
(17) SP. G., + NO. 4 ____
(21) MOISTURE CONTENT, + NO. 4 ____ %
(24) FILL UNIT WEIGHT, WET 125.8 DRY 102.6 lbf/ft³
(25) FILL MOISTURE CONTENT, w_f 22.6 %
(26) FIRST SPECIMEN UNIT WEIGHT, WET 128.9 DRY 105.1 lbf/ft³
(27) MAX. UNIT WT. AT FILL MOISTURE CONTENT WET 129.2 DRY 105.4 lbf/ft³
(28) OPT. MOISTURE CONTENT, w_o 22.0 %
$w_o = w_f + [zm \times (1 + w_f/100)]$
(29) $w_o - w_f$ -0.6 wet %
(30) C VALUE $\frac{(24)}{(26)}$ 97.6 %
(31) D VALUE $\frac{(24)}{(27)}$ 97.4 %

④ COMMENTS - 18 SPACES AVAILABLE

③ **RAPID METHOD CONTROL DATA**

			FIRST SPECIMEN	SECOND SPECIMEN	
POINT NO.		A	B	C	
ADDED WATER (%)		-2.0	0.0	+ 2.0	
MASS-MOLD & WET SOIL (lbm)		14.13	14.37	14.16	
MASS-MOLD (lbm) NO. 15		7.94	7.94	7.94	
MASS-WET SOIL (lbm)		6.19	6.43	6.22	
VOL. OF MOLD (ft³)		0.0499	0.0499	0.0499	
*WET UNIT WEIGHT (lbf/ft³)		124.0	(26) 128.9	124.6	
CONVERTED UNIT WEIGHT (lbf/ft³)		126.6		122.2	

Y_1/Y_2 CHART (ONE PERCENT)

PT. 1		PT. 1	
PT. 3		PT. 2	
Y_1 (1-3)		Y_2 (1-2)	
z_m		Y_m	
RLC		PT. 1	
$w_o - w_f = z_m + RLC$	(29)	MAX. WET UNIT WT.	(27)

Y_2/Y_4 CHART (TWO PERCENT)

PT. B	128.9	PT. C	122.2
PT. A	126.6	PT. A	126.6
Y_2 (B-A)	+2.3	Y_4 (C-A)	-4.4
x_m	1.5	Y_m	2.6
$z_m = x_m +$ ADDED WATER @ PT.A	-0.5		
RLC	-0.1	PT. A	126.6
$w_o - w_f = z_m + RLC$	(29) -0.6	MAX. WET UNIT WT.	(27) 129.2

PLUS NO. 4 (OVERSIZE)

(12) MASS-WET + NO. 4 & PAN ____ lbm
(13) MASS-PAN, NO. ____ lbm
(14) MASS-WET + NO. 4 (12)-(13) ____ lbm
(15) MASS + NO. 4 IN WATER ____ lbm
(16) VOL. OF + NO. 4 [(14)-(15)] /62.4 ____ ft³
(17) SP. G OF + NO. 4 (14)/[(14)-(15)] ____
(18) MASS-DRY + NO. 4 & PAN ____ lbm
(19) MASS-PAN, NO. ____ lbm
(20) MASS-DRY + NO. 4 (18)-(19) ____ lbm
(21) MOISTURE CONTENT + NO. 4 [(14)-(20)]/(20) × 100 ____ %
(22) VOL. OF -NO. 4 (7)-(16) ____ ft³
(23) MASS-WET -NO. 4 (10)-(14) ____ lbm
*(24) WET UNIT WT. (23)/(22) ____ lbf/ft³

FILL MOISTURE CONTENT

DISH NO.	CL15
MASS-WET SOIL + DISH	410.5 g
MASS-DRY SOIL + DISH	370.2 g
MASS-DISH	191.7 g
MASS-WATER	40.3 g
MASS-DRY SOIL	178.5 g
MOISTURE CONTENT (25)	22.6 %

AUXILIARY TESTS USBR 5300 - 89

IN PLACE UNIT WEIGHT PERFORMED IN ACCORDANCE WITH USBR 7205 - 89

+ # 4 SPECIFIC GRAVITY PERFORMED IN ACCORDANCE WITH USBR 5320 - 89

*ASSUME 1 lbf = 1 lbm

( ) STORED

Figure 3 - *Example of Rapid Compaction data sheet.*

This method is accurate to within ±0.5% water and ±1 pcf. Since water content is critical to compaction of soils with fines, a control method should be accurate to ±1% or better. The rapid method can be adapted for use with other compaction tests such as the "modified" effort of D 1557, but the soil should be checked and adjusted according to the ASTM procedure.

Control Philosophy and Practice -- Cohesionless Soils

The proctor impact test is not used by Reclamation for control of cohesionless soils used in drains and shells of dams. Terzaghi proposed the concept of relative density of cohesionless soils. Often, sands and gravels would be broken down with impact compaction tests. Since these soils were best compacted by vibration, the relative density test was developed. Reclamation played a key role in the development of this test [18]. The advantage of this test is the ability to predict engineering parameters which had been developed based on the relative density concepts of Terzaghi.

Numerous studies have been performed to relate relative density to engineering properties such as strength and compressibility. This research is still performed today. The test, however, is subject to two sources of error, since two tests are performed. Some operators and even researchers have used nonstandard methods which introduce more errors. In a major ASTM symposium on relative density testing in 1972, test errors were thoroughly evaluated for both relative density and impact compaction [19]. Tests were performed by a large group of agencies and private companies. Tavenas concluded that relative density testing was not reliable for control and it appeared that impact tests were better. From our perspective, with internal laboratories having better control and equipment, accuracy was better and errors were acceptable. In our practice, large quantities of soil are moved and the soil performance is evaluated on a statistical basis where random errors are not as important. Impact testing was also quite variable and was susceptible to errors in uniform soils. It is unfortunate that the 1970's ASTM study was performed prior to development of reliable table calibration methods. Most of the errors reported could be attributed to variation of table performance. Many of the tables used in those studies may have been found not to supply sufficient/consistent compactive effort.

Reclamation performs the test with the vibrating table in accordance with ASTM procedures. A calibration method using linear variable transducers and oscilloscope is used on a surcharged table [20]. Field laboratories are inspected at 1- to 2-year intervals. These procedures can be found in the ASTM and *Earth Manual* test procedures. With 20 years of experience with these tables, we have found some of the tables are difficult to maintain in calibration. However, we have found many methods to greatly increase table reliability. One critical component is the power supply. The power supply should be of sufficient voltage and conditioning equipment is often required. Lately we have just found that cutting the corners off the table, increases the energy delivered to the specimen.

To alleviate the vibrating table problems, research has been performed to develop a new vibrating table test. Figure 4 shows this hammer test. This test is based on British Standards Institute (BSI) procedures using either pedestal mounted or handheld vibrating

"Kango" type hammers [21, 22, 23]. This type of test was proposed by the Federal Highway Administration as a test method for road base course testing in the past [24, 25]. Extensive research has been performed by Reclamation relating vibrating table maximum density to vibratory hammer maximum density [26, 27]. Relative density molds were used in the test to handle a maximum particle size of 3-inches (75 mm). Our test results indicate the vibrating hammer maximum density equals or exceeds the performance of the vibrating table. Reclamation has tried to interest ASTM and other federal agencies in these tests -- but we have found little interest. Current earthwork practice apparently is using impact testing for control of these materials. Reclamation has not implemented the use of these hammers in their field laboratories due to lack of consensus interest, but may begin standardization efforts at ASTM.

Cohesionless soils are used in the filters, drains, and shells of an embankment. Gravel drains have maximum sizes of 3 to 5 inches. An advantage to the relative density test is that the test can use a maximum particle size of up to 3-in. (75 mm). Actually, as long as the percentage exceeding the control fraction is less than about 30 to 40%, the test can be used reliably with maximum particle sizes of over 3 in. (75 mm). A major drawback to the relative density test is the time and effort to run the tests. It takes up to 2 hours to perform the test.

Reclamation typically specifies a relative density requirement of 70%. For structure backfill and heavy loads the required relative density is 80%. With uniform sand filters, over-compaction can sometimes lower the permeability of the soil by an order of magnitude. In these cases we have limited compaction to no more than 80% relative density.

Relative Compaction Control -- Cohesionless Soils

On some projects we have performed control as a percentage of the vibrated maximum density. Use of control by "relative compaction" was discussed by Lee in 1971 [28]. In this method of control, the percentage of laboratory maximum density is used to determine the degree of compaction. We have applied this method to control of shell and miscellaneous fills.

Equations proposed by Lee to relate relative compaction to percent relative density have been confirmed in our studies of filter materials. The equation is:

$$RC = 80 + 0.2Dr \tag{1}$$

where;

RC = Relative Compaction, percentage of vibrated maximum
Dr = Relative Density -- %

The impact compaction test can be used in place of the vibrated maximum density test, but there is not agreement on the compactive effort to use. Many comparisons indicate that modified compaction effort agrees well with vibrating tables. However, modified compactive effort can cause particle breakdown. Often the standard effort of

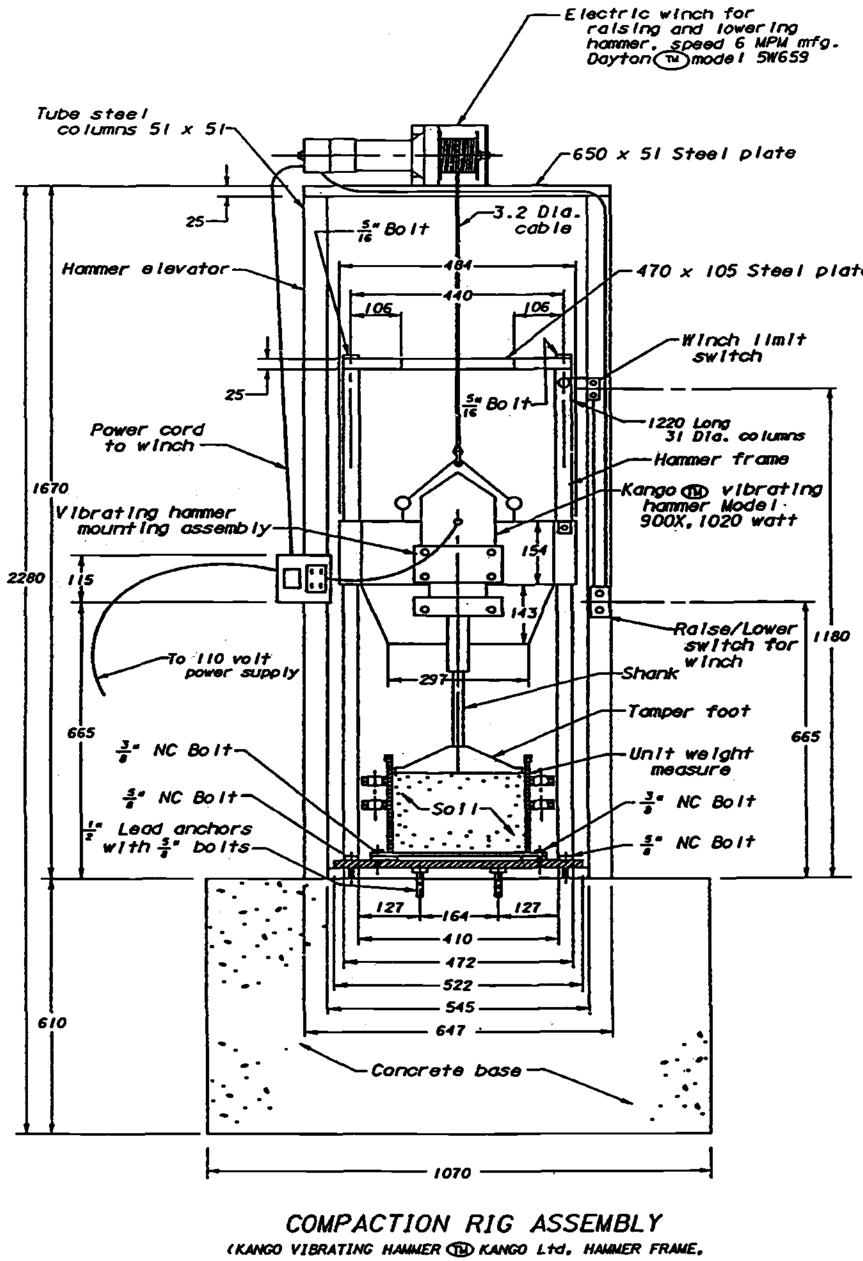

Figure 4 - *Vibratory hammer test apparatus.*

compaction can also result in a density close to vibrated maximum. Impact tests of sands should be performed dry to avoid errors in testing. We like to add an extra lift in the compaction mold when performing impact tests on sands.

Equation (1) is useful to relate relative compaction to relative density. Using equation (1),

Dr	RC
100	100
90	98
80	96
70	94
60	92
50	90

Typical Reclamation specifications require a minimum RC of 95% for most earthwork. For soils requiring additional compactive effort, an RC = 98% is specified which is about equal to 90% Dr. Specifications should never specify less than 95% relative compaction for cohesionless soils. Our research indicates that once relative density drops below 60%, the material is much more compressible, and could be subject to earthquake liquefaction. According to the above tabulation, specification of relative compaction of less than 93% will result in an unacceptable level of compaction. And what is of concern is, for those used to using "modified" efforts of compaction for fine grained soils, allowance of a 90% degree of compaction, often specified for soils with fines, is dangerous for clean coarse soils.

A 1% change in RC is about equal to a 5% relative density. Thus, use of relative density is more sensitive to changes in soil density. Uniform soils with a small differences between minimum and maximum are not controlled well by relative compaction. The use of relative density control is more reliable in these materials.

Control of Mixed Soils

For soils containing 10 to 20% fines, it is difficult to determine whether rapid method or relative density will control. Our specifications state that in these mixed soils, both tests should be run to determine which one will require the highest fill density. This issue is usually resolved at the start of construction.

In-place Density Testing

For silty and clayey soils, it is standard Reclamation procedure to use an 8-inch diameter sand cone. The depth of the hole is generally 10-12 inches. This provides us with 40 lb_m of soil. The rapid method requires 22.5 lb_m of minus #4 soil for the test. It is standard practice to use material from the test hole for the rapid test, providing irrefutable proof of the degree of compaction.

For soils with more than 5% gravel, the gravel is screened and washed. The volume of rock is determined by weighing in air and weighing in water. The gravel water content is assumed to be the surface saturated dry condition, and on large projects the water content is correlated to the specific gravity of the gravel. The volume and mass of gravel can then be computed and the wet density of the minus #4 control faction calculated. The control fraction density can then be used with the rapid compaction method and D ratio reductions according to Figure 1 can be applied to the D ratio.

A common error in sand cone testing is squeezing in material wet of optimum. Sand cone test holes begin to squeeze when proctor needle readings are less than 500 lb/in^2. These errors are easily detected by calculating the high degree of saturation in excess of 100%. Error can be reduced by using cribbing to distribute operator pressure.

Density sand must be carefully monitored to assure accurate test results. We tend to use very uniform density sand in the #16 to #30 sieve size range. Finer sands can have problems with humidity and variation in moisture. Our density sand is protected by heat tape in the storage bins and kept sealed in containers to avoid humidity effects.

Nuclear gages are used in sand filters and in roller compacted concrete (RCC). With RCC, the consistency is such that a sand cone hole will squeeze in during testing and have errors. The nuclear gage can be reliably calibrated with RCC material because the production plant and ingredients are well controlled. For RCC construction, a vibe table is used for maximum density determination. The vibrating hammer test discussed earlier is used for control of stiffer coarse grained soil cement material. These are special control cases. For fine grained soil cement, generally less than 10% gravel, rapid compaction testing is used.

Nuclear gages are not often used for soils which are controlled by the rapid method at Reclamation. The sand cone test is a reliable measure of wet density and soil must be excavated at the test site anyway. Nuclear water content corrections are not needed for the rapid method, therefore nuclear density could be successfully coupled with rapid control testing.

For in-place density of coarser soils we use large sand cones and test pits with either sand or water replacement. All of these methods are standardized at ASTM and Reclamation. For rockfill control, the large test pits with water replacement are used.

Special measures are required when the in-place density of natural alluvium must be determined. These soils are bedded in uniform layers. If the density test hole mixes two uniform layers, the resulting mixture may be well graded. The maximum density of the combined well graded mixture will be much higher than that for the individual uniform layers. If the layers are combined the calculated in-place relative density will be very low and possibly even negative values can be predicted. For thin uniform layers, smaller hole volumes must be used to avoid mixing layers.

Statistical Control of Earth Construction Data

Statistical control of earth control data is important on large earth moving projects. Reclamation has written several summary papers summarizing statistical data on many projects [15,16]. Recently, Reclamation has developed a computer program, PCEARTH, which performs many statistical analyses. The program keeps databases on borrow areas and has the following five control areas to accumulate data:

1.) Soils with Fines - Controlled by Rapid Method - or you can input a lab maximum
2.) Cohesionless Soils - Controlled by Relative Density - or Relative Compaction
3.) Physical Properties - Gradation control for aggregates, borrow data
4.) Soil Cement - fine grained soil cement controlled by Rapid Method
5.) Soil Cement Slurry - batching and compression strength monitoring - pipes

An example of PCEARTH tabular output is shown in Figure 5. This program runs on IBM PC's under Windows 3.1 and Windows 95, and is a DOS-based program. The program generates tabular and graphical data summaries. The program can be downloaded from the Internet www.usbr.gov/merl at no cost.

State of Earthwork Control Practice

Often Reclamation personnel are requested to check control methods used on such projects as dams or clay liner/cover applications for hazardous waste facilities for other federal or state agencies. Earth control practice today, in both private and government practice, is not in good condition. There are increased pressures to reduce testing costs for control purposes. Testing laboratories today often do not participate in writing specifications and often incorrect testing specifications are cited. Use of modified compaction effort has migrated from state transportation departments into most of the practice today. The specifications rarely discuss how to handle oversize corrections. Some specifications allow for the contractor to arrange for their own contract testing, allowing for potential conflict of interests.

Observations of private practice indicate the nuclear gage is used for most all control testing. Operators have rarely performed accurate application of moisture or count corrections in the field. If these corrections are not made, errors in moisture determination can exceed 1% , which may not be acceptable depending on the method of control. An extreme example of incorrect application is the use of nuclear gages on a mine spoil material, rich in lead bearing soils. Our check testing with sand cones revealed that the nuclear gages were measuring soil densities of 10 pcf greater than true insitu density. These types of errors are unacceptable in engineering practice.

There is increased reliance on the use of only a few compaction curves for the borrow. This is acceptable for smaller projects, or where the soil doesn't vary, but on larger projects or where the soil varies, more laboratory compaction tests should be performed. If the person using the nuclear gage or any method does not understand the compaction properties of the soil an incorrect typical curve can be selected.

Why does practice today favor the use of impact compaction testing for control of clean coarse grained soils? Probably due to the cost considerations listed above. Also, the difficulty in obtaining reliable performance of vibrating tables has frustrated some agencies. The impact test sometimes results in breakdown and alteration of the material grading curve invalidating the test. Further, the ASTM impact tests are limited to a maximum of 30% gravel and causes confusion in oversize corrections. There have been numerous claims regarding use of impact compaction for control of road and runway base course soils which often are too coarse for control by impact methods.

09/14/1995

U.S. Bureau of Reclamation
SUMMARY OF FIELD AND LABORATORY TESTS OF COMPACTED FILL
CONTROLLED BY THE LABORATORY COMPACTION METHOD

Page 1

PROJECT: Judge Roy Bean Reservoir
FEATURE: Pumping Plant
SPECIFICATION NO: LWP-1870

SPECIFICATION REQUIREMENTS:
Min percent compaction (D-Value): 95.00 %
Max DRY Wo-Wf: 2.00
Max WET Wo-Wf: -2.00
IF D-Value ≥ 98.00 % the Max DRY Wo-Wf is extended to 4.00

PERIOD OF REPORT: 11/01/1994 - 11/30/1994
TOTAL MATERIAL PLACED: 16,840 Cubic Yards
UNITS PER ACCEPTED TEST: 200 Cubic Yards

FILL NAME: 1 - COMPACTED BACKFILL

TEST NUMBER		LOCATION			FIELD DENSITY TESTS										LABORATORY VALUES					COMPACTION CONTROL VALUES						
												MINUS #4														
MONTH DAY SHIFT NUMBER TYPE STATUS	BORROW	STATION	OFFSET	ELEV	METH OF COMP	USBR TEST METH	WET DEN TOT MAT (PCF)	MOIS TOT MAT (%)	PLUS #4 (%)	SPEC GRAV +4	SPEC GRAV -4	DRY DEN (PCF)	MOIS CONT (%)	DEG OF SAT (%)	COMP TEST METH	COMP CYL (PCF)	MAX DRY DEN (PCF)	OPT MOIS CONT (%)	DEG OF SAT (%)	Wo-Wf (%)	C (%)	D (%)	PENE NEED (PSI)	VISUAL SOIL CLASS	OTHER TESTS	REMARKS
11-01-A-01-R-A	2	3+00	10R	5912	SR	7205	136.8	11.6	0.0	AV	AV	122.6	11.6	88	R	134.2	120.5	10.8	77	-0.8	101.9	101.7	NA	SC	N	
11-02-A-01-R-A	2	2+15	120R	5887	SR	7205	130.0	9.4	4.5	2.54	AV	117.8	9.4	62	R	128.2	120.1	11.6	82	2.2	100.5	98.1	NA	SC	N	
11-02-A-02-R-A	2	2+30	135R	5896	SR	7205	134.2	10.1	0.0	AV	AV	121.9	10.1	75	R	132.8	121.4	11.2	82	1.1	101.1	100.4	NA	SC	N	
11-03-A-01-R-R	2	2+40	135R	5904	VR	7205	124.0	11.0	5.0	2.56	AV	110.7	11.0	59	R	134.2	121.1	11.4	83	0.4	91.6	91.4	NA	SC	N	
11-03-A-02-X-A	2	2+40	135R	5904	VR	7205	134.5	10.4	0.0	AV	AV	121.8	10.4	77	R	132.6	121.6	11.4	84	1.0	101.4	100.2	NA	SC	N	RE 11-03-94A1
11-04-A-01-R-A	2	1+90	170R	5906	SR	7205	126.5	10.0	0.0	AV	AV	115.0	10.0	61	R	127.0	116.4	11.9	75	1.9	99.6	98.8	NA	SC	N	
11-10-A-01-R-R	2	2+30	134R	5901	VR	7205	118.3	10.1	0.0	AV	AV	0.0	0.0	0		0.0	0.0	0.0	0	0.0	0.0	0.0	NA	SC	N	
11-10-A-02-X-A	2	2+30	134R	5901	VR	7205	130.6	10.1	0.0	AV	AV	118.6	10.1	68	R	131.8	120.7	11.4	82	1.3	99.1	98.3	NA	SC	N	RE 11-10-94A1
11-11-A-01-R-A	2	2+80	100L	5885	VR	7205	135.1	10.9	0.0	AV	AV	121.8	10.9	81	R	133.4	120.3	10.7	76	-0.2	101.3	101.2	NA	SC	N	
11-14-A-01-R-A	2	2+90	100L	5896	SR	7205	137.9	11.2	0.0	AV	AV	124.0	11.2	89	R	133.6	120.1	11.0	77	-0.2	103.2	103.2	NA	SC	N	
11-14-A-02-R-A*	2	3+00	120L	5895	SR	7205	125.8	9.6	0.0	AV	AV	114.8	9.6	58	R	131.0	121.4	11.2	82	1.6	96.0	94.6	NA	SC	N	ACC/BY/INSP
11-15-A-01-R-A	2	2+30	103L	5890	PT	7205	135.7	9.3	6.0	AV	AV	123.1	9.3	72	R	130.6	121.4	10.9	80	1.6	103.0	101.4	NA	SC	N	
11-16-A-01-R-A	2	2+40	110L	5895	PT	7205	128.7	10.4	5.3	AV	AV	115.4	10.4	64	R	133.4	120.8	10.4	75	0.0	95.5	95.5	NA	SC	N	
11-17-A-01-R-A	2	2+60	103L	5894	PT	7205	127.4	9.5	0.0	AV	AV	116.3	9.5	60	R	127.2	119.3	11.6	80	2.1	100.1	97.5	NA	SC	N	ACC/BY/INSP
11-21-A-01-R-R	2	2+81	75L	5903	SR	7205	124.4	7.8	0.0	AV	AV	115.4	7.8	48	R	125.2	120.2	10.8	76	3.0	99.4	96.0	NA	SC	N	
11-21-A-02-X-A	2	2+81	75L	5903	SR	7205	136.4	9.9	0.0	AV	AV	124.1	9.9	79	R	131.0	120.0	11.2	79	1.3	104.1	103.4	NA	SC	N	RE 11-21-94A1
11-21-A-03-R-A	2	3+20	75L	5908	SR	7205	134.1	9.3	3.4	AV	AV	122.0	9.3	69	R	129.0	119.9	11.0	77	1.7	103.4	101.8	NA	SC	N	
11-29-A-01-R-A	2	2+15	79L	5893	PT	7205	131.3	10.8	0.0	AV	AV	118.5	10.8	72	R	128.2	118.1	12.8	85	2.0	102.4	100.3	NA	SC	N	
ACCEPTED TESTS THIS PERIOD: 16						AVG.	132.3	10.3	1.5	2.54		119.7	10.3	72		131.1	120.1	11.3	80	1.0	100.7	99.7				
						S.D.	3.8	0.8	2.4	0.00		3.3	0.8	10		2.4	1.4	0.6	3	1.0	2.5	2.5				
						C.O.V.	2.9	8.2	**.*	0.00		2.8	8.2	14		1.9	1.1	5.2	4	93.0	2.5	2.5				

AV: Assumed Values used for Specific Gravity dependant calculations. +#4=2.65 -#4=2.65

Figure 5 - *Example output from program PCEARTH.*

The industry seriously needs a test method for control of base coarse materials containing 10 to 15% fines and up to 1.5 to 3 inch maximum size. Use of the existing impact compaction tests is not reliable for these materials because of excessive oversize. The vibrating hammer test would likely be a solution for this problem. Reclamation's current design would provide the most consistent maximum density data. There is a simpler handheld method which is also feasible for private industry use. The vibrating table test would likely give reliable data on these soils when tested in the dry condition.

State agencies often dictate earth control procedures for hazardous waste applications. These agencies often do not have personnel experienced in earthwork control. These agencies should consider more proven methods of control especially, when the public safety is at risk. Unfortunately, the error prone control methods listed above persist.

Conclusions

Reclamation uses the rapid method of control for soils containing fines. The rapid method is a three-point adjusted wet density impact compaction test. The rapid method of control is based on "standard" effort. The use of "standard" effort is sufficient for most earthwork needs. Most control practice outside of our agency is based on a "modified" effort of compaction. There is only about a 5% difference in degree of compaction between the two tests.

The minus #4 fraction of the soil is used in the rapid compaction control. Simple gravel correction factors are used. These correction factors are included in our specifications. These factors could also be used in ASTM testing.

Sand cone tests are used along with the rapid method. Enough soil is taken from the test hole to run the test. A rapid compaction test is performed on every sand cone test. This provides irrefutable data as to the degree of compaction.

Nuclear gages are used for roller compacted concrete and filter materials.

The relative density test is a very reliable test for cohesionless soils. It can control coarse materials of 3 to 7 inch maximum size. The test suffers from errors and the tables are difficult to keep calibrated. For miscellaneous fills, use of a percentage of laboratory maximum dry density can be used for control -- but for uniform soils, use of relative density allows for closer, more accurate control.

The state of earthwork control practice today is disappointing. Many errors persist and there is increasing pressure to cut testing costs.

A new vibratory hammer test should be considered for standardization by ASTM. This test would solve current problems with control of coarse-grained soils.

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Variability in Field Density Tests

Noorany, I., Gardner, W. S., Corley, D. J., and Brown, J. L., "**Variability in Field Density Tests,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: This paper describes a comparative study of the three most commonly used field density tests: sand-cone, nuclear, and drive-cylinder. In order to evaluate the range of variability of these tests, a large-scale soil compaction apparatus was constructed so that soil could be compacted to a known water content and dry density under close laboratory control conditions before running field density tests. The equipment consisted of a 4-ft (122-cm) mold, and a hydraulic system for compacting the soil in ten 4-in. (10-cm) thick layers inside the mold. A cohesive soil with gravel up to ¾-in. (19 mm) in size was used. Five series of tests were performed on this soil compacted to five different water contents and dry densities. In each series, side-by-side sand-cone, nuclear, and 3-in. (7.6-cm) drive-cylinder tests were made. It was found that sand-cone test results were closer to the placement values than the nuclear test results. This was partly due to inaccuracies in water content readings by the nuclear device. When the water content data measured by the nuclear device were ignored and the placement water contents measured by oven-drying were used instead, the results of the nuclear tests became more accurate, but the range of variability in the measured data did not decrease significantly. The drive-cylinder test had a bias toward underestimating the actual field density, primarily due to sampling disturbance. Despite careful control of test conditions, data from all three test methods had a wide range of variability.

Keywords: soils, compaction control, field density, sand-cone, nuclear, drive-cylinder, statistical evaluation, large-scale tests

Field Density Tests

The most commonly used in-place density tests are the sand-cone, nuclear, and drive-cylinder. Each test has a number of limitations that can lead to measurement errors

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and variability in the results. The key limitations are briefly discussed in the following sections.

The standard sand-cone test, ASTM Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method (D 1556), requires hand excavating a density hole and measuring the mass of the material excavated as well as the volume of the hole. The hole volume is measured indirectly by pouring sand in the hole and measuring the mass of the sand required to fill up the hole. The standard sand-cone device for pouring sand into the density hole has a base diameter of 6.5 in. (16.5 cm), and is suitable for testing soils that do not have significant amounts of coarse material larger than 1.5 in. (38 mm). A larger apparatus with 12-in. (30-cm) base diameter and proportions similar to those of the standard cone is used for bigger test holes when particles larger than 1.5 in. (38 mm) are prevalent. For soils containing significant amounts of coarse particles, the ASTM Density of Soil and Rock in Place by the Sand Replacement Method in a Test Pit (D 4914) or Density of Soil and Rock in place by the Water Replacement Method in a Test Pit (D 5030) are appropriate. The sand-cone test is not suitable for soft or friable soils, or for soils that deform easily, or for those soils that may undergo a volume change in the excavated hole during the test.

The in-place density test by nuclear method, ASTM Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth) (D 2922), and in-situ moisture content by nuclear method, ASTM Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth) (D 3017), are used to measure the wet density and the water content of compacted soils. These methods are suitable to test depths up to 12 in. (30 cm), but the measured densities may be affected by chemical composition and heterogeneity of the soil being tested. The technique also exhibits spatial bias, in that the apparatus is more sensitive to the density of the soil in close proximity to the surface. Furthermore, oversize rocks or large voids may cause incorrect density readings. The same factors also affect the measurement of water content by nuclear device except that the chemical composition of the soil may dramatically affect the measurement.

The drive-cylinder method, ASTM Test Method for Density of Soil in Place by the Drive-Cylinder Method (D 2937), is used for obtaining a relatively undisturbed soil sample by driving a thin-walled cylinder into the soil. Drive cylinders with diameters of 3 to 5.5 in. (7.5 to 14 cm) or larger have be used for measuring densities of fine-grained soils. The test is not recommended for soft, highly plastic soils, non-cohesive soils, or any soil that can easily deform. According to the D 2937 standard, the use of this method in soils containing particles coarser than 4.75 mm "may not yield valid results if voids are created along the wall of cylinder during driving, or if particles are dislodged from the sample ends during trimming." Furthermore, the ASTM standard stipulates that when this test is used as a basis of acceptance of a compacted fill, the drive-cylinder volumes must be as large as practical and not less than 0.03 ft^3 (850 cm^3). Despite these limitations, a drive cylinder commonly used in California is a 3-in. (7.6-cm) in diameter cylinder with a volume of 293 cm^3, which is too small for compaction control or forensic determination of relative compaction of fills.

Variability in Field Density Tests

A study for evaluating variability in the results of field density tests was carried out during compaction observation and testing of three structural fills with the participation of nine geotechnical engineering firms in San Diego, California (Noorany, 1990). The objective of the study was to determine the statistical variations in field density values when measured by different operators and different methods. It was found that the measured relative compactions had a wide range, 17 to 20% of the mean, with relative compaction values as low as 77%, even though each site had been tested and approved as having met the minimum required relative compaction of 90%. The variability was particularly high for the site with significant amounts of gravel and cobbles. It was not possible to determine from this investigation whether the variability in the field density values was primarily due to actual variations in the fill's density or due to errors in the field density test methods.

The next logical step in investigating the potential variability in field density tests is to compact a large volume of soil in the laboratory, where the placement water content and dry density can be closely controlled, and then run in-place density tests. A study of this type using a box filled with soil compacted with a hand-held compactor was reported by Dreasen (1984). In the current study, a large-scale compaction test system was developed and five series of comparative field density tests were performed. Each series included density tests by using nuclear method, sand-cone method, and drive-cylinder method. The test equipment, soil properties and test results are described in this paper.

Test Equipment

The compaction system consisted of a large mold and a hydraulic compactor. The compaction mold was a rigid soil bin made of steel, with an inside diameter of 46 in. (117 cm) and a depth of 48 in. (122 cm) in three segments coupled together. The compaction mold was assembled on the base platform of a steel frame 7-ft by 7 ft (2.1 m by 2.1 m) in horizontal dimensions and 14 ft (4.3 m) high. The base platform consisted of a thick steel plate supported on a grid of welded beams. The top of the frame had two girders that supported a 150-ton (662-kg) double-acting hydraulic cylinder. The hydraulic cylinder pushed a loading ram which was connected to a compaction foot with an imprint 46 in. (117 cm) long, 7 in. (18 cm) wide on the ends and 3.5 in. (9 cm) wide in the middle. The ends of the compaction foot were curved to fit the mold snugly. After each application, the foot was raised and rotated before the next application. The downward travel of the compaction foot was monitored by means of a displacement transducer, thereby controlling the thickness of each compacted lift. The displacement transducer was mounted on a separate frame to avoid any effects of the elastic deformation of the main frame.

The hydraulic cylinder activating the downward push of the compaction foot was controlled by a push-button electrical switch, and the movement of the foot was measured to 0.01 in. (0.25 mm) on the digital readout of the displacement transducer. An initial zero reading was established by lowering the compaction foot to the bottom of the empty mold and loading the foot to a pressure of the same magnitude typically observed during compaction of the soil layers. Thereafter, the thickness of various lifts was controlled by lowering the foot to exactly 4.00 in. (10 cm) above the base for the first lift, 8.00 in. (20 cm) for the second lift, and so on.

When compaction operations were completed, the base plate with the mold was pulled out of the compaction frame, so that tests could be made on the compacted soil.

Soil Placement and Compaction

For each test series, the soil was moisturized to the desired water content in 1.5 ft^3 (0.04 m^3) batches in a large mixer. The mixed soil was then stored in tightly sealed containers for several days. In order to eliminate any variations between different batches of moisturized soil, all containers were then emptied in a pile and mixed with shovels to a uniform mix and once again placed in sealed containers and stored for at least one day before compaction. The necessary mass of soil for each 4-in. (10-cm) lift was calculated from the volume of each lift and the desired unit weight (wet density). The volume of each lift was determined by measuring the inside circumference of the mold at 4-in. (10-cm) intervals by means of a pi-tape. To improve uniformity of soil placement, the soil for each lift was weighed in four equal quarters and placed in four quarter-circle zones and leveled by hand trowels. Typically, the thickness of the placed and leveled soil was approximately 6.5 in. (16.5 cm). For each lift, two water content samples were taken and the average of these two measurements was used as the representative water content of the lift. The loosely placed lift was then compressed into a 4-in. (10-cm) thickness by activating the hydraulic system, which pushed the compaction foot down in steady motion. This was accomplished in three stages, first, the entire surface area of each lift was compressed into a 5-in. (12.7-cm) thickness, next to a 4.5-in. (11.4-cm) thickness, and finally to a 4-in. (10-cm) thickness.

The process of compaction of ten 4-in. (10-cm) lifts in the 4-ft (1.22-m) diameter mold took about 10 hours. The mold was then covered overnight and density tests commenced the next morning. During the entire process of soil preparation, compaction, and density tests, a humidifier maintained a high level of humidity in the laboratory.

Soil Properties

The soil used in this study was obtained from a test boring in a residential fill at Villa Trinidad development in San Diego, California. The fraction finer than ¾ in. (19 mm) used for this test program was classified as clayey sand (SC). The soil's grain size distribution curve is shown in Figure 1. The soil had a liquid limit of 35, a plasticity index of 21, and a specific gravity of solids of 2.66. The results of the ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (D 1557) Method C compaction test are shown in Figure 2. The maximum dry density from this test was used as a reference for computing relative compaction, RC, for five series of tests in the large-scale mold. The water content and dry density values for these five series of tests are also shown in Figure 2.

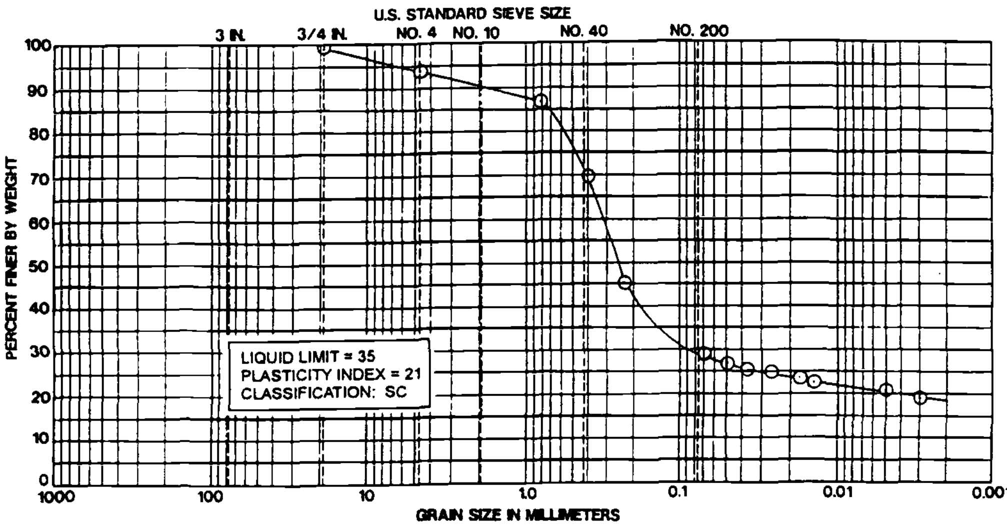

Figure 1 – *Results of Classification Tests*

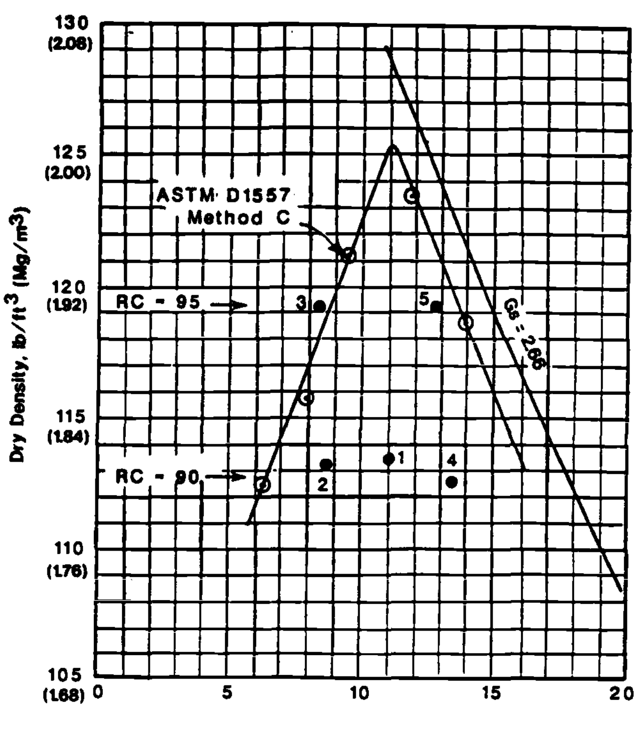

Figure 2 – *Results of Compaction Test, and Placement Conditions for Test Series 1 through 5*

Placement Data

Table 1 summarizes the placement water content and dry density data for five series of tests. Each "mean" value is the average of 10 measurements for 10 compacted layers, and the "range" is the difference between the highest and the lowest value of 10 measurements. It can be seen that uniform placement conditions were achieved in all five test series with a narrow range for water content and dry density. Nevertheless, it is seen that in compacting large volumes of soil, even under tight laboratory conditions, it is very difficult to control water content any closer than 0.7% and dry density any closer than about 1.5 pounds per cubic foot (0.02 Mg/m^3) or about ±1% of the target value.

Table 1 – *Placement Data*

Test Series	Water Content, % Mean	Range	Dry Density Mean lb/ft^3 (Mg/m^3)	Range lb/ft^3 (Mg/m^3)	Relative Compaction RC [1], %	Range
1	11.4	0.7	113.3 (1.82)	1.3 (0.021)	90.3	1.0
2	8.7	0.6	113.0 (1.81)	1.5 (0.024)	90.1	1.2
3	8.3	0.4	118.6 (1.90)	1.4 (0.022)	94.6	1.1
4	13.3	0.7	112.4 (1.80)	1.1 (0.018)	89.6	0.9
5	12.8	0.7	119.2 (1.91)	1.5 (0.024)	95.0	1.2

[1] RC denotes relative compaction or percent compaction

Density Tests

Three types of density tests were made on the compacted soil: nuclear, sand-cone, and drive-cylinder. Table 2 shows the number of each type of test in each test series.

Table 2 – *Number of Density Tests*

Test Series	Number of Tests Nuclear	Sand Cone	Drive Cylinder
1	89	5	13
2	32	12	12
3	40	8	11
4	44	12	12
5	48	12	12

Starting from the top of the compacted soil, nuclear moisture and density measurements were made in two or more locations to a depth of 8 in. (20 cm). Then, after hand excavating the soil to a depth of 8 in. (20 cm), another series of measurements were made and so on. In each test location and at every depth, two sets of readings were taken in two directions 180° apart. All test locations were selected a minimum distance of 12 in. (30 cm) away from the side and the base of the mold.

The nuclear-density tests were performed by four experienced soil technicians and engineers licensed for nuclear density testing; three of them were from local geotechnical consulting firms, one was from a public agency. Each operator brought his nuclear-density instrument and his density calibration block, on which he ran a calibration check before commencing tests on the compacted soil. No attempt was made to calibrate various nuclear density instruments against one another; the objective was to see how the results would turn out when different operators calibrate their own equipment before use.

The sand-cone tests were made using the standard 6.5-in. (16.5 cm) in diameter sand cone, at 6- to 8-in. (15 to 20 cm) depth intervals at different locations within the mold. Excavated soil was weighed to 0.01 g. Water contents were measured by oven drying.

The drive-cylinder tests were run using a 2.8-in. (7.1-cm) high cylinder with 3-in. (7.6-cm) outside diameter, 2-7/8-in. (7.3-cm) inside diameter, and a volume of approximately 0.01 ft^3 (284 cm^3). This type of drive-cylinder is often used in compacted structural fills in California. After driving the cylinder into the soil using the equipment described in D 2937, the soil adjacent to the cylinder was excavated by hand tools to remove the cylinder without disturbance. The presence of gravels up to ¾ in. (19 mm) interfered with trimming the ends of the cylinder, and dislodging any gravel and filling and packing the hole typically led to a decrease in the measured density.

Measured Water Content Data

Table 3 summarizes the water content data measured using different field density tests. Because the water contents in sand-cone and drive-cylinder tests were measured by oven drying, the results are close to the actual placement water content in each series. Excluding the nuclear method, the measured water content data in Table 3 has an average standard deviation of 0.30, an average coefficient of variation of 2.69, and an average range of 1.1%. In contrast, the water contents measured by nuclear method resulted in considerably lower accuracy with an average standard deviation of 0.77, an average coefficient of variation of 7.96, and an average range of 2.9%. This is because the nuclear method is an indirect procedure for measuring water content by counting the hydrogen present in the form of water. The chemical composition of the soil may affect the measurement, and adjustments may be necessary by calibration tests on samples of known water content, as described in D 3017. Furthermore, measurements by different operators using different instruments can lead to considerable variability in the results. In this study, the nuclear device used by one of the four operators gave consistently lower water contents than the other three devices, and appeared to be off in calibration. This is not uncommon, and a water content correction factor, or direct measurement of water content by oven drying is necessary for every project.

Table 3 – *Summary of Moisture Content Data from Various Tests*

Test Series	Placement, w [1] %	Test Method	Measured w			
			Mean	SD [2]	CV, % [3]	Range [4]
1	11.4	Nuclear	12.2	0.64	5.25	2.6
		Sand Cone	11.5	0.43	3.74	1
		Drive Cylinder	11.5	0.26	2.26	0.9
2	8.7	Nuclear	8.9	0.55	6.18	1.9
		Sand Cone	8.5	0.20	2.35	0.7
		Drive Cylinder	8.5	0.36	4.23	1.4
3	8.3	Nuclear	7.0	1.06	15.1	3.7
		Sand Cone	8.3	0.12	1.45	0.4
		Drive Cylinder	8.3	0.17	2.05	0.6
4	13.3	Nuclear	11.8	1.21	10.23	4.4
		Sand Cone	12.9	0.23	1.78	0.9
		Drive Cylinder	13.2	0.20	1.52	0.7
5	12.8	Nuclear	13.0	0.40	3.10	1.8
		Sand Cone	12.8	0.45	3.51	1.7
		Drive Cylinder	12.9	0.33	2.56	1.1

[1] w denotes water content in percent
[2] SD denotes standard deviation.
[3] CV denotes coefficient of variation (the ratio of SD to mean).
[4] Range is the difference between the highest and the lowest values.

Measured Relative Compaction Data

Table 4 presents a summary of the measured relative compaction data by different test methods. Because all of the relative compaction (RC) values were computed based on a single laboratory maximum dry density value of 125.4 lb/ft^3 (2 Mg/m^3), this summary in effect reflects the variability of the in-place dry density measurements, but it is presented in terms of the final compaction control parameter, R.C.

Although the data trends for the five test series are not identical, the sand-cone method had the lowest average standard deviation and range overall. In order to assess the accuracy of the test method, the mean values of the measured RC are compared with the placement RC values in each test series. To illustrate this more clearly, the placement RC, the measured RC, and the range of RC values from Table 4 are plotted in Figures 3, 4, and 5. The horizontal bands marked with "P" in these figures indicate the placement range of values (from Table 1); the vertical lines show the range of measured values in each test series, and the dots depict the mean of the measured values. From these plots, it can be concluded that:

- The sand-cone method (Figure 3) had the best accuracy, with the measured mean values in most of the test series being closest to the actual placement value. Nevertheless, the measured RC values had a range of 4% to 9%, with extreme values being as much as 5% off the placement value.
- The nuclear method (Figure 4) was somewhat less accurate, in that the measured mean RC values in five series of tests were not as close to the placement values as in the sand-cone tests. Besides, nuclear tests showed a higher degree of variability with a range of 4.2 to 10.8%, and extreme values about 10% off the placement value.

Table 4 – *Summary of Relative Compaction Data*

			Measured RC			
Test Series	Placement RC [1] %	Test Method	Mean %	SD [2]	CV [3] %	Range [4] %
1	90.3	Nuclear [5]	87	2.33	2.68	10.8
		Adjusted Nuclear [6]	87.6	2.13	2.44	9.8
		Sand Cone	90	1.62	1.80	4
		Drive Cylinder	88	2.30	2.61	8
2	90.1	Nuclear	87.7	1.03	1.17	4.2
		Adjusted Nuclear	87.8	0.95	1.08	3.9
		Sand Cone	92.5	2.10	2.26	7
		Drive Cylinder	86	2.27	2.64	8
3	94.6	Nuclear	94.6	2.45	2.59	10.2
		Adjusted Nuclear	93.4	2.15	2.30	8.8
		Sand Cone	95.5	1.32	1.39	4
		Drive Cylinder	91	2.13	2.34	8
4	89.6	Nuclear	90.6	3.16	3.49	10.6
		Adjusted Nuclear	89.3	2.26	2.53	8.5
		Sand Cone	90.5	2.39	2.63	9
		Drive Cylinder	88.5	2.14	2.41	8
5	95.0	Nuclear	92.8	1.17	1.27	4.6
		Adjusted Nuclear	92.9	1.08	1.16	4.3
		Sand Cone	95.5	1.71	1.79	5
		Drive Cylinder	93	1.49	1.60	5

[1] RC denotes relative compaction, percent of maximum dry density of 125.4 lb/ft^3 (2.0 Mg/m^3) in D 1557 test.
[2] SD denotes standard deviation.
[3] CV denotes coefficient of variation, (ratio of SD to mean).
[4] Range is the difference between the highest and the lowest values.
[5] Nuclear based on water contents measured by nuclear density device.
[6] Adjusted Nuclear based on actual placement water contents measured by oven drying.

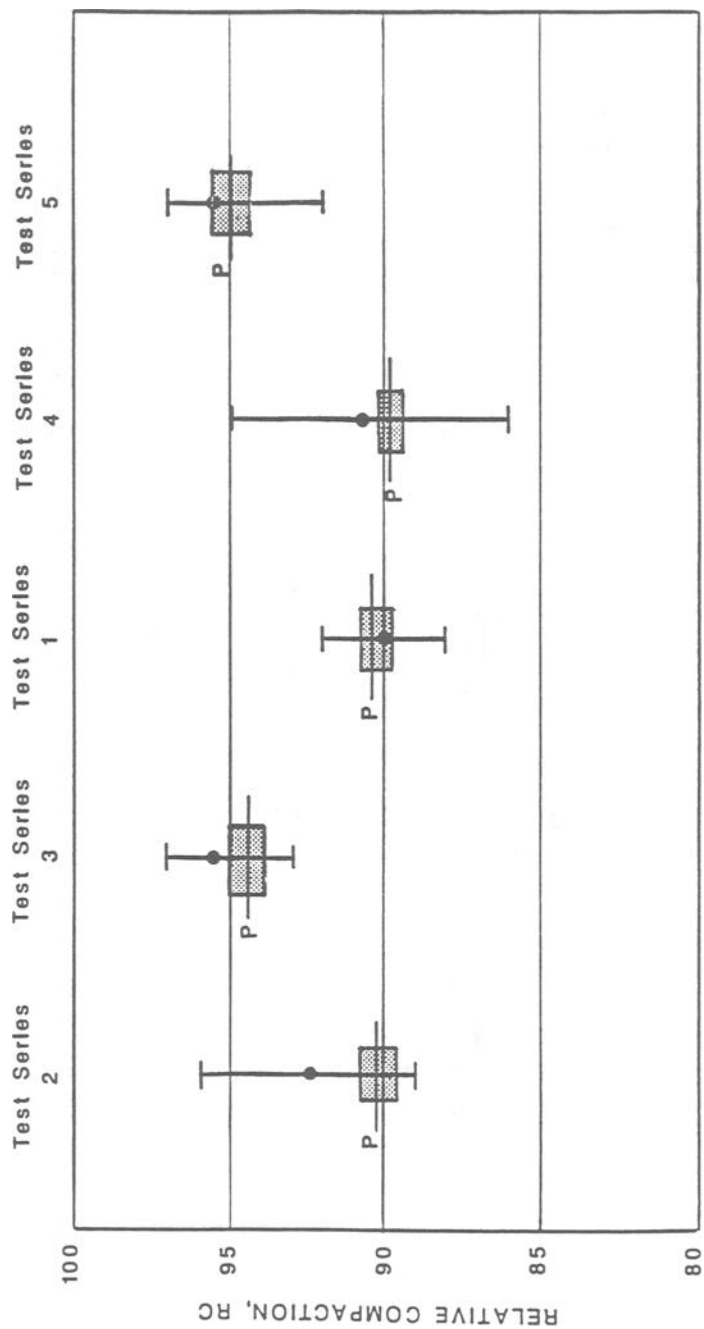

Figure 3 – *Summary of Data from Sand-Cone Tests*

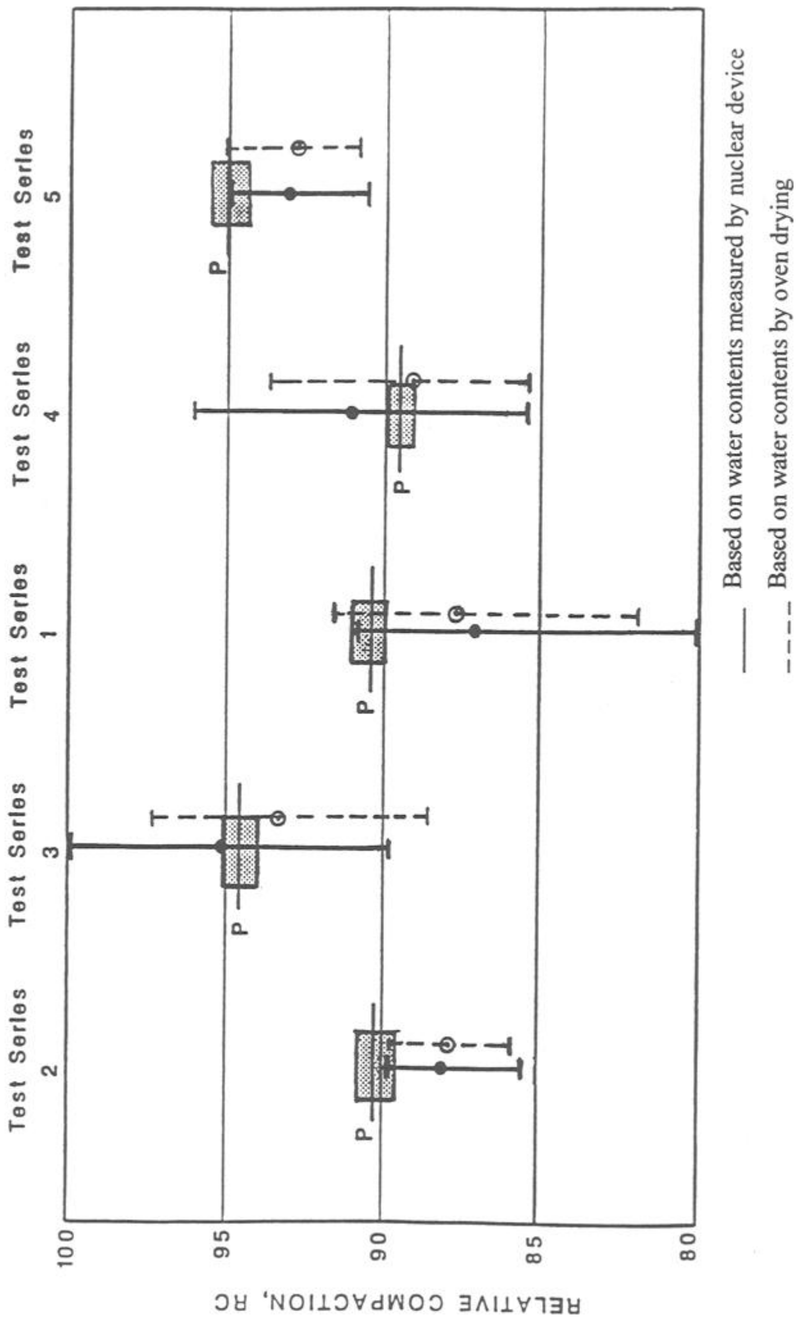

Figure 4 – *Summary of Data from Nuclear Tests*

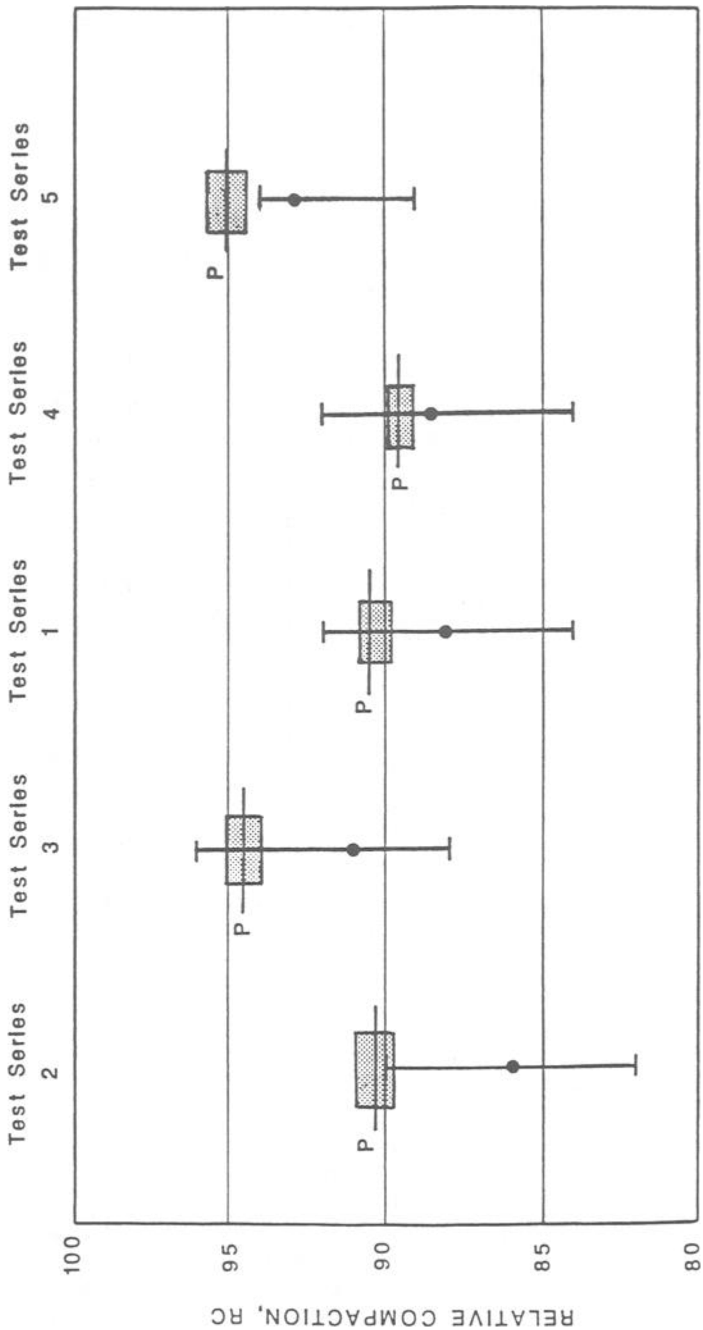

Figure 5 – Summary of Data from Drive-Cylinder Tests

However, when the water contents measured by nuclear device were ignored and dry densities were computed based on the placement water contents measured by oven drying, the standard deviation and range values decreased, indicating a higher degree of accuracy. These computed values are denoted as "Adjusted Nuclear" in Table 4, and they are also shown in Figure 4. However, it can be seen in Table 4 and Figure 4 that even when the water content variable was eliminated, the wide range in the RC did not decrease significantly.

- The drive-cylinder method (Figure 5) was less accurate than the sand-cone and the nuclear methods. The test had a bias in that most of the measured densities were lower than the actual placement densities, that is, the mean RC values (dots in Figure 6) for all test series were lower than the placement values: about 4% lower for soil compacted dry-of-optimum (Test Series 2) and 2% lower for soil compacted wet-of-optimum (Test Series 5). Aside from this bias, the range of measurements was about 8%, and the extreme measured values were as much as 8% lower than the placement value.
- Referring to Figures 3, 4, and 5, regardless of the type of density test, the ranges of measurements (vertical lines) were significantly higher than the placement ranges (bands marked with P). The differences are indicative of the test-related variability unless we suspect that the actual placement densities had some local variations more than the ranges indicated in Table 1. This is unlikely in view of all the precautions described in the section of Soil Placement and Compaction.

Summary and Conclusions

A comparative study of three types of field density tests was made based on data from many tests performed in a large soil bin. The soil used was a clayey sand (SC) with some gravel with a maximum size of ¾ in. (19 mm). In five series of tests at moisture-density combinations ranging from dry-of-optimum to wet-of-optimum and at relative compactions of 90% and 95% based on the D 1557 compaction test, it was possible to control water contents in the range of 0.4 to 0.7% and placement densities within ± 1% of the target value. The analysis of the density test results indicate that for the soil type used:

- The sand-cone method had the best accuracy; nevertheless, it measured relative compaction values that were as much as 5% off the placement value. The actual variability of data from this type of test under field conditions could be higher than those found in this study because of the lower degree of control on calibration of sand unit weight and other field conditions.
- The nuclear method had a wider range of variability and gave relative compaction values as much as 10% different from the placement value. A significant source of error was inaccurate water content readings by nuclear probe compared to direct measurement of water content by oven drying. It was also found that a nuclear-density device might have good repeatability, but at the same time lack accuracy because of improper equipment calibration and/or soil-related calibration. Thus, the standard procedure for calibrating the nuclear device with a density block does not

guarantee that the equipment will measure water content and density accurately. It is necessary that the nuclear device be calibrated for every type of soil at every site against direct measurements of soil density by sand-cone or similar method, and water content by oven drying. This study showed that when the nuclear density data are adjusted based on water contents directly measured by oven drying, more accurate results and a lower standard deviation and variability can be achieved.

- The 3-in. (76-mm) drive-cylinder method had a bias toward under-estimating the field density and relative compaction. The measured mean values in five test series were as much as 4% too low on the dry side of the optimum and 2% too low on the wet side of the optimum. Some measurements were as much as 8% lower than the placement value. The primary reasons for measuring low densities appeared to be the small size of the cylinder, and the adverse influence of gravel. The presence of gravel created voids along the wall of the cylinder during driving, and produced lower densities when gravel had to be dislodged from the sample ends during trimming. Results of the drive-cylinder tests confirm the ASTM D 2937 statement that in-place density measurements using such small drive cylinders should not be used as a basis of acceptance of compacted fills, particularly if the soil contains gravel.
- In this study, a limited number of tests were also made using a 3-in. (7.6-cm) California split-tube drive sampler with 2.4-in. (6-cm) ring liners. This sampler is thick-walled and does not meet the "area ratio" and the "clearance ratio" requirements stipulated in ASTM D 2937. When the sampler was driven by means of the Standard Penetration Test (SPT) hammer, it had a poor sample recovery ratio of 67%. The number of tests was too small for an adequate evaluation of the results. However, both in terms of size and sample disturbance, the quality of the samples taken with this type of drive sampler is lower than those taken by the 3-in. (7.6 cm) drive cylinder. Thus, the drive sampler would also be expected to provide unreliably low estimates of the in-place density.

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NRCS Experience with Field Density Test Methods Including the Sand-Cone, Nuclear Gage, Rubber Balloon, Drive-Cylinder, and Clod Test

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Reference: McCook, D. K., and Shanklin, D. W., "**NRCS Experience with Field Density Test Methods including the Sand-Cone, Nuclear Gage, Rubber Balloon, Drive-Cylinder, and Clod Test,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384*, D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Engineers of the Natural Resources Conservation Service (formerly the Soil Conservation Service), USDA, instituted a testing program to determine the reliability and reproducibility of several types of field density test methods. Tests were performed at several embankment construction sites in the Southeastern United States. Several different soil types were encountered in the embankment projects. On test pads constructed under normal field conditions, researchers measured the compacted dry density and water contents by the nuclear gage, sand-cone, drive-cylinder, and clod tests. The research compares the results of the various tests and discusses the conclusions of the researchers on reliability and accuracy of the various tests.

Conclusions of the studies include important aspects of each test that were found to explain variations and errors in the tests that were performed. The paper emphasizes errors common to the tests evaluated. Recommendations based on the studies of the NRCS are included. Advantages and disadvantages of methods are discussed. Data from the study sites is included to illustrate repeatability and comparisons of the various methods to each other. Data from the study sites are included to illustrate repeatability and comparisons of the various methods with each other.

Keywords: density, compaction, sand-cone, drive-cylinder, clod test, nuclear method, nuclear gage, field density tests, water content, earth fill

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Introduction

Natural Resources Conservation Service personnel conducted a testing program aimed at comparing results of earth fill density and water content measurements by several different tests over a two-year period. The program was conducted several years ago, and some standards have been revised since the testing was performed. A number of standard tests were used, which are summarized below.

1. ASTM Test Method for Density of Soil and Soil Aggregate in-place by Nuclear Methods (Shallow Depth) (D2922)
2. ASTM Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method (D1556)
3. ASTM Test Method for Density of Soil in Place by the Drive-Cylinder Method (D2937)
4. ASTM Test Method for Determination of Water (Moisture) Content of Soil by Direct Heating Method (D4959)
5. Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock (D2216)
6. Test Method for Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester Method (D4944)
7. Clod Test Method for Determination of Unit Weight of Soil. NRCS Method.

In a study at an Oklahoma embankment project, nuclear and sand-cone density test methods were compared. At an embankment site in Mississippi, drive-cylinder, sand-cone and nuclear methods were compared. Water contents were also measured by several methods, including nuclear, Speedy (carbide gas pressure meter), quick-dry and oven-dry methods. In a study at a Tennessee embankment, the sand-cone and nuclear gage methods were compared. In a study at a Georgia site, the sand-cone and clod test methods were compared with nuclear gage results. This report also summarizes results of tests run by construction personnel in Kansas and Indiana. The Kansas study compared the drive-cylinder and the nuclear gage methods; the Indiana data are for the rubber balloon method compared with the nuclear gage test. This report summarizes data from those investigations and includes conclusions and recommendations.

Oklahoma Data

Introduction

Nineteen companion sand-cone/nuclear density measurements were made at one site and seven comparisons made at another site in Oklahoma. A 4-inch diameter sand-cone was used for the Oklahoma testing, which was acceptable at the time the work was performed. Currently, NRCS uses only 6-inch diameter sand-cone devices to obtain the required volume of test hole. Every effort was made to conform to ASTM standard test procedures in effect at the time of the project.

A Troxler Model 3411B nuclear density gage was used. Readings were taken with a probe depth setting of 8 inches (in direct transmission), generally on the normal test interval (1-minute setting). Most nuclear measurements were made with a k value of 0. Later, some corrections were made mathematically based on k factors derived from oven-dry water content and nuclear water content measurements. It would have been preferable to obtain correction factors prior to the testing.

Site 1 Data, Oklahoma

Soil – Companion tests (sand-cone and nuclear) were performed in two types of soil. One series of eight tests was run in a sandy, very silty, moderately plastic CL soil. Eleven tests were run in a silty, very fine-grained nonplastic SM soil.

Test Methods – The nuclear gage was initialized prior to testing each day using the standard count procedure furnished by the manufacturer. In each test, a smooth, flat surface was prepared, and a probe hole made with the tools furnished with the nuclear gage. Next, nuclear water content, wet density, and dry density measurements were taken. Usually, three separate 1-minute readings were taken, and results averaged. At one location, twenty separate 1-minute readings were taken for a separate statistical examination. After removing the nuclear gage, a sand-cone test was performed immediately under the nuclear gage template. Water content samples were collected both from the material removed for the sand-cone determination (this soil was quick-dried) and from soil surrounding the sand-cone hole for oven-dry water content determinations. The water content samples for the direct heat tests were about 490 grams, while the oven-dry samples were about 100 grams.

At six locations in the clay fill and at four in the sandy fill, clod samples were collected in the immediate vicinity of the sand-cone tests and carefully preserved in airtight bags. Dry density/water content measurements were made later in the NRCS soil laboratory at Ft. Worth using the clod test method. The clod test used is not a standard ASTM test method. The procedure is very similar to one published by the U.S. Bureau of Reclamation (United States Bureau of Reclamation 1990). An ASTM Standard Test Method for this procedure is in development.

Test Results in Clay Fill – Table 1 summarizes results of the tests performed at eight locations on the CL fill material. Because these water content/density tests were taken from a relatively uniform area of compacted fill, variations should be relatively small between the eight test locations. To examine this, an average and a standard deviation were computed for each type of measurement.

Conclusions from Clay Fill Tests – Essentially the same results were obtained by the three methods of density measurement. The average nuclear test wet density is only 13 kg/m^3 greater than the sand-cone average and 25 kg/m^3 greater than the clod test average. The nuclear wet density values had a standard deviation that was about half of that of the sand-cone test. Standard deviations were less for the nuclear test than the other test methods for water content measurements. Water content samples 100 grams in size may not be large enough for accurate determination of k factors, in the opinion of the researchers. The nuclear gage measures the water content in a relatively large portion of soil, and larger oven-dry samples are necessary for good correlations.

TABLE 1 - *Comparison of Nuclear Gage, Sand-Cone, and Clod Tests, Clay Fill at Site 1, Oklahoma*

	Nuclear Data					Sand Cone Data					Clod Test Data		
Test No.	Wet Density kg/m3	Uncorr. w %	k factor	Corr. w %	Dry Density kg/m3	Wet Density kg/m3	Quick Dry w %	Oven Dry w %	Dry Density kg/m3	w% used	Wet Density kg/m3	Oven Dry w, %	Dry Density kg/m3
1	2,163	15.2	-8	14.3	1,893	2,174	14.2	14.4	1,904	direct *	2,170	13.3	1,915
			-18	13.2	1,911				1,901	oven **			
2	2,154	15.9	-8	15.0	1,874	2,134	15.9	14.5	1,842	direct *	2,181	13.3	1,925
			-18	13.8	1,892				1,864	oven **			
3	2,149	16.0	-8	15.1	1,867	2,150	15.5	14.8	1,862	direct *	2,122	15.6	1,835
			-18	13.9	1,887				1,873	oven **			
4	2,166	16.6	-8	15.7	1,872	2,136	16.6	15.1	1,832	direct *	2,077	15.2	1,803
			-18	14.5	1,892				1,856	oven **			
5	2,157	16.3	-8	15.4	1,869	2,174	16.2	14.7	1,871	direct *	2,104	15.0	1,830
			-18	14.2	1,889				1,896	oven **			
6	2,163	15.9	-8	15.0	1,882	2,155	16.6	15.9	1,848	direct *	2,138	15.9	1,844
			-18	13.8	1,901				1,860	oven **			
15	2,178	16.0	-8	15.1	1,879	2,115	15.8	14.1	1,827	direct *	NO TEST		
			-18	13.9	1,899				1,854	oven **			
16	2,128	16.6	-8	15.7	1,870	2,112	15.3	15.2	1,832	direct *	NO TEST		
			-18	14.5	1,889				1,833	oven **			
AVG	2,157	16.1	-8	15.2	1,873	2,144	15.8	14.8	1,852	direct *	2,132	14.8	1,857
			-18	14.0	1,895				1,868	oven **			
σ(n-1)	14.68	0.45	-8	0.46	8.5	24.0	0.8	0.6	26.1	direct *	39	1.1	49.5
			-18	0.43	8.2				22.3	oven **			

* - Direct w% ASTM Standard Test Method D4959 ** - Ovenw % ASTM Standard Test Method D2216

Supplemental Tests Clay Fill – To evaluate the inherent variability of nuclear measurements, twenty (20) 1-minute readings were taken at one location. The following variability was observed.

Wet Density (kg/m^3)			w,% (Uncorrected)		
Avg.	σ(n-1)	$\frac{\sigma}{\bar{x}}$ (%)	Avg.	σ(n-1)	$\frac{\sigma}{\bar{x}}$ (%)
2 163	4.16	0.19	16.3	0.36	2.23

Test Results in Sandy Fill – Table 2 summarizes results of eleven companion test locations in the sandy shell materials at Site 1. A water content correction factor, k, of -12 was used to obtain the corrected nuclear figures. Again, this correction factor was obtained after running oven dry water content determinations at each location, using rather small samples.

Conclusions Sandy Fill Tests – As in the clay fill tests, the nuclear and sand-cone methods obtained the same results, for practical purposes. The average nuclear method wet density was 20 kg/m^3 less than the sand-cone test wet density. The nuclear test standard deviation is 1/3 less than the sand-cone standard deviation. The standard deviation in the nuclear water content is about half of the deviation in the quick dry water content measurements.

Site 2 Data, Oklahoma

Soils – Seven companion nuclear sand-cone tests were made in a clayey, very sandy silt at this site. The soils were somewhat unusual in that they contained a high percentage of soluble salts. The NRCS Laboratory ran soluble salt determinations on samples from all 7 locations, and results showed soils had from 7% to 18.4% by dry weight soluble salts. The chemical composition of these salts was not determined.

Test Methods – The test methods used were essentially the same as those used at Site 1 discussed previously. Large changes in relative humidity during the day made it difficult to calibrate the sand. The sand was calibrated several times during the day when problems were noted.

Test Results – Table 3 summarizes the test results. The sand-cone and nuclear methods measured wet densities of about the same values (about 41 kg/m^3 difference between the average values. Again, if we assume the area of earth fill tested was uniform, the variance in measurements should reflect the repeatability or accuracy of the method used. The standard deviation of the nuclear wet density tests was about half of that of the sand-cone tests.

The unusual chemical or mineralogical composition of the soils on this site may have caused errors in the both nuclear wet density measurements and water content measurements. More data would be needed to determine if one should apply a correction factor to the wet density measurements in unusual soils such as those at this site. The small differences between the sand-cone and nuclear measurements could be accounted for solely by experimental errors, and no correction to the nuclear method wet density measurements was considered necessary by researchers.

TABLE 2 - *Comparison of Nuclear Gage, Sand-Cone, and Clod Tests, Sandy Fill, Site 1 Oklahoma*

Test No.	Nuclear Data					Sand Cone Data			Clod Test Data		
	Wet Density kg/m3	Uncorr. w %	Assumed k factor	Corr. w %	Dry Density kg/m3	Wet Density kg/m3	Direct Heat * w %	Dry Density kg/m3	Wet Density kg/m3	Oven Dry ** w %	Dry Density kg/m3
7	2,045	10.2	-12	8.8	1,879	2,106	9.8	1,918	2,099	10.5	1,900
8	2,086	10.6	-12	9.2	1,911	2,114	10.2	1,918	2,032	10.2	1,844
9	2,053	9.9	-12	8.6	1,890	2,064	8.7	1,899	1,942	9.6	1,772
10	2,099	10.2	-12	8.9	1,928	2,082	8.2	1,924	2,066	10.6	1,868
11	2,106	10.6	-12	9.3	1,926	2,078	5.7	1,966			
12	2,072	10.1	-12	8.8	1,904	2,107	9.6	1,923			
13	2,120	11.1	-12	9.7	1,933	2,098	9.2	1,921			
14	2,131	9.6	-12	8.3	1,968	2,226	8.5	2,051			
17	2,138	9.8	-12	8.5	1,970	2,155	8.9	1,979			
18	2,120	11.3	-12	10.0	1,927	2,158	10.3	1,957			
19	2,056	8.6	-12	7.3	1,916	2,058	8.4	1,898			
AVG	2,093	10.2	-12	8.9	1,923	2,113	8.9	1,941	2,035	10.2	1,846
σ(n-1)	33.0	0.74	0.00	0.7	28.1	49.50	1.3	45.0	67.4	0.5	54.1

* - Direct water content is according to ASTM Standard Test Method D4959

** - Oven water content is according to ASTM Standard Test Method D2216

TABLE 3 - *Comparison of Nuclear Gage and Sand-Cone Tests, Clayey, Very Sandy Silt Fill, Site 2, Oklahoma*

Test No.	Nuclear Data					Sand Cone Data		
	Wet Density kg/m3	Uncorr. water %	Assumed k factor	Corrected water %	Dry Density kg/m3	Wet Density kg/m3	Oven Dry * water %	Dry Density kg/m3
29	2043	20.5	-16	18.50	1724	2083	16.9	1,782
30	2034	19.2	-16	17.30	1734	2050	16.5	1,759
31	2030	20.7	-16	18.80	1709	2098	21.2	1,731
32	2027	20.8	-16	18.90	1705	2046	19.4	1,714
33	2035	20.3	-16	18.40	1719	2094	17.8	1,778
34	2061	20.6	-16	18.70	1736	2085	18.9	1,753
35	2030	21.8	-16	19.90	1693	2088	19.6	1,746
AVG	2037	20.6	-16	18.64	1717	2078	18.6	1752
σ(n-1)	11.6	0.77	0.	0.77	15.6	20.9	1.65	24.4

* - Oven water content is according to ASTM Standard Test Method D2216

Conclusions from Sandy Fill Tests – Problems with calibration of the Ottawa sand due to high humidity and the small size of the sample used for the 4-inch diameter sand-cone test probably contributed to its greater variability compared to the nuclear test method. Except for one measurement, the clod test obtained very similar dry unit weight measurements as the other tests.

Mississippi Data

Introduction – Twenty-three companion nuclear, sand-cone, and drive-cylinder measurements were made on an earth fill at Big Creek, Site 4. Water contents were measured with the nuclear gage, a Speedy (carbide gas pressure meter) water content, and oven dry procedures. The nuclear gage was a Troxler model 3411B. Standard count procedures were performed at the start of testing. A probe depth of 6-inch in direct transmission was used for all measurements. Mississippi personnel prior to this testing had determined a water content correction factor, but results were quite variable, and it is likely that somewhat different soils were encountered in this subsequent testing. Both normal speed and slow speed settings were used.

A 6-inch diameter sand-cone was used and the Ottawa sand was calibrated prior to testing. The drive-cylinder used was a 5-1/8 inch diameter apparatus that was locally manufactured (not commercially available readily). This is the normal density test method used by Mississippi Construction personnel; a drawing of the apparatus is in the *NRCS National Engineering Handbook*, Chapter 19. Two Speedy (carbide gas pressure) water content meters that had been calibrated previously were used. A standard drying oven was used for the oven-dry tests. Some oven-dry samples were 1,000 grams in size and some 500 grams. The larger samples inhibited complete drying in a reasonable time due to over-loading the small oven available.

Several sand-cone tests were disregarded because results were obviously in error due to missed weights, poor sand behavior during high humidity, etc. Intermittent showers created very adverse conditions on one of the days. The large number of tests run and many different personnel involved also contributed to errors.

Big Creek, Site 4 Data

Soils – The first ten test locations were in a relatively homogeneous portion of the fill. The soil tested was a clayey, silty, fine sand probably classifying as an SC. In the other thirteen test locations, a layer of moderately plastic to plastic, silty, sandy clay classifying as a CH soil occurred over the SC material. Sand-cone and drive-cylinder specimens contained varying amounts of each material. The portion of each soil type selected for the water content determination greatly influenced the resulting measurements, especially when small samples were taken using the Speedy (carbide gas pressure meter) water content meter.

Test Methods – An area was smoothed for the nuclear gage template, the nuclear probe was set at 6 inches, and either three 1-minute readings (normal speed) or one 4-minute reading (slow speed setting) were made. At a few locations, measurements were made in the same probe hole with another nuclear gage of the same type. All readings were taken with a k value of zero setting and corrected later mathematically. Sand-cone and drive-cylinder tests were made immediately under the nuclear gage location. Speedy (carbide gas pressure meter) water content tests were made on samples from the drive-cylinders. Oven-dry water contents were measured on the sand-cone samples and several of the drive-cylinder samples as well.

Test Results – Tables 4 and 5 summarize results of the test measurements. Note that several sand-cone tests are dashed because test results were obviously erroneous. The sand-cone dry densities were calculated using oven-dry water contents, while the drive-cylinder dry densities are given for both Speedy (carbide gas pressure meter) water content and oven-dry water content determinations. Nuclear water contents were corrected with a k value of -25.

Analysis of Test Results – The three tests measured about the same average wet density for each area of the fill tested. The standard deviation of the nuclear wet density measurements was about half of the standard deviation of the sand-cone wet density measurements, but only slightly less than the standard deviation of the drive-cylinder measurements, for the first ten tests. The Speedy (carbide gas pressure meter) water content test had a larger standard deviation than the oven-dry or nuclear methods of water content measurement.

In the test data on the layered fill, (second thirteen tests shown in Table 5), again, all three methods measured about the same wet densities. The standard deviation was higher for all three methods in this set of data, undoubtedly due to the variability of the soil being tested. The slightly lower standard deviation in the nuclear water content measurements compared to the other tests is attributed to the larger size of soil sample being measured in this method.

Conclusions - Based on this data, the nuclear gage obtains, for practical purposes, the same results as the sand-cone method and drive-cylinder method. Based on this data, and the experience of the group performing the tests, the drive-cylinder method was more reliable than the sand-cone method at this site. Of twenty-three sand-cone tests performed, nine tests were discarded. Results were ignored because the tests contained obvious errors or poor field conditions prevented completing the tests. Test results in the layered fill were difficult to interpret.

Special Testing – Several "side-purpose" tests were run in the Mississippi investigation which are of some interest. To investigate the influence of the size of sample selected for water content determination, the following procedure was performed. A Nuclear gage water content was measured on a uniform area of the fill in the SC soil in which the first ten groups of measurements were made. Then, a large volume underneath the nuclear template was sampled for oven-dry water content measurement. Twelve samples, each about 100 grams (moist), were taken from the large sample. A corrected nuclear water content of 15.44% was obtained.

TABLE 4 - *Comparison of Nuclear Gage, Sand-Cone, and Drive-Cylinder Tests, Uniform SC Fill, Big Creek, Site 4, Mississippi*

	Nuclear Data					Sand Cone Data			Calibrated Cylinder Tests		
Test No.	Wet Density kg/m3	Uncorr. w %	Assumed k factor	Corrected w %	Dry Density kg/m3	Wet Density kg/m3	Oven Dry * w %	Dry Density kg/m3	Wet Density kg/m3	Carbide Meter ** w %	Dry Density kg/m3
1	2,170	18.3	-25	15.3	1,882	2,170	14.3	1,899	2,139	14.8	1,864
2	2,180	17.2	-25	14.3	1,907		14.0		2,178	14.0	1,910
3	2,150	16.8	-25	13.9	1,887	2073.6	12.3	1,846	2,182	14.4	1,909
4	2,138	17.0	-25	14.1	1,873	2142.4	14.7	1,868	2,126	18.6	1,793
5	2,120	19.8	-25	16.8	1,815	2080	14.5	1,817	2,123	19.0	1,785
6	2,168	17.1	-25	14.2	1,898	2163	14.3	1,893	2,142	17.7	1,821
7	2,107	14.7	-25	11.8	1,885		11.0		2,104	17.2	1,795
8	2,114	16.0	-25	13.1	1,869		12.6		2,150	18.6	1,813
9	2,130	14.6	-25	11.7	1,907		11.5		2,104	14.0	1,846
10	2,174	16.4	-25	13.5	1,916	2139	15.0	1,860	2,194	13.5	1,933
AVG	2,145	16.8	-25	13.9	1,884	2128	13.4	1,864	2,144	16.2	1,847
σ(n-1)	27.0	1.55	0	1.52	28.6	45.8	1.44	33.9	31	2.14	57.8

* - Oven water content is according to ASTM Standard Test Method D2216

** - Carbide meter water content is according to ASTM Standard Test Method D4944

TABLE 5 - *Comparison of Nuclear Gage, Sand-Cone, and Drive-Cylinder Tests, Layered SC/CH Fill, Big Creek, Site 4, Mississippi*

	Nuclear Data					Sand Cone Data			Calibrated Cylinder Tests		
Test No.	Wet Density kg/m3	Uncorr. water %	Assumed k factor	Corrected water %	Dry Density kg/m3	Wet Density kg/m3	Oven Dry * water %	Dry Density kg/m3	Wet Density kg/m3	Carbide Meter ** water %	Dry Density kg/m3
11	2,022	23.6	-25	20.5	1,701	1,957	22.4	1,599	2,026	22.1	1,659
12	2,042	22.1	-25	19.1	1,714	2,077	19.3	1,741	2,032	21.2	1,677
13	1,982	28.8	-25	25.6	1,578	2,038	29.9	1,569	1,944	24.1	1,566
14	2,058	23.8	-25	20.7	1,705		20.1			20.4	
15	1,981	25.3	-25	22.2	1,621	2,008	22.0	1,646	2,058	24.9	1,648
16	2,029	20.4	-25	17.4	1,728	1,899	18.2	1,607	2,080	20.1	1,733
17	2,010	21.9	-25	18.9	1,690	2,144	17.3	1,828	2,053	16.9	1,756
18	2,054	24.3	-25	21.2	1,695		21.6			17.0	
19	1,974	24.9	-25	21.8	1,621	1,982	24.5	1,592	2,069	25.2	1,653
20	2,035	23.2	-25	20.1	1,695	2,061	20.3	1,713	2,053	20.0	1,711
21	2,104	18.3	-25	15.3	1,825	2,094	15.8	1,809	2,062	21.0	1,705
22	2,037	22.4	-25	19.4	1,706		20.7			23.3	
23	1,987	24.7	-25	21.6	1,634	2,042	23.9	1,648	2,064	23.8	1,668
AVG	2,024	23.4	-25	20.3	1,686	2,030	21.2	1,675	2,044	21.5	1,678
σ(n-1)	37.3	2.54	0	2.49	61.7	71.3	3.60	92.7	39	1.61	53.0

* - Oven water content is according to ASTM Standard Test Method D2216

** - Carbide meter water content is according to ASTM Standard Test Method D4944

Oven-dry water contents ranged from 13.36 to 17.92% (4.6% spread) with an average of 15.32% and a standard deviation of 1.29%. Previous tests in Oklahoma indicate individual nuclear measurements of water content on normal speed have a standard deviation of about 0.36%. It may be inferred that the nuclear readings have less variance than that caused by the selection of small size samples for oven-dry water content determinations.

The differences in two separate nuclear gages' measurement of wet density and water content were also investigated on this site. At several test holes, separate readings of wet density and water content were made with two meters, both Troxler model 3411B. The same probe hole was used for both readings. Eleven test locations were measured. Table 6 summarizes test results. The difference in wet density measurement between the two meters averaged 21.8 kg/m^3. The measured water contents had an average difference of 0.6%. These differences are slightly higher than the standard deviations where repetitive measurements were made with one gage, based on the Oklahoma data. In the Oklahoma data, a standard deviation of 4.16 kg/m^3 was observed in repetitive measurements (of wet density) using one gage. A standard deviation in water content of 0.36% was observed in single gage repetitive measurements in Oklahoma.

Tennessee Data

Introduction – Two sets of data were gathered at Mud Creek, Site 7, in Tennessee. Because all of the comparison tests were performed in natural foundation soil, the uniformity of the measurements was poor. The data are not completely reproduced in this paper. Only the general observations of the researchers and a brief summary of some data are included. Comparisons were primarily between the nuclear gage and a 6-inch sand-cone. A Troxler model 3411B nuclear gage was used in direct transmission mode testing.

Calibrating the sand used for the sand-cone test was difficult on this site. Even with highly experienced personnel, repeated measurements didn't obtain consistent enough results for reliability. After trial-and-error, removal of the minus #40 fraction of the density sand produced more consistent data.

Soils – Soils were nonplastic to slightly plastic ML or CL-ML classifications. The soils have about 70% silt with less than 10% sand.

Test Methods – Sand-cone tests were performed under the nuclear template outline. Nuclear readings were all on slow setting (4-minute). Water content correction factors (k factors) were obtained "after the fact" because preliminary data were not obtained. A probe setting of 8 inches was used for the nuclear test. A 6-inch diameter sand-cone apparatus was used. Water contents were determined using both direct heat and oven-dry methods. Oven-dry samples were 500 grams, and direct heat samples were 600 grams or larger. Where both sand-cone and drive-cylinder tests were made, they were made as close together as possible under the nuclear template.

TABLE 6 - *Comparison of Two Nuclear Gages, Big Creek, Site 4, Mississippi*

Test No.	Wet Density, kg/m3		Value Difference	Difference %	% Moisture		Value Difference	Difference %
	Meter A	Meter B			Meter A	Meter B		
111	2,162	2,168	6.4	0.3%	16.8	17.3	0.5	2.9%
112	2,163	2,141	22.4	1.0%	15.5	16.2	0.7	4.4%
12	2,042	2,027	14.4	0.7%	22.1	22.5	0.4	1.8%
13	1,982	1,957	25.6	1.3%	28.8	29.4	0.6	2.1%
14	2,058	2,024	33.6	1.6%	23.8	23.6	0.2	0.8%
115	1,981	1,997	16.0	0.8%	25.3	25	0.3	1.2%
116	2,101	2,099	1.6	0.1%	18.4	18.6	0.2	1.1%
19	1,974	1,968	6.4	0.3%	24.9	24.4	0.5	2.0%
20	2,037	1,997	40.0	2.0%	22.7	23.3	0.6	2.6%
21	2,104	2,074	30.4	1.5%	18.3	18.9	0.6	3.2%
22	2,037	1,998	38.4	1.9%	22.1	23.7	1.6	7.0%
23	1,987	1,960	27.2	1.4%	24.7	25.5	0.8	3.2%
24	1,982	1,962	20.8	1.1%	23.3	24.6	1.3	5.4%
Average			21.8	1.1%			0.6	2.9%
Std. Dev.			12.4	0.6%			0.4	1.8%

Test Results – In the first several sets of comparisons, the nuclear reading consistently obtained values of dry density that were 30 to 70 kg/m^3 higher than the sand-cone measured values. In a second series of tests, the nuclear gage values of dry density were very near the sand-cone test values. This difference in results was attributed solely to the screening out of fine sand fraction from the density sand prior to calibration. The more reliable calibration improved the correlation very significantly.

The corrected nuclear water and oven-dry water contents compared well. The direct heat method consistently obtained water contents about 1% higher than the oven-dry method, as would be expected, because the soils tested had some organic matter in them.

Conclusions – Very careful procedures are necessary to obtain reliable sand-cone test results. Calibrating density sand that contains a significant amount of minus #40 sieve particles proved to be impossible on this project. After screening out the minus #40 particles, very repeatable results were obtained.

Georgia Data

Soils – Six comparisons were made in micaceous SM, SC, CL, and ML classification soils in an earth fill project. The majority of the samples classify as SC. The mica content was quite high by visual observation.

Test Methods – A Campbell-Pacific model MC-2 nuclear gage was used in these tests. The sand-cone method and the clod test methods were used for comparisons to the nuclear method in this study. Tests were performed in the same general manner as on previous sites. Sand-cone tests were performed under the nuclear template outline using a 6-inch sand-cone. Both direct heat and oven-dry water contents were made for the sand-cone tests. Samples were 500 grams or larger. Water content correction factors for the nuclear gage were based on the oven-dry water content.

The Campbell Pacific gage uses a different methodology for correcting for water content than the Troxler gage and a k value as such is not used on the Campbell-Pacific gage. For comparison purposes, we calculated k values for each test as if a Troxler gage were being used. Nuclear readings were all taken with no water content correction dialed into the machine. A probe depth of 6 inches in direct transmission and a slow (4-minute) reading time were used in the nuclear measurements.

Small clod samples were obtained at each location for a dry unit weight and water content determination at the Soil Mechanics Laboratory. The sand-cone testing on this site was the most consistent of the jobs tested due to repeatable calibrations of the density sand. One factor thought to contribute to the good performance of the sand was the storage in a closed canister and the screening out of fines before each use.

Test Results – A complete summary of test results is shown in Table 7. The nuclear and sand-cone tests measured similar results. The nuclear gage test had an average wet density that was about 21 kg/m^3 less than that of the sand-cone test. However, the nuclear wet density measurements had a standard deviation almost half of the standard deviation of the sand-cone results. The clod test densities were considerably lower than the other two measurements; this is attributed to rebound of the micaceous materials, because the clods were not confined after obtaining them.

TABLE 7 - *Comparison of Nuclear Gage, Sand-Cone, and Drive-Cylinder Tests, Big Cedar, Site 9, Georgia*

Test No.	Nuclear Data					Sand Cone Data			Clod Tests		
	Wet Density kg/m3	Uncorr. w %	Assumed k factor	Corrected w %	Dry Density kg/m3	Wet Density kg/m3	Oven Dry * w %	Dry Density kg/m3	Wet Density kg/m3	Oven Dry * w %	Dry Density kg/m3
1	2070	25.6	-56	18.6	1746	2,048	18.4	1,730	2,024	17.6	1,721
2	2096	28.5	-56	21.3	1728	2,086	16.4	1,792	2,019	18.6	1,703
3	2102	22.8	-56	15.9	1814	2,127	16.2	1,831	1,989	16.0	1,714
4	2109	22.3	-56	15.5	1827	2,175	17.0	1,859	2,024	16.5	1,737
5	2056	24.7	-56	17.7	1747	2,105	16.0	1,814	1,933	15.6	1,672
6	2056	19.3	-56	12.6	1826	2,078	14.3	1,818	1,992	14.6	1,738
AVG	2082	23.9	-56	16.9	1781	2,103	16.4	1,807	1,997	16.5	1,714
σ(n-1)	23.7	3.16	0	2.99	45.7	44.0	1.34	43.9	35.1	1.43	24.8

* - Oven water content is according to ASTM Standard Test Method D2216

A larger k factor for water content correction on the nuclear gage was used than on the other soil types that were previously tested in other states. A value for k of – 56 was used on this site based on oven dry measurements. At other sites, values of k varied from – 8 to – 25. No positive k factors were calculated in any of the soils studied. At only a few sites have NRCS engineers encountered unusual soils with a positive k factor.

Conclusions – The excellent results obtained in the sand cone testing at this site was attributed by researchers to keeping the sand in closed containers and carefully screening the sand before use. The high k factor at the Georgia site is attributed to the mineral composition in the highly micaceous soils. The clod test measured significantly lower average dry unit weights than the other test methods. This could be attributable to rebound of the micaceous soils upon sampling.

Kansas Data

Soils – Most of the soils tested were silty clays classifying as CL soils. Most of the soils had very little sand or gravel, so the drive-cylinder was an appropriate testing device.

Test Methods – Kansas's construction personnel obtained data with the drive-cylinder and nuclear gages. These tests were made in the course of normal density testing associated with the construction of several earthen embankments. The data are for six different Troxler 3411B gages. Sixty-four comparisons were made at seven different sites. The drive-cylinder used for the comparative testing was a 3-½ inch diameter, 4-inch long cylinder. This met the ASTM Standard Test Method for Density of Soil in Place by the Drive Cylinder (D 2937) in effect at that time. A larger drive-cylinder is now required. The nuclear probe depth was 6 inches. Most nuclear water contents were measured with a k factor entered into the machine. Water content samples for the drive-cylinder were about 100 grams in size. Water contents were determined by the oven-dry method. The normal (1-minute) reading time was used for the nuclear gage. Drive-cylinder tests were made under the nuclear template between the probe and sensor.

Test Results – The complete data set is not repeated here. The primary interest of this study is the differences in the measurements of dry density between the two methods compared. The largest discrepancy noted was 152 kg/m^3; in 27 of the 64 tests the difference measured was 50 kg/m^3 or more. Of 27 tests where the differences were greater, only 3 tests had a nuclear measurement that was higher than the drive-cylinder measurement of dry density. Many of the larger discrepancies occur on lower density clays that would be expected to compress more. Figure 1 illustrates the distribution of differences in the two measurements.

The drive-cylinder measurement of dry density is higher than the nuclear gage in 48 of the 64 tests. Generally, NRCS researchers noted the same trend for other sites where this comparison was made. Larger diameter drive cylinders would probably not have caused as much disturbance of the sample, in the opinion of NRCS researchers.

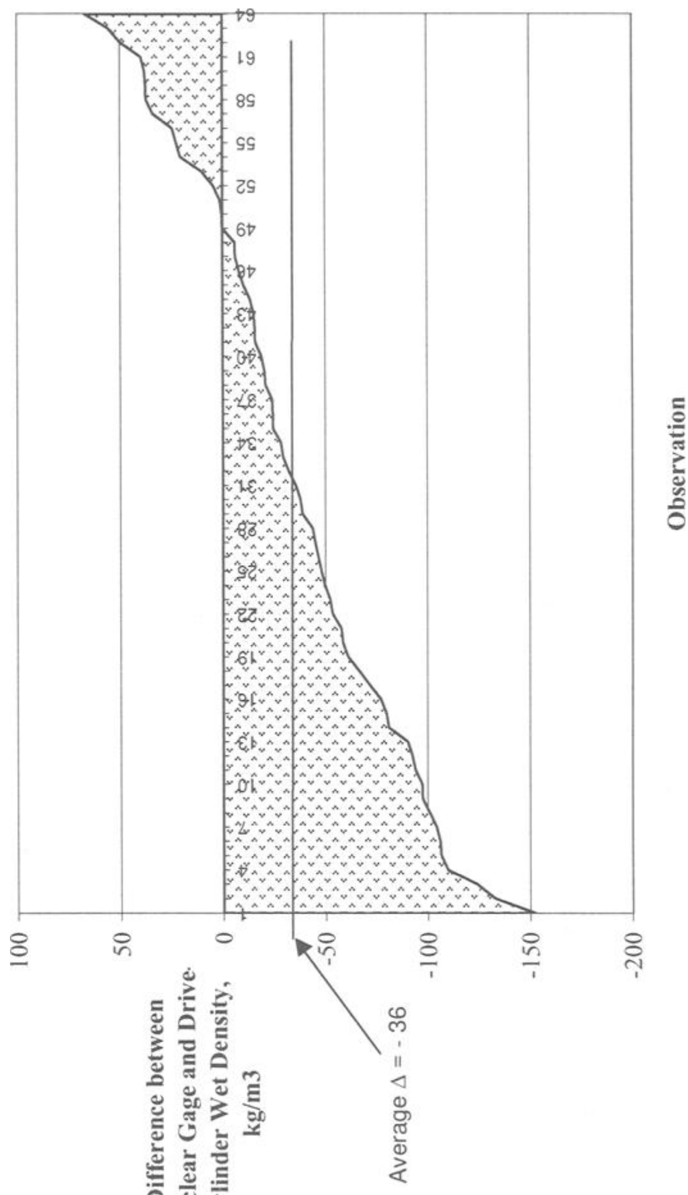

Figure 1 - *Kansas Comparisons between Nuclear Gage and Drive-Cylinder Wet Density Measurements*

The data did not always clearly indicate whether a k factor was used in the nuclear water content measurements for all of the data. Reporting forms should always include this information. The largest difference between the nuclear and oven-dry water content measurement was 5.7% water content. On only 20 of 64 tests were the nuclear water contents less than the oven dry water contents. Where correction factors were definitely employed, differences between nuclear water contents and oven-dry were less than 2.5% water content. The water content data shows the importance of using a water content correction factor in measuring by the nuclear gage. Overall, k factors were not as high as expected for similar clay soils.

Conclusions – In 42 of the 64 comparisons, the difference between the nuclear and the drive-cylinder measurement of wet density was less than 48 kg/m^3. The nuclear method measured an average wet density 36 kg/m^3 less than the drive-cylinder method. For practical field purposes, this is excellent agreement, about the same as comparative measurement differences by other methods. Where used carefully, with properly determined water content correction factors, the nuclear gage measures similar water content and wet density values as the drive-cylinder and oven-dry methods. Selection of sample location probably has a larger influence on the test result than does the method of measurement.

Indiana Data

Soils – Soils are primarily sandy clays classifying as CL. Most were only slightly-to-moderately plastic.

Test Methods – Indiana construction personnel furnished data on comparative density and water content tests made with the nuclear method and the rubber balloon method. Test data were gathered in the normal testing program at one flood retarding embankment and an upstream compacted blanket project. Thirty-three companion measurements were made over a period of about 3 years. The nuclear gage used was a Troxler 3411. A 4-inch probe depth in direct transmission and a normal (1-minute) reading time were employed. The rubber balloon test was performed immediately under the nuclear template. The rubber balloon was a smaller device commonly used at that time. This device commonly results in a hole volume of less than 700 cubic centimeters (0.025 cubic foot).

Water contents were determined by the oven-dry method using 100-gram size samples. All nuclear water content readings were determined with a water content correction dialed into the machine.

Test Results – The complete data set is not repeated here. The primary interest of this study is the differences in the measurements between the two methods. The largest difference in measurement of the wet density between the two methods is 102 kg/m^3; in 25 of 33 tests the difference measured was 50 kg/m^3 or less. Figure 2 illustrates the distribution of differences in the two measurements. The rubber balloon measurement of wet density is lower than the nuclear gage in only 7 of the 33 tests. The nuclear method measured an average wet density 10 kg/m^3 greater than the rubber balloon.

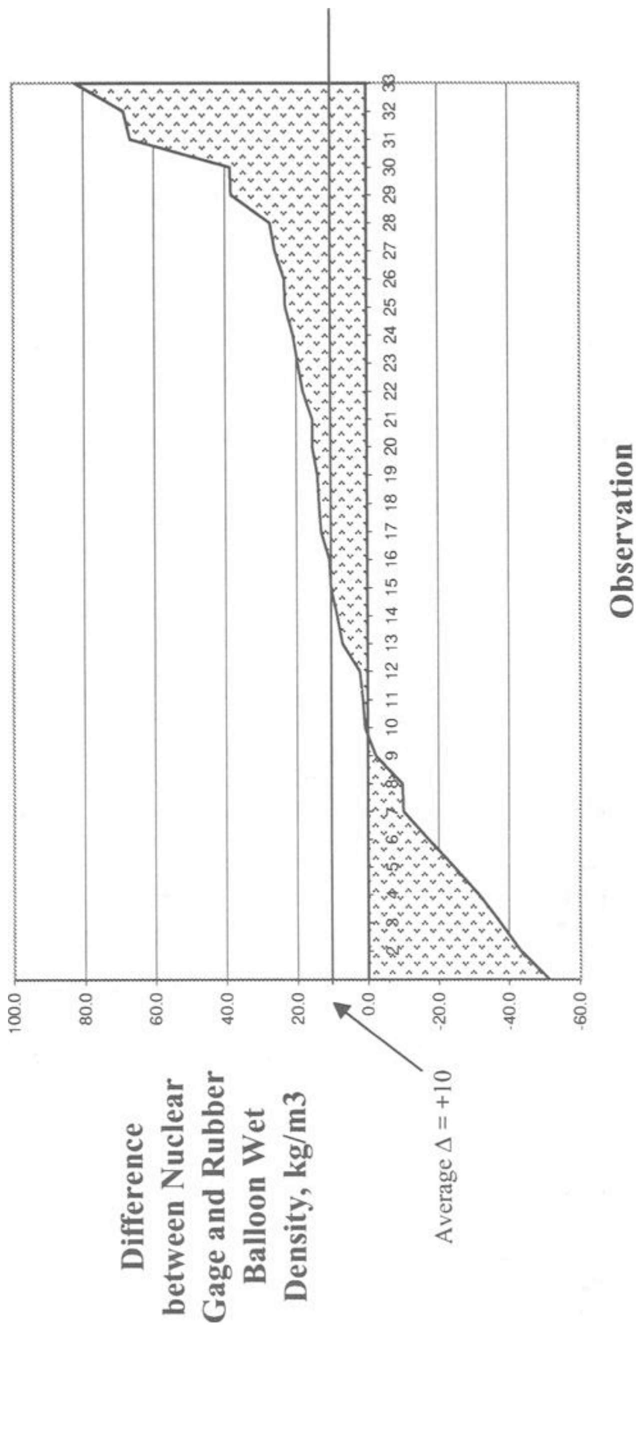

Figure 2 - *Indiana Study - Difference Between Nuclear Gage and Rubber Balloon Measurements*

Because the rubber balloon apparatus can be lifted with excess pressure, and some compression of the soil by the pressure of the balloon can occur, the measured volume could be too large. The result would be that the measured density would be lower than a more accurate measurement. The largest difference between the nuclear and the oven-dry water contents is 6.3%. The nuclear water content averaged 1.4% higher than the oven-dry water content for the 33 samples. This data indicates that similarly to the Kansas study, k factors were used that were not high enough. Only for 6 of 33 measurements were the nuclear water contents less than the oven-dry water contents.

Conclusions – For practical purposes, the nuclear gage obtains the same measurement of density as the rubber balloon, provided other tests are carefully run, and water content corrections are properly obtained for use with the nuclear gage.

General Conclusions

1. For practical field purposes, the nuclear method (direct transmission) for measuring wet density and water contents is equivalent to other methods of wet density measurement, when all procedures all carefully performed.
2. Calibration and measurement technique associated errors are generally less with the nuclear method than with the sand-cone method.
3. The standard deviation of results between different test locations for a single method are similar to standard deviations in results between methods at a single location on a uniform fill.
4. The larger volume of soil tested with the nuclear method compared to other conventional methods provides a more representative sampling of a soil mass. On heterogeneous soils, the nuclear gage obtains an average density reading that is likely to be more accurate than smaller samples representative of other test methods.
5. The primary variable associated with nuclear measurements is the water content correction factor appropriate for each soil type. The correction is dependent on the soil chemistry, which is mostly related to the clay fraction.
6. The sand-cone test was found to have more intrinsic variability than the nuclear gage or the drive-cylinder methods for wet density measurement.
7. The nuclear method of measurement for both density and water content is much faster than other methods. At least six nuclear measurements may be made in the time that one of the other methods of measurement can be made in the experience of the NRCS researchers.
8. Using the slow (4-minute) speed setting on the gage, compared to the normal (1-minute) setting improved the accuracy of nuclear measurements in the opinion of the researchers. However, the small improvement in accuracy is probably not justified for routine field compliance testing.
9. Results show little reason to prefer the 6-inch or 8-inch probe depth. Either appeared to work satisfactorily.

10. The drive-cylinder method of density measurement has fewer sources of error inherent in it than the sand-cone method. More consistent results are obtained. However, compression of the sample during sampling may occur and the measured wet densities may be erroneously high on some soils. Using a 4-inch or larger diameter sampler is essential. The 5-1/8-inch sampler used in Mississippi produced good results.
11. Calibration of sand used in the sand-cone method is a very critical aspect of the test affected by humidity and fines in the density sand. Calibration is often not easily repeatable within tolerable limits.
12. The clod test is a useful field density test that can be performed quickly and accurately. Results are comparable to other methods.
13. The rubber balloon apparatus may cause compression of softer soils, resulting in incorrectly low density measurements.

General Recommendations

1. Before using a nuclear gage on a site for density testing, water content correction factors should be obtained for each soil type anticipated, using at least 500-gram-size oven water content samples.
2. After performing nuclear measurements, the area tested should be exposed (with a spade) to inspect material tested and verify the water content correction factor to be used.
3. The nuclear data may be regarded with a high degree of confidence if ASTM standard test procedures are followed.
4. The drive-cylinder, rubber balloon, or clod methods are recommended when correlating nuclear results if possible. Gravelly soils may not permit use of some methods. The sand-cone test provided the least consistent results of the methods studied. Errors are more likely because the sand-cone test has greater numbers of steps to follow than the other methods, in the researchers' opinion.
5. Sand-cone tests require extreme care in calibrating the sand to obtain consistent results. Humidity and fines in the sand are special problems.
6. Because more tests can be performed in a given time with the nuclear gage, it has a higher likelihood of obtaining statistically valid results. A primary reason for preferring this testing method is its speed.

Reference

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Compactive Effort Applied by Hand-Operated Compactors

Reference: Filz, G. M., Duncan, J. M., and Brandon, T. L., **"Compactive Effort Applied by Hand-Operated Compactors,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: The rate of energy transfer between compactor and compacted soil is directly related to compaction effectiveness. In this study, a vibrating plate compactor and a rammer compactor were instrumented to determine contact forces and energy transfer rates. The compactor energy measurements and the corresponding unit weight measurements of the compacted soil were in good agreement with the results of conventional laboratory compaction tests. Consequently, it is possible to determine the amount of operating time required for these compactors to impart a particular compactive effort to a lift of soil of known volume. It was found that a Wacker BPU2440A vibrating plate compactor imparts the standard Proctor (ASTM D 698) compactive effort to one cubic meter of soil in about 18 minutes and a Wacker BS60Y rammer compactor imparts the standard Proctor compactive effort to one cubic meter of soil in about 12½ minutes. Such knowledge may be useful in contractors' estimates of construction costs and, as an adjunct to unit weight measurements, in quality control and quality assurance programs.

Keywords: compactive effort, compaction energy, compaction control, compaction equipment, rammer compactor, vibrating plate compactor

Introduction

Compaction is used to improve the strength, compressibility, and hydraulic conductivity characteristics of soil. When performance specifications are employed on construction projects, compaction control for fine-grained soils is achieved by requiring that the fill be compacted to a dry unit weight that equals or exceeds a certain percentage of the maximum dry unit weight measured in laboratory compaction tests. The laboratory tests employ a specified compactive effort, which is the amount of energy applied per unit volume of soil. In the field, the construction contractor is responsible for controlling the soil water content and applying the compactive effort necessary to achieve the required

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dry unit weight. Verification that the specifications have been satisfied is based on field measurements of the dry unit weight of the compacted soil, without consideration of the amount of energy applied by the contractor's compaction equipment.

Random and systematic errors in field and laboratory testing can occur, creating uncertainty and controversy during construction. Further, because the compaction characteristics of soils typically vary from point to point in a fill, laboratory results are sometimes applied incorrectly to field measurements of soils with different characteristics.

If the amount of compactive effort applied by the contractor's compaction equipment were known, this would provide another piece of information that would be useful in construction quality control and quality assurance programs. Knowing the compactive effort and the compaction water content, an independent assessment of the relative compaction of the fill could be made as an adjunct to comparisons of field and laboratory measurements of dry unit weight.

Values of energy transfer rates for a variety of compaction equipment would be useful, but such information is not generally available. This paper provides energy transfer rates for two hand-operated compactors: a 62 kg (137 lbf) rammer compactor and a 125 kg (275 lbf) vibrating plate compactor. Measurements of the dynamic contact force and energy transfer rate for these compactors were made during a series of instrumented retaining wall tests whose primary purpose was to study the effect that compaction has on lateral earth pressures. The force and energy measurements were made while operating each compactor on moist silty sand and on dry fine sand.

This paper provides a review of previous compactor force and energy measurements and descriptions of the equipment, materials, and procedures used in this testing program. The resulting force and energy measurements provide a basis for comparing the effectiveness of two hand-operated compactors in densifying soil.

Previous Compactor Force and Energy Measurements

Whiffen (1954), D'Appolonia et al. (1969), and Toombs (1972) have indirectly measured the force from vibratory roller compactors by using embedded earth pressure cells. The total dynamic compactor force, including the force from the static roller load, can be estimated by comparing pressure cell output during dynamic compactor loading to pressure cell output during static loading from the known roller load. According to Seed and Duncan's (1983) interpretation of these measurements, the total dynamic force from a vibratory roller is about 2 to 3 times the static roller load. Uncertainties in this approach include differences between the static and dynamic response of the embedded earth pressure cells, as well as nonlinearity in embedded earth pressure cell response to surface loads.

A different procedure for estimating the dynamic compactor force for a vibratory roller was employed by Yoo and Selig (1979), who proposed a model of the compactor represented by two lumped masses with springs and dashpots to represent the compactor suspension and soil. They measured accelerations on a roller compactor during operation and selected values of the model parameters (stiffness and damping of the compactor suspension and stiffness and damping of the soil) to give the best fit to the measured accelerations. They then used the model to calculate the dynamic force during roller operation. For a self-propelled, 8,800 kg (19,500 lbf) gross weight vibratory roller compactor operating in the 25 to 35 Hz frequency range, Yoo and Selig calculated a total peak dynamic load of 1.4 times the static roller load. The accuracy of this approach relies on the ability of the model to match the actual dynamic response of the compactor and soil.

Selig (1982) indicates that compactive effort for towed roller compactors can be determined by dividing the compacted volume into the product of the towing force and the distance traveled by the roller.

The Light Equipment Manufacturer's Bureau (LEMB) has developed a method for rating hand-operated, vibrating plate compactors (LEMB 1981b). This method provides a force rating but not an energy rating. The force rating is the calculated peak centrifugal force from the counter-rotating eccentric weights that are attached to the base plate of a vibrating plate compactor. The magnitude of the peak centrifugal force is given by

$$Q_0 = m_e\, e\, \omega^2 \tag{1}$$

where

Q_0 = the peak centrifugal force, N (lbf),
m_e = the mass of the eccentric weights, kg (lbf·s^2/in.),
e = the distance from the center of rotation to the center of mass of the eccentric weights, m (in.), and
ω = the rotation rate of the eccentric weights, rad/s.

For the model BPU 2440A vibrating plate compactor used in this study, the manufacturer's rated centrifugal force is 24.0 kN (5400 lbf).

It is noted that the centrifugal force calculated using Eq 1 is different from the contact force between the base plate and the soil. There are two reasons for the difference: 1) the soil provides flexible support instead of the rigid restraint implicit in the centrifugal force calculation, and 2) the weight of the compactor applies force to the soil that is not included in the centrifugal force calculation. The first effect results in contact force magnitudes that are generally below centrifugal force magnitudes. The second effect serves to increase contact forces. Since hand-operated compactors are light, the first effect will dominate and the actual contact forces will generally be less than the centrifugal force. Exceptions could occur if the compactor is operating near resonant frequency or if it is operating on a hard material so that the base plate loses contact during part of each cycle. High contact forces could be generated at impact in the latter situation.

The LEMB has also developed a method for rating hand-operated, rammer compactors (LEMB 1981a). In their method, a 19 mm (3/4 in.) diameter hardened steel ball is fastened to the bottom of the ramming shoe, and the compactor is operated on a 25 mm (1 in.) thick steel plate. Indentations made by the steel ball on the plate are measured and compared to indentations produced during a calibration procedure in which a weight, with the steel ball attached, is dropped onto the metal plate from various heights. Thus, the LEMB procedure yields a measure of energy delivered to the steel plate per blow from the compactor. A rated force is also calculated in the LEMB procedure by dividing the rated energy by the "standard soil deflection" which is defined by the LEMB to be 6.35 mm (0.25 in.). For the rammer compactor model used in this study, the manufacturer's rated energy is 78.4 J (57.8 ft-lbf) per blow, and the rated force is 12,300 N (2775 lbf).

The rated energy determined by the LEMB method should be a fairly realistic measure of the energy the compactor will apply to soil, since energy is the basis of the rating method. On the other hand, the rated force is only a nominal value since it is based on an assumed soil deflection. Variations in soil stiffness between dry and wet soil, for example, will be accompanied by variations in deflection and contact forces, even if the energy per blow remains constant. Consequently, actual compactor forces are not expected to be the same for all soils, and they could be quite different from the rated force. Even if the deflection during impact on a particular soil is 6.35 mm (0.25 in.), the rated force represents an average, and the peak force would be higher.

Compaction Equipment

Two hand-operated compactors were used in this study: a vibrating plate compactor and a rammer compactor.

Vibrating Plate Compactor

The vibrating plate compactor is a Wacker model BPU 2440A, which is powered by a 3,800 W (5 horsepower), 4-cycle engine that drives counter-rotating eccentric weights. The eccentric weights rotate at a frequency of about 100 Hz on axles fixed to a steel base plate that contacts the soil. A schematic diagram of the vibrating plate compactor is shown in Figure 1a. The operating weight of the compactor is 125 kg (275 lbf).

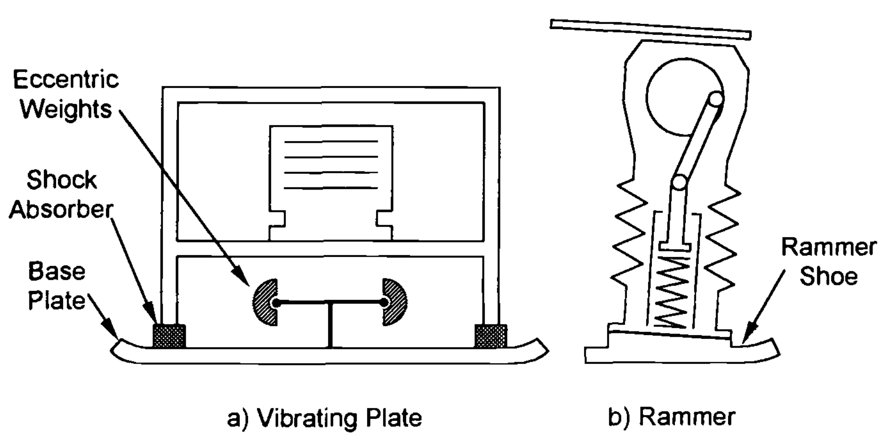

Figure 1 – *Schematic Diagrams of a) Vibrating Plate and b) Rammer Compactors*

Forward and reverse directional control of the vibrating plate compactor is provided by shifting the eccentric weight on one shaft out of phase with the eccentric weight on the other shaft. This is accomplished by a hydraulic system actuated from a control on the operator's guide handle. In Figure 1b, the eccentric weights are shown in their neutral position and the resultant centrifugal force is vertical. When the eccentric weights are held out of phase, a horizontal force component occurs. In addition to providing directional control, the phase shift also reduces the net eccentricity of the eccentric weights compared to the neutral position.

For the particular compactor used in this study, measurements of mass times eccentricity and rotation rate were made to calculate the centrifugal force using Eq 1 (Filz and Brandon 1993). The measured values of mass times eccentricity are listed in Table 1, which shows that the mass times eccentricity is higher with the eccentric weights held in the neutral position than in the forward position. The average measured frequency of rotation of the eccentric weights was 99 Hz, whereas the manufacturer's rated frequency for the model BPU 2440A is 90 Hz. Frequencies of 90 and 99 Hz (565 and 622 rad/s) were used to calculate the centrifugal force from Eq 1, and the results are listed in Table 1. The values of centrifugal force in Table 1 are lower than the manufacturer's rated centrifugal force of 24.0 kN (5400 lbf). This result indicates that the measured mass times eccentricity of the eccentric weights was less than the value used in the manufacturer's rating.

Table 1 – *Peak Centrifugal Force for the Vibrating Plate Compactor*

Eccentric Weight Position	Mass Times Eccentricity, kg-m (lbf-s^2)	Peak Centrifugal Force, kN (lbf)	
		At 90 Hz	At 99 Hz
Neutral	0.0578 (0.0130)	18.5 (4150)	22.3 (5020)
Forward	0.0406 (0.00914)	13.0 (2920)	15.7 (3540)

Rammer Compactor

The rammer compactor is a Wacker model BS 60Y, which is powered by a 3,000 W (4 horsepower), 2-cycle engine that drives a ramming shoe into contact with the soil at a percussion rate of about 10 blows per second. The ramming shoe is made of polyethylene with a steel bottom plate. A schematic diagram of the rammer compactor is shown in Figure 1b. The compactor weighs 62 kg (137 lbf). An operator can exert directional control by tilting the compactor in the desired direction of travel.

Soil Types

Three types of soil were used in the testing: Yatesville silty sand No. 1, Yatesville silty sand No. 2, and Light Castle sand.

Yatesville silty sand is an alluvial soil from the foundation of Yatesville Lake Dam on Blaine Creek in Lawrence County, Kentucky. About 47 percent of the Yatesville silty sand passes the No. 200 sieve. It is non-plastic, and its group symbol is SM according to ASTM Test Method for Classification of Soils for Engineering Purposes (ASTM D 2487-85). Two batches of the Yatesville silty sand were used in the testing. They are designated Yatesville silty sand No. 1 (YSS1), which was used in tests EP 1 through EP

12, and Yatesville silty sand No. 2 (YSS2), which was used in tests EP 13 and EP 14. For YSS1, the maximum dry unit weight is 1,920 kg/m^3 (120 lbf/ft^3), and the optimum water content is 12.5 percent, according to ASTM Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb (2.49-kg) Rammer and 12-in. (305-mm) Drop (ASTM D 698-78). For YSS2, the maximum dry unit weight is 2,000 kg/m^3 (125 lbf/ft^3) with an optimum water content of 11 percent, according to test method ASTM D 698-78.

The Light Castle sand, which was used in tests EP 15 and EP 16, is clean, fine sand consisting predominantly of subangular grains of quartz. The Light Castle sand was obtained from a quarry in Craig County, Virginia. About 68 percent of the sand passes the No. 40 sieve and less than 1 percent passes the No. 200 sieve. Its group symbol is SP according to ASTM Test Method for Classification of Soils for Engineering Purposes (ASTM D 2487-85). The maximum dry unit weight is 1700 kg/m^3 (106 lbf/ft^3), according to ASTM Test Method for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table (ASTM D 4253-83). The minimum dry unit weight is 1420 kg/m^3 (88.5 lbf/ft^3), according to ASTM Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density (ASTM D 4254-83).

Instrumentation Systems

The transducers, data acquisition systems, and data reduction methods employed to measure dynamic contact forces and energy transfer rates are described in detail by Filz and Brandon (1993). Piezoelectric transducers were used to measure forces and accelerations at various locations on the compactors. A Hall effect device was used to measure the rotation rate of the eccentric weights in the vibrating plate compactor. The data acquisition system was operated at a sampling rate of 50,000 Hz. The dynamic contact forces for the vibrating plate compactor were obtained by summing the products of the masses and accelerations of the compactor components. The dynamic contact forces for the rammer compactor were calculated by adding the force on top of the rammer shoe to the mass times acceleration of the rammer shoe. The energy transfer rates were obtained from the force measurements by integrating the dynamic contact force over the vertical distance traveled by the vibrating plate base or rammer shoe while it was in contact with the soil. The vertical position of the vibrating plate base or rammer shoe was obtained by double integrating the base plate or shoe acceleration.

Test Procedures

The compactor force and energy measurements described in this paper were made during backfilling of lateral earth pressure tests performed using the Instrumented Retaining Wall Facility at Virginia Tech. The test facility, the lateral earth pressure test procedures, and the test results are described in detail elsewhere (Sehn and Duncan 1990, Filz and Duncan 1992, Filz and Duncan 1993, Filz and Brandon 1994, and Filz and Duncan 1996). Information relevant to the compactor force and energy measurements is presented here.

The backfill area adjacent to the instrumented wall is 1.8 m (6 ft) wide by 3.0 m (10 ft) long. The backfill was placed in loose lifts of sufficient thickness to produce compacted lifts 0.15 m (6 in.) thick. Thirteen lifts were placed in each test so that the total height of backfill against the instrumented wall was about 2.0 m (6.5 ft) at the end of each test. Each lift was compacted by several passes of the hand-operated compaction equipment.

Table 2 provides data for the 14 instrumented retaining wall tests that can be used for evaluating the effectiveness of the vibrating plate and rammer compactors. These tests are designated EP 3 through EP 16. Table 2 lists the compactor type, compaction time, soil type, water content, and compacted dry unit weight for each test. Compactor force and energy measurements were only obtained during tests EP 12 through EP 16.

Table 2 – *Compactor Use in the Instrumented Retaining Wall Tests*

Test Number	Soil Type[a]	Water Content, percent	Compactor[b]	Compaction Time, s/m^3 (s/ft^3)	Dry Unit Weight, kg/m^3 (pcf)
EP 3	YSS1	13.7	Vib. Plate	378 (10.7)	1829 (114.2)
EP 4	YSS1	10.1	Vib. Plate	304 (8.6)	1642 (102.5)
EP 5	YSS1	9.3	Vib. Plate	343 (9.7)	1642 (102.5)
EP 6	YSS1	9.7	Vib. Plate	332 (9.4)	1632 (101.9)
EP 7	YSS1	11.1	Vib. Plate	343 (9.7)	1724 (107.6)
EP 8	YSS1	12.1	Vib. Plate	360 (10.2)	1761 (109.9)
EP 9	YSS1	12.5	Vib. Plate	389 (11.0)	1805 (112.7)
EP 10	YSS1	11.8	Vib. Plate	360 (10.2)	1759 (109.8)
EP 11	YSS1	13.5	Vib. Plate	427 (12.1)	1767 (110.3)
EP 12	YSS1	12.3	Vib. Plate[d]	381 (10.8)	1767 (110.3)
EP 13[c]	YSS2	12.7	Rammer[d] Vib. Plate	456 (12.9) 226 (6.4)	1914 (119.5)
EP 14[c]	YSS2	10.1	Rammer[d] Vib. Plate	307 (8.7) 162 (4.6)	1900 (118.6)
EP 15	LCS	<0.1	Rammer[d]	304 (8.6)	1701 (106.2)
EP 16	LCS	<0.1	Vib. Plate[d]	290 (8.2)	1677 (104.7)

[a] "YSS1" indicates Yatesville silty sand No. 1, "YSS2" indicates Yatesville silty sand No. 2, and "LCS" indicates Light Castle sand.
[b] "Vib. Plate" indicates the Wacker BPU 2440A vibrating plate compactor. "Rammer" indicates the Wacker BS 60Y rammer compactor.
[c] Both the vibrating plate and rammer compactors were used in tests EP 13 and EP 14.
[d] Compactor force and energy measurements were obtained.

Force and Energy Measurements

Table 3 provides a statistical summary of all the compactor force and energy measurements made during this study. Measurements of the dynamic force from the

Table 3 – *Summary of Compactor Force and Energy Measurements*

	Compacted Soil					Peak Force, kN (lbf)		Energy, J per cycle (ft-lbf per cycle)	
Test Number	Soil Type[a]	Dry Unit Weight, kg/m^3 (pcf)	Water Content, percent	Compactor[b]	Number of Measurements	Average	Standard Deviation	Average	Standard Deviation
EP 12	YSS1	1,767 (110.3)	12.3	Vib. Plate	12	5.4 (1215)	0.67 (150)	5.64 (4.16)	1.86 (1.37)
EP 13	YSS2	1,914 (119.5)	12.7	Rammer	10	22.4 (5040)	2.0 (460)	75.5 (55.7)	4.3 (3.2)
EP 14	YSS2	1,900 (118.6)	10.1	Rammer	9	32.6 (7330)	4.3 (970)	66.4 (49.0)	6.1 (4.5)
EP 15	LCS	1,701 (106.2)	< 0.1	Rammer	15	21.3 (4780)	5.0 (1120)	70.8 (52.2)	5.7 (4.2)
EP 16	LCS	1,677 (104.7)	< 0.1	Vib. Plate	16	5.8 (1310)	1.2 (280)	5.44 (4.01)	1.78 (1.31)

[a] "YSS1" indicates Yatesville silty sand No. 1, "YSS2" indicates Yatesville silty sand No. 2, and "LCS" indicates Light Castle sand.

[b] "Vib. Plate" indicates the Wacker BPU 2440A vibrating plate compactor. "Rammer" indicates the Wacker BS 60Y rammer compactor.

vibrating plate compactor were made during tests EP 12 and EP 16. Measurements of the dynamic force from the rammer compactor were made during tests EP 13 through EP15. In all cases, the measurements were made during operation of the compactor after the standard number of compaction passes had been completed on a lift. Thus, the force and energy measurements are for the compacted soil condition, not the loose soil condition.

Vibrating Plate Compactor

A typical set of force, base position, and energy traces versus time for the vibrating plate compactor during forward compactor travel is shown in Figure 2. The peak compressive forces are about 5.3 kN (1200 lbf) and peak tensile forces are about 1.3 kN (300 lbf). The operating frequency is about 98 Hz. The base position trace in Figure 4 shows a peak to peak displacement amplitude of about 1.5 mm (0.06 in.). The location of the zero position is arbitrarily set at the midpoint of the position range.

Peak tensile forces for all the measurements on the Yatesville silty sand average 1.8 kN (410 lbf) and peak tensile forces for all the measurements on the Light Castle sand average 1.1 kN (250 lbf). A small tensile stress, or "suction," between the surface of the moist Yatesville silty sand and the smooth base plate of the vibratory compactor during rapid unloading would account for the tensile forces measured on the Yatesville silty sand. For the Light Castle sand, a short term air pressure reduction of 7 kPa (1 psi) in the sand beneath the compactor base plate as it rapidly pulls up from the sand would account for the tensile force measured in this case.

The measured peak compressive force from the vibrating plate compactor for all the tests summarized in Table 3 ranged from 4.6 to 7.5 kN (1030 to 1680 lbf). These forces are much less than the manufacturer's rated centrifugal force of 24.0 kN (5400 lbf). The reasons for the discrepancy are that 1) the manufacturer's rating seems to be based on a higher value of mass times eccentricity than was measured for the compactor used in this study, as was shown in Table 1, and 2) the rating calculation is for eccentric weights rotating about a shaft fastened to a fixed support, whereas soil provides flexible support.

The average measured energy transfer per cycle for all the vibrating plate compactor measurements summarized in Table 3 is 5.54 J (4.08 ft-lbf) per cycle.

Rammer Compactor

A typical set of force, rammer shoe position, and energy traces for the rammer compactor is shown in Figure 3. The peak force in Figure 3 is about 24.5 kN (5500 lbf) and the impact duration is short, less than about 0.005 seconds. The operating frequency is about 10.6 hertz. Between impacts, the instrumentation system returns a measurement of zero force on the base of the rammer shoe, which is correct for the shoe not being in contact with the soil. The position trace shows that the rammer shoe moved through a vertical distance of about 63 mm (2.5 in.) during each cycle. The location of the zero position is arbitrary, and in Figure 3 it is set at the approximate point of rammer shoe contact with the soil, as indicated by the beginning of the force pulse. The energy applied by the compactor to the soil accumulates as a step function, increasing at a rate of about 75 J (55 ft-lbf) per impact.

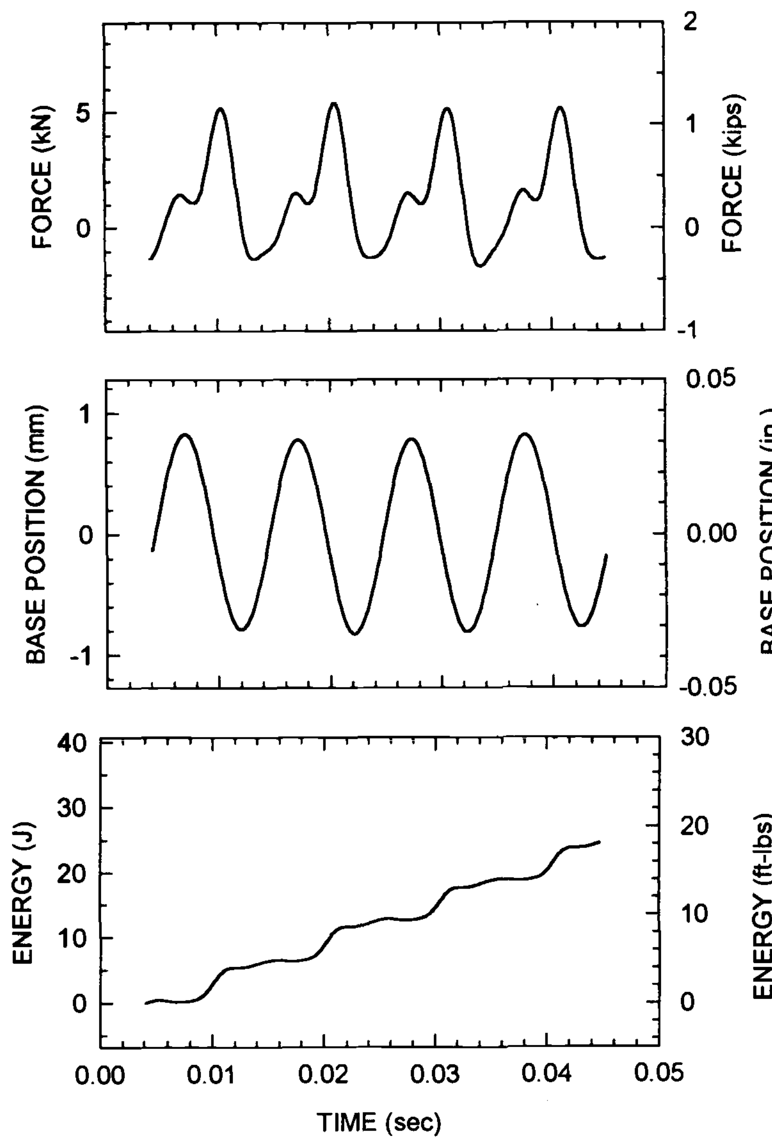

Figure 2 - *Typical Force, Position, and Energy Traces for the Vibrating Plate Compactor*

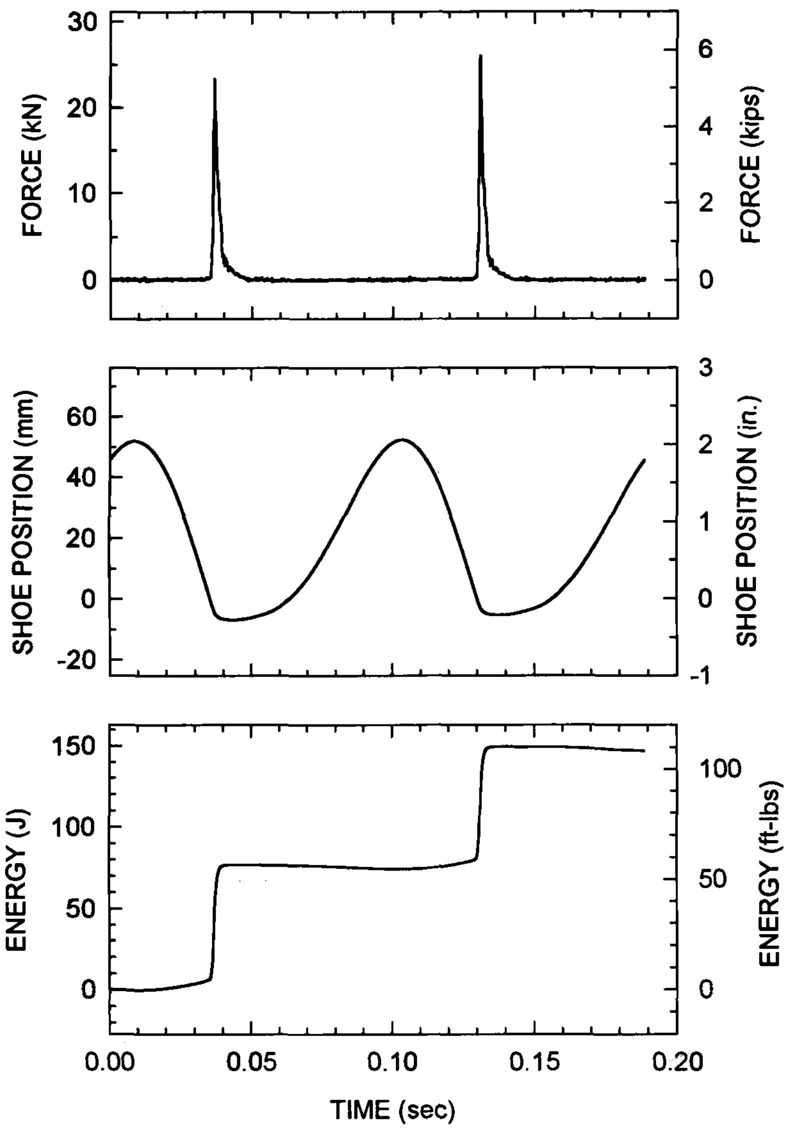

Figure 3 – *Typical Force, Position, and Energy Traces for the Rammer Compactor*

The first impact in Figure 3 is shown in Figure 4 with an expanded time scale. The approximate points of rammer shoe contact and departure from the soil are also shown. Figure 4 indicates that the soil in this test deformed about 8 mm (0.3 in.) during rammer impact.

The average measured energy per cycle for the rammer compactor for all the tests summarized in Table 3 is 71.0 J (52.4 ft-lbf), which is close to the manufacturer's rated energy of 78.4 J (57.8 ft-lbf). However, the measured peak contact forces ranged from 15.7 to 38.0 kN (3520 to 8550 lbf). The substantial variation in peak force is due to variations in soil stiffness as soil type and moisture condition changed from test to test. For example, the average of the peak force measurements for test EP 13, in which the backfill was soft because it was compacted wet of optimum, is 22.4 kN (5040 lbf). On the other hand, the average of the peak force measurements for test EP 14, in which the backfill was stiff because it was compacted dry of optimum, is 32.6 kN (7330 lbf). For all cases, the measured peak forces are greater than the manufacturer's rated force of 12.3 kN (2775 lbf). As mentioned previously, the manufacturer's rated force represents an average force corresponding to "the standard soil deformation" of 6.35 mm (0.25 in.) during impact. Peak forces are expected to be higher.

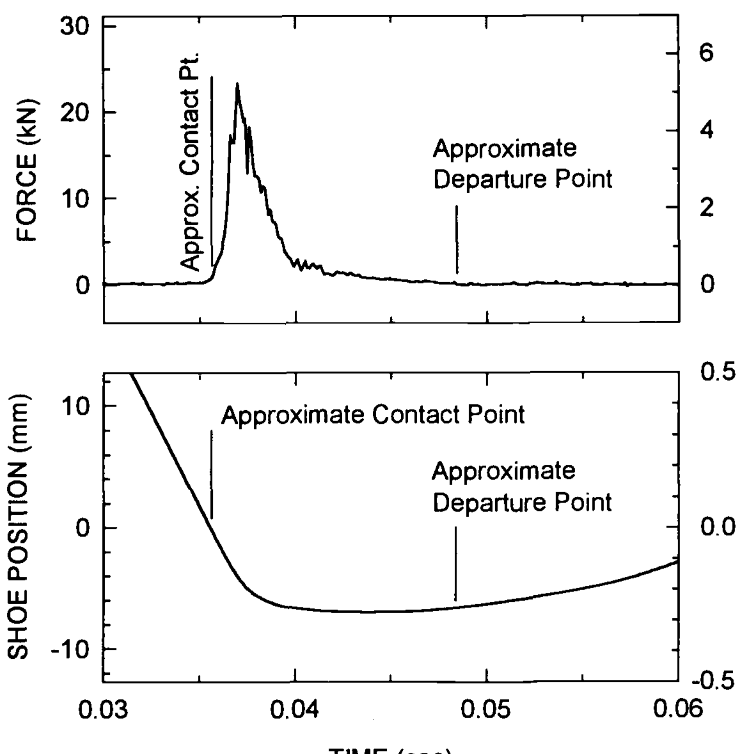

Figure 4 - *Expanded Force and Position Traces for the Rammer Compactor*

Table 4 – *Compactive Efforts in the Instrumented Retaining Wall Tests*

Test Number	Compactor[a]	Compaction Time, s/m^3 (s/ft^3)	Energy Transfer per Cycle, J (ft-lbf)	Compactor Period, s	Compactive Effort, kN-m/m^3 (ft-lbf/cu ft)
EP 12	Vib. Plate	381 (10.8)	5.64 (4.16)	0.0102	212 (4420)
EP 13[b]	Rammer	456 (12.9)	75.5 (55.7)	0.0946	363 (7590)
EP 13[b]	Vib. Plate	226(6.4)	5.54 (4.08)[c]	0.0101[c]	124 (2600)
EP 13[b]	Combined	682 (19.3)	--	--	488 (10190)
EP 14[b]	Rammer	307 (8.7)	66.4 (49.0)	0.0881	232 (4840)
EP 14[b]	Vib. Plate	162 (4.6)	5.54 (4.08)[c]	0.0101[c]	90 (1870)
EP 14[b]	Combined	469 (13.3)	--	--	321 (6710)
EP 15	Rammer	304 (8.6)	70.8 (52.2)	0.0888	241 (5030)
EP 16	Vib. Plate	290 (8.2)	5.44 (4.01)	0.0100	158 (3300)

[a] "Vib. Plate" indicates the Wacker BPU 2440A vibrating plate compactor. "Rammer" indicates the Wacker BS 60Y rammer compactor.
[b] Both the vibrating plate and rammer compactors were used in tests EP 13 and EP 14.
[c] Because measurements of the vibrating plate energy were not obtained during tests EP 13 and EP 14, average values from tests EP 12 and EP 16 were used for these calculations.

Compactive Effort

For the instrumented retaining wall tests in which compactor force measurements were made, the compactive effort (transferred energy per unit volume of compacted soil) can be computed because the energy per cycle, compactor period, compaction time, and compacted volume are known. The calculated compactive efforts for tests EP 12 through EP 16 are shown in Table 4.

Estimates of the compactive efforts for the instrumented retaining wall tests without compactor force measurements can be made by noting that the average values of transferred energy per cycle listed in Table 4 are not strongly dependent on the properties of the soil being compacted. In particular, the transferred energy per cycle for the vibrating plate compactor was 5.64 J (4.16 ft-lbf) in test EP 12 on moist Yatesville silty sand No. 1 and 5.44 J (4.01 ft-lbf) in test EP 16 on dry Light Castle sand. The transferred energy per cycle for the rammer compactor was 75.5 J (55.7 ft-lbf) in test EP 13 on wet Yatesville silty sand No.2, 66.4 J (49.0 ft-lbf) in test EP 14 on dry Yatesville silty sand No. 2, and 70.8 J (52.2 ft-lbf) in test EP 15 on dry Light Castle sand. The peak force, on the other hand, is dependent on the properties of the soil being compacted, as discussed previously.

The compactive efforts for the tests during which compactor force measurements were not made, i.e., tests EP 3 through EP 11, can be estimated using the compaction times listed in Table 2 and the approximately constant transferred energy per cycle measured for the vibrating plate compactor in tests EP 12 and EP 16. This results in compactive efforts ranging from 168 to 235 kN-m/m^3 (3,500 to 4,900 ft-lbf/cu ft) for tests EP 3 through EP 12, which are all the tests backfilled with Yatesville silty sand No. 1. The dry unit weights resulting from this level of compactive effort are compared with the dry unit weights resulting from the Standard Proctor (ASTM D 698-78) and Modified Proctor (ASTM D 1557-78) compactive efforts in Figure 5. The figure shows that, because the compactive effort is lower for the instrumented retaining wall tests than for the Standard Proctor test, the maximum dry unit weight is lower and the optimum water content is higher.

A similar comparison was made for Yatesville silty sand No. 2. In this case, two non-standard, low-energy laboratory compaction curves were obtained in addition to the Standard Proctor (ASTM D 698-78) and Modified Proctor (ASTM D 1557-78) curves. All four curves are shown on Figure 6, along with the results from tests EP 13 and EP 14. The compaction curve shown for the instrumented retaining wall tests in Figure 6 is for the compactive effort of test EP 14 only, since test EP 13 was compacted wet of optimum and, consequently, does not serve to provide good definition of the compaction curve.

The maximum dry unit weights from Figures 5 and 6 are plotted versus compactive effort in Figure 7. The trend is for increasing maximum dry unit weight with increasing compactive effort. The maximum dry unit weights for Yatesville silty sand No. 2 are larger than those for Yatesville silty sand No. 1. In both cases, the results from the instrumented retaining wall tests are in good agreement with the laboratory test data. This agreement indicates that the measured energies for these hand-operated compactors have the same effect on dry unit weights as equal energies applied in compaction tests performed in the laboratory.

As mentioned previously, it appears as though the transferred energy is not strongly dependent on the characteristics of the soil being compacted. Consequently, the data in Table 4 can be used to compute the average energy transfer rate for each compactor. These rates are listed in Table 5, along with other information about the two compactors used in this study, including the time required to develop the standard Proctor energy in the compacted soil. It can be seen in the table that the vibrating plate compactor imparts the standard Proctor compactive effort to one cubic meter of soil in about 18 minutes, and the rammer compactor accomplishes this energy transfer in about 12½ minutes. The efficiency with which these compactors transfer energy from their engines to the soil being compacted can be computed by dividing the engine powers into the energy transfer rates. As shown in Table 5, the efficiencies of the vibrating plate compactor and the rammer compactor are about 15 percent and 26 percent, respectively. It would be interesting to know whether similar values of efficiency apply to other compactors of the same types.

Conclusions

The two hand-operated compactors used in this study are commonly employed to compact backfill in confined areas and adjacent to structures such as walls and culverts. The compactors are different in their frequency of operation, energy per cycle, energy transfer rate, and peak contact force. These differences are summarized in Table 5.

For the vibrating plate compactor, peak forces ranged from 4.6 to 7.5 kN (1030 to 1680 lbf) and averaged 5.65 kN (1270 lbf). Measured peak forces were much less than the manufacturer's rated centrifugal force. The average rate of energy transfer to the soil was 548 W (404 ft-lbf/s), which means that it takes about 18 minutes to impart the

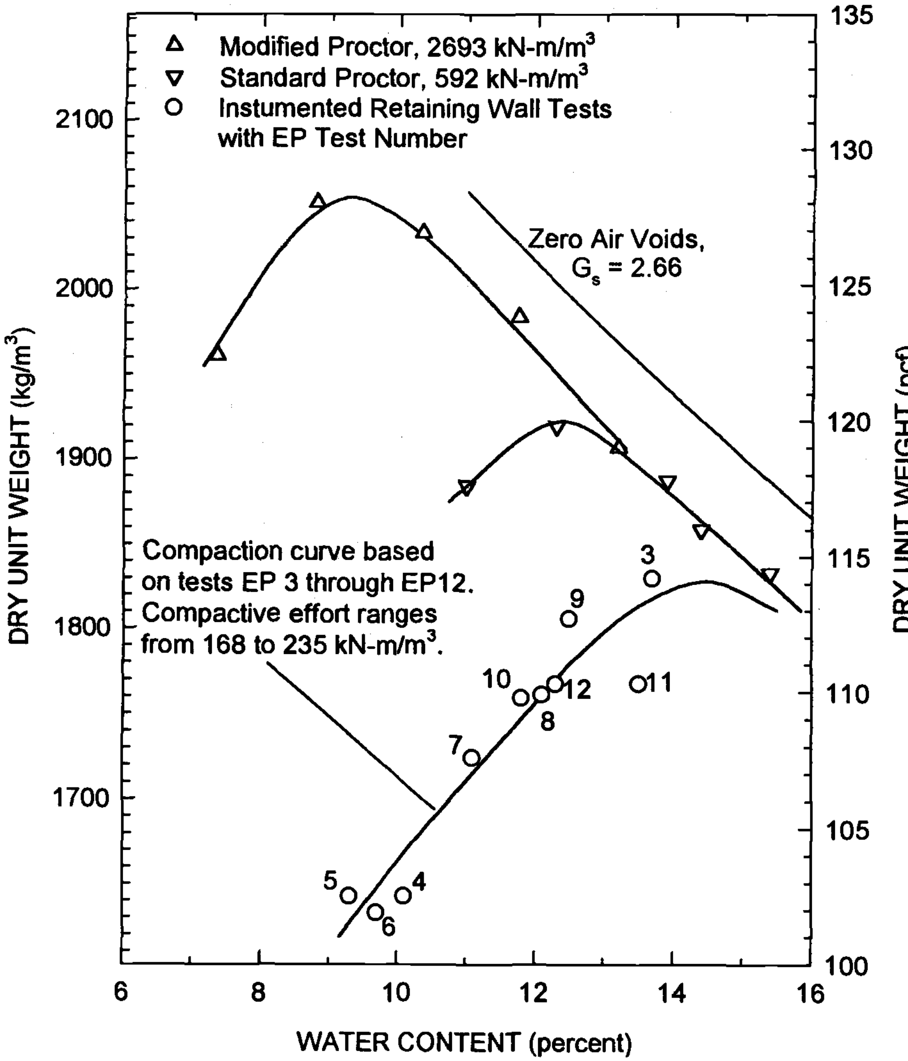

Figure 5 – *Compaction Curves for Yatesville Silty Sand No. 1*

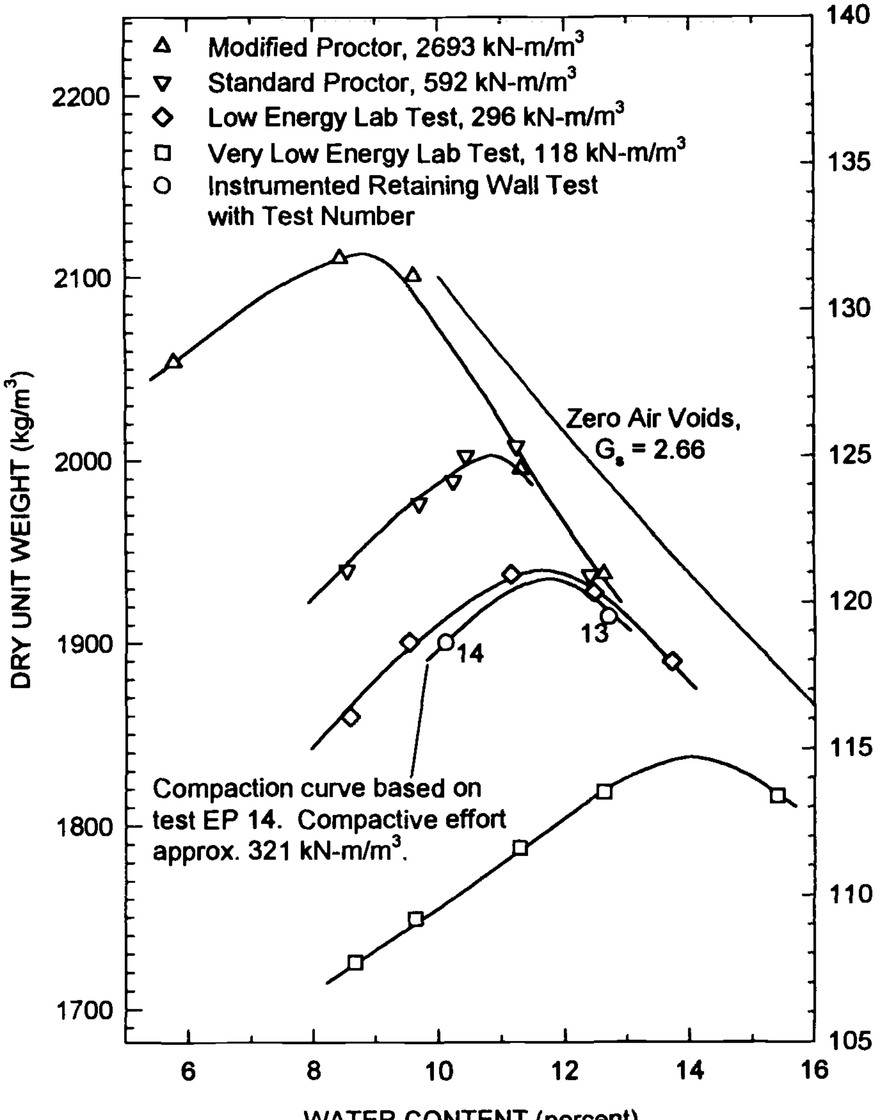

Figure 6 - *Compaction Curves for Yatesville Silty Sand No. 2*

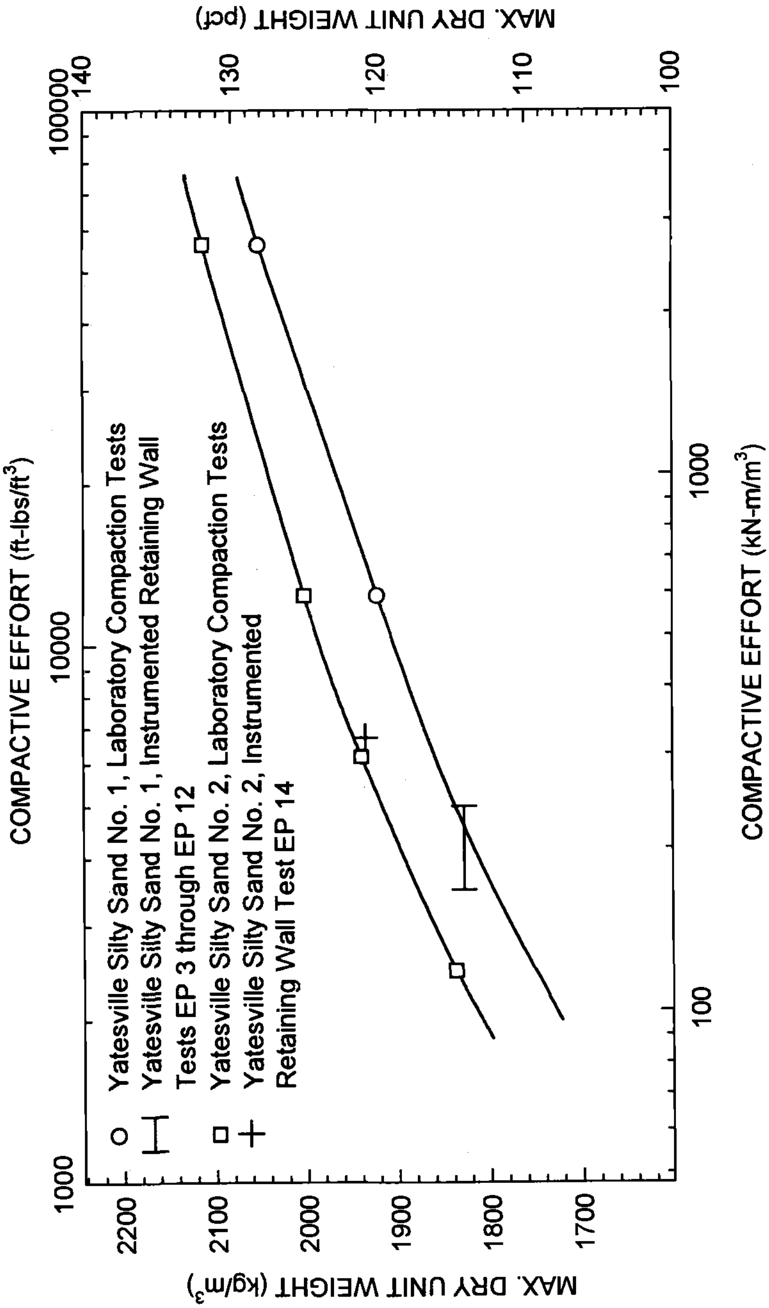

Figure 7 – Comparison of Compactive Efforts Applied in Laboratory Tests and by Hand-Operated Compaction Equipment

Table 5 – *Comparison of Compactor Performance*

Compactor[a]	Engine Power, W (hp)	Frequency, Hz	Range of Peak Contact Forces, kN (lbf)	Energy Transfer per Cycle[b], J (ft-lbf)	Time Rate of Energy Transfer[b], W (ft-lbf/s)	Time to Impart the Standard Proctor Energy[b], s/m^3 (s/ft^3)	Compactor Efficiency[b], percent
Vib. Plate	3730 (5)	99	4.6 to 7.5 (1030 to 1680)	5.54 (4.08)	548 (404)	1082 (31)	15
Rammer	2980 (4)	11.1	15.7 to 38.0 (3520 to 8550)	71.0 (52.4)	784 (578)	756 (21)	26

[a] "Vib. Plate" indicates the Wacker BPU 2440A vibrating plate compactor. "Rammer" indicates the Wacker BS 60Y rammer compactor.
[b] Average values.

standard Proctor compactive effort to one cubic meter of soil. This energy transfer rate is about equal to 15 percent of the power of the compactor's engine.

For the rammer compactor, the peak contact forces measured in this research ranged from 15.7 to 38.0 kN (3520 to 8550 lbf) and averaged 2.45 kN (5500 lbf). The peak contact force increased with increasing soil stiffness. The average measured energy transfer was 71.0 J (52.4 ft-lbf) per blow, and this value is close to the manufacturer's rated energy of 78.4 J (57.8 ft-lbf) per blow. However, the manufacturer's rated force of 12.3 kN (2775 lbf) is much lower than the measured peak forces. The average rate of energy transfer to the soil was 784 W (578 ft-lbf/s), which means that it takes about 12½ minutes to impart the standard Proctor compactive effort to one cubic meter of soil. This energy transfer rate is about equal to 26 percent of the power of the compactor's engine.

It is generally recognized that rammer compactors are better than vibrating plate compactors at compacting cohesive soils. This difference may be due, in part, to the ability of the high rammer contact force to overcome interparticle forces and break the soil down into a more compact arrangement.

Larger compaction-induced lateral earth pressures are expected in backfill compacted with the rammer compactor than in backfill compacted with the vibrating plate compactor because larger vertical stresses are induced in the backfill with the rammer compactor. High compaction-induced lateral earth pressures can cause cracking or excessive deformation of structures.

In instances where the two compactors achieve the same relative density or compaction, settlements of structures founded on backfill compacted with the rammer compactor will probably be smaller because the higher compaction-induced lateral stresses produced by the rammer compactor render the backfill less compressible.

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Equation for Complete Compaction Curve of Fine-grained Soils and Its Applications

Reference: Li, H. and Sego, D. C., "**Equation for Complete Compaction Curve of Fine-grained Soils and Its Applications,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384*, D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: A complete compaction curve for fine-grained soil and its characteristics are discussed. The formulation of the relationship to model the curve is introduced. The equation is versatile in its ability to quantify the shape, size and position of compaction curves. It is capable of describing the compaction curve from the dry to very wet condition and can be used to predict a family of compaction curves for a given soil for different input of compaction energies. All parameters in the equation have specific physical definitions. Simple procedures to obtain all parameters directly from a standard compaction curve are developed and described in detail. The examples show that the proposed equation is good at representing the soil compaction curve and demonstrates excellent agreement between laboratory test data and the predicted curves. The family of curves predicted by the equation would improve the practice of the one-point method in the field.

Keywords: compaction curve, equation, fine-grained soils, family curves, prediction, application

Since Proctor's first paper published in 1933, the compaction method has become one of the most widely used soil improvement techniques around the world. However, most laboratory and field test programs are concerned with the physical properties of the soil near or at the maximum dry density. The compaction curve itself over its whole range has seldom been studied in detail.

Only a few efforts have been made to mathematically describe the compaction curve. Almost all use second, third- or fourth-order polynomial equations (Hilf 1990, Bradet 1996, Howell et al. 1997). Although using a polynomial equation is simple, its application is restricted, as the regression parameters it uses are pure fitting parameters. It is common that these values change by up to three-orders of magnitude or even from positive to negative values (Howell et al. 1997). The other disadvantage

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is that the equation works well over only a limited moisture content range for a specific test. The predicted value may become negative or predict that the compaction curve exceeds the zero air void curve (ZAVC) which is impossible. In addition, compaction curves for the same soil using different compactive efforts usually have similarly shaped curves, but a polynomial equation has no specific parameter designed to account for this soil characteristic.

An equation has been developed by Li and Sego (1998) which can overcome all the above shortcomings and is considered useful. This paper summarizes the characteristics of the complete compaction curve, describes the equation, and introduces the newly developed procedures to obtain all parameters from compaction curve on a given soil.

The Characteristics of the Complete Compaction Curve for Fine-grained Soils

The complete compaction curve for a soil differs from the traditionally presented compaction curve with respect to the moisture range considered. It starts from the completely dry condition and ends well wet of the optimum water content. With the existence of the dry and wet conditions, the complete compaction curve allows a rational formulation to be defined.

The first time the complete compaction curve appeared is probably in Turnbull and Foster (1957) for graded crushed stones. It was found that for non-cohesive soil, the dry density increases as the water content decreased to zero.

Faure (1981) might be the first to study the complete compaction curve for fine-grained soil. Faure (1981), Saini and Chow (1984) and Faure and Da Mata (1994) studied the evolution of these curves with increasing clay content, clay mineralogy, and compactive effort. The complete curve for a fine-grained soil shows a nearly constant dry density on the dry side before the water content reaches a point called the *compaction sensitivity threshold* (CST) (Fig. 1). Faure (1994) presented compaction curves with this feature for thirty-four materials prepared by mixing clay with either fine sand or coarse sand, and thirty-six natural soils from France and four from Brazil. Results from standard compaction test and kneading compaction test at the University of Alberta using glacial lacustrine clay also illustrate the same features (Li and Sego 1999).

Figure 1 shows a typical complete compaction curve. When the soil is very dry, the dry density of fine-grained soil is almost constant (γ_{dd}). If using the degree of saturation instead of dry density, this characteristic can be seen in Fig. 2 where the degree of saturation increases linearly with water content. The linear portion is called the approach line. As water content approaches CST, the degree of saturation departs from the linear relationship and increases rapidly. From CST to point M, the degree of saturation increases rapidly between CST and point A then the rate of increase slows until it reaches a constant value called the maximum saturation (S_m).

In this paper, a new technique has been used to determine the position of CST as shown in Fig. 2. Drawing a tangent line through the inflection point on the S versus w curve, the line meets the approach line at a point where the degree of saturation equals the saturation at the CST (S_{CST}). This approach provides a standard method to locate the CST point on a compaction curve.

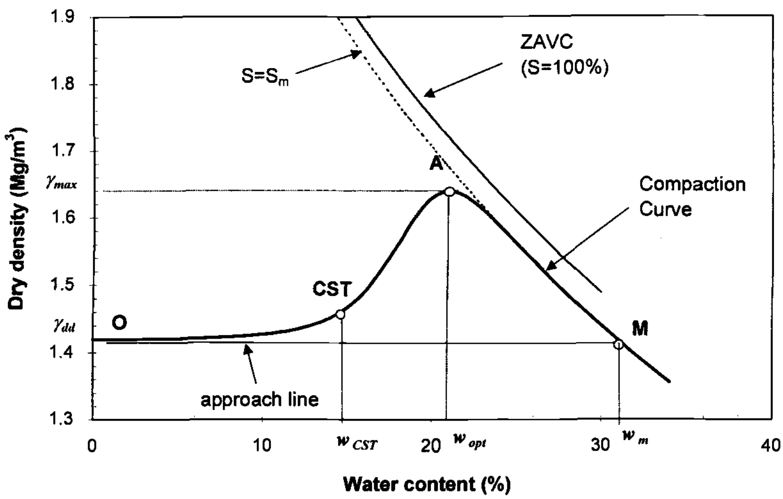

Fig. 1 - *Sketch of complete compaction curve ($\gamma_d - w$)*

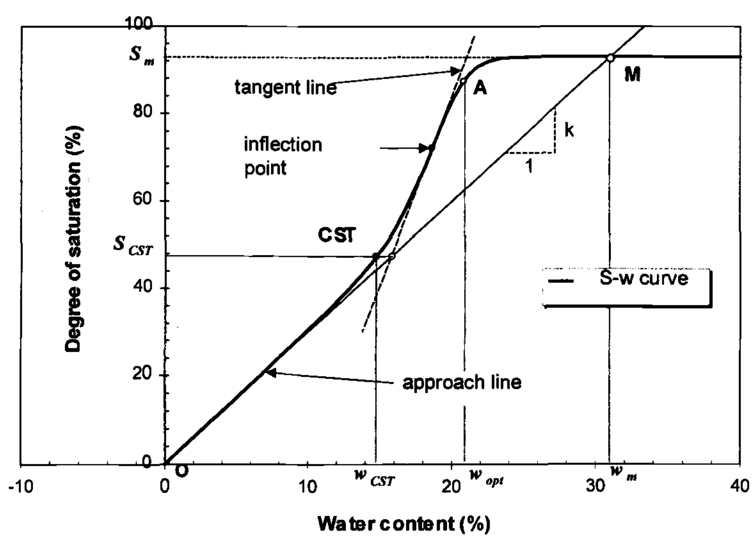

Fig. 2 - *Characteristics of the degree of saturation versus water content curve (S-w)*

The Boundary Conditions

The fact that the dry density at the very dry condition remained constant (γ_{dd}) is an important characteristic of fine-grained soils. Unfortunately, this has been neglected for years. It is the limit on the dry side of the complete compaction curve. In Fig. 2, the slope of approach line (k) is related to γ_{dd} as follows:

$$k = \frac{S_{dd}}{w_{dd}} = \frac{1}{\frac{\gamma_w}{\gamma_{dd}} - \frac{1}{G}} \tag{1}$$

where S_{dd} and w_{dd} are the degree of saturation and water content of soil compacted when completely dry or at the very dry condition ($w << w_{CST}$). γ_w is the density of water and G is specific gravity of soil particles.

It is assumed that the maximum degree of saturation (S_m) is reached at point M where the compaction curve intersects the approach line of the S versus w curve (Fig. 2). This assumption does not result in large errors because the degree of saturation only changes by a small amount on the wet side of the optimum water content. The water content at M (w_m) can be easily determined as:

$$w_m = \frac{S_m}{k} \tag{2}$$

For fine-grained soil, it is also shown that the maximum degree of saturation (S_m) usually remains constant and does not change as the compaction effort changes (Seed et al. 1960, Lee and Haley 1968). In practice, one can find S_m from the wet side of the compaction curve which runs roughly parallel to the zero air void curve (ZAVC) (Hausmann 1990). Thus S_m is another important referenced value and it controls the limit on the wet side of the compaction curve.

The Shape Factor

When a fine-grained soil is densified under a constant compactive effort but with varying moisture contents, a typical dry density versus water content relationship becomes apparent. The shape of the compaction curve is mostly related to the particle size distribution of the soil and compaction method. For the same soil when compacting it using the same compaction method, many laboratory tests and field tests show the same shape of the compaction curves when it is compacted using different compactive energy or effort (Turnbull and Foster 1958, Lambe 1958, Seed et al. 1960, Lee and Haley 1968). In embankment construction, different compaction equipment is generally used and they have different weights and use a different number of passes to represent different field compaction effort. It is also true when different thicknesses of lifts are compacted because the compactive effort differs with depth within the lift (Turnbull and Foster 1958, Parson 1992). Therefore, knowing the family of compaction curves is useful for the designer to evaluate the use of different layer thicknesses or different compaction efforts in the field.

Unfortunately, this important characteristic cannot be used to predict the compaction curve since no equation is available to quantify the generalized shape of the family of compaction curves for a given soil. Li and Sego (1998) discussed a shape factor n that reasonably represents this important characteristic. It can be used to predict the compaction curve since it remains unchanged for the same soil while compacting the soil using a given compaction method.

The Compactable Moisture Range

In addition to the shape of the curve, the dry density of a soil increases in only a certain moisture content range. During soil compaction, water at a certain content or volume becomes effective at lubricating the soil particles and decreasing the matric suction. This allows the density to increase. Increasing the clay content helps the soil particle to pack together at smaller water content (i.e. CST is smaller) (Faure 1981, 1994). Since the maximum saturation changes little for fine-grained soil, higher plasticity generally leads to a larger compactable moisture range.

The Formulation

The approach used to analytically represent the compaction curve employs the degree of saturation (S) versus water content (w), which then can be converted to the compaction curve.

$$S = f(w) \tag{3}$$

The equation for the compaction curve therefore can be written as follows:

$$\gamma_d = \frac{G\gamma_w}{1 + \dfrac{wG}{f(w)}} \tag{4}$$

where γ_d is the dry density of soil.

This converts as illustrated in Fig. 2 to give a more convenient curve for use. The degree of saturation and water content are both dimensionless. So the shape of the curve and its parameters are unique for a soil.

The derivation of the mathematical expression of $f(w)$ has been included in Li and Sego (1998). The equation for S versus w can be written as

$$S = S_m - \frac{S_m}{w_m} \cdot \frac{(w_m - w)^{n+1}}{(w_m - w)^n + p^n} \qquad w<w_m \tag{5}$$

where S_m is the maximum degree of saturation, w_m is the water content when S_m is reached, n and p are two parameters which control the shape and width of the compaction curve.

One should observe that the degree of saturation for the dry condition is not zero as predicted by Equation 5. This may cause large dry density according to Equation 4 when the water content approaches zero as shown in Curve A of Fig. 3. During tests with very high compaction energy, the error may not be neglected since the compaction curve moves to the left and close to dry condition. This concern can be solved by employing the revised relationship presented by Li and Sego (1998).

$$S = S_m - S_m \times \left(\frac{w_m - w}{w_m} \right)^{n+1} \times \left(\frac{w_m^n + p^n}{(w_m - w)^n + p^n} \right) \qquad w<w_m \qquad (6)$$

Equation 6 describes curve B in Fig. 3 and it intersects γ_{dd} at zero moisture content.

The analytical curves shown in Fig. 4 indicate the compaction curve shifts vertically when S_m changes. As discussed previously, S_m usually is the constant for different compactive efforts for a given fine-grained soil.

Parameter w_m can be obtained by extending a horizontal line from the point of an air-dried soil sample (γ_{dd}) to cross the compaction curve on the wet side (Fig. 1). The parameter k, w_m or γ_{dd} are interrelated as shown in Equations 1 and 2, so w_m can be calculated from γ_{dd} in Equations 1 and 2. A decrease of w_m can also be considered as an increase of γ_{dd}, which means the compaction curve shows higher density on the dry side of the optimum moisture content but changes little on the wet side (Fig. 4).

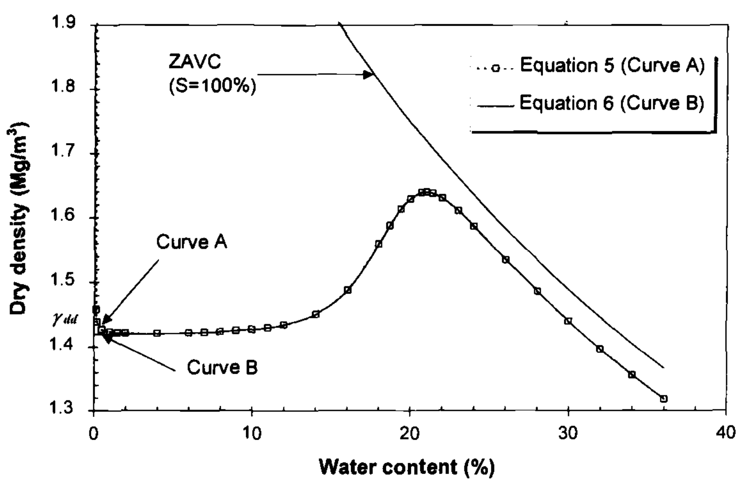

Fig. 3 - *Complete compaction curves by using Equations 5 and 6*

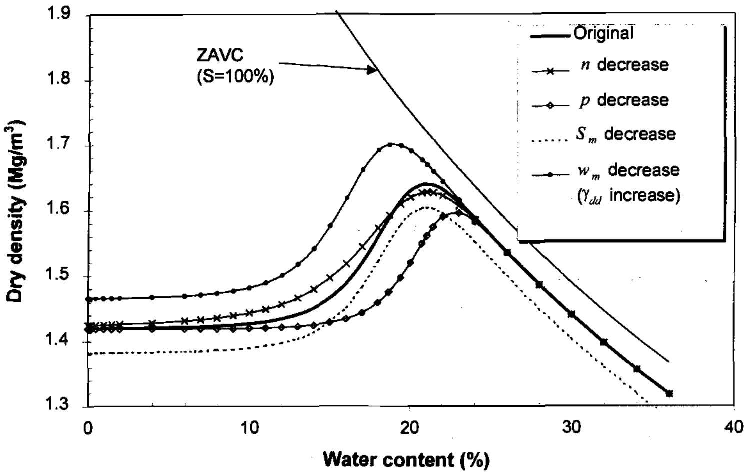

Fig. 4 - *Influence of the parameters n, p,* S_m, *and* w_m *on the complete compaction curve*

This is the general observation when a higher compaction effort is used in compaction tests. As the water content has little influence on the value of γ_{dd}, the compaction energy is the only external factor. Therefore, γ_{dd} is the index of compaction energy used during a test.

Figure 4 shows the influence of parameter *n* on the dome shape of the compaction curve. When *n* increases, the compaction curve becomes narrower and achieves a higher optimum dry density. The curve tends to flatten when *n* decreases. This indicates that *n* controls the shape of the compaction curve. The shape of a compaction curve depends on both the soil fabrics and compaction method used. As discussed before, when using the same compaction method with more energy to compact a soil, the compaction curves have a similar characteristic shape that is shifted up and to the left. Therefore parameter *n* can be assumed as a constant for a particular soil. Compaction curves with a characteristic shape thus can be quantified by using a constant value of *n*. This provides the engineer with the ability to predict a family of compaction curves for a given soil for different compactive energy inputs. Therefore, parameter *n* will be called the shape factor because of its important influence on the shape of the compaction curves. For natural soils it varies in a range between 4 and 12.

Parameter *p* relates to the width of the upper curved part of compact curve, which can be called compactable moisture range (Fig. 4). Parameter *p* allows the equation to define the size of this range without changing its shape factor (*n*) and boundary conditions (defined by S_m and γ_{dd}). It can be called the index of the compactable moisture range. Because *p* is a measure of dome width for a compaction curve, it is directly related to w_{CST} of a soil that depends on the clay content of the soil according to Faure (1991). The width of the compactable moisture range is about 1~1.5 times p.

Determination of Parameters n and p from a Compaction Curve

As discussed, the parameters S_m and w_m (or γ_{dd}, k) can be easily obtained using Figs. 1 and 2. The authors recently developed a simple procedure to determine n and p directly from a compaction curve. It is a great advantage to avoid having to use non-linear regression of the compaction data and to ensures the equation can easily be used in everyday engineering practice.

Determination of the Shape Factor (n)

A most recent study has developed a simple procedure to get n directly as illustrated in Fig. 5. Same scale has to be used in both S and w axes.

Procedure: From origin O, draw a tangent line on a S-w curve to find point A (point with maximum dry density), or use the optimum water content (w_{opt}) obtained from the compaction curve to determine A as shown in Fig. 5. Extend line MA to cross the X-axis at point B. The difference in water content between points M and B equals to nw_{opt}. Thus,

$$n = \frac{nw_{opt}}{w_{opt}} = \frac{BM'}{OA'} \tag{7}$$

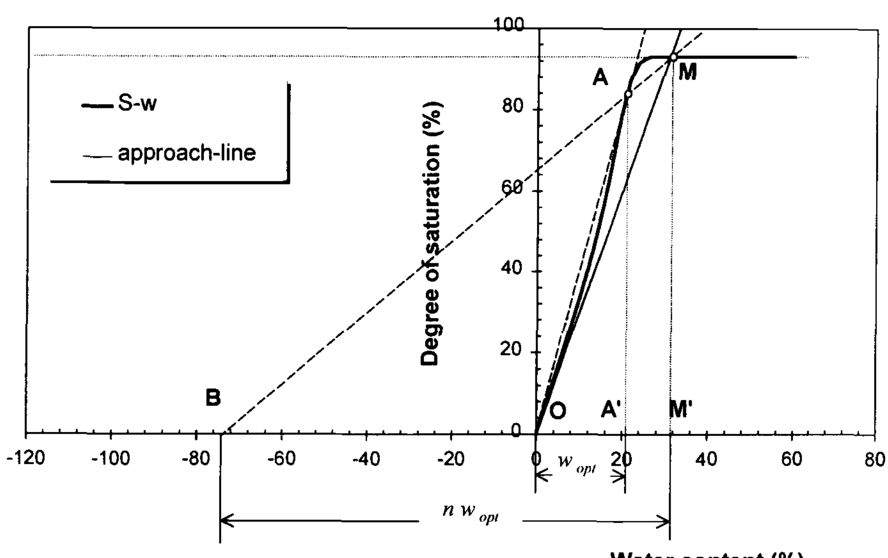

Fig. 5 - *Determination of parameter n*

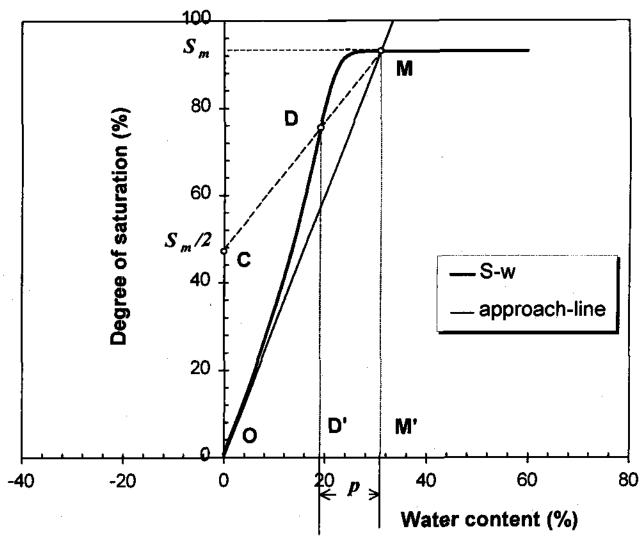

Fig. 6 - *Determination of parameter p*

Using Fig. 5, from the coordinate of points M', A' and B, BM' = 31%-(-74%) = 105%, OA' = 21%, therefore value of n is determined to be 5.

Determination of the Index of the Compactable Moisture Range (p)

Figure 6 introduces a simple method to estimate a realistic value of p. Using same scale in axes is also required.

Procedure: Point C is a point on S-axis with coordinates of $(0, S_m/2)$. Draw a line MC to cross the S versus w curve at point D. Distance along X-axis between M and D is equal to p ($M'D'$).

In this case, $M'D'$ = 31%-19% = 12%, so p is equal to 12%.

Create a Family of Compaction Curves

The shape factor provides an opportunity to predict a family of compaction curves which are prepared using the same compaction method but using different compactive efforts.

As discussed by Li and Sego (1998), the degree of saturation at the point CST (S_{CST}) for different compactive efforts appeared to remain constant. This assumption is based on observations from the results of dynamic and kneading compaction tests presented by Faure (1994) and Li and Sego (1998). This concept is also partly supported by the observation reported in the literatures that the degree of saturation at the point of maximum dry density is generally the same when a soil is subjected to different compaction efforts (Lambe 1958, and Seed et al. 1960).

From the definition of CST, S_{CST} can be derived from Equation 5 as follows:

$$S_{CST} = S_m - kp \cdot \left(\frac{n+1}{n-1}\right)^{\frac{n+1}{n}} \tag{8}$$

As discussed, the soil shape factor is the same for a given soil when subjected to a given compaction method. The maximum degree of saturation does not generally change for a given fine-grained soil. Based on these assumptions, a relationship is derived with surprising simplicity.

$$kp = \frac{S_m}{w_m} \cdot p = const \tag{9}$$

Given Equation 9, one only needs a single test to create a whole compaction curve for the same soil subjected to different compactive efforts.

The easiest compaction test can be carried out on an air-dried sample. Li and Sego (1998) found a logarithm linear relationship between compaction effort and γ_{dd} during the kneading compaction test. This once again is partly supported by the fact that there is an approximate logarithm linear relation between compaction effort and maximum dry density (Hausmann 1990).

The procedure to create a family of compaction curves is described as follows:

- Obtaining all four parameters from a complete compaction curves as previously described.
- For fine-grained soil, assume the shape factor (n) and maximum degree and saturation (S_m) remain constant for all levels of compactive effort.
- Carrying out a compaction test on an air-dried sample to measure γ_{dd} at a given compaction energy and calculate k and w_m using Equations 1 and 2.
- From Equation 9 establish the index of compactable moisture range (p).
- Using Equation 5 or 6 to calculate the compaction curves for the different levels of compactive effort.

Figure 7 shows a family of compaction curves for Pleistocene Lacustrine clay (P_l) compacted at the University of Alberta using the kneading compaction method. The soil was obtained from overburden soils at Syncrude Canada Ltd., Fort McMurray, Alberta. The soil was air-dried and then had a prescribed amount of water added. The compaction test was conducted using a CS 1000 Electronic-Hydraulic Kneading Compactor which is described in ASTM Standard Practice for Preparation of Bituminous Mixture Test Specimens by Means of California Kneading Compactor (D 1561-92). At first, the samples were tamped using three layers of soil under a foot pressure of 700 kPa with 50 applications per layer. The parameters of the curve were obtained by the regression method using Equation 6, where S_m is 88.8%, w_m is 27.2%, n is 11.38 and p is equal to 11.3%. These parameters were then used to create the other curves at different compactive energy levels.

Additional results of laboratory compaction tests carried out at different compaction energy show good agreement with the predicted family of curves (Fig. 7). The pneumatic compaction test is a little different from the kneading compaction test because it has 100% coverage and no kneading effects, but its results also show a similar shape in compaction curve.

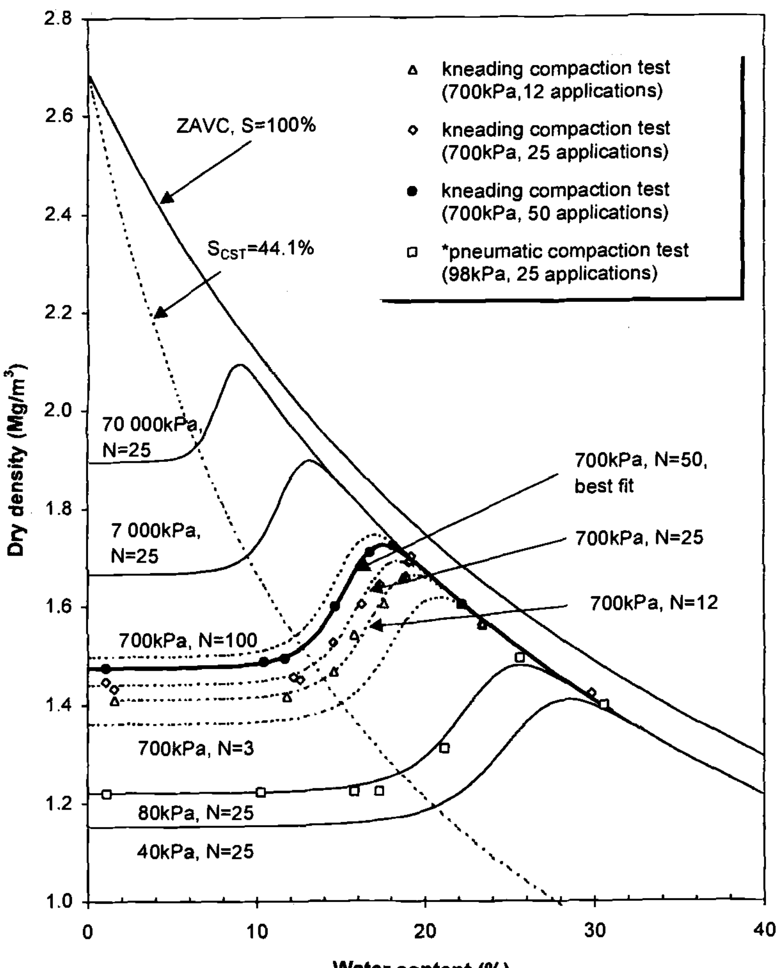

Fig. 7 - *A family of compaction curves of glacial lacustrine clay under different compactive efforts (energy)*

Conclusions

An equation for use with fine-grained soil based on the relationship between the degree of saturation and water content is proposed. The equation is dimensionless and can be used to describe the complete compaction curves for a fine-grained soil. The parameters in the equation are related to the soil properties and compaction method and are easily determined from a complete compaction curve. The equation has the ability to create a family of compaction curves that result from use of different compactive efforts. The suggested equation shows excellent comparison between laboratory test data and the predicted curves.

Acknowledgments

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Robert Scavuzzo[1]

Construction Quality Control Testing of Compacted Fills: Optimum Moisture-Density Values

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Reference: Scavuzzo, R., **"Construction Quality Control Testing of Compacted Fills: Optimum Moisture-Density Values,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbots, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Construction quality control testing is critical in ensuring that compacted fills are properly placed. One element in the successful implementation of compacted fill placement is the appropriate assignment of optimum moisture-density values for varying fill material types encountered during construction. For decades the U.S. Bureau of Reclamation (USBR) has acquired laboratory moisture-density test data for a variety of material types from the 17 western states in which the USBR operates. A comparison between optimum moisture-density data published by the USBR is made with laboratory test results obtained from years of quality control testing of compacted fills on commercial projects throughout the Denver-Metro area. Optimum moisture content-maximum dry density curves are presented which provide a visual representation of the variability in the data which can be anticipated for each soil type evaluated. This data provides confirmation of the USBR database and can be used in estimating appropriate moisture content-dry density values, evaluating the validity of laboratory compaction test results obtained, as well as assessing the state of compaction of in-place fill material.

Keywords: quality control testing, compacted fills, optimum moisture-density, construction control

Fill Placement Process

Ideally, material from the borrow area to be used for fill placement is identified by the earthwork contractor and submitted to the laboratory for testing. Atterberg limits testing and gradation analysis are performed to ensure that the material meets the project specification requirements for its proposed use. Once approved, optimum moisture-density relationships are established and fill placement begins. Fill placement is monitored at the testing frequency as required by project specifications. Borrow area materials do not change during the fill placement operation such that the optimum

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moisture-density relationship initially established is valid throughout the entire fill placement process.

The reality of the fill placement process is often very different from the ideal scenario presented above. On project sites that require large quantities of import material, multiple borrow areas are often used to obtain the required amount of material. Experience has shown that on many jobs up to a dozen or more borrow areas are selected by the earthwork contractor, and material identified as coming from a single source changes during the fill placement process such that the optimum moisture-density relationship initially established is no longer applicable.

Quality control field technicians are routinely called to a jobsite after fill placement has begun and are directed by the contractor to piles of material that are "representative" of material that has already been placed. Field technicians are then asked to perform moisture-density tests for "information only" to assess the state of compaction of the material in-place so that work can continue with some degree of confidence that moisture-density requirements are being met prior to obtaining laboratory moisture - density relationships for the variety of material being placed.

Knowledge of anticipated optimum moisture content-dry density values can be valuable information for both the earthwork contractor and quality control field technician to help ensure that fill is being placed per project specifications (Monahan 1986).

For decades the U.S. Bureau of Reclamation (USBR) has acquired laboratory moisture-density test data for a variety of material types from the 17 western states in which the USBR operates. This data, published in the USBR Design of Small Dams Manual (USBR 1987), has been used as an information source for moisture-density relationships.

Presented are optimum moisture content-dry density averages and anticipated ranges for six soil types established from over 1400 standard laboratory compaction tests performed in the CTC-Geotek laboratory. Optimum moisture content-dry density curves for the soil types evaluated are also presented. This data provides confirmation of the USBR database and can be used in estimating appropriate moisture content-dry density values, evaluating the validity of laboratory test results obtained as well as assessing the state of compaction of in-place fill material.

Laboratory Test Results

Atterberg limits obtained from laboratory tests performed in accordance with ASTM Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (D 4318) for over 1400 samples were used to classify the soil in accordance with ASTM Test Method Classification of Soils of Engineering Purposes (Unified Soil Classification System) (D 2487). Soils resulting in a Unified Soil Classification System (USCS) dual or borderline classification were not incorporated into the data presented.

Optimum moisture content and maximum dry density relationships were obtained in accordance with ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3)) (D 698) were compiled for the 1427 soil samples classified.

Average, standard deviation, minimum, and maximum optimum moisture content and dry density values for the six USCS soil types compiled are summarized in Table 1. Values obtained in the CTC-Geotek laboratory are compared to the results published in the United States Bureau of Reclamation (USBR) Design of Small Dams Manual (USBR, 1987) in Table 1. Optimum moisture content-maximum dry density values for each of the USCS soil types evaluated are presented in Figures 1 through 6 providing a visual representation of the variability in the data which can be anticipated for each soil type.

Fat Clay - CH

Material which classified as Fat Clay (CH) in the USCS represents approximately 9.5% of the total CTC-Geotek sample population of 1427 and approximately 7% of the total USBR sample population of 497. Average values of average optimum moisture content and maximum dry density of 24.7% and 97.3 lb/ft^3 (1559 kg/m^3) were obtained, respectively, from the 135 laboratory Standard compaction tests performed.

A review of the data presented in Table 1 for the laboratory test results obtained from the 135 tests performed in the CTC-Geotek laboratory to the 36 tests published by the USBR indicates a good correlation between optimum moisture content and maximum dry density values obtained from the two data sets. Optimum moisture content-dry density values for the 135 tests performed on samples classified as Fat Clay (CH) in the USCS are presented in Figure 1.

A further statistical review of the data summarized in Table 1 indicates that approximately 73% of the 135 laboratory Standard compaction tests performed were within plus or minus one standard deviation of the average moisture content and average maximum dry density values obtained.

Lean Clay - CL

Material which classified as Lean Clay (CL) in the USCS represents approximately 38% of the total CTC-Geotek sample population of 1427 and approximately 44% of the total USBR sample population of 497. Average values of optimum moisture content and maximum dry density of 19.5% and 105.5 lb/ft^3 (1690 kg/m^3) were obtained, respectively, from the 540 laboratory Standard compaction tests performed.

A review of the data presented in Table 1 for the laboratory test results obtained from the 540 tests performed in the CTC-Geotek laboratory to the 221 tests published by the USBR indicates a slightly lower average maximum dry density and a corresponding slightly higher average optimum moisture content for the CTC-Geotek data set. Optimum moisture content-dry density values for the 540 laboratory compaction tests performed on samples classified as Lean Clay (CL) in the USCS are presented in Figure 2.

A further statistical review of the data summarized in Table 1 indicates that approximately 71% of the 540 laboratory Standard compaction tests performed were within plus or minus one standard deviation of the average moisture content and average maximum dry density values obtained.

Table 1 - *Average Laboratory Standard Compaction Values*

USCS Soil Type	Maximum Dry Density lb/ft^3 (kg/m^3) CTC GEOTEK	USBR[1]	Optimum Moisture Content (%) CTC GEOTEK	USBR[1]	Number of Tests CTC GEOTEK	USBR[1]	Values Listed
	97.3 (1559)	95.3 (1527)	24.7	25.0	135	36	Average of all values
CH	5.0 (80)	6.6 (106)	3.5	5.4			Standard deviation
Fat Clay	83.9 (1344)	82.3 (1318)	17.0	16.6			Minimum value
	109.2 (1749)	107.3 (1719)	36.5	41.8			Maximum value
	105.5 (1690)	109.3 (1751)	19.5	16.7	540	221	Average of all values
CL	5.0 (80)	5.5 (88)	3.0	2.9			Standard deviation
Lean Clay	90.8 (1454)	90.0 (1442)	12.0	6.4			Minimum value
	123.0 (1970)	121.4 (1945)	29.5	29.2			Maximum value
	119.2 (1909)	116.6 (1868)	12.5	12.5	376	123	Average of all values
SM	6.7 (107)	8.9 (143)	3.1	3.4			Standard deviation
Silty Sand	96.6 (1547)	92.9 (1488)	7.0	6.8			Minimum value
	136.4 (2185)	132.6 (2124)	25.5	25.5			Maximum value
	115.4 (1849)	118.9 (1905)	14.7	12.4	342	73	Average of all values
SC	6.6 (106)	5.9 (95)	3.3	2.3			Standard deviation
Clayey Sand	92.1 (1475)	104.3 (1671)	8.5	6.7			Minimum value
	130.6 (2092)	131.7 (2110)	27.3	18.2			Maximum value
	103.2 (1653)	103.3 (1655)	21.3	19.7	24	39	Average of all values
ML	10.9 (175)	10.4 (167)	5.8	5.7			Standard deviation
Silt	87.5 (1402)	81.6 (1307)	8.8	10.6			Minimum value
	129.7 (2078)	126.0 (2018)	29.7	34.6			Maximum value
	86.5 (1386)	85.1 (1363)	34.2	33.6	10	5	Average of all values
MH	3.8 (61)	2.3 (37)	3.3	1.6			Standard deviation
Elastic Silt	78.4 (1256)	82.9 (1328)	28.8	31.5			Minimum value
	92.3 (1479)	89.0 (1426)	41.0	35.5			Maximum value
				Total Samples:	1427	497	

[1] From Reference (USBR 1987)

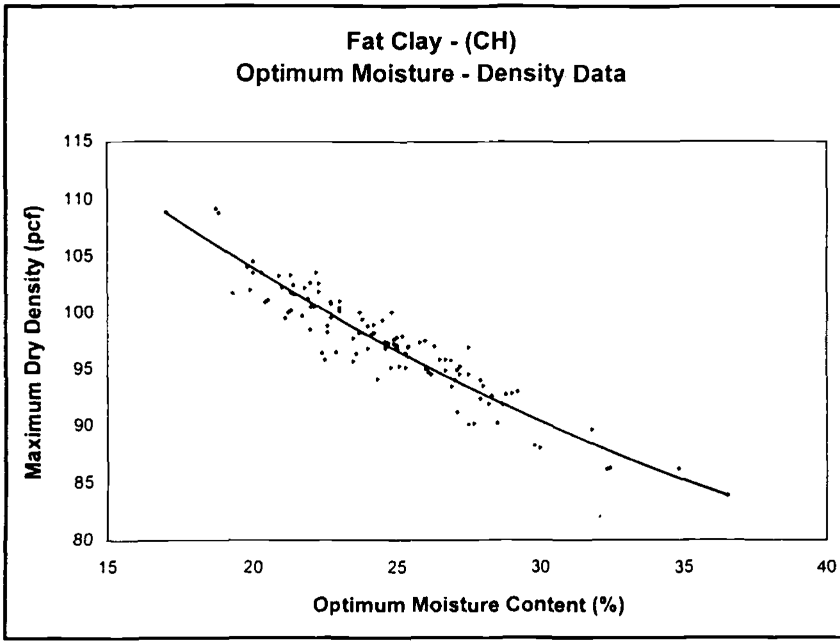

Figure 1 - *Fat Clay (CH) Optimum Moisture-Density Data*

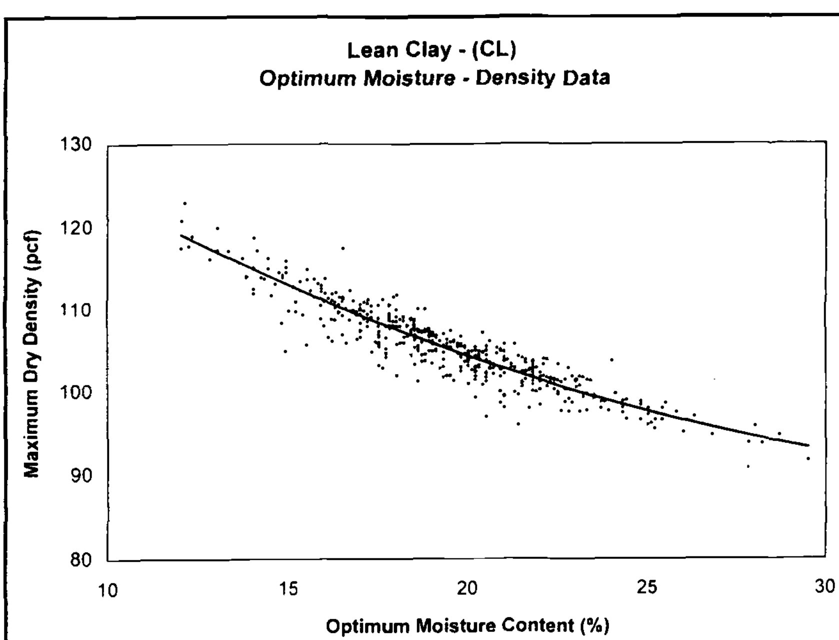

Figure 2 - *Lean Clay (CL) Optimum Moisture-Density Data*

Silty Sand - SM

Material which classified as Silty Sand (SM) in the USCS represents approximately 26% of the total CTC-Geotek sample population of 1427 and approximately 25% of the total USBR sample population of 497. Average values of optimum moisture content and maximum dry density of 12.5% and 119.2 lb/ft^3 (1909 kg/m^3) were obtained, respectively, from the 376 laboratory Standard compaction tests performed.

A review of the data presented in Table 1 for the laboratory test results from the 376 tests performed in the CTC-Geotek laboratory to the 123 tests published by the USBR indicates a slightly higher average maximum dry density value obtained for the CTC-Geotek data set. An average optimum moisture content of 12.5% was obtained from both the CTC-Geotek and USBR data set. Optimum moisture content-dry density values obtained from the 376 laboratory standard compaction tests performed on samples classified as Silty Sand (SM) in the USCS are presented in Figure 3.

A further statistical review of the data summarized in Table 1 indicates that approximately 79% of the 376 laboratory standard compaction tests performed were within plus or minus one standard deviation of the average optimum moisture content obtained, and approximately 70% were within plus or minus one standard deviation of the average maximum dry density obtained.

Clayey Sand - SC

Material which classified as Clayey Sand (SC) in the USCS represents approximately 24% of the total CTC-Geotek sample population of 1427 and approximately 15% of the total USBR sample population of 497. Average values of optimum moisture content and maximum dry density of 14.7% and 115.4 lb/ft^3 (1849 kg/m^3) were obtained, respectively, from the 342 laboratory Standard compaction tests performed.

A review of the data presented in Table 1 from the 342 tests performed in the CTC-Geotek laboratory to the 73 tests published by the USBR indicates a slightly lower average maximum dry density and a corresponding slightly higher average optimum moisture content for the CTC-Geotek data set. Optimum moisture content-dry density values obtained from the 342 laboratory standard compaction tests performed on samples classified as Clayey Sand (SC) in the USCS are presented in Figure 4.

A further statistical review of the data summarized in Table 1 indicates that approximately 75% of the 342 laboratory standard compaction tests performed were within plus or minus one standard deviation of the average optimum moisture content obtained, and approximately 70% were within plus or minus one standard deviation of the average maximum dry density obtained.

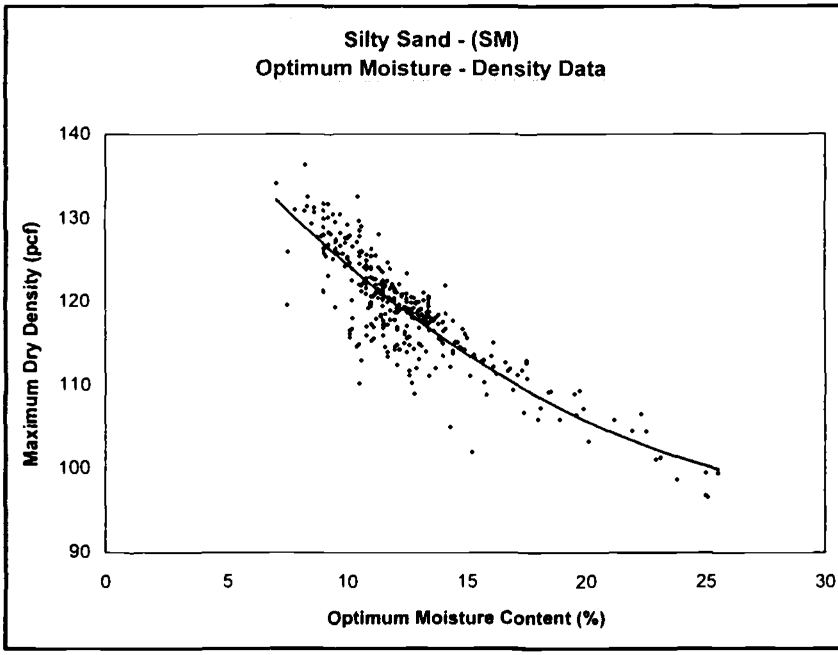

Figure 3 - *Silty Sand (SM) Optimum Moisture-Density Data*

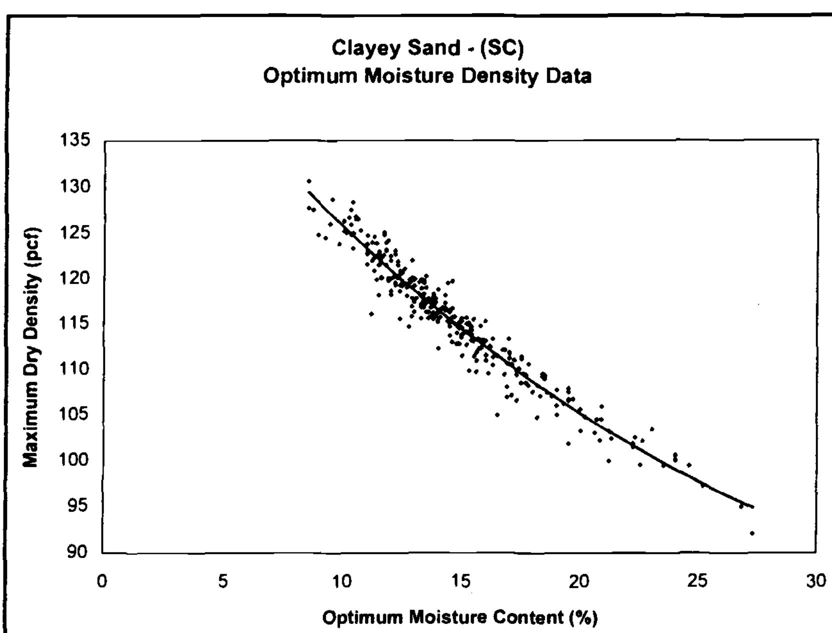

Figure 4 - *Clayey Sand (SC) Optimum Moisture-Density Data*

Silt - ML

Material which classified as Silt (ML) in the USCS represents approximately 2% of the total CTC-Geotek sample population of 1427 and approximately 8% of the total USBR sample population of 497. Average values of optimum moisture content and maximum dry density of 21.3% and 103.2 lb/ft^3 (1653 kg/m^3) were obtained, respectively, from the 24 laboratory Standard compaction tests performed.

A review of the data presented in Table 1 from the 24 tests performed in the CTC-Geotek laboratory to the 39 tests published by the USBR indicates a good correlation between average optimum moisture content and average maximum dry density values obtained from the two data sets. Optimum moisture content-dry density values obtained from the 24 laboratory Standard compaction tests performed on samples classified as Silt (ML) in the USCS are presented in Figure 5.

A further statistical review of the data summarized in Table 1 indicates that approximately 71% of the 24 laboratory Standard compaction tests performed were within plus or minus one standard deviation of the average optimum moisture content and average maximum dry density obtained.

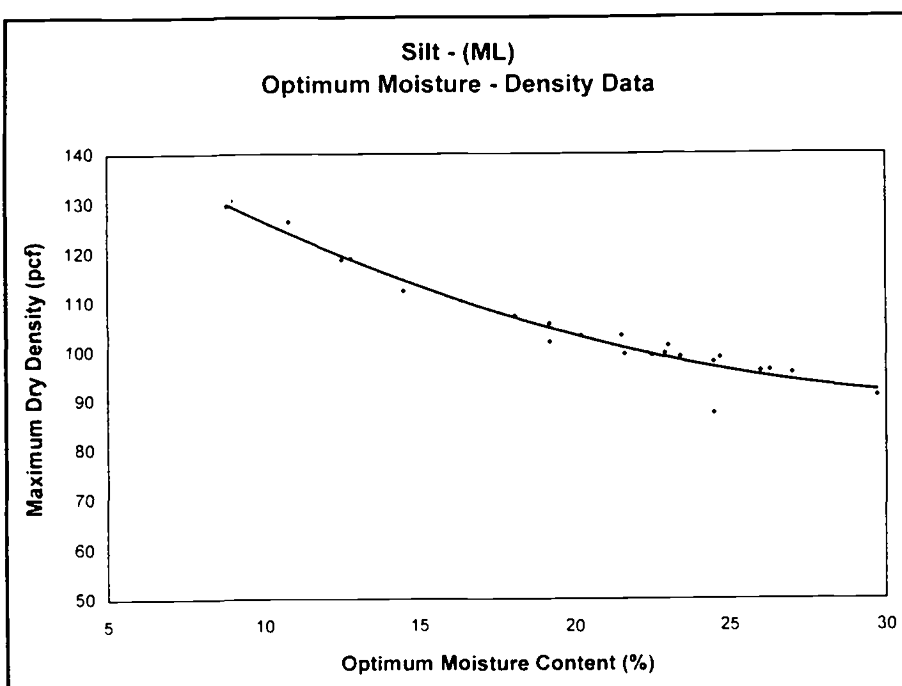

Figure 5 - *Silt (ML) Optimum Moisture-Density Data*

Elastic Silt - MH

Material which classified as Elastic Silt (MH) in the USCS represents less than 1% of the total CTC-Geotek sample population of 1427 and approximately 1% of the total USBR sample population of 497. Average values of optimum moisture content and maximum dry density of 34.2% and 86.5 lb/ft^3 (1386 kg/m^3) were obtained, respectively, from the 10 laboratory Standard compaction tests performed.

A review of the data presented in Table 1 from the 10 tests performed in the CTC-Geotek laboratory to the 5 tests published by the USBR indicates a good correlation between average optimum moisture content and average maximum dry density values obtained from the two data sets. Optimum moisture content-dry density values obtained from the 10 laboratory Standard compaction tests performed on samples classified as Elastic Silt (ML) in the USCS are presented in Figure 6. The shape of the curve provided in Figure 6 is most likely due to the minimal data in the Elastic Silt (MH) data population and not a true material type attribute.

A further statistical review of the data summarized in Table 1 indicates that approximately 80% of the 10 laboratory Standard compaction tests performed were within plus or minus one standard deviation of the average optimum moisture content and average maximum dry density obtained.

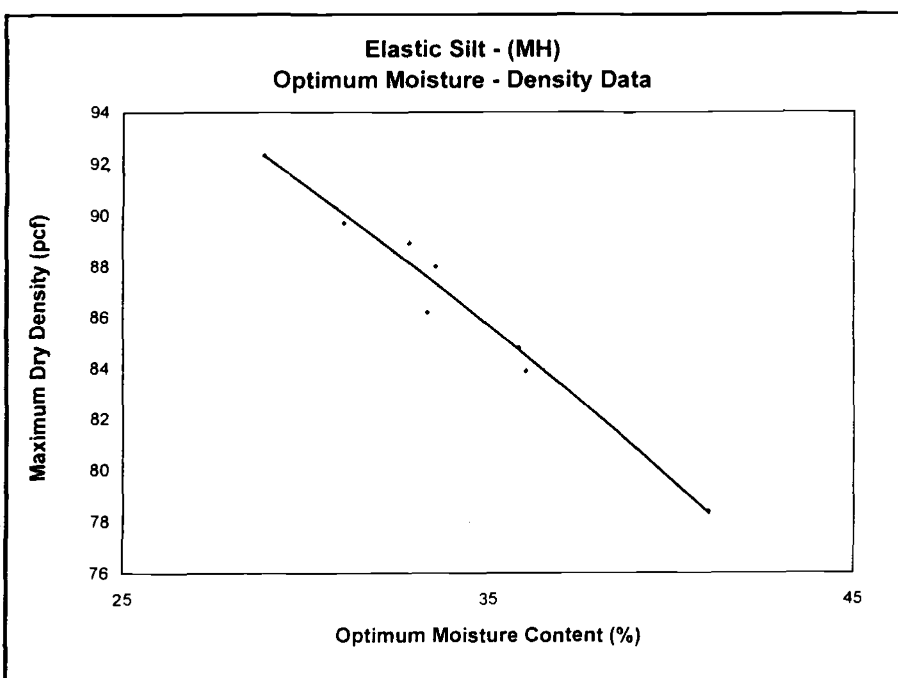

Figure 6 - *Elastic Silt (MH) Optimum Moisture-Density Data*

Optimum Moisture-Density Data Usage

Anticipated values of average optimum moisture content and maximum dry density can be obtained for the six soil types evaluated from the data summarized in Table 1. In general, comparison between the CTC-Geotek laboratory compaction test results and the published USBR data indicates a good correlation between the two data sets. These average values can be used by earthwork contractors, as well as quality control technicians in estimating appropriate moisture content-dry density values prior to obtaining laboratory compaction test results.

Optimum moisture content-maximum dry density curves shown in Figures 1 through 6 provide a visual representation of the variability in the data which can be anticipated for each soil type. In addition, these curves provide a correlation between optimum moisture content and a corresponding maximum dry density range for each soil type. These relationships can be used by laboratory managers, engineers, and earthwork contractors to evaluate the validity of laboratory compaction test results obtained. In the example illustrated in Figure 7, fill material is classified in the laboratory as a Lean Clay (CL) and laboratory compaction test results indicates an optimum moisture content of 20% and a maximum dry density of 104.5 lb/ft^3. Based upon the maximum and minimum values contained in the data set at an optimum moisture content of 20%, an anticipated maximum dry density range between approximately 100.5 lb/ft^3 to 107.5 lb/ft^3 can be established, verifying the laboratory value of 104.5 lb/ft^3 obtained.

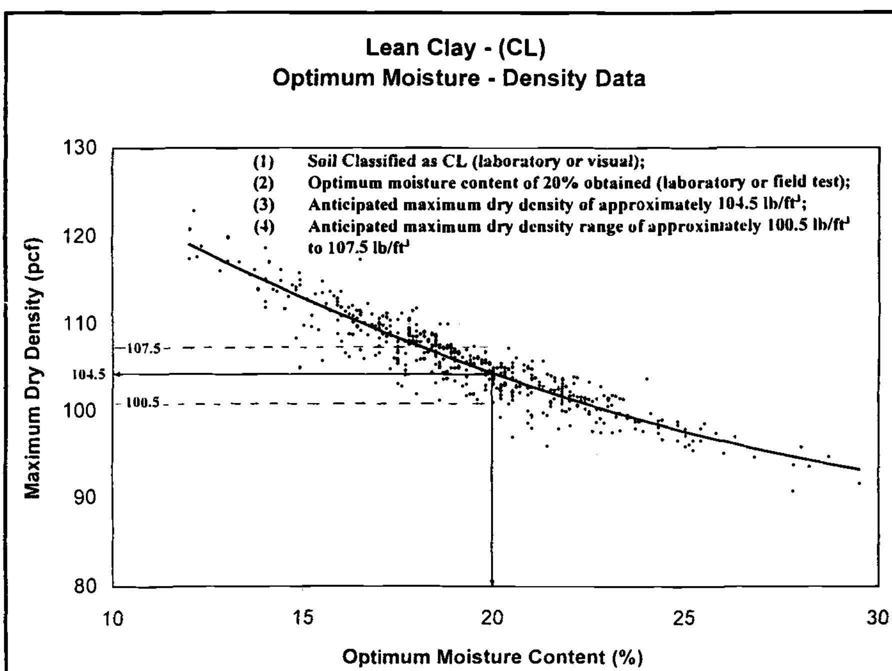

Figure 7 - *Optimum Moisture-Density Usage Example*

In this way, compaction test results received from the laboratory can be compared to the optimum moisture content-maximum dry density relationship anticipated from the appropriate material type curve to verify their validity.

Optimum moisture content-maximum dry density correlations provided in Figures 1 through 6 can also be used to assess the state of compaction of in-place fill material prior to obtaining laboratory compaction test results. In the example illustrated in Figure 7, fill material is visually classified as a Lean Clay (CL) by the quality control field technician. A visual assessment is also made of the placement moisture condition, i.e., below, at, or above optimum. Nuclear density gauge testing is performed indicating an in-place moisture content of 20% which, for this example, was estimated to be approximately the optimum moisture content for the fill material being evaluated. The corresponding in-place dry density obtained from nuclear density gauge testing can be compared to optimum moisture content-maximum dry density relationship anticipated from the appropriate material type curve. From this analysis, guidance can be provided to the earthwork contractor such that fill placement can continue with some degree of confidence that moisture-density requirements are being met prior to obtaining laboratory moisture-density data for the material being placed.

Summary

The average optimum moisture content-dry density values and/or the soil type curves presented are not intended to replace the need for site specific laboratory testing to establish appropriate moisture-density relationships as required by project specifications. The data presented provides confirmation of the USBR database and can be used to estimate appropriate optimum moisture content-dry density values, evaluate the validity of laboratory compaction test results obtained, as well as assessing the state of compaction of in-place fill material prior to obtaining laboratory compaction test results.

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Christopher D. Hardin[1] and Greg D. Icenhour[2]

Historical Perspectives on Earthwork Engineering and Creating a Passion for Earthwork Excellence

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Reference: Hardin, C. D., and Icenhour, G. D., **"Historical Perspectives on Earthwork Engineering and Creating a Passion for Earthwork Excellence,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384*, D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Earthwork engineering practices have been reasonably well defined since the early 1900s. Knowledge of the historical approach to solving subsurface soil problems can be gained by reviewing the writings of the pioneers of soil mechanics. Of particular interest are the writings of Karl Terzaghi, the father of modern soil mechanics. Application of Professor Terzaghi's attitude towards consulting engineering will result in a passion for earthwork engineering excellence. The authors have observed the practical application of earthwork engineering principles in a manner which provides quality earthwork projects at a reasonable cost.

Keywords: earthworks, engineering, subsurface soil problems

Introduction

It is a human tendency when problems exist to develop new written methods and to enforce these methods by carefully written contracts or lawsuits. Although this approach typically provides a short term sense of justice it frequently does little to correct the problem. This paper addresses a similar tendency which is increasingly influencing the field of earthwork engineering. As our society has become more litigious, the cost of these failures becomes more expensive. The impact of our current approach to earthwork engineering is obvious in the number of lawsuits, damaged structures and an overall sense of failure that frequently exists during or after the completion of many large earthwork projects. The purpose of this paper is to describe several problems associated with

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current practices in earthwork engineering and attempt to provide positive methods for creating a passion for earthwork engineering excellence.

The examples used in this paper to illustrate these issues are based on the author's design, laboratory and field experience on this and other earthwork projects. Over the past 14 years the authors have learned to "live with the soil" in an effort to understand its fascinating and ever changing characteristics. The most important lesson learned is that soil and nature frequently create situations that cannot be reliably evaluated using solely analytical methods. Non-homogeneous soil conditions tend to be the rule instead of the exception. The changing conditions provide constant challenging situations for the geotechnical engineer. As frequently described by the early practitioners of soil mechanics the constant changing nature of soil continually places us "at the borderline between science and art."

During our evaluation and correction of earthwork engineering problems on over 40 sites in the past 10 years, we have noticed a related, but non-technical problem which has equal and potentially greater influence on the successful completion of projects. This problem is the influence of simple human errors on the practice of earthwork engineering. Currently the methods and procedures in the field of soil mechanics and earthwork engineering are reasonably well defined and these procedures have eliminated many of the uncertainties that existed during the creation of the core principles in the early and mid 1900's. Unfortunately, the number of defeats due to non-technical components and need for improvement appears to have actually increased in recent years. The increase in human error appears related to the reality that many geotechnical engineers have lost the ability to "think with the hips" in areas where non-standard solutions are required. As we have increased our technical knowledge and become better educated, it appears that we have lost appreciation for regular and consistent practical application of principles to ensure that we understand their deeper meaning.

In particular, over the last 7 years our firm has noticed that the primary cause for failure in the area of earthwork engineering is often a simple misunderstanding of the principles that guided the founding fathers of soils mechanics and/or an inability to communicate these principles to the field and construction personnel. Most often it appears that our response as engineers is to write more voluminous or perfected test methods instead of working on effectively applying the test methods we already have. As expected, this has resulted in continued errors and what has become an almost comical application of geotechnical principles and test methods by engineers and technicians who simply do not understand what they are doing. The longer the tendency exists to talk about our problems instead of going to the field to solve them, the more young engineers we will train to think in the same erroneous patterns until eventually we will be in danger of losing the culture that makes quality earthwork possible. Our concern is that eventually engineers who have the moral courage to apply the principles of sound soil mechanics and earthwork engineering will become suspected as "high risk" and "bad engineers" because they fail to follow the habits of engineers who have not spent enough time "living with the soil". Our response to these errors is to make a concerted effort to return to the original and simplified principles of earthwork engineering by reviewing the writings of Karl Terzaghi and by making an attempt to apply these principles daily using modern methods of team management and peer review.

A Review of the Writings of Karl Terzaghi

A review of the writings of Karl Terzaghi has proved to be both inspiring and discouraging when compared to the current state of the practice of geotechnical engineering. The following sections are quotes from several speeches we found to be the most enlightening:

> *"A consultant is a person who is supposed to know more about a subject under consideration than his client. Once an engineer has acquired a reputation for superior knowledge and discovers that there is a demand for his services, his future career depend on what he expects to get out of life. If he longs for financial success and social prestige, he will find that his aims can hardly be satisfied without creating an organization. Once the organization exists he becomes a slave to it. His income increases, but so do his worries. Sometimes he has sleepless nights because he does not know how to handle all the orders rained into his lap, and at other times, because his overhead charges exceed his income. In any event, the Tax Collector sees to it that his income does not assume staggering proportions. He may still believe that he is a consultant, but in reality he has turned into a business man and executive, equipped with all the prerequisites for stomach ulcers.*
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> *On the other hand, if he derives his principal satisfaction from practicing the art of engineering, he will desist from establishing an organization and concentrate all his efforts on broadening his knowledge in the field of his choice. In order to be successful in this pursuit he must be not only willing but eager to spend at least half of his time on unprofitable occupations such as research or the digest of his observational data. Therefore, his money making capacity remains limited, but in exchange he has fewer worries and retains his freedom of action. This is the type of occupation which has turned out to agree with my disposition."*
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> (Terzaghi, 1958)
>
> *"I realized, in the course of the years, that the knowledge accumulated in a human brain has no practical value unless its owner has the moral courage to use*

it as the basis for decisions. Last but not least, I became more and more impressed by the importance of never missing an opportunity to find out, by direct observation, the difference between forecasts and the real developments."
(Terzaghi, 1957)

On acquiring the ability to "think with the hips":

"*... this capacity can only be acquired by first absorbing with the head everything that is to be known and then to get it into the subconscious by continuously practicing it.*"

"*To acquire competence in the field of earthwork engineering one must learn to live with the soil. One must love it and observe its performance not only in the laboratory but also in the field, to become familiar with its manifold properties.*"
(Terzaghi, 1957)

In addition to the above quotations, our review of the early writings of Professor Terzaghi reveals him as a highly independent and innovative thinker. An article by Laurits Bjerrum (Bjerrum, et. al., 1960) indicates the following key aspects of Terzaghi's method of working:

1. He studies the subsurface geology and topography and formulates a hypothesis regarding the subsoil conditions.

2. It is characteristic that he almost never prepares a detailed program for his subsoil explorations in advance and he maintains complete flexibility to develop the exploration step-by-step.

3. No essential detail escapes his attention and he knows by heart the configuration of the terrain the results of the test borings and all the data concerning the groundwater conditions.

4. Every single part of the work he does himself. He rarely uses assistants on his jobs and he prepares all his own drawings because it keeps him in personal contact with the job.

The writings of Professor Terzaghi give us a glimpse of the past that can provide great hope for the future. His definition of what it means to be a consultant provides unique insight into what may be the root of the problem. In our pursuit to "publish or perish" or "be billable or unemployed" we may have unintentionally abandoned our greatest joy and satisfaction.

"The greatest joy for an engineer involved in the construction of earthwork projects is the successful completion of projects in accordance with good engineering principles."

C. Hardin, 1999[1]

As family men and company owners who do their best work as part of a team, we respectfully disagree with Mr. Terzaghi that an organization will automatically tend to enslave the owner who creates it. As we look to a new millennium in earthwork engineering there are great possibilities if we seek to create small, efficient, and team-oriented organizations focused on providing excellence in earthwork engineering. It is our experience that this type of team-oriented environment, with a pre-planned limitation on financial compensation, can maintain the independence necessary to be a consultant in the spirit of Terzaghi's writings. The following provides practical examples on how these principles can be best applied.

Project Case History

Over the past ten years as consultants working in the earthwork engineering field we have observed numerous defeats and victories which have taught us and continue to teach us what works and does not work. Our company has completed earthwork engineering projects and field density testing services on over 50 landfill projects, 2 speedways, and over 50 residential development projects. The following sections provide a case history of a project we found to be most fascinating and noteworthy. We have purposely omitted the name of the project and project consultants to focus our discussion on the educational aspects of the project.

Speedway Repair Project

Our firm's work on this speedway project involved the development and implementation of a remedial action plan for controlling subsurface drainage, grout stabilizing areas that experienced subsidence, and controlling heave at several locations on the racetrack. Our firm was part of the original design team that developed the remediation plan. We also served as the primary earthwork engineering firm working with the contractor to install the remedial design in 35 days or less. The project consisted of over 20 different components ranging from altering the track geometry to the installation of an innovative pavement drainage system that would maintain stability on 22 degree slopes. One of the most challenging parts of the project is that the $4 million corrective action had to be

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[1]Hardin, C., GCI, Charlotte, NC, personal communication with Greg Icenhour, GCI, Charlotte, NC, May, 1999

designed and installed in less than 40 days to accommodate an upcoming televised race event.

The most significant geotechnical aspect of the project consisted of identifying and controlling over 40 groundwater seeps that originally manifested during a nationally televised racing event. To understand the cause of the seeps and to categorize a potential flow pattern, our firm developed a seep location map. The seep location map was also correlated to construction materials testing data that was prepared during the initial race track construction. Our firm suspected improperly compacted fill materials were contributing to the problem since the project was constructed in a very rapid time period with over 1 million cubic yards placed in less than 8 months.

One of the first things our firm looks for when investigating a problem earthwork project is data that appears "too perfect". The data developed by the testing firm for this speedway project had very few failing and subsequently passing retest locations. Numerous tests were considered "passing" by the geotechnical engineer even though they failed to meet the moisture criteria. Review of these data indicated a high probability that the test results falsely indicated acceptable structural fill in areas which were actually filled with sub-standard structural fill materials.

Our visual observations and data review as the racetrack was excavated for repair confirmed our hypothesis. The fill material was placed below the specified compaction criteria at the two areas exhibiting major seeps. The review of the technician's test reports also indicated that attempts were made to mix potentially expansive clay materials with sand seams to achieve a low plasticity fill material beneath the banked turns of the speedway. There was no control of the borrow source by a qualified geotechnical engineer and it appeared that equipment operators were the only ones classifying the soils, using color as the primary identification method. Expansive clay mixed with intermittent sand seams placed in the speedway embankments allowed water to saturate the expansive clay, thereby softening and/or causing swell conditions beneath the pavement.

In addition to improperly placed speedway embankment fill material, there also appeared to be a problem with the interpretation and use of the groundwater information developed during the initial geotechnical evaluation. The initial geotechnical evaluation indicated groundwater 10 to 12 feet above the elevation of the bottom of the track surface at two locations. Rainfall data three months prior to drilling indicated average to above average rainfall. During placement of the embankment fill in the seep areas, the construction testing staff did not observe the presence of groundwater since it was one of the driest seasons on record. This change in conditions and the lack of a subsurface drainage system allowed groundwater to flow through the sand seams and surface on the race track along the pavement seams. Based on conversations with the geotechnical engineer of record and one of the main technicians at the site during construction, it is our understanding that there was no coordination between the design and construction testing staff because they were managed as two different sections and profit centers within the same company.

The remedial activities for the seepage problem involved development of an innovative steep slope drainage system that combined pavement and landfill closure technologies. The drainage section consisted of a cement treated, coarse aggregate base of AASHTO No. 57 stone. The combination of materials and technologies allowed the installation of a high volume drainage system on 22 degree slopes. For slopes flatter than 11 degrees, a conventional composite geonet system was installed beneath the aggregate base course. Both the cement treated aggregate and geonet systems were routed to a french drain and piping system connected to the speedway's storm water collection system. It is estimated that removing the racetrack and installing the drainage system cost approximately $1.5 million.

In hindsight, it would be easy to think that both the speedway owner and geotechnical engineer should have had the foresight to avoid the problems that were encountered on this project. It was, after all, a very expensive fix that caused great hardship for the owner and no small amount of fear for the geotechnical consultant. After reviewing the data, the scope of services of the contractor and engineer, and the time constraints of the project there may have been very little that could have been done to avoid the problems encountered. As an engineer who has also struggled with numerous difficult projects it appears that one of Terzaghi's Rules of Professional Activities and Relationships most appropriately expresses our view of the situation:

> *"Engineering is a noble sport which calls for good sportsmanship. Occasional blundering is part of the game. Let it be your ambition to be the first one to discover and announce your blunders. If someone else gets ahead of you, take it with a smile and say, "thank you for your interest". Once you begin to feel tempted to deny your blunders in the face of reasonable evidence you have ceased to be a good sport. You are already a crank and a grouch."*
>
> (Bjerrum, et.al., 1960)

> *" It is not good sportsmanship to repeat the same mistakes over and over again. Let us all learn from this project."*
>
> C. Hardin, 1999[2]

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[2]Hardin, C., GCI, Charlotte, NC, personal communication with Greg Icenhour, GCI, Charlotte, NC, May, 1999

Practical Ideas For Creating A Passion for Excellence In Earthwork Engineering

So that the recommendations for creating a passion for excellence in earthwork engineering did not become too theoretical, we found it is necessary to focus on the main defeats or problems we have encountered or observed in the past 12 years. Based on our experience the main causes of problems in earthwork engineering and construction materials testing are as follows:

- Improper Proctor selection with almost no one-point or field Proctor confirmation -- most technicians and engineers pick the Proctor curve that makes the data pass or look proper;

- Lack of review by a senior geotechnical engineer who understands how to use the Proctor curve and the proper implementation of field density methods;

- Limited control of the quality of the material coming to the site by qualified personnel who have the ability to classify the soils and stop the equipment if necessary;

- Almost no oven dry confirmation of the nuclear density moisture contents as required by ASTM.

- Almost no cross checking of the nuclear field density utilizing large drive cylinders or sand cones as required by ASTM.

- An overall attitude that field density testing is "technicians" work and beneath the intellectual ability of engineers. It is interesting for us to observe that earthwork engineering is almost never mentioned during the completion of large earthwork construction projects. Most of the work on earthwork construction projects is done by testing firms or soils testing laboratories. We may all be getting the quality of service we are expecting.

- Little or no coordination by most companies between the geotechnical design engineer and field technician. On projects where our firm has been called to fix problems we have yet to meet a technician or a field engineer who has read and understood a copy of the geotechnical design report prepared by "others".

As you can see none of the items listed above have to do with failing test methods or inadequate contracting skill. It is interesting to note that the technician or contractor are most often the parties who receive the greatest blame if problems occur on a project. It is our experience that it is most often the failure to take responsibility for a simple lack of communication which yields a divided team concept and is the primary reason for most of the problems in earthwork engineering. A simplified version of the items listed above — the epidemic in low quality earthwork engineering appears to be more relational and less technical.

To create a passion for earthwork engineering in the spirit of the founding fathers we believe it is easier to start small rather than large. If the problem with the current practice in earthwork engineering is more relational, then it is a logical conclusion that companies with a strong professional relationship base can provide the best application of the technical principles required for high quality earthwork. Within our small company and others like ours we are observing the creation of a yeast that will easily influence the dough of companies who are more intent on making the organization and income the primary focus of engineering.

We have observed that many young engineers are willing to pursue a less profitable career marked by an eager willingness to find out why and how things work if they are valued and respected for their efforts. The owners, staff engineers and technicians of our firm and similar firms are excited about obtaining our principal satisfaction from doing quality engineering work and have therefore embraced the concept of keeping our lifestyles reasonable to maintain this satisfaction. We believe it is time for a revolution of quality earthwork engineering driven by a combination of quality research, laboratory and field testing, and most importantly, quality engineers who have an appreciation for both the technical and human aspects of our work. Our goal is to be one of the many "standard bearers" by providing a human example of the proper application of test methods. It is our professional opinion that living examples of how to do quality earthwork engineering will be more lasting and powerful. Actions always speak louder than words.

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Applications and Lessons Learned in the Field

F. Barry Newman[1] and Samuel G. Mazzella[1]

Compaction Control to Minimize Settlement of Fill Supporting a Shopping Center

Reference: Newman, B. F. and Mazzella, S. G., "**Compaction Control to Minimize Settlement of Fill Supporting A Shopping Center,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: A sloping site was regraded to a nearly level configuration for a strip shopping center. This resulted in one end of the building being underlain by rock at finished subgrade and the other end by up to 35 feet of compacted soil-rock fill. In addition, abandoned coal mine workings were present under the portion of the building with the rock subgrade. A portion of the building was supported by reinforced concrete drilled pier foundations through the abandoned mine to prevent damage due to mine subsidence. The other portion of the building was supported by spread footings bearing on compacted fill. This paper describes how ASTM testing methods were employed in controlling compaction of mixed soil-rock fill to limit total and differential settlements to tolerable values. Settlements after ten years have been minimal.

Keywords: compaction, settlement, fills, earthwork, foundations

Introduction

Level ground is a rare commodity in the hilly terrain of the Appalachian Plateau of southwestern Pennsylvania. The creation of level ground by excavating hills and filling valleys to provide suitable sites for commercial development frequently leads to classic foundation dilemma -- the filled area is generally more compressible than the excavated area. This condition can lead to excessive differential settlement of structures partially founded on compacted fill and partially on excavated areas. This paper presents a case history where careful compaction control of a mixed soil-rock fill was used to limit differential settlements to tolerable amounts.

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Proposed Development

Figure 1 shows a conceptual plan view of the shopping center. The building is a single story metal frame and masonry wall structure having a total length of about 1000 feet. Maximum column loads are about 100 kips.

Figure 1 also shows the conceptual grading of the site for the shopping center. The maximum depth of excavation was about 30 feet near the east end of the building, and the maximum depth of fill was about 35 feet under the south end of the building. The end result of the regrading was a relatively gently sloping site for the building and the parking area in front.

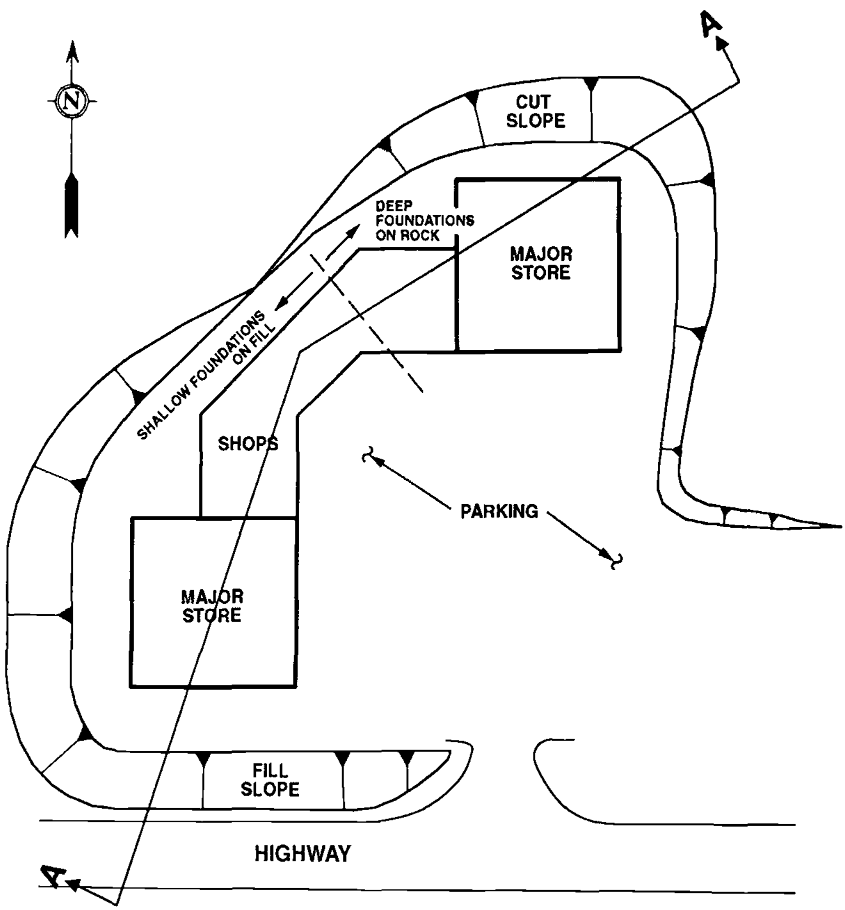

Figure 1 — *Conceptual Plan of Shopping Center*

Geology

The project site is underlain primarily by Pennsylvanian age rocks of the Lower Monongahela Group, Pittsburgh Formation and the Upper Conemaugh Group, Casselman Formation. Rock types varied from soft shale to hard sandstone. The Pittsburgh Coal, the basal member of the Pittsburgh Formation, was deep mined into the hillside in the late 1890's by the room and pillar method. These abandoned mine workings are about 20 feet below the finished grade along the eastern portion of the shopping center and outcropped along the southwestern portion of the site. Figure 2 shows a generalized geologic section A-A along the centerline of the shopping center building, and Figure 1 shows the plan location of Section A-A.

Foundation Considerations

The abandoned mine workings contained partially open voids up to several feet high. The plan for regrading the site included the overexcavation and removal of the shallower portions of the mine workings to the base of the Pittsburgh Coal seam to eliminate the potential for mine subsidence. The limit of overexcavation of mine workings from below the building was established where the rock became too hard to excavate economically. The presence of the abandoned mine workings below the eastern portion of the building meant that there was a high probability that future subsidence could damage the structure. Reinforced concrete drilled pier foundations extending into the sound rock (siltstone) below the base of the coal seam were therefore used to support the wall and roof loads of the building. The concrete floor slabs were constructed as slabs-on-grade with more reinforcement than normal to bridge potential sinkholes. Thus, they could be repaired if subsidence were to occur.

Since the eastern end of the building would be supported on drilled pier foundations on rock, this end was not anticipated to settle. One way to limit differential settlements is to place the entire structure on drilled piers on rock. However, the developer desired to avoid the premium cost of drilled pier foundations, and therefore elected to construct a well compacted fill and support the western portion of the structure on spread footings. This method is suitable provided that the fill is compacted to reduce settlements to tolerable values.

Fill Materials

The on-site materials that were available for construction of the fill below the building were residual soils formed by chemical weathering of sandstones and shales. These soils tend to be clayey silts to silty clays with rock fragments having Unified Soil Classification System (USCS) symbols ranging from CL and ML to GC and GM. As the excavation progressed further into rock, the fill became coarser and transitioned to rock fill with little soil. The grading plan was established to utilize the most soil-like materials below the building area and to use the rockier fill in parking areas where compaction and settlement were less critical. A total of 245,000 cubic yards of excavated material was

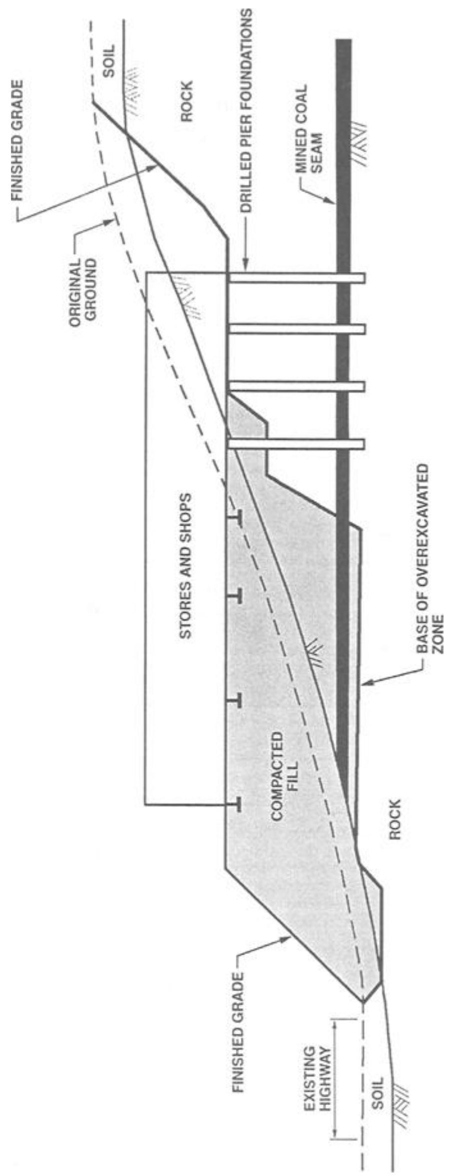

Figure 2 — *Generalized Section A-A*

generated for this project and only 205,000 cubic yards of fill were required to achieve the finished grade. This allowed the construction monitoring personnel to select or reject materials to incorporate into the fill.

Compaction Control

The fill under the building (structural fill) was specified to be compacted to at least 95 percent of the maximum dry density obtainable according to "Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb (4.54-kg) Rammer and 18-in (457-mm) Drop," (ASTM Designation D 1557-78). The fill placed in other areas (non-structural fill) was specified to be compacted to at least 90 percent of the maximum dry density according to ASTM D 1557-78. A rock correction was performed according to "Standard Practice for Correction of Unit Weight and Water Content for Soils Containing Oversize Particles," (ASTM Designation D 4718-87) because of the significant percentage of rock greater than the 3/4-inch size that would be present in the fill.

A field laboratory was placed onsite to conduct the tests necessary to control compaction. The fill excavated from the cut area was continuously changing. Therefore, Proctor tests of the fine fraction of the mix (-3/4-inch) were conducted throughout the grading operation as the changes in material types were identified. The parent rock that composed the coarse fraction of the mix (+3/4-inch) was also variable and therefore specific gravity values were determined based on material-type (primarily sandstone and shale).

The general procedure used to run the field density tests of the compacted fill and then to properly determine the target dry densities was as follows:

1. Determine the field total wet density and average moisture content of the fill using a nuclear gage in the direct transmission mode.
2. Excavate a representative sample of fill from below the center of the nuclear gage and pass the sample through a 3/4-inch sieve.
3. Determine the percent passing the 3/4-inch sieve based on moist unit weights and then determine the moisture content of the coarse and fine fractions.
4. Conduct a "one point" modified Proctor on the material passing the 3/4-inch sieve to identify which Proctor curve to use to determine the maximum dry density and optimum moisture content for the fine fraction.
5. Determine the corrected wet density of the minus 3/4-inch portion of the field sample when factoring in the percentage of the particles greater than 3/4-inch according to ASTM D 4718.
6. Compute the corrected dry density of the minus 3/4-inch portion of the field density test material.
7. Compare the corrected dry density of the minus 3/4-inch material in the field from step 6 to the maximum dry density for the minus 3/4-inch material determined in Step 4 to determine the percent relative compaction achieved.

ASTM D 4718 indicates that when the percentage of +3/4-inch materials exceeds 30 percent, the procedure is no longer appropriate. On this project, we used the above outlined procedure for mixtures up to about 35 percent of +3/4-inch material. When the percentage of material greater than 3/4-inch exceeded about 35 percent, the material was classified as "rock fill" and was placed and compacted according to a "method specification." Since the voids between particles are not filled with soil and the rock particles may deteriorate and settle with time, "rock fill" was not placed as structural fill under the building.

As the project progressed and test data was accumulated, some generalizations were made to expedite the testing procedures. Since the rock portion of the fill for this project was comprised of two types of rock (shale and sandstone), samples were collected and several tests for bulk specific gravity were performed. The average bulk specific gravities were 2.4 for shale and 2.6 for sandstone and average moisture contents were 3 to 4 percent. Using the equations from ASTM D 4718, two sets of tables were generated:

- The first table listed corrected soil wet density of minus 3/4-inch material as a function of the rock bulk specific gravity, the percentage of plus 3/4-inch rock, and the uncorrected total wet density as determined by the nuclear gage.
- The second table listed the corrected soil moisture content of minus 3/4-inch material as a function of the rock moisture content, the percentage of plus 3/4-inch rock, and the uncorrected total moisture content as determined by the nuclear gage.

To permit a rapid determination of the percent relative compaction, the truck of the field representative had all the equipment necessary to perform the testing at the fill placement location. This reduced the number of trips to the field laboratory. For this project, 16 moisture-density relationships were determined. The maximum dry densities of the minus 3/4-inch soils ranged from 115 to 132 pounds per cubic foot, and the optimum moisture contents ranged from 9 to 14 percent.

The observation of the placement and compaction of the fill is as important as the testing. The fill placement and compaction was monitored continuously by a geotechnical engineer or an engineering geologist. Loose lift thicknesses under the building were limited to 8 inches and random maximum particle sizes were limited to 6 inches. The compaction was performed by several passes of a segmented wheeled roller or a towed vibratory roller. The location of the field density test was then selected as "representative" of the typical condition of the lift.

The project specifications did indicate that the moisture content of the fill had to be near to the optimum for compaction as determined by ASTM D 1557. On this project, materials that were a few percent too wet or too dry relative to the optimum for compaction could not be compacted to the minimum dry density required with the available equipment. Therefore, percent relative compaction actually governed the acceptability of the fill and moisture contents had to be near optimum to achieve the specified degree of compaction.

This project involved the placement and compaction of about 130,000 cubic yards of structural fill to support the building. Approximately 240 field density tests were conducted in the structural fill. An area was compacted until the measured relative compaction achieved was a minimum of 95 percent. The average relative compaction achieved was 97 percent. Additionally, 75,000 cubic yards of non-structural fill were placed and compacted and approximately 64 field density tests were conducted. The non-structural fill was compacted to a minimum relativc compaction of 90 percent of the maximum dry density and to an average relative compaction of 94 percent.

Long-Term Performance

The building has been in service for about 10 years. A line of bricks on the rear of the building was recently surveyed to determine how much relative settlement has occurred. The south end of the structure over about 35 feet of fill was found to be less than ½ inch lower than the north side, which is founded on rock. There are no differential settlement cracks in the walls of the building or any other signs of distress. The walls contain periodic vertical crack control joints and two major control joints near the transition between the rock- and soil-supported portions of the building. Differential movements at these joints are negligible.

Conclusions and Recommendations

A building was successfully supported partially on rock and partially on about 35 feet of compacted soil and rock fill with post construction settlements of less than ½ inch over a ten-year period. The degree of compaction that achieved this performance was a minimum of 95 percent of the maximum dry density obtainable according to ASTM Test Designation D 1557 with rock corrections according to ASTM D 4718. The proper application of the ASTM standards in conjunction with appropriate design and construction procedures resulted in a fill with negligible long-term settlement and low compressibility under building loads.

K.R. Peaker,[1] H. Lohse,[1] and S. A. Ahmad,[1]

Compaction Control Bermuda Sports Centre

Reference: Peaker, K. R., Lohse, H., and Ahmad, S. A., **"Compaction Control Bermuda Sports Centre,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: The new national sports centre in Bermuda encompasses numerous facilities, including playing fields, which are located in an area previously filled with up to 6 m of loose granular materials. It was decided to re-compact the upper 3 to 4 m of the fill in order to provide a serviceable playing surface. A geotechnical investigation, which included both test pits and sampled boreholes, found that the fill was composed of aeolinite bedrock, which is a weakly, cemented friable limestone. This was found to overlie undisturbed bedrock. The limestone was composed of about 60 percent solid particles of shell fragments, ooliths and pellets and about 10 percent calcite cement. The porosity was about 30 percent. Normal compaction testing was difficult since the only nuclear gauge on the island had not been calibrated for several years and the Proctor values for the excavated limestone gave results lower than those measured in situ. It was decided to determine the maximum density of the fill using trial test strips. The test strips were found to be the most suitable method of evaluating the compaction. It was found that the aeolinite could be compacted in 300 mm lifts with heavy vibratory compactors.

Keywords: trial test strips, compaction control, aeolinite, Bermuda

Introduction

The new national sports centre in Bermuda encompasses numerous facilities, including two soccer fields, a cricket pitch, pitch hockey field and a running track surrounding the fields as well as an indoor aquatic centre, gymnasium and covered grandstand.

The fields and ancillary structures are located in an area previously filled with up to 6 m of loose granular materials. It was required to provide a playing field that had negligible settlement at the final grade as well as to support lightly loaded ancillary structures and light posts. The engineering solution was to remove and replace and compact all or part of the fill. The depth of fill re-compacted would depend upon the risk,

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which the client was prepared to take. In order for the client to have minimal to no risk, all of the fill would have to be excavated and then replaced in a well compacted state. If the upper 3 to 4 m of loose fill was re-compacted, then settlements of 30 to 40 mm were predicted to occur over time. This amount of settlement could be tolerated and be remediated with normal periodic maintenance and this alternative was selected for the construction of the fields.

Soil Conditions

The geotechnical investigation (Shaheen & Peaker Limited 1998) in this area consisted of both test pits and sampled boreholes. The test pit initially found up to 4.3 m of loose fill, which was the limit of the equipment. The fill was comprised primarily of white to beige fine to medium sand with traces of limestone. The fill in the test pits was well compacted in the upper 0.5 to 0.8 m due to surface traffic but below this depth the fill was loose and the test pit readily caved. The boreholes confirmed that the fill was loose and Standard Penetration Test N values ranged from 3 to 10 blows per 300 mm. In addition the boreholes revealed that the fill had a maximum depth of about 6 m.

This fill originated from the aeolinite bedrock of Bermuda, which was formed on land in sand dunes, comprised of carbonate detritus. This detritus was subsequently cemented and the amount of cementation varied with the age of the rock. Commonly the younger rock formations contain lenses of uncemented sand. Between the various geological formations are geosols, which are fossil soil layers formed on earlier sand deposits (Rowe 1990).

The geological map of Bermuda (Vacher et al. 1989) indicates that the bedrock at the site is the Rocky Bay Formation of Pleistocene age. This is a weakly cemented friable aeolinite consisting of about 60 percent solid particles of shell fragments, ooliths and pellets and approximately 10 percent calcite cement. The porosity of the limestone was reported to be 30 percent (MacDonald 1994). When this material is broken up and compacted the bulking factor is approximately one, i.e. the excavated material will probably occupy the same volume once it is well compacted.

A number of Standard Proctor tests were carried out during the construction period in order to attempt to relate insitu densities to laboratory results. However, the aeolinite did not break up in the field in the same manner as a Proctor sample and the values were of limited use. The values are as follows:

(1) 1694 kg/m3 at 16.4% (S&P)
(2) 1685 kg/m3 at 16.4% (site)
(3) 1662 kg/m3 at 16.5% (site)
(4) 1611 kg/m3 at 19.0% (D&J)

Compaction

Compaction testing was difficult, as there were no testing facilities at the site. Only one nuclear gauge was available on the island, a Troxler 3430, which was borrowed from the Ministry of Works and Engineering. There are no provisions on the island for periodic calibration and the gauge had not been calibrated since it was purchased several years earlier. The gauge was therefore used to give readings for the trial strips. It was not important whether the readings were accurate or not, since the change in the readings was used to determine the success of the compactive efforts.

The filling of the playing fields was undertaken with both older excavated aeolinite bedrock as well as fresh excavated rock from the site. The older material was sufficiently broken up so it could be compacted using normal compaction techniques for granular soil.

The freshly excavated bedrock posed a potential problem during compaction. When the aeolinite is excavated it breaks up into a gap graded material consisting of large fragments and fine to medium sand. The large fragments could prevent adequate compaction by allowing the formation of voids in the compacted fill. Standard Proctor tests on the rock resulted in a well-broken rock or sand which had lower maximum dry densities that were less than the values obtained in situ. In some cases the resulting degrees of compaction were measured to be about 110% of the standard Proctor maximum dry density.

It was difficult to relate the density measured in the field to the laboratory results and the calibration of the nuclear gauge was questionable. It was therefore decided to establish the maximum density of the aeolinite fill with trial compaction strips. These would be carried out on both the older material as well as the freshly excavated rock.

A trial strip is site specific and equipment specific. It is not possible to transfer the results to other sites. To work reasonably well, the fill material should be made of a uniform type for the entire project; the compacted thickness of each layer should be constant; the compaction equipment and density measuring device must remain the same.

The purpose of a trial strip is to calibrate the equipment and soil so that a quality control system can be established for the site compaction work.

To achieve the necessary calibration, the soil should be at optimum moisture content. A handful of soil at optimum moisture content will form a "tight ball" if the fist is clenched. Well graded soil compacts well. This type of soil gives a definite curve when plotting compactive effort vs. density. Uniformly graded soil is difficult to compact and gives a flat curve of density vs. compactive effort. The freshly excavated rock, which is gap graded, gave erratic plots of density vs. compactive effort.

To run the compaction trial strip, a flat area of ground was selected where the surface consisted of aeolinite bedrock. The procedure was as follows

(3) A layer of fill about 300 mm thick, 2.5 m wide and 15 m long was spread without making any effort to compact it.

(2) A series of readings were made with the nuclear gauge to establish the average value for the loose fill.

(3) Using the selected compactor one pass was made and the reading was established for one pass. This was repeated for an additional 5 to 6 passes and the results were plotted.

The compaction equipment used for the various trial strips included (1) Cat 977L bulldozer, (2) Vibratory smooth drum roller (Ingersol Rand DD-90), (3) Vibratory sheep's foot type compactor, and (4) Dual drum vibratory asphalt roller (Ingersol Rand).

The initial trial strips, undertaken in August, tested the older sand fill and the freshly excavated aeolinite using the compactive efforts of a Cat 977L bulldozer tracks and a vibratory smooth drum compactor (Ingersol Rand DD-90). The trial strips consisted of two lifts about 200 to 300 mm thick. Density measurements were made with the nuclear density gauge in the upper 150 mm of each lift. The results of these tests are shown on Figures 1 and 2.

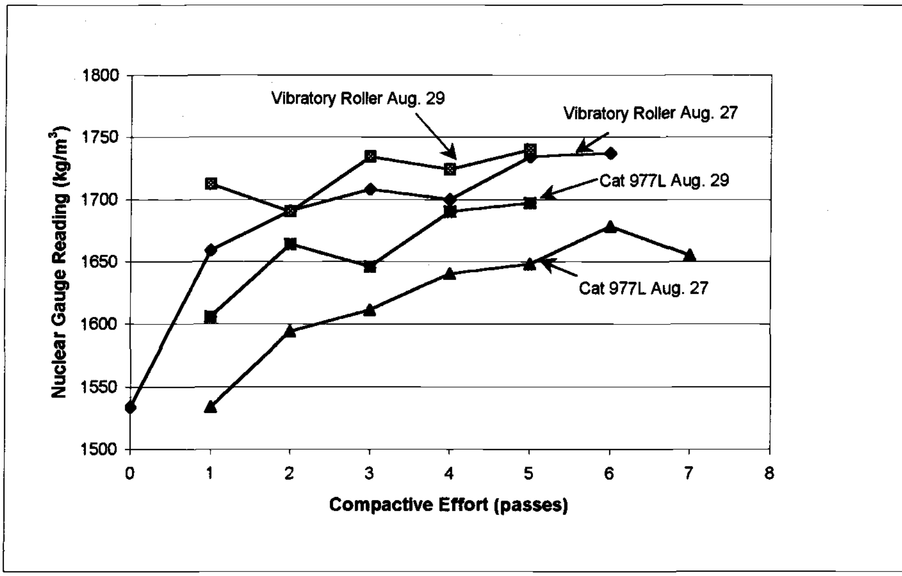

Figure 1 - Trial Strip on Fill

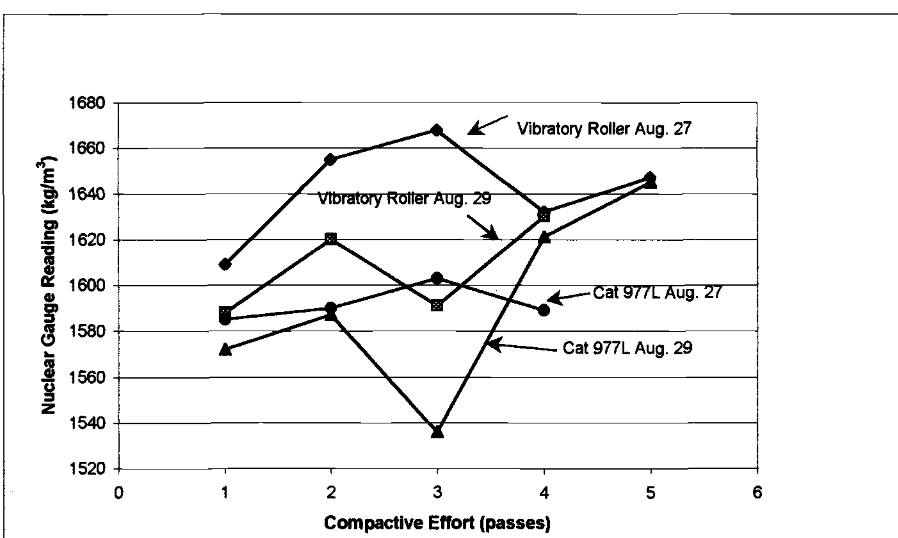

Figure 2 - Trial Strip on Freshly Excavated Aeolinite

The test curves for the older fill are relative smooth as would be expected and the older fill compacts uniformly. The greatest compaction was achieved with the vibratory roller. The bulldozer tracks densities corresponded to about 96 to 97 percent of the those achieved with the vibratory roller.

The test curves for the freshly excavated aeolinite were more widely scattered. During the August 27 trial, the bedrock was ripped to a depth of 450 to 600 mm and large pieces of rock were present in the fill. The lower results achieved by the Cat 977L on August 27 could be attributed in part to the tracks riding the large pieces of rock. The vibratory roller was more successful with this material. On August 29 the rock was ripped to a depth of about 150 mm to achieve a smaller size. On this date the compaction with the Cat 977L was more successful. The trials on the aeolinite indicated that the density did not increase significantly after 4 to 5 passes and large variations in the density occurred due to the presence of larger pieces of rock.

The maximum density for the freshly excavated aeolinite was found to be around 1640 to 1660 kg/m^3, which corresponded reasonably well with the laboratory maximum dry densities.

Additional test strips were undertaken in September to determine if the freshly excavated limestone could be adequately compacted in 300 mm thick lifts. In this trial about 300 to 400 mm of loose excavated limestone (ripped to a depth of about 150 mm) was compacted with a heavy vibratory sheep's foot type compactor. Density measurements were made at the top of the lift to a depth of 150 mm In total, nine passes were made to determine the maximum density. Once a maximum density was achieved the top 150 mm of the trial strip was removed and the density of the lower 150 was measured. These test results are shown in Figure 3.

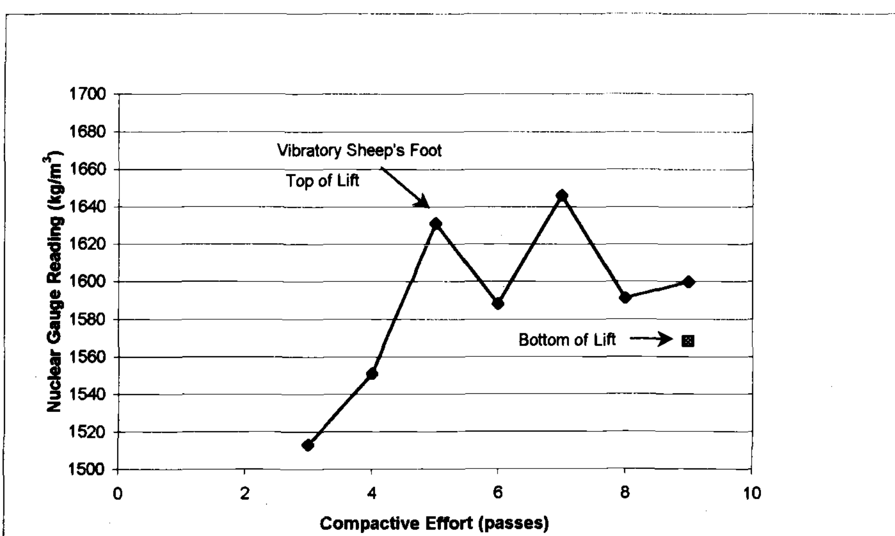

Figure 3 - Trial Strip on Freshly Excavated Aeolinite

The maximum density of the freshly excavated aeolinite was achieved after about 4 to 5 passes with the vibratory sheep's foot compactor. This density was approximately 1620 kg/cm^3 and corresponded reasonably well with the previous trial strips. The density of the lower half of the lift was found to be 1568 kg/m^3 which corresponds to about 98 percent of the maximum achieved at the surface of the trial strip.

Additional trial strips were made in September using an Ingersol Rand (10 ton) vibratory double steel drum roller. A lift thickness of 300 mm was used and both 150 and 300 mm readings were made with the nuclear gauge. These test results are shown in Figure 4.

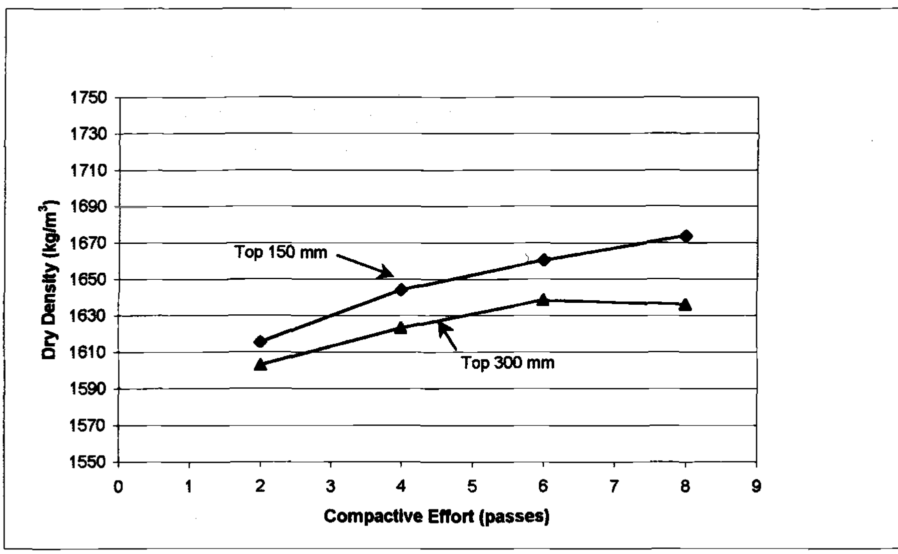

Figure 4 - Trial Strip on Freshly Excavated Aeolinite

The results indicate that after 2 passes with the roller, the compaction of the surface 150 mm of the aeolinite fill was about 97 percent of the probable maximum. The compaction of the upper 300 mm of the lift was about 96 percent of the probable maximum.

Conclusions

Where little engineering skills, laboratory facilities and calibration of nuclear equipment are available, the trial strips are considered to be the most suitable method to determine the maximum density of the compacted aeolinite bedrock fill.

Using trial strips it was found that the weakly cemented aeolinite in Bermuda could be adequately compacted in about 300 mm lifts. The compactive effort was minimized by ripping the rock in thin layers, in our case about 150 mm, and by using heavy vibratory compactors. The use of a vibratory sheep's foot compactor did significantly improve the compaction. The aeolinite broke up and compacted just as easily with vibratory drum compactors. Static compaction did not achieve the required results and is not recommended for compaction of aeolinite bedrock.

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Dynamic Compaction of Surface Mine Spoils to Limit Settlements Within Commercial Developments

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Reference: Satyapriya, C.K, and Gallagher, P. E, **"Dynamic Compaction of Surface Mine Spoils to Limit Settlements Within Commercial Developments,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Throughout the Appalachian areas of the Eastern United States, site developers face the task of constructing extensive cut and fill situations in order to obtain building sites. Along with the expense of large filling operations, oftentimes they are required by the geotechnical engineer to monitor the fills for settlements before the foundation work can begin. This monitoring period can at times extend for 18 months. A developer will incur severe financial burdens during this monitoring period. CTL Engineering has incorporated the use of a technique commonly known as "dynamic compaction" to precompress these fills such that the post - construction monitoring period is substantially reduced. Five case studies have been completed where the fills were conventionally constructed and the building pads dynamically compacted. Three of the sites were extensively instrumented to monitor settlements during and subsequent to construction. The data will show that the areas that received dynamic compaction did not differentially settle under very large cut-fill depths and comparative fills where conventional compaction techniques were used settled more than 7 inches. Settlement plates, standard penetration testing and various survey networks were used to evaluate the performance of the dynamic compacted sites. The sites have been in operation for up to five years and have not exhibited any signs of differential settlement.

Keywords: dynamic compaction, differential settlement, precompression, Settlements Plates

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Introduction

Dynamic compaction is a technique of densifying soils by repeatedly dropping dead weights on the ground surface. This type of ground improvement has been used since Roman times and in various countries. In the U.S. a number of sites have been treated by dynamic compaction (Drumheller & Shaffer, 1997 and Satyapriya, 1988). Typically the energy from this repeated compaction densifies the soil mass to depths up to 30 vertical feet (10 m), The grid spacing of the impact nodal points is 7 to 15 feet (2 to 5 m) on centers depending upon drop height, weight, and proposed land use. Figure 1 is an illustration of dynamic compaction and the stress distribution within the effective depth.

Since 1982, CTL Engineering has been involved with over thirty sites in Ohio, Kentucky, North Carolina and West Virginia where this method was used. The subsurface conditions at these sites varied from a domestic waste landfill to surface mine spoils and loose deposits. Due to the economics of this technique and the increasing number of marginal sites being considered for construction, dynamic compaction is expected to enjoy rapid growth in the future. The fact that the fills are precompressed, is an enormous benefit to developers who can accelerate the time schedule for tenants to occupy their retail space by not waiting for conventional consolidation to occur. Also, this technique will allow for the commercial buildings to be founded upon spread footings in lieu of deep foundations. This will result in significant cost savings, in addition to the fact that the need for specialty contractors for foundation construction is eliminated.

Design and Construction Considerations

This technique has been used to improve sites with almost all types of materials and stratigraphy. The effectiveness of dynamic compaction depends on several variables: magnitude of the weight, height of the weight's free fall, number of drops per location, distribution of the drop locations, homogeneity and isotropy of the soil, soil strength, and the degree of the soil's saturation. However, dynamic compaction has been successfully used in a variety of subsurface conditions.

Generally, the effectiveness of this technique decreases as the cohesive soil content increases. Dynamic compaction is most effective when the soils are unsaturated and the groundwater is at least six feet below the surface.

The typical weight of the hammer used ranges between 6 and 22 tons (5443 to 19,958 kg) but weights as heavy as 40 tons (36,287 kg) have been used (Drumheller & Shaffer, 1997). Weights are usually dropped with a single line, using modified cranes from heights exceeding 25 feet (7.5 m). Studies have indicated that single lines are most effective, with attached lines reducing efficiency by as much as 20 percent (Dumas & Beaton).

The maximum practical effective depth has been found to be about 40 feet (12 m). Beyond this depth, other techniques may have to be used in conjunction with dynamic compaction.

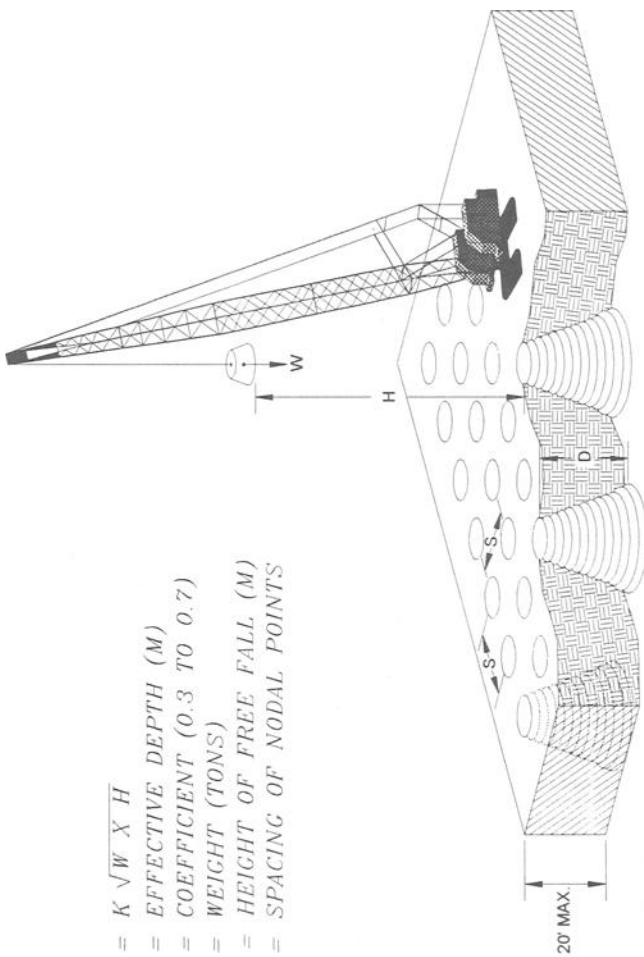

FIG.1— DYNAMIC COMPACTION DIAGRAM

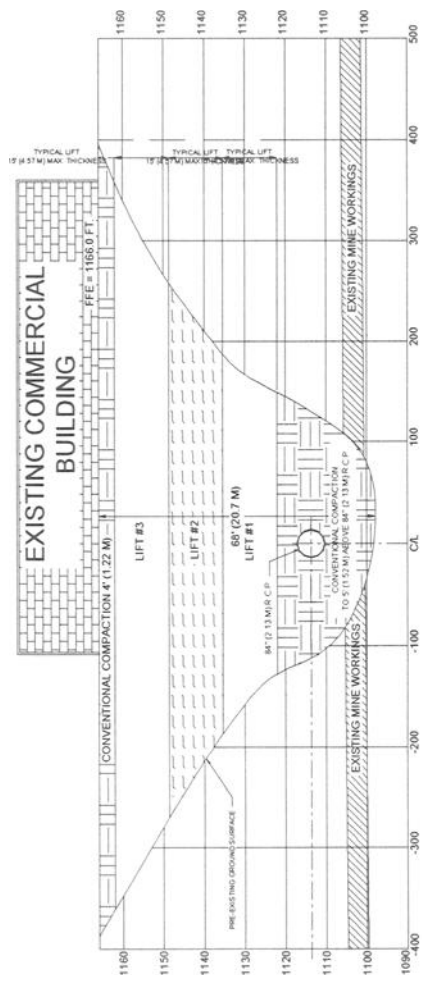

FIG. 2— TYPICAL SECTION AT SITE #1, ST.CLAIRSVILLE, OHIO

The effective depth of compaction has generally been related to the square root of the product of the weight (tons) and height (meters). The coefficient of this relationship has been found to vary between 0.3 and 0.7, depending on site conditions. This relationship may be expressed as:

$$D = K * \sqrt{(W * H)}$$

D = effective depth in meters
K = coefficient ranging between 0.3 and 0.7
W = weight in tons
H = height of free fall in meters

Dynamic compaction can be carried out at distances as close as 20 feet (6 m) from adjoining structures. This distance to which dynamic forces are transmitted is site specific. The weight, when it strikes the surface produces P, S, and R waves. The P and S waves produce the required densification of the soil. However, the R waves and other surface traveling waves are harmful, as they tend to loosen the surface soil and may damage adjacent structures if they are too close. Drop heights may be reduced in the proximity of structures to reduce the potential of damage. Owners, contractors and engineers should therefore consider monitoring all structures and underground utilities in the vicinity.

Where new fills are to be constructed, the soils are placed in horizontal lifts of 8 in. (0.2 m) and conventionally compacted to densities exceeding 95% Standard Proctor. Since dynamic compaction will allow for precompression of the fill, in deep fill areas the lift thickness can be increased to 18in. (0.5 m). While the 18 in. (0.5 m) lift requirement accelerates the completion of the fill in comparison to 8 in. (0.2 m) lifts, there is a potential for significant time dependent settlements to occur in the fill. Application of dynamic compaction to the fill will reduce the time dependent settlement. The time to complete the dynamic compaction is significantly quicker than conventional construction using 8 in. (0.2 m) fill lift thickness. The time to accomplish dynamic compaction at a site is dependent on the factors described before and the number of drops at a nodal point. The number of drops of the dead weight at the same location has a diminishing return. Typically no more than 7 drops should be applied at any location. Application of dynamic compaction results in craters as shown in Fig. 1. In addition, field experience regarding the creation of craters during the dynamic compaction phase is that 5 drops of the weight should not create a depression exceeding 36 in. (1 m). Modification of the weight / height ratio must be made to accommodate a maximum 36inch (1 m) crater formation.

Case Studies

Five selected case studies have been presented. However, three of the sites are presented with site design considerations, instrumentation/data acquisition, and performance evaluations. The site information is presented as follows.

St. Clairsville, Ohio – Commercial Development

This site (see Fig. 2) is located along I-70 100 miles east of Columbus, Ohio and includes 80 acres (32.3 hectares) of existing mine spoil. The thickness of mine spoil within the building(s) limits, generally single story masonry block supported structures, ranged from zero (rock) to 70 feet (21 m). The site preparation for this site entailed handling 2.5 million cubic yards (approx. 2.5 million cubic meters) of on-site mine spoils/soils in order to provide a 45 acre (18.2 hectares) level building pad. This site was constructed using 18 in. lifts (0.5 m) conventionally compacted to densities exceeding 95% standard proctor and dynamically compacted at intervals of 15 feet (4.5 m). Dynamic Compaction was applied at nodal points of 7 feet (2 m) using a 12-ton (10,886-kg) weight falling 30 feet (9 m). The number of drops at any location was less than 5 drops. Crater depths ranged from 12 in. (0.3 m) to about 30 in. (0.75 m). The last four (4) feet (1.2 m) of the fill was placed using conventional compaction procedures with a lift thickness of 8 inches (0.2 m). The masonry block walls and the columns were constructed using spread footings with an allowable bearing value of 3000 pounds per square foot (143.6 kN/m^2) and reinforced similar to a grade beam system

An extensive instrumentation program, which includes surveyed settlement plates, pre and post construction geotechnical drilling, and monitoring benchmarks set within the buildings, was completed and is on-going at this site. Table 1 shown below summarizes the data obtained from the instruments. Notice that within the 70 feet (21 m) of dynamically compacted fills underlying the buildings, only ¼ in. (6.4 mm) of settlement has occurred over the past 4 years. In contrast, the parking areas surrounding the buildings that were conventionally compacted settled 3.55 in. (90 mm).

Table No. 1 – Settlement Data for St. Clairsville

	Total Settlements (in.)/(mm)		
Fill Conditions	**At Completion**	**To Date (1/99)**	**Average SPT N-Values**
Dynamic Compacted Fills	6.5/165.1	6.75/171.5	22
Non-Dynamic Compacted Fills	0.75/19.1	4.30/109	12

With more than 1 million square feet (90,000 sq. m.) of finished commercial floor space completed at this site, no structural distress has been observed. Finally it is of utmost importance to note that the foundation units were constructed immediately upon completion of the fills without any time being allowed for consolidation related settlements.

Clarksburg, West Virginia – Commercial Development

This site is located along I-79, 90 miles south of Pittsburgh, PA in Clarksburg, WV and includes 65 acres (26.3 hectares) of existing mine spoil. The thickness of mine spoil within the store limits ranged from zero (rock) to 50 feet (15 m). The site preparation entailed handling 1.5 million cubic yards (approx. 1.5 million cubic meters) of on-site mine spoils/soils in order to provide a 25 acre (10.1 hectares) level building pad. This site was constructed using 18 in. lifts (0.5 m) conventionally compacted to densities exceeding 95% standard proctor and dynamically compacted at intervals of 15 feet (4.5 m). Dynamic compaction was applied using the same spacing of nodal points, weight and height of fall as the St. Clairsville project. The number of drops at any location was also less than 5 drops. Crater depths ranged from 12 in. (0.3 m) to about 36 in. (1 m). The last five feet (1.5 m) of the fill was placed using conventional compaction procedures with a lift thickness of 8 in. (0.2 m) at a minimum 100% standard proctor density. The masonry block walls and the columns were constructed using spread footings with an allowable bearing value of 3000 pounds per square foot (143.6 kN/m^2) and reinforced similar to a grade beam system.

An extensive instrumentation program, which includes surveyed settlement plates, pre and post construction geotechnical drilling, and monitoring benchmarks set within the buildings, was completed and is on-going at this site. Table 2 shown below summarizes the data obtained from the instruments. Within the 50 feet (15 m) of dynamically compacted fills underlying the buildings, only ½ in. (1.3 mm) of settlement has occurred over the past 2 years. In contrast, the parking areas surrounding the buildings that were conventionally compacted settled 3.0 in. (76 mm).

Table No. 2 – Settlement Data for Clarksburg

	Total Settlements (in./mm)		
Fill Conditions	**At Completion**	**To Date (1/99)**	**Average SPT N-Values**
Dynamic Compacted Fills	4.5/114	5.0/127	35
Non-Dynamic Compacted Fills	0.5/13	3.5/89	15

With more than 250,000 square feet (22,504 m^2) of finished commercial floor space completed at this site, minor structural distress has been observed in the form of expansion joint movements of 1.25 in. (31.75 mm). Finally, it is of utmost importance to note that the foundation units were constructed immediately upon completion of the fills without any time being allowed for consolidation related settlements.

Morgantown, West Virginia – Commercial Development

This site is located along I-68 five miles east of Morgantown, WV and includes 60 acres (24.3 hectares) of existing mine spoil. The thickness of mine spoil within the store limits ranging from zero (rock) to 80 feet (24 m). The site preparation entailed handling 2.0 million cubic yards (approx. 2 million m^3) of on-site mine spoils/soils in order to provide a 35-acre (14.2 hectares) level building pad. This site was constructed using 18 in. lifts (0.5 m) conventionally compacted to densities exceeding 95% standard proctor and dynamically compacted at intervals of 15 feet (4.5 m). Dynamic Compaction was applied at nodal points spaced at 7 feet (2 m) using a 9-ton (8165-kg) weight falling 30 feet (9 m). The number of drops at any location was less than 5 drops. Crater depths ranged from 12 in. (0.3 m) to about 30 in. (0.75 m). The last five feet (1.5 m) of the fill was placed using conventional compaction procedures with a lift thickness of 8 inches (0.2 m) at a minimum 100% standard proctor density. The masonry block walls and the columns were constructed using spread footings with an allowable bearing value of 3000 pounds per square foot (143.6 kN/m^2) and reinforced similar to a grade beam system.

An instrumentation program, which includes surveyed settlement plates, pre and post construction geotechnical drilling, and monitoring benchmarks set within the buildings, was completed and is on-going at this site. Table 3 shown below summarizes the data obtained from the instruments. Notice that within the 80 feet (24 m) of dynamically compacted fills underlying the buildings, only ¼ inch (6.4 mm) of settlement has occurred over the past year. In contrast, the parking areas surrounding the buildings that were conventionally compacted settled 1.75 inches (44.5 mm).

Table No. 3 – Settlement Data for Morgantown

	Total Settlements (in./mm)		
Fill Conditions	**At Completion**	**To Date (1/99)**	**Average SPT N-Values**
Dynamic Compacted Fills	5.25/133.4	5.5/139.7	30
Non-Dynamic Compacted Fills	0.75/19	2.5/63.5	11

With more than 300,000 square feet (27,000 m^2) of finished commercial floor space completed at this site, no structural distress has been observed. Finally, it is of utmost importance to note that the foundation units were constructed immediately upon completion of the fills without any time being allowed for consolidation related settlements.

In addition to the above three sites, various other sites containing surface quarry or mine spoil have been improved using Dynamic Compaction. These other sites were not significantly instrumented. Following is a brief description of two other sites.

Lowe's Store – Steubenville, Ohio

This site is located along S.R. 22 west of Steubenville, Ohio. Existing mine spoil thickness within the store limits ranged from none (rock) to about 70 feet (21 m). Similar to the St. Clairsville site, this site was constructed using 18 in. (0.5 m) lifts and Dynamically Compacted at intervals of 20 feet (6 m). Dynamic Compaction was applied at nodal points of 10 feet (3 m) using a 12-ton (10,886-kg) weight falling 30 feet (9 m). The number of drops at any location was less than 5 drops. Crater depths ranged from 12 in. (0.3 m) to about 30 in. (0.75 m). The last six feet (1.8 m) of the fill was placed using conventional compaction procedures with a lift thickness of 8 inches (0.2 m). The masonry block walls and the columns were constructed using spread footings with an allowable bearing value of 3000 pounds per square foot (143.6 kN/m^2).

Four-Story Steel Frame Office- Building and Two Three- Story Office Buildings – Fifth Avenue, Columbus, Ohio

These buildings were constructed over quarry mine spoil ranging in thickness between 60 feet and 80 feet (18 to 24 m) adjacent to the Scioto River in Columbus, Ohio. Since excavation and replacement of the variably compacted quarry spoil was not an acceptable option, dynamic compaction of the spoil in place was considered. Dynamic compaction was applied at the surface of the mine spoil using a 12-ton (10,886-kg) weight falling in excess of 40 feet (12 m). Due to the presence of cohesive soil pockets a layer of granular material as cover was placed prior to the applied Dynamic Compaction. Dynamic Compaction resulted in crater depths of up to 36 inches (1 m). In isolated locations crater depths were found to be greater than 48 inches (1.2 m). These areas were cut down to the bottom of the craters and re-compacted. The craters were cut to the bottom and an ironing pass using the same 12-ton (10,886-kg) weight falling about 10 feet (3 m) was applied. All the buildings have been constructed using spread footings using an allowable bearing value of 4000 pounds per square feet (191.5 kN/m^2). The four-story building was completed in 1987 and the two three-story buildings were completed in 1992. No distresses have been observed. The four-story building was located approximately 60 feet from an adjacent building. Vibration monitoring on the existing building indicated no discernable vibrations. Although, during the pounding, air vibrations and noise caused some concern for the occupants.

Conclusions

Dynamic compaction has successfully been used to economically prepare sites for buildings. Precompression of the mine spoil allows for the building to be supported on conventional spread footings constructed immediately after the fill is placed. Dynamic compaction eliminates the monitoring period between placement of the fill and the completion of time-dependent settlement of the fill under its own dead weight. Construction of deep foundations in the fill, which has a potential for settlements, may result in negative skin friction. Cost savings from the use of shallow footings and acceleration of project completion will outweigh the additional cost of dynamic compaction. Typical dynamic compaction costs vary from about \$ 0.50 to about \$ 2.00 per square foot (\$0.014 to \$ 0.056 per m), depending on the size of the project, complexity of the project and site constraints. Dynamic compaction will be used more frequently as suitable sites diminish.

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Compaction and Performance of Loess Embankments

Reference: Zhang, L., Du, J., and Hu, T., **"Compaction and Performance of Loess Embankments,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384*, D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: This paper summarizes the practice of highway embankment compaction in the loess plateau of northwestern China, based on a field trip and the related laboratory studies. A large number of high loess embankments were built across gullies. The compaction was based on the standard Proctor method (ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort, D698-91) during 1950 - 1985, and the modified Proctor method (ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort, D1557-91) after 1985. The performance of these embankments is described. Stability analysis and centrifuge tests are conducted to confirm the observations and improve designs. Storm water ponds are found to be critical to both stability and settlement. For embankments compacted using the standard Proctor method, progressive failure would start with any further erosion if the slopes were steeper than 1:0.75.

Keywords: embankment, compaction, stability analysis, settlement, loess, erosion

Review of Compaction Practices

This paper concerns construction of highway embankments in the eastern Gansu and northern Shannxi area, which is at the center of the "Loess Plateau" in China. This area has semi-arid climate; the annual precipitation is only 250 – 600 mm. The loess thickness varies from a few meters to 50 meters. The soils found in the west of the area are largely Q_3 and Q_4 quaternary deposits and those in the east are Q_2 and Q_3 deposits. Silt (d = 0.074 - 0.002 mm) comprises more than 60% and clay (d < 0.002 mm) comprises about 20% of the soil. The natural dry density, void ratio, and natural moisture content are in the range of 1140 - 1600 kg/m^3, 0.78 - 1.50, and 7.0% - 23.0%, respectively, and the

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liquid and plastic limits are 21.7% - 32.5% and 14% - 21%, respectively. The friction angle and apparent cohesion from the consolidated-undrained tests are 23° - 39° and 10 - 65 kPa, respectively.

Pre-1950

Although a lot of loess embankments were built before 1950, few of them survived the harsh erosion and wars. The compaction of ancient embankments was done manually, typically using hand hammers or big stone rams of 50 – 100 kg operated by several people. The rule-of-thumb for moisture control was "being a lump when it is held in hand but breaks apart when fallen to the ground" (Ninxian County Government 1992). Figure 1 shows the remainder of an ancient embankment in Xiantou. It was built before 1900 and major renovations were made in 1918, 1955, and 1982 ~ 1984. The present embankment serves a major highway. It is 54 m high, 180 m long, and 11 m wide at the crest. The ancient fill is exposed to the sun. The fill materials are moderately cemented so that the nearly vertical slope stands approximately 40 m high.

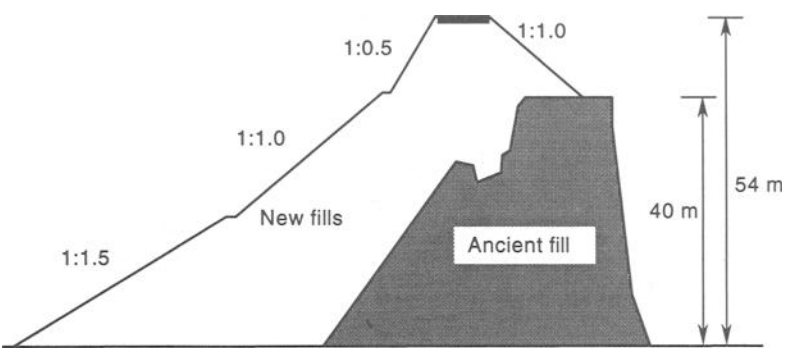

Figure 1 - *Xiantou Embankment Embodying Ancient Fills*

1950 - 1985

The period between 1950 to 1985 witnessed construction of thousands of embankments for unclassified roadways. Compactions based on the standard Procter compaction (ASTM Test Method D698-91) were enforced for most embankments. Only clean and inorganic loess soils were selected as fill materials. The contents of clay and coarse sand were limited to less than 25% and 20%, respectively. Such fill materials had a plastic index in the range of 10 - 14%. The maximum dry density and optimum moisture content were 1630 ~ 1700 kg/m^3 and 17 ~ 20%, respectively. The construction moisture content was controlled within 2% around the optimum depending on the natural moisture content, and the relative compaction was required to be no less than 0.95. In Eastern Gansu, the typical compaction procedures were (Ninxian County Government 1992),

- Remove the surface debris and organic soils,
- Compact the foundation,
- Spread and compact fills in lifts. For compaction using flat rollers, the lift thickness was about 0.2 m and seven passes were usually needed for each lift. For manual compaction, the spreading thickness was limited to 0.1 m.

It was interesting to note that the slopes of most of the embankments built in this period were very steep, so that the embankments were commonly called "loess bridges" in the area. These steep structures were normally built at ridges and therefore subjected, to a lesser degree, to the effects of water infiltration. Table 1 lists the slopes recommended for use in that period (Ninxian County Government 1992), among which the slope 1:0.3 was used at top of a number of embankments. Figure 2 shows Miqiao embankment as rebuilt in 1958. The slope varies from 1:0.3 at the top 34 m to 1:0.75 at the bottom 6.0 m. The design of such steep slopes was influenced by the limit slope concept. According to the concept, the engineered optimum slope should take a configuration similar to the shape of the critical slip surface. An exponential curve was developed for configuring the side slopes (Shannxi Highway Design Institute 1960).

Table 1 - *Recommended Slopes for Loess Embankments (after Ninxian County Government 1992)*

Soil type	Height of embankment (m)		
	< 20	20 – 30	30 – 40
Q_2 deposit	0.2 – 0.5	0.3 – 0.75	0.1 – 1.0
Q_3 and Q_4 deposits	0.5 – 0.75	-	-

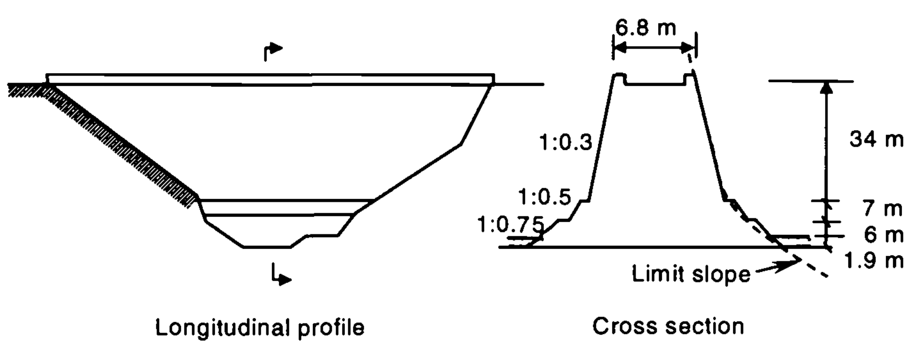

Figure 2 - *Miqiao Embankment as Built in 1958*

Compaction near the side slopes of such embankments was not attainable by machinery and had to be conducted manually. The construction technique shown in Figure 3 was conventionally adopted. Stacked rafters were used to hold the fill near the slope. The rafters were in turn held in place by reed bundles or straw ropes. After the current lift was compacted, the reed or rope connections to the rafters for the previous lift were cut, the rafters were removed to be used again, and the reeds or ropes were left in the embankment, which in fact resembled modern reinforced walls.

Summarized in Table 2 are some of the high "loess bridges" built or renovated during 1950 - 1985. Note that the flatter slopes (1:1.2 ~ 1.75) were adopted mostly for the lower elevations when these embankments were widened in early 1980's.

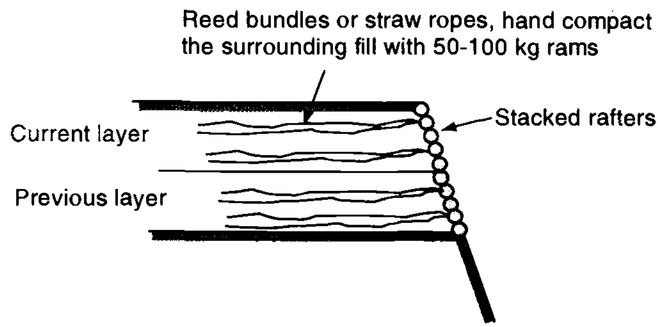

Figure 3 - *Compaction of Steep Embankments*

Table 2 - *Existing Embankments Built during 1950 – 1985*

Name	Location	Length (m)	Height (m)	γ_d (kN/m^3)	w_{op} (%)	Current slope	Year built
Nancang	Nancang town	120	59.3	16.7	14.0	1:0.75~1:1.5	1957
Xiantou	East of Xiantou	180	54	-	-	1:0.75	1955
Lujiayan	West of Lujiayan	160	46.4	-	-	1:1.2	1967
Miqiao	West of Miqiao	140	42.0	-	-	1:0.4~1:1.0	1957
Leijia	State road 309 K1643	101	89.0	17.1	16.5	1:0.75~1:1.75	1970
Wangyan	State road 309 K1641	-	60.0	17.1	16.5	1:0.75~1:1.5	1976
Liujiagou	Yijun County	62	19.2	16.6	18.2	1:0.5~1:1.5	1950's
Wujiayan	Yijun County	43	42.1	16.3	19.7	1:0.16~1:1.2	1950's
Hanzhuan	Huanglin County	62	29.1	16.0	17.3	1:0.35~1:1.25	1950's
Shijiazhuang	Luochuan County	27	61.0	16.6	18.3	1:0.33	1950's

1985 - 1990's

The age of expressways (the interstate system) in China did not start until the late 1980's. Embankments for expressways differ significantly from those for old highways:

- Interaction of fills with water. The roadways have to cross gullies according to the need of road alignment. This requires that embankments be built in the gullies with considerable water passage. Consequently, culverts are needed and the fill materials below the culvert elevations have to interact with storm water ponds.
- Post construction settlement. The pavement structures that account for the majority

of the total cost allow a post-construction settlement less than 0.2 - 0.3 m.

To meet the above challenges, a new compaction standard equivalent to the modified Proctor compaction (ASTM Test Method D1557-91) was adopted by the Chinese highway design and construction codes (China Department of Transportation 1987, 1991), and slopes were designed much flatter.

In order to evaluate the stability and settlement of embankments, a soil sample was recovered at Jinnin. The silt fraction comprised up to 79% of the soil and the clay fraction comprised 16%. The liquid and plastic limits were $\omega_L = 30\%$ and $\omega_p = 17\%$, respectively. The maximum dry density, minimum void ratio, and optimum moisture content were $\rho_d = 1910$ kg/m^3, $e_{min} = 0.44$, and $\omega_{op} = 12.5\%$, respectively. According to the new specifications (China Department of Transportation 1987, 1991), the relative compaction is specified to be $K \geq 0.93$ for subgrades and $K \geq 0.90$ for embankment fills. Figure 4 shows a standard profile used for embankment design for the Lanzhou-Xian freeway (State Road 312). Table 3 gives several embankments in the freeway between K603 – K635. Storm water ponds are observed upstream of these embankments, except for these with bottom culverts. The infiltration of water in the loess fill would decrease slope stability and cause a significant increase in settlement, which will be discussed in the next section.

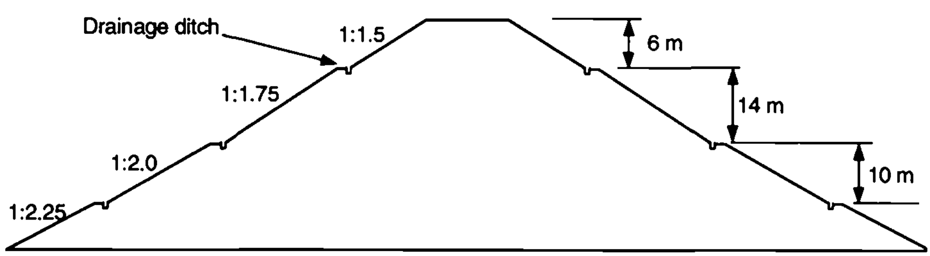

Figure 4 - *Standard Profile of Embankments Compacted According to ASTM Test Method D1557-91 (Modified Proctor)*

Table 3 - *Summary of Embankments in the State Road 312 Between K603 – K635*

Location	Height (m)	Slope	Storm water passage	Water proof measures
K603	28.0	-	Bottom ϕ3.0 m culvert	-
K608	50.0	-	Tunnel 30 m beneath crest	-
K615+120	25.6	1:1.0	Bottom ϕ3.0 m culvert	2 m lime stabilized surface
K619+068	46.8	1:1.75-1:2.5	Mid-level culvert	Geomembrane below 15 m
K621+787	40.0	1:1.75-1:2.5	Two ϕ1.8 m culverts	Vertical clay core
K624+470	49.0	1:1.75-1:2.0	Blank ditches	Vertical clay core
K629	30.0	1:1.50-1:1.75	Mid-level ϕ1.6 m culvert	None

Performance of Embankments

Settlement

The major problems of the embankments built during 1950 - 1985 were excessive settlement and difficulty in maintenance. The fill materials compacted according to the ASTM Test Method D698-91 were identified as slightly collapsible soil, which would collapse and settle due to water infiltration. Moreover, the fills were relatively permeable (coefficient of permeability $k \approx 5 \times 10^{-7}$ m/s) and the surface water could penetrate to a great depth. Thus, large settlement would be expected in the first one or two wet seasons (July to September). For instance, the middle of the Xiantou embankment settled 1.4 m, i.e. 2.6% of the embankment height, in the 8 years after the end of construction in 1984. Most settlement occurred in the second and third years after construction. Afterwards, settlement gradually ceased to develop.

The steep slopes also made it difficult to properly maintain the slopes. As a result, many embankments were eroded by storm water and their slopes became near-vertical cliffs in less than ten years after construction. Consequently, many of the embankments were not durable. In addition, extra settlements were induced due to stress rearrangement, as the side slope became steeper and steeper. For example, the Wangyan embankment was 60 m high and 8 m wide at the crest when built in 1977 (Figure 5). Gradually, the lower part of the sunny side slope was eroded into a cliff more than 50 m in height. The shady slope was also eroded to about 1:0.75 from the original 1:1.0 slope. Settlement started to develop in 1990 and accumulated to 1.5 m (2.5% of height) by 1992. This sudden increase in settlement indicated the initiation of progressive failure of the slope. In response, rehabilitation was started in 1992, with reinforcement of the shady slope at 1:1.25 for the upper 25 m and 1:1.5 for the lower 31 m, as shown in Figure 5.

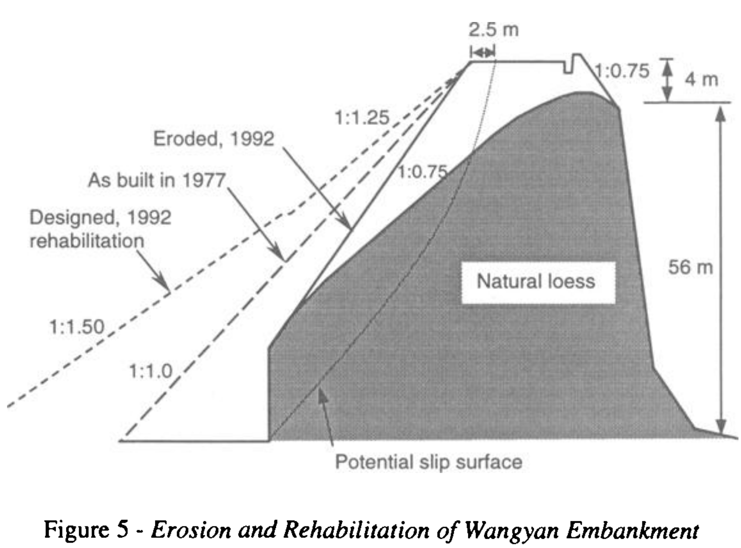

Figure 5 - *Erosion and Rehabilitation of Wangyan Embankment*

The above observed settlement is of the typical values for similar embankments. The Roadbed Construction Specifications (China Department of Transportation 1987) recommend a settlement of 2.5% of the height for 10 - 20 m high embankments. The Design Guidelines for Railways in Regional Soils (China Department of Railways 1992) recommend settlements of 0 ~ 2.5% of the height for embankments lower than 20 m and 1.0 ~ 1.5% for those higher than 20 m. Further, the suggested settlements are also influenced by local annual precipitation (Zuo 1988), as shown in Table 4.

Table 4 - *Suggested Embankment Settlement with Respect to Annual Precipitation (after Zuo 1988)*

Height of embankment (m)	Annual precipitation (mm)		
	< 300	300 ~ 500	> 500
6 ~ 12	3%	4%	5%
12 ~ 24	2 ~ 3%	3 ~ 4%	4 ~ 5%
> 24	1 ~ 2%	2 ~ 3%	3 ~ 4%

The new design and construction methods (China Department of Transportation 1987, 1991) employ the modified Proctor compaction method and adopt the improved embankment profile (Figure 4). The deformation modulus of the compacted soil increases whereas the coefficient of permeability decreases considerably to below 10^{-9} m/s. The surface fill affected by storms in turn reduces to a crust of about 2-m thick. Consequently, the embankment settlement is significantly reduced. For instance, the maximum crest settlement of Dukang embankment of 65 m high was 0.61 m, less than 1% of its height, three years after the end of construction.

It should be noted that settlement control for the new embankments was still based on the experience of the embankments built during 1950 ~ 1985. For instance, the embankments in Table 3 utilized compensation fills of 1.5% of the embankment height, which resulted in the typical crest settlement pattern shown in Figure 6. The maximum settlements occur at two shoulder locations while the central area elevates, which damages road serviceability. Such settlement distribution is due to the differences in the stress-strain behaviors of the fill and foundation materials. Particularly, the stiffness of the fills compacted with the modified Proctor method is greater than the natural soil stiffness, in contrary to the fills compacted according to the standard Proctor method.

Figure 7 illustrates the calculated settlement in the longitudinal section for a 30-m height embankment by Huang (1992) using a three-dimensional finite element method. The side slopes of the gully is 1:0.7. The calculation reveals rather uniform settlement across the gully at all elevations within the embankment, with the settlement at the central area (0.933 m maximum) slightly smaller than that at the side locations (0.954 m maximum). As such, the post-construction settlement configuration illustrated in Figure 6 would be produced, if the thickness of the compensation fill were designed proportional to height. According to the calculated settlement in Figure 7, uniform compensation fills would be more appropriate for design of high embankments that adopt the modified compaction standard.

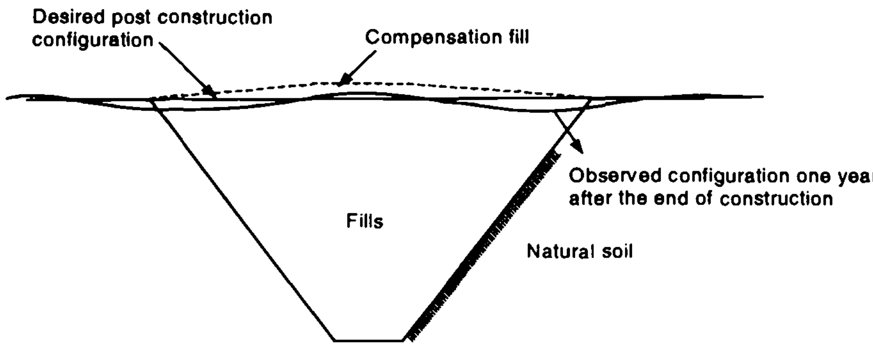

Figure 6 - *Observed Post-construction Settlement*

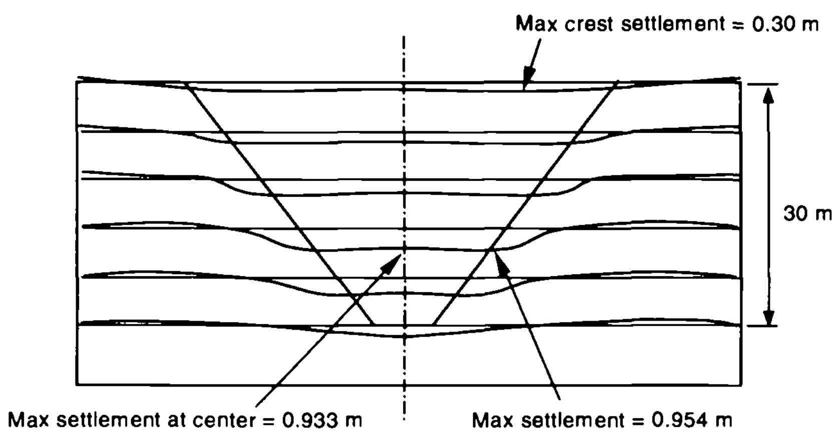

Figure 7 - *Predicted Settlement Distribution in Longitudinal Section (after Huang 1992)*

Stability

Stability analysis was carried out for three of the embankments in Table 2, built during 1950 - 1985. Table 5 lists the parameters and factors of safety of these embankments. Note that the parameters were obtained by direct shear tests using unsaturated samples corresponding to the construction moisture content (Shannxi Highway Design Institute 1960). Therefore, the cohesion should be considered as "apparent cohesion" that included the contribution of soil suction. The program REAME (Rotational equilibrium analysis of multi-layered embankments) by Huang (1982) was employed to perform the analysis using the simplified Bishop method.

Table 5 - *Stability Analysis of Embankments Built Using Standard Compaction*

Name	Location	Height (m)	Dry unit weight	w_{op}	Apparent cohesion (kPa)	Friction angle (degree)	Safety factor
Liujiagou	Yijun County	19.2	16.6	18.2	29.4	19.0	1.33
Wujiayan	Yijun County	42.1	16.3	19.7	39.2	31.0	1.31
Hanzhuan	Huanglin County	29.1	16.0	17.3	11.8	26.0	1.59

To investigate and compare the stability of embankments of different densities, samples of four different relative compactions, i.e., 0.94, 0.90, 0.85, and 0.80, were prepared and tested under saturated and optimum moisture conditions. Table 6 lists the peak strength parameters from consolidated undrained tests (Zhang et al. 1993). Table 7 summarizes the variations of safety factors with embankment height, slope, and upstream

Table 6 - *Friction Angle and Apparent Cohesion of Loess Samples Compacted to Four Densities (Total Stress Parameters)*

Dry density (kg/m^3)	1800		1720		1630		1530	
Degree of compaction	0.94		0.90		0.85		0.80	
Moisture state	Unsatur.	Satur.	Unsatur.	Satur.	Unsatur.	Satur.	Unsatur.	Satur.
Unit weight (kN/m^3)	19.85	21.03	19.01	20.53	17.93	19.97	16.86	19.34
Friction angle (°)	34.60	30.00	33.00	28.00	31.30	27.00	29.50	21.00
Apparent cohesion (kPa)	144.00	66.00	65.00	60.00	58.00	54.00	50.00	40.00

Table 7 - *Sensitivity Analysis of Stability of 30 m and 63.8 m Embankments*

Height of Embankment (m)	Strength parameters	Upstream pond depth	1:1.2	1:1.3	1:1.4	1:1.5
30.0	Foundation: c=30.4 kPa, φ=25°	H_w=0	1.95	1.99	2.03	2.08
	Fill: c=141.1 kPa, φ=34.6°	H_w=10.0 m	1.52	1.54	1.56	1.59
30.0	Foundation: c=30.4 kPa, φ=25°	H_w=0	1.78	-	-	-
	Fill: c=82.3 kPa, φ=34.6°					
30.0	Foundation: c=25.8 kPa, φ=21.3°	H_w=0	1.61	1.66	1.70	1.74
	Fill: c=119.9 kPa, φ=29.4°	H_w=10.0 m	1.27	1.29	1.32	1.32
		H_w=20.0 m	1.14	1.16	1.18	1.21
63.8	Foundation: c=30.4 kPa, φ=25°	H_w=0	1.60	1.64	1.69	1.73
	Fill: c=141.1 kPa, φ=34.6°	H_w=20.0 m	1.27	1.29	1.32	1.36
63.8	Foundation: c=25.8 kPa, φ=21.3°	H_w=0	1.33	1.37	1.40	1.46
	Fill: c=119.9 kPa, φ=29.4°	H_w=20.0 m	1.06	1.08	1.11	1.14
		H_w=40.0 m	0.93	0.95	0.97	1.00

storm water pond depth. Evident from the table is that the safety factor decreases substantially with an increase in the upstream water pond depth. Particularly, the 63.8-m high embankment would fail when it retains a water pond of 2/3 height of the embankment, if the strength parameters are reduced 15% from their peak values. Consequently, seepage control is an important issue in embankment design. In practice, no culverts are allowed to be located above the half height of the embankment.

Table 8 - *Comparisons of Embankments Compacted Uniformly at Relative Compactions of 0.94, 0.90, 0.85, and 0.80 (after Zhang et al. 1998)*

Test No.	Relative compaction γ_d/γ_{dmax}	Total height (m)	Slope	Time elapsed (year)	Upstream pond depth (m)	Slope water infiltration	Maximum settlement (m)	Maximum horizontal displacement (m)	Remark
M-0	0.94	63.80	1:1.50	4.63	No	No	0.260	0.033	
			1:1.50	7.15	23.4	Yes	0.360	0.048	
			1:1.20	8.70	23.4	Yes	0.400	0.059	
			1:1.00	10.25	23.4	Yes	0.650	0.078	
			1:0.63	13.35	23.4	Yes	4.200	3.053	Failed
M-2	0.90	63.80	1:1.75	4.05	No	No	0.574	0.149	
			1:1.75	5.60	23.4	No	0.574	0.319	
			1:1.50	7.15	23.4	No	0.681	0.425	
			1:1.20	8.70	23.4	No	0.681	0.468	
			1:0.75	10.25	23.4	No	0.711	0.915	
			1:0.75	10.75	23.4	Yes	0.830	-	Failed
M-3	0.85	63.80	1:1.75	4.05	No	No	1.212	0.170	
			1:1.75	5.60	23.4	No	1.425	0.808	
			1:1.50	7.15	23.4	No	1.638	0.915	Cracking
			1:1.20	8.70	23.4	No	1.851	1.042	Cracking
			1:1.00	10.25	23.4	No	2.425	1.360	Cracking
M-4	0.85	63.80	1:1.75	4.05	No	No	1.382	0.170	
			1:1.50	5.60	No	Yes	1.630	0.213	
			1:1.20	7.15	No	Yes	1.914	0.234	
			1:0.75	8.70	No	Yes	2.404	0.234	
			1:0.50	10.25	No	Yes	2.808	0.298	
M-9	0.80	63.80	1:1.75	4.05	No	No	2.446	0.213	
			1:1.75	5.60	23.4	No	3.233	0.808	
			1:1.50	7.15	23.4	No	3.744	1.080	
			1:1.20	8.70	23.4	No	3.999	1.170	

Extensive centrifuge tests were also carried out to investigate the stability and settlement of loess embankments. Table 8 summarizes five series of tests with varying upstream pond depths and a slope surface infiltration corresponding to the annual precipitation of 300 mm (Zhang et al. 1998). The embankments were compacted to 0.94, 0.90, 0.85, and 0.80, respectively, of the maximum density obtained from the modified Proctor tests. If the embankment is built with 0.94 relative compaction, the settlement would start to increase markedly at 1:1.0 slope, and the embankment would fail at 1:0.63 slope under the most unfavorable combinations. If compacted with the recommended relative compaction, 0.90, the embankment would fail at 1:0.75 slope. The relative compaction of 0.85 corresponds roughly to the standard compaction (dry density ρ_d = 1700 kg/m^3). At this density, the embankment would experience appreciable settlement, and the lateral displacement would accelerate at 1:1.0 slope if it retained a 23.4-m water pond (Model M-3). If the embankments were built at ridges (no water ponds), the settlement and lateral displacement would not increase significantly at a slope flatter than 1:0.75 and would sustain at slopes steeper than 1:0.50 even with the effect of the design storm infiltration (model M-4). This is in agreement with the field observations that embankments at ridges could be built with steep slopes such as 1:0.3 but would experience a significant settlement. Embankments steeper than 1:0.75 were considered marginally stable (e.g., Wangyan embankment using standard compaction). Moderate erosion on the slope would bring the embankment to the point at which both settlement and horizontal displacement would develop significantly with any further loss of slope fill. Cracks would thus develop that introduces more water into the fills and accelerates the process of slope failure.

Erosion Control

The design profile currently employed (Figure 4) is able to meet the settlement and stability requirements. However, it has an increased erosion exposure since the embankment slopes are much flatter than with older embankments. To reduce erosion of the slope surfaces, several berms are constructed on both sides of the embankment, and an open ditch is built on each berm to collect the surface runoff (Figure 4). The ditches have to be sealed properly to prevent any concentrated leakage.

A variety of biotechnical measures (grasses, trees) have also been tried for erosion control in addition to the ditches. However, these measures are not quite successful, because the vegetation develops poorly in the semi-arid area. Sound erosion control techniques for loess embankments remain to be developed.

Conclusions

Loess is a special silty soil with relatively high apparent cohesion. Embankments approximately 40 m high have been constructed with very steep slopes (up to 1:0.3) at ridges using the standard compaction method. However, such embankments experienced relatively large settlement and were not durable, since progressive failure would start with erosion. The interaction between loess fill and water, particularly the upstream storm-water ponds, proves to be the most critical factor affecting settlement and stability. An embankment that meets the settlement requirements of modern classified highways should be constructed using the modified Proctor method with approximately 0.90 relative compaction. The design profile currently employed is able to meet the settlement and stability requirements under the design surface water infiltration and storm ponds of half embankment height. For better serviceability, the compensation fill for settlement control should be uniform along the embankment, rather than proportional to the height.

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James R. Talbot[1]

Evaluation of Procedures Presented in TR-26 and TR-27 for Design and Construction of Earth Fills Using Soil Containing Oversize Rock Particles

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Reference: Talbot, J. R., **"Evaluation of Procedures Presented in TR-26 and TR-27 for Design and Construction of Earth Fills Using Soil Containing Oversize Rock Particles,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: In the early 1960's, the Soil Conservation Service (now Natural Resources Conservation Service) established some procedures for agency use in making laboratory and field tests for control of density and water content of compacted soil containing variable amounts of rock particles. The procedures included guidance for when the rock particles would undergo some breakdown during the compaction process. This paper summarizes the procedures, providing an awareness of them to interested parties, and compares them to those established by others. Although the procedures in TR-26 and TR-27 are less than definitive in some instances, the guidance regarding evaluation of breakdown and testing methods for these soils helps to satisfy the need for evaluating these soils. Further studies using the procedures in TR-26 and TR-27 may be useful in providing more usable techniques for design and construction testing of soils containing rock that is subject to particle breakdown during the compaction process.

Keywords: compaction of soil, compaction testing of soil containing rock, rock breakdown, non-durable rock, compaction testing.

Introduction

Some procedures for making laboratory and field tests for the control of density and water content of soils containing rock fragments larger than the No. 4 sieve were established for agency use in 1965 by the Soil Conservation Service (SCS) of the U.S. Department of Agriculture (presently the Natural Resources Conservation Service (NRCS)). The procedures established by the SCS were mainly for design and construction of small to medium-size earth dams which were being constructed in large numbers at that time for flood control under the Small Watershed Program of Public Law 566 (PL-566). Most dams constructed by SCS in this time period contained one or more zones of earth fill that generally consisted of fine-

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grained soils. Some dams were constructed of gravelly soils in the outer shell zones, while others were constructed of materials from alluvial river deposits or from rock materials excavated from formations in the abutments where spillways were excavated. These excavated rock formations and alluvial river deposits sometimes contained rock size particles up to 12 or 15 inches in diameter. Only a few of the dams designed and constructed by the SCS were rock fill dams which were similar in size and volume to some of the larger rock fill dams designed and constructed by other government agencies.

The density and water content testing procedures established by SCS in the 1960's for soils containing rock particles were specifically for the size and types of dams designed by the agency. The SCS procedures did not incorporate the use of very large diameter compaction molds and field-testing rings used by others. Some of the SCS dams were constructed in areas where the rock formations forming the abutment spillway areas were mainly softer rock such as shale, claystone, or siltstone. The testing procedures established by the SCS at this time were somewhat unique because they included some guidance on testing soils with rock fragments that undergo some disintegration (break down) during the process of constructing and compacting the fill. The procedures and guidance were included in SCS Engineering Division Publications, Technical Release No. 26 and Technical Release No. 27 (TR-26 and TR-27).

The title of TR-26 is "The Use of Soils Containing More Than 5 Percent Rock Larger Than the No. 4 Sieve." The purpose of TR-26 was to furnish guidance on procedures to be followed in sampling, testing, and construction control of soils to be used for earth fills which contain more than 5 percent of the particles larger than the No. 4 sieve. The main objective of TR-26 is proper construction control of rocky materials by determining the gradation and density that should be used in a testing program for various amounts and durability of the rock materials.

The title of TR-27 is "Laboratory and Field Test Procedures for Control of Density and Moisture of Compacted Earth Embankments." The purpose of TR-27 was to coordinate procedures used for controlling moisture and density of compacted earth materials in laboratory testing and field construction control. Test procedures (laboratory and field tests) for soil materials with no rock and various amounts of rock fragments are recommended in TR-27 for durable, moderately durable, and nondurable rock materials.

TR-26 and TR-27 were prepared on the premise that the moisture-density relationships and the resulting strength, compressibility and permeability of rocky soils after compaction in an embankment are considerably influenced by the amount and gradation of the rock fraction (that portion of the soil having particles larger than 4.75 mm or the No. 4 sieve). The final gradation of the material as placed depends on the resistance of the rock to breakdown and disintegration by mechanical manipulation and by natural weathering. Design values for strength, permeability, and compressibility should be based on tests made on specimens possessing the characteristics of the soil after it has been excavated, transported, spread, mixed, and compacted in an earth fill.

Description of Materials and Related Terms for which Guidance is Provided

The following descriptions and definitions are provided for the materials included in TR-26 and TR-27 and are shown in the summary tables of recommended testing requirements:

1. *Rock or Rock Fraction* - Those particles in the earth mass larger than the No. 4 sieve, consisting of unaltered fragments of mineral solids that have retained the structure and composition of natural geologic formations.
2. *Soil or Soil Fraction* - The portion of the earth mass smaller than the No. 4 sieve consisting of individual particles derived from physical and chemical weathering of rock and minerals.
3. *Oversize* - The rock portion consisting of particles larger than a certain size which were screened-off and not used in laboratory tests was referred to as "oversize." Oversize usually consists of those particles larger than the No. 4 sieve. For some testing situations, oversize may refer to that portion larger than the 3/4 inch sieve.
4. *Durable Rock* - The rock will not break down or disintegrate significantly during excavation and compaction operations or from the action of natural weathering processes. (Moh's hardness greater than 4).
5. *Moderately Durable Rock* - The rock will break down into smaller sizes during excavation and compaction operations, but can be separated from the soil by wet sieving methods without breaking down further. This rock includes moderately soft to hard sandy shales, siltstones, moderately weathered granoite, gneiss, shist, slate, cherty limestone, marble, etc. (Moh's hardness scale of plus 2, 3, or 4). The dry unit weight of the rock will generally be greater than 110 pcf. Pulverized rock materials will have low plasticity with plasticity index values less than 15.
6. *Nondurable Rock* - The rock breaks down easily into smaller size particles during excavating and compacting operations and is *not* sufficiently durable or stable under the action of water to be separated from the soil fraction without breaking down further. Each test which is made to determine the gradation of the soil requires some manipulation of the soil which, in turn, causes further breakdown of the rock into smaller sizes. In many cases, the rock is less dense and softer than the soil after compaction. Rock in this group includes very soft to soft plastic clay shales, highly weathered to moderately weathered clayey siltstones, soft limestones, schists, and etc., (Moh's hardness scale of 2 or less).
7. *Soil Group I* - Soils containing durable rock.
8. *Soil Group II* - Soils containing moderately durable rock.
9. *Soil Group III* - Soils containing non-durable rock.
10. *Soil Subgroup A* - Soils containing 65 percent or more fine fraction (material passing the No. 4 sieve).
11. *Soil Subgroup B* - Soils containing 35 to 65 percent fine fraction.
12. *Soil Subgroup C* - Material with less than 35 percent fine fraction.

Selection of Engineering Properties Values for Design

The engineering properties values discussed in TR-26 are shear strength, permeability, and compressibility. Two ways for determining engineering properties for design purposes are explained as follows:

- The preferred approach is to perform laboratory tests on specimens with gradation and density conforming to the values obtained from samples taken from test fills built by using the specified construction procedures. This approach is expensive and involves a time delay and arrangements required for constructing a test fill as part of the design.
- An alternate approach is to perform laboratory tests on specimens for which the gradation and density are established based on past experience with similar materials or on tests performed on potential borrow soils that may be altered by removal of some oversize materials to accommodate test equipment and procedures. This procedure requires evaluation of the gradation and density of the fill as it is being placed in order to determine the need for re-evaluating the design of the structure.

Since it is very difficult to simulate or estimate the amount of rock breakdown that may occur during the construction process, TR-26 suggests that when strength or permeability are critical, construction specifications may require that construction procedures be adjusted during the process of constructing the fill to produce an earth fill having the characteristics assumed for the design. This may be particularly important when materials with rock fractions subject to breakdown and disintegration are used in critical sections of the earth fill.

The problems or items referred to as requiring special consideration are as follows:

- The grading and density characteristics that should be used in laboratory test specimens for materials with rock larger than 3 inches.
- The grading and density characteristics that should be used in laboratory test specimens for materials with rocks that break down during placement and compaction of the fill.
- The testing methods that should be used for conducting embankment construction control testing for materials that contain durable and less than durable rock fragments.
- Investigating, sampling, and identifying materials containing rock particles that break down during construction.

Design and Construction Control Procedures

The basis for the guidance in TR-26 is that the rock fraction does not have a substantial influence on the engineering properties (shear strength, consolidation, and permeability) of the soil mass when the percent passing the No. 4 sieve is greater than 65 percent (less than 35 percent retained on the No. 4 sieve). The engineering properties of compacted materials with 35 percent to 65 percent passing the No. 4 sieve are significantly affected by both the compacted characteristics of the minus 4 fraction and the size, amount, gradation, and character of the plus 4 fraction. When materials are so coarse that less than 35 percent passes the No. 4 sieve, there are generally not enough finer particles to fill the voids and the engineering properties are related to placement conditions and overall density of the mass. The basis for selecting these percentages as the division between categories is not given in the documents and backup information was not found. It is assumed these percentages are based on the experience of the authors of TR-26.

TR-26 provides guidance for design and construction control for each group (Group I - Durable, Group II - Moderately Durable, and Group III - Nondurable) and for three ranges of rock fraction percentage (<35%, 35% to 65%, and >65%). Table 1 provides a summary of the recommended design and construction control procedures. All the details of the guidance in TR-26 is not provided in this paper. It is recommended that interested individuals obtain a copy of TR-26 from the NRCS for further study.

In Table 1, "Mass" refers to the measurement of the mass density (performing tests on the total soil including all the rock particles).

For soils containing durable rock (Group I soils) up to 35 percent retained on the No. 4 sieve, the testing and design is based on the soil fraction (that portion passing the No. 4 or the 3/4 inch sieve). All laboratory and field testing during construction is performed only on the soil fraction (that fraction of the fill material having a maximum size equal to that used in the compaction test method).

For Group I soils containing from 35 percent to 65 percent retained on the No. 4 sieve, the testing and design is based on the total mass, or in some cases, on the minus 3/4 inch fraction. The minus 3/4 inch fraction is used when the ASTM or other test method requires it. When the rock content is variable, the design is to be based on the rock content that is least desirable for the purpose of the fill. Construction control is specified as a certain number of passes with a specified size of roller (method specification) or by moisture and density testing of the minus 3/4 inch material. When moisture-density testing is performed, the control density should be determined on compaction tests made on material taken from the same location in the fill as the in place density tests.

TR-26 suggests that method specifications may be desirable for materials with plastic fines due to difficulty in making physical separation of particle sizes. However, a caution is given that close inspection and careful evaluation of methods of compaction and occasional embankment density tests be made to ensure fulfillment of minimum design requirements.

For Group I soils containing more than 65 percent durable rock (less than 35 percent passing the No. 4 sieve), laboratory testing is not recommended. The design is based on engineering properties of the mass developed by special field tests or by correlative experience with similar materials. Compaction is usually by method specifications (number of passes of a specified size and type of roller).

For soils containing moderately durable rock (Group II soils), the coarse particles will break down during normal placement and compaction, but the rock fractions are hard and resistant enough to allow physical separation of various particle sizes without significant breakdown. Design and compaction control depends on the degree of breakdown that has occurred after compaction. For Group II soils containing more than 35 percent passing the No. 4 sieve after compaction, test fills are recommended to determine the breakdown and to determine the properties of the fill for important zones of critical structures. For less important zones or noncritical fills, design is based on the compacted density of the mass. In this case, the amount of processing of the fill material may need to be controlled so that the amount of breakdown in the constructed fill matches the breakdown assumed in design.

TABLE 1 - *Design and Construction Testing Requirements for Soils Containing Particles Larger than No. 4 Sieve*

Rock Fraction Durability	Soil Fraction Plasticity	Gradation	Construction Control	Testing	Design and Specifications	Sample Size
Group I - Rock is Durable, Hard, Water Stable	Low Plasticity Soil Fraction	65% or more passing #4	< #4	< #4	< #4	50 lbs.
		35%-65% retained #4	Mass or < 3/4"	Mass or < 3/4"	Mass or < 3/4"	250 lbs.
		<35% passing #4	Method that produces desired density	Usually none	Mass Density	250 lbs.
	Plastic Soil Fraction	Same as low plasticity matrices except that density tests may be limited when gravel contents approach 35 percent				
Group II - Rock is Moderately Durable and can be separated from soil		65% or more < #4	< #4	< #4	< #4	300 lbs.
		35%-65% retained #4	Mass - (Breakdown if specified)	Mass	Mass (Breakdown if needed)	
		<35% passing #4	Method that produces desired density	Usually none	Mass density	300 lbs. (If required)
Group III - Soft Rock, Unstable in Water			Mass Density	Mass Density	Mass Density	150 lbs.

For Group II soils with less than 35 percent passing the No. 4 sieve after compaction, the design will be based upon engineering properties of the mass as determined from special tests and correlative experience. Method specifications will be used with some mass density tests to ensure the desired density has be obtained.

For all Group III soils, test fills are highly recommended before design unless engineers have documented experience for the soil or for very similar materials. The test fills will provide information for evaluating breakdown and compaction characteristics. Undisturbed samples can usually be obtained from the test fills for determining design properties and construction specifications. Mass density tests which compare the constructed fill density to the test fill density are recommended for construction control.

TR-26 also provides requirements for investigation and sampling including the information to be observed or determined and the sampling requirements for soil materials containing durable, moderately durable, and nondurable rock fragments. The observations include the geologic source and history of the materials, the maximum size particles, the percentage passing the 6 inch, 3 inch, 1-1/2 inch, 3/4 inch, and No. 4 sieves, and any experience or history of the use of the materials in engineering structures. Other items to be identified about the material include hardness, plasticity, bulk density, structure, texture, porosity and permeability (estimated), water stability (amount of slaking in water), mineralogy, acidity, soluble salt content, and an estimate of the amount of breakdown expected.

The investigation requirements further identify the size of samples to be taken for laboratory testing and evaluation of the material performance based on studies, experience with similar materials, test fills with undisturbed cores, and testing or observations made on the samples obtained. Specific methods for performing an investigation, making observations, collecting samples, making field tests, and determining the amount of breakdown during a test fill operation are included in an example in TR-26.

Test Procedures for Control of Density and Water Content of Compacted Earth Embankments

The main guidance provided by TR-27 deals with field compaction control of materials containing more than 35 percent rock fragments. The specific ASTM tests procedures to be used for various percentages of rock fragments are provided and guidance on their use given. Test procedures in SCS test procedures (Soil Conservation Service 1963) are also referred to. Guidance on the compaction method used to determine design values and field construction control are given for each material (soil group and percent rock fragments). The tests referenced are ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (D 698), ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (D 1557), the Sand Cone Test, the Rubber Balloon Test, the Calibrated Cylinder Test, the Rapid Method of Compaction Control Test, the Oven Drying Test, the Quick Dry (Direct Heating), the Drying by Alcohol Burning Test, and water content by the Speedy Moisture Meter Test. SCS test procedures (Soil Conservation Service 1963) are referenced for most of these tests.

A template (metal ring or rectangular frame) and plastic liner used with water or sand are recommended for the most soils containing more than 35 percent rock fragments and having a maximum particle size of over 4 inches. The plastic liner is generally always used with water to prevent the water from infiltrating into the compacted soil, but is also recommended for soils containing coarser rock particles to reduce the potential for sand to migrate into voids between rock particles. These tests are similar to the present ASTM Test Method for Determination of Density of Soil In Place by the Sand Replacement Method in a Test Pit (D 4914) and ASTM Test Method for Determination of Density of Soil In Place by the Water Replacement Method in a Test Pit (D 5030). The guidance stresses of the percent compaction and deviation from optimum water content are to be determined by comparing the in-place density and water content with a compaction test made on material taken from the fill near the in-place test. The in place test and the compaction test are to be made on materials having the same maximum particle size (if oversize is removed for one test, it must be removed for the other test also).

Table 2 provides a summary of the guidance on the methods recommended for the various soil groups and the maximum particle sizes commonly found. The guidance is specific to the point that the compaction control test method, test hole volume, and test hole dimensions are given. Details on these testing procedures are outlined further in TR- 27 which is available from the NRCS.

Comparison to Current ASTM Test Methods and Methods Used by Others

A summary of testing methods for constructing embankments from soils containing large particles used by the U.S. Army Corps of Engineers (USACE), the U.S. Bureau of Reclamation (USBR), and the California Department of Water Resources (CDWR) was published in 1988 (United States Committee on Large Dams 1988). These organizations developed the first field density testing procedures for soils containing rock. The United States Committee on Large Dams reference of 1988 gives a history of this development work and provides descriptions of the tests and the large equipment recommended for performing the tests.

Rockfills containing boulders up to 24 or 36 inches in diameter are common in the USACE work. The USACE test procedures (U.S. Department of the Army 1977, 1982, 1969, and 1972) include field density tests using rings up to 6 feet in diameter. Generally, the USACE tests do not involve guidance on testing material containing rock that breaks down during construction.

Most of the tests on material containing rock fragments developed by the CDWR resulted from the construction of Oroville Dam between about 1959 and 1965. All the materials in Oroville Dam included a considerable percentage of material coarser than the No. 4 sieve. Even the impervious core materials contained over 50 percent by weight larger than the No. 4 sieve. Field density tests on the pervious (shell) zones were accomplished using 4-foot and 6-foot-diameter rings with plastic liners and water used to measure the volume of the density samples taken from the fill. On Oroville Dam, the laboratory density was determined in a 27-inch-diameter by 30-inch-high mold. Other laboratory test apparatus used by the CDWR for Cedar Springs, Del Valle, and Pyramid Dams to determine the maximum density included 6-foot-diameter by 6-foot-high manhole rings. CDWR reports and manuals (California Department of Water Resources 1964a, 1962a, 1962b, 1968, and 1964b) give the details of the tests developed.

TABLE 2 IN PLACE DENSITY TEST REQUIREMENTS

Soil Group	Percent Rock (+ No. 4)	Maximum Particle Size in Mass (Dia)	Compaction Control	In Place Density Tests				
				Method	Test Hole Vol. (Cu. Ft.)	Test Hole Dimensions		
						Top (in)	Bottom (in)	Depth (in)
IA IIA	< 35	2"	< No. 4	6½" sand cone	0.10	6.5	4	8
		2" - 4"	< No. 4	12" sand cone	0.40	12	8	12
IB IIB	35 - 65	2"		6½" sand cone	0.10	6.5	4	8
		2" - 4"	Group IB generally controlled on < 3/4" or mass	12" sand cone or 12" template w/plastic liner	0.40	12	8	12
		4" - 6"		24" to 30" template with plastic liner	1.00 to 2.00	24 to 30	12 to 18	12
		6" - 12"	Group IIB generally controlled on mass	48" template w/ plastic liner	10.0 to 14.0	48	24	18
IC IIC	>65	4"	Mass & Method	12" sand cone or 12" template w/plastic liner	0.2 to 0.4	12	Variable	12
		6"	Mass & Method	30" template w/plastic liner	1.00 to 2.00	30	Variable	12
		12"	Mass & Method	48" template w/ plastic liner	10.0 to 14.00	48	Variable	18
		15"	Mass & Method	72" template w/ plastic liner	15.0 to 18.0	72	Variable	18
III	5 - 65	Indeterminate	Mass	6½" sand cone	0.10	6.5	4	8

Prior to 1933, the USBR controlled compaction of materials containing a substantial amount of rock particles using performance or method specifications which specified a lift thickness, type and size of compacting equipment, and the number of passes. The USBR completed a study of compaction characteristics of gravely soils in 1957 (U.S. Bureau of Reclamation 1957) wherein a large compaction device was developed for testing material up to 3 inches in diameter. Since the 1957 study, the USBR has used various dam construction projects to develop a rather sophisticated testing program for testing rock fills or soils containing rock particles. Their procedures (U.S. Bureau of Reclamation 1990) include detailed procedures using the sand cone, as well as ring or rectangular templates ranging from 2-feet square to 9-feet in diameter for testing in place density of fills.

The previous cited test procedures are generally for durable rock and do not involve guidance on testing material containing rock that breaks down during construction. These organizations may have studied compaction of soils containing rock that is less than durable; however, the test procedures generally are not specific to this problem.

Evaluation of Procedures Presented in TR-26 and TR-27

There are no reports cited in TR-26 and TR-27 relative to studies upon which the guidance is based. It is possible the guidance was based on judgment from experience in constructing with shales and other less durable materials. These documents may have been prepared to provide some guidance based on experience until further studies could be made. The use of test fills to evaluate the amount of breakdown and to provide some indications of engineering properties for soils containing rock that breaks down seems to be sound advice.

The SCS guidance in TR-26 and TR-27 does not go into the details of making large laboratory tests to determine the maximum density for comparing with the in place density of the earth fill. The guidance recommends large template with liner and water or sand for determining the in place density of the fill. The current ASTM standards or the test details of other organizations can be used to perform these tests. The SCS guidance is somewhat vague or lacking as to how the in place density test results are to be used to determine that sufficient compaction has been achieved.

SCS guidance in TR-26 and TR-27 advocates the use of test fills for those soils that contain rock particles that are moderately durable or nondurable. Test fills can be used to demonstrate the amount of breakdown to expect. Undisturbed samples can be obtained from test fills and specimens obtained for laboratory testing of shear strength, compressibility, or permeability provided the rock fragments are not too large for trimming the specimens or performing the tests. If the rock particles are too large for performing tests, some visual observations can be made and engineering judgments made relative to the compaction procedures and the resulting engineering properties.

The procedures presented in TR-26 and TR-27 received very limited use in SCS. The equipment required for making the large tests is expensive and the tests are time consuming. The resources of most field construction offices were not adequate to use these methods. The procedures were perceived as not clear and definitive in some cases. This may have caused some confusion among field engineers regarding the procedures to be used for certain conditions.

The SCS construction office in West Virginia purchased equipment for making large in place density tests, including several circular, steel templates ranging from 30 inches to 72 inches in diameter, a truck equipped with a hoist for lifting heavy items on and off the truck, tanks with meters for measuring water, jack hammers for use in excavating rocky soils, and barrels for use as soil containers. This equipment was used for design and construction testing on several flood control dams constructed in West Virginia using soils containing shale rock or river deposits with appreciable gravel, cobble, and boulder size particles. At least two other state construction offices of SCS borrowed the West Virginia SCS truck and equipment for making tests on rocky soils used in earth dam construction. The use of the procedures in these instances was successful; however, most SCS offices do not have the resources in personnel and equipment to use the procedures more extensively.

Conclusions and Recommendations

Although the procedures Presented in TR-26 and TR-27 were not used extensively by SCS over the years, they were used successfully on several flood control dam projects in at least three states. A lack of personnel and equipment resources is likely the main reason these procedures were not used more extensively. There may have also been some confusion resulting from the lack of specific details on when the procedures are to be applied and how to use the test results to determine compliance with compaction requirements for an earth fill. These shortcomings may have caused some field engineers to avoid using the procedures.

TR-26 and TR-27 contain some guidance on testing soil containing rock fragments that breakdown to smaller particles during the excavation, conditioning, and compaction processes which are generally not included in procedures published by others. Since TR-26 and TR-27 are internal documents for agency use, many researchers and users of compaction testing procedures are not aware of them. TR-26 and TR-27 may provide some information and help to researchers and others who may be developing procedures for testing and using soil containing rock fragments in compacted earth fills. Further studies using test fills and other guidance are recommended to further verify the procedures and to develop more definitive requirements.

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Comparison of Laboratory Data and Field Performance for Fills Subject to Hydrocompression

Reference: Kropp, A. L. and McMahon, D. J., "**Comparison of Laboratory Data and Field Performance for Fills Subject to Hydrocompression,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Over 120 laboratory response-to-wetting (hydrocompression) tests on engineered fill samples using various procedures were compiled from the literature and from private files of geotechnical engineers in the State of California. These data were grouped into three categories according to the level of relative compaction used for the fill; these categories were 85 to 89%, 90 to 91%, and 92 to 95% relative compaction. In addition, field performance data from nine sites were compiled, which recorded the magnitude of hydrocompression in deep fills compacted with a minimum 90% relative compaction specification. Comparison of the magnitude of hydrocompression settlements calculated from the laboratory testing with the recorded values at project sites showed general agreement.

Keywords: deep fills, hydrocompression, field performance, compaction specifications

Introduction

Residential development in California in the last couple of decades has frequently resulted in mass grading of hillside areas, often involving the removal of ridges and hilltops and the filling of canyons and valleys to produce relatively flat building lots. The fills created by these operations have been up to about 120 feet deep, a significant increase in the depth of fill compared to older development, which generally occurred in flatter terrain. The compaction specifications for these fills commonly consisted of a minimum of 90% relative compaction based on laboratory maximum densities determined by ASTM Test Method (D 1557). During the 1980's, significant distress was observed in a number of the structures built on relatively deep canyon fills. This distress was often attributed to post-construction wetting of the fill due to infiltration of rainfall, and more importantly, landscape watering. In some cases, the wetting was aggravated by

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breaks in paving, drains, water supply lines and sewer pipes that were part of the distress. This wetting resulted in compression within the deeper portions of the fill; and the behavior was termed hydrocompression. The costs of repairs for some sites have been estimated to be nearly $100 000 000 (Lawton et al. 1992).

Although geotechnical engineers involved in the construction of earth dams were aware of the problem of settlement of deep fills due to hydrocompression (Nobari 1968, Nobari and Duncan 1972, Leononds and Davidson 1984), the practicing geotechnical engineering community working on residential fills did not seem to recognize the potential for problems until the publication of the landmark case history paper by Brandon et al. in 1990. Some earlier papers had dealt with this issue (for example, Cox 1987, Nwabuokei and Lovell 1986, and Lawton et al. 1989), and a focused research proposal to explore specifically the compressibility of compacted fills was circulated in 1988 (Noorany 1988), but this issue did not seem to be widely recognized within the practicing engineering community in California.

Studies of collapsible natural soils were identifying the mechanism of collapse (Houston et al. 1988, El-Ehwary and Houston 1990), but these publications did not specifically address compacted fills. However, with the publication of the Brandon et al. (1990) paper, it appears that the general geotechnical engineering community began to appreciate that potentially serious problems could be posed by hydrocompression within compacted fills. Following this publication, a variety of excellent papers have been published dealing with this issue and are listed in the references. A nice review of the history of the literature is given by Rogers (1998). Disagreements about whether to run tests with loading after wetting (Lawton et al. 1991) or wetting after loading (Noorany 1992) seem to have been resolved by combining the two test methods to obtain the maximum information per testing dollar spent (Houston and Houston 1997).

In 1995, the authors conducted a survey of geotechnical engineering design companies throughout the State of California regarding hydrocompression issues. Thirty-four firms completed a four-page survey, with some choosing the option to remain anonymous. These materials, in conjunction with in-house file materials from the firm that employs the authors, provided an important extension of hydrocompression data presented in the literature. Materials from these sources, in conjunction with the technical literature and personal communications, were utilized in this comparative study.

Laboratory Test Data

The results of 123 response-to-wetting tests from 12 sources were compiled to illustrate the hydrocompression behavior of soils placed as engineered fill and compacted to varying levels of relative compaction. Ten data sets were obtained from the geotechnical literature (Lawton et al. 1989, Brandon et al. 1990, Noorany 1990, Stark and Bixby 1990, Alwail et al.1992, Lawton et al. 1992, Rogers 1992, Kropp et al. 1994, Vicente et al. 1994, and Lamb and Hourihan 1995) and four were obtained in response to a survey of engineers practicing in California. It should be noted that 62 of these tests, or roughly half of the entire data set, are from one source (Noorany 1990). All of the 123 laboratory response-to-wetting tests are presented in Figures 1a to 1c. The tests have been separated into ranges of relative compaction, including 85 to 89% relative compaction (Figure 1a), 90 to 91% relative compaction (Figure 1b), and 92 to 95%

relative compaction (Figure 1c). In addition, the average for the data on each curve is presented and a composite of all of averages for each relative compaction interval is presented in Figure 1d. These data represent a very broad range of soil types ranging from gravels to expansive clays, so there is a significant overlap in the response-to-wetting behavior for the data points in each range of relative compaction. Unfortunately there was insufficient data to present individual relative compaction ranges for each soil type. Even with this broad overlay, Figure 1d shows the clear trend that higher densities achieved by higher relative compaction standards generally reduce hydrocompression while at the same time increasing the amount of swell that occurs near the surface. It should also be noted that these laboratory data have been presented without regard to% saturation, although some of the data sources did include study of the effects of water content.

Field Data

Ground surface movements related to hydrocompression or swell in response to wetting at 8 sites in California have been compiled in Figures 2a to 2d. As illustrated on Table 1, seven of these sites appear in the geotechnical literature. Data from the remaining site plus additional data for some of the same sites were submitted in response to the authors' survey of practice in California (Anonymous 1995, Eliahu 1995, Lamb 1995, Moran 1995, Shires 1995, and Vicente 1995). The compaction specifications for these sites generally consisted of a minimum of 90% relative compaction, and some also specified a minimum water content. Similar magnitudes of settlement were recorded for all of the Southern California sites (Figures 2a and 2b) while the Northern California sites show significantly different magnitudes (Figures 2c and 2d). The site with data shown in Figure 2c generally consisted of sandy and gravelly fill compacted dry of optimum water content and may represent a worst case for potential settlement. The Northern California sites represented in Figure 2d typically contained expansive clay fill materials and hydroswell tends to dominate the response to wetting behavior.

Table 1 – *Data for eight sites with damaging fill movements*

SITE #	REFERENCE	USCS	YEARS AFTER CONSTRUCTION	SETTLEMENT (IN)	THICKNESS OF FILL (FT)
1	Kropp et al. (1994)	GM, GP, SM	3	18	85
2	Brandon et al. (1990)	CL	15	18	100
3	Brandon et al. (1990)	CL	15	18	100
4	Vicente et al. (1994), Vicente (1995)	MH, CH	9	18	150
5	Confidential N. Cal. Site	CH	12	?	?
6	Lamb & Hourihan (1995)	SC, CL	5	4.5	35
7	Lamb & Hourihan (1995)	SC, CL	12	5.7	70
8	Lamb & Hourihan (1995)	SC, CL	11	10.4	85

It should be noted that the data from Kropp et al. and from Vicente et al. have been corrected for surface settlement that occurred prior to the initiation of settlement monitoring. In the case of the data from Kropp et al. 1994, one of the affected condominium buildings crossed a cut-fill contact. In this case, floor survey data was used to estimate surface settlements of 6 inches in 60 feet of fill prior to the commencement of survey monitoring. In the case of Vicente et al. 1994, data was available from several other consultants for nearby fills constructed at the same time with the same soils. These data were used to estimate surface settlement of 18 inches in 150 feet of fill prior to the commencement of survey monitoring. In both cases these corrections were applied only to the deeper portions of the fill that were subject to hydrocompression (as determined by laboratory testing). A squared function was used in the correction because this approximated the measured laboratory behavior in the range of interest.

Comparison of Laboratory and Field Data

Several investigators have performed laboratory testing for swell/hydrocompression on samples from sites where significant ground movements have been measured (Brandon et al. 1990, Noorany et al. 1992, Noorany and Stanley 1994, Vicente et al. 1994, and Kropp et al. 1994). These tests were usually performed in accordance with ASTM Test D5333, Standard Test Method for Measurement of Collapse Potential of Soils, although this test was not formally adopted until 1992. In each of these cases relatively good correlation was obtained between the predictions of movement from the laboratory testing and the actual movements recorded (refer to references for more detail).

To evaluate the more general case of the wide range of soil types represented by the 123 laboratory tests, we have compiled all of the laboratory and field data from Figure 1 and Figure 2, in Figure 3. To compare the two data sets, the ground surface settlements predicted by the average strain (the average line for each range of compaction shown in Figure 1d) were used to calculate predicted averages of movement, plotted as lines in Figure 3. The surface movement was calculated using the procedure described by Nwabuokei and Lovell (1986). This involves determination of representative vertical strains throughout the profile by laboratory tests, and then integration of these strains over the full depth of the fill to predict the movement at the ground surface.

In general there is good agreement between the movements predicted by the averages and the field data shown in Figure 3. While this comparison between lab and field data appears to support suggestions that relative compactions in the field were sometimes less than the 90% relative compaction specified (Pradel et al. 1992), it should be noted that there is significant scatter for any given range of relative compaction, and significant overlap between the data for different ranges of relative compaction. The scatter and overlap are due to the wide range of soils included in the data shown in Figure 1a to 1c. Each case history has a better correlation between its laboratory data and field data, as would be expected. Relative compactions of less than 90% are not required to explain the observed movements.

As shown in Figure 3, increasing the relative compaction to 92 to 95% increases the swell near the surface and decreases the hydrocompression in the deeper portions of the fill. Decreasing the relative compaction to 90% decreases the swell near the surface, but increases hydrocompression-induced settlements at depth. In both cases, differential movements occur between the shallower and deeper portions of the fill. Changing the relative compaction alone does not eliminate the potential for significant differential movements and associated damage.

The differential movements beneath a proposed building can be minimized by maintaining a relatively uniform fill thickness beneath any single structure, by using different compaction specifications for deeper and near-surface portions of the fill, or by wetting the fill prior to constructing any buildings. Alternatively, foundations can be designed to accept or resist larger differential movements. The need for any of these measures can be reduced by using compaction criteria based on percent saturation rather than on relative compaction, which is discussed in a companion paper in this publication (McMahon and Kropp, 2000).

Conclusions

There is in general good agreement between predicted hydrocompression-induced settlements based on laboratory response-to-wetting testing and observed field performance. This suggests that current laboratory testing techniques combined with current analytical techniques are adequate to describe hydrocompression-induced settlements. However, the hydrocompression-induced settlements can be unacceptably high, even when the minimum acceptable compaction is increased to 92 to 95% (without regard to percent saturation). Increasing the minimum relative compaction alone may not be adequate to eliminate unacceptably-high differential settlements in deep fills supporting residential structures.

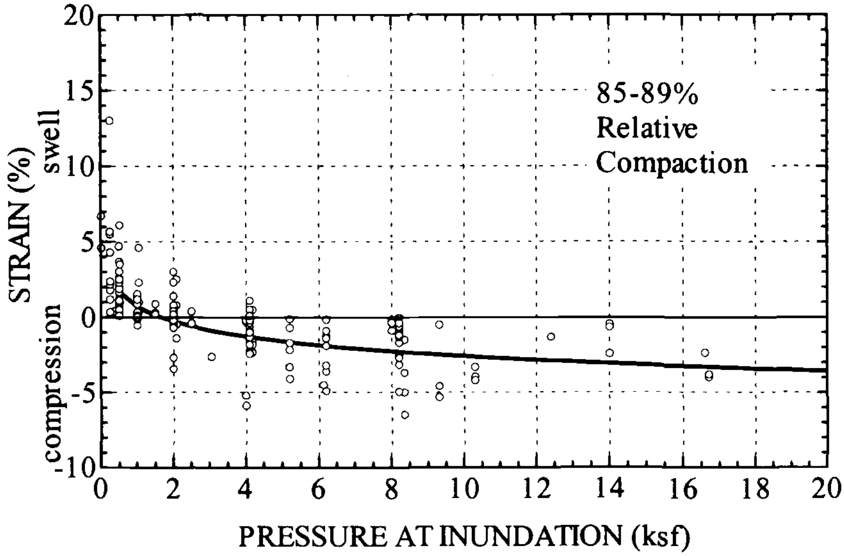

Figure 1a - *Laboratory response-to-wetting data and average for 85-89% relative compaction.*

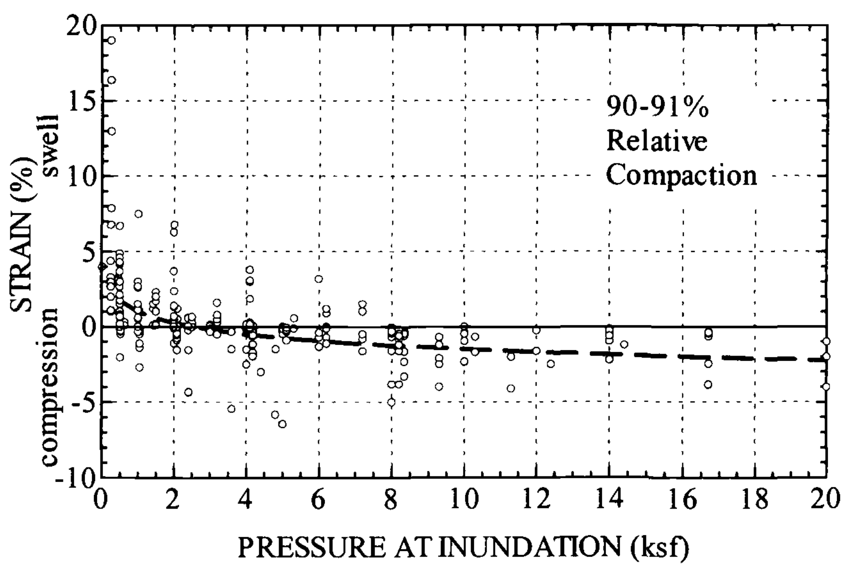

Figure 1b - *Laboratory response-to-wetting data and average for 90-91% relative compaction.*

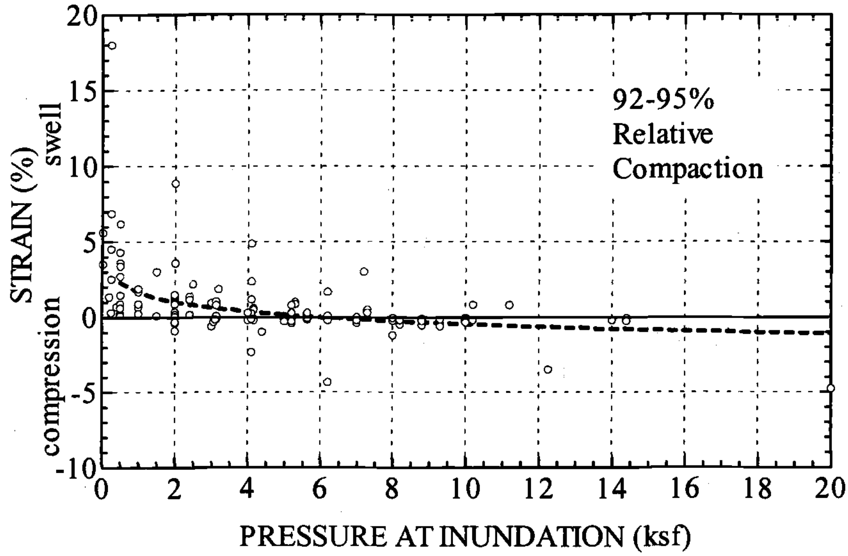

Figure 1c - *Laboratory response-to-wetting data and average for 92-95% relative compaction.*

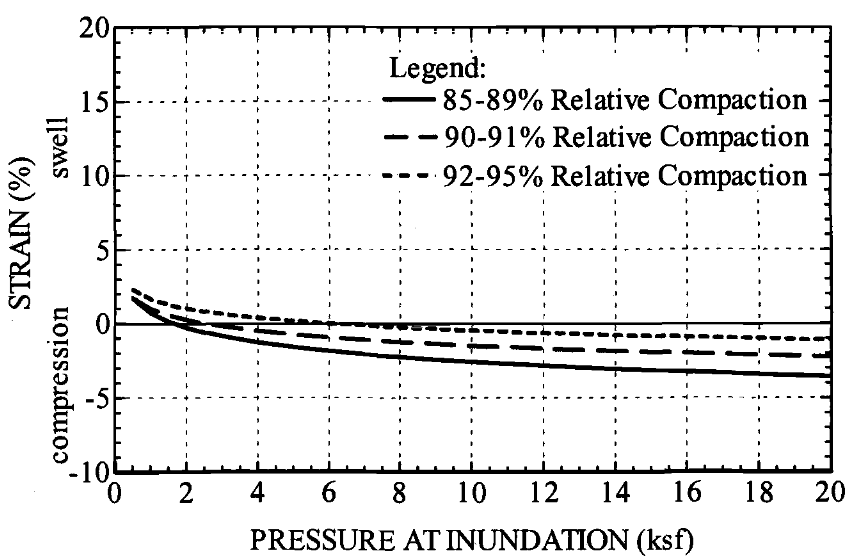

Figure 1d - *Comparison of averages for 85-89%, 90-91% and 92-95% relative compaction.*

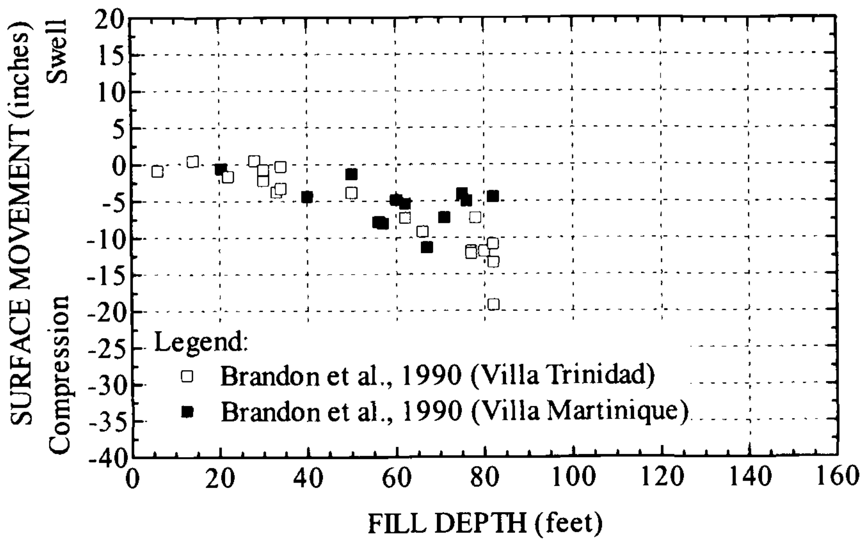

Figure 2a - *Measured surface movement data for two Southern California sites.*

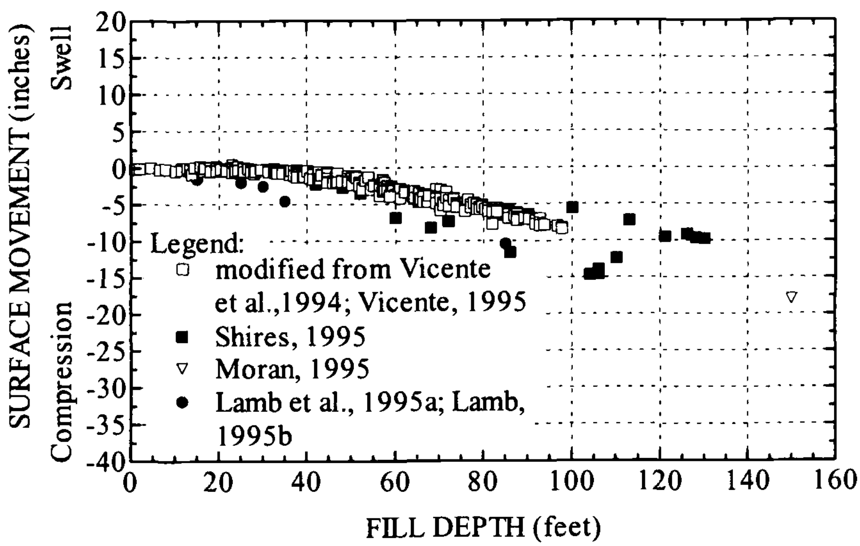

Figure 2b - *Measured surface movement for several Southern California sites.*

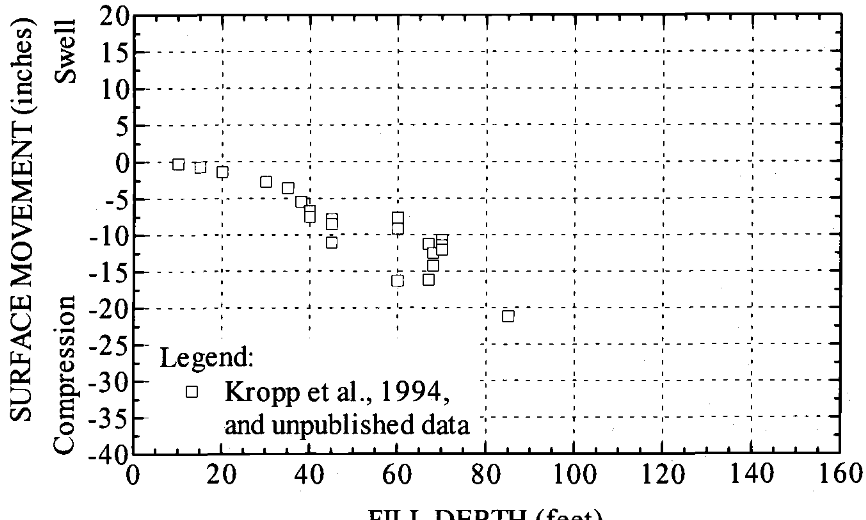

Figure 2c - *Measured surface movement data for a Northern California site.*

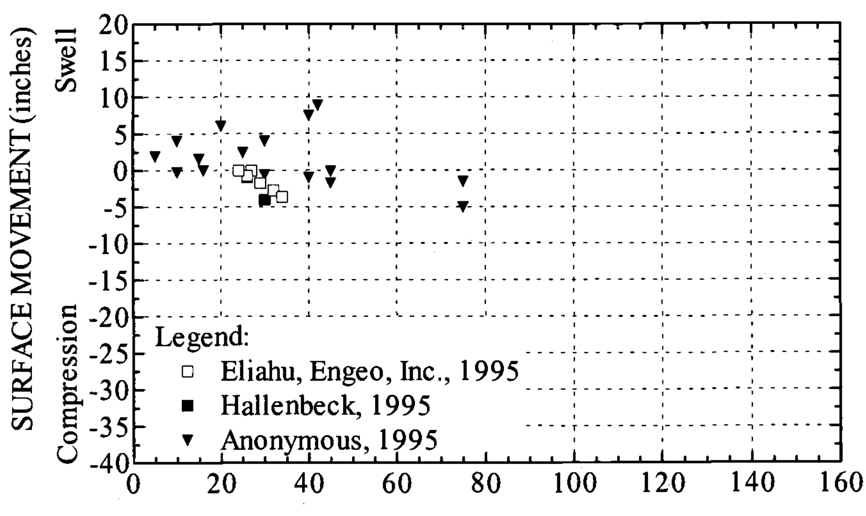

Figure 2d - *Measured surface movement data for several Northern California sites.*

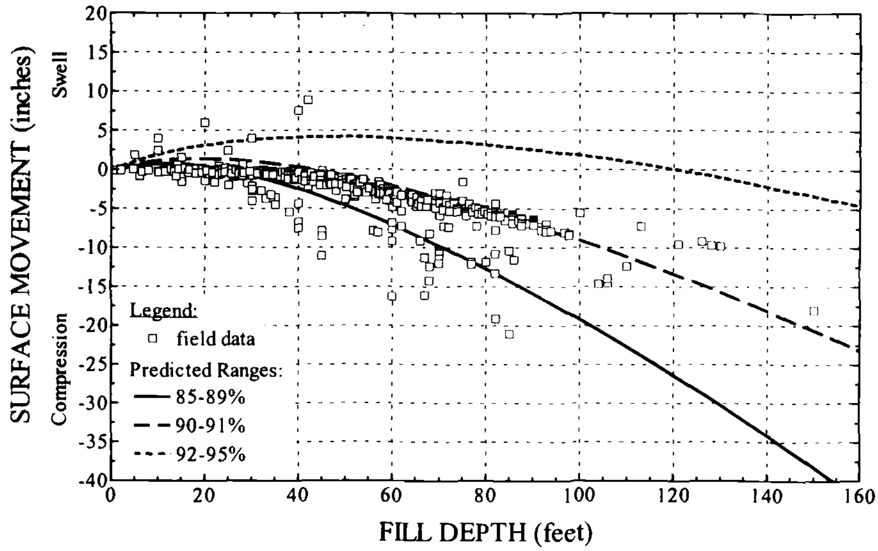

Figure 3 - *Comparison of the field data shown in Figure 2a to 2d with the predicted hyrdrocompression-induced settlements for the average response to wetting data for each of the ranges of relative compaction shown in Figures 1a to 1c.*

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Proposed Compaction Specifications to Minimize Hydrocompression-Induced Settlements in Fills Supporting Residential Structures

Reference: McMahon, D. J. and Kropp, A. L., "**Proposed Compaction Specifications to Minimize Hydrocompression-Induced Settlements in Fills Supporting Residential Structures,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Laboratory compaction tests, field compaction tests, response-to-wetting tests and case history data are examined in terms of percent saturation in order to help design engineers reduce hydrocompression-induced settlements in fills caused by post-construction wetting. Over 900 laboratory compaction tests (ASTM D1557) are examined to show that the maximum dry density is, on average, achieved at about 85% saturation, and that the line of optimums derived from compaction tests with lower compactive effort is roughly parallel to the 85% saturation line. Laboratory response-to-wetting tests and controlled wetting tests are used to show that the largest portion of hydrocompression-induced settlements occur below about 85% saturation. Case history data is used to illustrate that the percent saturation typically achieved during construction of residential fills is as low as 50%, and averages about 60%. Post-construction wetting is shown to commonly increase this value to as high as 95%, with an average of about 80%. This wetting can result in potentially damaging hydrocompression-induced settlement within the fills. Based upon this understanding of the importance of percent saturation, a new type of compaction specification is proposed to reduce hydrocompression-induced settlements in fills by specifying a zone of acceptable dry density/water content combinations based on achieving a minimum percent saturation. This new specification is also shown to have the advantage of providing a wider range of acceptable density and water contents (compared to increasing the minimum density or increasing the water content). The proposed specifications based on percent saturation are shown to make construction more practical than specifications based solely on density and water content. These specifications can be easily implemented by field technicians, greatly reducing the potential for damaging hydrocompression.

Keywords: deep fills, hydrocompression, compaction specifications, saturation

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Introduction

The scale of mass grading in hillside areas for residential development in California has increased over time, resulting in more frequent cases of deeper fills having differential movements that damage structures. Hillside grading frequently involves the removal of ridges and hilltops and the filling of canyons and valleys to produce relatively flat lots for residential development. Typical compaction specifications for these residential fills in California consist of a minimum of 90 to 95% relative compaction, based on laboratory maximum densities determined by D1557. Some specifications also include a minimum water content, for example, at or above the optimum water content, as determined by the same test. Fills can be relatively deep (up to 150 ft) and are often of non-uniform thickness, particularly as they transition from fill to cut. Residential structures built on these non-uniform fills can be subjected to large total and differential movements. The causes of the differential movements include swell of near-surface fill soils and cut areas containing expansive silts and clays, and settlement of the deeper portions of the fill, which can occur for all soil types. The swell (sometimes called hydroswell) and settlement (termed hydrocompression) occur in response to wetting of the fill mass, which frequently occurs months and years after the completion of construction. Post-construction wetting occurs due to infiltration of rainfall and, most importantly, landscape watering. In extreme cases, the differential movements can cause breaks in paving, drains, water supply pipes and sewer pipes, resulting in additional localized wetting.

The depths of residential fills have increased over the years, reaching depths that historically have only been employed for the construction of earth dams or freeway embankments. Earth dams have been shown to have significant post-construction hydrocompression-induced settlements as portions of the dam are wetted during filling of the reservoir (Nobari 1968, Nobari and Duncan 1972). Not surprisingly, the post-construction movements due to hydrocompression seen in fills constructed for residential development have been similar in magnitude to those observed in dams. Unfortunately, typical residential structures are more sensitive to differential settlement than earth dams. While earth dams may settle several feet without distress to the dam, residential structures are typically designed to withstand movements of only inches. In California it is common to observe cosmetic and/or structural distress when differential movements across a residential structure exceed about 1 in. in 20 ft. Significant distress has been observed in a number of structures built on relatively deep canyon fills where large differential settlements have been attributed to hydrocompression. The costs of repairs for some sites have been estimated to be nearly $100 000 000 (Lawton et al. 1992).

Because of the concerns regarding hydrocompression-related damages, the authors conducted a survey of geotechnical engineering design companies throughout the State of California in 1995. This survey was limited to companies in California, because most of the hydrocompression of deep fill case histories have been reported for sites in California. Thirty-four firms completed a four-page survey, with some choosing the option to remain anonymous. This information, in conjunction with in-house files from the firm that employs the authors, provided an important extension of hydrocompression data presented in the literature, and are utilized in this study.

The current challenge for the engineer designing a deep fill supporting residential structures is either to reduce post-construction differential ground movements, increase the capacity of residential structures to withstand those movements, or both. This paper focuses on compaction specifications designed to reduce differential movements caused by hydrocompression. To provide a theoretical background for the proposed compaction specifications, this paper examines hydrocompression from the perspective of a key variable, percent saturation (%S). Laboratory tests, case history data, compactive effort and compaction tests are all examined in terms of percent saturation.

Degree of Saturation During Compaction

In California residential projects, the maximum dry density and optimum moisture content for the soils are most commonly determined by the Modified Proctor test ASTM D1557, with a compactive effort of 56 250 ft-lb/ft^3. Previously, the Standard Proctor test (ASTM D698) with a compactive effort of 12 400 ft-lb/ft^3, and to a lesser extent, the California Impact test (California Standard 216-G), with a compactive effort of 37 000 to 44 000 ft-lb/ft^3, were used by various engineers. However, the Standard Proctor test was generally replaced by the Modified Proctor test when it was found that deep fills compacted to densities governed by the Standard Proctor test suffered significant settlement problems from insufficient compaction (Hunt 1986). If different laboratory compaction tests are performed on a soil using varying compactive efforts, different maximum dry density and optimum water content points will be determined for each test. In general, increasing the compactive effort increases the maximum dry density and decreases the optimum water content, as shown schematically in Figure 1. The line drawn through the maximums achieved by each of the compaction tests is termed the "line of optimums." The line of optimums should not be confused with the optimum

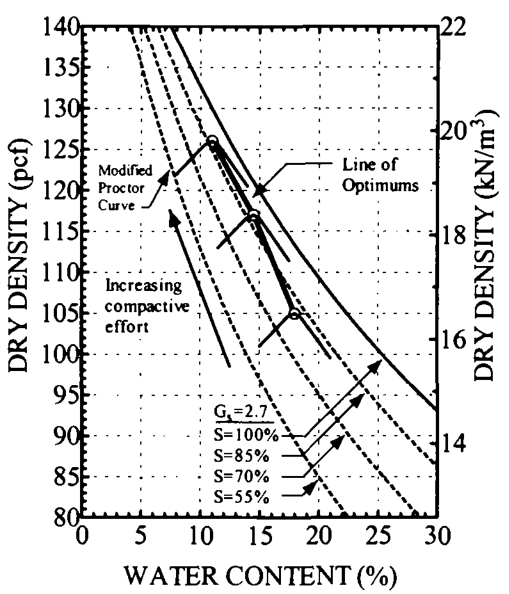

Figure 1: The maximum dry density increases and the optimum water content decreases for a given soil compacted with increasing compactive effort. The line drawn through the maximums achieved by each of the compaction tests is termed the "line of optimums."

water content, which is represented by a vertical line through the maximum dry density achieved with a particular compaction energy, for example as shown for the Modified Proctor test in Figure 1. The line of optimums is important because significant changes in soil structure, strength, yield characteristics and other properties have been shown to occur when soils (especially clayey soils) are compacted to densities and water contents above the line of optimums (Seed and Chan 1959). Of particular interest for this paper is that a significant change in hydrocompression behavior is expected when soils are compacted to a dry density and water content above the line of optimums. This change in hydrocompression behavior is probably due to a combination of higher percent saturation at the time of compaction as well as increased shearing in the soil and resultant changes in the soil structure that occur when soils are compacted wet of the line of optimums. The increased shearing when compacting wet of the line of optimums is probably due to a reduction in soil suction and concurrent weakening of the soil structure at higher saturations.

To evaluate the relationship between the line of optimums and the degree of saturation lines, examples of compaction curve data were obtained from the literature where both the Standard Proctor and the Modified Proctor tests were performed for the same soil. This data is presented in Figure 2. The specific gravity of these soils was generally not provided, but since most soils have a specific gravity between 2.6 and 2.8, 85 and 100% saturation lines are plotted on Figure 2 for these two specific gravity values. Note that the arrows on Figure 2 denote that the lines of optimums are generally clustered around the 85% degree of saturation lines for specific gravity values of 2.6 to 2.8 (especially at dry density values above 110 lb/ft^3). Since engineers do not typically run a series of varying effort compaction tests on the same soil, it is convenient to note that the line of optimums appears to extend roughly parallel to the degree of saturation lines. Therefore the line of optimums can be approximated by drawing a line through the maximum dry density and optimum water content point, parallel to the percent saturation lines.

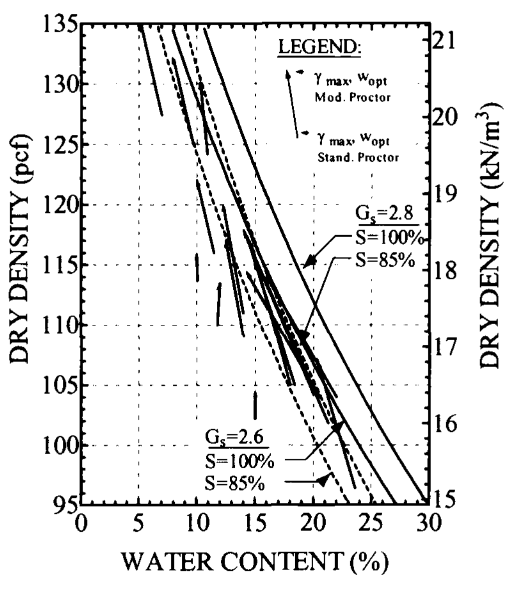

Figure 2: Lines of optimums for nineteen soils where both Modified and Standard Proctor test results are available. Increasing compactive effort from the Standard to the Modified Proctor test increases the maximum dry density and decreases the optimum water content; on average the effect of increased compactive effort is to shift the maximums roughly parallel to the percent saturation lines.

A large amount of maximum dry density data is available for soils with unknown specific gravity. Figure 3a shows the maximum dry densities and optimum water contents of 887 Modified Proctor tests (D1557). These data were collected from the literature, from the files of the firm that employs the authors, and from respondents of the survey on the practices of California engineering companies conducted by the authors. These data are plotted with the 85% saturation lines for specific gravity values of 2.4, 2.6, 2.8, and 3.0. While there are about 50 or so points (less than 10% of the data) that plot away from the 85% saturation lines, most of these points are from soils derived from diatomaceous soils or bedrock. Although diatomaceous soils can be shown to follow the same behaviors described in this paper as non-diatomaceous soils, caution must be used when designing for diatomaceous soils; the water content and saturation data are skewed by water held within (or occluded from within) the individual hollow siliceous cells that compose the individual diatoms. Excluding the data for diatomaceous soils, there is a clear trend that the maximum dry density/optimum water content data is centered around the 85% saturation lines. For comparison, Figure 3b shows the same 887 data points plotted with 100% saturation lines for the same specific gravity ranges. Although not shown on Figures 3a or 3b, the compaction data was compared against other percent saturation values for the same specific gravity ranges. The 85% saturation lines are displayed in Figure 3a because they have the best correlation with the compaction data, although there is significant scatter (even for the non-diatomaceous soils).

The actual percent saturation at the maximum dry density can be calculated if the specific gravity is known. Unfortunately the specific gravity test (D854 or D5550) is rarely performed by engineers designing fills supporting residential structures. A series of twenty-six maximum dry density and optimum water content points (as determined by ASTM 1557) were gathered from the literature on soils where the specific gravity (G_s) of the soil was known. The values of specific gravity ranged from 2.59 to 2.80. The twenty-six maximum dry density and optimum water content points are plotted in Figure 4a with 85 and 100% saturation lines for specific gravity values of 2.6 and 2.8; this figure provides continuity with the data previously presented on Figures 2, 3a, and 3b where the specific gravity was unknown. The values of percent saturation calculated for each of the maximum dry density and optimum water content points are plotted in Figure 4b. Most of the points plot between 80 and 90% saturation, and the average percent saturation for the maximum dry density and optimum water content points shown in Figure 4 is about 85%. The data in Figures 4a and 4b agree well with the data shown in Figure 3a.

Degree of Saturation in Response-to-Wetting Tests

When performing laboratory tests on a soil to determine appropriate compaction specifications to minimize hydrocompression, engineers typically perform a series of response-to-wetting tests for the soil at various densities (to simulate various relative compaction levels) and water contents (see Noorany and Stanley 1995). After performing such a series of tests, the engineer then tailors the compaction specification to limit the range of acceptable compaction to the densities and water contents that had the

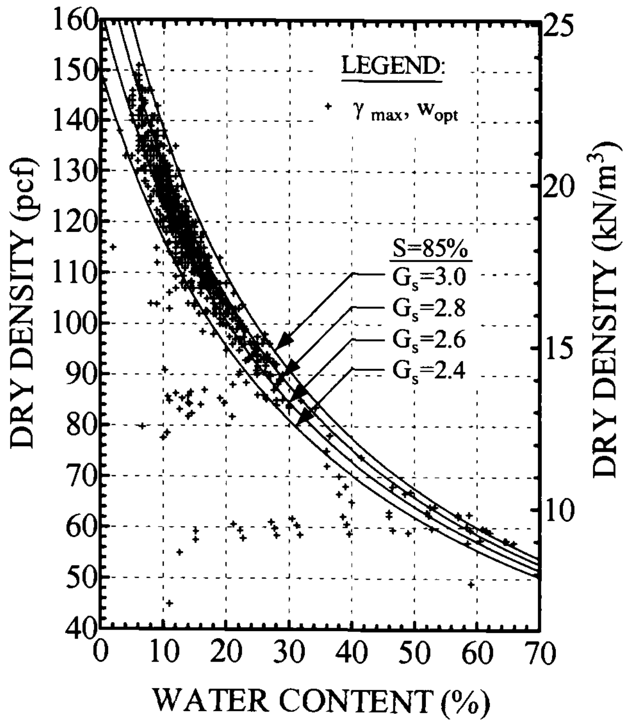

Figure 3a: The maximum dry density and optimum water content points for 887 modified Proctor tests (D 1557) have an excellent correlation with 85% saturation lines for specific gravities of 2,4, 2.6, 2.8 and 3.0.

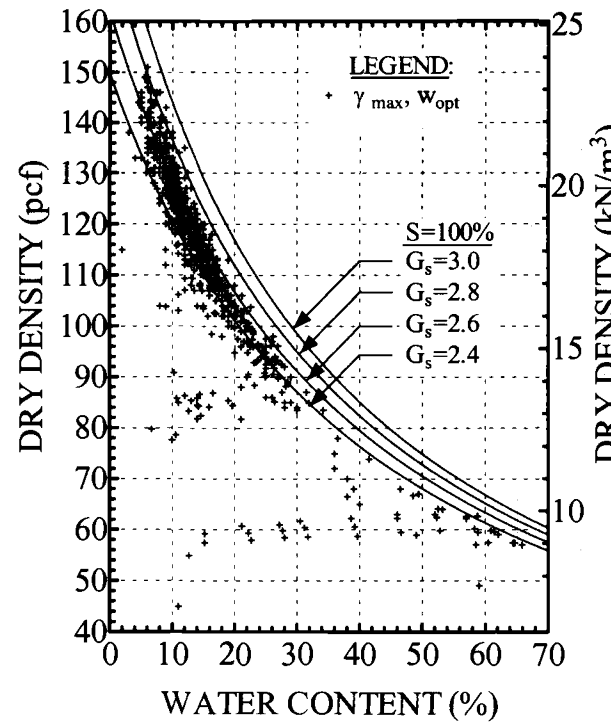

Figure 3b: For comparison with Figure 3a, the maximum dry density and optimum water content points for 887 modified Proctor tests (D 1557) are shown with 100% saturation lines for specific gravities of 2,4, 2.6, 2.8 and 3.0.

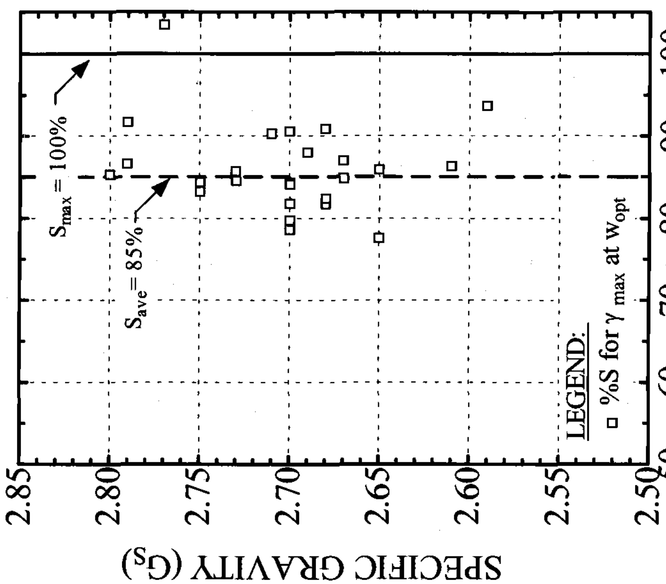

Figure 4b: Percent saturation at the maximum dry density and optimum moisture content for twenty-six soils with known specific gravities (same data as in Figure 4a).

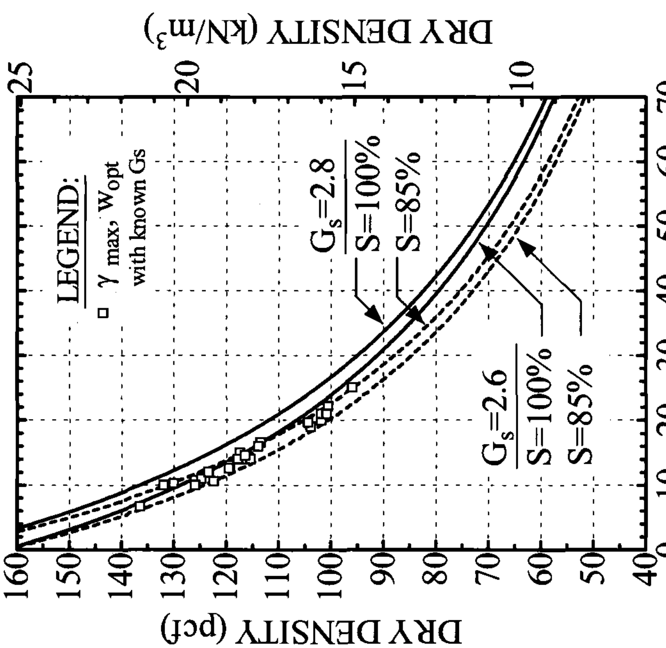

Figure 4a: Maximum dry density/optimum water content data for twenty-six soils of known specific gravity (see Figure 4b for specific gravity data).

best hydrocompression and hydroswell behavior. For example, Figures 5a and 5b show data collected by Noorany at the Villa Trinidad site studied by Brandon et al. (1990) (revised from Figures 2 and 3 of Noorany and Stanley 1995). Figure 5a shows the compaction curve determined by D1557, as well as the water contents and densities at which the response-to-wetting tests shown in Figure 5b were performed. The authors have added labels for optimum water content and relative compaction levels for clarity.

Often tests are performed without regard to the degree of saturation achieved during compaction. Since Noorany and Stanley measured specific gravity, the percent saturation achieved during compaction (just prior to initiating the response-to-wetting tests) was able to be calculated. The specific gravity measured was 2.66. The calculated percent saturation values for the Noorany data are shown in boxes on Figure 5b. In addition, Figure 5a shows lines of 40, 55, 70, 85 and 100% saturation that were calculated for a specific gravity of 2.66. As can be seen for the calculated saturation on Figure 5b, in general there is less hydrocompression (at higher vertical stresses) for samples that were compacted to a higher percent saturation.

Other important aspects of response-to-wetting tests are the percent saturation that is achieved upon soaking, and the level of saturation beyond which there is limited hydrocompression strain. In most laboratory response-to-wetting tests, samples are allowed free access to water by soaking, and typically the final degree of saturation is between about 92 and 98% (for example, Noorany et al. 1992 and Lawton et al. 1992). Some of the investigators at Purdue University (for example DiBernardo and Lovell 1980, and Nwabuokei and Lovell, 1986) have used back-pressure saturation to achieve 100% saturation. These investigators reported that relatively little additional hydrocompression strain occurred when back-pressure saturation was applied to the fully soaked sample to achieve a 100% saturation.

With regard to the relationship between the saturation level and the magnitude of hydrocompression, Lawton et al. (1992) indicated that there was a critical degree of saturation beyond which negligible hydrocompression would occur, and this level corresponded to the degree of saturation defined by the line of optimums. For their soil the line of optimums corresponded to a degree of saturation of about 80%. In tests where the saturation was controlled incrementally, Houston and Houston (1997) indicated that naturally deposited silts have a limited increase of collapse (hydrocompression in a naturally deposited soil) strain when the degree of saturation reaches approximately 85%. Of interest in the data by Houston and Houston is the correlation of the degree of saturation to the matric suction of the soils, shown on the right vertical axis of Figure 6. The trend of this data is that the matric suction becomes nearly constant (essentially zero) when the 85% degree of saturation level has been achieved. It appears that a good estimate is that most of the hydrocompression potential of a soil is realized by the time it has reached about 85% saturation. This is probably due to a reduction in soil suction and concurrent weakening of the soil structure at higher saturations.

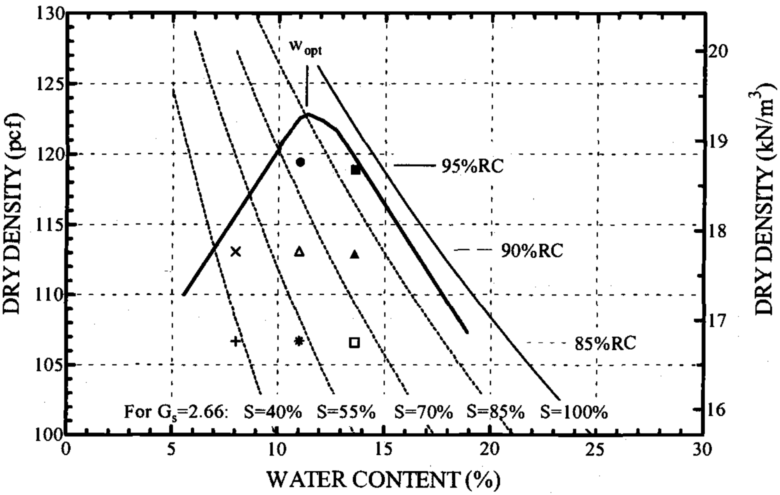

Figure 5a: After Noorany and Stanley 1995, the response-to-wetting test results drawn in Figure 5b were run for soils compacted to the densities and water contents shown (symbols and lines of relative compaction, "RC", correlate with Figure 5b).

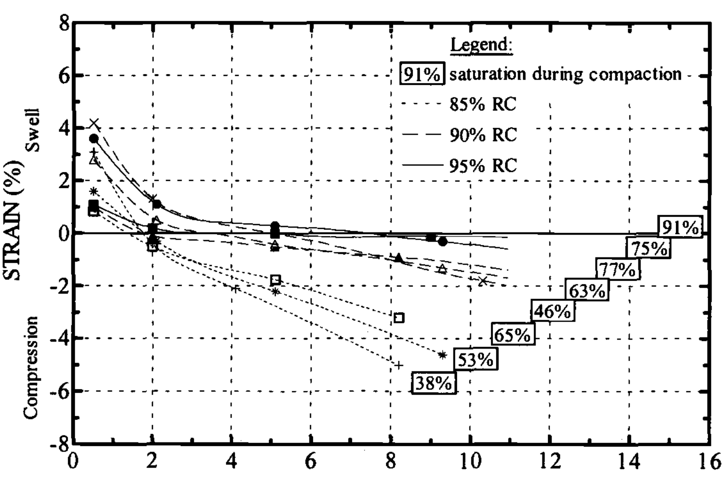

Figure 5b: Modified from Noorany and Stanley 1995, response-to-wetting test results for soils compacted to the water contents and densities shown in Figure 5a (symbols and lines of relative compaction correlate with Figure 5a). In general, increasing saturation during compaction decreases hydrocompression at higher loads. Calculated percent saturation values are in boxes.

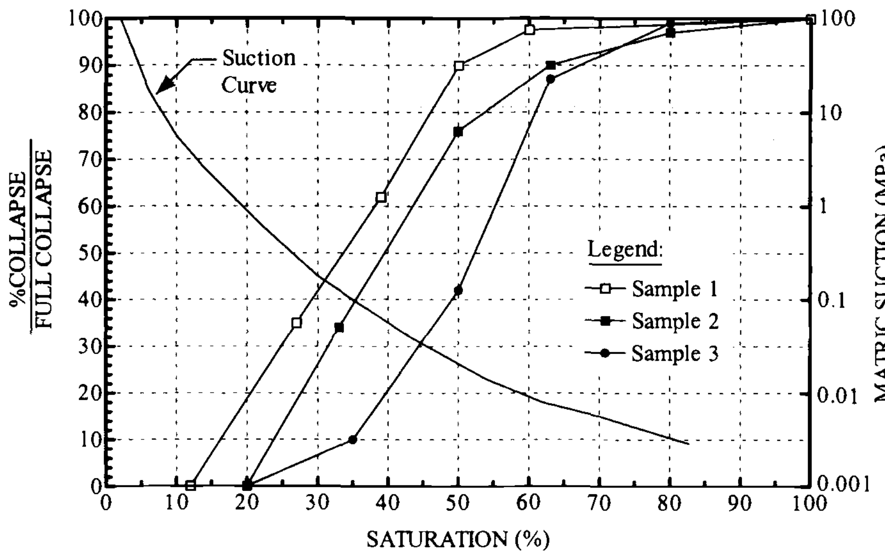

Figure 6: Collapse data for natural silts (after Houston and Houston 1997), showing collapse (hydrocompression in a naturally deposited soil) as a percent of the maximum collapse, measured as a function of percent saturation. Also note that the matric suction is essentially constant (near zero) at saturations above about 85%.

Degree of Saturation in Fills

The initial and final degrees of saturation in compacted fills were compiled for eight sites where a minimum 90% relative compaction specification (using D1557) was provided and where damaging hydrocompression had occurred (Brandon et al. 1990, Vicente et al. 1994, Kropp et al. 1994, Lamb and Hourihan 1995, Anonymous 1995, Eliahu 1995, Lamb 1995, Moran 1995, Shires 1995, and Vicente 1995). As shown on Table 1, four of these sites are described in the literature while the remaining four were submitted to us in response to the survey conducted by the authors. The initial degree of saturation and the final degree of saturation values are presented on Figure 7. It should be noted that at Site 1, the quarry fill site in Northern California, some of the soils were very gravelly and sandy, and these soils have relatively low degree of saturation values. Fill materials at the remaining sites consist of silty or clayey sands, silts or clays, and all of these materials typically had higher degree of saturation values, both during and after construction. In addition, the degree of saturation at some sites may continue to increase for up to 10 to 15 years after construction; thus the term "final" means the final measurement taken, and increased saturation levels may occur after the last measurement (such as at Site 6). Figure 7 illustrates that the degrees of saturation during construction were within a relatively narrow band, often varying by less than 10 percentage points at each site. Furthermore, the average degree of saturation during construction was about 60%.

Figure 7 also shows that post-construction wetting occurred for all of the residential fills studied, resulting in wider ranges of degrees of saturation. Data with a calculated saturation of above 100% was not plotted because saturation values above 100% are impossible; the data was cut off at 95% to reflect that the maximum degree of saturation normally seen under field conditions is about 95%. Note that the average degree of saturation after construction for sites monitored for over 10 years (except Site 8) are between 80 and 90%. In each of these case histories, the increase in the degree of saturation due to wetting after construction resulted in significant differential movements and post-construction distress to residential structures.

Degree of Saturation in Typical Compaction Specifications

Historically, compaction specifications for fills supporting residential structures in California required only a minimum relative compaction, usually 90% relative compaction. More recently, California practice has incorporated a minimum of 95% relative compaction for the deeper portions of fills. Whether 90 or 95% relative compaction was specified, another recent practice has been to specify a minimum water content, often defined as the optimum water content, but occasionally defined as the optimum water content plus 2 to 4% (usually when a minimum of 90% relative compaction is recommended). These minimum water content specifications have been developed, at least in part, to address differential movements caused by hydroswell and hydrocompression.

Typical compaction specifications using a minimum of 90% relative compaction and a minimum water content of the optimum water content are summarized in Figure 8a. While it is theoretically possible to achieve relative compaction and water contents approaching the zero-air-voids line (s=100%), only the area of the acceptable zone below the compaction curve is shaded to allow a more realistic comparison of the different compaction specifications. This example uses a typical soil with a maximum dry density of 126 lb/ft^3, an optimum water content of 11%, and a specific gravity of 2.7. Also shown on Figure 8a are 55, 70, 85 and 100% saturation lines. As shown by these saturation lines, the saturation at the maximum dry density/optimum water content is about 85%. In this typical compaction specification, the lower left portion of the zone of acceptable compaction has degree of saturation values of as low as about 60%. Recall that the average saturation during construction for the case history data shown in Figure 7 was about 60%, and that deep fills constructed with this type of specification have the potential for significant hydrocompression-induced settlements.

To address the potential hydrocompression that can occur (as well as reduce expansion potential of expansive fill materials), some California engineers have increased the minimum water content. The lower shaded triangular area in Figure 8b shows a typical compaction specification using a minimum of 90% relative compaction and a minimum water content of optimum plus 2%. The lower left portion of the zone

Table 1: Data for eight sites with damaging fill movements

SITE #	REFERENCE	USCS	YEARS AFTER CONSTRUCTION	SETTLEMENT (IN)	THICKNESS OF FILL (FT)
1	Kropp et al (1994)	GM, GP, SM	3	18	85
2	Brandon et al (1990)	CL	15	18	100
3	Brandon et al (1990)	CL	15	18	100
4	Vicente et al (1994) Vicente (1995)	MH, CH	9	18	150
5	Confidential N. Cal. Site	CH	12	?	?
6	Lamb & Hourihan (1995)	SC, CL	5	4.5	35
7	Lamb & Hourihan (1995)	SC, CL	12	5.7	70
8	Lamb & Hourihan (1995)	SC, CL	11	10.4	85

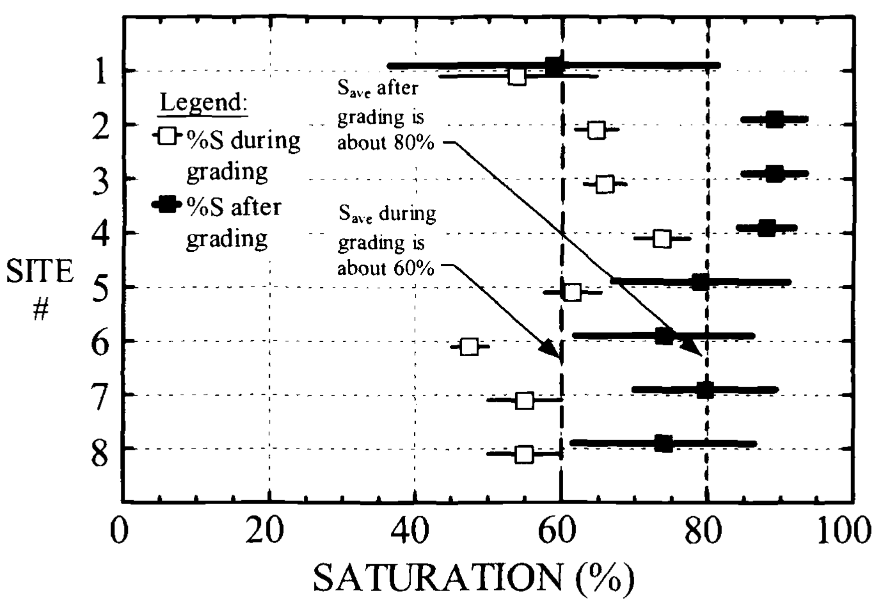

Figure 7: Ranges and averages of percent compaction during grading and several years later (years shown in Table 1) for eight fills with damaging differential movements.

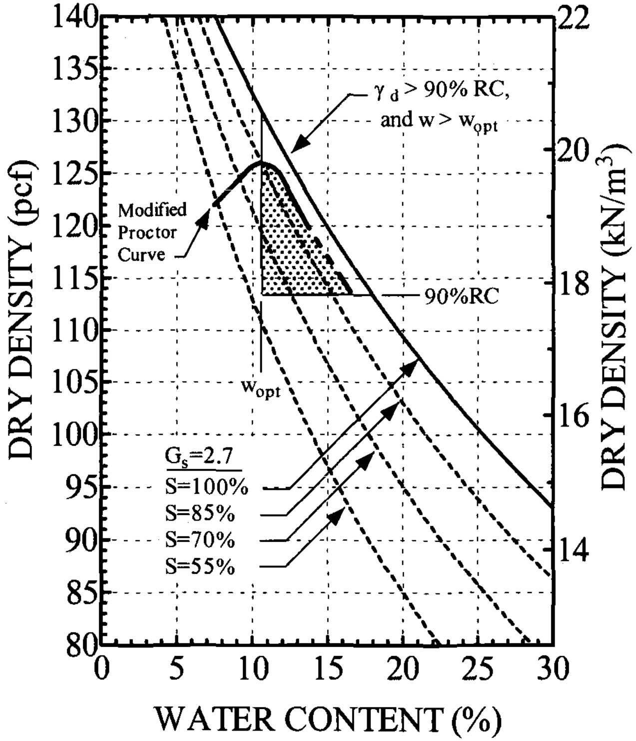

Figure 8a: Typical compaction specifications with a minimum of 90% relative compaction and a minimum of the optimum water content.

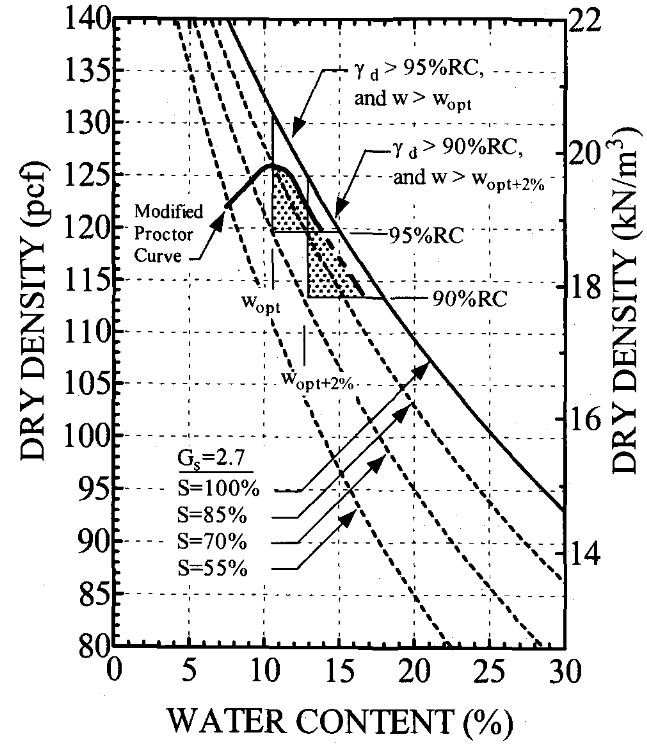

Figure 8b: Typical compaction specifications with a minimum of 90% relative compaction and optimum plus 2%, and compaction specifications with a minimum of 95% relative compaction and a minimum of the optimum water content.

of acceptable compaction still has degree of saturation values of as low as 70%. Other California engineers have addressed the potential hydrocompression by increasing the minimum relative compaction. The upper shaded triangular area in Figure 8b shows a typical compaction specification using a minimum of 95% relative compaction and a minimum water content of the optimum water content. The lower left portion of the zone of acceptable compaction also has degree of saturation values of as low as 70%. As evidenced by data from Sites 2, 3 and 4 on Figure 7, deep fill sites with materials compacted to 65 to 75% saturation experience significant settlement problems. Perhaps in response to these types of behaviors in deep fills, other engineers have increased the minimum relative compaction and minimum water content at the same time, defining triangular zones of acceptable relative compaction even smaller than those on Figure 8b.

Note that the two typical compaction specifications shown in Figure 8b use a minimum relative compaction and a minimum water content, which define triangle-shaped zones of acceptable relative compaction. In order to reduce hydrocompression (i.e. achieve high degrees of saturation), a triangular zone based on minimum relative compaction and minimum water content must necessarily increase the minimum water content and/or the minimum relative compaction. Those changes in specification define smaller triangles, which unfortunately preclude areas of relative compaction and water content that would have acceptable hydrocompression behavior because they have high degrees of saturation. For example, in the lower shaded triangular area defined in Figure 8b, increasing the minimum water content precludes soils compacted above 95% relative compaction and with saturation values above 85%. Similarly, in the upper shaded triangular area defined in Figure 8b, increasing the minimum relative compaction to 95% precludes soils compacted to above 90% and with saturation values above 85%. In both cases the smaller triangle-shaped areas preclude areas with low hydrocompression potential. Specifications based on a single value of minimum relative compaction and a single value of minimum water content that are designed to limit hydrocompression necessarily limit the area of the zone of acceptable compaction, and, in fact, limit the area more than is required.

Recommended Specifications

An excellent overview of contemporary practices regarding structural fill compaction is presented by Noorany (1997), and an outstanding discussion of the various types response-to-wetting tests (i.e. saturation before or after loading) is presented by Houston and Houston (1997). Noorany's recommendations include using different compaction criteria in the upper portion of a fill (where hydroswell behavior may predominate) and the deeper portion of a fill (where hydrocompression is the primary behavior). The discussion by Houston and Houston (1997) appears to help resolve whether to saturate samples before or after loading (contrast Lawton et al. 1992 and Noorany 1992). Houston and Houston recommend using a combined procedure, where the sample is saturated after significant loading, but some additional data may be collected for even higher load increments after saturation. The recommended specifications presented below specifically address the deeper portions of the fill where hydrocompression is a concern, and assume response-to-wetting tests performed with a

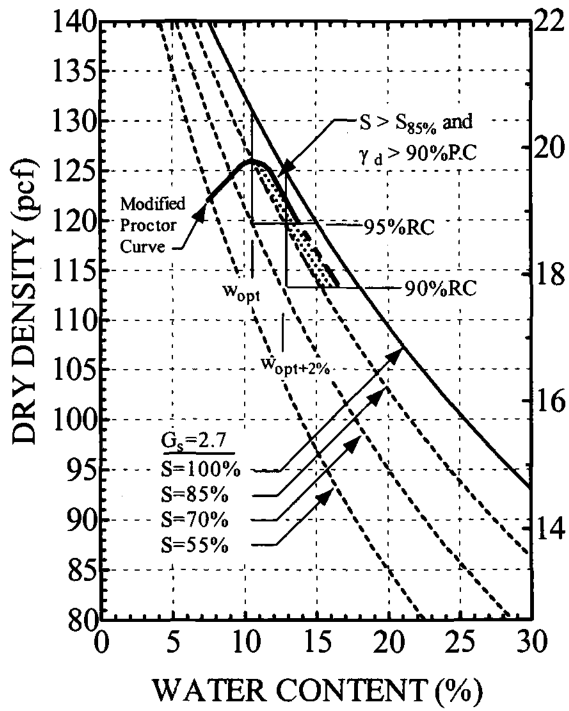

Figure 8c: Compaction specifications based on a minimum of 85% saturation (calculated from a measured specific gravity) and a minimum of 90% relative compaction.

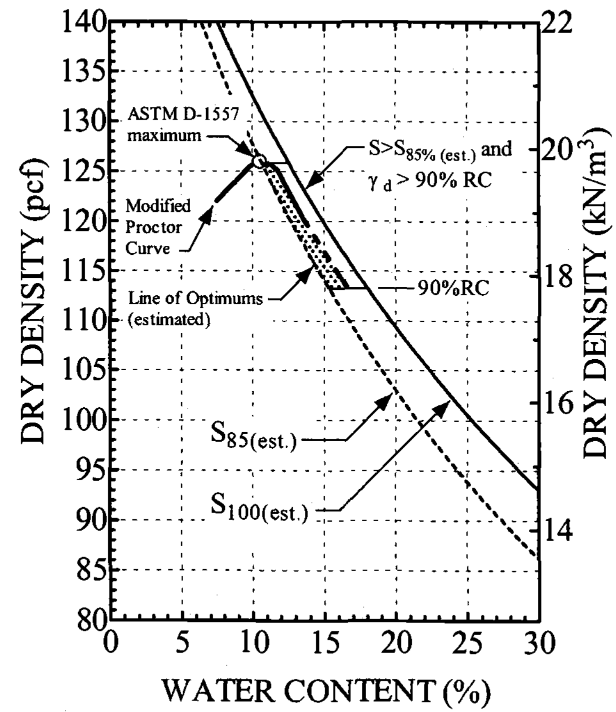

Figure 8d: New compaction specifications with a minimum saturation of the line of optimums (estimated), determined by drawing a line through the maximum dry density and water content point determined by ASTM D-1557, and by assuming the line of optimums and the maximum have saturations of 85%.

saturated-after-loading procedure that is possibly extended with some additional load increments. Corrections for the oversized portions of the soils (Noorany and Houston 1995) should be made where appropriate.

In the deeper portions of a fill mass where it is important to maintain a low hydrocompression potential, the authors propose a new type of compaction specification that is based on the percent saturation achieved during compaction. As discussed earlier, it appears that nearly all of the hydrocompression potential is eliminated above about 80 to 85% saturation. A compaction specification based on this understanding would therefore substantially reduce the hydrocompression potential. Such a specification is shown graphically in Figure 8c, where the optimum water content, optimum water content plus 2% and 95% relative compaction lines have been added for comparison with Figures 8a and 8b.

However, strict compliance with the compaction specification shown in Figure 8c would require testing of the specific gravity of the soil D854 or D5550 which is rarely performed by engineers designing residential fills due to both precedent and the difficulty of accurately performing these tests. Fortunately, it is relatively easy to estimate the saturation level where hydrocompression behavior is significantly reduced (typically 80 to 85% saturation). As illustrated on Figure 4, the average saturation of the maximum dry density and optimum water content points determined by D1557 is about 85%. Since this is the saturation level where improved behavior will likely be achieved, a line of constant saturation can be approximated by drawing a line through the maximum dry density/optimum water content point and parallel to the lines of 100% saturation. This line coincidentally approximates the line of optimums, above which there are generally beneficial improvements to soil structure, strength, and yield characteristics (Seed and Chan 1959) particularly for clayey soils and higher compactive effort. The zone of acceptable compaction using this type of specification based on the estimated minimum percent saturation present at the maximum dry density and optimum water content point on the compaction curve (typically about 85% saturation) is shown in Figure 8d, where the minimum relative compaction has been arbitrarily defined as 90% relative compaction, a common regulatory requirement. Lower allowable relative compactions might be considered by the design engineer and regulatory agencies in cases where the soils are highly expansive clay soils, especially for shallow portions of the fill, where hydroswell is expected to result in large differential movements due to post-construction wetting. Comparing Figures 8a through 8d, the zone of acceptable compaction shown in Figure 8d is similar in size to either of the triangular areas in Figure 8b, while at the same time limiting the hydrocompression potential by maintaining relatively high saturation during compaction.

As in all cases of deep fills with the potential for hydrocompression-induced settlements, after the zone of acceptable compaction has been estimated, the potential hydrocompression-induced movements should be checked by performing response-to-wetting tests (as described by Houston and Houston, 1997) on samples with the worst possible properties. A series of tests should be performed, similar to the series of tests described by Noorany and Stanley (1995). This is critically important because the properties of the site-specific soils must be determined and because the degree of saturation level where improved hydrocompression behavior is achieved is only approximated by the line of optimums; generalized, non-site-specific curves should not be used for a complete design. However, fewer response-to-wetting tests are required using the proposed new procedure. Continuing with the example shown in Figure 8d, the worst hydrocompression-induced settlements would occur for a soil with low relative compaction and low percent saturation, for example with 90% relative compaction at a water content of about 16%. The worst expansion in shallow clay soils would occur at low relative compaction and high water content, for example at a relative compaction of 100% and a water content of about 12.5%. Adjustments to the compaction specifications may be required if the predicted movements from the response-to-wetting tests are unacceptably large. Other measures should be considered, as discussed in great detail by Noorany (1997) such as maintaining structures above fills of uniform thickness, post-construction wetting prior to construction of sensitive structures, or designing foundations to accommodate the movements, particularly where the differential movements predicted by testing are unacceptably high.

In California, testing for relative compaction in the field (for example D1556 or D2922) is often performed by technicians working under the supervision of an engineer. These technicians have been trained to measure density and water content and compare them with acceptable values, but are generally not (currently) trained to calculate saturation values (even if the specific gravity is known). For this reason, engineers may have a concern that implementing compaction criteria based on the concept of percent saturation would make it difficult for their field technicians to quickly and economically determine the acceptability of the compaction achieved during construction. However, this issue is eliminated if technicians are supplied with a plotted zone of acceptable relative compaction for each soil, such as is shown in Figure 8d. The technicians can still determine acceptability of the compaction by measuring water content and density as they have done in the past. Acceptability is simply determined by comparing the measured water content and density with the plotted zone of acceptable relative compaction for each soil type.

Engineers, contractors, and owners using compaction specifications such as shown on Figure 8d should not be surprised if more compactive effort is required compared to previous standards. This will especially be true where specifications similar to the one shown on Figure 8a are still in use. A slightly smaller increase in effort will be required for the proposed specifications when specifications such as those shown on Figure 8b are in use. The increased compactive effort may cost more money. However, the increased compactive effort at higher percent saturation values helps to reduce differential settlements and damage caused by hydrocompression.

Conclusions

The proposed new compaction specifications based on a saturation level that corresponds to the peak point on a compaction curve (instead of a minimum water content) should reduce hydrocompression-induced settlements due to inevitable post-construction wetting. Hydrocompression is limited by this specification because most hydrocompression occurs below a saturation of about 80 to 85%. High degree of saturation lines are roughly parallel to the line of optimums, and soils compacted to a degree of saturation level above the line of optimums (not the optimum water content) are likely to have improvements in other soil properties, such as soil structure, strength and yield characteristics. The critical level of degree of saturation can be approximated by drawing a line through the maximum dry density/optimum water content point determined by D1557, and parallel to the lines of high saturation. This approximation reduces the need for specific gravity testing, and, combined with other common design procedures (uniform fill depths beneath a structure; different compaction specifications for shallow and deeper portions of the fill) allows the design engineer to define a larger zone of acceptable compaction. The larger zone of acceptable compaction provides for easier fill control and ease of construction, and therefore is a very practical improvement to current typical specifications, which usually require a minimum relative compaction and a minimum water content. When designing fills supporting residential structures or other deep fills where hydrocompression is a concern, engineers should consider using specifications based on a minimum percent saturation as a way to reduce hydrocompression.

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The Effect of Soil Composition and Moisture Content on Dry Density and Hydraulic Conductivity of Clays

Reference: Darban, A. K., Foriero, A., and Yong, R. N., "**The Effect of Soil Composition and Moisture Content on Dry Density and Hydraulic Conductivity of Clays,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbots, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Soil composition and moisture content have significant effects on the resulting bulk density of clayey soils because of the rearrangement of the solid particles and chemical stabilization. In this study, kaolinite and kaolinite with calcium carbonate (KC), silica gel (KS), and both calcium carbonate and silica gel (KSC) were compacted at different moisture contents according to ASTM standards. These mixtures of compacted clays were then subjected to leaching by distilled water, followed by a solution of heavy metals. Experimental results indicate that the dry density and coefficient of hydraulic conductivity are significantly influenced by the soil constituents. Specifically, kaolinite exhibits a low dry density and a high coefficient of permeability when compared to the other soils (KS, KC and KSC). As permeant flow approaches a stationary regime, greater pore volumes of effluents result, and an increase in the coefficient of permeability is observed in all types of soils. Tests confirm that chemical reactions are responsible for permeability increases up to a constant value. Expressly, kaolinite mixtures with silica gel or calcium carbonate exhibit a coefficient of permeability almost one order less than that of kaolinite. Results indicate the importance of silica gel as an additive for the reduction of the permeability in clay soils.

Keywords: soil, limestone, amorphous silica, compaction, hydraulic conductivity

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Introduction:

Compacted soils (fill) are commonly used for the construction of dams, embankments, and to support building foundations and pavements. Compacted clays are often used in landfill liner systems as engineered-barriers for containment and attenuation of contaminants in the leachates generated in the landfill. Compaction changes the physical properties of the fill thereby increasing the shear strength and reducing the consolidation-settlement. In practice the dry-density is the measure generally utilized in criterion calculations of the relative compaction. North American practice specifies fill-compaction to 95 percent or more of the maximum dry density, as determined in the laboratory by the Proctor test (ASTM D698-91 Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort) or ASHTO standard M57-80. Regarding clay liner specification, only low hydraulic conductivity (less than 1×10^{-7} cm/s) is to be attained in most of the federal and provincial regulations. In recent years, guidelines have also been compiled for selecting appropriate soil properties and compaction methods that are likely to result in low hydraulic conductivity (Gordon et al. 1984, Daniel 1990). Consequently the need for effective compaction techniques and selecting appropriate material is invaluable.

The most common characteristics warranted by a compacted fill are resistance to seepage, piping and erosion. In general, organic soils are not suitable as fill material because of their similarity to soft clays, low shear strength, and variable permeability. Similarly, certain factors also indicate that dispersive soils are not recommended because of their highly erosive and piping susceptibility. Moreover, other soils must be excluded because of their gypsum content which induces piping and uneven settlement.

To determine the appropriate fill material extensive analyses of such factors as particle-size distribution, permeability, compatibility, and plastic limits are required (Benson et al. 1994). The reason being that performance of earth fill is dependent on soil composition, density, and the layer-thickness of the specified materials. Consequently, one expects widespread use of recycled materials in earth fills only if a better understanding of the material's behavior, durability, and chemical stability under various loading and environmental conditions is acquired. Moreover, the economic efficiency of such materials (large volumes at low costs) will determine their success in recycled usage of earth fills. Indraratna et al. (1994) also emphasize the need for light-weight fill material and solid waste products.

This study examines the influence of the soil composition on dry density and hydraulic conductivity for compacted clay soils. Finally, a consideration of the potential use of fly ash and limestone for the treatment of the loose soil is investigated.

Materials and Method

The soil used in this experimental study is composed of kaolinite, silica gel and limestone. Kaolinite is commercially known as Hydrate PX (Georgia Kaolin Co.). It is frequently used in the construction of backfills and foundation for testing small-scale retaining walls. Consistency tests yielded a liquid limit of 49% and plastic limit of 33%.

The maximum dry unit weight obtained with the standard Proctor test (ASTM) resulted to 14.2 kN/m^3 and an optimum moisture content of 29%. The cohesion of this clay ranges between 16.3 and 23.8 kN/m^2 for a friction angle ranging between 18.4 and 21.7° as reported by Lesniewska and Porbaha (1998). Kaolinite was selected in this investigation for a number of reasons; for one kaolinite is the least reactive of clay minerals and commonly utilized in soil studies to limit reaction effects. Moreover, kaolinite is very low in amorphous content (virtually no quartz, smectites, carbonate or organic matter) and consequently practical to study the effects of both the absence and presence of the carbonate and amorphous materials. Silica gel, on the other hand, was used because it acts as an amorphous material in the soil. Finally, calcium carbonate (limestone) was added because of the natural carbonate content in soils is around 10-15%. The content-ratio of silica gel : kaolinite and limestone : kaolinite in the present tests averaged between 1 and 10%. This in accordance with the experimental results of Indraratna (1996) who indicates that increases in lime content induces a significant drop in strength. He explains this by the consumption of the available silica during the hydration. The same finding was reported by Wang (1990) in the case of silica gel.

Soil mixtures were air dried and ground to pass a 2 mm sieve. They were then subjected to a variety of chemical and physical tests including a pH measurement test, a cation-exchange-capacity (CEC) determination and a specific-surface-area (SSA) measurement. The soil pH was measured at a soil-water solution ratio of 1:2 with a Beckman pH meter (pH/ISE type). The specific surface areas were determined using ASTM C1251-95 (Standard Guide for Determination of Specific Surface Area of Advanced Ceramic Materials by Gas Adsorption). The CEC of the soil was determined by the silver thiourea method (Chabra et al. 1975). Results of physico-chemical properties of the prepared soil material; kaolinite (K), kaolinite + silica gel (KS), kaolinite + calcium carbonate (KC), and kaolinite + silica gel + calcium carbonate (KSC) are summarised in Table 1.

Table 1 — *Prepared soil physico-chemical properties.*

Properties	Kaolinite (K)	Silica gel (S)	10% silica gel+kaolinite (KS)	10% carbonate+ kaolinite (KC)	5% carbonate+ 5% silica gel + kaolinite (KCS)
PH	4.5 ±0.5	6.3 ±0.2	5.15 ±0.4	7.07 ±0.3	7.01 ±0.4
CEC (meq/100 g)	8 ±0.4	82 ±5	67±5	17 ±2	55 ±3
Surface Area (m^2/g)	12 ±0.2	276 ±10	118 ±7	66 ±4	97 ± 7

Prior to column testing, the optimum moisture content of the treated soil was determined using ASTM D698-91 (Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort). Moisture-density curves of the prepared soil material; kaolinite (K), kaolinite + silica gel (KS), kaolinite + calcium carbonate (KC), and kaolinite + silica gel + calcium carbonate (KSC) are demonstrated in Figure 1.

Column leaching tests were carried out to the effect of soil constituents on the coefficient of hydraulic conductivity. The procedure taken in each tests is described in the ASTM D4874-95 (Standard Test Method for Leaching Solid Material in a Column Apparatus). The prepared dry soil was mixed with distilled water raising the optimum moisture content slightly above the 3% mark (Phifer et al. 1995). The soil was then placed in a plastic container and allowed to equilibrate in a humid room for at least 24 hours. Subsequently the soil was statically compacted in a lucite cell to its maximum dry density. This was achieved in 3 layers of 16 mm, each layer requiring a pressure of 1500 psi (10342.5 kPa).

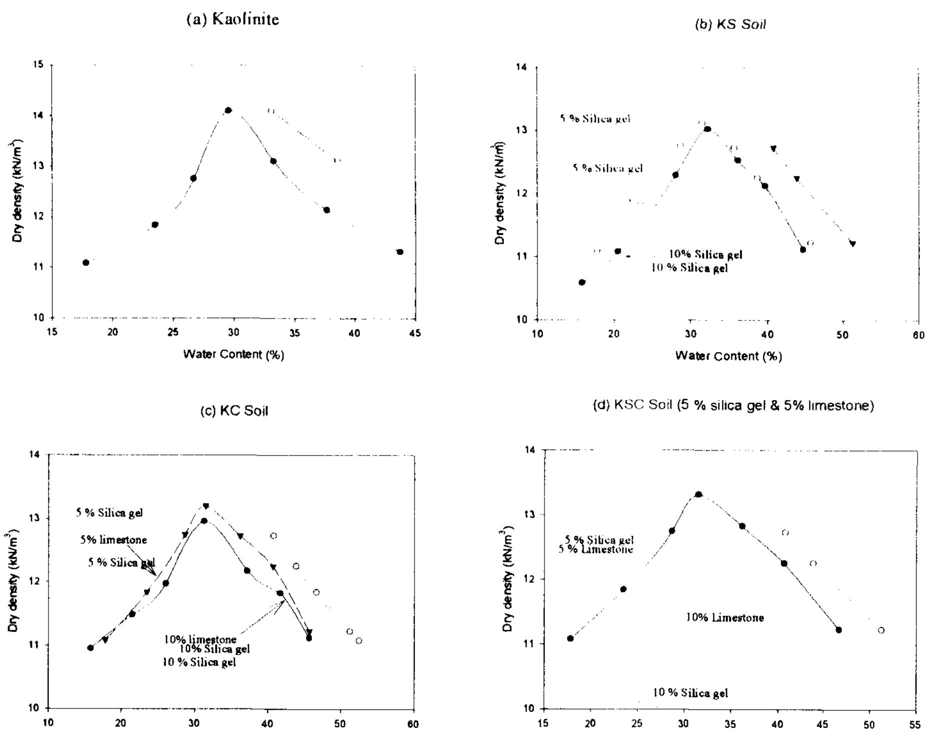

Figure 1—*Variation of dry unit weight with moisture content of kalinite clay blended with silica gel and limestone.*

A 5 mm thick porous stone was placed on top of the soil core to ensure a uniform distribution of the hydraulic pressure. A similar stone was utilized at the base of the column in order to collect and channel the effluent to the drainage outlet. A schematic picture of the cell is shown in Figure 2.

An air pressure of 18.30 kPa (equivalent to a hydraulic head of 2 m) was applied on the top boundary of the cylindrical soil sample. This resulted in a constant hydraulic gradient of 40. Results of permeability tests (Cabral and Yong 1993) for kaolinite clay demonstrate that there are slight changes in the coefficient of the permeability using this value of the hydraulic gradient.

The general procedure adopted for the column leaching tests herein is as follows: First, establish a steady state flow of distilled water through the soil sample; subsequently the permeant solution in the influent reservoir was exchanged for a solution of heavy metals spiked with chloride salts. Two types of heavy metal solutions were chosen, namely, lead and zinc.

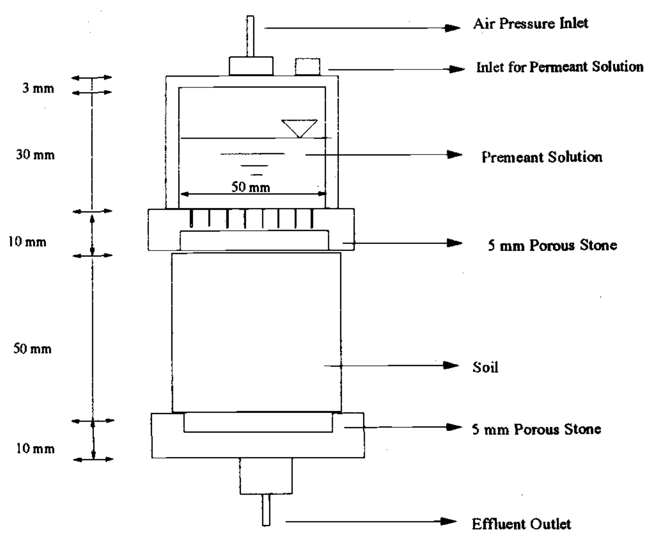

Figure 2 —*Leaching cell.*

Results and Discussions

Effect of Soil Composition on Compaction

Compaction curves shown in Figure 1 indicate a general trend. Specifically, the dry unit weight of all the samples decreased with increases of additives. This is partly explained by the specific gravity of the two additives being less than that of the kaolinite. In general the optimum moisture content of blended samples also increased with the percentage of additives. Moreover a difference in dry unit weight was also observed when the additive was varied from 5% to 10% (KS and KC soil).

Both limestone and silica gel absorbed water during the hydration process thereby increasing their optimum moisture content–this can be considered an advantage when working with wet fills. As shown in Figure 1c, increasing the limestone content lowers the compaction curve. Moreover, curves approach the fully saturated line for KSC and KC clays (Figures 1c and 1d).

However, the KSC clay behaves somewhat differently. Its compaction curve approaches the fully saturated line. In addition, this clay mixture behaves in a similar manner to the KC mixture. These results corroborate the findings of Kinuthia et al. (1999), who stress the effect of chemical stabilization of a divalent salt on the consistency and compaction of lime-stabilized soils. One cannot over emphasize that a decrease in the dry density of the blended soil is beneficial in reducing mass movements for fill material.

Effect of Soil Composition on the Coefficient of Permeability

Results of permeability tests on kaolinite and kaolinite mixtures at different pore volumes (K, KS, KC and KSC) are shown in Figure 3. One deduces that the coefficient of hydraulic conductivity is influenced by the soil constituents. The results agree with the findings of Benson et al. (1994), who emphasize the effect of specific surface area and double layer thickness on reduction of hydraulic conductivity of clay soils. However, introduction of the lead and zinc permeant did not affect significantly the value of the hydraulic conductivity for a particular soil. This is attributed to the low concentration of permeant solution which does not affect the diffuse double layer of the clay soil.

Kaolinite exhibits the highest coefficient of permeability when compared with the three other soils (Figure 3). As flow progresses more pore volumes of effluent result, and the coefficient of permeability begins to increase. Results indicate that the degree of saturation of the clay initially increases and proceeds towards a full saturation value yielding a constant coefficient of permeability.

The increase in the permeability of clays permeated by heavy metal solutions, observed in tests at high pore volumes of effluent, is linked to the reduction in the diffuse double layer thickness. Specifically, this is due to the replacement of the monovalent ions or exchange of calcium in carbonate soil from the solution by divalent heavy metals (Yong et al. 1992). However, as indicated in Figures 3b to 3d the kaolinite mixtures with silica gel or calcium carbonate additive exhibit a coefficient of permeability approximately one order less than that of kaolinite. These results substantiate the importance of silica gel and limestone in reducing the value of the coefficient of permeability. A valid explanation is that amorphous material acts as a cementing agent (Yong and Sethi 1980). On the other hand, the lower permeabilities observed in the cases of KC and KSC soils are a consequence of the chemical stabilization of kaolinite with limestone as emphasized by Kinuthia et al. (1999).

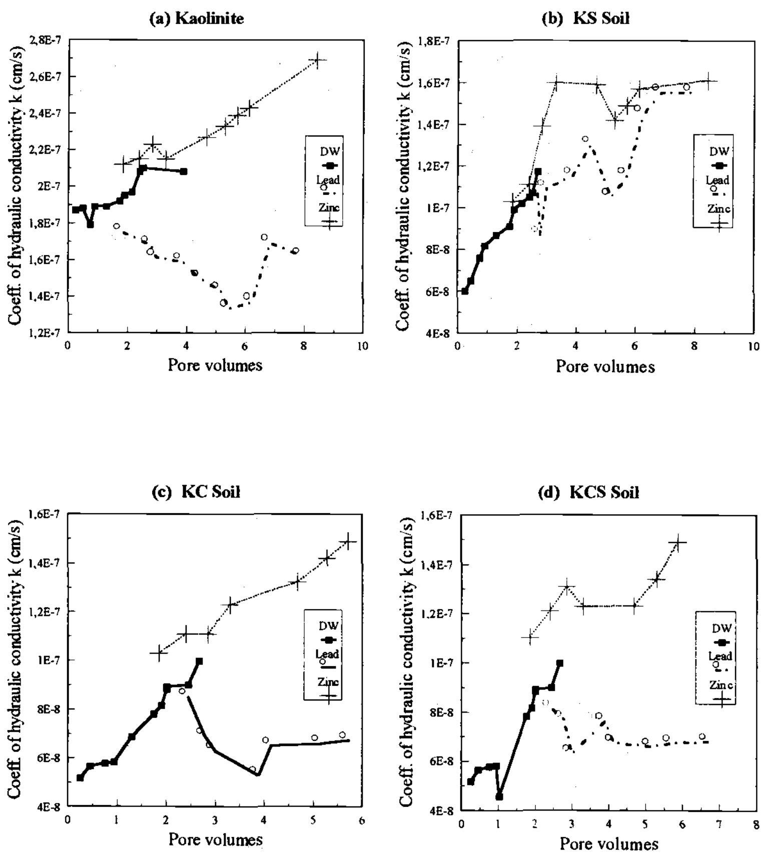

Figure 3 — *Coefficient of hydraulic conductivity in different clay soil.*

An illustration of time versus effluent pore volumes for each clay soil is plotted in Figure 4. As demonstrated by this graph, soil constituents have a significant effect on the value of the permeability of the clay soil. Measurements of the effluent pH as a function of pore volume, for each soil (K, KS, KC, KSC), are shown in Figure 5. Data of pH-effluent were recorded after each pore volume to determine the buffer-capacity (the resistance of the soil to a change in pH-value is called soil buffer capacity,Yong et al. 1995) of the soil with respect to the acidic leachate in the input solution. Also, one deduces from Figure 5 that more H^+ ions are introduced into soil column soil as the pore volumes increase, and consequently the pH of the soil solution decreases.

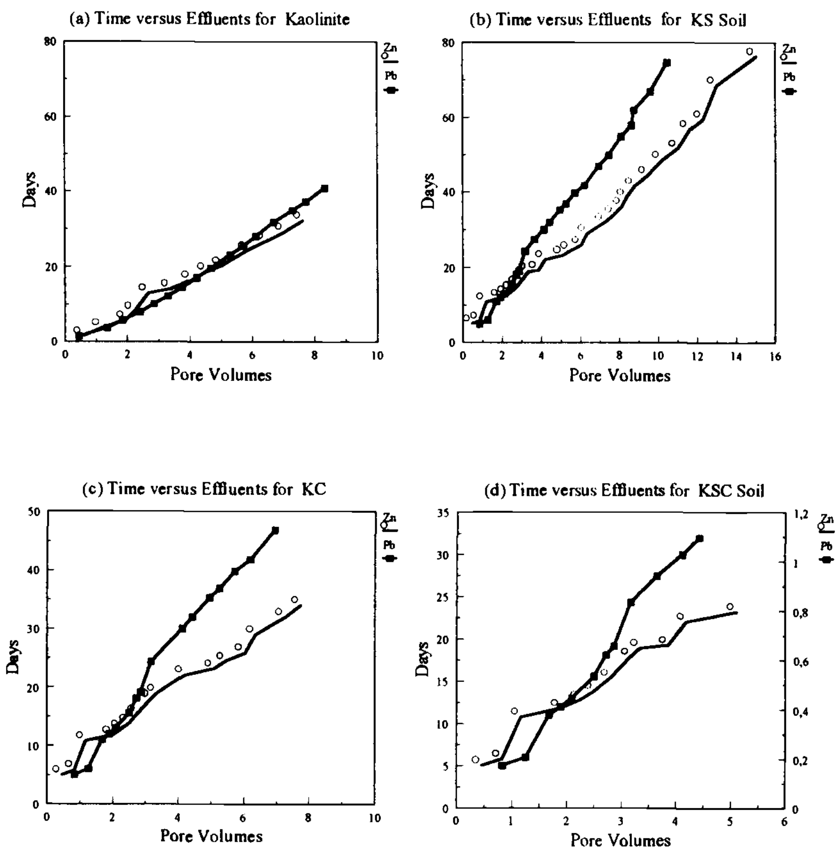

Figure 4 — *Flow in different soils.*

On the other hand for the carbonated soil (Figures 5c and 5d), an increase in pH- value is initialy observed. The resistance of the soil to a change in pH-value varies as the column receives a continuous load of acidic solution. One should take notice that the buffer capacity is also dependent on the soil constituents.

Finally, the pH-effluent of kaolinite soils is readily affected by increases in pore volumes when compared with the other tested clay soils in our experimental program. Kaolinite soils generally possess a low initial pH. On the other hand, KC soils display high effluent pH due to their high carbonate content (Phadungchewit, 1990).

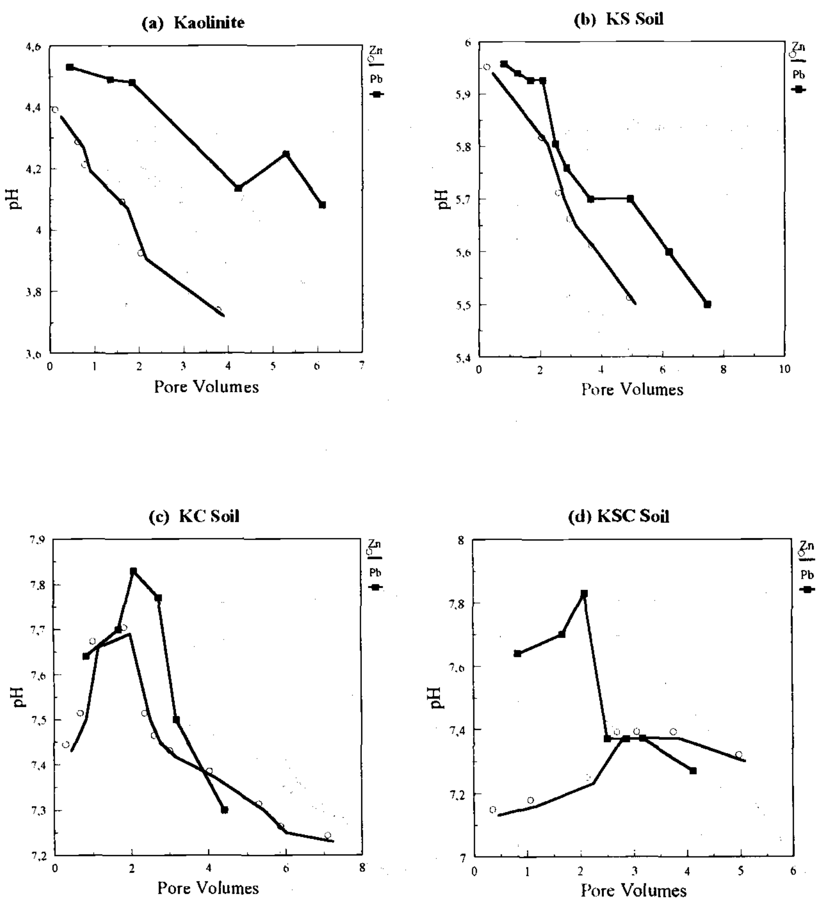

Figure 5 — *Soil pH variation with flow.*

Conclusion

The effect of soil constituency on the compaction behavior, the coefficient of permeability and the compatibility of particular clay soils was thoroughly examined. It was clearly demonstrated that chemical stability is directly linked to the properties of the parent soil, and to the cationic processes that take place as soon as the mix of material is in contact with water. Consequently, one cannot overemphasize the significance of the constitution of the parent material and the factors influencing chemical stabilization. These factors are also affected by various loading and environmental conditions related to the performance of earth fill materials. Moreover, the importance of the hydraulic conductivity on the performance of the earth fill materials should be definitely accounted for along with environmental influences. One concludes that soil materials with high cation exchange capacity (CEC), high specific surface area (SSA) and limited amorphous and carbonate content are valid constituents for earth fills. Finally, the addition of limestone for soil treatment provides a compatible chemical stabilization due to ion exchange process.

Acknowledgment

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Soil Liner Construction and New Compaction Technology

Pascal Thériault,[1] Rosa Galvez-Cloutier[1], and Thierry Winiarski[2]

Impact of Heavy Metals (Pb, Zn, and Cr) on the Hydraulic Conductivity of Sand/Bentonite Liner

Reference: Thériault, P., Galvez-Cloutier, R., and Winiarski, T., "**Impact of Heavy Metals (Pb, Zn, and Cr) on the Hydraulic Conductivity of Sand/Bentonite Liner,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384*, D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: This research was conducted to study the changes in permeability of a sand/bentonite liner made under standard compaction conditions and permeated with solutions containing the heavy metals Pb^{+2}, Zn^{+2} and/or Cr^{+3}. The fine particle size distribution and the free swelling capacity of the bentonite were taken as indicators of the impact of contaminants on the liner's properties.

Generally, particles of bentonite were coarser when in contact with solution containing heavy metals. Its free swelling capacity decreased when heavy metal concentrations increased. It is shown that with heavy metal solutions the permeability of a sand/bentonite liner increased by three orders of magnitude. The ion Cr^{+3} which had a greater impact on the free swelling capacity may have led to a rapid change in permeability. When permeability increased rapidly, heavy metals were present earlier and in higher concentration at the outlet of the liner.

Keywords: sand/bentonite liner, particle size distribution, free swelling capacity, permeability, heavy metal, compaction

Introduction

Sand/bentonite liners are used at hazardous waste disposal sites to prevent liquid wastes and leachate from contaminating groundwater. The imperviousness of these liners

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can be influenced by factors such as the content and mineralogy of bentonite, the fines content, the particle size distribution, the density and porosity of the sand, and the degree of compaction and saturation of the liner (Marcotte et al. 1994, Chapuis 1990). The influence of chemicals on the hydraulic conductivity is a major concern in determining the long-term performance of clay liners used for waste impoundments (Meegoda and Rajapakse 1993). Some studies have shown that waste chemicals of higher concentrations tend to cause bentonite to shrink and increase its permeability (Petrov and Rowe 1997, Wu and Khera 1990, D'Appolonia 1980). The chemicals that have the greatest effect on the expansion of the soil show the largest changes in the permeability (Wu and Khera 1990). Quigley stated in 1993 that "since they (bentonites) are very susceptible to double layer and c-axis contraction, bentonites are the most temperamental of the barrier clays and have not received nearly enough laboratory and field study."

Heavy metals form a group of contaminants commonly found in several kinds of wastes including sludges and landfill leachates. Depending on the type and origin of wastes, the leacheates generated may have undesirable levels of concentration of several heavy metals. The heavy metals that have received the most attention with regard to accumulation in soils, uptake by plants, and contamination of groundwater included lead (Pb), cadmium (Cd), copper (Cu), zinc (Zn), nickel (Ni), chromium (Cr), and mercury (Hg). The concentration of these heavy metals may range from 0-100 ppm in municipal solid wastes to 100 - ~10 000 ppm in sewage sludges, mining wastes, and various industrial wastes such as those originating from the electroplating, pulp and paper, and chemical industries (Yong et al. 1993).

This paper presents the results of a study on the permeability of a sand/bentonite liner made under standard compaction conditions and permeated with solutions containing the heavy metals Pb, Zn and/or Cr. The variation of permeability was compared with changes in particle size distributions and the swelling capacity of the bentonite.

Materials and Methods

The commercially available bentonite used in this study was NL Baroid National Standard Western Bentonite. It is currently used in the construction of sand/bentonite liners (Marcotte et al. 1994). It is composed of 85% sodium-rich montmorillonite and it had a cation exchange capacity (CEC) of 92 meq/100g. Ottawa sand was used for liner construction. It was a uniform fine graduated sand which had particles between 60 and 200 μm, an effective diameter (d_{10}) of 68.7 μm and it contained 16.4% under 80 μm. Three heavy metals, lead (Pb), zinc (Zn) and chromium (Cr) from nitrate salts were chosen as contaminants. This leads to heavy metals in the ionic form of Pb^{+2}, Zn^{+2} and Cr^{+3}. Aqueous solutions were made using deionised water.

In the following paragraphs, solution concentrations are first expressed as mass concentrations in mg/L to make eventual comparisons possible with literature. In fact, the contaminants making part of leachates or waste solutions are generally quantified in mass concentrations. In this paper, the corresponding charge concentrations are expressed in meq/L. The charge (meq/L) corresponds to the ionic concentration, expressed as mol/L, multiplied by the valence of the ion.

Particle Size Distribution Tests

The particle size distribution of the bentonite was measured using laser diffraction analysis with a Mastersizer Microplus from Malvern Instruments Ltd. The test consisted of the measurement of the particle size distribution of 0.1 g of bentonite mechanically dispersed by agitation in 500 mL of aqueous solution. The duration of the dispersion and the energy used were the same for each test. A test, which established the initial distribution, was performed with deionised water. The solutions were made with Pb, Zn or Cr in concentrations of 10 mg/L and 100 mg/L. This leads to solutions of 0.10 meq/L to 5.76 meq/L.

Free Swelling Tests

To observe the swelling capacity of the bentonite face to a change in its chemical environment, 2 g of bentonite (equivalent to 3 mL dry volume) were sprinkled in small, approximately equal portions (0.05 – 0.10 g) in 100 mL of solution in a graduated cylinder. Each portion was allowed to be deposited on the bottom of the cylinder before adding more sample. The bentonite reached its maximum free swelling capacity in less than 48 hours at which swelling results were taken. The initial free swelling capacity of the bentonite was determined with deionised water. Swelling was observed also in contaminated solutions with Pb, Zn or Cr in concentrations of 10 mg/L, 100 mg/L and 1000 mg/L. The charge concentration of solutions varied between 0.10 meq/L and 57.70 meq/L. The free swelling capacity F_s was expressed as (Wu and Khera 1990):

$$F_s = 100(V_f - V_i)/V_i \ (\%) \tag{1}$$

where

V_i = initial dry volume of bentonite, mL, and
V_f = final volume of expanded bentonite in solution, mL.

Permeability Tests and Breakthrough Curves

Based on the results of trial batches, a bentonite content of 12% was selected for liner construction to achieve permeability in the order of 10^{-8} cm/s, to assure retention of heavy metals (Galvez-Cloutier and El-Herraoui 1998), and to prevent washing of fines (Marcotte et al. 1994). The optimum moisture content and the maximum dry unit weight for the sand/bentonite mixture were determined according to ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (D 1557).

The permeability tests were conducted using rigid leaching cells. The permeability was estimated according to Darcy's law for saturated conditions. Sand/bentonite mixture was compacted to achieve 95% of its maximum dry unit weight. A thin latex membrane was used to prevent the occurrence of sidewall leakage. Before the tests, each soil column was saturated with deionised water.

In order to accelerate percolation, an air pressure applied on the solution held above the soil column produced a constant hydraulic gradient of 286. The tests were terminated when 10 pore volumes passed through each soil column. One cell was permeated with deionised water and was used as reference for comparisons. Three other cells were tested with three different solutions. The first solution contained 1000 mg/L of Pb^{+2} and 1000 mg/L of Zn^{+2} producing a total charge concentration of 40.24 meq/L. The second one contained 1000 mg/L of Zn^{+2} and 1000 mg/L of Cr^{+3} equivalent to a charge of 88.28 meq/L. Mixing all three heavy metals, each one in a concentration of 1000 mg/L, made the third one with a total charge of 97.94 meq/L.

During column percolation, heavy metal concentrations were measured using atomic absorption spectrometry, at the end of every pore volume passage. The results were plotted as breakthrough curves.

Results and Discussion

Particle Size Distribution Tests

Particle size distributions of the bentonite in solutions containing Cr^{+3} in concentrations of 10 mg/L and 100 mg/L are plotted in Figure 1. The size distributions are expressed in percentage volume by volume, as frequency curves. One principal mode was observed for the bentonite with and without heavy metals. As the mass concentration of heavy metal increased, the principal mode of particles moved farther to the right on the x-axis, meaning the formation of coarser particles. The results obtained for Pb^{+2} and Zn^{+2} followed similar trends. Thus, an increase of the mass concentration was associated with an increase of charge concentration as it varied from 0 meq/L for water to 0.58 meq/L and 5.76 meq/L for Cr^{+3} (10 ppm) and Cr^{+3} (100 ppm), respectively.

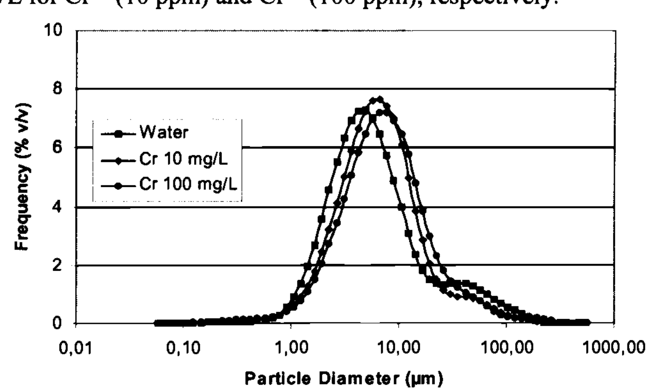

Figure 1 – *Influence of the concentration of Cr on the particle size distribution*

The influence of the ion type on particle size distribution is shown in Figure 2. The distributions were obtained with Pb^{+2}, Zn^{+2} or Cr^{+3} in an equal concentration of 100 mg/L. These solutions gave charge concentrations of 0.96 meq/L, 3.06 meq/L and 5.76 meq/L, respectively. As shown in Figures 1 and 2, particle size was coarser with an increase of charge concentration, expressed in meq/L. However, it seemed (Figure 2) that a maximum impact was reached when the curves of Zn^{+2} (100 mg/L) and Cr^{+3} (100 mg/L) overlapped despite further increase of charge (3.06 to 5.76 meq/L).

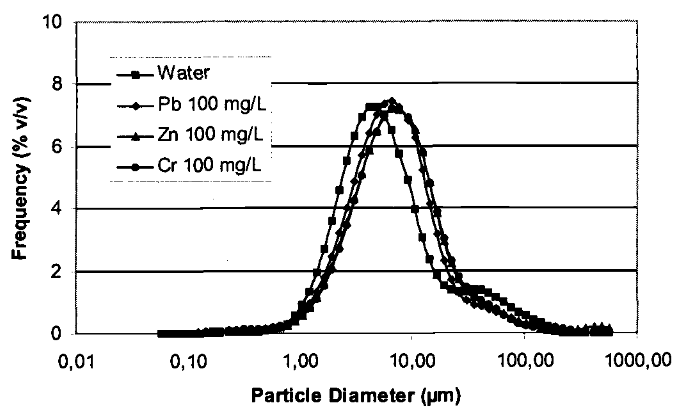

Figure 2 – *Influence of solutions containing 100 mg/L of Pb, Zn or Cr on particle size*

An increase of charge concentration may represents an increase of ion concentration and/or valence. According to the diffuse double layer (DDL) equation (Mitchell 1976):

$$\frac{1}{K} = \sqrt{\frac{DkT}{8\pi\eta_o e^2 v^2}} \qquad (2)$$

where

1/K = "thickness" of the double layer
D = dielectric constant of the medium
k = Boltzmann constant
T = temperature
η_o = electrolyte concentration
e = unit electronic charge
v = cation valence

an increase of ion concentration and of valence may reduce the double layer thickness. Thus, a reduction of the double layer thickness may inhibit the dispersion of the bentonite particles, leading to the formation of bigger aggregates.

Free Swelling Tests

The free swelling capacity (F_s) of bentonite interacting with selected solutions as a function of heavy metal mass concentration and charge is presented in Figures 3 and 4, respectively. The disposition of heavy metals and their concentrations on the x-axis of Figure 3 represents the order from left to right for which the charge concentration increases. The points of the curve (Figure 4) followed the same order. In deionised water, F_s reached 1000%. It remained nearly constant up to an individual concentration of Zn of 100 mg/L or for a charge concentration of 3.00 meq/L. Concentrations higher than that produced a significant decrease of the free swelling capacity. The greater impact was found when interacting with Cr^{+3} concentration of 1000 mg/L, corresponding to the maximum charge concentration of 57.70 meq/L. Thus, reducing the free swelling capacity to 200%. Solutions with Cr^{+3} showed the greater impact because of its higher valence. As expected from equation 2, a higher ionic concentration and a higher ion valence, reduced the double layer thickness, which in turn inhibited the expansion of bentonite.

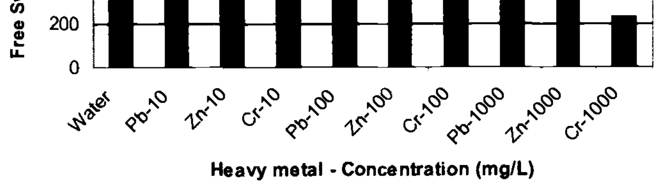

Figure 3 - *Free swelling capacity of bentonite against mass concentration*

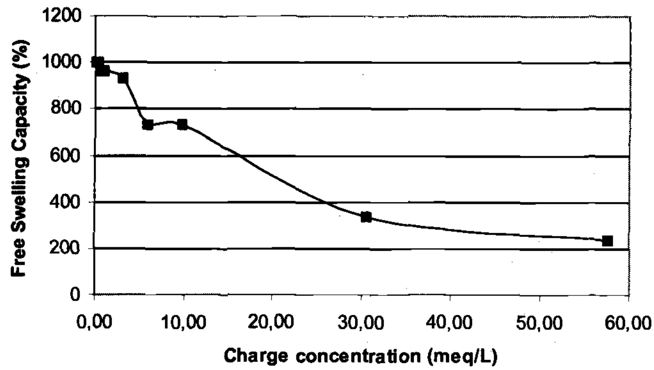

Figure 4 - *Free swelling capacity of bentonite against charge concentration*

Permeability Tests and Breakthrough Curves

The variations in permeability with time of the sand/bentonite mixture permeated with different loads of contaminants are presented in Figure 5. The compaction of the soil to achieve 95% of the maximum dry unit weight leaded to a total pore volume equal to 16 mL. The permeability of the s/b mixture with deionised water was nearly constant and it was in the order of 10^{-9} cm/s. Generally, with solutions containing heavy metals the permeability increased by three orders of magnitude. With Zn-Cr and Pb-Zn-Cr solutions, which represented a charge concentration of 88.28 meq/L and 97.94 meq/L, respectively, the permeability increased rapidly after the passage of the first pore volume. It increased progressively with the Pb-Zn solution which had a charge concentration of 40.24 meq/L.

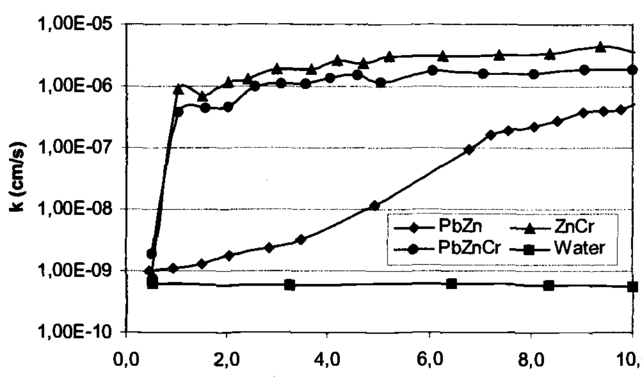

Figure 5 – *Permeability (k) of s/b mixture permeated with heavy metal solutions*

The relative concentrations (C/Co) of effluents are plotted in Figures 6, 7 and 8, for selected solutions. As mentioned earlier, sand/bentonite mixture permeated with Pb-Zn solution showed a progressive increase on its permeability (Figure 5). The metals Zn^{+2} and Pb^{+2} reached the end of the soil column after 5 and 6 pore volumes, respectively (Figure 6). After 10 pore volumes, the concentrations of Pb^{+2} and Zn^{+2} were still increasing and were lower than 80% of the initial concentration. The migration of Pb^{+2} was retarded compared to that one of Zn^{+2}.

Sand/bentonite mixture permeated with Zn-Cr and Pb-Zn-Cr solutions showed a rapid increase on its permeability within the first pore volume (Figure 5). The heavy metals migrated to reach the end of the soil column within the first pore volume (Figures 7 and 8). The out-coming concentrations of the heavy metals increased up to 80% to 100% of the initial concentration after the passage of approximately 5 pore volumes. The migration of Cr^{+3} was retarded compared to that one of Pb^{+2} and Zn^{+2}.

Based on the CEC of the bentonite and on the mass of s/b mixture, each cell liner made of 12% bentonite/sand ratio gave an exchange capacity of 10.15 meq. Theoretically, the liner exchange capacity was sufficient to retain all heavy metals present in Pb-Zn solution after the passage of 10 pore volumes, which gave a solution charge of 6.44 meq. However, results (Figure 6) showed that the liner capacity was not totally used. Same thing happened with solutions Zn-Cr and Pb-Zn-Cr. After the passage of 5 pore volumes the relative concentrations of heavy metals were up to 80% to 100% at the end of the soil column (Figures 7 and 8). Then, the total charges of Zn-Cr and Pb-Zn-Cr solutions were 7.06 meq and 7.84 meq, respectively, which is not enough to fulfill the exchanged sites of the liner (10.15 meq). The passage of heavy metals may have been due to the increase in permeability, thus leaving inadequate reaction time for ion exchanges and retention.

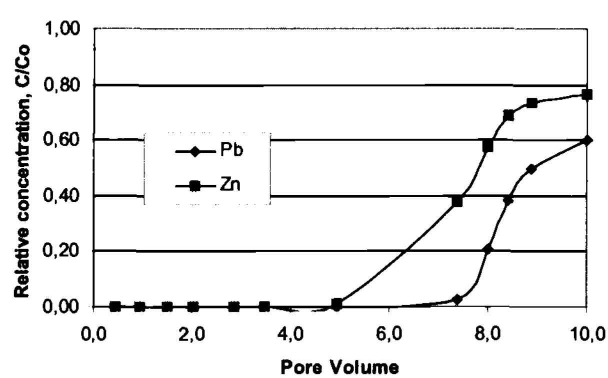

Figure 6 – *Breakthrough curves from the permeation of Pb-Zn solution*

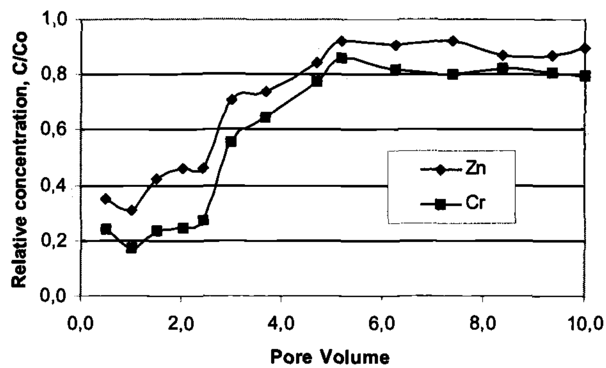

Figure 7 – *Breakthrough curves from the permeation of Zn-Cr solution*

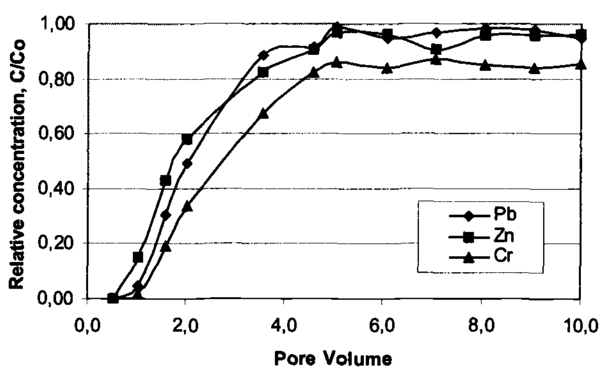

Figure 8 – *Breakthrough curves from the permeation of Pb-Zn-Cr solution*

Summary and Conclusion

The expression of heavy metals in terms of charge concentrations, in meq/L, was more representative of the impact of contaminated solutions on the properties of bentonite and sand/bentonite liner.

Generally, particles of bentonite were coarser when the charge concentration increased. This may lead to bigger pores when liners are subjected to the percolation of "charge rich" leachates, thus resulting in higher permeabilities. A charge concentration of nearly 3.00 meq/L produced a maximum impact on the particle size distribution. Charges higher than approximately 3.00 meq/L impacted significantly on the free swelling capacity. Compared with the permeability with water, the permeability of 12% bentonite/sand mixture permeated with solutions that contained heavy metals increased, generally, by three orders of magnitude. A charge of 40.24 meq/L changed progressively the permeability of sand/bentonite liner. By doubling this charge the permeability reached three orders of magnitude higher after the sole passage of one pore volume. The passage of heavy metals may have been due to the increase in permeability that left inadequate time for ion exchanges and retention. Heavy metals were present in higher concentrations earlier at the end of the soil column when permeability increased rapidly.

A few solutions to this problem could consist of increasing the thickness of the liner, adding more bentonite to the mixture, using a clay which offers more appropriate properties, diminishing the charge of the leachate, etc.

Acknowledgments

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Craig H. Benson[1] and Gordon P. Boutwell[2]

Compaction Conditions and Scale-Dependent Hydraulic Conductivity of Compacted Clay Liners

Reference: Benson, C. H. and Boutwell, G. P., "**Compaction Conditions and Scale-Dependent Hydraulic Conductivity of Compacted Clay Liners,**" *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: A database was compiled containing index properties, compaction data, and hydraulic conductivity data from 51 clay liners. The hydraulic conductivity data included large-scale field measurements (K_F) and small-scale laboratory measurements (K_L) on specimens collected from the field in 74-mm-diameter sampling tubes. Analysis of the database shows that K_F is comparable to K_L when compaction is wet of the line of optimums, but K_F can be up to 266 times larger than K_L when compaction is dry of the line of optimums. Similar results were obtained from a stochastic simulation model. Discrepancies between K_F and K_L exist when macropores are present (i.e., compaction dry of the line of optimums), because macropores are inadequately represented in small laboratory specimens. To achieve, $K_F \sim K_L$ compaction specifications should require that the percentage of data falling wet of the line of optimums (P_o) exceed 80-90% and the difference in initial saturation (ΔS_i, which is the mean degree of saturation at compaction less the degree of saturation corresponding to the line of optimums) exceed 2 to 4%. A "line of optimums" compaction specification is recommended to facilitate achieving these recommended values for P_o and ΔS_i.

Keywords: compacted clay, clay liners, hydraulic conductivity, field hydraulic conductivity, laboratory hydraulic conductivity, compaction conditions, landfills

Compacted clay liners are used alone or in conjunction with a geomembrane as hydraulic barriers in waste containment systems. Because their primary purpose is to impede flow, clay liners should be constructed with methods that result in low hydraulic conductivity. While many factors influence the hydraulic conductivity of compacted clays, the most influential factors are compaction water content and compaction effort (Mitchell et al. 1965, Boutwell and Hedges 1990, Benson and Trast 1995, Benson et al. 1999). Varying either of these factors can result in the hydraulic conductivity varying by as much as six orders of magnitude (Benson and Daniel 1990). Accordingly, proper compaction control is essential to successfully construct a clay liner with low hydraulic conductivity (Benson et al. 1999).

Because hydraulic conductivity is the key property of a clay liner, measurements of hydraulic conductivity are nearly always made during construction of clay liners.

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Specifying the method to measure the hydraulic conductivity is one of the most contentious issues in quality assurance testing (Benson et al. 1999). Large-scale field methods are preferable because numerous investigators (Daniel 1984, 1987; Elsbury et al. 1990; Sai and Anderson 1990; Trautwein and Williams 1990; Trautwein and Boutwell 1994; Benson et al. 1994a; Trast and Benson 1995; Benson et al. 1999) have shown that the field-scale hydraulic conductivity (K_F) of clay liners can be much higher than the hydraulic conductivity measured on small "undisturbed" specimens collected in the field and tested in the laboratory (K_L). For example, in a review of 85 case histories, Benson et al. (1999) show that K_F/K_L can be as large as 300. However, field hydraulic conductivity measurements are usually expensive, time consuming, and difficult to integrate into the construction process. Also, field measurements made *after* construction is complete do little good in terms of achieving a clay liner with low hydraulic conductivity. Consequently, laboratory measurements on small specimens collected in thin-wall sampling tubes remain popular for assessing the hydraulic conductivity of clay liners.

Examination of field data from numerous sites has shown qualitatively that the difference between K_F and K_L depends to great extent on compaction conditions (Reades et al. 1990, Benson and Boutwell 1992, Benson et al. 1994a, Trast and Benson 1995). The purpose of this paper is to demonstrate quantitatively how K_F/K_L varies with parameters describing the compaction condition. A database was compiled using data collected from 51 different clay liners where compaction conditions, index properties, K_F, and K_L were documented sufficiently well to ensure their reliability. The database was then analyzed to identify parameters describing compaction conditions that correlated well with K_F/K_L. A stochastic simulation model was also developed to confirm that the empirical correlations developed by analyzing the database were grounded in basic principles.

Background

Benson and Boutwell (1992) and Benson et al. (1994a) have shown that differences between K_F and K_L exist when the pores controlling flow at field-scale are inadequately represented in laboratory specimens and that the presence of such pores is related to the compaction condition. In particular, when compaction does not eliminate interclod macropores, the pores in a specimen removed in a typical thin-wall sampling tube (74-mm-diamter) are more representative of small micropores in the matrix of a clay clod instead of the larger interclod macropores conducting flow at field-scale. These large macropores control K_F (Fig. 1a). Because large interclod macropores are not represented in small laboratory specimens, K_L is lower than K_F. Conversely, when clay is compacted in a manner that eliminates interclod macropores, micropores conduct flow at field scale. These pores are adequately represented in a specimen collected in a typical sampling tube (Fig. 1b). As a result, K_L is essentially the same as K_F.

The existence of large interclod macropores is closely tied to the compaction condition (Garcia-Bengochea et al. 1979, Acar and Oliveri 1990, Benson and Daniel 1990). Dry of the line of optimums, clay clods are strong relative to the compactive effort applied and thus are difficult to remold. As a result, clay compacted dry of the line of optimums generally has large macropores between clods (Fig. 1a) and higher hydraulic conductivity (Garcia-Bengochea et al. 1979, Benson and Daniel 1990). As the water content increases, clay clods become softer and are more easily remolded, which eliminates macropores (Benson and Daniel 1990, Shackelford and Javed 1991). The transition point where macropores are eliminated corresponds approximately to the line of optimums. Wet of the line of optimums, most if not all of the large macropores are eliminated, leaving only micropores and low hydraulic conductivity (Fig. 1b) (Benson and Daniel 1990). Accordingly, K_F should be larger than K_L for clay liners compacted dry of the line of optimums and, as the compaction becomes wetter relative to the line of optimums, K_F should become increasingly similar to K_L (Benson and Boutwell 1992).

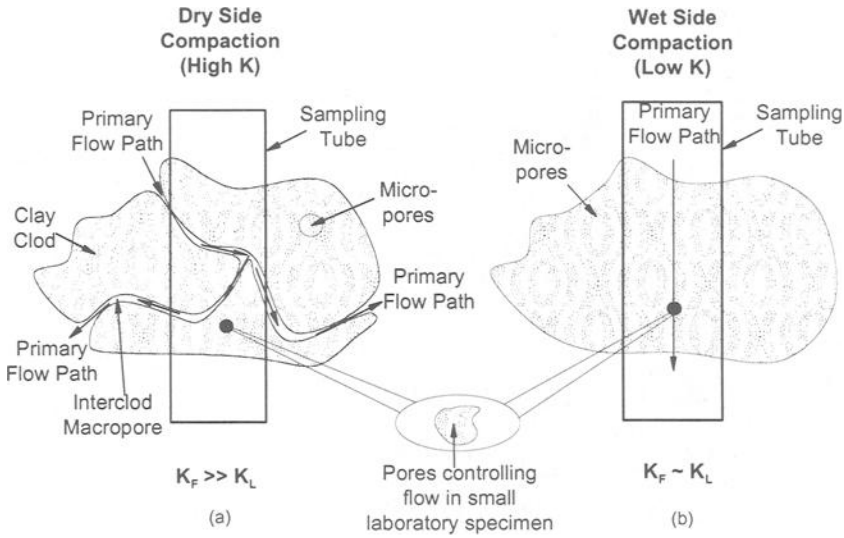

FIG. 1 -- *Schematic showing field-scale and laboratory-scale flow paths in clay compacted on dry and wet sides of the compaction curve.*

There are exceptions to this trend. For example, clays carefully pulverized prior to compaction (e.g., claystones processed with a road reclaimer) can be comprised of small clay clods. The pores between these small clay clods can be adequately represented in small laboratory specimens regardless of the compaction water content. Also, the pores in soil-bentonite mixtures are generally sufficiently small so that they are adequately represented in small laboratory specimens (Kraus et al. 1997) regardless of the compaction condition.

Database

A database was compiled consisting of 51 clay liners or test pads where K_F and K_L were measured. A portion of this database is also contained in Benson and Boutwell (1992) and Benson et al. (1999). The database in this paper contains additional data not included in Benson and Boutwell (1992) and, unlike Benson et al. (1999), the database in this paper includes data from clay liners designed to have hydraulic conductivity less than 10^{-5} cm/s. These clay liners were included in the database because they are often used in covers.

For inclusion in the database, a clay liner had to meet several criteria. First, large-scale measurements of hydraulic conductivity were required. These measurements had to be made using sealed double-ring infiltrometers (SDRIs) per ASTM *Standard Test Method for Field Measurement of Infiltration Rate Using a Double-Ring Infiltrometer with a Sealed-Inner Ring* (D 5093) or with lysimeters. Second, laboratory hydraulic conductivity tests had to be conducted using ASTM *Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible-*

Wall Permeameter (D 5084) on undisturbed specimens removed from the liner or pad with thin-wall sampling tubes (diameter = 74 mm). Laboratory-compacted specimens were excluded. Third, the liner had to be at least 300-mm thick to be representative of modern practice. Fourth, at least ten field measurements of water content and dry unit weight were required so that basic statistics could be computed. Characterization also had to include at least the Atterberg limits and the percentage fines. Finally, liners affected by desiccation or other environmental stresses were excluded. Scale-dependence of hydraulic conductivity could then be expressed simply as K_F/K_L.

Currently there is no consensus on how suction at the wetting front (H_s) should be incorporated into data reduction for SDRI tests. Thus, H_s was assumed to be zero in all SDRI tests. Wang and Benson (1995) show that this assumption can result in K_F being slightly overestimated, but the overestimate is less than a factor of two provided the depth of the wetting front (D_F) is at least half the thickness of the clay liner (L). In all of the SDRI tests summarized in this study, D_F/L was > 0.5.

Basic characteristics of sites in the database are summarized in Table 1. All of the sites except Sites 2 and 24 are from real liners or test pads built for commercial or industrial applications. The projects were located in 18 states and one Canadian province. Field-testing was conducted by 21 geotechnical firms. A wide variety of soils is represented. The plasticity index ranges from 5 to 58 and the percent fines ranges from 30 to 99. The average relative compaction (i.e., average dry unit weight, γ_d, divided by maximum dry unit weight, $\gamma_{d,max}$) varied from 86% (based on standard Proctor, SP) to 98% (based on modified Proctor, MP). The mean water content ranged from 3.5% below SP optimum water content to 7.1% above SP optimum water content. Liners constructed using MP were compacted at mean water contents of 0.3% to 6.0% above their optimum water contents.

The ratio K_F/K_L ranged from 0.12 to 266, which suggests that very large K_F/K_L, such as those measured by Day and Daniel (1985) and Elsbury et al (1990), are rare. A similar finding is reported by Benson et al. (1999). Furthermore, at 10 sites, K_F/K_L was less than 1; i.e., K_F was *lower* than K_L. This occurs because multi-lift redundancy, afforded by liners constructed with multiple lifts, reduces the "overall" or "equivalent" hydraulic conductivity of a soil liner (i.e., K_F). The effect of redundancy is not reflected in specimens extracted from individual lifts for laboratory testing (Boutwell and Rauser 1990, Benson et al. 1994b). Also, backpressuring may have resulted in higher degree of saturation during permeation in the laboratory than in the field, which would increase K_L relative to K_F.

Analysis

The background discussion established that K_F/K_L should be related to the wetness of the soil at compaction relative to the *line of optimums*. Basing the analysis on the line of optimums is essential. A single arbitrary optimum water content (e.g., from a standard or modified Proctor compaction test) depends on the compactive effort employed. In the field, the compaction effort rarely matches that in laboratory tests and is hardly ever known (Tritico and Langston 1995). Thus, a comparison based on a prescribed optimum water content is intrinsically flawed, unless by chance the compactive effort in the field matches that in the laboratory. Basing the comparison on the line of optimums avoids this pitfall, because by definition the line of optimums is the locus of optima for all compactive efforts (Benson et al. 1999).

Two parameters describing the compaction condition relative to the line of optimums were found to be strongly related to K_F/K_L. The first parameter is the percentage of field compaction data points falling wet of the line of optimums (P_o). The second parameter is the difference in initial saturation (ΔS_i), which is the difference between the mean degree

TABLE 1 -- *Summary of database*[a].

Site	Source	PI	Fines (%)	K_L (cm/s)	K_F/K_L	Test Type	$w-w_o$ (%)	Relative Compaction	$<S_i>$ (%)	P_o (%)	ΔS_i (%)
1	Benson & Boutwell (1992)	10	65	$3.2x10^{-8}$	8.8	SDRI-FP	0.10	93 m	83.0	44	-3.0
2	Benson & Boutwell (1992)	14	89	$2.0x10^{-7}$	100.0	SDRI-FP	-3.50	91	64.1	0	-21.0
3	Benson & Boutwell (1992)	25	85	$3.0x10^{-9}$	50.0	SDRI-FP	2.60	99	86.3	28	-4.2
4	Benson & Boutwell (1992)	10	85	$8.0x10^{-9}$	1.1	LY	1.50	102	93.2	98	15.0
5	Benson & Boutwell (1992)	34	95	$5.0x10^{-9}$	2.2	SDRI-PP	3.40	96	88.0	80	2.6
6	Benson & Boutwell (1992)	20	75	$8.0x10^{-9}$	11.2	SDRI-FP	0.30	97	79.1	32	-7.5
7	Benson & Boutwell (1992)	19	88	$2.4x10^{-8}$	3.5	SDRI-FP	2.00	92 m	75.0	64	-10.9
8	Benson & Boutwell (1992)	17	91	$1.8x10^{-8}$	2.2	SDRI-FP	5.70	82 m	82.2	60	-8.2
9	Benson & Boutwell (1992)	13	75	$9.0x10^{-9}$	13.3	SDRI-FP	0.80	94 m	76.2	8	-12.6
10	Benson & Boutwell (1992)	26	87	$9.0x10^{-9}$	1.0	SDRI	3.00	93	85.1	95	-3.5
11	Benson & Boutwell (1992)	21	71	$1.0x10^{-8}$	0.7	LY	6.00	91 m	93.3	90	5.8
12	Benson & Boutwell (1992)	31	60	$8.0x10^{-9}$	4.0	LY	5.1	90 m	86.4	50	3.0
13	Benson & Boutwell (1992)	30	79	$2.0x10^{-9}$	1.3	LY	3.70	92 m	95.1	75	-7.4
14	Benson & Boutwell (1992)	28	75	$3.0x10^{-9}$	0.7	LY	3.00	97 m	96.2	98	4.6
15	Benson & Boutwell (1992)	17	86	$1.3x10^{-8}$	1.0	SDRI-PP	2.60	98	94.0	100	8.2

[a]Notes: PI = plasticity index, Fines = percent finer than 75 μm, SDRI = sealed double-ring infiltrometer, PP = partially penetrating wetting front, FP = fully-penetrating wetting front, LY = lysimeter, m = based on modified Proctor compaction curve, w = average compaction water content, w_o = optimum water content based on standard or modified Proctor, NA = not available, $<S_i>$ = average degree of saturation at compaction, P_o = percentage of field compaction data falling wet of the line of optimums, and ΔS_i = mean degree of saturation at compaction less the degree of saturation corresponding to the line of optimums.

TABLE 1 -- *Summary of database (continued).*

Site	Source	PI	Fines (%)	K_L (cm/s)	K_F/K_L	Test Type	$w-w_o$ (%)	Relative Compaction	$<S_i>$ (%)	P_o (%)	ΔS_i (%)
16	Benson & Boutwell (1992)	31	90	$1.7x10^{-8}$	5.4	SDRI-PP	1.80	97	87.0	71	0.3
17	Benson & Boutwell (1992)	20	70	$4.8x10^{-8}$	0.4	SDRI-PP	2.00	102	89.1	78	12.5
18	Benson & Boutwell (1992)	58	99	$4.4x10^{-9}$	0.8	SDRI-PP	5.00	96	94.1	98	8.3
19	Benson & Boutwell (1992)	22	77	$3.7x10^{-8}$	0.8	SDRI-PP	4.00	98	91.3	91	13.5
20	Benson & Boutwell (1992)	17	31	$6.8x10^{-8}$	9.9	SDRI-PP	0.40	97	79.2	25	-7.6
21	Benson & Boutwell (1992)	17	30	$2.6x10^{-8}$	11.9	SDRI-PP	-1.40	102	83.1	25	-3.0
22	Benson & Boutwell (1992)	17	90	$1.5x10^{-8}$	20.7	SDRI-PP	-0.50	100	83.3	51	-2.8
23	Benson & Boutwell (1992)	34	95	$3.0x10^{-9}$	2.0	LY	3.00	96	92.0	100	13.0
24	Benson & Boutwell (1992)	15	92	$7.5x10^{-7}$	2.5	LY	1.40	93	82.2	77	-11.9
25	Benson & Boutwell (1992)	18	52	$1.5x10^{-8}$	0.7	SDRI-PP	3.60	93	89.1	95	0.2
26	Benson & Boutwell (1992)	16	91	$1.9x10^{-8}$	2.3	LY	3.10	95 m	90.0	81	3.8
27	Benson et al. (1999)	23	94	$3.0x10^{-9}$	266	SDRI-FP	-0.4	98	83.8	8	-6.2
28	Trast & Benson (1995)	23	94	$2.5x10^{-7}$	1.0	SDRI-FP	1.5	99	92.8	80	4.3
29	Benson et al. (1999)	42	96	$2.4x10^{-8}$	0.8	SDRI-FP	NA	NA	93.7	NA	-3.0
30	Trast & Benson (1995)	38	94	$2.5x10^{-8}$	2.0	SDRI-PP	-3.5	106	87.9	60	-0.7
31	Khire et al. (1997)	15	64	$3.2x10^{-6}$	0.6	LY	5.6	86	86.4	78	0.7
32	Khire et al. (1997)	5	70	$2.2x10^{-7}$	1.4	LY	4.6	86	85.2	96	0.2
33	Authors' Files	18	73	$1.4x10^{-8}$	1.1	LY	2.0	99	96.6	89	7.4

TABLE 1 -- *Summary of database (continued).*

Site	Source	PI	Fines (%)	K_L (cm/s)	K_F/K_L	Test Type	$w-w_o$ (%)	Relative Compaction (%)	$<S_i>$ (%)	P_o (%)	ΔS_i (%)
34	Benson et al. (1999)	41	88	2.5×10^{-9}	3.5	SDRI-FP	2.9	94	89.5	71	-0.5
35	Benson et al. (1999)	46	94	9.1×10^{-9}	1.7	SDRI-FP	2.1	97	93.3	81	0.4
36	Dunn & Palmer (1994)	22	93	2.2×10^{-8}	3.0	SDRI-FP	4.9	92	88.2	59	0.1
37	Dunn & Palmer (1994)	26	52	3.2×10^{-8}	1.2	SDRI-FP	7.1	93	79.8	88	6.9
38	Dunn & Palmer (1994)	15	97	3.5×10^{-8}	43	SDRI-FP	0.1	100	84.6	42	-3.4
39	Dunn & Palmer (1994)	29	62	2.3×10^{-8}	13.9	SDRI-FP	3.5	91	75.7	40	-1.1
40	Trast & Benson (1995)	14	85	2.1×10^{-8}	10	SDRI-FP	-1.5	100	83.4	17	-8.8
41	Trast & Benson (1995)	16	89	3.0×10^{-8}	6.7	SDRI-FP	2.6	102	77.2	6	-10.4
42	Trast & Benson (1995)	18	74	2.1×10^{-8}	4.5	SDRI-FP	0.1	100	89.4	57	-0.3
43	Bergstrom et al. (1995)	10	74	2.2×10^{-8}	0.8	SDRI-FP	0.0	98	90.8	84	1.0
44	Authors' Files	19	NA	3.0×10^{-8}	3.6	SDRI-PP	2.2	95	86.9	65	-2.3
45	Authors'Files	23	59	1.6×10^{-8}	5.1	SDRI-PP	0.4	100	85.6	75	-1.5
46	Benson et al. (1999)	27	99	3.0×10^{-8}	1.3	SDRI-PP	3.9	96	97.9	92	7.5
47	Benson et al. (1999)	15	87	1.3×10^{-8}	3.1	SDRI-PP	0.1	99	87.0	80	7.0
48	Authors'Files	23	94	1.5×10^{-8}	26.7	SDRI-FP	0.5	100	87.5	42	2.5
49	Benson et al. (1999)	19	97	3.0×10^{-8}	1.2	SDRI-PP	-1.5	102	97.0	78	2.8
50	Benson et al. (1999)	17	74	9.1×10^{-9}	3.3	SDRI-FP	2.3	96	83.3	77	3.4
51	Benson et al. (1999)	17	48	4.9×10^{-8}	0.3	SDRI-PP	1.7	96	83.4	50	-1.6

of saturation at the time of compaction and the degree of saturation corresponding to the line of optimums. For compaction wet of the line of optimums, $\Delta S_i > 0$. The following describes how P_o and ΔS_i are computed.

Computations

The parameter P_o was calculated for each clay liner as illustrated using Fig. 2. The combinations of compaction water content (w) and dry unit weight (γ_d) measured during construction were plotted along with the compaction curves developed before and/or during construction. The line of optimums was identified and the number of the w-γ_d combinations falling wet of the line of optimums (N_w) was counted along with the total number of w-γ_d measurements (N). The parameter P_o was then calculated as

$$P_o = \frac{N_w}{N} \times 100 \qquad (1)$$

For the data shown in Fig. 2, $N_w = 15$, $N = 35$, and $P_o = 44\%$.

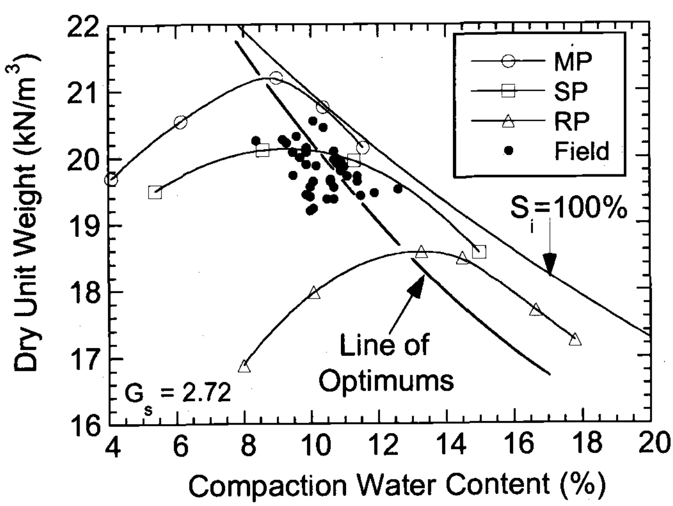

FIG. 2 -- *Example showing computation of P_o. RP corresponds to reduced Proctor effort as defined in Daniel and Benson (1990).*

Difference in initial saturation (ΔS_i) was computed by first determining the average initial saturation ($\langle S_i \rangle$). Initial saturation (S_i) is the degree of saturation at the time of compaction (Boutwell and Hedges 1989, Benson et al. 1994b) and is computed with the compaction water content (w) and dry unit weight (γ_d) using the traditional formula for degree of saturation, i.e.

$$S_i = \frac{w}{\gamma_w / \gamma_d - 1/G_s} \quad (2)$$

where γ_w is the unit weight of water and G_s is the specific gravity of solids. To compute $\langle S_i \rangle$, S_i is computed for each w-γ_d combination using Eq 2 and the average of these S_i is determined:

$$\langle S_i \rangle = \frac{1}{N}\sum_{i=1}^{N} S_i \quad (3)$$

The next step is to compute the S_i corresponding to the line of optimums, S_{io}. Generally the line of optimums corresponds to approximately S_{io} = 85% (Benson and Boutwell 1992, Blotz et al. 1998). However, soil-specific values for S_{io} were computed by determining S_i at optimum for each compaction curve developed for the soil (i.e., by using optimum water content and maximum dry unit weight in Eq 2). The average of these S_i was used as S_{io}. The parameter ΔS_i was the computed as

$$\Delta S_i = \langle S_i \rangle - S_{io} \quad (4)$$

For the example shown in Fig. 2, $\langle S_i \rangle$ = 83%, S_{io} = 86%, and ΔS_i = -3%.

Results of Database Analysis

A graph of K_F/K_L vs. P_o is shown in Fig. 3. The ratio K_F/K_L decreases with increasing P_o. Also, no apparent difference exists between the data added in this paper and that from Benson and Boutwell (1992). The trend line (dashed) falling along the central tendency of the data corresponds to

$$\log(K_F / K_L) = 1.77 - 0.019P_o \quad (5)$$

Eq 5 was fit by least-squares regression (R^2 = 0.85). For low values of P_o, K_F/K_L can be nearly 300, whereas $K_F/K_L \sim 1$ when P_o is greater than 90%. This latter observation gives rise to the question: "How many combinations of w-γ_d can fall dry of the line of optimums before K_F will be significantly higher than K_L?" The data in Fig. 3 suggest that no more than one or two out of every ten measurements can fall dry of the line of optimums if K_F is to be approximately equal to K_L.

There is one distinct outlier in Fig. 3 that deserves mention (Site 51, P_o = 50%, K_F/K_L = 0.3). The test pad at Site 51 was constructed with gravelly clay. Gravel in the clay abraded along the sampling tubes during pushing and extraction of the sample, resulting in disturbance and small cracks. As a result, hydraulic conductivities from the small laboratory specimens tested for Site 51 were significantly (~ 4 times) higher than K_F measured with the SDRI (6x10^{-9} cm/s). The K_F obtained from Site 51 using an SDRI is considered to be particularly reliable because nearly identical values for K_F were obtained from two-stage borehole tests (5x10^{-9} cm/s) and large (300-mm diameter) block samples (6x10^{-9} cm/s).

A graph of K_F/K_L vs. ΔS_i is shown in Fig. 4. The ratio K_F/K_L decreases with increasing ΔS_i, and K_F/K_L is approximately 1 for ΔS_i > 0. As in Fig. 3, the new data confirm the relationship reported in Benson and Boutwell (1992). The trend line (dashed)

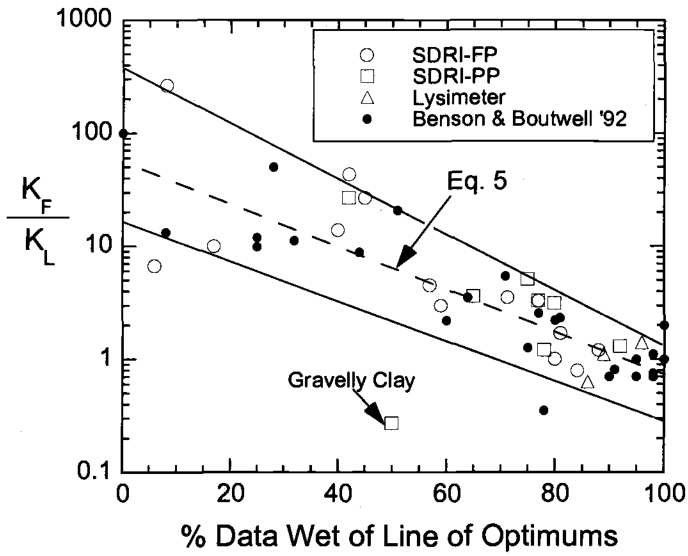

FIG. 3 -- *Relationship between K_F/K_L and P_o. Data are segregated by type of test used to measure K_F and new data vs. data from Benson and Boutwell (1992).*

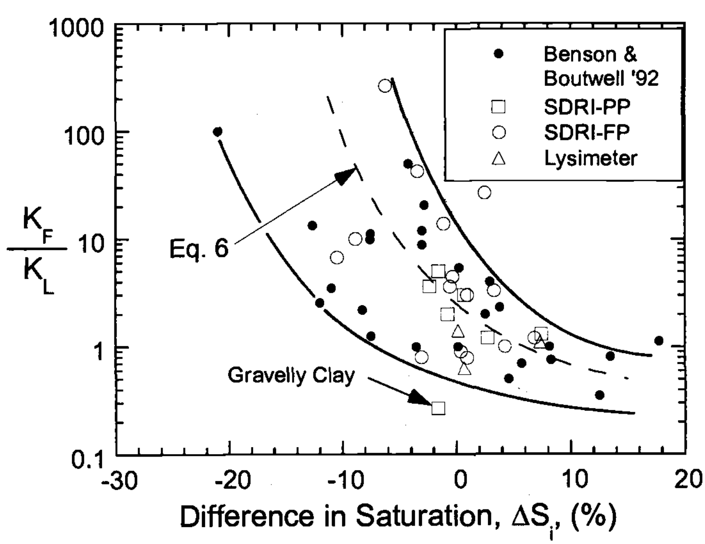

FIG. 4 -- *Relationship between K_F/K_L and ΔS_i. Data are segregated by type of test used to measure K_F and new data vs. data from Benson and Boutwell (1992).*

falling along the central tendency of the data corresponds to

$$\log(K_F / K_L) = 0.9\exp(-0.1\ \Delta S_i) - 0.5 \tag{6}$$

Eq 6 was fit by eye. The trend in Fig. 4 suggests that K_F and K_L will be practically the same if compaction is wet enough of the line of optimums so that ΔS_i is greater than 2 to 4%.

Stochastic Simulation Model

To confirm the findings shown in Figs. 3 and 4, a model was developed to simulate the effect that compaction conditions have on the "equivalent" or "overall" field hydraulic conductivity of a soil liner. A stochastic approach was used to simulate the variability of water content and dry unit weight that is typically observed in the field.

Distribution for Water Content and Dry Unit Weight

A bivariate normal distribution was used to simulate variability of water content and dry unit weight. Eq 7 is the probability density function for the bivariate normal distribution

$$f(w,\gamma_d) = \frac{1}{2\Pi\sigma_w\sigma_d\sqrt{1-\rho^2}}\exp\left\{-\frac{1}{2(1-\rho^2)}\left[\left(\frac{w-\mu_w}{\sigma_w}\right)^2 - 2\rho\left(\frac{w-\mu_w}{\sigma_w}\right)\left(\frac{\gamma_d-\mu_d}{\sigma_d}\right)+\left(\frac{\gamma_d-\mu_d}{\sigma_d}\right)^2\right]\right\} \tag{7}$$

where μ_w and μ_d are the means of w and γ_d, σ_w and σ_d are the standard deviations of w and γ_d, and ρ is the coefficient of correlation between w and γ_d. The bivariate normal distribution was selected because it simulates scatter that is typically observed in the field during construction of clay liners. For example, compaction data from a clay liner constructed for a hazardous waste landfill on the Gulf Coast of the United States are shown in Fig. 5 along with combinations of w and γ_d generated from a bivariate normal distribution. The parameters μ_w, μ_d, σ_w, σ_d, and ρ for the bivariate normal distribution were computed from the compaction data measured during construction. The simulated data and the field data are comparable.

Spatial variability of w and γ_d is characterized by σ_w, σ_d, and ρ. Low variability in w and γ_d corresponds to low σ_w, σ_d, and large ρ. High variability in water content and dry unit weight corresponds to low σ_w, σ_d, and low ρ. If compaction is along the wet side of the compaction curve, ρ is negative. Positive ρ occurs for compaction on the dry side of the compaction curve.

Values for σ_w, σ_d, and ρ were obtained from a database created using construction data from 67 clay liners in landfills in North America (Benson et al. 1992, 1994b). Three conditions were considered: low variability, moderate (or typical) variability, and high variability. Conditions corresponding to low variability were assigned the 5th percentile standard deviations from the database (σ_w = 1.3%, σ_d = 0.36 kN/m^3) and the 95th percentile of ρ (ρ = -0.91). Conditions corresponding to high variability were assigned the 95th percentile standard deviations (σ_w = 4.2%, σ_d = 0.96 kN/m^3) and the 5th percentile of ρ (ρ= -0.49). Moderate conditions were assigned median values, i.e.: σ_w = 2.2%, σ_d = 0.61 kN/m^3, and ρ = -0.81.

Monte Carlo simulation was used to generate realizations of water content and dry unit weight. Because the properties of compacted clay liners typically exhibit little spatial correlation structure (Benson 1991), no serial correlation was specified for subsequent pairs of w and γ_d. Bivariate normal deviates were generated using the Box-Muller method (Box and Muller 1958). As shown in Fig. 5, this simulation approach yielded random deviates that appear like actual field data.

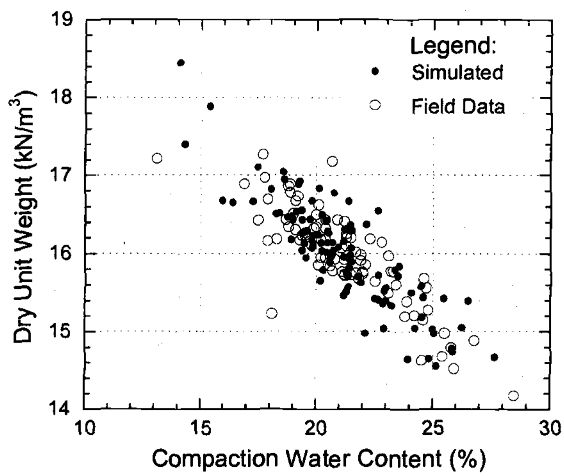

FIG. 5 -- *Simulated and measured combinations of water content and dry unit weight. Field data are from a clay liner constructed for a hazardous waste landfill located on the Gulf Coast of the United States.*

For all analyses that were conducted, a set of compaction-hydraulic conductivity curves for a glacial till from Wisconsin (Fig. 6) were used to define reasonable values for μ_w and μ_d and the relationship between hydraulic conductivity, water content, and compactive effort. The curves are from Othman and Benson (1993) and are based on laboratory-compacted specimens. Other curves could have been used and similar results would have been obtained.

Using laboratory compaction and hydraulic conductivity data to simulate field-scale hydraulic conductivity-water relationships is simplistic. In the field, hydraulic conductivity is likely to be more sensitive to water content than would be suggested by Fig. 6 (e.g., see Elsbury et al. 1990, Benson 1994). Nevertheless, to the writers' knowledge, no data are currently available to describe the relationship between hydraulic conductivity, water content, and compactive effort in the field. The results obtained with the model should be viewed in light of this simplification.

Computing Distribution of Hydraulic Conductivities

A hydraulic conductivity was assigned to each combination of water content and dry unit weight that was sampled from the bivariate normal distribution. The hydraulic conductivity was selected using the following procedure

(i) A combination of water content (w_i) and dry unit weight ($\gamma_{d,i}$) is sampled from the bivariate normal distribution using the random number generator.

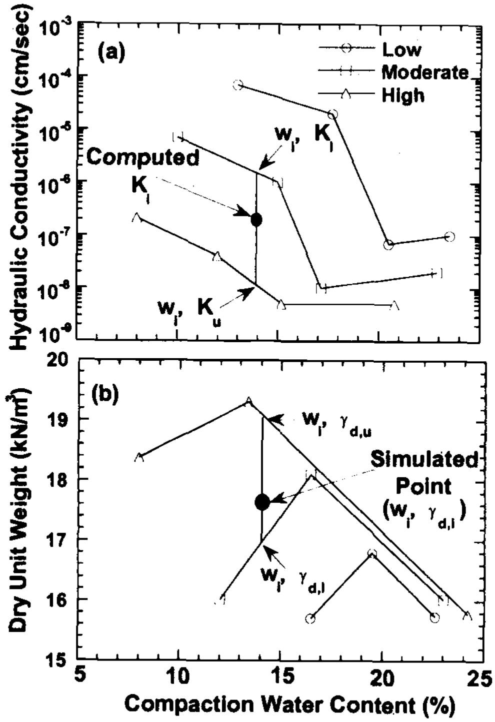

FIG. 6 -- *Hydraulic conductivity-water content curves (a) and compaction curves (b) used in simulation.*

(ii) The w_i and $\gamma_{d,i}$ are compared to the compaction curves corresponding to low, moderate, and high compactive effort (Fig. 6b). The relative distance between the sampled combination of water content and dry unit weight and the adjacent compaction curves (ξ_i) is computed by

$$\xi_i = \frac{\gamma_{d,i} - \gamma_{d,l}}{\gamma_{d,u} - \gamma_{d,l}} \tag{8}$$

where $\gamma_{d,l}$ is the dry unit weight on the lower compaction curve corresponding to w_i and $\gamma_{d,u}$ is the dry unit weight on the upper compaction curve corresponding to w_i (Fig. 6b).

(iii) The hydraulic conductivity K_i corresponding to w_i and $\gamma_{d,i}$ is approximated using the log-linear relationship shown in Eq 9, which is based on the hydraulic conductivity curves (Fig. 6a) corresponding to the compaction curves (Fig. 6b) used in Steps i and ii.

$$\log K_i = \log K_l - \xi_i(\log K_l - \log K_u) \tag{9}$$

In Eq 9, K_l is the hydraulic conductivity corresponding to w_i on the lower compaction curve and K_u is the hydraulic conductivity corresponding to w_i on the upper compaction curve (Fig. 6a).

(iv) Steps i through iii are repeated for each combination of w_i and $\gamma_{d,i}$ sampled from the bivariate normal distribution.

Simulation

Simulations were conducted for a series of μ_w and μ_d that could be realized for the compaction curves in Fig. 6b and a compaction specification requiring the water content to exceed optimum (16.5%) and the dry unit weight to exceed 95% of $\gamma_{d,max}$ (17.19 kN/m^3) for moderate effort (e.g., standard Proctor). Although this "percent compaction specification" has drawbacks (see later discussion), it is the most common type of compaction specification. Benson et al. (1999) report that a percent compaction specification is used for more than 90% of the clay liners being constructed today. The parameters μ_w and μ_d ranged from 16.5 to 19.5% and 17.27 to 18.37 kN/m^3, respectively. Twenty-one sets of simulations were conducted to uniformly cover values of μ_w and μ_d possible within the compaction specification. For each set of μ_w and μ_d, the parameters σ_w, σ_d, and ρ were set to simulate low, moderate, and high variability Altogether 63 simulations were conducted

Each sampled pair (w_i, $\gamma_{d,i}$) was compared to the compaction criterion. If the generated values were outside the compaction criterion or corresponded to S_i>100, they were rejected as if a technician had tested the soil and required that it be re-worked. Samples of w and γ_d were collected until 100 points falling within the compaction criterion were obtained.

The hydraulic conductivity (K_i) corresponding to each combination of w_i and $\gamma_{d,i}$ was computed using Eqs 8 and 9 (i.e., 100 realizations of K_i were computed for each simulation). This set of hydraulic conductivities was assumed to represent the spatial distribution of hydraulic conductivity in the liner corresponding to the combinations of w_i and $\gamma_{d,i}$ that were generated. The "overall" or "equivalent" field hydraulic conductivity (K_{FS}) was assumed to equal the geometric mean of the K_i, i.e.

$$K_{FS} = \exp\left(\frac{1}{N}\sum_{i=1}^{N} \ln K_i\right) \tag{10}$$

In this case, N = 100. The subscript 'S' is used to denote simulated conditions. Benson et al. (1994c) show that the geometric mean is a conservative estimator of the equivalent average field hydraulic conductivity of a soil liner with heterogeneous hydraulic conductivity. In this analysis, K_{FS} is assumed to be analogous to K_F measured with an SDRI or lysimeter.

Laboratory-scale hydraulic conductivity (K_{LS}) was assumed to equal 1×10^{-8} cm/s, which corresponds to the hydraulic conductivity obtained for compaction wet of optimum with moderate compactive effort for the soil in Fig. 6a. Because the pores controlling flow are very small when the soil is compacted wet of optimum, K_{LS} is analogous to the laboratory-scale hydraulic conductivity (K_L) described in the database.

Results from Simulation Model

Results of the simulations are presented in Fig. 7 in terms of K_{FS}/K_{LS} vs. P_o or ΔS_i along with trend lines from the field data (Eq 5). As with the field data, K_{FS}/K_{LS} decreases as P_o increases (Fig. 7a). More of the compaction data falling wet of the line of optimums results in lower K_{FS}. Furthermore, Fig. 7a suggests that P_o should exceed approximately 80% if K_{FS}/K_{LS} near 1 is to be obtained. A similar result was determined from analyzing the field data (e.g., for $K_F/K_L = 1$, P_o should be > 90%, see Fig. 3).

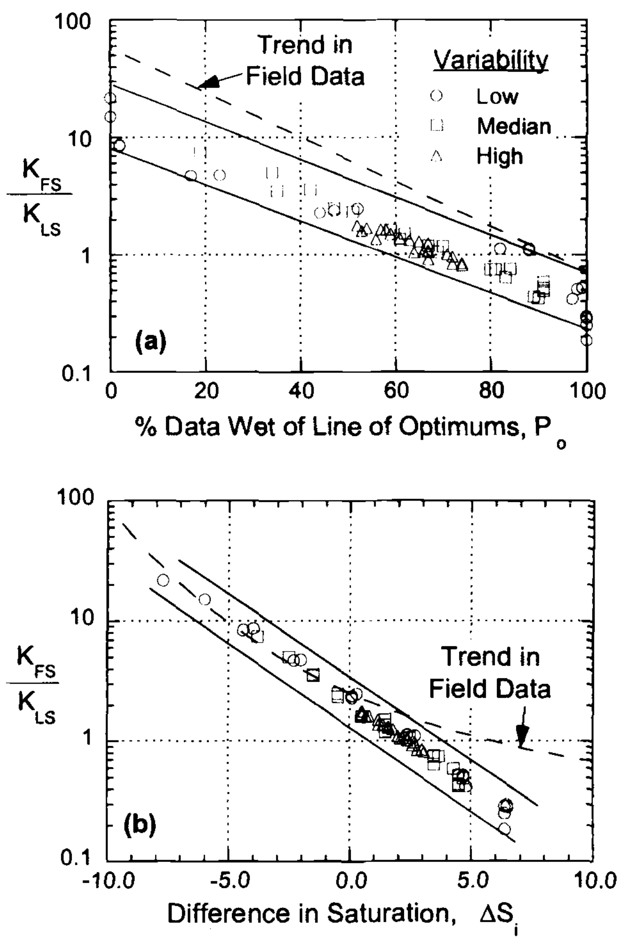

FIG. 7 -- *Results of simulations along with trend lines from the field data: (a) K_{FS}/K_{LS} vs. P_o and (b) K_{FS}/K_{LS} vs. ΔS_i.*

The trend line from the field data in Fig. 7a falls above the simulation results. This may occur because clay compacted dry of optimum in the field can have significantly higher hydraulic conductivity than laboratory-compacted specimens prepared at the same water content and dry unit weight (Elsbury et al. 1990, Benson 1994). The field hydraulic conductivity is higher dry of optimum because clods in laboratory-compacted specimens are much smaller than clods in the field, and therefore laboratory-compacted specimens contain smaller macropores than macropores in the field (Benson and Daniel 1990). This effect is in addition to the aforementioned difference between the size of pores existing in the field and those captured in samples collected in thin-wall sampling tubes.

The simulation model also includes only a single set of hydraulic conductivity-water content curves corresponding to a single soil. In contrast, the database contains data from a breadth of soils, some of which may have hydraulic conductivity that is more sensitive to water content than indicated in Fig. 6a. For example, Benson and Daniel (1990) found that the hydraulic conductivity of a highly plastic Gulf Coast clay compacted in the laboratory using the standard Proctor method can vary six orders of magnitude from dry to wet of optimum water content. The hydraulic conductivities shown in Fig. 6 span only three orders of magnitude for a given effort. The modeling results also exhibit less scatter than the field data, because other factors affect hydraulic conductivity in the field (e.g., variations in soil composition, clod size effects, and testing errors) in addition to those included in the model.

The relationship between K_{FS}/K_{LS} and ΔS_i is shown in Fig. 7b. The graph shows the expected trend; as ΔS_i increases, K_{FS}/K_{LS} decreases. For $\Delta S_i > 2.5\%$, K_{FS}/K_{LS} is close to 1. Again, this quantity is in reasonable agreement with the field data. And, as in Fig. 7a, the simulation results exhibit less scatter than the field data.

A counter-intuitive aspect of Fig.7 is that the largest and smallest values of K_{FS}/K_{LS} are realized when the water content and dry unit weight have *low* variability. The very high and very low values of K_{FS}/K_{LS} occur with low variability because the sampled values of water content and dry unit weight have little scatter about their mean. Hence, when the mean is dry of the line of optimums, nearly all of the sampled combinations of water content and dry unit weight fall dry of the line of optimums and K_{FS} is much higher than K_{LS}. In contrast, when the mean falls wet of the line of optimums, nearly all of the sampled combinations of water content and dry unit weight fall wet of the line of optimums and K_{FS} is comparable to K_{LS}. As the variability in water content and dry unit weight increases, these effects are tempered because the data are not as closely grouped near the mean.

Summary and Recommendations

The field data and modeling results show that large field-scale measurements of hydraulic conductivity (K_F) are likely to be comparable to small-scale measurements of hydraulic conductivity made in the laboratory (K_L) on specimens collected in thin-wall sampling tubes (74-mm diameter) when compaction is wet of the line of optimums. Compaction wet of the line of optimums results in remolding of clods and elimination of large interclod macropores. As a result, pores controlling flow in the field are comparable to those contained in small laboratory specimens, and K_F and K_L are similar. In contrast, when compaction is dry of the line of optimums, large interclod macropores control the field hydraulic conductivity, and these pores are inadequately represented in small laboratory-scale specimens. As a result, K_F can be appreciably larger than K_L.

To achieve $K_F \sim K_L$, compaction specifications should require that the percentage of data falling wet of the line of optimums exceed 80-90% and the difference in initial saturation (ΔS_i) exceed 2 to 4%. However, these recommendations will not ensure K_F/K_L

= 1. Other factors unrelated to compaction control (e.g., desiccation and freeze-thaw, variations in soil properties, etc.) may result in macroscopic defects and excessive field-scale hydraulic conductivity.

The criteria P_o > 80-90% and ΔS_i > 4% may not be achieved with traditional "percent compaction" specifications that require the water content to exceed an optimum water content and that the dry unit weight to exceed a percentage of the corresponding maximum dry unit weight. A typical percent compaction specification is shown in Fig. 8a. Approximately half of the "acceptable zone" falls dry of the line of optimums. Thus, the contractor is equally likely to compact the liner dry of the line of optimums or wet of the line of optimums. The key factor affecting the portion of the acceptable zone where compaction occurs is the water content of the soil in the borrow source and not engineered control of the compaction process. In contrast, a specification based on the line of optimums (Fig. 8b) implicitly requires compaction wet of the line of optimums, and will likely result in $K_F \sim K_L$. Methods to develop a compaction specification such as the one shown in Fig. 8b are described in Mundell and Bailey (1985), Boutwell and Hedges (1989), and Daniel and Benson (1990).

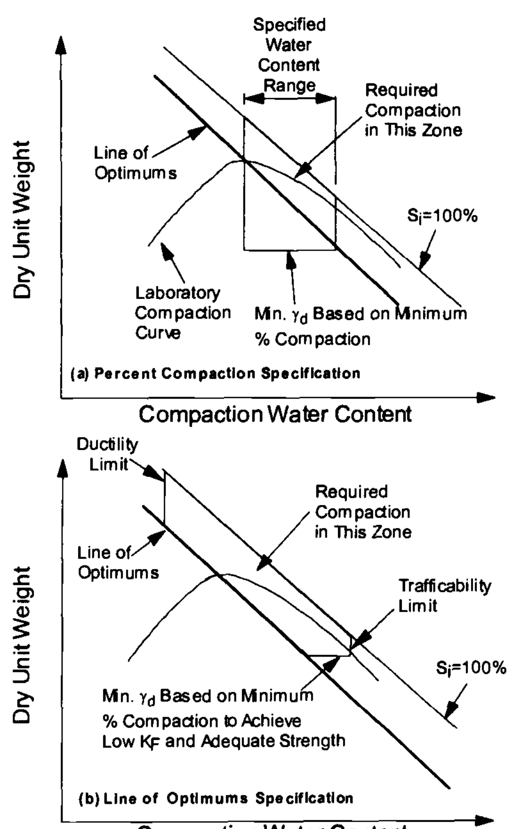

FIG. 8 -- *Schematics of (a) "Percent compaction" specification and (b) "line of optimums" specification (adapted from Benson et al. 1999).*

The authors recommend that a "line of optimums" specification be used when constructing compacted clay liners, in part to facilitate $K_F \sim K_L$. In addition, Benson et al. (1999) found that all liners in their database that were constructed with at least 80% of the compaction data falling wet of the line of optimums had $K_F < 10^{-7}$ cm/s. Thus, use of a "line of optimums" compaction specification results in low K_F, which is usually the objective when constructing compacted clay liners.

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Variability of Initial Subgrade Modulus at Ohio SHRP Test Road

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Abstract: Premature deterioration of highway pavements is often attributed to the base and subgrade. In highway construction projects, construction quality control during the subgrade preparation involves nuclear moisture-density testing at specified depths at equally spaced stations along the entire project length. However, the authors' field study at the Ohio-SHRP Test Road revealed that this conventional approach was not effective for minimizing the variability of the subgrade stiffness. First of all, it was not possible to achieve a uniform moisture-density (or dry density) condition throughout the entire project. Secondly, even when the nuclear test data indicated that a fairly uniform relative compaction was established within an area, the subgrade stiffness could still vary significantly. The reasons for this were that the conventional approach could provide only very localized data on the state of the soil compaction and that the relative compaction alone could not be a reliable indicator to describe the soil stiffness. An alternative approach is to employ a rapid, nondestructive stiffness test method, such as Falling Weight Deflectometer (FWD), Dynaflect, and soil stiffness gage (SSG), in the field to monitor the stiffness of the entire thickness directly during the subgrade preparation.

Keywords: subgrade modulus, resilient modulus, soil compaction, QC/QA, pavement performance, in-situ testing

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Introduction

Construction and maintenance of highway pavements cost billions of dollars to state and federal governments each year. While pavements usually perform close to the design standards, premature localized deterioration is encountered frequently and is a sign that further improvements are needed in pavement construction methods. This localized distress may be in the form of rutting, cracking, or other types of failure. Under repeated traffic loading and environmental cycles, the dimensions of these areas often continue to expand. Assuming that the pavements are designed to acceptable standards, nonuniformity or variability in physical properties within each pavement layer is considered to be responsible for these progressive failures. Premature deterioration is often attributed to problems in the base and subgrade. There are some tools available to detect the areas of progressive deterioration, such as falling weight deflectometer (FWD), Dynaflect, ground penetrating radar (GPR), and dynamic cone penetrometer (DCP).

To support the Strategic Highway Research Program (SHRP), the Ohio Department of Transportation (DOT) constructed in August 1996 a 4.8-km (3-mile) long test road on the U.S. Rt. 23, near the city of Delaware, about 40 km (25 miles) north of Columbus, Ohio (Ohio Research Institute for Transportation and Environment 1997). The test road encompassed four Specific Pavement Studies (SPS) formulated by the Strategic Highway Research Program (SHRP):

SPS-1 Strategic Study of Structural Factors for Flexible Pavement.
SPS-2 Strategic Study of Structural Factors for Rigid Pavement.
SPS-8 Study of Environmental Effects in the Absence of heavy Traffic.
SPS-9 Asphalt Program Field Verification Studies.

The northbound lanes were constructed of portland cement concrete (PCC) for the SPS-2 experiment. The southbound lanes were constructed of asphalt concrete (AC) for the SPS-1 and SPS-9 experiments. On a ramp section to the southbound lanes, PCC and AC sections were developed to address the SPS-8 experiment. Site topography was fairly flat, and fine-grained soils were found. A uniform subgrade material was preferred. However, as shown in Table 1(a), three different soil types (A-4, A-6, A-7-6) were encountered. Table 1(b) lists basic compaction properties of the three types of subgrade soil. Groundwater table at the site was typically encountered 1.3 m (4.3 ft) below the top of the subgrade.

Table 1 - *Soil Types Identified at Ohio SHRP Test Site*

(a) Locations of Three Soil Types

AASHTO Soil Classification	SHRP Section (PCC)	SHRP Section (AC)
A-4	N/A	390110, 390160, 390902
A-6	390202, 390205, 390207 390211, 390262	390111
A-7-6	N/A	390107

Table 1 - *Soil Types Identified at Ohio SHRP Test Site*

(b) Basic Compaction Properties of Three Soil Types

AASHTO Soil Classification	Optimum Moisture Content, OMC (%)	Maximum Dry Density or Dry Unit Weight kg/m^3 (pcf)
A-4	13.5	1,874.3 (117.0)
A-6	14.6	1,835.9 (114.6)
A-7-6	15.8	1,794.2 (112.0)

Specific pavement base types/combinations utilized in the test sections included: dense-graded aggregate base (DGAB), asphalt-treated base (ATB), permeable asphalt-treated base (PATB), permeable cement-treated base (PCTB), and lean concrete base (LCB). During construction, structural response and environmental parameter sensors were installed in 34 of the 40 sections by research teams from six universities in Ohio (Akron, Case Western Reserve, Cincinnati, Ohio, Ohio State, and Toledo), with Ohio University taking a leadership role. The pavement response parameters included horizontal strains in pavement, vertical displacement of each pavement layer, vertical pressure at layer interfaces, and joint opening in the PCC pavement. Environmental/seasonal parameters included temperature in pavement, frost penetration depth, water table depth, moisture content and soil suction in subgrade. In addition, an on-site weather station was set up to record the climatological data.

QC/QA During Ohio Test Road Construction

Subgrade soil was compacted by a sheepsfoot-roller. During the subgrade preparation, specifics of the Ohio DOT Item 203.12 (1997) were enforced. Applicable compaction specifications were:

- Soil subgrade with maximum laboratory dry unit weight of 1,681-1,920 kg/m^3 (105-119.9 pcf) shall be compacted to no less than 100% of the maximum dry density. The maximum dry unit weight shall be determined as determined by AASHTO T99 or other approved method.
- Subgrade under new pavement and paved shoulders shall be compacted to a depth of 0.30 m (12 inches) below the surface of the subgrade and to a width of 0.46 m (18 inches) beyond the edge of the surface of the pavement, paved median, paved shoulder, or to the back of the adjacent curb and gutter.

The quality control described above represents typical practices by a highway agency in the U.S. It focused only on the dry unit weight and did not address any quality control measure on the in-place moisture content of the subgrade soil layers during the field compaction.

Basic Theory of Soil Compaction and Resilient Modulus

Theory of soil compaction is presented in a relatively simple manner in many geotechnical engineering textbooks. Proctor (1933) established a fundamental theory of soil compaction through his laboratory experiment method. Lambe (1958a) discovered that the soil's micro-structure could differ significantly between the dry and wet sides of the optimum moisture content (OMC). Soils compacted dry side of the OMC have flocculated structures (particles arranged randomly), while soils compacted on the wet side of the OMC have dispersed structures (particles arranged more uniformly). To make the matter more complicated, the position of the OMC in the dry unit weight vs. moisture content plot is not absolutely fixed and can move as a function of the compaction method and compaction energy applied per unit soil volume.

Because of this difference in the micro-structure, dissimilar behaviors are often observed between the two sides of the OMC (Lambe 1958b). A soil compacted dry of the OMC behaves like a brittle material, showing a high peak strength and a small strain at failure. A soil compacted wet of the OMC behaves more like a ductile material, showing a lower peak strength and a larger strain at failure. Under low pressure, a soil compacted wet of the OMC is more compressible. Under higher pressure, the opposite trend is observed. Initial tangent modulus on the stress vs. strain curve becomes smaller as the molding moisture content increases.

Development of a mechanistic design approach for flexible pavements has led to several laboratory studies on the behavior of cohesive soils under repeated (pulsed) loading. This new design procedure uses resilient modulus (modulus under the repeated loading) to characterize material property of each pavement layer. Thompson and Robnett (1976) performed a series of resilient modulus tests on subgrade soils found in Illinois. They saw through correlations that the degree of saturation had a significant influence on the magnitude of resilient modulus at both 95% and 100% compaction rates. Johnson (1986) measured resilient modulus of subgrade soils in eastern Tennessee area. According to his study, the resilient modulus decreased from about 97 to 62 MPa (14 to 9 ksi) when the degree of saturation increased from low 80% to high 90%. Li and Selig (1994) analyzed eleven sets of resilient modulus data found in literature and proposed a general method to estimate the resilient modulus of compacted fine-grained subgrade soils. They observed that three factors had a major influence on the magnitude of resilient modulus. They were loading condition (deviator stress), soil type and micro-structure, and soil physical state (moisture content, dry density). They observed that resilient modulus could vary between 14 MPa and 140 MPa (2 ksi and 20 ksi) for the same soil due to the changes in these factors. Under a constant dry density, their best-fit curve to the available data indicated that the resilient modulus increased nonlinearly as the molding moisture content decreased within a range of OMC $\pm$ 5%.

In-Situ Subgrade Testing

Being part of the SHRP SPS studies, extensive in-situ testing was performed on various pavement materials/layers during construction. The field tests included nuclear moisture/density tests on soil layer compacted as well as Falling Weight Deflectometer (FWD) testing on the subgrade, base, and pavement after completion of each layer (Sargand et al. 1998). The FWD tests on the subgrade were performed about the same time as the nuclear moisture/density tests of the subgrade layers.

Subgrade moisture and density data were measured by a nuclear gage at a depth of 305 mm (12 in.) along the centerline of the driving lane. A minimum of three readings were taken within each test section. The field test data are presented in Figures 1 and 2. Each entry in these plots is the average of four readings taken at 90° apart. According to the test data, the measured moisture contents and dry density values were bounded between 6% and 16% and 1,750 kg/m^3 and 2,100 kg/m^3 (109 and 131 pcf), respectively.

Falling Weight Deflectometer (FWD) tests were conducted on each test section every 15.2 m (50 ft.) along the centerline of the driving lane and in the right wheel path (Sargand et al. 1998). Subgrade FWD tests were completed just after the subgrade soil had been compacted. The circular plate had a diameter of 150 mm (5.9 in.), and the tests involved four different weights. Elastic modulus can be back-calculated by using a solution obtained by Boussinesq for a circularly loaded rigid area (Lambe and Whitman 1969):

$$D = \frac{\pi r\left(1 - \mu^2\right)P}{2E} \tag{1}$$

where D = uniform deflection of the circular plate; r = radius of the circlar plate; μ = Poisson's ratio of the soil (= 0.4); P = average applied pressure; and E = average elastic modulus of the soil.

The above is a theoretical equation and can only estimate the elastic modulus within the realm of its assumptions (homogeneous, isotropic, and elastic half-space; a uniform deflection). One fundamental difference between these two test methods is that the nuclear moisture/density tests measured the average physical conditions that existed only within the top 305 mm (12 in.) zone, while the FWD data represented the overall quality of thicker subgrade zone in a cumulative manner. Tables 2(a) and 2(b) list the maximum, mean, and minimum moduli of the subgrade for each section. In 12 of the 19 SPS-2 sections and 7 of the 17 sections from SPS-1 and SPS-9 combined, the standard deviation exceeded the minimum modulus, indicating that a relatively large fluctuations often existed in the magnitude of the elastic moduli. The back-calculated modulus varied from 16.9 MPa to 409 MPa (2.45 ksi to 59.33 ksi) in the SPS-2 experiment zone, from 14.3 MPa to 254 MPa (2.07 ksi to 36.84 ksi) in the SPS-1 and 9 zones.

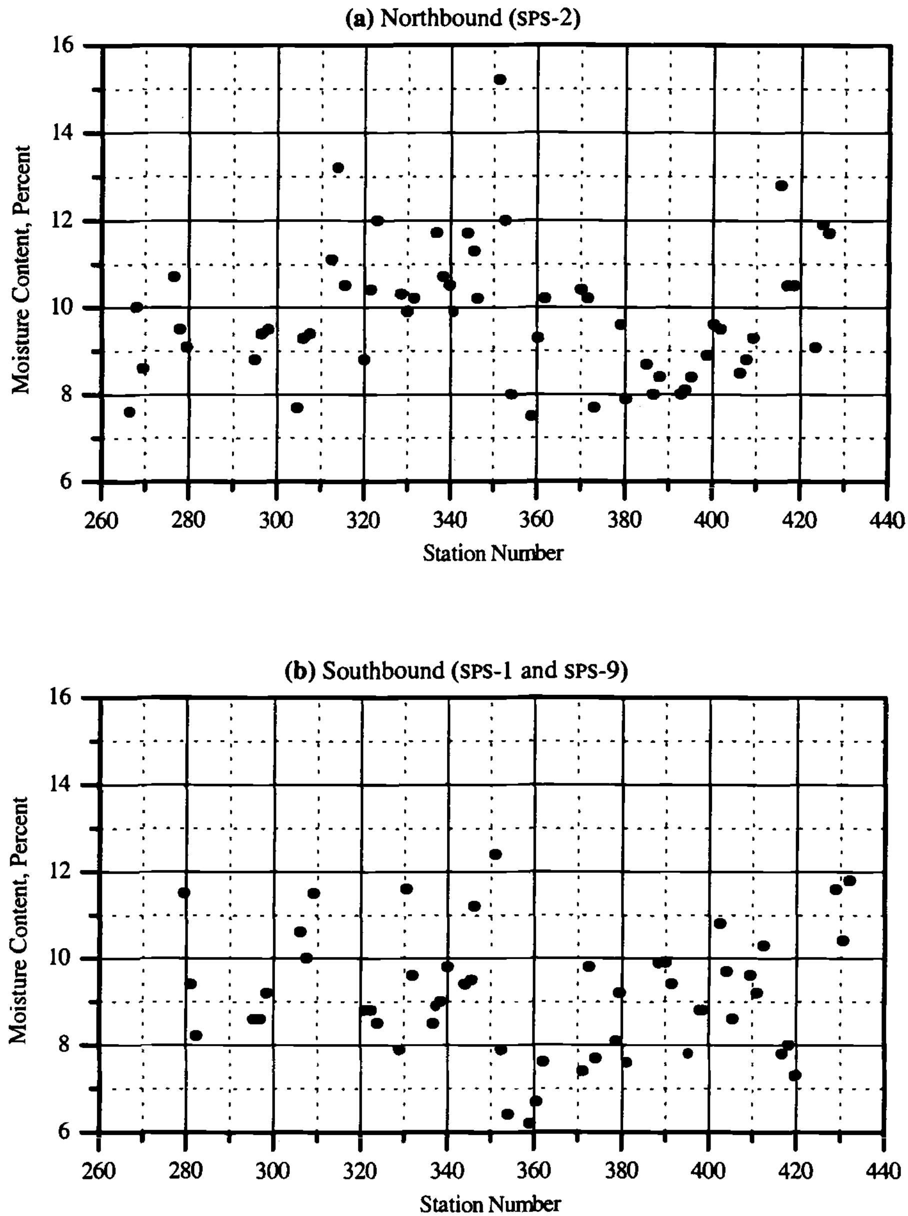
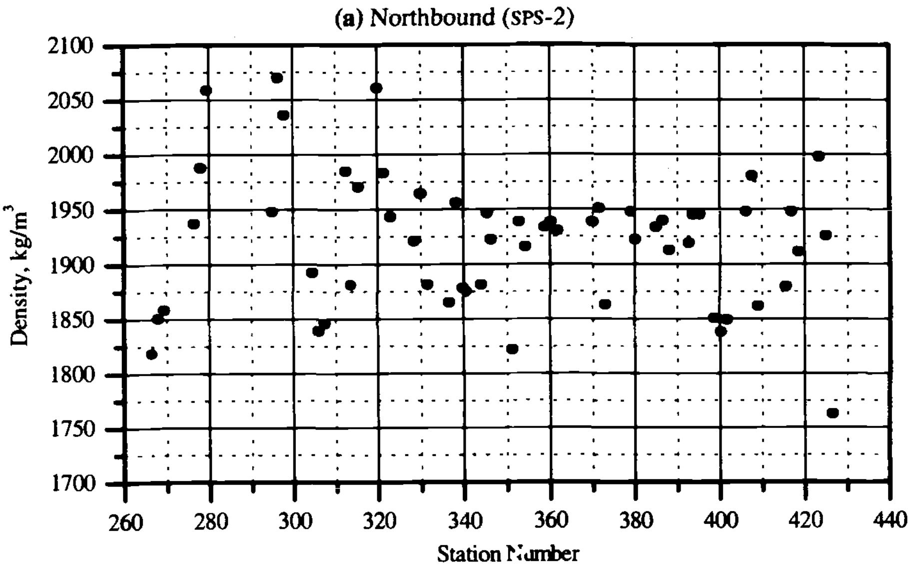


Figure 1 - *Moisture Content Measurements at Various Locations of Ohio Test Road*



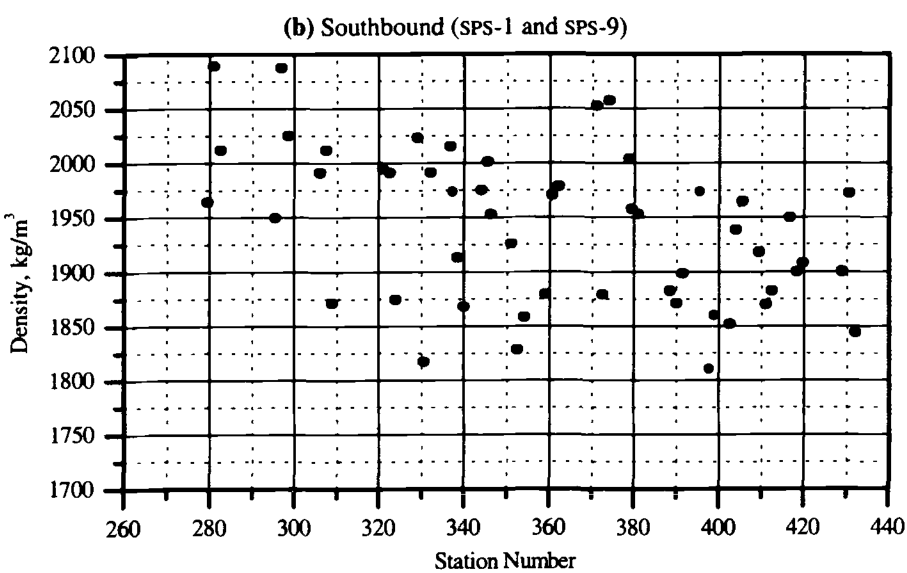


Figure 2 - *Dry Density Measurements at Various Locations of Ohio Test Road*

Table 2(a) - *Variations of Back-Calculated Elastic Moduli for SPS-2 Experiment Sections*

SHRP Section No.	Back-Calculated Ave. Elastic Modulus MPa (ksi):			
	Minimum	Mean	Maximum	Std. Deviation
390201	18.6 (2.7)	62.4 (9.1)	117.0 (17.0)	28.6 (4.1)
390202	23.7 (3.4)	123.4 (17.9)	255.0 (37.0)	70.0 (10.1)
390203	58.8 (8.5)	103.0 (14.9)	171.9 (24.9)	28.2 (4.1)
390204	69.3 (10.1)	205.3 (29.8)	409.2 (59.4)	95.4 (13.8)
390205	16.9 (2.5)	64.3 (9.3)	151.6 (22.0)	37.1 (5.4)
390206	17.5 (2.5)	87.8 (12.7)	166.3 (24.1)	46.1 (6.7)
390207	67.3 (9.8)	117.8 (17.1)	183.9 (26.7)	36.2 (5.3)
390208	68.6 (10.0)	112.7 (16.3)	222.2 (32.2)	39.0 (5.7)
390209	17.3 (2.5)	71.6 (10.4)	176.4 (25.6)	54.1 (7.8)
390210	21.7 (3.1)	71.1 (10.3)	130.5 (18.9)	31.4 (4.6)
390211	61.6 (8.9)	109.3 (15.9)	145.2 (21.1)	21.2 (3.1)
390212	63.1 (9.2)	140.9 (20.4)	251.2 (36.4)	49.0 (7.1)
390259	20.5 (3.0)	79.0 (11.5)	135.5 (19.7)	33.9 (4.9)
390260	24.3 (3.5)	101.5 (14.7)	196.6 (28.5)	41.6 (6.0)
390261	23.1 (3.4)	124.1 (18.0)	228.6 (33.2)	43.9 (6.4)
390262	41.4 (6.0)	107.8 (15.6)	222.4 (32.3)	42.6 (6.2)
390263	26.4 (3.8)	93.7 (13.6)	176.8 (25.6)	42.7 (6.2)
390264	17.2 (2.5)	34.3 (5.0)	87.2 (12.6)	15.8 (2.3)
390265	65.9 (9.6)	88.7 (12.9)	112.2 (16.3)	18.3 (2.7)

Table 2(b) - *Variations of Back-Calculated Elastic Moduli for SPS-1 and SPS-9 Experiment Sections*

SHRP Section No.	Back-Calculated Ave. Elastic Modulus MPa (ksi):			
	Minimum	Mean	Maximum	Std. Deviation
390101	28.2 (4.1)	80.6 (11.7)	155.6 (22.6)	40.1 (5.8)
390102	54.5 (7.9)	140.5 (20.4)	234.8 (34.0)	58.3 (8.5)
390103	40.5 (5.9)	108.2 (15.7)	172.1 (25.0)	30.2 (4.4)
390104	56.1 (8.1)	116.2 (16.9)	228.8 (33.2)	48.7 (7.1)
390105	74.5 (10.8)	107.2 (15.5)	156.5 (22.7)	22.8 (3.3)
390106	45.7 (6.6)	123.3 (17.9)	190.6 (27.6)	40.9 (5.9)
390107	54.8 (7.9)	115.6 (16.8)	195.3 (28.3)	39.4 (5.7)
390108	81.9 (11.9)	130.7 (19.0)	205.3 (29.8)	44.0 (6.4)
390109	27.0 (3.9)	79.4 (11.5)	186.4 (27.0)	39.2 (5.7)
390110	33.6 (4.9)	89.3 (13.0)	159.3 (23.1)	37.5 (5.4)
390111	27.5 (4.0)	124.7 (18.1)	254.5 (36.9)	62.0 (9.0)
390112	20.6 (3.0)	95.3 (13.8)	196.0 (28.4)	43.4 (6.3)
390159	14.3 (2.1)	39.8 (5.8)	84.3 (12.2)	22.0 (3.2)
390160	72.5 (10.5)	128.5 (18.6)	210.4 (30.5)	38.6 (5.6)
390901	62.3 (9.0)	186.0 (27.0)	423.2 (61.4)	99.6 (14.4)
390902	33.4 (4.8)	106.9 (15.5)	222.2 (32.2)	47.8 (6.9)
390904	48.7 (7.1)	98.8 (14.3)	215.5 (31.3)	41.1 (6.0)

Laboratory Resilient Modulus Tests

Fifteen representative bag samples of the subgrade soil were transported to the ORITE material testing laboratory and subjected to resilient modulus testing (according to the SHRP Protocol P-46). These included six samples (390106, 390107, 390108, 390110, 390111, 390160) from the SPS-1 experiment, six (390202, 390205, 390207, 390209, 390211, 390262) from the SPS-2 experiment, two (390809, 390810) from the SPS-8 experiment, and one (390902) from the SPS-9 experiment. Based on the soil classification data, these soil samples can be divided into three groups:

A-4 Group ------------ 390110, 390160, 390809, 390810, 390902.
A-6 Group ------------ 390111, 390202, 390205, 390207, 390211, 390262.
A-7-6 Group ---------- 390107.

The SHRP requires that each soil be tested at/near the in-place moisture and density conditions. However, in the current investigation each sample was compacted close to the field dry density at a minimum of three moisture contents to examine the effect of moisture content on the resilient modulus. Resilient modulus (M_R) is defined as:

$$M_R = \sigma_d/\varepsilon_R \tag{2}$$

where σ_d = repeatedly applied deviatoric stress; and ε_R = elastic (or recoverable) axial strain.

The resilient modulus test equipment utilized in the study was a state-of-the-art system featuring a large triaxial chamber, an electro-servo actuator, and a computerized load command generation and data acquisition (Masada 1998). Each soil sample was initially air-dried, moistened for 24 hours prior to the test, and recompacted inside a 152-mm (6-in.) split mold according to the SHRP P-46. Each specimen was subjected to 500 conditioning load cycles (σ_d = 4 psi, σ_3 = 6 psi) prior to the actual testing, as seen in Table 3.

There were some common resilient behaviors that were observed among all three soil types. Resilient modulus remained relatively high at low levels of deviator stress. As the deviator stress increased, the resilient modulus sharply declined and became almost constant above the deviatoric stress of 62 kPa (9 psi). Effect of confining stress was negligible on the magnitude of resilient modulus.

Figures 3(a) through 3(c) present the typical test results for each soil type. As shown in Figure 3(a), the resilient modulus of A-7-6 soil sample declined by more than 80% when its moisture content was raised from 10.5% to 21.8% (the dry density/unit weight stayed about the same at 1,831.1 kg/m^3 or 114.3 pcf). For the specimens belonging to the A-6 soil group, moisture content was varied between 7.6% and 20.5% and the dry density/unit weight was maintained mostly within 1,803.9. $\pm$ 70.5 kg/m^3 (112.6 $\pm$ 4.4 pcf) in the laboratory. The resulting resilient modulus ranged from 13.1 to 206.8 MPa (1.9 to 30.0 ksi). For the specimens belonging to the A-4 soil group, moisture content was varied between 11% and 21.5% and the dry density/unit weight was maintained mostly within 1,781.4 $\pm$ 96.1 kg/m^3 (111.2 $\pm$ 6 pcf) in the laboratory. The resulting resilient modulus ranged from 14.5 to 180.6 MPa (2.1 to 26.2 ksi). In spite of some inherent scattering of the test data, detailed examinations of the test results indicated that:

- Resilient modulus decreased as the moisture content increased. This trend was seen clearly for A-6 and A-7-6 soil groups but not for A-4 soil sample.
- A concave downward, nonlinear, bell-shaped relation existed between the resilient modulus and moisture content.
- The higher the clay content was, the more sensitive the magnitude of the resilient modulus was to changes in moisture content.
- An increase in dry unit weight led to a higher resilient modulus at low moisture contents but to a lower resilient modulus at high moisture contents.

Table 3 - *Load Sequences Utilized in Subgrade Resilient Modulus Testing*

Load Sequence No.	Confining Pressure σ_3, kPa (psi)	Deviator Stress σ_d, kPa (psi)	Number of Repetitions
0	41.4 (6.0)	27.6 (4.0)	500
1	41.4 (6.0)	13.8 (2.0)	100
2		27.6 (4.0)	100
3		41.4 (6.0)	100
4		55.2 (8.0)	100
5		68.9 (10.0)	100
6	27.6 (4.0)	13.8 (2.0)	100
7		27.6 (4.0)	100
8		41.4 (6.0)	100
9		55.2 (8.0)	100
10		68.9 (10.0)	100
11	13.8 (2.0)	13.8 (2.0)	100
12		27.6 (4.0)	100
13		41.4 (6.0)	100
14		55.2 (8.0)	100
15		68.9 (10.0)	100

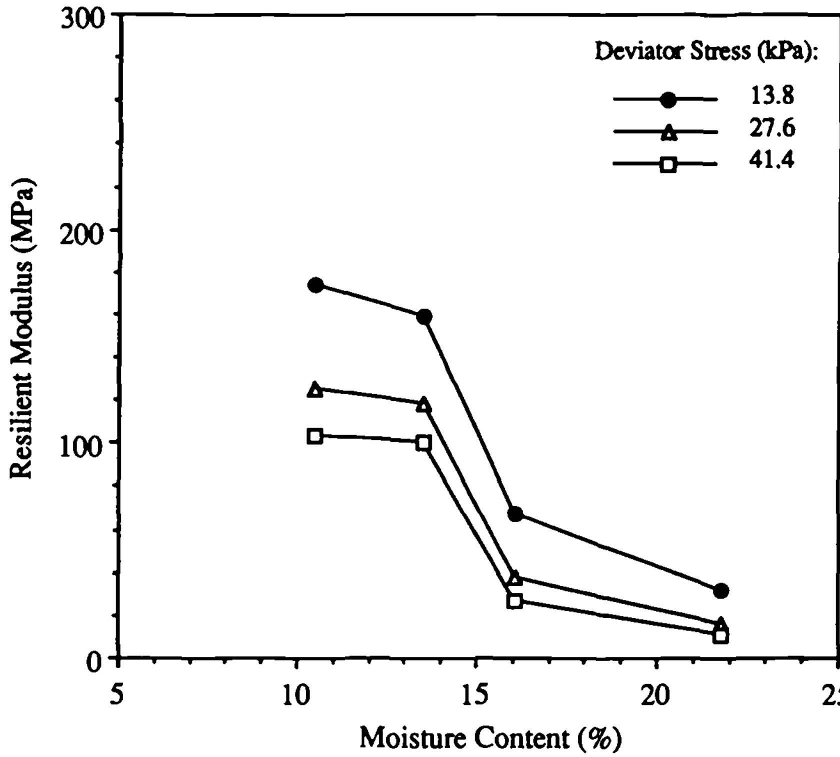


Figure 3(a) - *Resilient Modulus Vs. Moisture Content Plot for A-7-6 Soil Specimens*

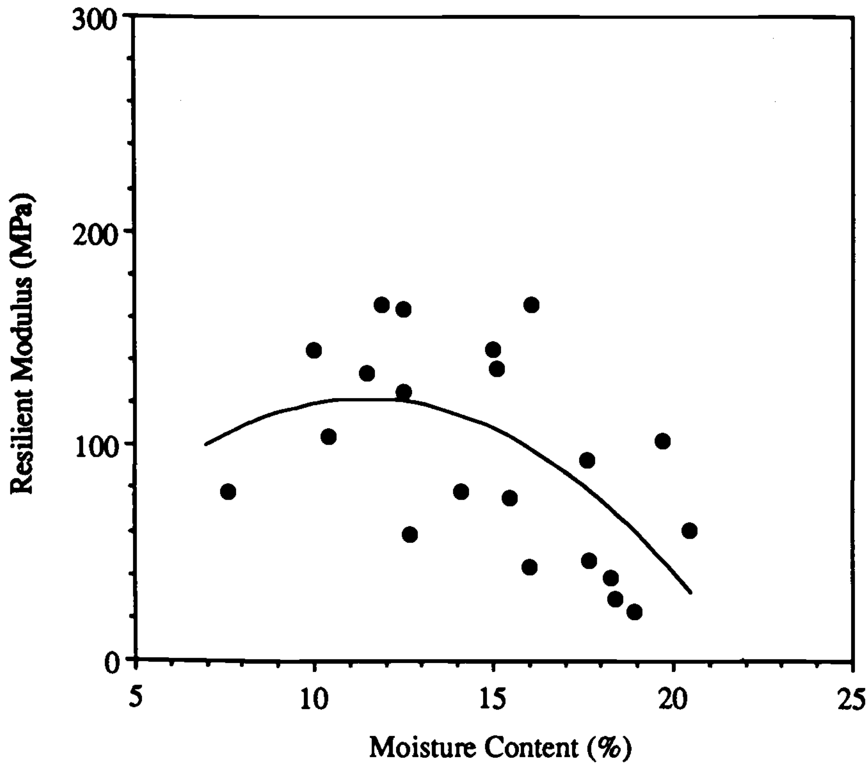


Figure 3(b) - *Resilient Modulus Vs. Moisture Content Plot for A-6 Soil Specimens (Deviator Stress = 13.8 kPa)*

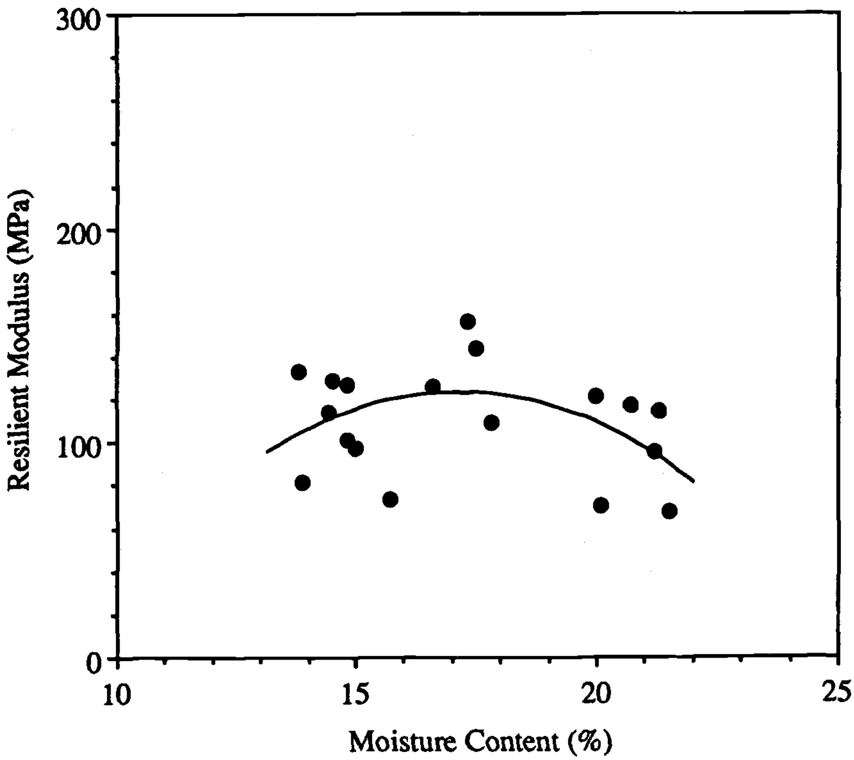


Figure 3(c) - *Resilient Modulus Vs. Moisture Content Plot for A-4 Soil Specimens (Deviator Stress = 13.8 kPa)*

Conclusions

In the Ohio SHRP Test Road study, a nuclear gage was utilized to monitor the in-place dry density condition within the top 305 mm (12 in.) of the recompacted subgrade. Also, initial variability of subgrade stiffness was back-calculated using the FWD test data. The results showed that the subgrade stiffness varied greatly along the length of each test section and through the entire project, although the subgrade soil layers all satisfied the applicable compaction requirements. When the test results from the above two in-situ test methods were compared, it was not possible to establish a correlation between the dry density and subgrade stiffness.

A series of resilient modulus tests were conducted on the representative subgrade soil samples in the laboratory. The test results showed that the modulus under simulated traffic loading can vary significantly due to variations in the moisture content and deviatoric stress even when the dry unit weight stayed about the same. Similar findings have been reported in literature previously. The laboratory test results also indicated that the resilient modulus of the subgrade soil sample with higher clay content was more sensitive to changes in the moisture content.

Excessive rutting was observed at a limited number of locations in Ohio SHRP Section 390101 (Sargand et al. 1998). A forensic study of this section revealed that this premature distress in the pavement was caused by insufficient stiffness of the subgrade soil at these locations. This reinforced the belief that during subgrade construction monitoring of the dry density (or relative compaction) alone would not be sufficient to assure the pavement performance. Its stiffness must be measured and controlled. There are some techniques already available to do so, which include Falling Weight Deflectometer (FWD), Dynaflect, and dynamic cone penetrometer (DCP). Recently, a new portable, nondestructive device (soil stiffness gage or SSG by Humboldt Manufacturing Co., Norridge, Ill.) has been developed that can measure the stiffness at a rate of one test per minute. Details on this device can be found in an article authored by Fiedler et al. (1998).

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Quality Control of Earth Fills Using Time Domain Reflectometry (TDR)

Reference: Lin, C-P., Siddiqui, S. I., Feng, W., Drnevich, V. P., and Deschamps, R. J., **"Quality Control of Earth Fills Using Time Domain Reflectometry (TDR)"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384,* D. W. Shanklin, Ed., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: Measurement of soil density and water content in compacted fills is the principal means of quality control to assure adequate performance. Current testing methods have various limitations, including the use of hazardous materials, limitations in accuracy, the need for extensive calibration, or the test duration. A new technique using time domain reflectometry (TDR) to measure the water content and density of soil is introduced. The purpose of this paper is to present a historical and theoretical background of this new approach. Prototype equipment was developed for routine use in the quality control testing of compacted soils. The method was evaluated with theoretical study and laboratory experiments. The results of the TDR method are compared with results from conventional methods on actual construction sites. The advantages and limitations of this new method are also discussed.

A TDR device is used to transmit an electromagnetic wave into the soil and receive a reflected waveform. The dielectric constant of the soil can be calculated from the travel time of the waveform. Water content and density are the basis used to assess the quality of a compacted earth fill. These are the same factors that affect the dielectric properties of soil; so a relationship between them can be established. A procedure to measure soil density in-situ using TDR was developed. This procedure obtains results comparable in accuracy to existing methods such as the nuclear density gage and the sand-cone test. The test duration is approximately 15 minutes. A new approach is under development to interpret the reflected waveform and extract additional information regarding the electromagnetic soil properties. This approach appears to be very promising and has the potential to provide even more rapid and accurate test results for assessing the water content and density of compacted soils.

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Keywords: water content, density, field compaction control, time domain reflectometry, dielectric constant

Nomenclature

a	calibration constant
b	calibration constant
c	the velocity of an electromagnetic wave in free space (2.997925×10^8 m/s)
CC	coaxial cylinder transmission line, a compaction mold with a rod passing through the soil along the centerline
CH	coaxial head, device that provides a transition from coaxial cable to the measuring probe
K_a	apparent dielectric constant
$K_{a,field}$	dielectric constant of soil in place
$K_{a,mold}$	dielectric constant of the soil in the mold
K_s	dielectric constant of soil solids
K_{fw}	dielectric constant of free water
K_{bw}	dielectric constant of bound water
K_{air}	dielectric constant of air
l	number of molecular water layers of tightly bound water
L	length of the probe in soil
MRP	multiple rod probe
S	specific surface of the soil particle
t	the time required by the signal to travel twice the length of the probe in soil
TDR	time domain reflectometry
v	propagation velocity of an electromagnetic wave
w	gravimetric water content
w_{field}	gravimetric water content of the soil in place
w_{mold}	gravimetric water content of the soil in the mold
α	fitting parameter
δ	thickness of one molecular water layer ($= 3 \times 10^{-10}$ m)
θ	the volumetric water content
θ_{bw}	volumetric bound water content
ρ_d	dry density of soil
ρ_w	density of water
$\rho_{d,field}$	dry density of the soil in place
$\rho_{d,mold}$	dry density of the soil in the mold
$\rho_{t,field}$	total density of the soil in the field
$\rho_{t,mold}$	total density of the soil in the mold
ρ_s	density of soil solid

Technical Terms

bound water – the water adhering to the surface of soil particles due to the interfacial effect at the interface of two phases.
dipole – a positive charge and a negative charge separated by some distance.
gravimetric water content – the ratio of the mass of water to the mass of soil particles in a soil sample.
dielectric permittivity – the electric property that measures the polarizability of a material.
polarizability – the extent of polarization in a material.
polarization – the separation of the centers of positive and negative charges in a material due to an externally applied electric field.
relaxation frequency – the frequency at which the frequency-dependent dielectric permittivity has dramatic change in value similar to a resonant phenomenon.
transmission line – typically consists of two conductors along which the voltage and current of signals are carried.
volumetric water content – the ratio of the volume of water to the total volume in a soil sample.

Introduction

Earthwork construction and compaction of fill soils are required on practically all civil engineering projects and are important components of construction in the transportation industry. The most common quality control tests used are the measurements of density and water content in compacted soils. Current testing methods have various limitations, including the use of hazardous materials, limitations in accuracy, the need for extensive calibration, or long test durations. Traditional compaction quality tests include the oven-drying method for determining water content and the sand–cone method for determining soil density. Determination of water content by oven drying is very time-consuming and delays interpretation of both laboratory and field tests. The sand-cone method is not applicable to granular soils with insufficient cohesion or particle attraction to maintain stable sides on a small hole or excavation. In the last several decades, the nuclear gage has gained popularity in compaction quality control. It measures both soil density and water content in the field. The nuclear gage requires calibration and uses hazardous materials, which leads to the necessity for safety training and expensive maintenance.

A new technique using time domain reflectometry (TDR) to measure water content and density of soil has been recently introduced (Siddiqui and Drnevich, 1995). Using a TDR device, the apparent dielectric constant of soil is obtained by measuring the velocity of an electromagnetic wave traveling through a coaxial line with soil as the insulating material. The dielectric constant is correlated with the water content and density of the soil. The objective of this paper is to present the theoretical background of this new approach and development of prototype equipment for compaction quality control. To demonstrate the methodology, field measurements at several compaction sites in Indiana were evaluated with the procedure.

Theoretical Background of TDR Method

A time-domain reflectometry (TDR) device is basically composed of a pulse generator and a sampling oscilloscope. These instruments are sometimes called cable radar. The pulse generator sends an electromagnetic pulse along a transmission line and the oscilloscope is used to observe the echoes returning back to the input. Such instruments have been used to locate faults in transmission lines since the 1930's. Fellner-Feldegg (1969) used them for measuring permittivity of liquids. Figure 1 is a system configuration of the TDR system. The TDR system used in this study is shown in Figure 2 in which a Tektronix® 1502B metallic cable tester is used as the TDR device. The transmission line of the system consists of the coaxial cable section and the measurement probe section. The probe section is inserted into the soil, as shown, to measure its dielectric property.

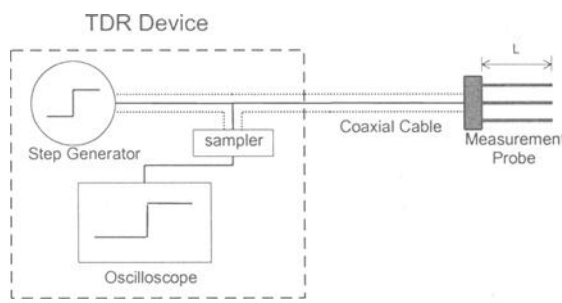


Figure 1 -- *TDR system configuration.*

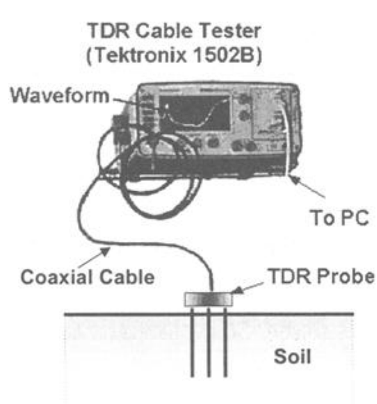


Figure 2 -- *Example of a TDR system.*

Dielectric Properties of Soil

Even though a dielectric material is electrically neutral, an externally applied electric field may cause microscopic separations of the centers of positive and negative charges, which thus behave like dipoles of charges. These charge-separation distances are on the order of atomic dimensions, but the vast numbers of dipole provide a significant effect. This phenomenon is referred to as the polarization of the dielectric. The permittivity (or dielectric constant), ε, of a material is a measure of its polarizability. It can be seen as the response of a material's electrical property to the application of an electric field. In general, permittivity is a complex number and is a function of the applied frequency of the electric field in the frequency domain. Davis and Annan (1977) indicated that the real part of the permittivity of soils does not appear to be strongly frequency dependent over the frequency range of 1 MHz to 1 GHz. They also indicated that the imaginary part was considerably less than the real part in this frequency range. Based on these conclusions, Topp et al. (1980) defined the apparent dielectric constant, K_a, as the quantity determined from measured velocity of the electromagnetic wave traveling through a transmission line. Briefly, the propagation velocity, v, of an electromagnetic wave in a transmission line is related to the apparent dielectric constant, K_a, as

$$v = \frac{c}{\sqrt{K_a}} \tag{1}$$

in which c is the velocity of an electromagnetic wave in free space (2.997925×10^8 m/s). The TDR technique measures the velocity of the electromagnetic wave traveling through a transmission line. As shown in Figure 2, the TDR device sends a step pulse down the cable which is reflected from both the beginning and end of the probe due to impedance mismatches. The two reflections cause two discontinuities in the resulting signal displayed on the TDR screen. The time difference between these two discontinuities is the time (t) required by the signal to travel twice the length (L) of the probe in soil. So the wave propagation velocity in soil is

$$v = \frac{2L}{t} \tag{2}$$

and the dielectric constant of the soil is (using Eqn. 1 and 2)

$$K_a = \left(\frac{ct}{2L}\right)^2 \tag{3}$$

In commercial TDR instruments, the term $ct/2$ is reduced to an apparent length, l, in the x-axis of the signal resulting in

$$K_a = \left(\frac{l}{L}\right)^2 \tag{4}$$

Measurement of Soil Volumetric Water Content

Because of the permanent dipole of the water molecule, the dielectric constant of water is very high (≈80 at frequencies below the water relaxation frequency). Dry soil is only polarizable by atomic and electronic polarization, leading to a low dielectric constant (typically it is less than 5). This difference makes it possible to measure the amount of water in soil by determining the soil dielectric constant. People have been investigating the use of the dielectric constant for measuring soil water content for about 40 years. References are given by Selig and Mansukhani (1975) to a number of papers which describe experimental techniques and electrical circuits used for water content measurement based on changes in the dielectric constant within the low radio frequency range. The accuracy of the measurements was limited partly because the dielectric constant of the soil at low frequencies is strongly frequency dependent and soil-type dependent due to the interface effect (Dukhin and Shilov, 1974).

Davis and Annan (1977) proposed the use of TDR, which has a dominant frequency from 1 MHz to 1 GHz, for measuring soil water content. Only after Topp et al. (1980) had published their calibration data, however, was the potential of TDR for soil science recognized. The availability of cable testers made by electronic manufacturers facilitated the introduction of TDR. The results of Topp et al. (1980) showed that the apparent dielectric constant is strongly dependent on the volumetric water content and relatively independent of soil density, texture, and salt content. The empirical equation, known as Topp's equation, was given as

$$\theta = -0.053 + 2.92\times10^{-2} K_a - 5.5\times10^{-4} K_a^{\,2} + 4.3\times10^{-6} K_a^{\,3} \tag{5}$$

where θ is the volumetric water content. In geotechnical engineering and construction quality control, water content w is usually measured in terms of gravimetric water content, and is simply referred to as "water content". Volumetric water content is related to gravimetric water content w by the following relationship

$$\theta = \frac{w\rho_d}{\rho_w} \tag{6}$$

where ρ_d is the dry density of soil, and ρ_w is the density of water. As an alternative to Eqn. 5, a linear calibration equation was proposed by Ledieu et al. (1986) and Alharthi and Lange (1987) as

$$\sqrt{K_a} = a + b\theta \tag{7}$$

where a and b are calibration constants: a = 1.545 and b = 8.787 in Ledieu et al. (1986); a = 1.594 and b = 7.83 in Alharthi and Lange (1987). Topp's equation is essentially the same as Eqn. 7 in the normal range of water content ($0.05 < \theta < 0.5$), with $a = 1.56$ and $b = 8.47$. Ledieu et al. (1986) reported that the calibration is improved if bulk dry density is included as

$$\sqrt{K_a} = a\rho_d + b\theta + c \tag{8}$$

where a, b, and c are calibration constants obtained by regression analysis: $a = 0.297$, $b = 8.79$, and $c = 1.344$ in Ledieu et al. (1986). Siddiqui and Drnevich (1995) tried to normalize the density effect and obtained an equation equivalent to Eqn. 8 with $c = 0$. They expressed the equation in terms of gravimetric water content as

$$\frac{\sqrt{K}\rho_w}{\rho_d} = a + bw \tag{9}$$

Equations 8 and 9 provide better calibration than Eqn. 5 and 7 because the density effect is taken into account. However, some prior knowledge of the soil dry density is needed to determine water content.

Measurement of Soil Density

The soil density is needed to calculate the gravimetric water content from the volumetric water content measured by TDR. It is also an important parameter for compaction quality control. Siddqui and Drnevich (1995) showed how, by making two separate TDR measurements, it is possible to measure in-place density of soil. A field probe can be used to measure the dielectric constant of soil in place ($K_{a,field}$). Some soil can be quickly taken from the location of the in-place measurement and compacted in a cylindrical mold to measure the dielectric constant of the soil in the mold ($K_{a,mold}$). Applying the calibration equation to the two measurements, two equations can be obtained

$$f(K_{a,field}, w_{field}, \rho_{d,field}) = 0 \tag{10}$$

and

$$f(K_{a,mold}, w_{mold}, \rho_{d,mold}) = 0 \tag{11}$$

where the function $f(.)$ represents a calibration equation such as Eqns. 5, 7, 8, or 9; w_{field} and w_{mold} are the gravimetric water content of the soil in place and in the mold, respectively; $\rho_{d,field}$ and $\rho_{d,mold}$ are the dry density of the soil in place and in the mold, respectively. The total density of the soil in the mold ($\rho_{t,mold}$) can be measured directly using a balance. The dry density of the soil in the mold ($\rho_{d,mold}$) can be calculated as

$$\rho_{d,mold} = \frac{\rho_{t,mold}}{1 + w_{mold}} \tag{12}$$

By assuming that the gravimetric water content of the soil in the mold is the same as the gravimetric water content of the soil in place (i.e. $w_{field} = w_{mold}$), three unknowns (w_{field}, $\rho_{d,field}$, $\rho_{d,mold}$) can then be solved by three equations (Eqn. 10, 11 and 12).

Design of TDR Prototype Equipment

One of the most important areas of TDR research has been the development of TDR probes or transmission lines. Depending on the purpose and application, different kinds of transmission lines have been developed and used successfully. However, those probes were developed mostly in soil science for permanent installations and long-term monitoring. They are not suitable for rapid installations and withdrawal following a one-time measurement. Siddiqui and Drnevich (1995) studied the factors that influence the wave transmission including the type, length, and geometry of the transmission line, and the spatial characteristics and volume of soil tested. Based on their results, transmission line components were designed and built to be robust, easy to use, and provide superior wave transmission for field measurements of water content and density.

The wave transmission line consists of a coaxial cable, a coaxial head (CH), and either a coaxial cylinder (CC) or a multiple rod probe (MRP). Each of these transmission components contains an inner conductor and an outer conductor. A typical coaxial cable consists of an inner conducting wire surrounded by a cylindrical casing that acts as the outer conductor.

The coaxial head (CH) actually consists of three parts, as shown in Figure 3: 1) a coaxial line similar to the coaxial cable; 2) a solid cylindrical head with Delrin® as the insulating material; and 3) multiple rod section consisting of a center rod and three perimeter conducting rods with air as the insulating material. The coaxial head (CH) provides a transition from coaxial cable to the measuring probe. It is designed such that it can be used both for field probe and compaction mold. The CH has four metal studs threaded into the metal head. The lengths of the central stud and two of the outer studs are the same 21 mm (0.825 in.). The fourth stud is threaded to provide adjustable length.

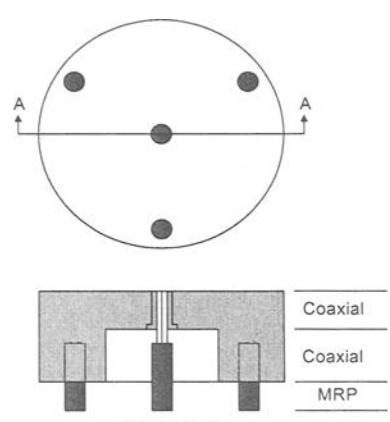


Figure 3 -- *Configuration of coaxial head.*

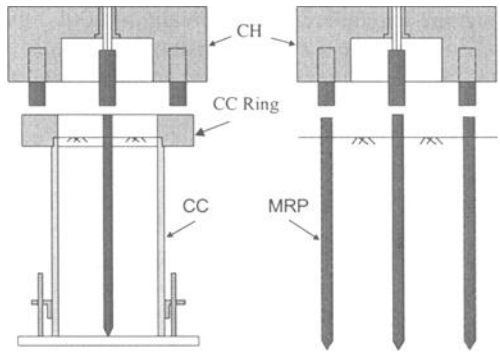


Figure 4 -- *Configuration of transmission lines.*

The coaxial cylinder (CC) transmission line consists of a CC mold, a ring and a central rod. The cylinder is filled with soil that serves as the insulating material. The central rod is made of stainless steel, 8 mm (5/16 in.) in diameter, and 234 mm (9.2 in.) long. It is driven through a guide template that rests on top of the CC mold. After the central rod is driven into the soil in the CC mold, the template is removed and the CC ring is placed on top of the CC mold. The CH is then placed on top of the CC ring and the threaded stud is adjusted until contact is made as shown in Figure 4.

The multiple rod probe (MRP) consists of a central rod and three perimeter rods of the same spacing as the CH. The rods are steel spikes, 9.5 mm (3/8 in.) in diameter and 254 mm (10 in.) long. They are inserted into the soil so that the soil acts as the insulating material. A detachable template is used to guide insertion of the spikes into the ground so that the configuration is the same as the CH. The conducting spikes are driven through the template and the template is removed. The CH is placed on the heads of the spikes so that the central stud sits on top of the central spike and the two equal outer studs sit on top of the two outer spikes. The threaded stud is adjusted until contact is made with the other spike as shown in Figure 4.

Different transmission line systems produce different waveforms and may require different interpretation. For the proposed TDR system, tests have been conducted to verify the first and second reflection points. Discontinuities in impedance occur at the connection of the coaxial cable and the CH, inside the CH, at the top of the soil surface, and at the end of the CC or MRP as shown in Figure 5(a). The goal of waveform analysis is to find the reflection points that occur at the top of the soil surface (Point 1), and at the end of the CC or MRP (Point 2). The first reflection point is at the peak of the waveform right before it starts to drop. The second reflection point will be around the portion of the waveform where it starts to rise to the steady state. The TDR system measurement is a waveform with dispersion. The first and second points are apparent reflection points obtained by an approximating procedure such as intersection of tangent lines, as shown in Figure 5(b). The first point represents the reflection from the top of the soil and the second point represents the reflection from the end of the probe. Waveform interpretation was performed with a computer connected to the TDR device. Details of the design drawing of the prototype equipment and computer algorithms used can be found in Feng et al. (1998).

Evaluation of Calibration Equations

The widespread use of TDR has resulted in a number of empirical calibration equations such as Eqns. 5, 7, 8, and 9. These empirical equations are different because they were obtained under different experimental conditions. The most relevant experimental factors are the type of soil used and the range of soil density tested. A calibration equation developed under a specific condition (i.e. soil type and density) may not be suitable for applications under different conditions. Theoretical mixing formulas were found to be more general and produced better calibration (Dirksen and Dasberg, 1993). Separating bound water in the soil matrix from free water, a four-phase soil mixing formula can be obtained from the volumetric mixing model proposed by Birchak et al. (1974).

$$K_a^{\alpha} = \left(\frac{\rho_d}{\rho_s}\right)K_s^{\alpha} + (\theta - \theta_{bw})K_{fw}^{\alpha} + \theta_{bw}K_{bw}^{\alpha} + \left(1 - \frac{\rho_d}{\rho_s} - \theta\right)K_{air}^{\alpha} \tag{13}$$

where K_s, K_{fw}, K_{bw}, and K_{air} are dielectric constants of soil solid, free water, bound water, and air, respectively; ρ_s is the density of soil solid; θ_{bw} is the volumetric bound water content; α is the fitting parameter that phenomenologically summarizes the geometry of the medium with respect to the applied electric field. For an isotropic and homogeneous medium, α becomes 0.5. The volumetric fraction of tightly bound water covering the mineral surfaces can be approximated by (Dobson et al. 1984)

$$\theta_{bw} = l\delta\rho_d S \tag{14}$$

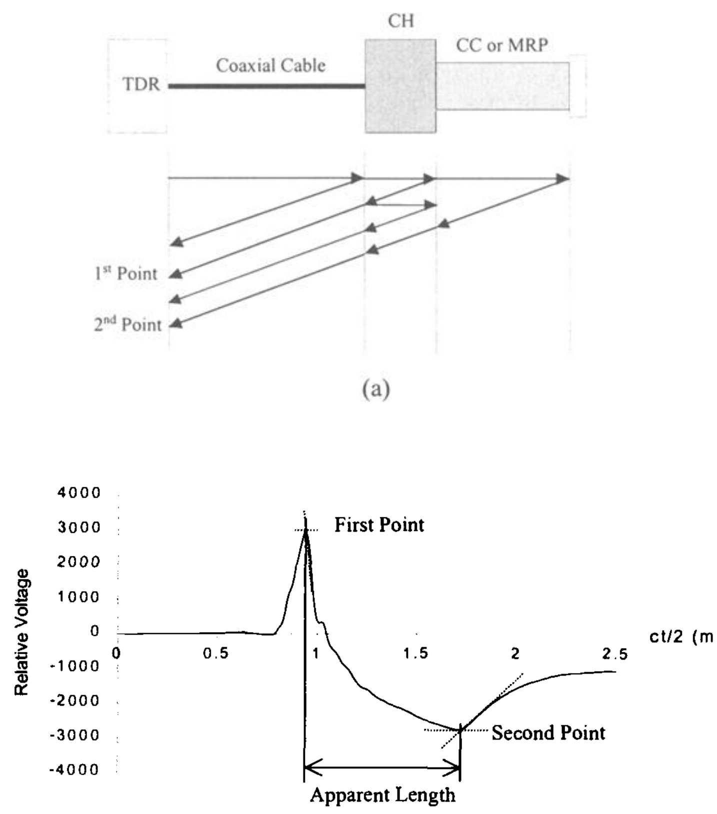
where l is the number of molecular water layers of tightly bound water, $\delta = 3 \times 10^{-10}$ m is the thickness of one molecular water layer, and S is the specific surface of the soil particle. Substituting $\alpha = 0.5$ and Eqn. 14, Eqn. 13 can be rewritten as

$$\sqrt{K_a} = \left(\frac{\sqrt{K_s} - \sqrt{K_{air}}}{\rho_s} + \left(\sqrt{K_{bw}} - \sqrt{K_{fw}}\right)l\delta S\right)\rho_d + \left(\sqrt{K_{fw}} - \sqrt{K_{air}}\right)\theta + \sqrt{K_{air}} \tag{15}$$

The theoretical mixing models are typically complex and involve several parameters not known a priori. The dielectric constants of air, water, and soil solid can be assumed invariant in practice. In terms of calibration parameters, it can be expressed as a function of soil type, density, and water content. Equation 15 becomes

$$\sqrt{K_a} = a(soil\,type)\rho_d + b\theta + c \tag{16}$$

Equations 15 and 16 serve as the theoretical basis for the empirical calibration equations. The apparent dielectric constant of soil is affected primarily by the volumetric water content and secondarily by the soil density and soil type. It is interesting to note that the empirical equations are special cases of Eqn. 15. If the soil-type effect is neglected, Eqn. 15 reduces to Eqn. 8. If soil type and density effects are both neglected, Eqn. 15 becomes Eqn. 7.



(b)

Figure 5 -- *Interpretation of waveform measured by the TDR system.*

Topp et al. (1980) concluded that the apparent dielectric constant is almost independent of the soil density from the experiments that only had 9% change in soil bulk density (ρ_d = 1.32 ~ 1.44 Mg/m^3). The laboratory work performed in Siddiqui and Drnevich (1995) indicates only marginal benefit in using one calibration equation over another. However, the experiments from which their conclusions were drawn were limited to a range in density of 1.40 to 1.65 Mg/m^3. The specific purpose of this study is to measure water content and density of soils used in highway construction for quality control purposes. Therefore, it is desirable to test actual compacted soils over the typical range of compacted densities to evaluate the calibration equations.

Experimental Method

Table 1 shows the characteristics of the soils used in the laboratory testing program. These are soils taken from actual construction sites in Indiana. TDR measurements were conducted during standard compaction and modified compaction tests. The testing device used was similar to the CC transmission line shown in Figure 4. The procedure for performing a test is as follows:

1. Prepare the soil at the desired water content and compact the soil in the compaction mold with standard compaction effort or modified compaction effort, following ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (D 698) and ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (D 1557).
2. Determine the total density of the soil in the compaction mold by measuring the mass of soil in the known volume.
3. Make a measurement using the TDR device and compute the dielectric constant.
4. Take a sample for measuring water content by oven drying ASTM Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock (D 2216).

Table 1 -- *Soil characteristic for experiments to evaluate the calibration equations.*

Soil Source	USCS	Liquid Limit	Plasticity Index	Density Range (Mg/m^3)	Water Content Range (%)
Vigo	CL	36	12	1.60-1.77	9-21
Hendricks I	CL	37	13	1.62-1.75	11-23
Indianapolis	ML	15	-	2.01-2.12	4-11
Hendricks II	CL	32	11	1.62-1.92	7-20
Bloomington	CL-CH	50	24	1.52-1.76	16-24

Results and Discussion

Topp's equation was used to compute the volumetric water content. The gravimetric water content was then obtained from the volumetric water content and the measured total density. The results were compared with the oven-dry water content, as shown in Figure 6. The standard error is 1.9% and the maximum error is 4.0%. Figure 6 illustrates that Topp's equation typically overestimates the water content of the denser soils (e.g. Indianapolis with a dry density of 2.01-2.12 Mg/m^3).

To obtain the calibration equation in the form of Eqn. 7, values of $\sqrt{K}$ were plotted with the actual volumetric water content as shown in Figure 7. A regression analysis results in:

$$\sqrt{K} = 2.13 + 7.00\theta \quad (17)$$

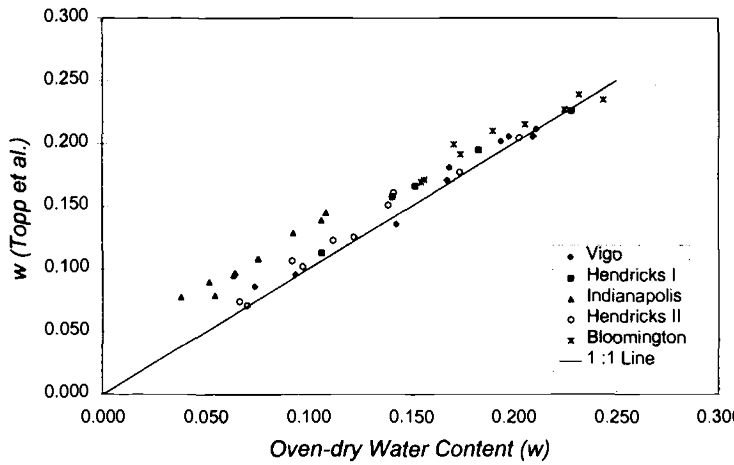


Figure 6 -- *Comparison of the oven-dry water content and the TDR measured water content for different types of soils (listed in Table 1) using Topp's empirical equation.*

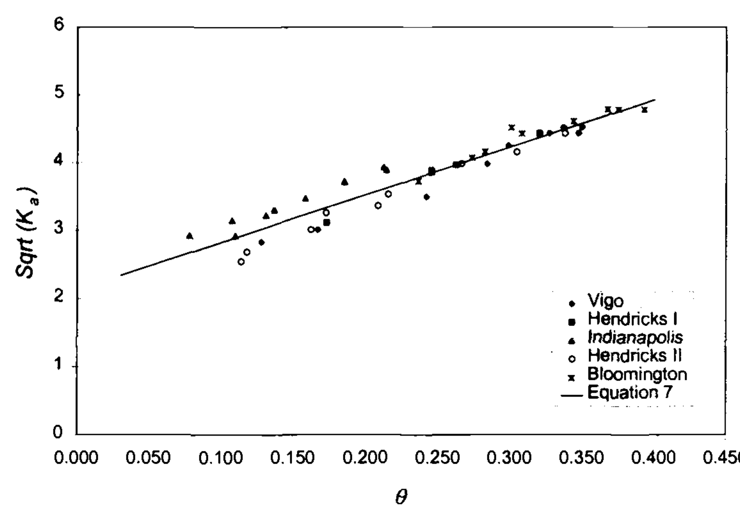


Figure 7 -- $\sqrt{K_a}$ *vs.* θ *relationship for different types of soils (listed in Table 1).*

and coefficient of determination (R^2) = 0.92. Using this equation and the measured total density, the water contents were backcalculated and compared with the oven-dry water content. The standard error of estimate is 1.5% and the maximum error is 3.2%. Equation 7 is shown to be an improvement over Topp's equation because it provides a good approximation of the theoretical models over limited ranges of soil density due to the added flexibility of the calibration constants a and b.

The density-compensating gravimetric water content equation (Eqn. 9) is plotted in Figure 8. A regression analysis of the data yields:

$$\frac{\sqrt{K}\rho_w}{\rho_d} = 1.03 + 8.24w \tag{18}$$

and R^2 = 0.98. Using this equation and the measured total density, the water contents were backcalculated and compared with oven-dry water content. For this case the standard error is 0.9% and the maximum error is 1.9%. Siddiqui and Drnevich (1995) showed only modest improvement of using Eqn. 9 over Topp's equation; however, when the range of soil densities was expanded, Topp's equation is shown to be inferior to Eqn. 9.

For each soil type, Eqn. 8 provides slightly better correlation than Eqn. 9. This result agrees with the form of calibration equation suggested by the theoretical volumetric mixing formula (Eqn. 18). However, when the data of all soils are used, Eqn. 8 resulted in almost the same R^2, standard error of estimate and maximum error as Eqn. 9. Since the parameter of soil type is unknown a priori, a calibration equation independent of soil type is desirable. For a variety of soils used in compaction as shown in Table 1, the correlation between dielectric constant and water content, normalized with respect to density, (i.e. $\sqrt{K}\rho_w/\rho_d$ vs. w) is not sensitive to the soil type. Hence, unless greater accuracy is required or highly plastic soil is encountered, Eqn. 18 can be used with approximately 1% standard error for water content measurement if the density of the soil is known.

New Method for Field Water Content and Density by TDR

Using Eqn. 9 as the calibration equation in Eqn. 10 and 11, and solving the simultaneous equations, the water content and dry density of the soil in the field can be written as

$$w_{field} = \frac{\sqrt{K_{a,mold}} - a\rho_{t,mold}/\rho_w}{b\rho_{t,mold}/\rho_w - \sqrt{K_{a,mold}}} \tag{19}$$

$$\rho_{d,field} = \frac{\sqrt{K_{a,field}}}{\sqrt{K_{a,mold}}} \times \frac{\rho_{t,mold}}{1 + w_{field}} \tag{20}$$

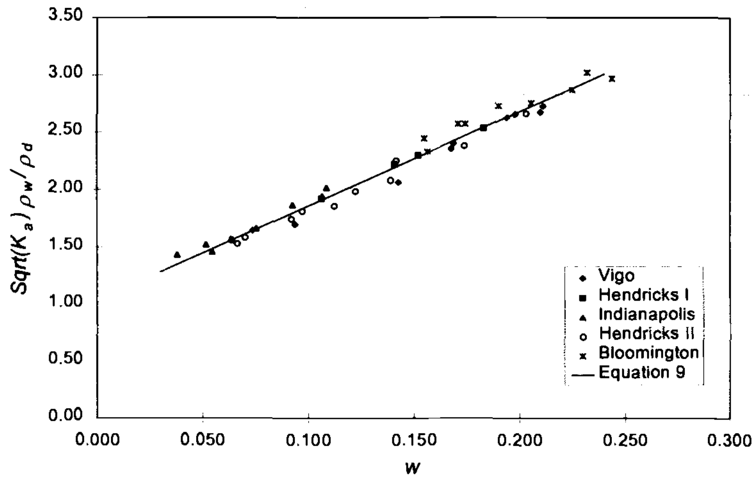


Figure 8 -- *Density-compensating water content relationship for different types of soils (listed in Table 1).*

where a = 1.03 and b = 8.24 as obtained in Eqn. 18. The calibration parameters can be determined for specific soils when necessary. The total density of the soil in the field can be measured independent of water content as

$$\rho_{t,field} = \frac{\sqrt{K_{a,field}}}{\sqrt{K_{a,mold}}}\rho_{t,mold} \tag{21}$$

An important feature of Eqns. 20 and 21 is that they are not functions of calibration parameters. These equations provide a means to measure density of soil in the field. They also can be used to obtain more accurate water content determinations because the soil density effect is taken into account.

Simulated Field and Field Evaluation of the TDR Method

The developed prototype equipment was used at several construction sites to assess accuracy of water content and density measurements under field conditions. Quality control tests in the field require determination of water content and compacted density. The sand cone test and nuclear density test are the most commonly used tests. When performing the sand cone test, the water content is determined by oven drying so that actual dry density is not known until the following day. The nuclear density gage can provide estimates of density and water content rapidly. However, the device requires calibration and uses hazardous materials. Measurements of water content using the nuclear density gage and the TDR method are compared to oven drying method, and measurements of density are compared between sand cone, TDR, and nuclear density tests where available. Much greater scatter in the data can be expected under field conditions relative to laboratory tests because of the inherent spatial variability of soils, water contents, and densities that will result under field conditions.

Simulated Field Tests

As there is no exact method of measuring in-place density to correctly assess the performance of the method developed for measuring density, it was necessary to conduct experiments in the laboratory under simulated field conditions to more accurately assess the performance of measuring in-place density. Simulated field experiments were conducted in the laboratory using the same devices as those used for the field experiments. The only difference was that the soil whose in-place density and moisture content was to be measured was compacted in a large mold. The MRP was installed in the central area of the soil in the mold. For each soil, compacting of the soil in the coaxial cylinder was done at different densities to cover a wide range of densities (loose to dense). The above procedure was repeated for soils prepared at other moisture contents to cover a wide range of moisture content.

The soil used for the simulated field experiment was a Crosby till, which has similar soil characteristic as the Hendricks II in Table 1. The soil-specific calibration parameters were used to calculate water content using Eqn. 19. The gravimetric moisture contents measured by TDR are compared with the oven dry water content in Fig. 9. The standard error and maximum error are 0.4%, and 0.7%, respectively. Fig. 10 shows the comparison between the measured total density and the actual total density and the agreement is good. The standard error of measurement is only 0.020 Mg/m^3. These results validate the procedure and equation developed for measuring density and moisture content.

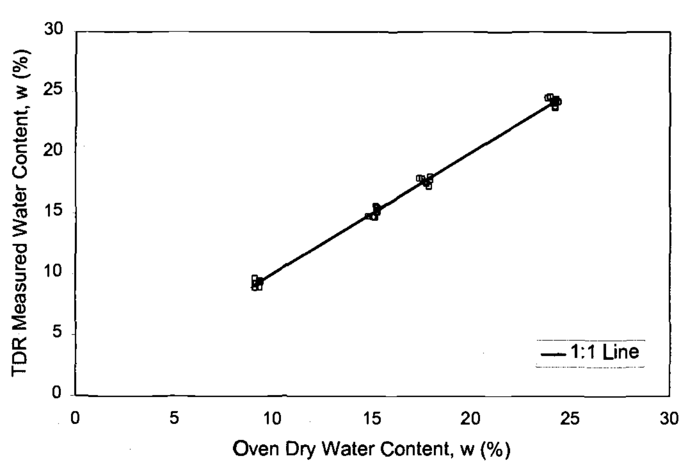


Fig. 9 – TDR water content compared to oven dry water content from simulated field tests.

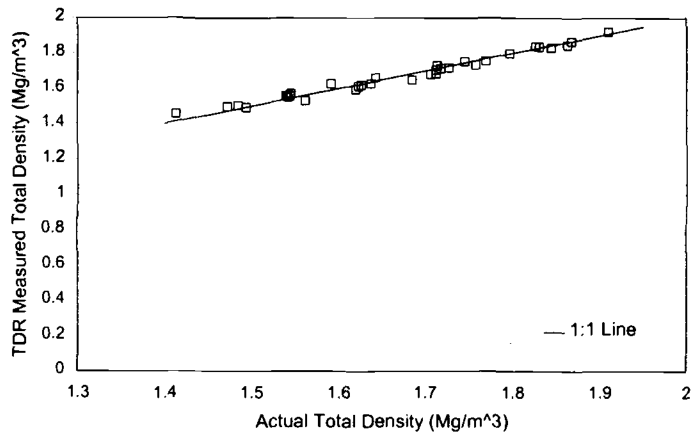


Figure 10 – TDR measured total density compared to actual total density in simulated field tests.

Field Tests

Forty-four quality control tests were performed at 11 construction sites. Table 2 provides the characteristics of the soil at these sites. Some sites have TDR tests and nuclear tests or sand cone tests, while other sites have all three types of tests.

Table 2 -- *Soil characteristics for the field tests.*

Soil Source	Soil Type	P#4 %	P#10 %	P#40 %	P#200 %	Liquid Limit	Plasticity Index
Crawfordsville	SW	64.9	44.2	15.9	0.3	22	9
Anderson I	SC	98.3	88.6	51.6	18.3	29	12
Decatur	SW-SC	93.6	72.1	29.1	5.8	39	20
Butler	SP-SC	99.2	98.8	65.6	5.2	34	16
Mt. Vernon	SM	90.5	87.1	74.1	39.8	27	4
Knox	SC	99.9	92.1	49.5	21.6	30	11
Anderson II	SP-SC	96.0	89.3	59.5	6.3	23	9
I-52 I	SW	88.9	76.0	50.9	10.0	--	--
I-52 II	GW	69.9	52.0	12.1	0.6	--	--
I-52 III	SP	96.9	85.8	61.9	10.4	--	--
Gas City	SC	97.5	94.2	58.7	12.5	31	13

At each test location the TDR, nuclear and/or sand cone tests were done as close as practical to each other (≈ 8 in.). The soil excavated for compaction in the coaxial cylinder was taken from within the boundary formed by the three outer spikes of the multiple-rod probe and uniformly from the entire depth penetrated by the spikes. The soils were placed directly in the coaxial cylinder to minimize moisture loss and were compacted uniformly using either moderate hand compaction or standard compaction effort. The excavated soil samples from the TDR tests and sand cone tests were brought back to the lab to determine the oven-dry water content. Equations 19 and 20 were used to calculate water content and density using the TDR method.

The water content measurements, obtained from the TDR and nuclear methods (if available) were compared with oven-dried values and are shown in Figure 11. The measurements indicate that the water contents obtained by the TDR method are more accurate than the nuclear density test. The standard error and maximum error for water content measured by TDR were 1.1% and 2.8% respectively, and were 1.8% and 4.4% for the nuclear method.

For a comparison of densities, twenty tests were performed with both the TDR and nuclear gage (direct transmission method with 6 in. depth) and eight of these tests also with sand cone. Figure 12 gives the comparison of total densities by sand-cone and nuclear methods to total densities by the TDR method. The results show significant variability in estimates obtained from the three methods. For the eight measurements where all three methods were used, the standard error and maximum error for dry density measured by TDR were 0.097 Mg/m^3 (6.1 pcf) and 0.140 Mg/m^3 (8.9 pcf), respectively, when compared to the sand cone test. The density obtained by the nuclear method compared more favorably to the sand cone test than did the TDR densities, yielding standard error of estimate of 0.066 Mg/m^3 (4.2 pcf) and maximum error of 0.138 Mg/m^3 (8.7 pcf).

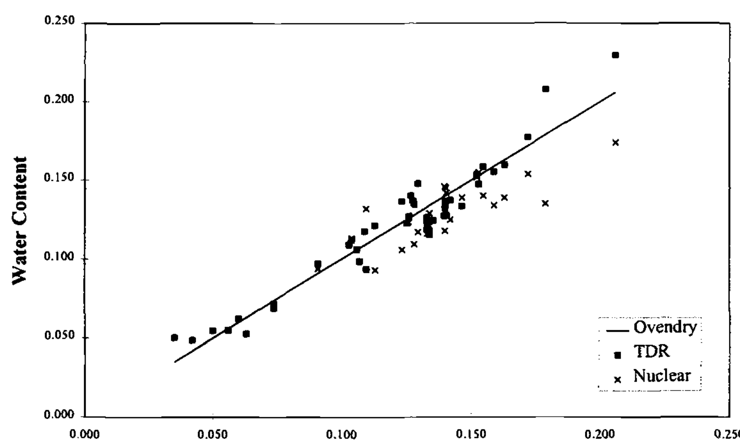


Figure 11 -- *Comparison of TDR water content and nuclear water content with oven-dry water content.*

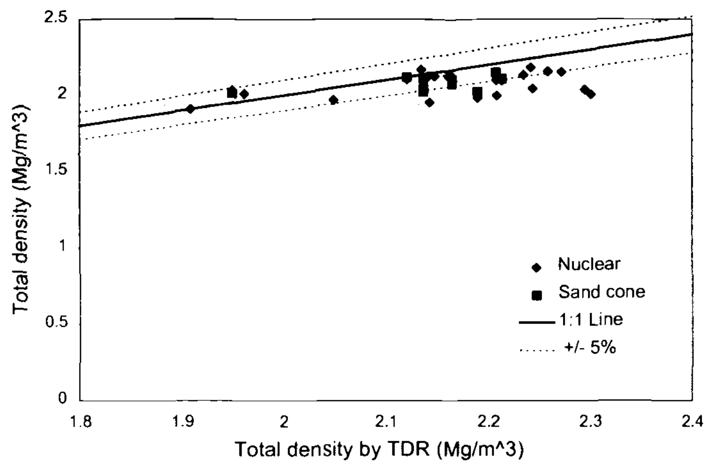


Figure 12 – *Comparison of sand-cone total density and nuclear total density with TDR measured total density.*

The ability to assess the accuracy of the TDR method for determining density was limited because the true density of the compacted soil is not known. The difference in density that exists among the three methods is attributed partly to the spatial variability of soil, water content, and density at the construction sites. For example, the nuclear gage showed a difference in density 0.04 Mg/m^3 (2.2%) at a distance of only 20 cm (8 in.) apart at the Knox site.

Conclusions

This paper evaluated and extended the methodology developed by Siddiqui and Drnevich (1995) for measuring soil water content and density using Time Domain Reflectometry (TDR). Prototype equipment was developed to be robust and easy to use for routine quality control testing of compacted soils. Calibration equations describing the relationship between the apparent dielectric constant of soils measured by TDR and soil water content were examined and compared by theoretical dielectric models and laboratory experiments. The density-compensating water content relationship (Eqn. 9) provided the best calibration and was adopted in the procedure of determining water content and density of compacted soil. The results of the TDR procedure were compared with conventional methods on actual construction sites in Indiana.

Laboratory tests were conducted over a broad range of soil types and densities. It was shown that the use of Topp's equation to define the relationship between volumetric water content and dielectric constant does not have sufficient accuracy because it does not account for variations in soil density. Accordingly, the calibration equation of the form proposed by Siddiqui and Drnevich (1995) is used. This calibration equation is not sensitive to the types of soils commonly used for fills, at least for the varieties tested in this program.

The time required to perform the TDR test under field conditions is approximately 15 minutes. The sand cone test can also be performed in approximately 15 minutes while the nuclear density test can be performed in approximately 5 minutes. In general, the nuclear gage provides estimates of density and water content more rapidly than the sand cone or TDR tests, is less accurate than the TDR test in estimating water content, and must be calibrated for a specific soil using sand-cone tests or other calibration techniques. Moreover, the nuclear gage uses a hazardous source requiring operators to take safety training and leading to expenses associated with operator licensing and equipment transportation, storage, maintenance, and disposal. The sand-cone test is time consuming, and cannot provide estimates of water content. The TDR test is also time consuming, provides accurate estimates of water content. The errors associated with estimated dry density, when compared to the sand-cone test, are slightly larger than the nuclear density test for the field sites in this research.

While the results of this study are quite encouraging, the TDR test takes more effort than the nuclear method for the measurement of water content and density. One especially attractive feature of the TDR method is that an on-site moisture-density curve can be obtained in less than an hour using the TDR method with the coaxial cylinder.

The present TDR test is based solely on the measurement of the apparent dielectric constant. It is the single parameter that is obtained from the travel time analysis of reflected waveform. However, progress is being made in the development of an approach to use other features of the reflected waveform, in addition to the apparent dielectric constant, in the interpretation process. Toward the latter part of this study it became obvious when comparing curves of different soil types and at different densities that the reflected waveforms contained much more information than is currently being utilized. A new study has been initiated to use the entire waveform to estimate water content, density, and soil type. The primary objective is to increase the accuracy of the density estimate while eliminating the need to excavate and compact the soil in the coaxial cylinder, thereby reducing the test time substantially.

Acknowledgments

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Soheil Nazarian[1] and Deren Yuan[2]

Quality Control of Compacted Layers with Field and Laboratory Seismic Testing Devices

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References: Nazarian, S., and Yuan, D., **"Quality Control of Compacted Layers with Field and Laboratory Seismic Testing Devices,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384*, D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: A procedure to measure the modulus and Poisson's ratio of each compacted layer shortly after placement is presented. Seismic technology has been used for this purpose. Simplified field and laboratory tests are suggested that can be performed and interpreted rapidly and nondestructively so that any problems during the construction process can be adjusted. The field and laboratory methods are incorporated in a manner that their results can be readily reconciled without any scaling or simplifying assumptions. Therefore, the laboratory tests can be used to develop the ranges of acceptable properties for a given material. Nondestructive field tests are performed to determine whether the contractor has achieved these levels. In this paper, the overall procedure and the field and laboratory devices are introduced, a procedure to establish the accuracy and repeatability of the methods is described, and several case studies are included to exhibit the preliminary uses of the procedure.

Keywords: quality assurance, modulus, field testing, laboratory testing, compacted fills

The primary material parameters that affect the performance of a constructed fill have been the shear strength, modulus and Poisson's ratio. Unfortunately, the acceptance criteria are typically based on the adequate density of the compacted materials. The primary goal of this paper is to provide a concept on using seismic methods which, in a rational manner, combines the results from laboratory and field tests with those used for quality control during construction. The devices and procedures introduced measure seismic wave velocities. Seismic wave velocities can be easily transformed to moduli using fundamentally correct relationships. Performing the simplified laboratory and field tests on fills will allow us to develop a database that can be used to smoothly unify the design procedures and construction

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quality control. Field procedures to measure the modulus and Poisson's ratio of each layer shortly after placement are also presented.

Several case studies are included to show some results that can possibly be obtained with some of the devices. These studies also demonstrate the issues that have yet to be resolved.

Overview of Methods

The modulus and Poisson's ratio of a material can be determined either with field testing or with laboratory testing. For a more sophisticated analysis, the behavior of the material in terms of variation in stiffness with the state of stress should be determined. This behavior is typically established by conducting time-consuming laboratory tests. Simplified laboratory tests can be used in conjunction with the more sophisticated ones during the design process. By combining the results from simplified and more comprehensive tests, one can either ensure compatibility or develop correlations that can readily be used in the field.

Based on the background information provided, the goal is to develop modulus-based tests that can be readily used in the field for quality control of any layer in a compacted earth fill. In the next section, the procedures necessary for this task are introduced. As in any other quality management program, acceptance criteria should be developed. The proposed acceptance criteria are based on seismic testing of specimens prepared in the laboratory. The specimens used for this purpose are similar to those used in determining the optimum moisture content and maximum dry density.

Seismic Field Tests

The Young's and shear moduli of a certain layer are measured nondestructively by generating and detecting the arrivals of compression, shear or surface waves. The historical development as well as the theoretical and experimental background behind these tests can be found in Baker et al. (1995).

The setup is shown in Figure 1. Typically, a seismic source and at least two receivers are needed. The surface of the medium is impacted, and the transmitted waves are monitored with the receivers. The reduction of data can be

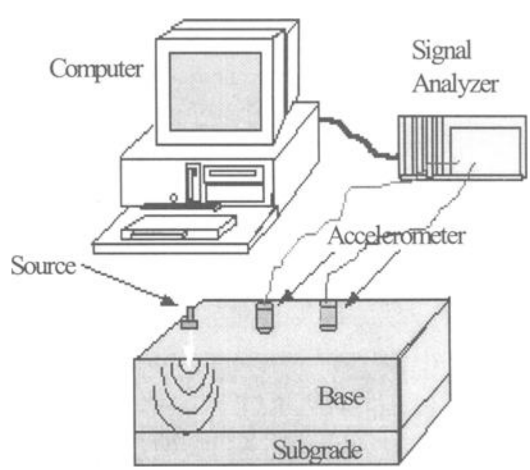

Figure 1 - *Schematic of Field Seismic Testing*

performed either in the time- domain or in the frequency-domain. These processes are described below.

Time-Domain Data Reduction - In the time-domain analysis, one relies on identifying the time at which different types of energy arrive at each sensor. The velocity of propagation, V, is determined typically by dividing the distance between two receivers, ΔX, by the difference in the arrival time of a specific wave, Δt. In general, the relationship can be written in the following form:

$$V = \frac{\Delta X}{\Delta t} \tag{1}$$

In the equation, V can be the propagation velocity of any of the three waves [i.e., compression wave, V_P; shear wave, V_S; or surface (Rayleigh) wave, V_R]. Knowing the wave velocity, the modulus can be determined in several ways. Young's modulus, E, can be determined from shear modulus, G, through Poisson's ratio (ν) using:

$$E \quad 2\,(1 + \nu)\, G \tag{2}$$

Shear modulus can be determined from the shear wave velocity, $V_{S,}$ using:

$$G = \rho\, V_S^2 \tag{3}$$

where ρ is the mass density. To obtain the modulus from the surface wave velocity, V_R is converted first to shear wave velocity using:

$$V_S = V_R\,(1.13 - 0.16\nu) \tag{4}$$

The shear modulus is determined then by using Equation 3.

Typical records from two sensors are shown in Figure 2 for a fill material. As an example, the arrivals of compression, shear and surface waves are marked on the figure. The compression wave (or P-wave) energy is reasonably easy to identify because it is the earliest source of energy to appear in

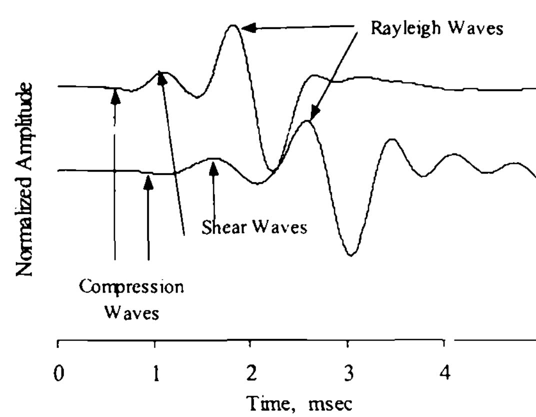

Figure 2 - *Typical Time Records*

the time record. Since less than 10% of the seismic energy propagates in this form, the peak compression wave energy in the signal is sometimes only several times above the inherent background noise. This limitation may make it difficult to consistently estimate the arrivals of these waves.

The shear wave (or S-wave) energy is about one-fourth of the seismic energy and, as such, is better pronounced in the record. The practical problem with identifying this type of wave is that it propagates at a speed that is close to that of a surface wave. As such, the separation of these two energies, at least for short distances from the source, may be difficult.

Surface (Rayleigh) waves contain about two-thirds of the seismic energy. As marked in Figure 2, the most dominant arrivals are related to the surface waves; as such, it should be easy to measure them. If a layer does not have surface imperfections and if the impact is "sharp" enough to generate only waves that contain energy for wavelengths shorter than the thickness of the top layer, this method can be used readily to determine the modulus. However, it may be difficult to meet these two restrictions. The frequency-domain analysis, even though more complex to implement, is by far more robust than the time-domain analysis.

Frequency-Domain Data Reduction - Since most of the energy in a seismic wave train is carried by surface waves, one can take advantage of the signal processing and spectral analysis to develop a more robust methodology for determining the modulus. This method is called the Spectral-Analysis-of-Surface-Waves (SASW; see Nazarian et al. 1995).

The goal with the SASW method is to generate and detect surface waves over a wide range of wavelengths. The time records collected with the setup described above are transformed to a so-called dispersion curve — a plot of velocity of propagation of surface waves versus wavelength. If the only goal is to determine the modulus of the top layer, the method becomes straight forward.

Consider the time records shown in Figure 2. By performing a fast Fourier transform on the two signals and then dividing the two transformed signals by one another, one obtains a phase spectrum (i.e. variation in phase with frequency). Such phases are shown in Figure 3. The phase shown in Figure 3 can be "unwrapped" and fitted by a straight line. As indicated by Baker et al. (1995), the slope of such a line, m, is directly related to Young's modulus, E, using

$$E = 2\ \rho\ (1 + \nu)\ [(1.13 - 0.16\nu)\ \frac{360\Delta X}{m}]^2 \tag{6}$$

where ν is Poisson's ratio and ΔX is the sensor spacing. As before, ρ is the mass density. Alternatively, one can construct a dispersion curve, as shown in Figure 3, and determine the average modulus of the top layer. In that case, the modulus is obtained from:

$$E = 2\ \rho\ (1 + \nu)\ [(1.13 - 0.16\nu)\ V_{ph}]^2 \tag{7}$$

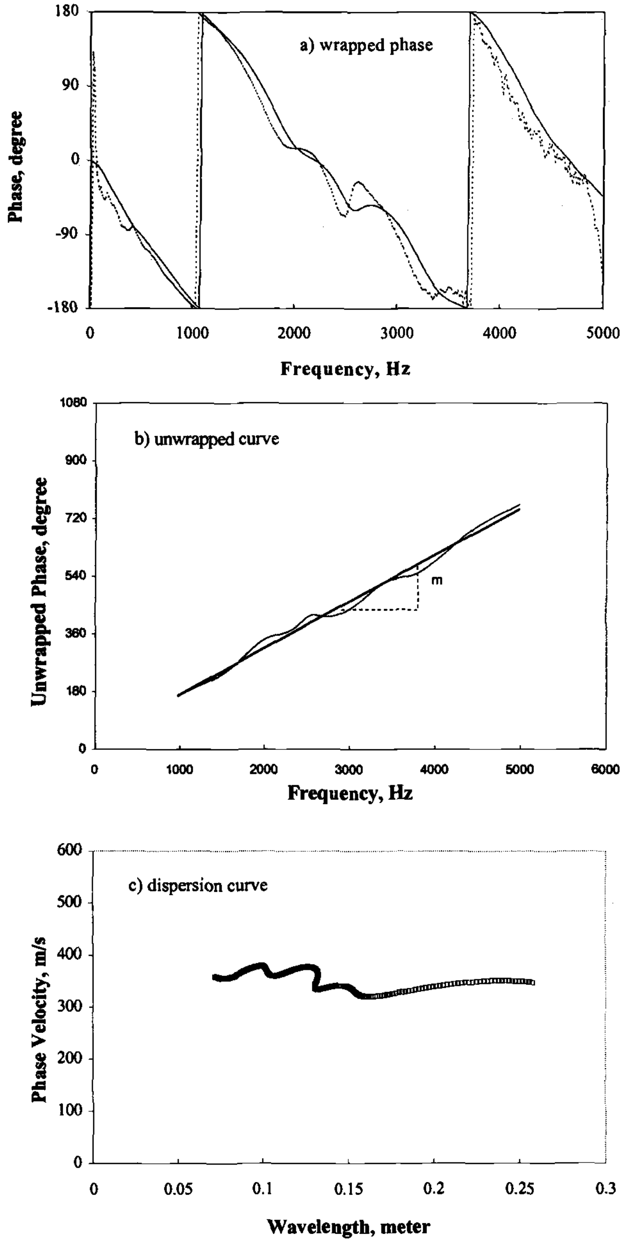

Figure 3 - *Typical Results from Frequency-Domain Analyses*

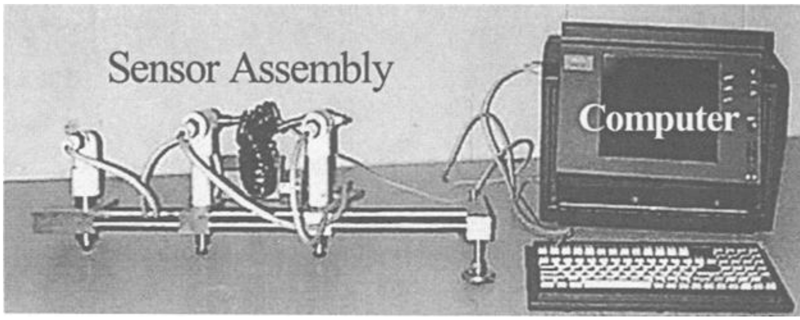

Figure 4 - *Automated Device for Field Testing*

where V_{ph} is the average phase velocity of the top layer. Baker et al. (1995) have developed a device that can perform this test in the field in less than 1 minute per point. The device is shown in Figure 4.

If the shear and compression wave velocities are known, Poisson's ratio, ν, can be readily determined using:

$$\nu = \frac{0.5\ \alpha^2\ -1}{\alpha^2 - 1} \tag{8}$$

where $\alpha = V_P / V_S$. (V_S and V_P are shear and compression wave velocities, respectively).

Seismic Laboratory Tests

One of the major goals of the project is to develop field tests that are compatible with laboratory results. As indicated before, the existing tests used to determine the modulus of geo-materials in the laboratory are cumbersome and time-consuming. Simplified laboratory tests can be used in conjunction with the more sophisticated ones during the design process.

A schematic of the test setup for the Free-Free Resonance test is shown in Figure 5. The specimen is either suspended by two wires or placed on a material which is substantially less stiff than the specimen (e.g., Styrofoam). An accelerometer is placed securely on one end of the specimen, and the other end is impacted with a hammer instrumented with a load cell. The signals from the accelerometer and load cell are used to determine the resonant frequency, f, as shown in Figure 6. Once the frequency, f, the mass density, ρ, and the length of the specimen, L, are known, Young's modulus, E, can be found from

$$E = \rho\ (2\ f\ L)^2. \tag{9}$$

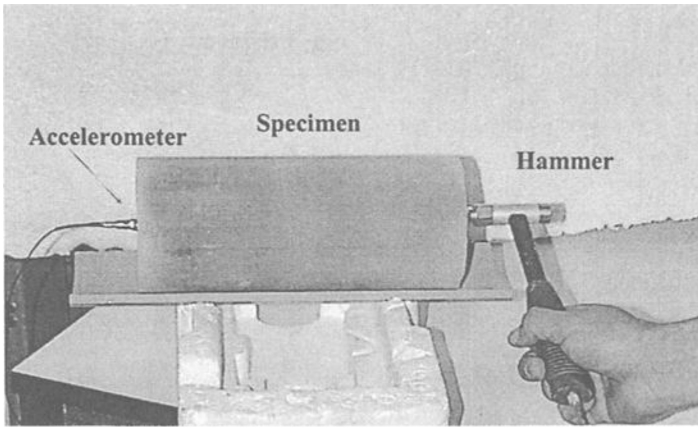

Figure 5 - *Picture of Free-Free Resonance Test*

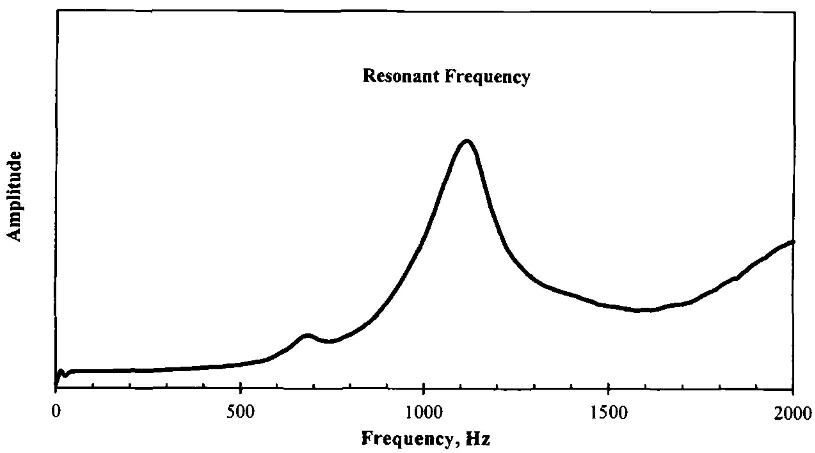

Figure 6 - *Typical Spectral Function from Free-Free Resonance Test*

Alternatively, the accelerometer can be placed in the radial direction, and the specimen can be impacted in the radial direction to determine the shear modulus (see Equation 4). Once again, the shear and Young's moduli can be combined to calculate Poisson's ratio.

In general, the method is quite repeatable and is nondestructive. Therefore, the conventional specimens to be tested can be used before they are placed in the loading frame. In less than 3 minutes, a specimen can be tested, and the test result can be obtained.

Case Studies

Several field and laboratory studies were carried out to determine the initial feasibility of the suggested tests. The results are summarized here.

A series of tests was carried out at a site near Horizon, Texas to determine the variation in modulus of a base and subgrade with the seismic method. Besides seismic tests, a dynamic cone penetration (DCP) device and a conventional nuclear density gauge were used. The granular base at the site was about 200 mm thick.

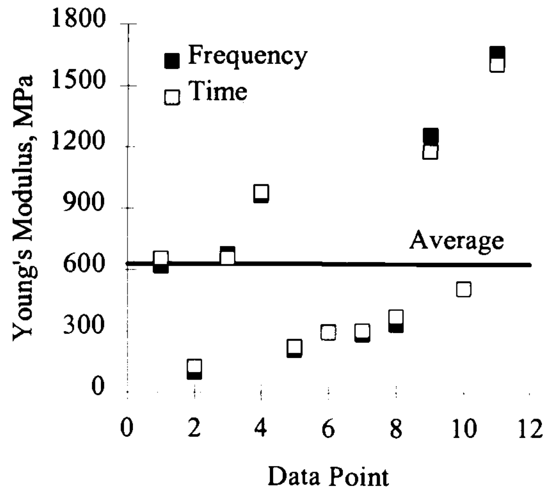

Figure 7 - *Variation in Modulus along a Section of Subgrade in Horizon*

The variation in seismic modulus with location for the prepared subgrade is shown in Figure 7. A total of eleven points each about 2 m apart were tested. The results from the time-domain and frequency-domain analyses are fairly close. This occurs because the subgrade material was compacted well and did not contain large gravel. If a material does not contain fine cracks and surface imperfections, the time-domain and frequency-domain analyses normally yield similar results.

The average moduli from the two methods are essentially the same, about 630 MPa. However, the moduli at most points are much less than the average value. The coefficient of variation is approximately 70%, indicating a large variability in the moduli. Such a large variability in modulus can be attributed to either a lack of precision of the method, the actual material variability, or both.

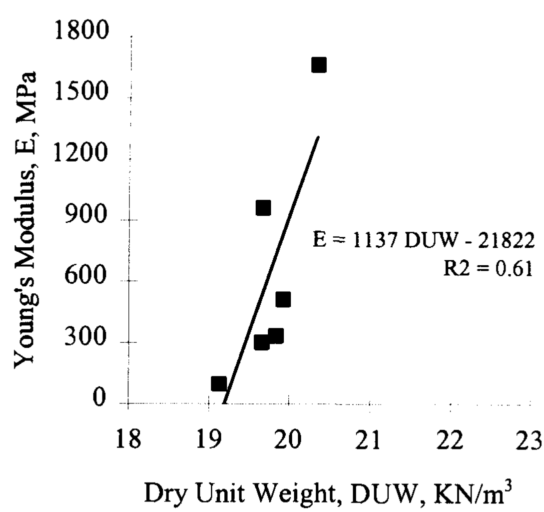

Figure 8 - *Variation in Seismic Modulus with Dry Unit Weight*

A laboratory study was carried out to determine the repeatability of the seismic method. Six boxes, 1 m x 0.6 m, were filled with the material used at the site. The density and the moisture content were controlled precisely to be very close to the optimum values. A 200-mm-thick layer of the material was placed in each box, and four seismic tests were carried out on top of each material. The results from these tests showed that seismic tests are rather precise and repeatable at a level of about better than 7%. Therefore, the variation in modulus should be related to the variation in material properties.

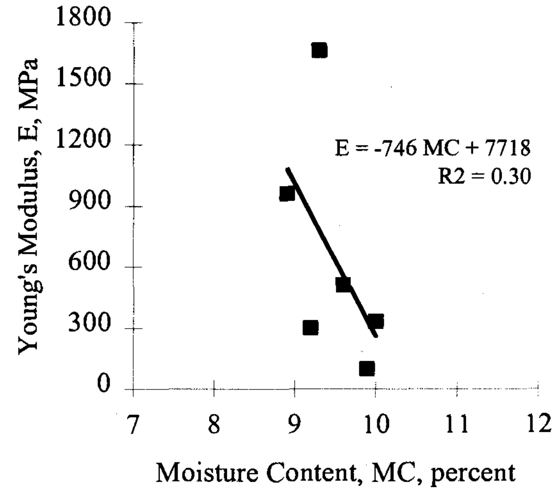

Figure 9 - *Variation in Seismic Modulus with Moisture*

In Figure 8, the variation of in-place dry density measured with a nuclear density gauge at six of the data points is related to seismic modulus. All points could not be tested because the nuclear device was needed for an ongoing project. A significant drop in modulus is associated with a small variation in dry density. Similarly, the variation in modulus with moisture content is shown in Figure 9. Once again, a mild correlation between the modulus and moisture content exists. This type of relationship was pursued further in the laboratory environment, as will be discussed later.

The variation in Poisson's ratio with location, as shown in Figure 10, varies between 0.34 and 0.43. The average Poisson's ratio is about 0.39. Typically, the lower moduli coincide with higher Poisson's ratios (compare Figures 7 and 10). The higher Poisson's ratios are usually related to wetter subgrades.

Similar tests were performed on the base material about 1000 m away from the subgrade site. A comparison of

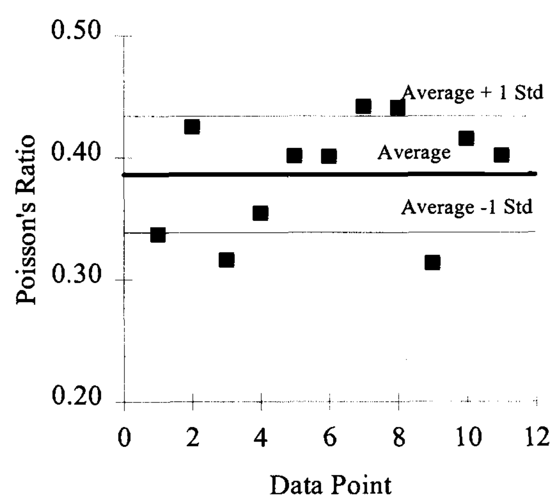

Figure 10 - *Variation in Poisson's Ratio*

different moduli related to this base is shown in Figure 11. The average seismic moduli is approximately 840 MPa with a coefficient of variation of about 23 percent. The resilient modulus of the base was about 470 MPa, which is almost 80 percent less than the seismic modulus. Using the free-free resonance test, the seismic modulus measured from laboratory specimens prepared to the average density and moisture of the base are about 745 MPa. However, when the specimens were prepared near the optimum water content as per Proctor test, the seismic modulus was about 227 MPa, corresponding to the average value along the thickness of the base.

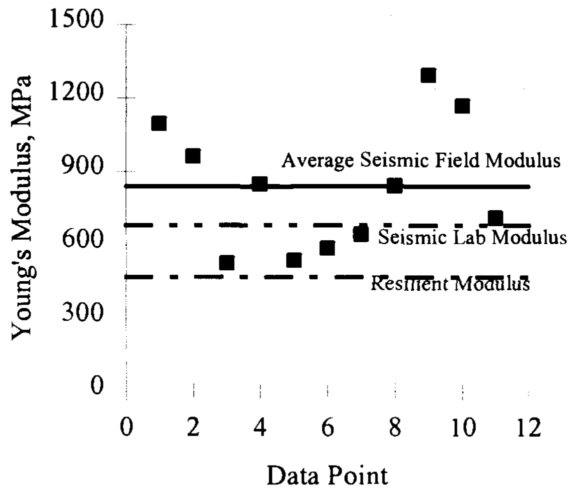

Figure 11 - *Comparison of Base Moduli from Different Methods*

Some scatter and a mild correlation in the results can be observed. Since a DCP test is rather localized, some anomalies such as larger gravel can influence the results. The best-fit line depicted in Figure 12 corresponds to a ratio of about 2200 between the modulus and CBR, which is higher than the ratio of 1500 suggested by AASHTO. However, it lies within the range of 750 and 3000 defined as reasonable in the AASHTO design.

From this case study, we learned that trends exist between field moisture content, density, CBR and seismic modulus. We also learned that specimens prepared as per Proctor procedure may yield moduli that are less than those measured in the field. However, if the laboratory specimens are prepared at the density and moisture level

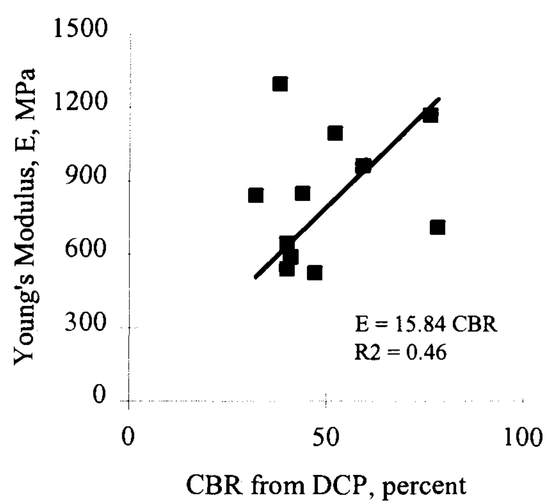

Figure 12 - *Variation in Seismic Modulus with CBR from DCP*

measured in the field, closer relationships between laboratory and seismic moduli can be developed.

One of the desirable aspects of a QA/QC program is to determine the variation in moduli with moisture content under a constant compactive effort, similar to the moisture-density curve determined with the Proctor method. The results from such a study are shown in Figure 13. The Proctor optimum moisture content of the material is about 6.5%. As such, three sets of specimens were prepared at nominal moisture contents of 5.5%, 6.5% and 7.5%. Inspecting the trends, a fourth specimen was also prepared at a moisture content of about 6%. From Figure 13, the maximum seismic modulus occurs at a moisture content closer to 6%. From this preliminary study, it seems that the maximum modulus may occur at a lower water content than the optimum water content determined by the Proctor method. This result is in concurrence with Stokoe's (1998) finding that "over-compaction" may result in a reduction in modulus.

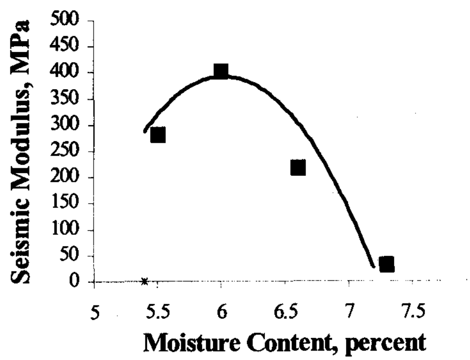

Figure 13 - *Variation in Modulus with Moisture During Compaction*

After the compaction of a layer is completed, it may be exposed to environmental factors that may impact its behavior. One of the major concerns with most fill materials is its water retention potential and the impact of the change in moisture content on the strength and stiffness parameter of a layer. To address these issues, we have adapted a test that will potentially allow the engineer to quantify these issues. To perform a test, a specimen (150-mm by 300-mm for coarse-grained materials or 100 mm by 200 mm for fine-grained material) is prepared. A PVC concrete mold is retrofitted within a compaction mold for this purpose. Several small holes are drilled into the bottom of the mold so that the specimen can quite readily access water. The specimen is prepared at the optimum moisture content as per Proctor method. The prepared specimen is then placed in an oven normally used for moisture content specimens and dried until all the water is removed. Since the test is nondestructive, the same specimen can be used over and over. When the water content is equal to zero, the specimen is weighed and placed in a pan filled with water. The free-free resonance test is performed on it daily. The gain in weight of the specimen and the change in modulus with time is then monitored until the water content is close to the optimum moisture content. By inspecting the change in modulus with moisture content, the behavior of the material can be judged.

The variation in modulus with moisture from one base material is shown in Figure 14. As observed, a significant difference (about an order of magnitude) in modulus can be detected. In the Southwest, the drying cycle can be associated with the change in the properties of the exposed soil during hot summer days after the completion of compaction. The soaking cycle can be related to occasional rain storms experienced in

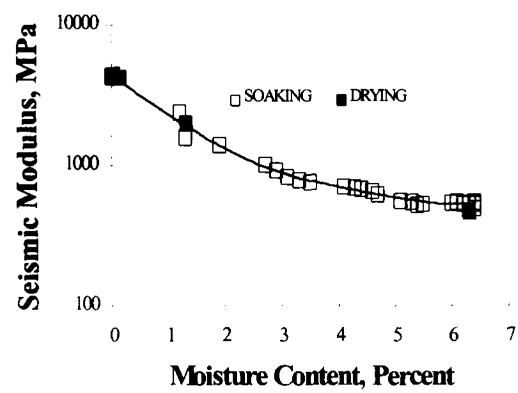

Figure 14 - *Variation in Modulus with Moisture During Soak Test*

the area. One known weakness of the method is that above the optimum moisture content, the absorption of water becomes quite difficult since the suction forces in the soil matrix become small. Even though the daily laboratory tests by themselves are not time-consuming (about 5 minutes), the long length of time that is required to perform these tests on one material (typically about 2 weeks) may be of concern. These two parameters are under consideration at this time.

To demonstrate the repeatability of the laboratory tests, two similar specimens were prepared from a subgrade. The reported optimum moisture content of the material was 12%. The average moisture contents of the two specimens at the start of testing were slightly lower than 12%, as shown in Figure 15. To expedite the tests, the specimens were not dried but only soaked. Based on the results in Figure 15, the two specimens follow the same trends. The moduli drop approximately by a factor of three as the moisture content increases to about 13%. Based on our experience, and a quick comparison of the rate of reduction in modulus with moisture in Figures 14 and 15, as the clay content of the soil increases, the modulus due to the absorption of water will decrease. In Figure 15, a systematic difference of about 15% between the moduli of the two specimens is observed. This difference, at this time, is attributed to experimental errors and to a lack of consistency in the preparation of the specimens.

Summary and Conclusions

A procedure to measure the modulus and Poisson's ratio of compacted fill shortly after placement is presented. Seismic technology has been used for this purpose. Simplified field and laboratory tests are suggested that can be rapidly and nondestructively performed and interpreted so that any problems during the construction process can be adjusted. The field and laboratory methods are incorporated in a manner that their results can be readily reconciled without any scaling or simplifying assumptions. Therefore, the simplified laboratory tests can be used to develop the ranges of acceptable properties for a given material. Nondestructive field tests are performed to determine whether the contractor has achieved these levels.

The laboratory test suggested is the free-free resonance test. This test can be performed in less than three minutes and is inexpensive, and its data reduction process is simple and almost instantaneous. The field seismic nondestructive tests are performed with an automated device that can collect data and analyze the results in less than 60 seconds per point.

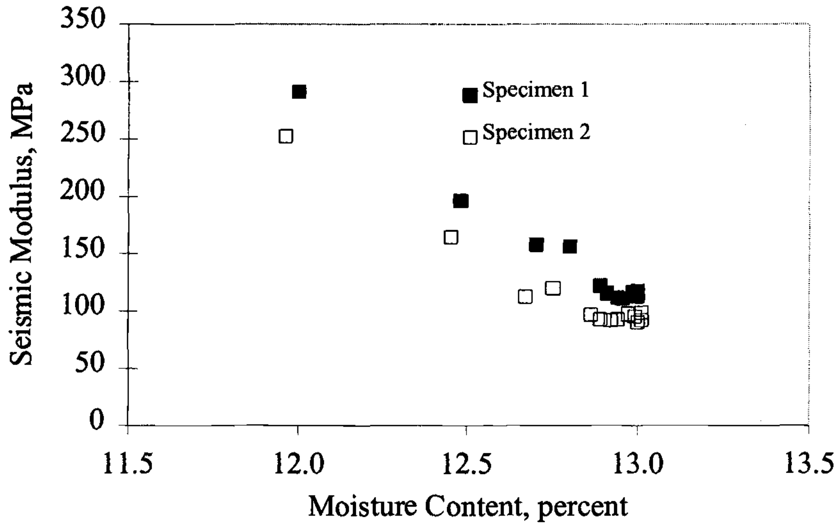

Figure 15 - *Variation in Modulus with Moisture Content for a Clayey Subgrade*

In this paper, the overall procedure is described, the field and laboratory devices are introduced, a procedure to establish the accuracy and repeatability of the methods is described, and several case studies are included to exhibit the preliminary uses of the procedure.

Based on this study, the following preliminary conclusions can be drawn:

- Seismic testing, both in the laboratory and in the field, is rapid and repeatable.
- Seismic modulus is sensitive to variations in dry density of the fill material.
- There is a good agreement between the seismic moduli measured in the field and in the laboratory as long as the laboratory specimens are prepared at the density and moisture content of the field materials (not Proctor or modified Proctor density and moisture content).
- A large variability in the fill modulus was observed with small changes in moisture content. Through laboratory experiments, such variability was attributed mostly to regional environmental conditions.

Acknowledgment

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Compaction and Performance of Loess Embankments

Reference: Zhang, L., Du, J., and Hu, T., **"Compaction and Performance of Loess Embankments,"** *Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384*, D. W. Shanklin, K. R. Rademacher, and J. R. Talbot, Eds., American Society for Testing and Materials, West Conshohocken, PA, 2000.

Abstract: This paper summarizes the practice of highway embankment compaction in the loess plateau of northwestern China, based on a field trip and the related laboratory studies. A large number of high loess embankments were built across gullies. The compaction was based on the standard Proctor method (ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort, D698-91) during 1950 - 1985, and the modified Proctor method (ASTM Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort, D1557-91) after 1985. The performance of these embankments is described. Stability analysis and centrifuge tests are conducted to confirm the observations and improve designs. Storm water ponds are found to be critical to both stability and settlement. For embankments compacted using the standard Proctor method, progressive failure would start with any further erosion if the slopes were steeper than 1:0.75.

Keywords: embankment, compaction, stability analysis, settlement, loess, erosion

Review of Compaction Practices

This paper concerns construction of highway embankments in the eastern Gansu and northern Shannxi area, which is at the center of the "Loess Plateau" in China. This area has semi-arid climate; the annual precipitation is only 250 – 600 mm. The loess thickness varies from a few meters to 50 meters. The soils found in the west of the area are largely Q_3 and Q_4 quaternary deposits and those in the east are Q_2 and Q_3 deposits. Silt (d = 0.074 - 0.002 mm) comprises more than 60% and clay (d < 0.002 mm) comprises about 20% of the soil. The natural dry density, void ratio, and natural moisture content are in the range of 1140 - 1600 kg/m^3, 0.78 - 1.50, and 7.0% - 23.0%, respectively, and the

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liquid and plastic limits are 21.7% - 32.5% and 14% - 21%, respectively. The friction angle and apparent cohesion from the consolidated-undrained tests are 23° - 39° and 10 - 65 kPa, respectively.

Pre-1950

Although a lot of loess embankments were built before 1950, few of them survived the harsh erosion and wars. The compaction of ancient embankments was done manually, typically using hand hammers or big stone rams of 50 – 100 kg operated by several people. The rule-of-thumb for moisture control was "being a lump when it is held in hand but breaks apart when fallen to the ground" (Ninxian County Government 1992). Figure 1 shows the remainder of an ancient embankment in Xiantou. It was built before 1900 and major renovations were made in 1918, 1955, and 1982 ~ 1984. The present embankment serves a major highway. It is 54 m high, 180 m long, and 11 m wide at the crest. The ancient fill is exposed to the sun. The fill materials are moderately cemented so that the nearly vertical slope stands approximately 40 m high.

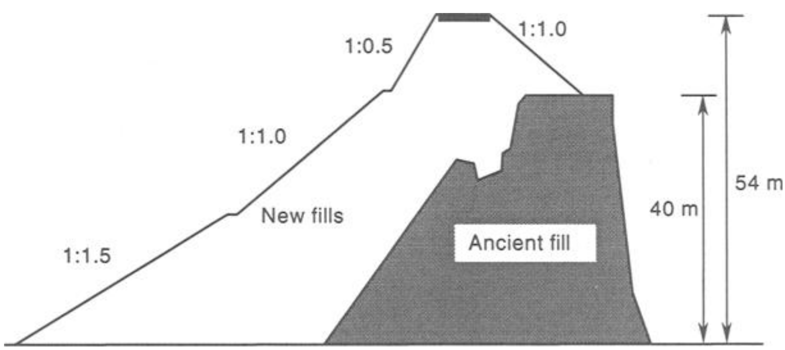

Figure 1 - *Xiantou Embankment Embodying Ancient Fills*

1950 - 1985

The period between 1950 to 1985 witnessed construction of thousands of embankments for unclassified roadways. Compactions based on the standard Procter compaction (ASTM Test Method D698-91) were enforced for most embankments. Only clean and inorganic loess soils were selected as fill materials. The contents of clay and coarse sand were limited to less than 25% and 20%, respectively. Such fill materials had a plastic index in the range of 10 - 14%. The maximum dry density and optimum moisture content were 1630 ~ 1700 kg/m^3 and 17 ~ 20%, respectively. The construction moisture content was controlled within 2% around the optimum depending on the natural moisture content, and the relative compaction was required to be no less than 0.95. In Eastern Gansu, the typical compaction procedures were (Ninxian County Government 1992),

- Remove the surface debris and organic soils,
- Compact the foundation,
- Spread and compact fills in lifts. For compaction using flat rollers, the lift thickness was about 0.2 m and seven passes were usually needed for each lift. For manual compaction, the spreading thickness was limited to 0.1 m.

It was interesting to note that the slopes of most of the embankments built in this period were very steep, so that the embankments were commonly called "loess bridges" in the area. These steep structures were normally built at ridges and therefore subjected, to a lesser degree, to the effects of water infiltration. Table 1 lists the slopes recommended for use in that period (Ninxian County Government 1992), among which the slope 1:0.3 was used at top of a number of embankments. Figure 2 shows Miqiao embankment as rebuilt in 1958. The slope varies from 1:0.3 at the top 34 m to 1:0.75 at the bottom 6.0 m. The design of such steep slopes was influenced by the limit slope concept. According to the concept, the engineered optimum slope should take a configuration similar to the shape of the critical slip surface. An exponential curve was developed for configuring the side slopes (Shannxi Highway Design Institute 1960).

Table 1 - *Recommended Slopes for Loess Embankments (after Ninxian County Government 1992)*

Soil type	Height of embankment (m)		
	< 20	20 – 30	30 – 40
Q_2 deposit	0.2 – 0.5	0.3 – 0.75	0.1 – 1.0
Q_3 and Q_4 deposits	0.5 – 0.75	-	-

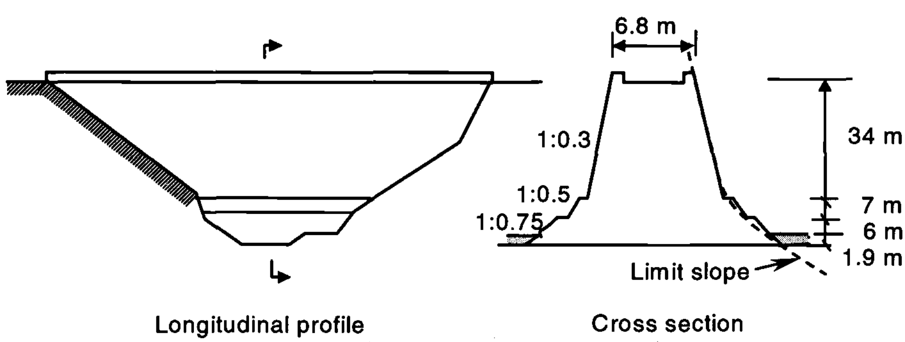

Figure 2 - *Miqiao Embankment as Built in 1958*

Compaction near the side slopes of such embankments was not attainable by machinery and had to be conducted manually. The construction technique shown in Figure 3 was conventionally adopted. Stacked rafters were used to hold the fill near the slope. The rafters were in turn held in place by reed bundles or straw ropes. After the current lift was compacted, the reed or rope connections to the rafters for the previous lift were cut, the rafters were removed to be used again, and the reeds or ropes were left in the embankment, which in fact resembled modern reinforced walls.

Summarized in Table 2 are some of the high "loess bridges" built or renovated during 1950 - 1985. Note that the flatter slopes (1:1.2 ~ 1.75) were adopted mostly for the lower elevations when these embankments were widened in early 1980's.

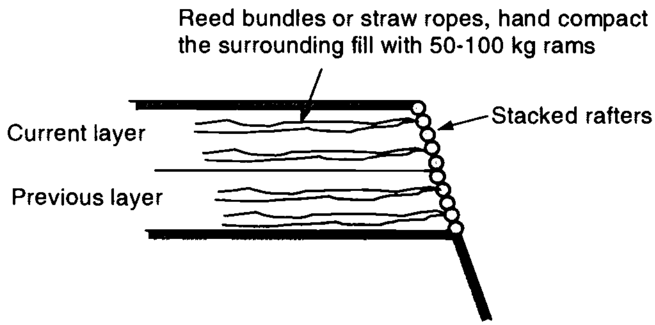

Figure 3 - *Compaction of Steep Embankments*

Table 2 - *Existing Embankments Built during 1950 – 1985*

Name	Location	Length (m)	Height (m)	γ_d (kN/m^3)	w_{op} (%)	Current slope	Year built
Nancang	Nancang town	120	59.3	16.7	14.0	1:0.75~1:1.5	1957
Xiantou	East of Xiantou	180	54	-	-	1:0.75	1955
Lujiayan	West of Lujiayan	160	46.4	-	-	1:1.2	1967
Miqiao	West of Miqiao	140	42.0	-	-	1:0.4~1:1.0	1957
Leijia	State road 309 K1643	101	89.0	17.1	16.5	1:0.75~1:1.75	1970
Wangyan	State road 309 K1641	-	60.0	17.1	16.5	1:0.75~1:1.5	1976
Liujiagou	Yijun County	62	19.2	16.6	18.2	1:0.5~1:1.5	1950's
Wujiayan	Yijun County	43	42.1	16.3	19.7	1:0.16~1:1.2	1950's
Hanzhuan	Huanglin County	62	29.1	16.0	17.3	1:0.35~1:1.25	1950's
Shijiazhuang	Luochuan County	27	61.0	16.6	18.3	1:0.33	1950's

1985 - 1990's

The age of expressways (the interstate system) in China did not start until the late 1980's. Embankments for expressways differ significantly from those for old highways:

- Interaction of fills with water. The roadways have to cross gullies according to the need of road alignment. This requires that embankments be built in the gullies with considerable water passage. Consequently, culverts are needed and the fill materials below the culvert elevations have to interact with storm water ponds.
- Post construction settlement. The pavement structures that account for the majority

of the total cost allow a post-construction settlement less than 0.2 - 0.3 m.

To meet the above challenges, a new compaction standard equivalent to the modified Proctor compaction (ASTM Test Method D1557-91) was adopted by the Chinese highway design and construction codes (China Department of Transportation 1987, 1991), and slopes were designed much flatter.

To evaluate the stability and settlement of embankments, a soil sample was recovered at Jinnin. The silt fraction comprised up to 79% of the soil and the clay fraction comprised 16%. The liquid and plastic limits were $\omega_L = 30\%$ and $\omega_p = 17\%$, respectively. The maximum dry density, minimum void ratio, and optimum moisture content were $\rho_d = 1910\ kg/m^3$, $e_{min} = 0.44$, and $\omega_{op} = 12.5\%$, respectively. According to the new specifications (China Department of Transportation 1987, 1991), the relative compaction is specified to be $K \geq 0.93$ for subgrades and $K \geq 0.90$ for embankment fills. Figure 4 shows a standard profile used for embankment design for the Lanzhou-Xian freeway (State Road 312). Table 3 gives several embankments in the freeway between K603 – K635. Storm water ponds are observed upstream of these embankments, except for these with bottom culverts. The infiltration of water in the loess fill would decrease slope stability and cause a significant increase in settlement, which will be discussed in the next section.

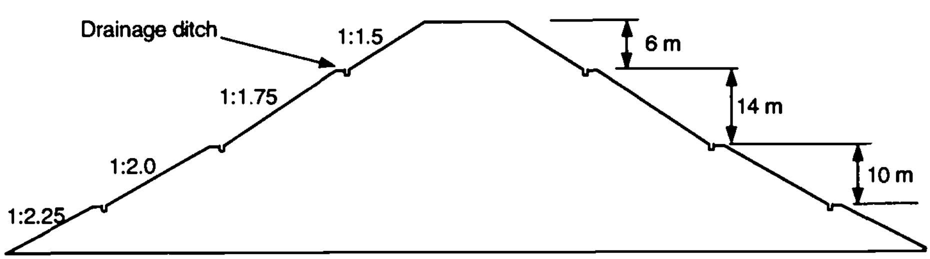

Figure 4 - *Standard Profile of Embankments Compacted According to ASTM Test Method D1557-91 (Modified Proctor)*

Table 3 - *Summary of Embankments in the State Road 312 Between K603 – K635*

Location	Height (m)	Slope	Storm water passage	Water proof measures
K603	28.0	-	Bottom ϕ3.0 m culvert	-
K608	50.0	-	Tunnel 30 m beneath crest	-
K615+120	25.6	1:1.0	Bottom ϕ3.0 m culvert	2 m lime stabilized surface
K619+068	46.8	1:1.75-1:2.5	Mid-level culvert	Geomembrane below 15 m
K621+787	40.0	1:1.75-1:2.5	Two ϕ1.8 m culverts	Vertical clay core
K624+470	49.0	1:1.75-1:2.0	Blank ditches	Vertical clay core
K629	30.0	1:1.50-1:1.75	Mid-level ϕ1.6 m culvert	None

Performance of Embankments

Settlement

The major problems of the embankments built during 1950 - 1985 were excessive settlement and difficulty in maintenance. The fill materials compacted according to the ASTM Test Method D698-91 were identified as slightly collapsible soil, which would collapse and settle due to water infiltration. Moreover, the fills were relatively permeable (coefficient of permeability $k \approx 5 \times 10^{-7}$ m/s) and the surface water could penetrate to a great depth. Thus, large settlement would be expected in the first one or two wet seasons (July to September). For instance, the middle of the Xiantou embankment settled 1.4 m, i.e. 2.6% of the embankment height, in the 8 years after the end of construction in 1984. Most settlement occurred in the second and third years after construction. Afterwards, settlement gradually ceased to develop.

The steep slopes also made it difficult to properly maintain the slopes. As a result, many embankments were eroded by storm water and their slopes became near-vertical cliffs in less than ten years after construction. Consequently, many of the embankments were not durable. In addition, extra settlements were induced due to stress rearrangement, as the side slope became steeper and steeper. For example, the Wangyan embankment was 60 m high and 8 m wide at the crest when built in 1977 (Figure 5). Gradually, the lower part of the sunny side slope was eroded into a cliff more than 50 m in height. The shady slope was also eroded to about 1:0.75 from the original 1:1.0 slope. Settlement started to develop in 1990 and accumulated to 1.5 m (2.5% of height) by 1992. This sudden increase in settlement indicated the initiation of progressive failure of the slope. In response, rehabilitation was started in 1992, with reinforcement of the shady slope at 1:1.25 for the upper 25 m and 1:1.5 for the lower 31 m, as shown in Figure 5.

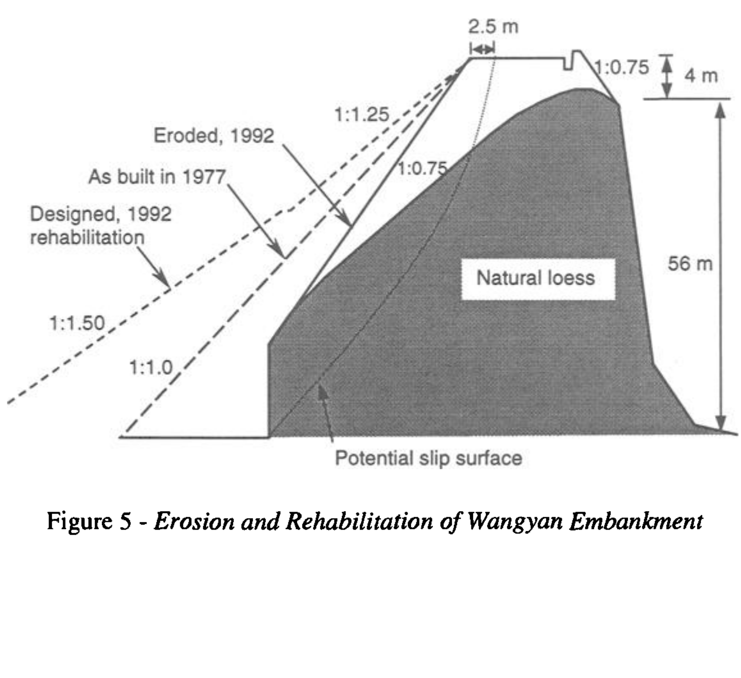

Figure 5 - *Erosion and Rehabilitation of Wangyan Embankment*

The above observed settlement is of the typical values for similar embankments. The Roadbed Construction Specifications (China Department of Transportation 1987) recommend a settlement of 2.5% of the height for 10 - 20 m high embankments. The Design Guidelines for Railways in Regional Soils (China Department of Railways 1992) recommend settlements of 0 ~ 2.5% of the height for embankments lower than 20 m and 1.0 ~ 1.5% for those higher than 20 m. Further, the suggested settlements are also influenced by local annual precipitation (Zuo 1988), as shown in Table 4.

Table 4 - *Suggested Embankment Settlement with Respect to Annual Precipitation (after Zuo 1988)*

Height of embankment (m)	Annual precipitation (mm)		
	< 300	300 ~ 500	> 500
6 ~ 12	3%	4%	5%
12 ~ 24	2 ~ 3%	3 ~ 4%	4 ~ 5%
> 24	1 ~ 2%	2 ~ 3%	3 ~ 4%

The new design and construction methods (China Department of Transportation 1987, 1991) employ the modified Proctor compaction method and adopt the improved embankment profile (Figure 4). The deformation modulus of the compacted soil increases whereas the coefficient of permeability decreases considerably to below 10^{-9} m/s. The surface fill affected by storms in turn reduces to a crust of about 2-m thick. Consequently, the embankment settlement is significantly reduced. For instance, the maximum crest settlement of Dukang embankment of 65 m high was 0.61 m, less than 1% of its height, three years after the end of construction.

It should be noted that settlement control for the new embankments was still based on the experience of the embankments built during 1950 ~ 1985. For instance, the embankments in Table 3 utilized compensation fills of 1.5% of the embankment height, which resulted in the typical crest settlement pattern shown in Figure 6. The maximum settlements occur at two shoulder locations while the central area elevates, which damages road serviceability. Such settlement distribution is due to the differences in the stress-strain behaviors of the fill and foundation materials. Particularly, the stiffness of the fills compacted with the modified Proctor method is greater than the natural soil stiffness, in contrary to the fills compacted according to the standard Proctor method.

Figure 7 illustrates the calculated settlement in the longitudinal section for a 30-m height embankment by Huang (1992) using a three-dimensional finite element method. The side slopes of the gully is 1:0.7. The calculation reveals rather uniform settlement across the gully at all elevations within the embankment, with the settlement at the central area (0.933 m maximum) slightly smaller than that at the side locations (0.954 m maximum). As such, the post-construction settlement configuration illustrated in Figure 6 would be produced, if the thickness of the compensation fill were designed proportional to height. According to the calculated settlement in Figure 7, uniform compensation fills would be more appropriate for design of high embankments that adopt the modified compaction standard.

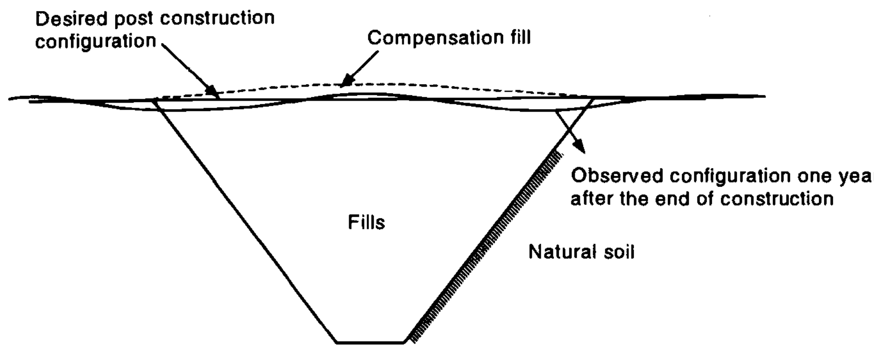

Figure 6 - *Observed Post-construction Settlement*

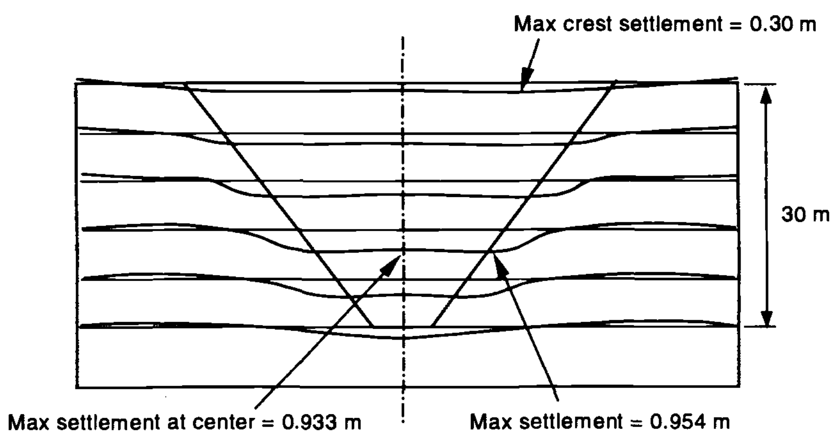

Figure 7 - *Predicted Settlement Distribution in Longitudinal Section (after Huang 1992)*

Stability

Stability analysis was carried out for three of the embankments in Table 2, built during 1950 - 1985. Table 5 lists the parameters and factors of safety of these embankments. Note that the parameters were obtained by direct shear tests using unsaturated samples corresponding to the construction moisture content (Shannxi Highway Design Institute 1960). Therefore, the cohesion should be considered as "apparent cohesion" that included the contribution of soil suction. The program REAME (Rotational equilibrium analysis of multi-layered embankments) by Huang (1982) was employed to perform the analysis using the simplified Bishop method.

Table 5 - *Stability Analysis of Embankments Built Using Standard Compaction*

Name	Location	Height (m)	Dry unit weight	w_{op}	Apparent cohesion (kPa)	Friction angle (degree)	Safety factor
Liujiagou	Yijun County	19.2	16.6	18.2	29.4	19.0	1.33
Wujiayan	Yijun County	42.1	16.3	19.7	39.2	31.0	1.31
Hanzhuan	Huanglin County	29.1	16.0	17.3	11.8	26.0	1.59

To investigate and compare the stability of embankments of different densities, samples of four different relative compactions, i.e., 0.94, 0.90, 0.85, and 0.80, were prepared and tested under saturated and optimum moisture conditions. Table 6 lists the peak strength parameters from consolidated undrained tests (Zhang et al. 1993). Table 7 summarizes the variations of safety factors with embankment height, slope, and upstream

Table 6 - *Friction Angle and Apparent Cohesion of Loess Samples Compacted to Four Densities (Total Stress Parameters)*

Dry density (kg/m^3)	1800		1720		1630		1530	
Degree of compaction	0.94		0.90		0.85		0.80	
Moisture state	Unsatur.	Satur.	Unsatur.	Satur.	Unsatur.	Satur.	Unsatur.	Satur.
Unit weight (kN/m^3)	19.85	21.03	19.01	20.53	17.93	19.97	16.86	19.34
Friction angle (°)	34.60	30.00	33.00	28.00	31.30	27.00	29.50	21.00
Apparent cohesion (kPa)	144.00	66.00	65.00	60.00	58.00	54.00	50.00	40.00

Table 7 - *Sensitivity Analysis of Stability of 30 m and 63.8 m Embankments*

Height of Embankment (m)	Strength parameters	Upstream pond depth	1:1.2	1:1.3	1:1.4	1:1.5
30.0	Foundation: c=30.4 kPa, φ=25°	H_w=0	1.95	1.99	2.03	2.08
	Fill: c=141.1 kPa, φ=34.6°	H_w=10.0 m	1.52	1.54	1.56	1.59
30.0	Foundation: c=30.4 kPa, φ=25°	H_w=0	1.78	-	-	-
	Fill: c=82.3 kPa, φ=34.6°					
30.0	Foundation: c=25.8 kPa, φ=21.3°	H_w=0	1.61	1.66	1.70	1.74
	Fill: c=119.9 kPa, φ=29.4°	H_w=10.0 m	1.27	1.29	1.32	1.32
		H_w=20.0 m	1.14	1.16	1.18	1.21
63.8	Foundation: c=30.4 kPa, φ=25°	H_w=0	1.60	1.64	1.69	1.73
	Fill: c=141.1 kPa, φ=34.6°	H_w=20.0 m	1.27	1.29	1.32	1.36
63.8	Foundation: c=25.8 kPa, φ=21.3°	H_w=0	1.33	1.37	1.40	1.46
	Fill: c=119.9 kPa, φ=29.4°	H_w=20.0 m	1.06	1.08	1.11	1.14
		H_w=40.0 m	0.93	0.95	0.97	1.00

storm water pond depth. Evident from the table is that the safety factor decreases substantially with an increase in the upstream water pond depth. Particularly, the 63.8-m high embankment would fail when it retains a water pond of 2/3 height of the embankment, if the strength parameters are reduced 15% from their peak values. Consequently, seepage control is an important issue in embankment design. In practice, no culverts are allowed to be located above the half height of the embankment.

Table 8 - *Comparisons of Embankments Compacted Uniformly at Relative Compactions of 0.94, 0.90, 0.85, and 0.80 (after Zhang et al. 1998)*

Test No.	Relative compaction γ_d/γ_{dmax}	Total height (m)	Slope	Time elapsed (year)	Upstream pond depth (m)	Slope water infiltration	Maximum settlement (m)	Maximum horizontal displacement (m)	Remark
M-0	0.94	63.80	1:1.50	4.63	No	No	0.260	0.033	
			1:1.50	7.15	23.4	Yes	0.360	0.048	
			1:1.20	8.70	23.4	Yes	0.400	0.059	
			1:1.00	10.25	23.4	Yes	0.650	0.078	
			1:0.63	13.35	23.4	Yes	4.200	3.053	Failed
M-2	0.90	63.80	1:1.75	4.05	No	No	0.574	0.149	
			1:1.75	5.60	23.4	No	0.574	0.319	
			1:1.50	7.15	23.4	No	0.681	0.425	
			1:1.20	8.70	23.4	No	0.681	0.468	
			1:0.75	10.25	23.4	No	0.711	0.915	
			1:0.75	10.75	23.4	Yes	0.830	-	Failed
M-3	0.85	63.80	1:1.75	4.05	No	No	1.212	0.170	
			1:1.75	5.60	23.4	No	1.425	0.808	
			1:1.50	7.15	23.4	No	1.638	0.915	Cracking
			1:1.20	8.70	23.4	No	1.851	1.042	Cracking
			1:1.00	10.25	23.4	No	2.425	1.360	Cracking
M-4	0.85	63.80	1:1.75	4.05	No	No	1.382	0.170	
			1:1.50	5.60	No	Yes	1.630	0.213	
			1:1.20	7.15	No	Yes	1.914	0.234	
			1:0.75	8.70	No	Yes	2.404	0.234	
			1:0.50	10.25	No	Yes	2.808	0.298	
M-9	0.80	63.80	1:1.75	4.05	No	No	2.446	0.213	
			1:1.75	5.60	23.4	No	3.233	0.808	
			1:1.50	7.15	23.4	No	3.744	1.080	
			1:1.20	8.70	23.4	No	3.999	1.170	

Extensive centrifuge tests were also carried out to investigate the stability and settlement of loess embankments. Table 8 summarizes five series of tests with varying upstream pond depths and a slope surface infiltration corresponding to the annual precipitation of 300 mm (Zhang et al. 1998). The embankments were compacted to 0.94, 0.90, 0.85, and 0.80, respectively, of the maximum density obtained from the modified Proctor tests. If the embankment is built with 0.94 relative compaction, the settlement would start to increase markedly at 1:1.0 slope, and the embankment would fail at 1:0.63 slope under the most unfavorable combinations. If compacted with the recommended relative compaction, 0.90, the embankment would fail at 1:0.75 slope. The relative compaction of 0.85 corresponds roughly to the standard compaction (dry density ρ_d = 1700 kg/m^3). At this density, the embankment would experience appreciable settlement, and the lateral displacement would accelerate at 1:1.0 slope if it retained a 23.4-m water pond (Model M-3). If the embankments were built at ridges (no water ponds), the settlement and lateral displacement would not increase significantly at a slope flatter than 1:0.75 and would sustain at slopes steeper than 1:0.50 even with the effect of the design storm infiltration (model M-4). This is in agreement with the field observations that embankments at ridges could be built with steep slopes such as 1:0.3 but would experience a significant settlement. Embankments steeper than 1:0.75 were considered marginally stable (e.g., Wangyan embankment using standard compaction). Moderate erosion on the slope would bring the embankment to the point at which both settlement and horizontal displacement would develop significantly with any further loss of slope fill. Cracks would thus develop that introduces more water into the fills and accelerates the process of slope failure.

Erosion Control

The design profile currently employed (Figure 4) is able to meet the settlement and stability requirements. However, it has an increased erosion exposure since the embankment slopes are much flatter than with older embankments. To reduce erosion of the slope surfaces, several berms are constructed on both sides of the embankment, and an open ditch is built on each berm to collect the surface runoff (Figure 4). The ditches have to be sealed properly to prevent any concentrated leakage.

A variety of biotechnical measures (grasses, trees) have also been tried for erosion control in addition to the ditches. However, these measures are not quite successful, because the vegetation develops poorly in the semi-arid area. Sound erosion control techniques for loess embankments remain to be developed.

Conclusions

Loess is a special silty soil with relatively high apparent cohesion. Embankments approximately 40 m high have been constructed with very steep slopes (up to 1:0.3) at ridges using the standard compaction method. However, such embankments experienced relatively large settlement and were not durable, since progressive failure would start with erosion. The interaction between loess fill and water, particularly the upstream storm-water ponds, proves to be the most critical factor affecting settlement and stability. An embankment that meets the settlement requirements of modern classified highways should be constructed using the modified Proctor method with approximately 0.90 relative compaction. The design profile currently employed is able to meet the settlement and stability requirements under the design surface water infiltration and storm ponds of half embankment height. For better serviceability, the compensation fill for settlement control should be uniform along the embankment, rather than proportional to the height.

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