Geotechnical Design Guide

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Introduction

Welcome to Engineering Ministries International's Geotechnical Design Guide! Whether you are a seasoned Geotechnical engineer or a volunteer hoping to provide soil-related recommendations, our hope is that this design guide will provide some guidance on Geotechnical engineering in the developing world, but ultimately that this guide is used to advance God's Kingdom here on earth.

No good Geotechnical resource is complete without quoting Jesus's famous words:

Everyone then who hears these words of mine and does them will be like a wise man who built his house on the rock (Matthew 7:24)

Let us be hearers and doers of Jesus' words.

EMI DESIGN PHILOSOPHY

Vision
People restored by God and the world restored through design.

Mission
To develop people, design structures, and construct facilities which serve communities and the Church.

Core Values
EMI revolves around the person of Jesus and serves the global Church to glorify God through:

> ***Design:*** *EMI works within the local context to design and construct culturally-appropriate facilities that are sustainable, affordable, and transformational.*
>
> ***Discipleship:*** *EMI develops people spiritually and professionally through intentional discipleship and mentoring.*
>
> ***Diversity:*** *EMI builds the Church by connecting people of diverse backgrounds, abilities and ethnicities to demonstrate our love for God, our love for the nations and the unity we share in Christ.*

PURPOSE OF THIS DESIGN GUIDE

This design guide was written primarily for the benefit of civil volunteers and staff with some familiarity with soils who provide design recommendations related to soils, whether for foundation pad subgrades, infiltration facilities, or pavements.

Secondarily, we hope that this design guide will be useful in engaging with Geotechnical professionals within our EMI network, whether they go on a project trip or are asked to consult remotely. The intent is that this design guide can be used by non-Geotechnical professionals to communicate succinctly with our Geotechnical volunteers.

Lastly, we hope that this design guide can be a resource to document and share with others around the world the ways in which earth structures are designed and constructed in the developing world. Our aim is to provide design recommendations that are appropriate to a local context.

The topics addressed in this design guide are by no means exhaustive and we expect this design guide to grow and change over time. If you have experience or information that might be pertinent to this design guide, please send your comments to Josiah.Baker@emiworld.org.

DESIGN GUIDE FEATURES

The purpose of this design guide is to support the vision, mission, and core values of EMI in Geotechnical engineering design. The standards and procedures contained in this guide combine experience of EMI staff and volunteers as well as many other external resources. Since each project, client, and context is unique, this guide does not propose one-size-fits-all solutions.

This design guide contains hyperlinks and references to ASTM test methods and procedures. Some sections have been reserved for future guide expansion, so the user must ensure the most current edition is being used. Supporting files provided include:

- ***Design Templates*** provide EMI standard design templates for project calculations.
- ***References*** provide external resources and supporting information for topics discussed in the design guide.
- ***Region Specific Guidelines*** provide more specific geological information for different regions.
- ***Training Videos*** provide additional information and guidance to support material presented in the guide.

ASSUMPTIONS AND LIMITATIONS

This design guide was developed based on the authors' experience practicing in the US. Many of the engineering principles described in this design guide are standard within Geotechnical design communities, but others, especially where structural codes govern, may not apply in certain regions.

This guide shall not be used as a substitute for engineering experience and judgments. All engineers are encouraged to practice within their area of competence.

CONTRIBUTORS

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EDITION

This document is a living document and is intended to capture best practices. To continually improve the contents of this document, please provide feedback to Josiah.Baker@emiworld.org.

Table 0- 1 Edition History

Edition	Date	Editors	List of Updates / Comments
Geotechnical Design Guide Version 1.0	February 2023	Josiah Baker, PE; Jennifer Laybourn, EIT; Wil Kirchner, Volunteer	Initial Publication

Contents

Global Guide

1 Introduction to Soil

1.1 WHAT ARE SOILS?

Soil, at a fundamental level, is a three-phase material in which solids (soil particles), liquids (water), and gas (air) interact. As shown in Figure 1, all these states of matter exist simultaneously and may change in proportion over time. Beyond this, physical properties such as particle shape, size, grain size distribution, and hardness affect soil behavior.

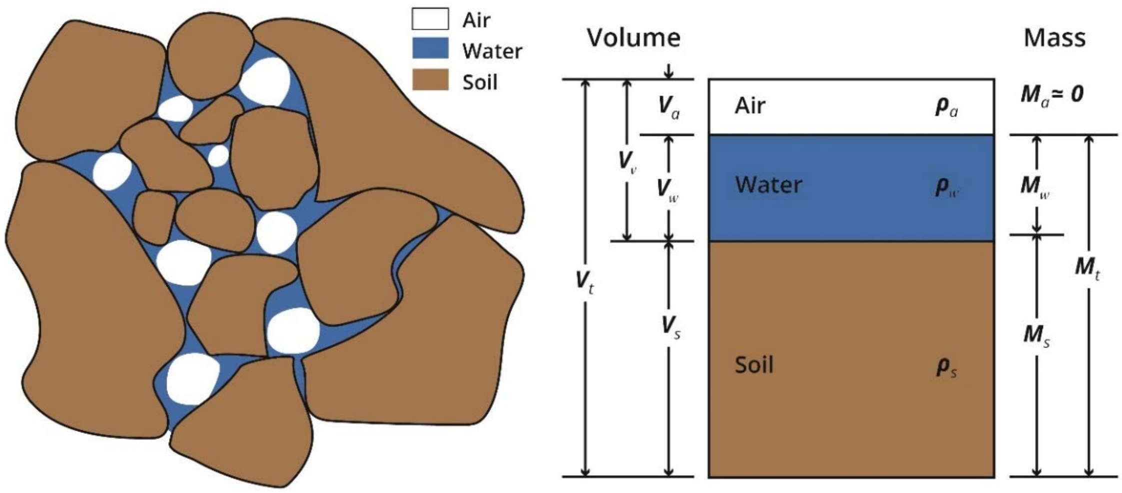

Figure 1. Typical soil matrix and equivalent phase volume and mass.

Another way to describe this is that soil is *particulate* rather than *continuous*. Most other construction materials used for engineering purposes are *continuous*, meaning that the material is a single, continuous mass. On the other hand, soil is *particulate* wherein the physics of soil mechanics is dominated by particle-to-particle interaction.

Figure 2. This parent rock will eventually decompose into granular soil.

Soil particles are most often eroded from weathered rock. The type of parent rock and the erosional and depositional processes affect the features of the derivative soil particles. For example, an igneous rock rich in iron oxides may weather into a red clay.

For the purposes of most EMI project trips, it is not strictly necessary to document parent rock material, but in many instances, it may be worthwhile to note nearby geologic features that may influence soil type, such as rock outcrops, rivers, and lakes. Chapter 17 of this design guide is dedicated to describing the local geology of each field office and may aid in background research for project sites.

1.2 PHASE RELATIONSHIPS

There are many important soil characteristics that are used to describe a soil. The most basic of these are *phase relationships*, which describe various volumetric and gravimetric features of soil. Some *phase relationships* will be referred to throughout the Geotechnical Design Guide, and for clarity and consistency in the way we describe soil, these terms are defined below. The Geotechnical Design Guide will not cover calculations or formulas for *phase relationships* as these can be found in most introductory geotechnical engineering textbooks.

Formulas for all the phase relationships are included in Appendix A.

Figure 3. Unknown subsurface conditions, like large boulders, can cause serious delays in construction!

1.2.1 POROSITY AND VOID RATIO

The main component of soil is the *soil matrix* or *skeleton*, the solid phase in a soil media. Contact forces, including normal and shear loads, are transferred between particles in the soil matrix. The free space between soil particles is called pore/void space which can be filled with water or air. Variation in pore space (and consequently the soil matrix) will affect many soil characteristics such as density, permeability, shear strength, and much more. The quantity of pore space is quantified using either *porosity* or *void ratio.*

Figure 4. Soil matrix of a well-graded sand.

Porosity (n) is the percentage of void space in the soil or rock, or the ratio of volume of voids over total volume. *Void ratio* (e) is like porosity but is instead the volume of voids compared to the volume of solids.

1.2.2 SATURATION AND WATER CONTENT

As shown in Figure 1, void space in soil is filled generally with either a gas (typically air) or a liquid (typically water). The amount of water stored in the void space can vary significantly over time and with distance to the groundwater table. The amount of water in soil is measured using either saturation or water content.

Saturation is the percentage of water, by volume, filling the void space. *Water content* is the ratio of water over solids. There are two ways to measure water content: *gravimetric* and *volumetric. Volumetric water content* is usually denoted with θ, while *gravimetric water content* is denoted with w .

Generally, gravimetric water content is much easier to measure than volumetric and is more commonly reported in Geotechnical reports, usually as a percentage. Most natural soils have a water content w less than 50%. Some clays, peats, and organic soils can have a water content exceeding 100%, which indicates there is more water than solids, and implies potentially unstable soil conditions.

1.2.3 DENSITY, UNIT WEIGHT AND RELATIVE DENSITY

Density and *unit weight* are two ways to measure the amount of soil and water per volume. *Density* (ρ) is the ratio of the soil and water **mass** over total volume. *Unit weight* (γ) is the ratio of the soil and water **weight** over total volume. The two expressions are related by the equation $\gamma = \rho g$ where g is the acceleration due to gravity. Because we are concerned with forces in engineering, *unit weight* is used more frequently.

There are several ways to describe *unit weight*. The *dry unit weight* γ_d is the unit weight when the soil is perfectly dry (S = 0%). The *saturated unit weight* γ_{sat} is the unit weight when the soil is fully saturated (all the pore space is full of water, S = 100%). The m*oist unit weight* γ_m is somewhere in between.

Soils below the groundwater table are affected by buoyancy forces of the water. This is sometimes directly accounted for in the unit weight of soil. The *buoyant* or *effective unit weight* γ' is the difference between the saturated unit weight and the unit weight of water, γ_w = 9.81 kN/m3 (62.4 pcf).

Most natural soils will have a moist unit weight between 12 to 20 kN/m3 (80 to 125 pcf), although it can be much lower for peaty or organic soils, or much high for well-compacted gravel. A well-compacted, well-graded gravel can have a moist unit weight exceeding 23 kN/m3 (145 pcf), which is comparable to most concrete[1].

For projects that involve a lot of fill placement, it is important to understand *relative density*, D_r . *Relative density* is a measure of how dense or compact a soil is compared to its theoretical minimum and maximum unit weights, $\gamma_{d,min}$ and $\gamma_{d,max}$, as determined by ASTM D4253 and D4254 laboratory procedures. A very loose soil has a relative density D_r near 0%, while a very dense soil has a relative density D_r near 100% (although it is possible to exceed 100% for mechanically compacted soils that receive compaction effort far exceeding laboratory effort).

[1] Coduto, page 50, chapter 3.1. Holts, Kovacs and Sheahan, page 28, chapter 2.3.

2 Classifying Soils

2.1 SOIL CLASSIFICATION SYSTEMS

Two main soil classifications systems exist which vary in purpose and methodology. The first is the United States Department of Agriculture (USDA) Soil Classification System, which is used predominately for agricultural purposes. The second is the Unified Soil Classification System (USCS), which is used predominately in engineering and geological applications. The two systems vary in the way soil classification is defined, especially for fine-grained soils. In broad terms, the USDA system is more focused on texture and particle size, while the USCS system is more concerned with soil behavior.

While both classification systems have important applications, most EMI projects will use the USCS system as it is more useful for engineering analyses and design purposes. The design guide will exclusively use USCS designations for soil classification.

The EMI Civil Engineering Design Guide provides a general overview of both soil classification systems, so if you will be classifying soils for agricultural purposes, please refer to Chapter 3 and Appendix A3 of the EMI Civil Design Guide to find a comparison mapping the two methods and various USDA testing methods for soil classification.

2.1.1 USCS SOIL TEXTURE

Soil classification using the USCS method is determined primarily by soil texture for coarse-grained soils and soil behavior for fine-grained soils. Soil texture refers to the physical description and feel of the soil. One way to quantify this is by soil particle size and distribution (the range of particle sizes). Individual soil particles can be classified according to the particle size as summarized in Table 1 below.

Table 1. USCS soil particle classification table (ASTM D2478).

RETAINED ON	MESH SIZE (mm)	PARTICLE DESCRIPTION
300mm (12in)	300	Boulder
75mm (3in)	75	Cobble
19mm (3/4in)	19	Coarse Gravel
No. 4	4.75	Fine Gravel
No.10	2.00	Coarse Sand
No. 40	0.425	Medium Sand
No. 200	0.075	Fine Sand
Passing No. 200	Fine-Grained Particles (Silts and Clays)	

2.1.2 COARSE-GRAINED SOILS

Coarse-grained soils are defined as soils which have more than 50% of particles (by weight) retained on the No. 200 sieve (0.075mm), meaning that most of the soil has a gritty or gravelly texture.

As shown in Figure 5, coarse-grained soils can be further classified as either sand or gravel. In the USCS classification, the No. 4 sieve (4.75mm) distinguishes coarse-grained soils as sands or gravels. When the majority of particles pass through the No. 4 sieve (4.75 mm), the soil is considered a sand while if most of the coarse-grained particles are retained on the No. 4, the soil is considered a gravel.

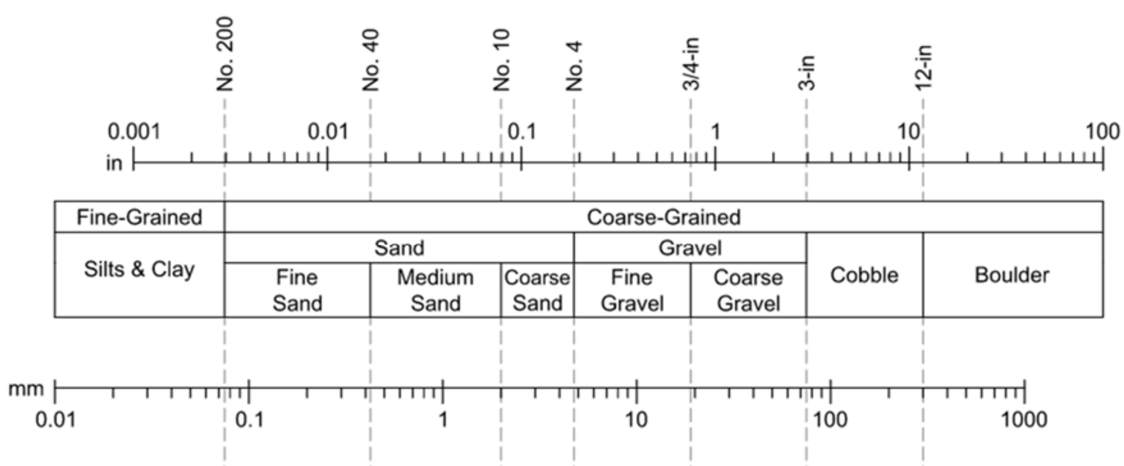

Figure 5. USCS soil classification chart (ASTM D2478).

Figure 6 below shows a typical gradation for poorly graded sand. Note that most of the particles are retained on the No. 200 sieve and pass through the No. 4 sieve.

Another important trait of coarse-grained soils is grain-size distribution. Grain-size distribution describes the range and relative representation of a soil's particle sizes. The USCS uses two types of grain-size distributions: well graded soils and poorly graded soils.

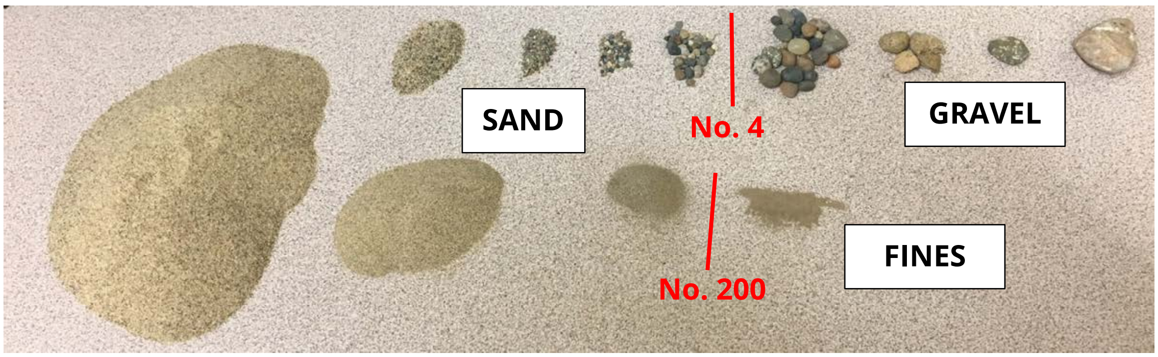

Figure 6. Poorly graded sand passed through a standard set of sieves.

Well graded (in pavement design its often referred to as dense graded) sands and gravels have a wide range of particle sizes with good representation. These materials are preferred for pavements and foundation support over other types of soils.

Poorly graded sands and gravels have either a small range of particle sizes or poor representation of intermediate particle sizes within the range. Poorly graded soils offer the benefit of increased water drainage over well graded soils.

Poorly graded soils can be further classified as either gap graded or uniformly graded. Uniformly graded soils typically have a very narrow range of particle sizes (functionally one particle size). Gap graded soils have a wider range of particles sizes, but there is a "gap" in particle distribution, meaning intermediate particle sizes are poorly represented. Figure 7 shows a diagram of three possible soil particle distributions. Notice the available pore space (white) in the two poorly graded distributions as opposed to the lack of pore space in the tightly packed, well graded distribution.

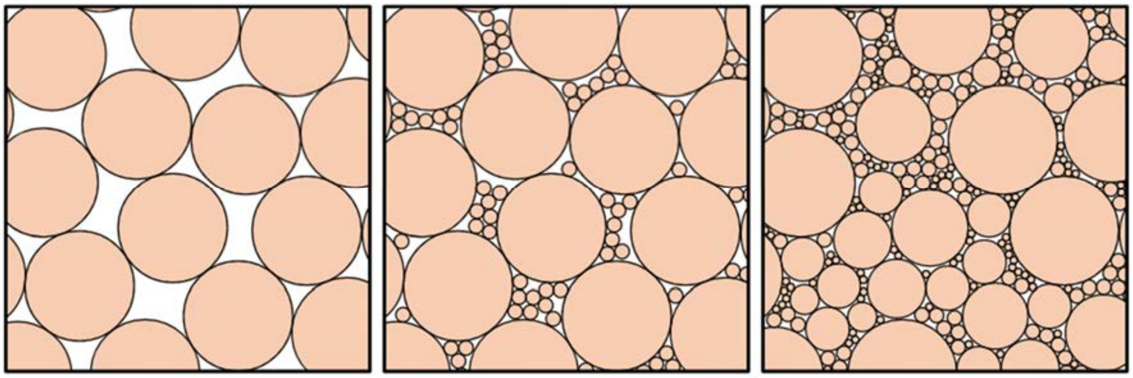

Figure 7. From left to right, conceptual section of uniformly graded (poorly), gap graded (poorly) and well graded soils.

2.1.3 FINE-GRAINED SOILS

When the average soil particle size is smaller than the No. 200 sieve, soil performance is controlled less by contact forces and physical particle features such as size, shape and distribution, but by characteristics unobservable to the naked eye. Due to the small particle size, forces such as Van der Waals forces, electrostatic forces, and chemical bonds begin to control soil performance causing fine-grained soils to be classified by soil behavior rather than by physical characteristics.

Fine-grained soils are broadly categorized into two types: silt and clay. Silts and clays differ fundamentally at particle-scale based on the parent rock material and will chemically react differently. Silts are characterized as either non-plastic or moderately plastic. Silts generally do not exhibit "putty-like" behavior and will only exhibit a "putty-like" behavior over a short moisture content range compared to a clay. Furthermore, silts tend to become brittle and crumble under at low moisture contents rather than harden.

In contrast, clays behave like a plastic material and exhibit "putty-like" behavior over a wider range of moisture contents than silts. Unlike silts, clays harden as moisture content decreases due to matric suction.

Figure 8. Typical Atterberg limit testing setup.

Another distinction between clays and silts is the response to changes in moisture content. Clays tend to react more to changes in soil moisture content. For example, drying may cause tension cracking while wetting can lead to clay particle expansion, causing the soil to expand. Silts may also swell or contract, but it is usually less extreme than in clays.

Fine-grained soil behavior is determined from Atterberg limit testing (shown in Figure 8) which evaluates soil characteristics with changes in water content. The test identifies the soils plastic and liquid limits which are explained below. Figure 9 plots the characteristics on a continuum showing that at a basic level, dry, fine-grained soils behave like a semi-solid, while wet soils behave like a semi-liquid. The soil states can be defined as follows:

- Semi-solid state: the soil may be remolded to a limited degree, but cracks will develop, or it will tend to crumble or harden.
- Plastic state: the soil can be remolded readily without cracking.
- Semi-liquid state: the soil flows under its own weight.

The natural water content of a soil varies in relation to its depth, location on a site, recent rainfall, season, proximity to the groundwater table, and the air humidity above ground. Understanding the Atterberg limits is important to knowing how the soil might behave with changes to water content.

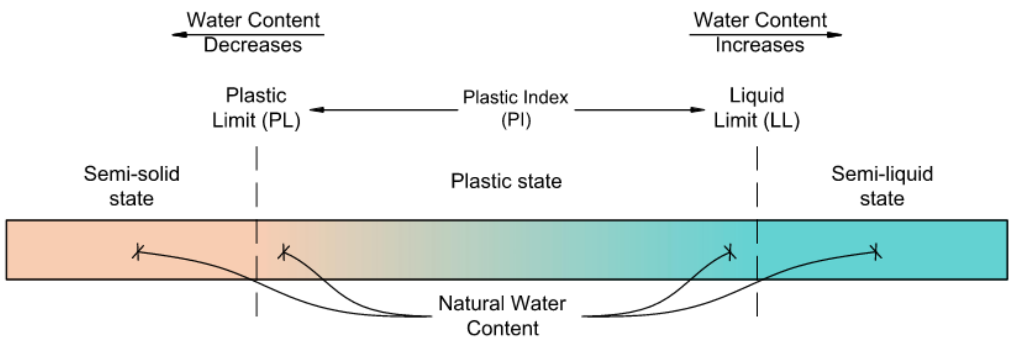

Figure 9. A bar representing the changes in fine-grained soil behavior with variation in water content.

The first Atterberg limit is the plastic limit (PL), which is the moisture content that marks the transition between semi-solid and plastic behavior. Soils in this state have some elasticity. Soils that are drier than the plastic limit may behave like a solid, having a brittle failure mechanism as with many clays, or may be friable and dusty as with some silts.

The second Atterberg limit is the liquid limit (LL) which is the moisture content marks the transition between plastic and semi-liquid behavior.

The liquid limit is used to distinguish low plasticity and high plasticity silts and clays. Fine-grained soils are sometimes referred to as fat clays for high plasticity clays, lean clays for low plasticity clays, and elastic silts for low plasticity silts.

The plastic index (PI) is the difference between the liquid limit and plastic limit and is a measure of a soil's responsiveness to changes in moisture content change

Generally, clays have a higher PI than silts and require more moisture to change, however this is determined graphically by plotting LL vs. PI as shown in Figure 10.

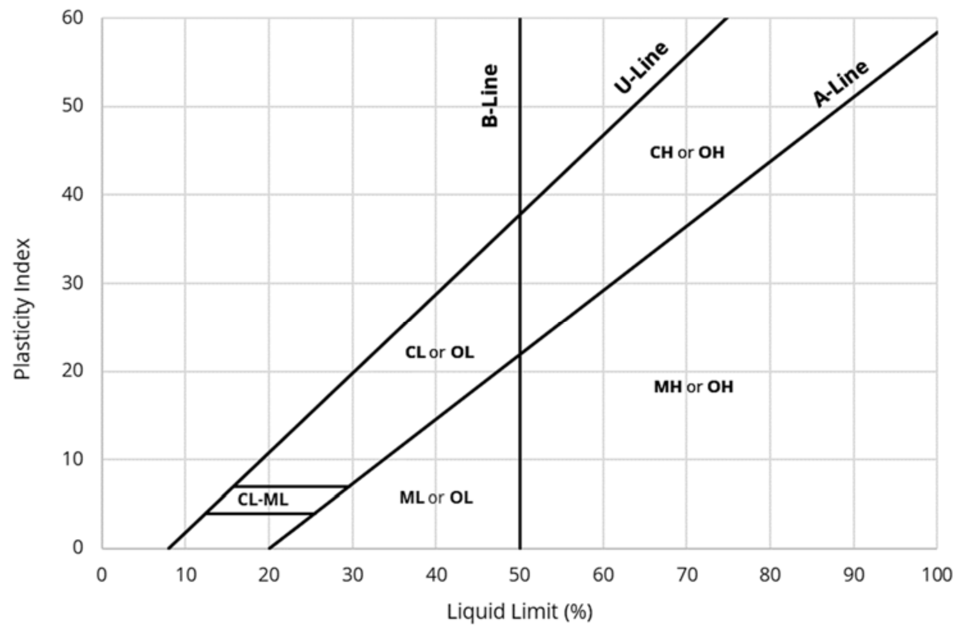

Figure 10. Casagrande Plasticity Chart

2.2 IDENTIFYING COARSE-GRAINED SOILS

2.2.1 FIELD IDENTIFICATION (ASTM D2488)

Since coarse-grained soils can be seen by the naked eye, field identification of coarse-grained soils is typically achieved by visual identification and soil texture. The flowchart included in Appendix B can be used to aid in field classification.

1. First, identify whether the soil is coarse-grained OR fine-grained. The USCS method classifies soil by weight, so some judgment is required in the field. If it seems the soil has greater than 50% coarse-grained particles **by weight**, it is a coarse-grained soil.
2. Once the soil is determined to be coarse-grained, it can further be classified as sand, gravel, cobble, or boulder based on the predominant particle size. Again, based on weight.
3. Finally, the group symbol can be determined by whether the soil is well graded or poorly graded as discussed above. GW is the group symbol for a well graded gravel. SP is the group symbol for poorly graded sand. All group symbols, and their definition, are included in the Boring Log Legend, in Appendix B. Cobbles and boulders do not have group symbols.

4. (Optional) If the soil seems to have a significant percentage of fines (>10%), the soil can be classified using a dual group symbol (e.g., GW-GC well-graded gravel with clay) or by using a different group symbol (e.g., GC clayey GRAVEL).
    a. This is difficult to identifying in the field and is best determined using laboratory analyses. However, if there is a significant percentage of fines (soil is sticky, or you can roll the fine-grained particles), please document this.
    b. For most EMI projects, the first group symbol is enough to justify many design assumptions. If a more refined classification is necessary based on design requirements, please see Laboratory Testing below or ASTM D2487 and D2488.
5. Identify the color according to the color chart provided in the Boring Log Legend in Appendix B.

Useful tools to aid in classifying coarse-grained soils in the field include a wash sieve (No. 200), portable sieve set, and portable scale. If you are having trouble determining whether your soil is a fine-grained or coarse-grained soil, you can quickly perform a Wash Sieve Test to determine the percent fines (see below).

Often, if your soil is gritty, even if you cannot see the particles, it is a sand (possibly SM silty-SAND or SP fine poorly graded SAND), but if it can be rolled or molded into a ball it is also possible that it is a low-plasticity silt (ML), which often has a high percentage of SAND. This distinction is often difficult to make, which is why bringing a wash sieve on the project trip might be helpful. More details on the Wash Sieve procedure can be found below.

A portable sieve set is helpful to determine major group symbols in the field. Since the soil is not oven dried, some air drying may be necessary. The soil must be relatively dry for use with the portable sieves. If the soil tends to clump, a portable sieve cannot be used without washing.

2.2.2 LABORATORY IDENTIFICATION (ASTM D2487 AND D6913)

Wash Sieve Testing

If your soil has a significant percent of fine-grained soils (over 10%), it is often necessary to “wash” the soil first. The wash sieve test utilizes a larger, reinforced No. 200 sieve to determine the percentage of fine-grained soils and coarse-grained soil.

Prior to washing the soil, the sample should be dried to determine moisture content and dry weight. Once this is completed, the soil is soaked with water and agitated using a rod or spatula for at least five minutes until the soil is fully dispersed. After soaking, the soil is placed in the wash sieve and rinsed until the water comes out clear. Soil may be lightly stirred by hand to aid in the washing process. Avoid washing gravel-sized particles or roughly pushing the soil down as this may damage the delicate No. 200 sieve mesh.

Finally, the wash soil is carefully placed into a drying pan. Once the sample has been dried, the percent fines can be calculated from the difference between the initial and washed weights.

Full Sieve Testing

For granular soils with a small fines content, or for soils that have been washed, the soil specimen may be passed through a full sieve test to aid in more accurately identifying the major group type and the gradation of the soil. It is important to select a good representation of sieves based on whether you have sandy or gravelly soil or somewhere in between.

2.2.3 FIELD IDENTIFICATION (ASTM D2488)

Accurately identifying fine-grained soils in the field, without the aid of laboratory testing, is more challenging than identifying coarse-grained soils and requires some experience and practice comparing soils to Atterberg results.

The procedures for identifying fine-grained soils described here have been modified from ASTM D2488. The flowchart provided in the Boring Log Legend in Appendix B can be used to aid in identifying fine-grained material.

1. Roll a ball of fine-grained soil between the palms or on a smooth, clean surface until the thread is less than 3mm in diameter. Note whether it can be rolled or not. If it cannot, it is possibly sand or low plasticity silt.
2. If the ball is too sticky or wet to roll, allow it to air dry or continue to work it until it can be rolled into a 3mm thread.
3. After rolling it into a thread, fold the sample and attempt to reroll it. Repeat this until the thread begins to crumble at 3mm. This is the plastic limit. Note the toughness of the soil as you roll it at the plastic limit. If it requires considerable pressure to roll, it is possibly a high plasticity CLAY. If only slight pressure is required, it may be a low or high plasticity SILT.
4. Lump the crumbled soil and attempt to knead it into a lump. Note whether you can form a lump. If it cannot lump together, it is possibly a high plasticity SILT or low plasticity CLAY. If it can lump, it is possibly a high plasticity CLAY.
5. Try to roll the soil beyond (drier than) the plastic limit. Note whether it can be rolled past the plastic limit. If it can be rolled several times past the plastic limit, it is a high plasticity CLAY.
6. Identify the color according to the color chart provided in the Boring Log Legend in Appendix B.

Another helpful observation to note is the dry and wet texture of the fine-grained soils. When dry, silts tend to break down more easily and may have a floury texture (rather than gritty, which is the case for sands). Clays will tend to form hard lumps when dry. When wet, some describe silts as having a silky or soapy texture without sticking, whereas clays tend to stick. Be cautious, these textures may not always be consistent with lab classifications!

For more information on identifying fine-grained soils in the field, please refer to "EMI Geotech Videos: Field Identification Soils".

2.2.4 LABORATORY IDENTIFICATION (ASTM D2488)

As described earlier in this chapter, laboratory procedures for identifying fine-grained soils utilizes results from Atterberg limit testing to classify fine-grained soils as either silt or clay, and whether it has low or high plasticity.

2.3 METHODS FOR DRYING SAMPLES

The three methods used for EMI projects when drying samples are the oven method, microwave oven method and air-drying method. Dry soil samples are required to identify soil moisture content for tests such as Atterberg limits, standard/modified proctor, and wash sieving.

A sample is considered dry when all free water has been removed. This is identified by taking several weight readings as the sample is drying. The sample is dry when the weight does not change between readings.

2.3.1 OVEN DRYING

Oven drying is considered the most accurate method for drying. To dry the sample, place a soil sample (generally 75 to 200g is a good size depending on the resolution of the scale) in the oven for a minimum of 24 hours.

The oven temperature shall be 105-110° C and remain constant (±5° C) for the duration of the sample drying. A higher temperature could cause the sample to break down, especially those with organic particles which can burn, and a lower temperature may not allow for complete evaporation. *Be careful as the sample can be extremely hot!*

The sample weight should be measured immediately after it is removed from the oven. Samples that are oven dried are generally drier than ambient moisture conditions and will tend to absorb moisture from the air.

2.3.2 MICROWAVE DRYING

Microwaves may be used as a substitute for oven drying in scenarios where a conventional oven is not available (project trips) or if time is essential. The microwave should have an output power rating of less than 1700W.

A 100g soil sample may be placed in the microwave for 5min intervals. Following each interval, the sample is removed, weighed, and thoroughly mixed to achieve a uniform heating. Do not overheat the sample.

After an interval, if the weight has not changed, the soil can be considered dry. Otherwise, continue to dry. *Be careful as the sample can be extremely hot!*

2.3.3 AIR DRYING

Air drying soils is the least effective method, because takes significantly longer and will not fully dry because of ambient moisture. The measured moisture content is highly dependent on the natural humidity in the air. Soil samples that are air dried should only be considered for qualitative and conceptual purposes and should not be used in applications that require precise moisture content readings.

To air dry, spread and crush the soil sample on a flat plan. Place the pan on an elevated, clean, and dry surface. Mix sample periodically to ensure uniform drying. Weigh sample after 72 hours and each subsequent hour until the sample weight does not vary by more than 0.1%.

3 Planning a Site Investigation

3.1 PROJECT SCOPE

To plan out and prepare for a Geotechnical site investigation, ensure that the project scope, EMI's involvement in the project, and ministry client's expectations are clearly defined and understood. These questions may aid in determining investigation type, location, and frequency:

- What does the client hope to achieve on a project site? Are they planning new buildings, embankments, pavements, farms, etc.? What is the occupancy type for each facility?
- Where will these facilities potentially be located?
- What is the level design for new facilities at this point of the project? Final structural designs necessarily require a more thorough site investigation. It may be necessary to recommend additional Geotechnical investigations after the initial master planning is complete.
- Are there any unusual topographic features or steep slopes? These will require additional consideration.
- Is significant cut and fill required to grade the project site?
- Are there any local requirements for Geotechnical investigations?

Many ministry clients may not prioritize or even expect that we would recommend a Geotechnical site investigation, and in many parts of the world, such investigations are not a normal part of the construction process. However, it is important to balance and consider client expectations, the standard of practice in the developing world, and the need for a thorough and safe design.

Before going on a project trip, talk through some of these questions with the project leader to clarify the scope of Geotech work.

3.2 DESK STUDY

Before going on the project trip and visiting the site, a lot can be learned about the project site beforehand in the comfort of your own home (or desk). A desk study is a preliminary assessment of the site using public resources and private (usually from the client) information. These resources might include:

- private and public utility maps,
- aerial or satellite photographs,
- topographic maps and lidar data,
- existing subsurface reports,

- geology reports and maps,
- well logs,
- flood insurance maps,
- soil surveys,
- professional site surveys, and many more.

The availability of these sources will vary greatly by country, so it's important to do research beforehand to know what information is available and what information is lacking. As much as possible, EMI desires to continue to grow our regional expertise by building up a database of publicly available, local resources. Country / office specific resources are available in Appendix C of this report. Chapter 17 also includes region-specific overviews and geologic maps for each field office. If you come across any local information that may be useful for a desk study, like those described above, please email Josiah.Baker@emiworld.org so these resources can be made available in future editions of this Geotechnical Design Guide.

Lastly, as engineers serving with EMI, we have access to a large network of volunteers and active staff that have a broad range of experience in Geotechnical engineering or working in certain regions. We must not neglect to collaborate with others in the network or those who are also going on a project trip. Ask your project leader if you'd like to connect with other like-minded professionals.

In summary, the intent of the desk study is not to fully understand a project site, but to help make informed decisions while planning the investigation and throughout the design process. Inadvertently, the desk study should answer some questions but will consequently produce more questions that can be resolved by the Geotechnical investigation. Additionally, it will help guide what to prioritize once you arrive on site.

3.3 TYPES OF FIELD TESTS

There are many types of in-situ subsurface investigation techniques including, geophysical methods, dynamic and pseudostatic penetration and direct sampling. Many of these techniques are not available in the developing countries, but Table 2 below compares the pros and cons of in-situ techniques that are likely available in your region.

Table 2. Overview of basic in-situ subsurface testing methods.

Field Test	**Description**	**Quality of Data / Sample**	**Advantages**	**Disadvantages**
Standard Penetration Test (SPT)	A split spoon sampler is dynamically driven 0.5m (1.5ft) at regular intervals. The number of blows required to advance the sampler is recorded.	Low / Highly Disturbed	1. Readily available 2. Standard of practice for most regions	1. Sample is highly disturbed 2. Low quality and poor resolution data (infrequent) 3. Sampler gets clogged by coarse soils 4. Unsuitable for gravel and rock
Portable Dynamic Cone Penetration Test (DCPT)	A cone is dynamically driven using a portable hammer. The number of blows required to advance the cone is counted at regular intervals.	Moderate / None	1. Inexpensive 2. Repeatable nearly continuous data 3. Great for limited site access and remote sites	1. No sample 2. Driving depth is limited to user energy 3. Unsuitable for gravel and rock
Cone Penetration Test (CPT)	A cone is pushed at a fixed rate using a hydraulic ram. Penetration resistance, sleeve friction, and porewater pressure are recorded by a computer.	High / None	1. Highest quality, continuous data 2. Accurately predicts soil type	1. Expensive 2. Unsuitable for gravel and rock 3. No sample 4. Highly specialized equipment

Field Test	**Description**	**Quality of Data / Sample**	**Advantages**	**Disadvantages**
Rock Coring	A borehole is created by using tungsten carbide rock bits. A circular diamond tipped bit is used to cut a core of rock, which is fed into the core barrel.	Moderate / Moderate, fractured cores	1. May use the same borehole as SPT method 2. Retrieves a cored sample 3. Produces an RQD parameter describing quality of rock 4. Most common method for investing rock	1. RQD has limited correlations to strength parameters 2. Expensive and time consuming 3. Not used in soil investigation
Hand Augering	A borehole is excavated using a portable hand auger. Soil is cored into an auger bucket for extraction.	None / Highly Disturbed	1. Inexpensive 2. Great for limited access and remote sites 3. Useful to confirm soil type and subsurface conditions quickly 4. Has different bucket styles for various soil conditions	1. Limited in depth 2. Disturbed sample 3. Difficult for gravels, cannot sample rock 4. Produces no quantitative design parameters

Field Test	**Description**	**Quality of Data / Sample**	**Advantages**	**Disadvantages**
Test Pits	A pit is excavated with hand tools or with an excavator or backhoe.	None / Highly Disturbed	1. Inexpensive 2. Sidewalls of the excavation remain relatively intact for in-situ inspection 3. Samples can be retrieved from sidewalls for additional testing	1. Limited to rig arm length 2. Disturbed sample 3. Highly invasive, damages a large area 4. Trenches must comply with safety regulations (see OSHA Excavation regulations) 5. Produces no quantitative design parameters
Thin-Walled Sampling	A thin-walled Shelby tube sampler is dynamically driven to retrieve a soil sample	None / "Undisturbed"	1. Produces the highest quality in-tact sample 2. Suitable for use in SPT boreholes	1. Limited to fine-grained soils and some sands if using a catcher 2. Produces no qualitative design parameters

The preferred type of in-situ field test will vary greatly between projects. For example, investigations for sites with limited access or on slopes, hand augers and dynamic cone penetration may be most suitable. Alternatively, high quality samples are required for laboratory testing for design of critical earth structures like a dam. In this case, thin-walled Shelby tube samples are most appropriate, and collection may be more critical than other tests. Additionally, some countries may have limited availability for geotechnical services like CPT and/or rock coring, so it is important that you understand what is available in the region.

3.4 TEST LOCATION AND FREQUENCY

Test locations should be prioritized in areas requiring significant earthwork and where new facilities will be constructed. In general, test locations should be located along the perimeter of new buildings, especially near building corners, or where supporting columns or heavy structural loads are expected. However, boreholes may be required in the center of the building if the footprint is large and square, or if significant loads are anticipated at certain locations of the buildings.

Additionally, tests in areas receiving a significant amount of earthwork (cut/fill) are needed to determine material properties and compaction requirements.

There is no accepted standard of practice for borehole frequency and location. This is best left to the judgment of the design professionals and project managers working on the project, and local governing regulations. As a rule of thumb, Table 3 below may be referenced for approximate borehole frequency. Note, this table does not consider the quality and uniformity of subsurface conditions. Uniform sites with high quality soils will require fewer tests. Highly variable sites with poor quality soils will require more tests.

Table 3. Frequency of borehole by facility type (Modified from Das (2010) and FHWA Circular No. 05 (2002))

Project Description	Spacing Guideline
Final Design – Low Rise Structure	One per 150-300m^2 footprint area
Final Design – Critical Facilities	One per 50-150m^2 footprint area
Roads	Every 250-300m along centerline of road
Slope Stability & Retaining Walls	Every 25-50m along top of slope with a few in section and toe of slope
Earth Embankments	Every 60-120m generally along the center of new cut / fill embankments
Isolated Structures (e.g., water towers, storage tanks, cantilevered traffic signs)	Two (min) depending on size
Master Plan & Conception Design / Feasibility Study	Depends on level of detail

The depth of boreholes preferably should extend to a firm, native soil layer or to bedrock, especially for projects with heavy or critical facilities. This is not always necessary if subsurface conditions are uniform or if the design loads are low. However, if deep foundations are expected (based on experience in the region), boreholes should extend well below the expected pile length. In every case, boreholes should extend below soft and questionable soils.

4 Field Investigations at EMI

4.1 INTRODUCTION

As EMI grows and field office's regional knowledge expands, EMI is increasingly providing detailed design recommendations compared to recommendations in the past and it is necessary for EMI teams to consider and evaluate the suitability of on-site soils for the design. This section will describe the field investigation tools that are available for each field office, and each tool's purpose, strengths, and limitations. It is important to understand each tool's use and the limitations of each test in order to provide the best recommendations and next steps for our ministry partners.

4.2 WARNING ABOUT UNDERGROUND UTILITIES

In many countries, it is illegal to dig or perform a geotechnical investigation without first performing a private and/or public utility locate. Not only is it illegal in some countries, but it can also be potentially dangerous to dig near electrical utilities.

Before performing a subsurface investigation, consult with your project leader and research digging requirements for the project country. In the developing country, underground utility locating services may not be available and as-built utility location is usually not documented. Always discuss test locations with the client or site manager prior to digging.

You should also collaborate with the project electrical and civil engineers. These engineers are responsible for incorporating existing utilities with the proposed design, and they should be able to assist in determining likely locations for underground infrastructure.

4.3 IDENTIFYING TEST LOCATION

Precise location of boreholes may be difficult to identify on remote sites. If possible, have the surveyors include the borehole or test location in the surveyor. Otherwise, for large sites, GPS coordinates from a phone app will have sufficient accuracy. For small sites, measure the borehole distance to other known features.

4.4 AUGERS

Hand augers are used to quickly identify the subsurface conditions of a project site. Augers allow for a soil profile characterization free of discontinuities or missed areas that can occur when using other equipment such as a split spoon sampler, which produces a sample every 400 to 500mm.

The procedure for using a hand auger is simple: hold the handle with one hand on each side of the stem. Rotate in a clockwise direction while maintaining slight downward pressure against the handle. Once the bucket of the auger is full, remove the auger by continuing to rotate while applying a slight upward pressure until the bucket is free to be removed. Once the bucket is removed from the hole, point the bucket at an angle and tap lightly with a rubber mallet to remove the sample. Carefully observe and document subsurface soil conditions according to the procedures described in Chapter 5.

Each auger kit includes two types of buckets: a clay bucket and a sand bucket. The sand bucket has a comparatively smaller and flatter opening than the clay bucket so that loose sands do not fall out during extraction. Using the appropriate bucket for the soil type will reduce frustration when extracting the sample.

Samples should be labeled and stored in sample containers or Ziplock bags to transport back to the office[2] for further examination and testing such as for water content and Atterberg limit. Samples should be stored in a cool, shaded area so that the sample does not lose moisture. The highly disturbed samples obtained from hand augering are not appropriate to directly perform laboratory shear or permeability tests on without remolding and justifying the remolding process.

Excavated depth is limited to the number of extensions and the underlying soil material. Sites with gravels, rocks, and construction debris/coarse fill are challenging to sample since the particle size is similar or larger than the auger opening. Boreholes in wet or highly sandy conditions may collapse, and hand augering also does not provide information on the relative density and strength of the subsurface soil.

4.5 DYNAMIC CONE PENETROMETERS (DCP)

The Dual-Mass dynamic cone penetrometer (DCP) is used to identify soil consistency / density and soil behavior (soil type) by dynamically driving a hammer into the subsurface soils. The procedures for completing a DCP test are as follows:

- After attaching the cone, cone sleeve, rod, and hammer. The 8kg hammer is used to advance the hammer assembly and is dropped at a consistent height,

[2] There may be challenges with traveling with soil samples between international locations. Check restrictions on importing soil at the destination where analysis is to occur.

allowing for a very accurate determination of the amount of energy applied to drive and advance the cone.

- According to ASTM[3] procedures, the driven depth should be recorded for each blow, however this requirement can be modified according to project needs. For example, the number of blows necessary to advance the hammer assembly 10cm can be recorded instead.
- Additional rods are added each meter until the desired depth or firm subsurface material is reached.
- As a general guideline, soil consistency / density increases as the number of blow counts increase.

The standard ASTM hammer available at each field office was designed primarily for pavement design application and there are empirical equations to estimate subgrade resilient modulus (M_r). However, other empirical equations have been produced outside of the ASTM framework to estimate bearing capacity. This analysis should be done cautiously, however, as the ASTM procedures do not identify material type.

Since DCP testing does not retrieve a physical soil sample, it is important to use this test in conjunction with auger samples to confirm subsurface conditions.

Alternative DCP hammer types and testing procedures exist that provide reasonable estimates for soil material type, behavior, and relative density independent of auger confirmation. Contact Josiah.Baker@emiworld.org if you would like more information on alternative DCP testing methods.

4.6 FIELD SAMPLERS

Details and guidelines for their use and application will be added to this section at a later date.

4.7 HAND TOOLS

The pocket penetrometer and pocket torvane are portable tools that estimate undrained shear strength of fine-grained / cohesive soils. Since these tools are small, always bring them on the project trip. Both the pocket penetrometer and torvane have limitations in terms of use and accuracy, but both will give a general understanding of the material's consistency. However, data from these tools should not replace undrained shear testing performed in a laboratory when such data are warranted in a design.

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[3] ASTM D6951, "Standard Test Method for Use of Dynamic Cone Penetrometer in Shallow Pavement Applications."

Given the variability of readings for both tools, both tools should be used and the results compared if enough soil is available. Several tests should be performed in the same area, and the average reading should be recorded. The most accurate test results are obtained from undisturbed, smooth surfaces, like an exposed vertical scarp, sidewall of a test pit, or a surface cut by a shovel. The pocket penetrometer may be used with SPT samples to varying levels of success, depending on how intact the specimen is.

4.7.1 POCKET PENETROMETER

The pocket penetrometer apparatus loads a small amount of soil axially (in compression) by using a spring-loaded piston. The procedures are as follows:

1. Push the plastic ring until it is flush against the body of the penetrometer and has a reading of 0. This “resets” the pocket penetrometer.
2. Advance the penetrometer at a rate slow enough to simulate undrained conditions.
3. Press into the soil until the engraved circular band on the tip is reached.
4. For softer silts and clays, a wider foot can be attached to the tip of the penetrometer. For the foot, push the penetrometer until the foot is flush with the ground surface.
5. Record the reading on the side of the plastic reading closest to the tip. The penetrometer uses units kg/cm2 (or tsf). Record to the nearest 0.1 kg/cm2.
6. Clean the penetrometer with a towel after each use.
7. Record penetration location, depth, boring if tested on a retrieved sample, testing orientation and other pertinent information.

While the pocket penetrometer does not record a shear force, assuming there is little lateral axial force (a good assumption since the penetration depth is shallow), we can estimate shear strength using the following relationship.

$$s_u = \frac{1}{2} k \sigma_1$$

Where σ_1 is the recorded penetrometer reading.

k is the foot correction factor. k = 1 with no foot. k = 0.0625 with the foot. (Note: diameter sizes may vary between penetrometer. Use your manufacturers recommendation)

s_u is the approximate undrained shear strength of the soil.

4.7.2 TORVANE

The pocket torvane shears a thin band of soil along the edge of the foot of the vane by rotating and shearing the in-situ soil against soil being held by the walls of the foot.

The procedures are as follows:

1. Rotate the pointer arrow counter-clockwise while holding the foot in place until the arrow points to 0. This "resets" the torvane.
2. Press the torvane into the soil the full depth of the blade. Avoid pushing too far.
3. Turn the torvane knob clockwise while maintaining constant vertical pressure to shear the soil. Rotation should be slow enough that failure occurs within 5 to 10 seconds of shearing (maximum shear is achieved). Continue to shear for one full rotation.
4. For harder or softer silts and clays, a smaller or larger diameter foot attachment may be used instead.
5. Record the maximum reading from the pointer arrow. The torvane uses units of 0.1kg/cm^2 (or 0.1tsf). Record to the nearest 0.01 kg/cm2.
6. Clean the torvane thoroughly, especially between blades.
7. Record penetration location, depth, boring (if tested on a retrieved sample), testing orientation and other pertinent information.

Unlike the pocket penetrometer, the torvane measures undrained shear strength directly. Corrections are only necessary when using the either the small or larger diameter vanes rather than the standard vane. Corrections are made using the following formula:

$$s_u = k\, s_{u,r}$$

Where $s_{u,r}$ is the recorded undrained shear strength reading.

k is the vane correction factor. k = 1 with the standard vane. k = 2.5 with the small diameter vane. k = 0.2 for the large diameter vane. (Note: diameter sizes may vary between vanes. Use your manufacturers recommendation)

s_u is the approximate undrained shear strength of the soil.

4.7.3 GEOLOGY ROCK HAMMER

The geology rock hammer is useful for mountainous or rocky projects where it is important to identify the characteristics and quality of surface rock. The blunt end of the hammer is used to break off a fresh piece of rock from a rock outcrop, exposing a fresh, unweathered surface for determining mineralogy and rock type. The blunt end is also used to (very qualitatively) characterize the quality and hardness of the rock.

Wear eye protection when using a geology rock hammer as shards of small rock and dusty debris can be launched quite far when striking the rock. As much as possible, avoid damaging the landscape in a way that detracts from its natural beauty!

4.8 TEST PITS, CUT SLOPES AND EXPOSED SURFACES

Take advantage of the opportunity to observe subsurface conditions through test pits, cut slopes and exposed surfaces. Observing the soil in its undisturbed, in-situ condition provides many benefits such as the ability to directly observe the bedding plane, particle size variation, boulder-sized particles, and many more features. Pocket penetrometers and torvanes can also be used on recently cut surfaces for an estimate of in-situ strength.

Grab samples to be transported back to a lab or the office for further identification and testing can be taken from these exposed surfaces. Note the grab sample location, test pit number, and depth on each grab sample bag.

Take time to observe your surroundings as you travel. Highway cut slopes through mountainous regions near a project site are a great resource for determining local geology and rock type as well as the relative stability of the cut surface.

Be wary of working near deep test pits and excavators. As much as possible, avoid entering a test pit and instead, observe the samples retrieved by the excavator, but if you need to observe the sidewalls of the excavation, have the excavator dig a ramp to the bottom of the test pit. **Never** enter test pits that have a depth exceeding 1.25m (4ft). **Never** walk directly behind an excavator. Instead, position yourself in a location where you are visible by the excavator.

4.9 GROUNDWATER AND WELLS

The groundwater location, well logs, drilling observations and professional well drillers are great sources of subsurface information to help understand the in-situ conditions. However, when drilling a well, make sure to have a representative engineer or construction manager available on site to monitor and document the construction technique, note subsurface conditions, and identify aquifer locations while new water wells are being constructed on a site. While well drillers can identify relative material hardness and often material type, they typically do not provide documentation of their findings.

Cuttings extracted from well construction can be used to give a rough idea of subsurface conditions. However, this must be done cautiously as cuttings are highly disturbed, often mixed with a stabilizing slurry, and the original depth of the cuttings once they come to the surface are often unknown.

In some parts of the world, well drillers are required to document subsurface conditions themselves and submit a report to the local government. Check in your local region to what well log information is available.

In many regions, a shallow or perched groundwater table exists within a few meters of the ground surface. It is important to identify the location of this groundwater table during your investigation, since this water will have a direct impact on the design. Projects may even be impacted by relatively deep groundwater tables through a capillary zone (area of partial saturation) or seasonal fluctuation, which may move the groundwater surface several meters in a year.

In future editions of the Geotechnical and Civil Design Guide, we intend to partner with the WASH team to further address the design and construction of shallow and deep wells. There is still much to be said concerning types of wells, well testing, construction techniques and aquifer types to name a few, but these will not be addressed in this edition of the Geotech Design Guide.

4.10 IDENTIFYING TEST LOCATION

Precise location of boreholes may be difficult to identify on remote sites. If possible, have the surveyors include the borehole or test location in the surveyor. Otherwise, for large sites, GPS coordinates from a phone app will have sufficient accuracy. For small sites, measure the borehole distance to other known features.

5 Estimating Design Parameters

5.1 RESERVED FOR FUTURE EDITIONS

This section of the Geotechnical Design Guide is reserved for future editions of the guide. The goal of the section is to provide guidance for field offices on what appropriate Geotechnical design parameters might be. This is difficult to achieve since soil, by nature, is highly variable.

If you have any feedback regarding what this section should include, experience with region specific soil characteristics, or general practice estimating soil parameters in your local context, please send an email to Josiah at Josiah.Baker@emiworld.org.

6 Soil Permeability

6.1 INTRODUCTION TO PERMEABILITY

Soil permeability measures the ability of a liquid (usually water) to move through a soil media. Permeability, usually denoted as '*K*', is an intrinsic soil parameter, meaning that it is a fundamental parameter that does not vary as conditions change (like pressure head or viscosity).

Permeability is a crucial parameter for any project involving water retention / detention facilities, perforated pipes, and leech fields / absorption systems, to name a few. Soil permeability is affected largely by the void spaces in the soil media. Coarser, uncompacted, and open-graded soils have greater void spaces and thus higher intrinsic permeability.

When permeability is measured in a lab, a distinction is often made between the dry (or partially saturated) permeability versus the saturated permeability. Due to the entrapment of air in the void space, dry soils will have a lower permeability than saturated soils. Generally, in Geotechnical engineering, we are concerned only with the flow of water through a saturated media, so *permeability* is denoted as either 'k' or 'k_{sat}'

Flowrate describes the volume of water ***exiting*** a soil media. Flowrate is defined by *Darcy's law* flow which states:

$$q = kiA$$

The flowrate is seen to be a function of hydraulic gradient i and the soil permeability k, in addition to the cross-sectional area of the soil mass. As the hydraulic gradient increases (or as head losses increases through a soil media), the flowrate also increases.

Soil infiltration, measured by infiltration testing (or percolation testing), is another important design parameter and is used on many EMI projects. *Soil infiltration* is the measure of flowrate ***entering*** a flowing ***through*** soil media. Without getting into technical details, it is important to note that while these terms are related, *permeability, flowrate*, and *infiltration* are different and great care should be taken when designing with these terms.

During field infiltration test, *infiltration rate*, tends to decrease over the duration of the test. Since the infiltration rate is the measure of water *entering* the soil media, the water will tend fill the void space first before flowing through the media. It is important to only report the infiltration rate for saturated soil conditions!

Table 4. Typical values of hydraulic conductivity for saturated soils (modified from Das 2010)[4].

Soil Type	*k*	
	cm/sec	ft/min
Clean gravel	100-1.0	200-2.0
Coarse sand	1.0-0.01	2.0-0.02
Fine sand	0.01-0.001	0.02-0.002
Silty clay	0.001-0.00001	0.002-0.00002
Clay	<0.000001	<0.000002

Table 4 provides a list of typical permeability rates based on the soil material type. Permeability is a useful design parameter for facilities like reservoir embankments which are designed to permit or limit the flow of water through the embankment.

Table 5. Typical infiltration and application rates for wastewater facilities (modified from EPA 1980)[5].

Soil Type	Infiltration Rate		Effluent Application Rate
	cm/sec	ft/min	Lpd/m^2
Gravel, coarse sand	>0.04	>0.08	Too coarse for sewage treatment
Coarse to medium sand	0.04-0.008	0.08-0.01	49
Fine sand, loamy sand	0.008-0.003	0.01-0.005	33
Sandy loam, loam	0.003-0.001	0.005-0.003	24
Loam, porous silt loam	0.001-0.0007	0.003-0.001	18
Silty clay loam, clay loam	0.0007-0.0004	0.001-0.0007	8

Table 5 provides typical infiltration rates compared to appropriate application rates for wastewater facilities. The design parameters in these tables are intended to be

[4]Das, Braja M., “Principles of Geotechnical Engineering”, table 7.1 (2010).
[5]Environment Protection Agency, “Design Manual: Onsite Wastewater Treatment and Disposal Systems”, table 7-2 (1980).

used for conceptual purposes only and **are not** a substitute for field and laboratory testing.

6.2 INFILTRATION (PERCOLATION) FIELD TEST

Prior to designing a stormwater or wastewater facility it is crucial to perform infiltration testing to confirm the soil's suitability and determine sizing requirements. There are several methods of producing percolation rates. For most EMI projects the Encased Falling Head approach is most suitable. The purpose of the Encased Falling Head method is to use a PVC pipe to control, isolate and design for vertical infiltration *only,* without allowing water to infiltrate out of the sides of the hole. This is critical since most infiltration facilities are **significantly wider than they are tall** and calculating infiltration this way can lead to significant, unconservative error. This test is generally not suitable for gravely soils, because it is difficult to seal the bottom of the PVC, which is necessary to prevent water from exiting through the gap between the hole and pipe and infiltrating through the sides of the hole.

It is important to excavate a preliminary borehole prior to infiltration testing to determine in-situ soil conditions. Record these subsurface conditions and, if possible, determine the location of the groundwater table. Following this initial borehole, consider these questions when considering additional testing and/or design:

- Is there shallow bedrock? If yes, the location may not have an adequate infiltration rate and storage may be limited.
- Is there shallow groundwater? If yes, the high groundwater level may limit the available storage and infiltration rates. Many jurisdictions require a certain distance from the bottom of the designed facility to the seasonal high ground water table.
- Are the subsurface soils predominantly sands or gravels? If yes, and long-term storage or water treatment is desired, these soils may not be suitable as the infiltration rates for gravels are too high and little filtration occurs. These soils are not suitable for septic leech fields.
- Are the subsurface soils clay? If yes, these soils will have very low infiltration rates and may not be suitable for retention ponds or leech fields that are designed to disperse water via infiltration.

Before beginning the infiltration test, talk with the project civil engineer to determine the desired elevation for proposed facilities. At a minimum, all facilities should be at least 0.5m above the groundwater table.

Infiltration Test Equipment Checklist:

- Hand Auger
- Rubber Mallet (useful to loosen soil from hand auger)

- Wood
- PVC Pipe
- Timer
- Tape Measure
- Glove, Hardhat, Vest
- Water (usually one paint bucket of water is sufficient for one hole)

6.2.1 ENCASED FALLING HEAD PROCEDURE

1. Excavate a preliminary borehole to determine in-situ subsurface conditions. If possible, determine the location of groundwater.
2. Excavate a second test borehole to the desired trench or pond bed depth. **The test should be a minimum of 0.5m above the seasonally high groundwater table.**
3. Place the PVC pipe in the borehole. Use the mallet and wood to ***drive the PVC pipe an additional 15cm (6in)***. This is crucial to control the soil that is being tested and to ensure that we are only testing vertical infiltration. Most stormwater and wastewater facilities span a wide area, thus most infiltration is vertical.
4. Presoak the soil. Fill the pipe with 30cm of water from the bottom of the borehole and allow it to soak for a minimum of 4 hours (overnight preferred, especially for clays). If the water seeps away entirely in less than 10 minutes after filling the hole twice, the test can proceed immediately. As much as possible, make sure the hole does not drain completely so that the subsurface soils can continue to soak.
5. Perform the infiltration test. Fill the pipe again with 30cm of water. Using a tape measure, make an initial reading by measuring the distance from the top of the pipe to the surface of the water. Every 10 minutes, measure and record this distance. These recordings may need to be more frequent for sands and gravels, which drain quickly – use your judgement!
6. The first trial is completed after 1 hour or if the water has seeped away completely in less than an hour.
7. The infiltration rate is taken as the DROP IN WATER LEVEL over TOTAL TRIAL TEST TIME, or 1 hr.
8. Additional trials should be completed by filling the pipe again with 30cm of water. A minimum of two trials should be completed until infiltration rates between subsequent tests are about the same.

6.3 TYPES OF INFILTRATION FACILITIES

Adding impervious surfaces such as paved walkways, parking, and buildings tends to increase the average runoff coefficient across a project site. Impervious surfaces increase the speed at which water flows across the surface of a site and decreases the time of concentration and amount of water that naturally infiltrates into subsurface soils. The impact of increased concentrated flow on the site should be studied in the way it effects on-site facilities, while also striving to minimize additional discharge (beyond what is natural) from exiting this site. If additional discharge cannot be avoided, the engineer should work with local governments to ensure compliance with local regulations.

If the designer chooses to treat and dispose of wastewater onsite, this is yet another source of water that must be dealt with on site. Design facilities for wastewater retention and infiltration look slightly different than stormwater infiltration, but the mechanisms behind the design are fundamentally the same: water infiltrating or percolating into the subsurface.

The two primary on-site systems that are used to control and/or limit the flow of water within the site and discharge it off the site are *detention* and *retention* facilities. Both facilities store water, at least temporarily, in order to mimic flowrates and volume prior to additional construction. Table 6 briefly compares these two types of facilities.

Table 6. Comparison of detention and retention facilities.

Type	Purpose	Discharge	Mechanisms for Water Dispersal	Types of Facilities
Detention Facility	Temporary Storage	Outlet controls discharge exiting the facility	Discharge Evapotranspiration Infiltration (small)	Rain Garden Dry Pond Wet Pond Underground Storage
Retention Facility	Permanent or Short-term	No discharge exiting the facility except during extreme conditions	Evaporation Infiltration	Rain Garden Dry Pond Wet Pond Dry Well Soakage Trench

EMI's Civil Engineering Design Guide Section 4 includes some provisions and recommendations for the design of wastewater facilities. The "EMI Water and

Wastewater Design Template" aids in sizing components of Septic Systems and Soakage Trenches.

As these design guides develop, we hope to include design recommendations for other stormwater facilities as listed in the table above.

7 Foundation Alternatives

7.1 SHALLOW VS. DEEP FOUNDATIONS

Foundations are physical systems by which structural loads are transferred from a building or engineering to the underlying subsurface soils. Geotechnical engineers categorize foundations in two ways: *shallow foundations* and *deep foundations*.

Shallow foundations are foundations that are (usually) constructed near the ground surface. Structural loads are transferred from a column, stem wall, or grade beam to a widened concrete pad (called the *footing*) in order to distribute the load over a larger area. The controlling factors for shallow foundation design are the soil *bearing capacity* and *settlement*.

Deep foundations are foundations that extend deep below the ground surface transferring loads through shear along the sides of the foundation and by end bearing at the bottom. Typically, foundation design of deep foundations is controlled by the ability of the subsurface soil and rock to resist vertical forces. However, in hard rock the design may be controlled by the structural integrity of the pile itself.

7.2 CONSIDERATIONS FOR SHALLOW FOUNDATIONS

Bearing capacity occurs when the driving forces (dead and live loads) exceed the available resistance in the soil, resulting in a rotational-like failure for isolated footings or punching failure in soft silts and clays. Usually, bearing capacity is not the primary mode of failure as weaker/looser soils will tend to consolidate and settle as new loads are applied.

Settlement occurs when as soil particles within a soil matrix tend to reconfigure into a more stable and compact arrangement as the effective stress increases.

Other terms that are often bounced around are *spread footings, footers, strip footings, continuous footings* and *isolated footings.* These are all fundamentally shallow foundations. *Spread footings* and *footers* are another term synonymous with shallow foundations. *Strip footings* and *continuous footings* refer to long spread footings supporting line loads.

7.3 RESERVED FOR FUTURE EDITIONS

This section of the Geotechnical Design Guide and Sections 10 and 11 are reserved for foundation design principles and will be added to in future editions of the guide. Many of EMI field offices are not equipped to provide foundation design recommendations, but as the need arises within EMI, these sections can be added to at a later date. Potential topics that could be included here or in a related appendix include:

- Design considerations for deep foundation

- Shallow Foundation Design (or references to design methodologies)
- Settlement Analyses and Boussinesq Theory for Load Distribution
- Principles of Deep Foundation Design
    - Alpha & Beta Method Overview
- Design Assumptions by Region
- Deep Foundation Design / Construction Alternatives by Region

If you have any feedback regarding what these sections should include, or experience with foundation design and construction in your region, please send an email to Josiah at Josiah.Baker@emiworld.org.

8 Shallow Foundation Design

8.1 RESERVED FOR FUTURE EDITIONS

This section of the Geotechnical Design Guide and Sections 9 and 11 are reserved for foundation design principles and will be added to in future editions of the guide. Many of EMI field offices are not equipped to provide foundation design recommendations, but as the need arises within EMI, these sections can be added to at a later date. Potential topics that could be included here or in a related appendix include:

- Design considerations for deep foundation
- Shallow Foundation Design (or references to design methodologies)
- Settlement Analyses and Boussinesq Theory for Load Distribution
- Principles of Deep Foundation Design
    - Alpha & Beta Method Overview
- Design Assumptions by Region
- Deep Foundation Design / Construction Alternatives by Region

If you have any feedback regarding what these sections should include, or experience with foundation design and construction in your region, please send an email to Josiah at Josiah.Baker@emiworld.org.

9 Deep Foundation Design

9.1 RESERVED FOR FUTURE EDITIONS

This section of the Geotechnical Design Guide and Sections 9 and 10 are reserved for foundation design principles and will be added to in future editions of the guide. Many of EMI field offices are not equipped to provide foundation design recommendations, but as the need arises within EMI, these sections can be added to at a later date. Potential topics that could be included here or in a related appendix include:

- Design considerations for deep foundation
- Shallow Foundation Design (or references to design methodologies)
- Settlement Analyses and Boussinesq Theory for Load Distribution
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If you have any feedback regarding what these sections should include, or experience with foundation design and construction in your region, please send an email to Josiah at Josiah.Baker@emiworld.org.

10 Retaining Walls & Slope Stability

10.1 IDENTIFYING UNSTABLE SLOPE CONDITIONS

Slope stability issues occur when sediment, rock, snow or another mass move downslope in response to gravity. The downslope movement is a natural process that can be accelerated by undercutting the slope base, clearing stabilizing vegetation, or diverting natural drainage. Unstable slopes on a project site should be identified prior to design and avoided. Common indicators of unstable slopes are discussed below. If you encounter one of these on site, additional research and analysis should be performed.

10.1.1 SCARPS AND TENSION CRACKS

The most obvious indicator of existing unstable slope conditions and historic landslides is the existence of scarps. Scarps are dramatic steep or vertical topographic drops that run approximately parallel to the slope contours. Scarps occur at the head, or top, of a landslide where the landslide body moves away from the original slope.

Figure 11. Tension cracks forming parallel to the crest of a slope indicating failure.

If left unresolved, the scarps will at best slump and separate from the slope to a more stable configuration, and at worst develop future landslides that pose a safety risk to existing facilities and their occupants.

Figure 12. Deep scarps forming behind residential construction.

Like scarps, tension cracks may also form parallel to the slope contours. Tension cracks may indicate the formation of a future scarp or may occur in the middle of the landslide area where the body of the landslide is separating from itself.

10.1.2 TRANSVERSE CRACKS

Transverse cracks, as shown in Figure 13 form along the sides or edges of a landslide. Typically, these cracks will not manifest at the ground surface until significant sliding has already occurred. Transverse cracks show the horizontal boundaries of the landslide area.

Figure 13. Transverse crack extending through flexible pavement surface.

10.1.3 BENT AND CURVED TREES

Bent and curved trees are an easy way to identify areas of slope movement or mass soil creep (gradual movement over time). However, use this indicator in combination with others as there are other situations that will result in bent trees such as movement toward an area with more sunlight, or heavy snowfall damage to young trees.

Bent trees (from a vertical position), especially older trees, indicate a location of slope movement and may hint at future landslides. Since the bent trees were able to grow vertically for some time, the bend suggests a sudden change in ground conditions, such as removal of material at the slope toe or changes in groundwater conditions, that result in relatively quick earth movement.

Curved trees (also referred to as “pistol-butted” tree) are distinguished in that the base of the trunk curved from pointing downslope to pointing vertical or slightly upslope. Curved trees indicate that soil creep occurred while the plant was young, causing the sapling to point in the horizontally toward the downslope direction. The plant will naturally attempt to grow vertically toward the sun and will correct the direction of its trunk growth as seen in Figure 14.

Figure 14. Pistol-butting in a large, old evergreen tree.

10.1.4 MOIST GROUND CONDITIONS

Steep terrain generally lends itself to rapid water movement, but if the ground surface is uneven, water may infiltrate into the subsurface, resulting in decreased slope stability. Figure 15 below is one example of a slope failure where excess ground moisture produced slick conditions and a small landslide across the sidewalk. Notice the moisture seeping out of the bottom of the landslide debris.

Figure 15. Slope failure due to excess ground moisture.

Sites on slopes should be inspected for areas where water may build up. The designer should also be wary of areas that are seasonally damp or areas where groundwater exits the ground surface in the form of springs. These areas will often have significant growth of hydrophilic plants, indicating that an area is poorly drained. It is worthwhile to become familiar with types of water-loving vegetation that grow naturally in the region.

10.1.5 DAMAGED FACILITIES AND UTILITIES

Existing facilities and utilities will often exhibit stress prior to any visibly noticeable movements in the slope. For example, building edges and corners near the edge of the slope might develop cracks in the concrete, doors and windows may become difficult to open and floors near the slope may begin to settle. Often, the distress is obvious, as is the case in Figure 16, but sometimes the distress is small, such as with hairline cracks in concrete.

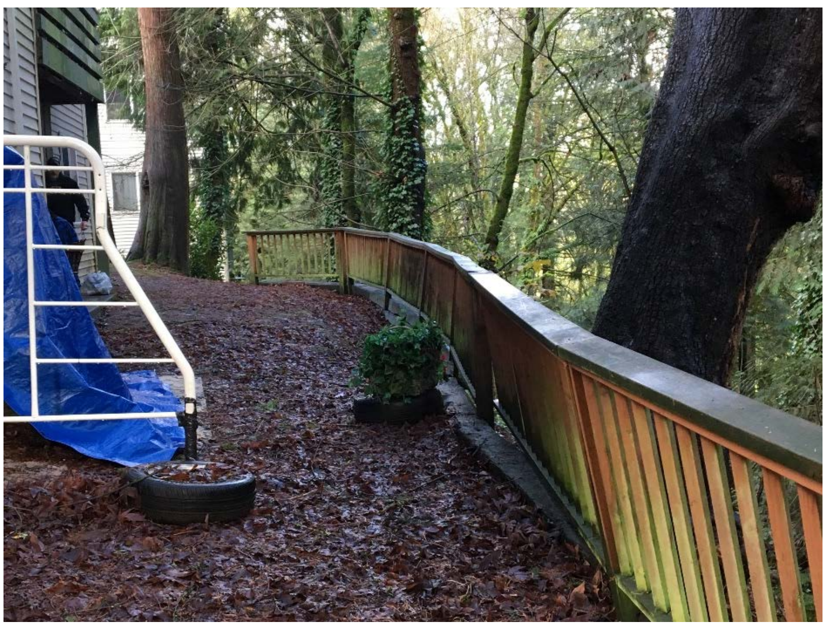

Figure 16. Slope failure resulting in damage to the border rail.

Landslides may also begin at or intercept with utility pipes where subsurface soils are disturbed by previous construction. Soil movement will then continue to accelerate as water and wastewater pipes break and release additional moisture into the sliding area.

10.1.6 LANDSLIDE DEBRIS

Areas with historic or ancient landslides may also be prone to future sliding, especially if there have been changes to the site that will decrease slope stability. Over time, features like scarps and tension cracks will slump and fill. Ancient landslide areas can be identified by the bowl-shaped depression created by the landslide movement. However, this is difficult to identify visually without the aid of survey or lidar data.

It is often easier to identify an old landslide area by the fan of landslide debris that has been transported downslope. The remaining debris forms an obvious lump that protrudes from the slope surface. It is hummocky and may cause water to build up (a potential future problem!).

Figure 17. Recent landslide debris. Over time, this will develop into a hummocky, uneven surfaces.

10.2 CAUSES OF SLOPE INSTABILITY

Slope movement occurs as the effect of friction on the potential sliding surface is reduced and the mass slides under gravity. Physical triggers are typically required to initiate slope failure and common triggers are described below:

- **Geologic Factors:** Weak, weathered bedrock, jointed rock, or bedrock that dips parallel to the slope can decrease stability. Cracks form in rocks as they naturally expand due to cooling or removal of surficial loading and reduce cohesion.

  Thin, weak bedding planes separating layers of rock from each other, often go undetected during the geotechnical investigation and can greatly reduce the perceived friction between layers.

- **Erosion or Removal of Toe Material:** Erosion by wind and water causes continuous removal of material on both natural and man-made slopes resulting in changes to the geometry of the slopes, eventually leading to slope failure. Additionally, rivers can erode the toe through normal stream flow or catastrophic flooding.

- **Increased Moisture Content:** Rainfall can weaken natural and man-made slopes by saturating the soils and reducing the apparent cohesion between soil particles, making them more prone to erosion. Water can also enter existing cracks and weaken the soil layers. Water induces lateral hydraulic pressures that tend to destabilize a slope. Finally, the excess water can add weight to the top of the slope and act as an additional surcharge load.
- **Removal of Vegetation**: Droughts, fires and human removal of vegetation such as logging and deforestation, decreases slope stability by removing roots that hold soil in place making it more resistant to erosion.
- **Lateral Ground Motions**: Seismically induced forces such as those from earthquakes, mining activity, nearby construction, etc. add dynamic lateral forces to the slope and decrease the soil's shear strength and stiffness making it more prone to failure. Failures typically occur in the undrained condition (when soils are more saturated).
- **Introduction of Surcharge Loads**: The addition of loads on top of the slope increases the gravitational load but adding load to the bottom of the slope can provide stability.
- **Over-steepened Surfaces:** Granular material has a natural angle of repose where the surface can stand without stabilization. Oversteepening this surface without stabilization will cause failure if the slope is modified.
- **Disturbed soils:** In general, soils and unconsolidated sediments are weaker than rocks since particles are not cemented together and lack significant compression from overlying materials.

10.3 TYPES OF SLOPE MOVEMENT

Before analyzing a slope failure or making repair recommendations, the designer must evaluate the type of slope movement or make an assumption about the type of slope movement. USGS has a useful diagram illustrating some of the different types of slope movements (see Figure 18).

Slope movement can be placed into three broad categories: fall, slide, or flow. During a *fall* (illustrations D and E) material drops through the air vertically or nearly vertically. This is common on steep rock faces, or when a slope is cut into dipping bedrock surfaces.

Slides occur as a soil mass moves along a plane of failure with minimal internal movement (illustrations A, B, and C). This is the convention definition of a landslide.

Flows occur as a soil mass moves initially along a plane of failure then beyond across the slope surface (illustrations F, G, and H). Flows exhibit “liquid-like” motion with debris flows or earthflows that are churned or mixed up.

Slope movements can be further classified. Here are a few more common types of slope movement:

- **Creep:** slow, shallow movement occurring in moderate to steep slopes (illustration I). Trees, fence posts or grave markers that are supposed to stand upright lean downslope.
- **Shallow Surface Movement (Translational Sliding):** shallow movement occurring across a flat and long geometry with respect to depth (illustration B). Occurs most often on slopes where there is a clear transition between subsurface materials or groundwater conditions.
- **Block Sliding:** movement of a soil or rock mass along joints, fissures or weakened zones allowing the mass to move as a block or wedge down the slope (illustration C). Often, the separated block is quite obvious and leaves an inclined failure surface behind. Failure could happen gradually over the span of multiple months or in a matter of seconds.
- **Circular (Rotational) Sliding:** movement of a soil mass across a circular slip surface occurring generally in over-steepened, relatively homogenous soils. The failure can occur on the face, about the toe, or at the base of the slope as shown in Figure 19.
- **Mud and Debris Flows:** movement that occurs as a mass of sediment loses strength and flows like a viscous fluid (illustration F and H). Flow failures can occur in both dry and saturated soils and the classification (mud vs debris) depends on particle size. If the flow material is sand-sized or smaller, it is considered a mudflow while gravel-sized or large is considered a debris flow.

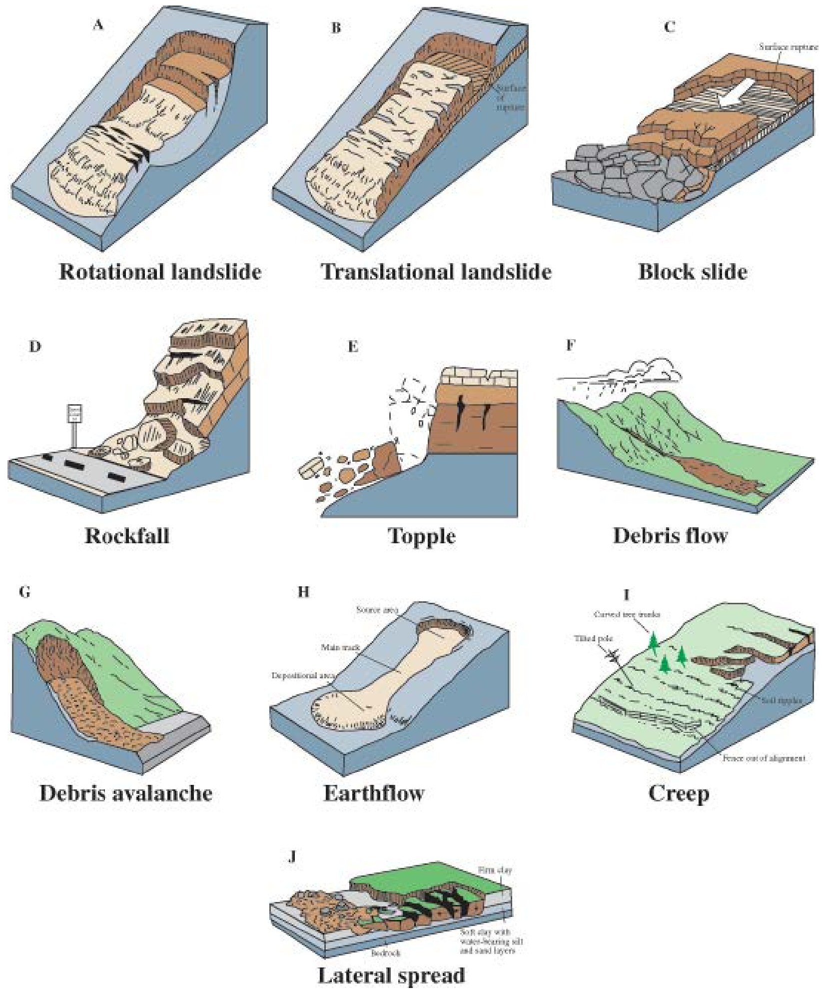

Figure 18. Types of slope failures (image from USGS[6]).

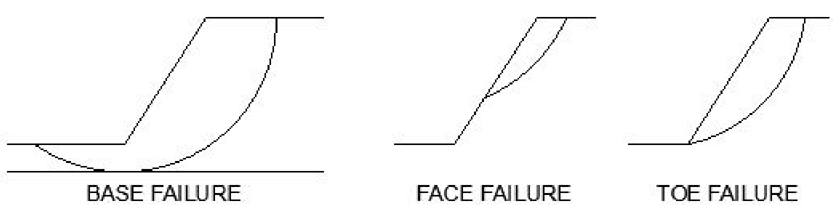

Figure 19. Circular Failure Modes

[6] United States Geological Survey, "Landslide Types and Processes", retrieved August 2, 2022.

10.4 FUTURE EDITIONS

This section of the Geotechnical Design Guide is reserved for future editions of the guide. Slope stabilization and repair is very complicated and often cost prohibitive. Future additional topics may include:

- Guidance on simplified slope stability analyses,
- Slope stabilization alternatives, and
- Retaining wall alternatives when grade changes are necessary.

Slope stabilization and retaining wall alternatives vary by region. If you have any feedback regarding what this section should include, or if you have information about what alternatives are available in your region (include photos!) please send an email to Josiah at Josiah.Baker@emiworld.org.

11 Vertical Pavement Design

11.1 PAVEMENT STRUCTURE OVERVIEW

Pavement structures are the physical infrastructure used to transfer traffic loads (transient and dynamic loads) from vehicles onto the soil subgrade. Pavements are categorized in two ways: *flexible pavements* and *rigid pavements.*

Flexible pavements are surfaced using Hot Mix Asphalt (HMA) and, as the name suggests, are comparatively more flexible than rigid pavements. Flexible pavements deflect to varying degrees with traffic loading. The intent of the asphalt surfacing is less to provide horizontal load transfer (although it does), but to keep aggregate base and subbase intact.

Rigid pavements are surfaced using reinforced Portland Concrete Cement (PCC) and deflect minimally with traffic loads. Rigid pavements distribute the load over a wider area due to steel reinforcement in the pavement structure. Hybrid and alternative pavement systems exist, but it is unlikely that these will be used on an EMI project.

A typical pavement structure consists of:

- Surfacing layer, as described above
- Base and subbase (also referred to as aggregate fill)
- Compacted and uncompacted roadbed soil (also referred to as subgrade outside of pavement design)

Base course material is generally finer than *subbase* material to provide a smooth and uniform layer for the surfacing / wearing course (asphalt or concrete).

The pavement structure can be further improved by use of pavement geotextiles, which aid in load distribution to the subgrade and controlling settlement.

11.2 PICKING A DESIGN ALTERNATIVE

During the master planning and concept design phase of the project, it may be appropriate to begin discussing which pavement structure is more suitable for the project. The client may want EMI to make recommendations for the pavement structure, otherwise, it may be prudent to discuss these design alternatives with the client to see what would best serve the ministry.

Some of the major design considerations when comparing flexible and rigid pavements are listed in the table below. Note, these considerations may not necessarily apply in a local context (e.g., asphalt is not available in the region) so it is important for you to know what is standard construction practice in the region.

Table 7. Comparison of flexible vs. rigid pavements.

Design Consideration	Flexible	Rigid
Material	Asphalt	Reinforced Concrete
Price	Cheaper	More Expensive
Construction Speed	Faster	Slower
Availability After Construction	Shortly after construction	Requires the concrete to cure
Load Distribution	Traffic load is transferred to subgrade exclusively through load distribution in surfacing, base, and subbase layers. Load distribution is significantly less than in rigid pavements.	Traffic load is transferred to subgrade first through steel reinforcement, then by load distribution in base and subbase.
Flexural Strength	Low	High
Thickness	Thicker	Thinner
Performance	10-15 years	30 years
Maintenance	Requires frequent maintenance	Requires minimal maintenance
Repairing Underground Utilities	Easy to remove and replace asphalt	Difficult to repair underground utilities
Joints	Jointless	JRC pavements requires joints, CRC pavements have transverse cracking
Durability	Low, may be damaged by oils and chemicals	High
Pavement Color	Dark, may be difficult to see at night	Light, may glare in the sun

11.3 COMPLETING A PAVEMENT STRUCTURE DESIGN

Once the pavement design alternative has been selected, we can begin the vertical pavement structure design. However, additional information is needed prior to calculating section performance including:

- **Client Expectation:** what does the client expect for performance, pavement lifecycle, and initial and lifetime cost? Some of these considerations can be discussed during preliminary planning for the project site.
- **Load Spectra:** what is the anticipated traffic load on the pavement structure? What is the anticipated traffic growth? Traffic load is a function of vehicle class (size) and frequency, which will vary by country and by site occupancy use. The load is summarized in the U.S. using an Equivalent Single-Axle Load (ESAL), which is a number used to represent the number of 18-kip (80-kN) single-axles that will drive over the pavement across its lifetime. EMI has developed an **ESAL calculator** to help estimate pavement demands for ministry projects.
- **Knowledge of local practice and regulations:** the availability of construction materials, general local practice / expertise, and any government regulations / guidelines may play a big role in the design process. Country specific resources are included later in this chapter.
- **Soil conditions**: the most important soil parameter to know in pavement design is the subgrade resilient modulus (M_r), a measure of the subgrade resistance to permanent vertical deformation. This parameter can be measured directly through laboratory samples (expensive), estimated in the field using a Falling Weight Deflectometer (often not available in developing countries) or correlated to dynamic cone penetration.

  Beyond resilient modulus, we also need to know drainage characteristics, moisture conditions, and other soil characteristics that might impact the design. Drainage is measured by how quickly water will drain or be removed from the subgrade surface and pavement structure. This is affected by the natural water content of the soil, its proximity to groundwater, and seasonal changes to the moisture content.

  The pavement designer should also be aware of high-risk soils that might affect pavement considerations such as expansive soils that shrink / swell significantly, soils prone to frost heaving, and peaty soils.
- **Material Parameters:** layer coefficients for pavement and aggregate material are usually provided by local governments or material providers based on the durability and quality of local sources. These design parameters have likely not been specified by developing governments, so the designer will have to make some design assumptions.

11.3.1 DESIGN METHODOLOGIES

Many municipalities perform pavement structure designs using slightly different methodologies that have been adapted to local conditions to suit local needs. Inputs required for different design methodologies will rely on the same inputs described in the previous section. For most EMI design projects, the AASHTO 1993 “AASHTO Guide for Design of Pavement Structures” is a suitable starting point for most flexible and rigid pavement designs. This document has been the design consensus for many decades and has since been used to develop region-specific design codes.

AASHTO 1993 is an empirical pavement design approach. There are other design methodologies that are referred to as “mechanistic” or “mechanistic-empirical”, but these analyses are too refined for most EMI design projects. Check with local design regulations to verify which design approach is approved for use in your area.

It is important to note that pavement design is a highly iterative process and that there is no “perfect” solution to pavement design. The designer will have to balance cost, performance, surface, base, and subbase thickness, and a variety of other variables before making final design recommendations.

11.4 GRAVEL PARKING LOTS AND LOW VOLUME ROADS

In some cases, because of the low level of anticipated traffic or low performance expectations, full pavement designs are not warranted for a project. A suitable alternative for the project may be a low-cost, unpaved gravel access road (low volume road) or gravel parking lot.

For preliminary design purposes, cost estimation, or when further analyses are deemed unnecessary, design thickness of compacted gravel for a parking lot or low volume road may be selected from Table 8 below. This table has been simplified from the South Dakota DOT (1995) “Rural Road Design, Maintenance, and Rehabilitation Guide” and from AASHTO 1993 Section II-69 “Low Volume Road Design” for regions with no frost conditions.

Table 8. Typical road base (well-graded crushed rock) thickness for parking lots and low volume roads.

Criteria	Gravel Thickness, mm (in)
High subgrade support, low truck traffic	100 (4)
Medium subgrade support, low truck traffic High subgrade support, medium truck traffic	150 (6)
Medium subgrade support, medium truck traffic	200 (8)
Medium subgrade support, high truck traffic	250 (10)
Low subgrade support or very heavy truck traffic	Complete a pavement structural design

12 Geologic Hazards

Before beginning the design process for a project, it is important to first consider geologic hazards and the ways in which they may impact the project scope and final design. Geologic hazards vary significantly by region and by the site's geologic environment. It is helpful to review geologic hazards in the context of local geology through the use of site-specific boreholes and local geologic maps, as summarized in Chapter 15.

12.1 SEISMIC-INDUCED HAZARDS

Seismic-induced hazards are not limited to ground shaking but also include landslides, soil liquefaction, lateral spreading, surface faulting and ground subsidence.

12.1.1 STRONG GROUND MOTIONS

The strong ground motions experienced during an earthquake (or from heavy blasts occurring during combat or near quarries) poses a direct risk to designed structures and may also trigger other seismically induced hazards.

Ground motions are the rapid ground acceleration and deceleration produced by seismic waves as faults and subduction zones slip. The severity of and areas effected due to ground shaking depends on topography, bedrock type and location and orientation of the fault rupture. Damage occurs as waves pass beneath and shake buildings, roads, and other infrastructure.

12.1.2 LANDSLIDES

The ground accelerations due to shaking reduce the stability of inclined rock and soil as strong ground motions introduce destabilizing horizontal and vertical forces, causing rock and soil masses to displace and slide. Even short duration accelerations can be enough to initiate failure. Seismically induced landslides often affect large areas and are difficult to repair and restabilize due to the large volumes of displaced and disturbed material.

Figure 20. Landslide in La Conchita, 1995 (courtesy of USGS)

12.1.3 SOIL LIQUEFACTION AND LATERAL SPREADING

Strong ground motions can cause loose, saturated soil (generally sands, but sometimes poorly-graded gravel and low plasticity silt) to loose particle-to-particle connection because of high, induced porewater pressure, causing the material to behave like liquid. For example, this phenomenon can be observed at the beach where agitating wet beach sand induces liquefaction.

Figure 21. Effects of liquefaction in New Zealand, 2011 (courtesy of Temblor)

Liquefaction typically occurs in unconsolidated, recently deposited sands and silts that are exposed to a high ground-water table. Deeper sands and silts are less prone to liquefaction due to the high overburden pressures induced by the material above. Liquefaction results in large ground deformations, which can damage buried pipe and cause structural damage.

Lateral spreading is the horizontal movement of liquefied material that can occur on relatively steep slopes that would otherwise remain stable in static conditions. Liquefied masses can move rapidly and span great distances, posing a high risk to people and structures.

Figure 22. Liquefaction induced lateral spreading (courtesy of Geomatrix)

Sand boils, another failure mechanism caused by liquefaction, are produced as excess pressure caused by strong ground motions forcibly eject water and sand from below the ground surface up to the surface. Small mounds of ejected sand may appear in flat areas subjected to strong ground motion.

Figure 23. Sand boil exposed at the ground surface (courtesy of USGS)

12.1.4 SURFACE FAULTING

Fault ruptures capable of producing strong ground motions generally result in visible, permanent deformation at the ground surface. This surface faulting may displace the ground surface vertically and laterally as little as a couple of millimeters up to several meters. The surface rupture can damage buried utilities, roads, dams, and structures.

Figure 24. Fault rupture extending to the ground surface (courtesy of newsatlas.com)

12.1.5 GROUND SUBSIDENCE

As ground shaking occurs, soils can densify and settle, leading to subsidence at the ground surface. Damage can occur to roads, utilities and structures including displacement, tilt, stretching, twisting, buckling or any combination.

Figure 25. Ground subsidence in loess deposits (courtesy of [Colorado Geological Survey])

12.2 PROBLEMATIC SOILS: KARST, LOESS, PEAT, ETC.

Certain problematic soils can pose unique challenges to designers due to their unpredictable or unusual behavior. These soils are often hard to control and are often the product of unique geological processes under specific climatic conditions.

12.2.1 LIMESTONE

Limestone formations on a project site are particularly challenging due to the solubility of limestone as it interacts with water. Limestone formations may produce unique geological features such as caves and karst formations with unpredictable characteristics that present difficult challenges for design engineers.

Limestone formations, and more specifically karst landscapes, typically form as a result of water eroding surface material and dissolving subsurface material through holes or groundwater. Karst landscapes include sinkholes, streams, caves, springs, and other features that have been created underground through the interaction of dissolving bedrock and water. Karsts become dangerous if the underground caves and voids collapse, causing sinkholes to form on the surface.

Karstic landscapes are generally associated with limestone, but may also occur in marble and gypsum formations, which are also soluble in water. Karst environments often develop in well-jointed, dense limestone formations near the surface with moderate to heavy rainfall and good groundwater circulation.

12.2.2 LOESS

Loess is fine-grained, silt sediment that has been deposited by the wind (aeolian). Due to the depositional process, these particles are loosely packed and often uniform in size depending on what the wind can typically lift and deposit in the region. The particles are generally pale in color and weak to the touch, crumbling in dry conditions or compressing easily in wet conditions.

Loess deposits are particularly prone to rapid subsidence due to the soil matrix configuration and generally governed by increases in moisture content. In dry conditions, silty loess soils rely on limited suction stress to maintain soil matrix stability, but as the soil moistens, the suction stress decreases, resulting in soil collapse.

12.2.3 PEAT

Peat soils are partially decayed vegetation and organic matter that has accumulated in waterlogged, oxygen deficient conditions with high acidity and nutrient deficiency. New peats generally occur in marshy lowlands, which have very low hydraulic gradients. alluvial/lacustrine

Peat deposits have a high water-holding capacity and can shrink and oxidize rapidly when water is removed. Additionally, its high compressibility and low bearing strength causes extreme settlement and decomposition of the organic fraction causes further subsidence.

As subsurface geology is very complex, do not be surprised to observe peats and other organic-laden soils well below the ground surface. These layers may be distinguished due to their dark color, fibrous texture, and organic odor.

Figure 26. Peat soil sample (courtesy of [David Stanley](#))

12.2.4 EXPANSIVE (REACTIVE) SOIL

Expansive soil deposits are typically rich in clay minerals that cause deposits to expand and contract with changes in moisture content. These soils can also be called reactive soils, as minerals within the soil react chemically to changing moisture content by changing volume. Some varieties of the montmorillonite clay mineral can swell up to 2,000 times their original dry volume when exposed to water. When the expansive soils dry, the loss of water causes them to shrink, producing tension cracks that can be both wide and deep at the peak of the dry season. Several tension cracks may combine to form a large grid or network of cracks and, to the touch, may seem to be quite strong. However, during the rainy season these soils may swell, and the resulting wet soil is often very soft, only capable of supporting small vertical loads.

Shrinking and swelling in fine-grained clay soils occurs because of the large fluctuation in suction stresses as moisture content changes. Dry clay soils have very high suction stresses, resulting in large tensile forces that separate the ground surface with tensile cracks. Moist and wet clays lose these suction stress (often electo-chemical forces), causing particles to expand or move apart. The resulting movement may be enough to induce uplift pressure that can exceed foundation loads imposed by light structures, rods, sidewalks and concrete slabs.

Problems associated with expansive soils are cracking of foundations and buried utilities, heaving and cracking of road surfaces and failure of wastewater disposal systems using infiltrative techniques.

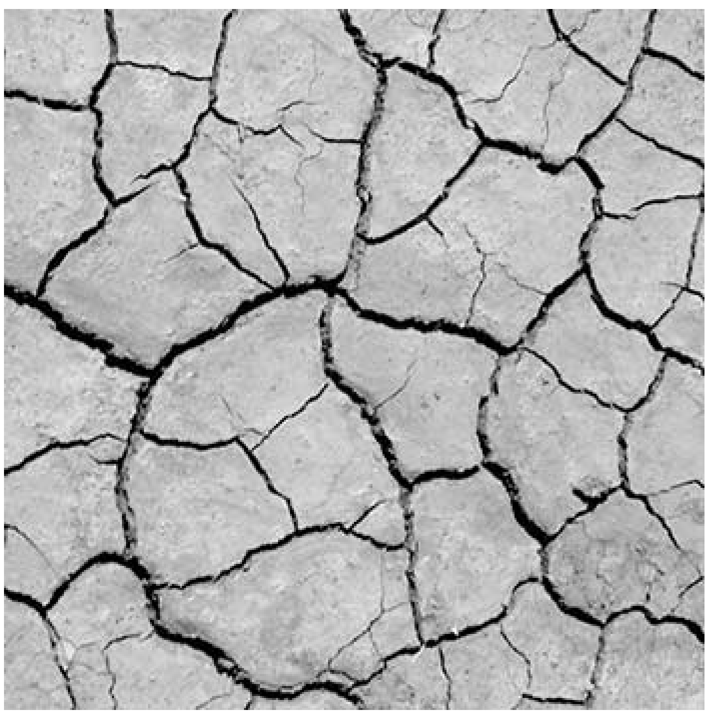

Figure 27. Expansive soil swell/shrinkage strains (Courtesy of WESI Geotechnica).

12.2.5 COLLAPSIBLE SOIL

Collapse-prone soils often occur in loose, dry, low-density deposits that decrease in volume or collapse when saturated for the first-time following deposition. Collapsible soils are geologically young materials such as alluvial-fan and debris flow sediments and wind-blown silts, like loess, with loose, “honeycomb” structure and high dry strength. As shown in Figure 6, the soil structure collapses when saturated leading to ground surface subsidence and differential settlements.

Many collapses are caused by human initiated activities such as the placement of poorly compacted fill, or irrigation, alterations to natural drainage and wastewater disposal on native soil. The increased water weakens the particle bonds and reduces the strength.

12.3 SCOUR

Scour is a specific type of erosion that occurs as sediment or engineered materials are removed from riverbanks. Scour occurs in rivers and channels because of increased water flowrate and intensity. These changes to a channel’s flowrate may be cause by human or natural causes, such as the construction of a new pier or the increased volume of water in a channel after a heavy storm. If the scour is severe, the loss of material can result in river-bank collapse, foundation subsidence and failure and road wash-out.

Figure 28. Scour around bridge pier (courtesy of USGS).

13 Earthwork

To some degree, most EMI projects will involve earthwork, whether for foundation construction, placement of new fill, pond shaping, or massive cut and fill. This section will cover some of the basic issues and considerations related to earthwork, earthwork construction safety as it pertains to vertical cuts and shoring, and erosion control for large earthwork projects.

At a fundamental level, successful earthwork requires an accurate estimation of available, suitable fill on a project site, the amount of fill required, and the amount of material to be imported or disposed of. It requires an understanding of native and compacted characteristics of the local soils and well as the topography of existing ground conditions.

13.1 BASICS OF EARTHWORK

When discussing earthwork, soils are said to "swell" and "shrink" when excavated or compacted from natural conditions as shown in Figure 29. Swell occurs as the soil becomes looser and disturbed when it is cut. Note that in a different context, the term "swell" may refer to fine-grained soils expanding with increases in moisture content rather than due to mechanical loosing of soil.

Similarly, soil "shrinks" when it is compacted. Although this is not always the case, the average unit weight of a native material will generally be greater when it is compacted and looser when it is cut, as shown by the equation below:

$$\gamma_{loose} < \gamma_{natural} < \gamma_{compacted}$$

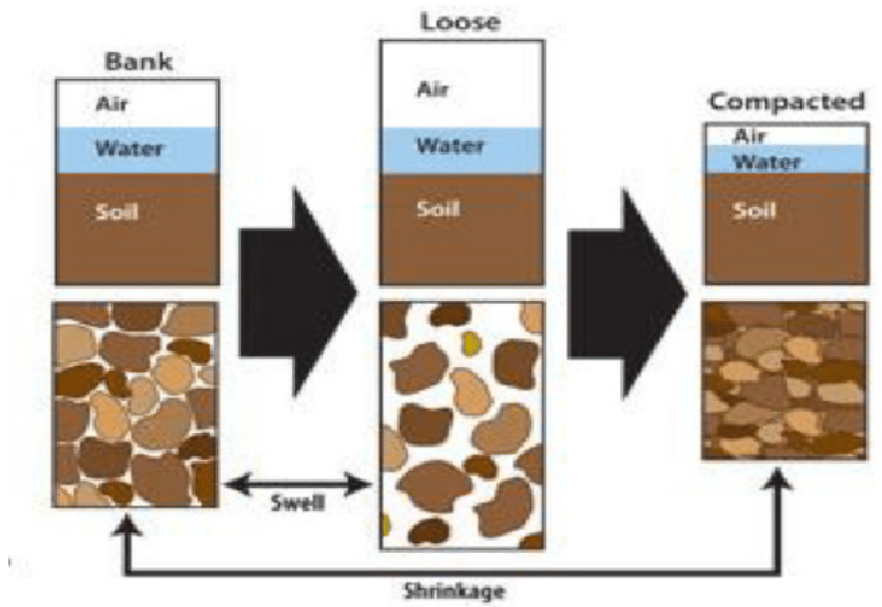

Figure 29. Conceptual diagram of shrinkage / swelling potential in soils.

"Swell" and "shrink" are important concepts for many reasons. Firstly, moving earth costs money and dump trucks have a limited volumetric capacity. Understanding the amount of soil required and number of dump truck trips will lead to more accurately estimating project cost. Secondly, most projects will "lose" soil volume due to compaction of native material. Knowing this "loss" potential will inform how much material will need to be imported.

The volumetric shrink / swell compaction will vary significantly by material type, native conditions, and compaction requirements, so it will be good practice to begin to develop a database of earthwork projects in your local region. A simple way to access this is by counting the number of dump truck runs required to move soil across the site and how many dump truck runs are used to import material. Below are equations used to calculate swell and shrinkage factors. Table 1 presents typical factors.

$$Swell\ Factor = \frac{V_{loose}}{V_{natural}} = \frac{\gamma_{natural}}{\gamma_{loose}}$$

$$Shrink\ Factor = \frac{V_{compacted}}{V_{natural}} = \frac{\gamma_{natural}}{\gamma_{compacted}}$$

Table 9. Conceptual swell / shrink factors for typical USCS soils, modified from Lindeburg "Civil Engineering Reference Manual, 14th ed." section 80-2 and Holtz, Kovacs and Sheahan, 2nd ed." section 2.

Material	Swell Factor	Shrink Factor
Clay	1.40	0.85
Silt	1.25	0.80
Sand	1.12	0.80
Gravel	1.12	0.80
Bedrock	1.60	1.00

13.2 CUT & FILL

Cut areas are defined as the part of the project where the proposed grade is lower than the existing grade. This will require existing soils to be cut, or removed, to achieve the desired elevation. Fill areas are those that require the placement and compaction of native cut or imported fills to raise the grade to the desired elevation. There are many techniques used to estimate cut and fill quantities including the use of software such as AutoCAD Civil3D. It is important to verify results produced using software with hand-calculated estimates as well.

Hand-calculation method will depend on the type of cut, project type (mass grading versus road alignment) and other factors such as the presence of piles or mounds of existing fill on site. For hand-calculations, keep the geometry and assumptions simple as these will produce usable estimates. For example, vertical road alignment can be determined by balancing the cut / fill areas using a profile section, or the volume and mass of a pile of fill can be estimated with a cone or prism.

For large sites, it is important to minimize the distance between cut and fill locations. The greater the distance between the locations is, the longer it will take to transport and construct, increasing earthwork and project costs.

13.3 TEMPORARY CUT SLOPES

Earthwork Safety is of utmost important when providing preliminary planning and final design recommendations on projects involving cut and fill of steep slopes or deep utility trenches. As a starting point use the publicly available US Occupational Safety & Health Administration (OSHA) standard for Excavation Sloping and Benching (1926 Subpart P) for safety recommendations. Additionally, check with your local government to determine what local rules and regulations will apply to earthwork in your area. Some of the most important considerations are summarized in this section, but additional research and planning must be performed.

OSHA classifies soils into four types of soils based on material type and undrained shear strength (s_u).

Table 10. OSHA Soil Classification for Trench Construction.

Category	Description	S_u, kPa (tsf)
Rock	Stable rock that can be excavated with vertical sides	N/A
Type A	Very stiff cohesive or cemented soils	> 144 (1.5)
Type B	Stiff cohesive soils or angular gravel	48 to 144 (0.5 to 1.5)
Type C	Soft to moderately stiff cohesive soils,	< 48 (0.5)

Refer to OSHA 1926 Subpart P Appendix A for full definitions of each soil type. The above definitions are not exhaustive and must be applied with caution. OSHA 1926 also includes special provisions for the following conditions:

- Fissured soils or soils that exhibit tension cracking when dried,
- Soils subjected to vibrations,
- Soils with a planar bed that dips toward the excavation,
- Dry, unstable rock,
- Disturbed or non-native soils,
- Submerged or saturated soils, and
- Other considerations that might weaken the soil.

There are many considerations for benching type, slope steepness, layered material, and lateral bracing requirements in OSHA 1926 Subpart P Appendix B. Overall, the maximum allowable grade for a cut slope (**not to exceed a vertical height of 6m**) for each soil type is summarized below:

Table 11. OSHA Definition of Allowable Slope by Soil and Rock Category

Category	Maximum Allowable Slopes, H:V (°)
Rock	Vertical (90)
Type A	¾H:1V (53)
Type B	1H:1V (45)
Type C	1½H:1V (34)

Permanent slopes cut and fill slopes should be evaluated for long term stability, especially in regions with high seismicity, saturated or variable groundwater conditions, and other geologic hazards that might destabilize the slope. Future editions of the Geotechnical Design Guide will address evaluating long-term stability of slopes in chapter 12.

Temporary protective systems that exceed 6m (20ft) should be designed by a professional engineer with experience specifying protective systems.

If there are cut slope width constraints on the project site or limitations based on soil type, temporary shoring may be necessary to stabilize trenches during utility construction. OSHA provides recommendations for various types of shoring systems including timber, hydraulic jacks, and pre-fabricated trench shields.

13.4 EROSION CONTROL

Heavy rainfall on the site may cause substantial uncontrolled surface erosion and mass earth transport on exposed surfaces if left unaddressed, particularly in sandy conditions. In general, it is best if site earthwork occurs during the dry season, however, rainstorms do occur throughout the dry season and preventative measures should be in place to limit erosion. Best management practices to minimize erosion are described below.

The final design should include specific provisions on where to use and place some of these best management practices in order limit surface erosion, especially near the creek embankment.

13.4.1 NATIVE VEGETATION

Native grasses and other vegetation should be planted immediately after earthwork is complete to establish new vegetation. This can be achieved by applying a mulch mix or native seeds directly on the ground surface. Mulch and seed may have to be reapplied several times after a heavy storm. Vegetation minimizes surface erosion and retains soils deeper below the surface once dense root networks have been established.

13.4.2 HARD SURFACES

Hard surfaces, such as pavements, buildings, and compacted gravel base (as shown in Figure 30), will prevent surface erosion from occurring. Surface runoff should not exit hard surfaces in an uncontrolled manner. Construction of hard surfaces should occur shortly after site grading to limit surface erosion.

Figure 30. Compacted gravel surfaces prevent the erosion or movement of the underlying subgrade.

13.4.3 SURFACE ROUGHENING

If native vegetation cannot be established or if construction is delayed, exposed graded sandy surfaces and slopes should be roughened using heavy equipment. The heavy equipment is used to create small ridges or pockets *perpendicular* to the slope. These ridges should be around 5cm tall to collect any sediment that might otherwise be moved across the surface of the slope.

Figure 31. Roughened surface after soil compaction.

13.4.4 WATTLES & STRAW LOGS

Wattles and straw logs are long rolls that physically prevent the movement of soil across the surface and are generally constructed of biodegradable materials. Wattles and straw logs work well on steep surfaces and should be placed perpendicular to the slope, as shown in Figure 32, to intercept moving sediment, especially where there is a significant change in grade such as at the toe of a slope.

Wattles and straw logs require regular maintenance to ensure sediment does not build up behind the log. Additionally, check for any erosion that might have occurred below or around the barrier.

Figure 32. Wattles placed perpendicular to the slope contours. Note the buildup of sand behind the wattle.

13.4.5 JUTE MESHES / EROSION CONTROL BLANKET

Another method to limit erosion on steep slopes involves installing sheets of biodegradable grids anchored across the face of the slope. Initially, the sheets are intended to physically limit the movement of sediment, but because it is a grid, vegetation can grow through the sheet, establishing permanent erosion control. Jute meshes and erosion control blankets can be used in conjunction with wattle and straw logs, as shown in Figure 33.

Figure 33. Jute mesh used to protect the slope from rainfall and surface runoff.

14 Understanding Fill & Compaction

14.1 INTRODUCTION

For many EMI projects, earthwork on a project site may be substantial. For example, earth may be moved from one side of a site to another, or there may be a need to important structural fill to reach a certain grade. For this reason, it is important to understand the types of fill material available for your project trip, and whether it is sourced on the site itself or imported from elsewhere.

There are many ways to describe construction fill material and many applications for their use. In general, fill can be summarized in three main categories:

1. Structural Fill: used to achieve desired elevation for locations on a project site that will experience significant vertical loading, such as beneath pavements and building. The material type varies depending on the anticipated loading, but in general structural fill is well-graded granular material. Because this fill supports structural elements, it is important that this material is specified and closely monitored for quality. Various types of structural fill are described below.
2. Drainage Rock: used to quickly permit the flow of water through the material. Drainage rock is most often open-graded, crushed rock and is most commonly applied in drainage facilities such as dry wells and French drains.
3. Non-Structural Fill: placed on areas of a project site where long-term consolidation (settlement) of fill material is permissible, such as landscaped areas, fields, or shaped earth. Often, it is desirable for non-structural fill to be highly organic to promote the growth of vegetation.

14.2 TYPES OF STRUCTURAL FILL MATERIAL

Structural fill material can be categorized into native structural fill, imported granular fill and recycled material. Each have specific characteristics that can make them more advantageous to use over others and it is important to understand what type of fill, if any, is on the project site.

14.2.1 NATIVE FILL

Native fill is fill material sourced locally and should be free of any organic / organic-laden soils, roots, uncontrolled construction debris, and other deleterious material. Well-graded sands and most types of gravels will perform well for structural and road base / subbase.

Fine-grained soils, especially high-plasticity silt and clay, may not be suitable for structural fill. These soils have a higher risk of shrinking and swelling, higher frost potential, and may compress significantly under heavy loads. However, these fine-grained soils have other important applications such as fill material for the compacted core of a pond embankment or pond clay-liner.

During a project trip, it may be helpful to identify potential sources of native fill on or in the vicinity of the project site. It is important to identify and select these sources based on their consistency and intended use. For example, if granular material is not available on a project site, the designer must weigh the pros and cons of importing more expensive granular fill versus using less robust and less workable fine-grained soils available on the site.

Native material that is not suitable for structural fill may be used elsewhere on the site for landscaping or general site grading. During construction, locations that receive this non-structural fill should be documented for reference.

14.2.2 IMPORTED GRANULAR FILL

As the project requires, granular fill may be imported to the project site for use as structural or road fill. Granular fill may come from a variety of sources, and aggregate suppliers will usually have different stocks of fill with differing material gradation based on the intend use.

Well-graded quarry rock (igneous) is the highest quality granular material. This rock is obtained by ripping or blasting and has a high percentage of fractured faces, increasing interlocking and shear strength. Other non-igneous rock sources, such as sandstones and siltstones, may be suitable for structural fill, but tend to be “softer” than igneous rock and may more readily weather and degrade.

Granular fill can also be sourced from alluvial material found near rivers and streams, gravel bars, or pits in alluvial valley basins. These fills are generally of lower quality than quarry rock for several reason:

- Individual grains are highly weathered due to river transport and may be rounded or subrounded
- May contain a high percentage of silt depending on the depositional process
- May be poorly-graded, especially in flood prone areas. As flood waters transport sediments, heavier sands and gravels will sink first while silts and clays may be suspended for some time.

It is important that alluvial fills be crushed and graded for use as structural fill. Rounded, poorly-graded sand and gravel should not be used for foundation pad construction and road subbase, but they may be used for drainage solutions.

Imported granular fill reliability may be limited due to the lack of construction quality control. The ASTM (Proctor) procedures for identifying maximum dry density do not cover oversized particles, so if the imported fill contains more than 30% of particles exceeding 19mm (3/4in), the Proctor test does not apply. Coarse granular fill will perform very well, especially as a road subbase material, but it will be difficult to set construction specifications.

14.2.3 RECYCLED MATERIALS

In many parts of the world, it is common to use recycled material like crushed concrete and brick for structural fill or road base / subbase. Due to the highly variable quality of recycled fill we recommend against using recycled material as fill, unless there is strict material quality control, project budget restrictions is a major consideration, or there are low performance requirements.

If recycled fill is chosen, it is important that the material be crushed well and have consistent gradation. Crushed concrete will have better performance than crushed brick or other material. As much as possible, it is best to have one material type. It is difficult to control what the recycled fill is made of, and it is often, repurposed construction debris. At the very least, ensure the recycled material is free from organics, vegetation and fine-grained soils.

14.3 PLACEMENT AND COMPACTION

14.3.1 COMPACTION MACHINES

Types of compaction machines and compaction techniques vary significantly by region. Surface, vibratory compactors are the most suitable solution for earthwork compaction (static rollers may also be a viable solution). Vibratory rollers vary in size, weight, compaction frequency, drum shape and compaction effort, depending on the compacted material.

The smooth drum roller (Figure 34) is the most common vibratory compaction technique and has 100% contact with the compacted surface. The smooth drum roller is suitable for granular fill and some fine-grained material.

Figure 34. Smooth drum roller used to compact crushed rock fill.

For some fine-grained soils, the sheepsfoot roller (Figure 35) is a better compaction solution. The sheepsfoot roller has rectangular protrusions that supply higher compacted loads to a smaller area. The shape of the sheepsfoot roller allow excess moisture to escape between the protrusions. Fine-grained soils are compacted until “walkout” is achieved, the point at which protrusions leave little to no impression in the soil.

Figure 35. Sheepsfoot roller used to compact sandy-silt soil.

Plate compactors (also called jumping jacks, Figure 36) are ideal for smaller applications or limited-access areas. The amount of delivered energy is considerably less than that of heavy drum rollers, requiring more passes and smaller lifts to achieve comparable compaction. Plate compactors can be used behind retaining walls to limit lateral dynamic loading against the face of the wall.

Figure 36. Vibratory plate compactor used to compact gravel fill behind a retaining wall.

Hand compaction tools like tampers are really only suitable for non-structural, low impact construction on well-graded sand such as slabs-on-grade or pedestrian walkways. Flooding a site is not a suitable compaction technique and should not be done as a substitute to achieve desired compaction. In situations of very loose fill, it may cause minor consolidation settlement, but will not be suitable for use as structural support. In other situations, flooding may cause problems with the subgrade. Additionally, allowing a project site with fill to sit fallow for a period of time does little to consolidate the fill, unless accompanied by the placement of surcharge loads.

14.3.2 CONSTRUCTION RECOMMENDATIONS

Earthwork is best performed later in the dry season when subgrade soils have had time to sufficiently dry. Earthwork can be done during rainy and early dry season; however, special attention must be given to the subgrade soils to ensure that soils are dry enough to compact and receive adequate compaction. Avoid compacting wet material as it may destabilize and loosen the soil. Compacting wet soil may also cause softening in sensitive clays or localized liquefaction in poorly-graded sands. In wet soils, compaction and loads from heavy construction equipment may cause fine-grained soils to squeeze out from underneath the equipment instead of densifying (also called “pumping”). For these reasons, it will be important that construction is closely monitored during earthwork.

Compaction is best performed at the optimum moisture content or slightly dry of optimum as compaction machines deliver more energy than laboratory testing of the material. **The optimum moisture content is typically close to the amount of moisture required to clump the soil into a ball with no visible moisture in the soil.** For fine-grained soils, optimum moisture content is usually below the plastic limit, so rolling threads should be difficult. The optimum moisture content is typically determined by the Proctor test described below in section 16.1.1.

For the best results, soil should be added in lifts and compacted to achieve consistent density throughout the fill.

The contractor should maintain separate, designated, well sorted piles of fill, such as one pile for native structural fill, another for landscaping and a another for imported granular fill. Depending on the season, piles should be covered to limit drying from the sun or becoming saturated during heavy rainfall.

It is best to cover recently compacted fill material immediately following placement. Examples include quickly beginning foundation pad construction or applying mulch and seed to re-graded areas. Large areas that will be exposed for long periods should be regularly moistened with a water truck to limit drying and control dust.

During earthwork construction, EMI should have a representative on site frequently to inspect, document technique and fill material, and ensure proper compaction is achieved, especially prior to foundation construction. EMI should also provide recommendations for fill material, compaction requirements, and quality control based on the material that is available to the client and the types of structures being built.

14.4 VERIFYING FILL COMPACTION

For project sites that will receive a significant amount of imported fill or cut/fill earthwork, it is important that compaction and material density is verified using in-situ **testing** rather than **visual inspection**. Material density is verified by comparing the in-situ density to laboratory determinations for maximum dry density as produced from standardized laboratory testing. Equipment to both determine the maximum dry density using the modified Proctor test (ASTM D1557) and in-situ verification of density using the Sand Cone method (ASTM D1556) are available to each EMI office.

14.4.1 MAXIMUM DRY DENSITY

The most common laboratory test for identifying maximum dry density is the Proctor test. There are two types of Proctor tests, Standard and Modified, which vary in the amount of applied dynamic load. For both tests, soil is placed and compacted in a metal mold using a calibrated hammer as shown in Figure 37.

Figure 37. Proctor Test Equipment - Hammer and Mold

The soil is compacted at several moisture contents which are plotted as shown on Figure 38. The maximum dry density is the highest point on the curve and the optimum moisture content is the accompanying moisture content at which it was compacted. The maximum dry density determined by the Proctor test is not the theoretical maximum compaction for a given soil and it is not unusual for in-situ density to exceed laboratory values based on compaction technique.

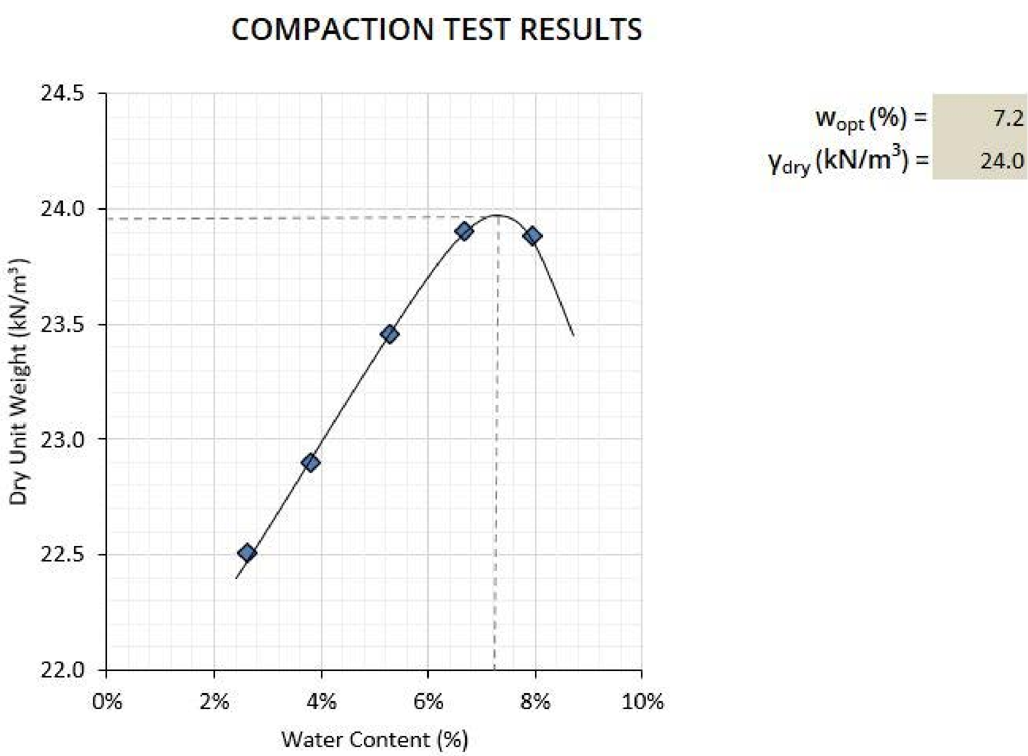

Figure 38. Proctor Test Results

14.4.2 IN-SITU TESTING

There are several methods for determining in-situ dry density and are categorized as either *destructive* or *non-destructive* techniques. Destructive techniques, as the name implies, are in-situ testing techniques that “destroy” a compacted surface. Destructive techniques work by physically removing a certain volume of fill and weighing the fill to determine the density. The technique to determine the in-situ volume depends on the type of test. Among these are the Sand Cone test, which uses calibrated sand to fill a hole to determine volume, and the Balloon Test, which uses a calibrated balloon attached to a pump to determine volume. To determine weight, destructive techniques require the fill material to be dried in order to determine moisture content, dry weight and dry density. This can take several hours to a day depending on the material. However, there are some approved provisions in the ASTM standard to quicken drying using a microwave.

The most common non-destructive technique is the *nuclear density gauge*. It offers several advantages over destructive techniques:

- Does not disturb compacted fill (although the disturbed area is often negligibly small and easy to replace),
- Easy to conduct,
- Instant results.

The main drawback of the nuclear density test is the high initial cost to purchase the equipment. The nuclear gauge itself also poses some risk to the user if not handled properly and may be subject to government regulation.

For most EMI projects, the Sand Cone test may be the most practical and economical in-situ solution, especially for smaller projects. The balloon test is not currently endorsed by ASTM (although the equipment is still available for purchase). The Sand Cone test additionally requires a supply of uniformly-graded, rounded sand.

15 Geology

15.1 INTRODUCTION

Other than human manipulation on a project site, geology alone is *the* controlling variable for subsurface soil conditions. Geology is a broad term that describes the study of the physical earth and how different features of the earth are affected by different depositional and erosional processes, plate tectonics, hydrogeology, and geomorphology to name a few!

Rather than describe all the possible types of rocks and surface geologies one might expect at a project site, the Geotech Design Guide will briefly describe the geologic setting for each EMI office. The design guide will also include a few region-specific geologic maps and geology resources that may be useful for future project trips. If you, the reader, come across any publicly available geological or soil references that you think should be included in future editions of this design guide, please email Josiah.Baker@emiworld.org.

For more information about the geological dating process referenced here, please refer to the following publications produced by the United States Geological Survey:

Divisions of Geologic Time (2010)

The designer should also be aware of projects in areas of high seismicity. Strong ground motions could pose a risk to project sites and facilities. Local design requirements may also be different if the site is in a high seismicity region. Be sure to check local requirements for your projects.

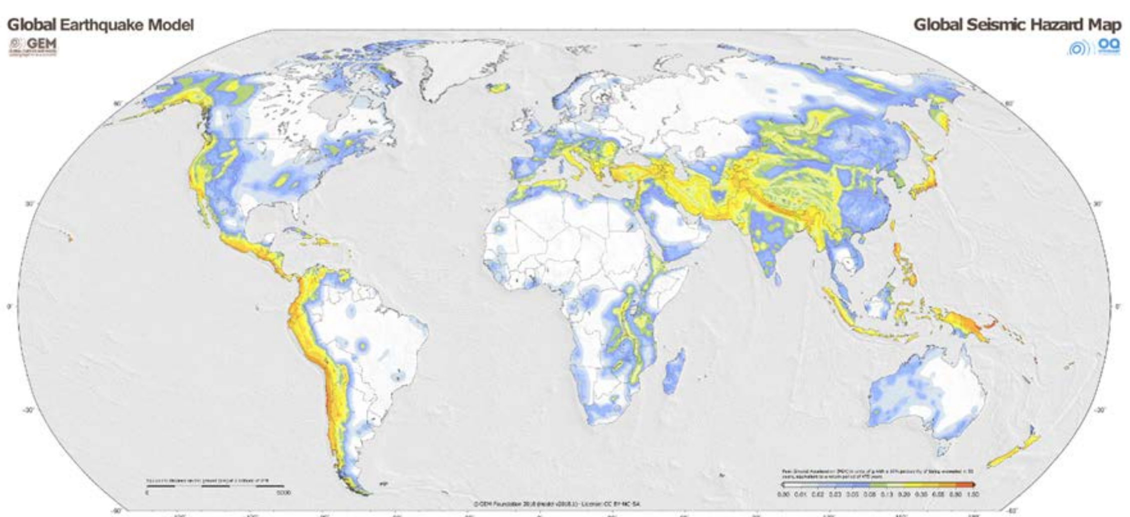

Figure 39. Global seismic hazard map showing peak ground acceleration (PGA) for an event having a 10% probability of being exceeded in 50 years (return period of 475 years).

Figure 39 is a global seismic hazard map[7] and may be helpful in determining if your project is in a seismically active region.

15.2 REGION-SPECIFIC GEOLOGY RESOURCES

CANADA OFFICE

Located near the foothills of the Rocky Mountains, the Canada office, in Calgary, Canada, is towered by the sandstones, mudstones, and shale mountains of the Brazeau formation and Alberta group (Mesozoic, Upper Cretaceous). These dominating mountains have been carved over time through glacial erosion to form the stunning displays at Banff and Jasper.

Calgary itself is in the Canadian Interior plains, with bedrock geology consisting of Sandstones and mudstones of the Porcupine Hills formation that dip toward the west (Tertiary, Paleocene).

The office is located within the lower terrace of the Bow River and the surface geology may consist of alluvial deposits eroded from the Rocky Mountains (Quaternary).

Geological Map of Alberta Canada (1999)

Surficial Geology of Alberta (2013)

Geologic Map of North America (2005)

CAMBODIA OFFICE

Most of the Kingdom of Cambodia is located in the heart of the Tonle Sap-Phnom Penh basin, including its capital Phnom Penh. This basin is home to the Tonle-Sap Lake, the biggest freshwater lake in Southeast Asia. Plate tectonic activity in Cambodia is rather subdued when compared to its neighbors, resulting in a relatively stable and shallow basin with little topographic relief.

Phnom Penh is located at the confluence of the Mekong and Tonle Sap Rivers. Given the flat nature of Cambodia and frequency of flooding of these rivers, the surficial geology consists of alluvium (Quaternary) for a large swath of land surrounding the bodies of water.

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[7] Global Earthquake Model (GEM), "Global Earthquake Hazard Map" (2018), accessed August 2022 at https://www.globalquakemodel.org/gem-maps/global-earthquake-hazard-map.

Mapped bedrock geology is limited in the area, but likely consists of sedimentary sandstones and siltstones as is typical near the area.

General Soil Map of Cambodia (1963)

Geological Map of Vietnam, Cambodia and Laos (1971)

INDIA OFFICE

The greater geologic context for South Asia is the collision of two large continental plates, the Indian Plate and the Eurasian Plate. Because these two plates have similar densities, the Indian Plate does not readily subduct underneath the Eurasian plate, resulting in the massive and tall Himalayan range.

Most of Northern India, beginning near Delhi and continuing toward Bangladesh, is located near the edge of this subduction zone, referred to as the Himalayan foreland basin, which forms the greater Ganges River watershed. This basin is the result of the Indian Plate subducting, or being pushed down, by the Eurasian Plate.

Near Delhi, surface geology consists of alluvium (Quaternary) deposited by the Yamuna River and other nearby tributaries to the north, and aeolian deposits (Quaternary) to the south and west, closer to the drier Indus Valley. Some Feldspathic sedimentary rock (Cambrian or Proterozoic) may be exposed or near the ground surface in southern Delhi.

Geology Map of Haryana

Geological Survey of India – Northern Region (2012)

Soil Regions – Northern Region (1981)

Geologic Map of South Asia (1997)

MENA OFFICE

The MENA region hosts some incredible geologic features, including:

- the Arabian Plate, a minor plate which separates the African and Eurasian plates, and the location of most Arabic countries,
- the Red Sea Rift, created by plate separation and sea floor spreading,
- numerous basins in the Arabian/Persian sea, containing some of the world's largest oil reserves,
- the Nile, Tigris, and Euphrates rivers which form the "cradle of civilization" hold incredible geological significance and provide freshwater to the region,
- the Dead Sea Rift valley is the lowest land elevation on earth,
- the Sahara Desert, comprised of limestone and sandstone (Mesozoic) which dominates geology in much of North Africa

Soil Map of Syria & Lebanon (1985)

Geologic Map of Egypt (1981)

Geologic Map of Israel (2014)

Geologic Map of the Arabian Peninsula (1997)

Geologic Map of Africa (1997)

MEXICO OFFICE

The Baja California peninsula was once a part of the North American continental plate that has since broken away and is now being carried and moved by the Pacific plate. This has resulted in a massive system of transform faults in the Gulf of California to the east.

The area surrounding Ensenada is mountains and rugged, consisting of sedimentary conglomerates (Tertiary), as well as intrusive andesitic and granitic rock (Jurassic-Cretaceous) related to the Sierra-Nevada range. Ensenada itself is situated in a small basin formed by minor regional faults.

Carta Geológico-Minera Ensenada (1997)

Geologic Map of North America (2005)

NICARAGUA OFFICE

The volcanic mountain ranges of Central American and Nicaragua were formed by the subduction of the Coco plate below the Caribbean plate at the Middle America Trench, off the west coast. This region is still very seismically (and volcanically) active, frequently having earthquakes exceeding $M_w > 6.0$.

Managua is situated in a unique depression called the Choco Pacific Basin that roughly parallels the Middle America Trench. This depression forms the Gulf of Fonseca to the northwest and Lake Managua and Nicaragua surrounding Managua.

In the vicinity of Managua, this basin is filled by volcanic flows, tuffs, and other volcaniclastic material (Quaternary) as well as other alluvial material (Quaternary) near Lake Managua.

Geologic Map of the Caribbean Region (1997)

Geologic Map of North America (2005)

SENEGAL OFFICE

Dakar, Senegal on the Cape Verde Peninsula is the western most location on continental Africa. The peninsula is geologically unique because it juts out from the rest of continental West Africa over100km.

Dakar, the western-most region of Cape Verde, shields the rest of the peninsula. It is formed by hard basaltic rock (Paleogene), which is not seen elsewhere in Western

Africa. These rocks originate from a volcanic field whose origins the science community has yet to fully understand.

The rest of the peninsula is bridged by calcareous marls and limestones (Eocene), as well as some minor dune deposits (Quaternaries). This land bridge is lined with several north-south trending faults.

Because of its unique geology, no significant freshwater bodies exist near Dakar. Difficulties meeting domestic, agricultural, and residential water demands for the city will persist well into the future.

Geologic Map of Senegal (2009)

Geologic Map of Africa (1997)

UGANDA OFFICE

Uganda sits at the center of the East African Rift System, which is composed of the Albertine Rift system to the west, and the Eastern Rift Valley to the east. The origin of these rifts is not well understood, but they play a significant role in the region. The diverging plates create distinct rift basins extending for thousands of kilometers.

Lake Victoria, one of the African Great Lakes and Africa's largest lake by area is located directly to the south of the capital Kampala. The rifts do not directly intersect here and there is no noticeable faulting in the area.

Bedrock geology of Kampala's gently undulating surface is ancient rock, consisting of mixed metaphoric quartzites, slates, and gneiss (pre-Cambrian). Locally, this rock is weathered into fine-grained silts and clays derived from the underlying parent material.

Geologic Map Uganda (1962)

Soil Map of Uganda (2001)

Geologic Map of Africa (1997)

UNITED KINGDOM OFFICE

The geology of the United Kingdom is known for its complexity and diversity and was formed through continental collisions over the ages. For example, rocks in northwest Scotland date back to the Hadean eon, which may be as old as the earth itself, while other formations in England were formed in the current eon (Quaternary). In Wales, there are many formations that date to the Cambrian eon (first complex life forms).

Within the greater London area, bedrock geology consists of a variety of sedimentary rock much of which is chalk and claystone. The age, source and origin of rocks in the London area is also quite diverse.

Geologic Map of Europe (1997)

US & GLOBAL OFFICE

Nestled in the foothills of the Rocky Mountains, the US & Global Office are located at the boundary of the Rocky Mountains and the Great Plains. The exact geologic formation of the Rocky Mountains is not yet well understood.

Many of the rock formations that build up the Rocky Mountains are pre-Cambrian. Surficial deposits in the Colorado Springs area are diverse, consisting in part of aeolian (wind) deposits, alluvium (river), and colluvium (gravity), all of which are young (Quaternary).

Perhaps one of the most famous rock formations in the area is the Garden of the Gods, which contains a deep bed of windswept, vertical sedimentary rock, which is uncharacteristic for the predominantly metamorphic Rocky Mountains.

Surface Geologic Map of the Pueblo Quadrangle (2002)

Geologic Map of North America (2005)

SOUTH AFRICA OFFICE

Geology in South Africa consists of two distinct and geologically complex regions: the Karoo supergroup, consisting of the Central Plateau and Lesotho Highlands, and the Cape supergroup, forming the mountain coastal regions to the south and west including Cape Town. South Africa is rich in rare earth and minerals.

The Cape Fold Belt was formed by orogenic movement of the Cape supergroup with the Karoo supergroup. The Cape Fold Belt consists predominantly of sedimentary shales and sandstones (Paleozoic). Along the west and southwest coast, there are massive, older granitic pluton intrusions (pre-Cambrian) that may have been exposed due to orogenic movement.

The EMI South Africa office is located north of the Table Mountain National Park which consists predominantly of uplifted sedimentary rock. Surface geology near the office consists of undifferentiated recent deposits (Tertiary to Quaternary), but it is likely colluvium and aeolian in nature.

Geologic Map of South Africa (2003) Plate 1 **Plate 2** **Plate 3** **Plate 4**

Geologic Map of Africa (1997)

Appendix A: Phase Relationships

Symbol	Dimension	Units	Description
s, d	-	-	Abbreviation: Solid, Dry
w	-	-	Abbreviation: Water
a	-	-	Abbreviation: Air
v	-	-	Abbreviation: Voids = Air + Water
sat	-	-	Abbreviation: Saturated = Solid + Water
t	-	-	Abbreviation: Total = Solid + Water + Air
$'$	-	-	Abbreviation: Buoyant
e	-	unitless	Void Ratio
g	LT^{-2}	m/s^2	Acceleration of Gravity
M	M	Kg	Mass
n	-	%	Porosity
S	-	%	Saturation
V	L^3	m^3	Volume
W	W	kN	Weight
w	-	%	Water Content
γ	WL^{-3}	kN/m^3	Unit Weight
ρ	ML^{-3}	kg/m^3	Density

Void Ratio $e = \frac{V_v}{V_s}$ $e = \frac{n}{1-n}$

Porosity $n = \frac{V_v}{V_t}$ $n = \frac{e}{1+e}$

Degree of Saturation $S = \frac{V_w}{V_v} \times 100$

Water Content $w = \frac{M_w}{M_s} \times 100$

Density $\rho_d = \frac{M_s}{V_t}$ $\rho_{\text{sat}} = \frac{M_s+M_w}{V_t}$ (for S = 100%) $\rho' = \rho_{\text{sat}} - \rho_w$

Unit Weight $\gamma = \rho g = \frac{W}{V}$ $\gamma' = \gamma_{\text{sat}} - \gamma_w$

Appendix B: Example Boring Log

See following three pages for an example boring log legend that can be appended to the end of soil boring logs.

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BORING LOG LEGEND
UNITED SOIL CLASSIFICATION SYSTEM (USCS) CHART

MAJOR DIVISION			GROUP SYMBOL	GROUP LETTER	GROUP NAME
Coarse-Grained Soils Contains less than 50% fines	**Gravel and Gravelly Soil** More than 50% of coarse fraction retained on the No. 4 sieve	Gravel with ≤ 5% fines		GW	Well graded GRAVEL
				GP	Poorly graded GRAVEL
		Gravel with between 5% and 12% fines		GW-GM	Well graded GRAVEL with silt
				GW-GC	Well graded GRAVEL with clay
				GP-GM	Poorly graded GRAVEL with silt
				GP-GC	Poorly graded GRAVEL with clay
		Gravel with ≥ 12% fines		GM	Silty GRAVEL
				GC	Clayey GRAVEL
				GC-GM	Silty, clayey GRAVEL
	Sandy and Sandy Soil More than 50% of coarse fraction pass the No. 4 sieve	Sand with ≤ 5% fines		SW	Well-graded SAND
				SP	Poorly graded SAND
		Sand with between 5% and 12% fines		SW-SM	Well graded SAND with silt
				SW-SC	Well graded SAND with clay
				SP-SM	Poorly graded SAND with silt
				SP-SC	Poorly graded SAND with clay
		Sand with ≥ 12% fines		SM	Silty SAND
				SC	Clayey SAND
				SC-SM	Silty, clayey SAND
Fine-Grained Soils Contains more than 50% files	**Silt and Clay**	Liquid Limit < 50		CL	Low plasticity (lean) CLAY
				CL-ML	Low plasticity silty CLAY
				ML	Low plasticity SILT
				OL	Low-plasticity ORGANIC silt or clay
		Liquid Limit ≥ 50		CH	High plasticity (fat) CLAY
				MH	High plasticity (elastic) SILT
				OH	High plasticity ORGANIC silt or clay
Highly organic soils				PT	Peat, fibrous organics
Fill				FILL	Non-native FILL soil and granular FILL
Bedrock				BR	BEDROCK

NOTES:

1. Unless otherwise noted, soil group was identified using field and sample observations according to ASTM D2488, *Visual-Manual Procedures for Soil Identification.* When additional laboratory data are available, soil classification is confirmed according to ASTM D2478.
2. Fines are material that passes the No. 200 sieve.

BORING LOG LEGEND
SOIL CLASSIFICATION GUIDE

Soil Description: *group name with group name modifier (group symbol)* – density or consistency if intact or if aided with in-situ testing; particle size range; moisture condition; plasticity of fines; color in moist condition; cementation; structure; geologic interpretation; additional comments (such as root structure, borehole stability, whether it is fill, odor if organic or contaminated, Atterberg Limit results for fine-grained soil, percent fines / sand / gravel for coarse-grained soil, reaction to HCl, maximum particle size, percent of cobbles and/or boulders).

Ex. *High plasticity silt with trace sand (MH)* – stiff; wet; gray; residual soil from siltstone of the Nye Formation. Thin root structure within top 10cm of retrieved sample.
Well-graded gravelly sand with silt (SW-SM) – moderately dense; moist; brown; fine to coarse sand, fine gravel; lightly cemented. Some woody debris, slight organic odor.

Moisture Description

Description	Criteria
Dry	Absence of moisture, dusty or dry to touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is near or below groundwater surface

Group Modifier

Description	Criteria
With Trace	Less than 5%
With Some	Between 5 and 30%
“Sandy” or “Gravelly”	More than 30%

Consistency and Density

Relative Density (Coarse-Grained)	N_{60}	Consistency (Fine-Grained)	N_{60}	s_u (kg/cm^2)	Description
Very Loose	0 – 3	Very Soft	0 – 1	< 0.125	Readily indented by the thumb
Loose	4 – 9	Soft	2 – 4	0.125 – 0.25	Thumb will penetrate soil about 25mm (1in)
Medium Dense	10 – 29	Medium Stiff	5 – 8	0.25 – 0.50	Thumb will indent soil about 6mm (1/4in)
Dense	30 – 49	Stiff	9 – 15	0.50 – 1.0	Thumb will not indent soil, but readily with thumbnail
Very Dense	50 +	Very Stiff	16 – 30	1.0 – 2.0	Thumbnail will not indent soil
		Hard	31 – 60	> 2.0	

NOTES:

1. Soil description was identified using modified procedures from ASTM D2488, *Visual-Manual Procedures for Soil Identification.*
2. Fines are material that passes the No. 200 sieve.

eMi designing a world of hope	**BORING LOG LEGEND** **SOIL CLASSIFICATION GUIDE**	

Reaction to HCl

Description	Criteria
None	No visible reaction to HCl
Weak	Some reaction, bubbles forming slowly
Strong	Violent reaction with bubbles forming immediately

Plasticity

Description	Criteria
Non-plastic	A 3mm (1/8in) thread cannot be rolled at any water content
Low	Thread can barely be rolled and lump cannot be formed when drier than the plastic limit
Medium	Thread is easily rolled and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. Lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. Thread can be rerolled several times after reaching the plastic limit. Lump can be formed without crumbling when drier than the plastic limit.

Rock Strength

Description	Criteria
Very Weak (R1)	Crumbles under firm blow with point of geologic hammer, can be peeled by a pocket knife
Weak (R2)	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geologic hammer
Medium Strong (R3)	Cannot be scraped by a pocket knife, specimen fractures with single blow of geologic hammer
Strong (R4)	Specimen requires more than one blow to fracture
Very Strong (R5)	Specimen requires many blows of geologic hammer to fracture
Extremely Strong (R6)	Specimen can only be chipped with geological hammer

Particle Shape

Description	Criteria
Flat	Particles with width / thickness > 3
Elongated	Particles with length / width > 3
Flat and elongated	Particles meet criteria for both flat and elongated

Cementation

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

Structure Description

Description	Criteria
Stratified	Alternating layers of varying material or color with layers at least 6mm (1/4in) thick
Laminated	Alternating layers of varying materials or colors with the layers less than 6mm (1/4in) thick
Fissured	Breaks along definite planes with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps without further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mas of clay
Homogeneous	Same color and appearance throughout

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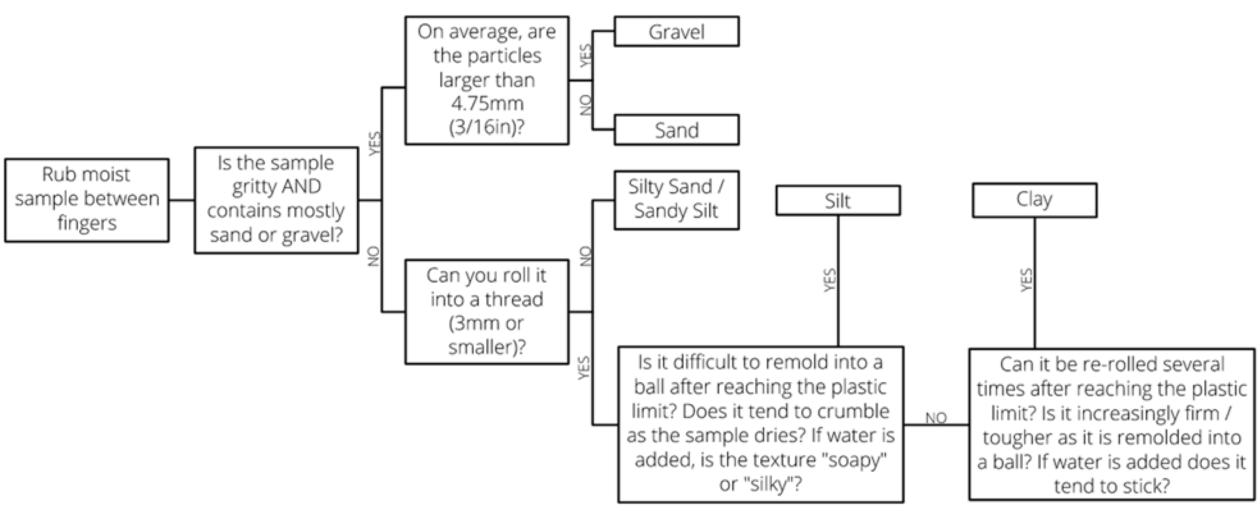
BORING LOG LEGEND
SOIL CLASSIFICATION

Color Guide

Soil Classification Flow Chart



Appendix C: General Desk-Study Resources

GENERAL RESOURCES

- Google Earth, especially historical aerial photography and measure tools
- Google Maps, especially topographic view
- USGS Earthquake Catalog
- USGS Store:
  - USGS provides current and historic government maps from around the world that are permitted for free distribution and large printing to the public
- USGS National Map:
  - This tool features all of USGS's publicly available geographical maps and data including topographic maps, GIS and elevation data (LIDAR), watersheds, hydrography, etc.
- QGIS:
  - This open-source software includes may publicly available plugins to download and view terrain data including:
    - The Mapzen Global Terrain tile,
    - Searching 'terrain' in the QMS plugin yields many useful elevation datasets

REGION SPECIFIC RESOURCES

North America

- National Geologic Map Database
- USGS Quaternary Fault Map
- USGS Topographic Maps
- USGS Hazard Maps
- NRCS Soil Survey Map:
  - This is the USDA's complete database of soil web surveys. The survey is generally intended for agriculture purposes, but it gives a rough overview of shallow subsurface ground conditions.

Appendix D: Region Specific Guidelines

This section of the Appendix is reserved for region specific information created in collaboration with each office based on their observations and experience regarding local geography and construction practices.