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Innovation in Transportation

DEVELOPMENT OF SUBGRADE STABILIZATION PROTOCOL

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16. Abstract Climatic factors and traffic loads are the primary driving forces of instability in subgrade soils. Seasonal precipitation and temperature changes are responsible for fluctuations in soil water content, which can in turn lead to volumetric changes in the soil and associated changes in subgrade performance. In extreme cases, fluctuations in post-construction water contents can result in permanent deformations that can render recently built pavements unsafe and inadequate to service traffic demands. When mechanical performance issues associated to subgrade instability become apparent during construction, corrective actions can be taken without the need for pavement demolition and reconstruction. However, in many cases issues manifest themselves several months after construction, when the environmental triggers reappear. The costs associated to fixing a road already in service increase dramatically, which is aggravated by the fact that the department is left to finance the corrective actions once it takes ownership from building contractors. The broad objective of this research project is to develop subgrade improvement guidelines for selection, design and construction. The combined guidelines are expected to enable geotechnical, pavement, materials, design, and construction engineers and project managers to apply the most cost-effective solutions available for the various subgrade conditions encountered on projects across the state. The establishment of subgrade improvement guidelines and specifications is expected to eliminate preventable subgrade failures; thus, reducing the associated construction and maintenance costs, and providing better and safer roads for the public.			
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PREFACE

Climatic factors and traffic loads are the primary driving forces of instability in subgrade soils. Seasonal precipitation and temperature changes are responsible for fluctuations in soil water content, which can in turn lead to volumetric changes in the soil and associated changes in subgrade performance. In extreme cases, fluctuations in post-construction water contents can result in permanent deformations that can render recently built pavements unsafe and inadequate to service traffic demands. When mechanical performance issues associated to subgrade instability become apparent during construction, corrective actions can be taken without the need for pavement demolition and reconstruction. However, in many cases issues manifest themselves several months after construction, when the environmental triggers reappear. The costs associated to fixing a road already in service increase dramatically, which is aggravated by the fact that the department is left to finance the corrective actions once it takes ownership from building contractors. The broad objective of this research project is to develop subgrade improvement guidelines for selection, design and construction. The combined guidelines are expected to enable geotechnical, pavement, materials, design, and construction engineers and project managers to apply the most cost-effective solutions available for the various subgrade conditions encountered on projects across the state. The establishment of subgrade improvement guidelines and specifications is expected to eliminate preventable subgrade failures; thus, reducing the associated construction and maintenance costs, and providing better and safer roads for the public.

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1. INTRODUCTION

Climatic factors are to a great extent the driving forces of instability in subgrade soils. Seasonal precipitation and temperature changes are responsible for fluctuations in soil water content, which can in turn lead to volumetric changes in the soil and associated changes in subgrade performance. In extreme cases (e.g., expansive and lime-treated sulfate bearing soils) fluctuations in post-construction water contents can result in permanent deformations that can render recently built pavements unsafe and inadequate to service traffic demands. When mechanical performance issues associated to subgrade instability become apparent during construction, corrective actions can be taken without the need for pavement demolition and reconstruction. However, in many cases issues manifest themselves several months after construction, when the environmental triggers reappear. The costs associated to fixing a road already in service increase dramatically, which is aggravated by the fact that the department is left to finance the corrective actions once it takes ownership from building contractors.

The objective of this report is to compile relevant available information on mechanical and chemical subgrade improvement. This information will serve as the basis for the development of selection, design, and construction guides intended to improve the performance and cost effectiveness of subgrade improvement projects across the state. The establishment of the guidelines and specifications is expected to eliminate preventable subgrade failures; thus, reducing the associated construction and maintenance costs, and providing better and safer roads for the public.

The introductory chapter provides a review of the state climate for context. Climate is not only the main trigger of subgrade instability, but also a key factor influencing construction procedures relevant to specific treatment options. Thus, this review offers a context for the literature review, and a base for the development of guidelines and specifications. The second chapter provides a review of project investigation practices. The third chapter covers mechanical and chemical stabilization methods in detail. The fourth chapter offers a series of recommendations to speed up implementation of best practices in the state. Chapters five, six and seven translate the available body of knowledge into state specific guidelines for selection, design, and construction of subgrade stabilization projects in New Mexico.

1.1 NEW MEXICO CLIMATE

New Mexico, the fifth largest state in the union, spans 121,412 square miles, most of which lies between latitudes 32° and 37° and longitudes 103° and 109° W, see Figure 1. The average elevation in the state is 4,700 feet (max 13,161 ft; min 2,870 ft). Most of the state is characterized by a mild, arid or semi-arid continental climate, with the exception of high mountains which exhibit a climate common to the Rocky Mountains (WRCC 2014). The state climate is divided into eight divisions based on topographic features: (1) the northwestern plateau, (2) the northern mountains, (3) the northeastern plateau, (4) the southwestern mountains, (5) the central valley, (6) the central highlands, (7) the southeastern plains, and (8) the southern desert (NOAA 2014), see Figure 1.

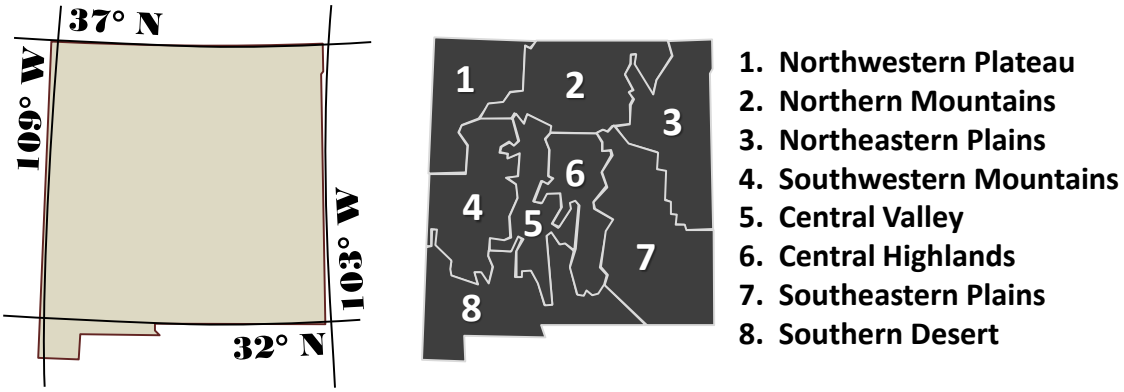


Figure 1. Geographic location and climate units within the state of New Mexico (NMSU 2014).

Mean annual temperatures across the state range from 64 °F (climate unit 7) to 40 °F or lower in the northern mountains (climate unit 2). Within the state, elevation is more important than latitude in determining temperature. While the difference in mean annual temperature between two locations at the same elevation, one in the extreme southwest and one in the extreme northeast, is merely 3 °F, the change between two points 15 miles apart with a difference in elevation of 4,700 feet is 16 °F (WRCC 2014). During the summer months daytime temperatures often exceed 100 °F at elevations below 5,000 ft (climate unit 8). The highest temperature in record is 116 °F (Orogrande 7/14/1934; Artesia 6/29/1918). The coldest month is January with average daily temperatures in the 50's at lower elevations and in the 30's at higher elevations. The record low is -50 °F (Gavilan 2/1/1951). The average range in daily temperature is 25 to 35°F. Figure 2 shows the mean monthly temperature in the different climate units between May 2011 and March 2013. Units 3, 5, 8, and 7 exhibit the highest mean monthly temperatures during the summer season whereas units 1, 2, 4, and 6 exhibit the lowest mean monthly temperatures during the winter season.

The State's main topographic features include mesas, mountain ranges, canyons, valleys, and dry arroyos. Scant rains and snows originate from sources of moisture in the Pacific Ocean and the Gulf of Mexico. Precipitation totals vary widely across the state and over time; from less than 10 inches in the southern dessert to more than 20 inches at higher elevations (WRCC 2014). Summer rains typically come in the form of brief but intense thunder storms that can release significant volumes of water over concentrated areas. Because of the terrain and the lack of vegetative cover, run offs from these storms frequently result in local flash floods. Run off converges at arroyos posing a threat to motorists and infrastructure where these meet roadways and bridges. 30 to 40% of the year's total precipitation falls between July and August, except in the San Juan Valley which receives only 25% of its annual precipitation during these months (WRCC 2014). Winter precipitation most often comes in the form of snow in the mountain areas, but it may also occur as either rain or snow in the valleys. Average annual snowfall ranges from less than 3 inches at the Southern Desert and Southeastern Plains to over 100 inches at Northern Mountains (WRCC 2014).

The relative humidity across the state ranges from an average of 65% at sunrise to near 30% in the mid afternoon. In warmer months, the relative humidity often drops to less than 20% and occasionally border 4% (WRCC 2014). Lower mountain temperatures result in higher average relative humidity compared to the valleys. The potential evaporation in the state is much greater than the average annual precipitation. Potential evaporation is the amount of water (in inches) that could be evaporated from surface sources if these were available. Between May and

October (the period of precipitation) evaporation ranges from 41 inches in the north central region to 73 inches in the southeast areas of the state (WRCC 2014). Therefore, the expected residence time of surface water following precipitation events is rather short.

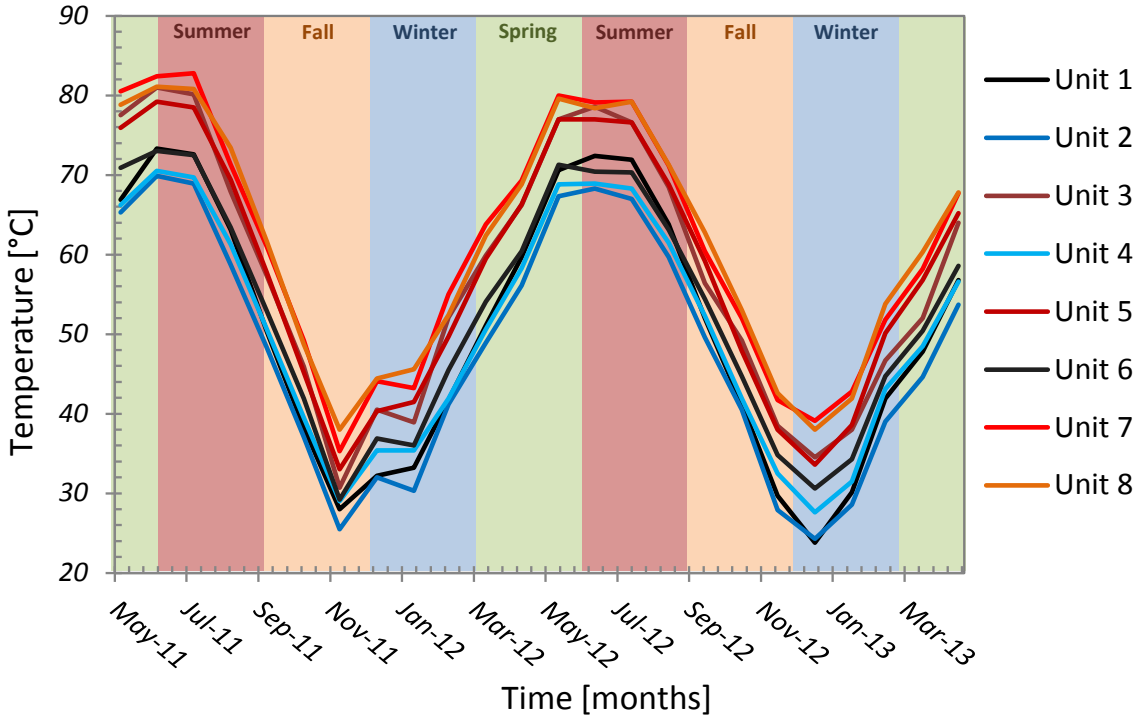


Figure 2. Mean monthly temperatures for the period between May 2011 and March 2013 (NOAA 2014).

Superimposing the district boundaries map and the climate units map, the following can be observed: (1) Districts 5 and 6 overlap by climatic units that exhibit the low winter temperatures, (2) the rest of the Districts include areas of high summer temperatures and low winter temperatures. See Figure 3.

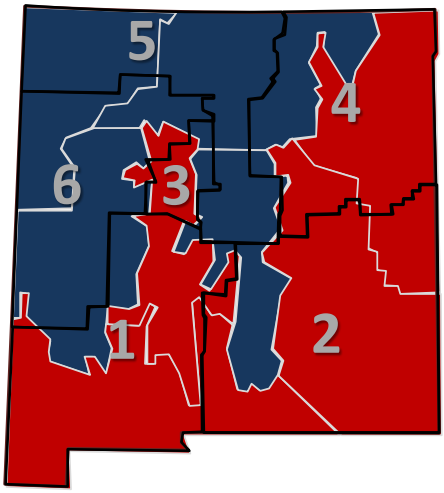


Figure 3. District boundaries superimposed over the state climate units map. Red areas indicate higher summer temperatures while blue areas indicate lower winter temperatures.

2. PROJECT INVESTIGATION

2.1 INTRODUCTION

The long-term performance of pavements depends to a large extent on the stability of subgrade soils; therefore, an evaluation of subgrade soil properties is essential for pavement designers to assess specific site conditions prior to the construction of roads, highways, and airfield runways. The techniques used to assess ground conditions can be divided into laboratory testing, in-situ (or field) testing, and geophysical methods. Laboratory characterization tests provide information about the engineering properties of the soil. Samples collected from the site are used to determine grain-size distribution, plasticity, organic content, and concentration of substances of interest i.e., sulfates and carbonates. Geotechnical field tests provide engineers with measurements of in-situ soil properties such as stiffness, shear strength and density, among others. Geophysical methods include seismic and electrical measurements generally conducted on the ground surface. Geophysical type tests allow for the identification of stratigraphic units of sufficiently contrasting properties, as well as low-strain stiffness, and moisture content, among others.

2.2 GIS PRELIMINARY STUDY

A soil stabilization map for the state of New Mexico has been created as part of this research project. The maps produced by the NMSU Department of Geography's Spatial Applications Research Center (SpARC) uses the methods described in Appendix C and are available through a web platform. This platform allows access to the maps through any internet capable device. The original files are housed on the NMSU Geography SpARC servers, and linked to the host servers of ESRI (Environmental Systems Research Institute). To access the ESRI site go to:
http://nmsu.maps.arcgis.com/apps/Viewer/index.html?appid=c60d9142e69943cd8266592764816d05

This link will bring the user directly to the "NMDOT: Soil Stabilization" mapping application. A variety of background maps and imagery are available for selection with this application using the "Basemap Gallery" found on the upper left tool bar.

The soil map layers are first selected by clicking the "Layers" icon from the tool bar. The available layers include, NMDOT Maintenance District boundaries, NMDOT Administrative District boundaries, New Mexico County boundaries, and the following Soil Index layers:

- ☐ SI 1, Depth 0 to 33 cm
- ☐ SI 1, Depth 34 to 66 cm
- ☐ SI 1, Depth 67 to 99 cm
- ☐ SI 1, Depth 100 plus cm
- ☐ SI .5, Depth 0 to 33 cm
- ☐ SI .5, Depth 34 to 66 cm
- ☐ SI .5, Depth 67 to 99 cm
- ☐ SI .5, Depth 100 plus cm

- ☐ SI 0, Depth 0 to 33 cm
- ☐ SI 0, Depth 34 to 66 cm
- ☐ SI 0, Depth 67 to 99 cm
- ☐ SI 0, Depth 100 plus cm

Selection is done by clicking the box next to each layer of interest, a check mark will appear for each of the desired layers. The transparency of the soils map units can be adjusted by selecting the expand key (>) next to any selected soil layer and choosing the "Opacity" tab. The sliding bar can be adjusted by the user. A low opacity value, for example, will allow roads and other landscape features to be seen through the soil layer polygon. ***A very important step in the use of this mapping application is to first select the county of interest from the "County Bookmark" using the Bookmark tool. Because of the large amount of mapping data being accessed, the user must zoom in past county level to activate selected soil data layers. Once the user has zoomed in sufficiently, the soil index layers will appear.*** The stabilization index is defined in terms of the plasticity index and the fines content as follows:

$$SI(PI, f_c) = \begin{cases} 1 & if \quad (PI \geq 12 \wedge f_c \geq 40) \vee (PI \geq 40 \wedge f_c \geq 12) \\ 0.5 & if \quad (PI \geq 40 \wedge f_c < 12) \vee (PI \geq 20 \wedge f_c \geq 12) \vee (PI \geq 5 \wedge f_c \geq 40) \\ 0 & else \end{cases}$$

where PI is the plasticity index and f_c the fines content. $SI = 1$ indicates that soils in the area are very likely to require stabilization, $SI = 0.5$ indicates that soils in the area may require stabilization, and $SI = 0$ indicates that soil in the area are unlikely to require stabilization.

2.3 SITE INVESTIGATION

Multiple field exploration methods are available for sampling and testing of subgrade soils. Specific methods can be chosen on the basis of design requirements, pre-existing information, sensitivity of the project, and equipment availability. In the case of sampling, soil samples are collected at the site and transported to laboratory facilities for testing. During in-situ testing, field measurements are recorded to assess the mechanical properties of the soil and their variation as a function of depth, yet soil samples are not necessarily recovered. Sampling and testing methods are complimentary. In fact, there are in-situ testing devices that can be used simultaneously to assess field properties and collect small soil samples for laboratory characterization (e.g., standard penetration test).

Guidance for subsurface exploration can be found in AASHTO R13: *Standard Practice for Conducting Geotechnical Subsurface Investigations*, ASTM D 420-98: *Standard Guide to Site Characterization for Engineering Design and Construction Purposes*, and state specific literature. In general, material testing standards are intended to reduce errors induced by sampling biases and to optimize sampling efficiency. Thus, random sampling principles are followed to guarantee an unbiased or averaged material sample. In the case of subsurface exploration for subgrade stabilization purposes, the objective is to identify and delineate specific soil types associated to construction or performance issues. Thus, the frequency of sampling is biased by positive hits on soil types of interest.

In the case of new roads, available guidelines recommend longitudinal sampling frequencies based on a visual assessment of surface uniformity. Sampling for alignments on uniform, flat, featureless terrain, can be conducted once every 2500 to 5000 ft. As variability in either ground conditions (i.e., color, texture, humidity, etc.) or topography increases, sampling becomes more frequent. Sampling frequency ought also to increase in areas prone to problems, such as:

i. low points and areas of poor drainage where water can accumulate,
ii. the boundaries between cut and fill sections, or
iii. failed areas on existing pavements .

However, recommended sampling frequencies seldom go beyond once every 1000 ft, as the costs associated to sampling and characterization make it unfeasible to attain higher resolution. The use of certain in-situ tests can effectively reduce costs and augment exploration coverage to frequencies up to once every 100 to 300 ft. (Cortes and Santamarina 2012; Jones et al. 2010; Mayne et al. 2002). The in-situ test results are used as an index to identify areas of interest where sampling is justified. Table 1 summarizes the sampling frequencies recommended by a variety of agencies. Note that the literature on subgrade stabilization tends to recommend low sampling frequencies (ADOT 2011; Jones et al. 2010; Little and Nair 2009).

Table 1. Recommended testing and sampling frequencies for subgrade exploration.

Agency	Frequency (samples per mile)	Depth (feet)	Reference
Arizona Department of Transportation	4	3 (of finished subgrade elevation)	(ADOT 2011)
California Department of Transportation	18* 4 ~ 11	2.5 (of finished subgrade elevation)	(Jones et al. 2010)
Nevada Department of Transportation	11 ~ 26	5 ~ 10	(Noori et al. 2005)
National Cooperative Highway Research Program	1 ~ 2	5	(Little and Nair 2009)

* in-situ testing using the dynamic cone penetrometer without sampling.

Despite the fact that soil stabilization methods typically target the upper 12 in of subgrade, in-situ testing and sampling extend to a depths between 2.5 and of 5 ft. (AASHTO 2012; Jones et al. 2010). The sample size varies depending on the type of laboratory testing to be conducted. Grain-size distribution and Atterberg limits can be conducted with as little as 10 lbs of soil, as long as the fines fraction (material passing sieve 200) exceeds 4 lbs. Additional tests, such as Proctor compaction, CBR, sulfate content, organic content, pH, and chloride content require sample sizes of up to 200 lbs. Mixture design testing may require soil samples of up to 500 lbs (Jones et al. 2010).

Clearly, sample size and sampling frequency have a significant impact on the cost of subgrade stabilization projects. A single ten mile stretch of road could require between 10 and 100 samples, that is, between 1 and 25 tons of soil. *It becomes imperative to identify a baseline series of laboratory tests to reduce the required sample size, and an in-situ test that could serve as an index to reduce the number of sampling sites.*

2.3.1 In-situ characterization

2.3.1.1 Standard Penetration Test (SPT)

The standard penetration test is the most frequently used test for geotechnical explorations in the U.S.. The SPT can be effectively used to assess the resistance to penetration in loose and dense clays and sands (McGregor and Duncan 1998). SPT soundings also provide disturbed soil samples that can be used to conduct laboratory characterization and index tests. Sample sizes vary depending on stratigraphy. Up to 4 lbs of soil can be recovered in a homogeneous medium; yet, if multiple stratigraphic units are present, the sample size for each unit is reduced in proportion to the relative thickness of the layer.

The SPT is conducted in a borehole, by driving a standard split-spoon sampler into the soil using repeated blows of a 140 lb hammer in a 30 inch free fall. The hammer is operated from the ground surface, and is connected to the sampler by extension rods. Before the test starts, the split-spoon is lowered down to the desired sampling depth. Then the hammer is used to drive it 18 inches into the soil while counting the number of blows required. At this point, the sampler is extracted, and the retrieved soil sample collected and packed in an air tight container for transport to the laboratory. The penetration resistance, *N*, is the number of blows required to drive the sampler for the last 12 inches of penetration.

SPT results are analyzed based on a series of correlations between the *N*-value and soil properties. The first correlations available, developed by Terzaghi and Peck, date back to the 1940's. Correlations between the SPT *N*-value and soil properties remain completely empirical and depend upon an international database of information (Clayton et al. 1995). Throughout the years, the evolution of equipment and methods, have resulted in a series of correction protocols for the *N*-values. These corrections take into consideration primarily the effects of using different types of hammers, N_{60}, and the effect of overburden pressure, $N_{1(60)}$ (McGregor and Duncan 1998).

Despite the wide spread availability of SPT equipment and expertise in the U.S., its use on highway projects remains centered on bridge foundations, embankments, and surveys of areas where slope stability issues are suspected. Conventional SPTs are better suited for a small number of deep explorations within a small footprint area. Conversely, subgrade exploration for stabilization purposes requires a significant number of near surface soundings spread over a wide footprint area. SPT soundings cost between $12 and $24 per foot in addition to the costs associated to post-hole closure ($4 to $9 per foot), equipment mobilization, and data reduction and analysis (Mayne 2007).

2.3.1.2 Cone Penetration Test (CPT)

The cone penetration test is a fast and reliable means of conducting geotechnical site investigations. Its use in highway projects includes the exploration of soft soils for support of embankments, retaining walls, and bridge foundations (Mayne 2007). CPT results are used to estimate in-situ soil properties such as: density, stiffness, friction angle, elastic modulus, and shear strength (Jamiolkowski et al. 2004; Lunne et al. 1997). The test results can also be used for identification of stratigraphic units containing different soil types (Hunt 2010; Jacobs 2004; Shuttle and Jefferies 1998). The evaluation of soil type is indirect and must be inferred from measurements coupled with a good understanding of the local and regional geology (Mayne 2007). CPT soundings can be used as complement to conventional rotary drilling and sampling methods, or in lieu of them (Mayne et al. 2002).

In the CPT, an instrumented cone is hydraulically pushed into the ground while continuous measurements of the soil response are collected. Correlations developed over years of experience are used by qualified engineers to interpret the collected data (Lunne et al. 1997; Robertson and Cabal 2010). The CPT can be used in fine-grained soils (clays and silts) and sands; however, the presence of gravel size or larger particles can affect the advance of the cone and lead to misinterpretation of test results. With typical depths of up to 100 ft, the test can be conducted in 60 to 90 minutes, providing rapid soundings and high resolution; however, no soil samples are recovered in the process.

Specific equipment configurations vary depending on test depth and subsurface conditions, but typically include: an instrumented cone tip, extension rods, a push system, and peripheral electronics. Cone tips are instrumented with load cells to measure point stress and sleeve friction. Additional sensors available allow for the determination of pore water pressure, resistivity, shear wave velocity, temperature, pH, and salinity (Mayne 2007). A mechanical type CPT probe is also available for soundings in very hard and abrasive strata. The push system typically consists of a hydraulic ram housed inside a rig that serves also as reaction frame.

A variety of systems are available to suit specific project needs. CPT devices range in size from small 'mini-pushing' units to very large truck and track vehicles (Lunne et al. 1997; Mayne 2007). CPT rigs need to provide adequate force to push the cone into the soil; thus, for deep soundings they tend to be heavy. Large rigs (> 30 tons) can be used to advance a cone up to 300 ft in soft soils (Robertson and Cabal 2010). CPT soundings cost between $6 and $9 per foot, in addition to post-grouting ($3 to $5 per foot), equipment mobilization, and data analysis. A recent survey of the state of the practice in CPT use showed that a very limited number of state DOTs use this technology on a regular basis, and, those who do, typically hire specialized contractors to carry out the test (Mayne 2007).

2.3.1.3 Dynamic Cone Penetrometer Test (DCPT)

The Dynamic Cone Penetrometer (DCP) is a field instrument used to assess the in-situ strength of unbound aggregate materials. The use of dynamic cones during the 1970's by the Transvaal Roads Department in South Africa, lead to the modern design that began to be gradually adopted by other nations in the 1980's (Wu and Sargand 2007). Pioneering use of the DCPT in the U.S. has been headed by California, Florida, Illinois, Minnesota, Kansas, Mississippi, and Texas; primarily for site characterization of aggregate bases and subgrade soils (Abu-Farsakh et al. 2004).

The dynamic cone test can be of great assistance in preliminary soil surveys, allowing road designers to rapidly identify areas of weak material. Depth averaged penetration rates can be used as an index to delineate the presence of weak soil layers, which can be potential candidates for stabilization (Jones et al. 2010). The use of DCPT in conjunction with density measurements offers an improvement in quality control and assurance. While density is commonly used to assess construction compliance to design specifications, it is not a measurement of the material strength. In fact, field data indicates that compliance with density specifications does not guarantee compliance with strength requirements (Abu-Farsakh et al. 2004; Wu and Sargand 2007). The DCP test can also serve as a tool for structural evaluations of existing pavements by coring through asphalt or concrete layers until the subgrade is exposed for the test to be conducted (Burnham and Johnson 1993).

DCP Test procedure

The cone is advanced by means of an 8kg falling hammer attached at the end of a steel extension shaft. Figure 4 shows the hammer, shaft, and cone tip. The DCP is operated by two people. One person is in charge of operating the hammer while maintaining the shaft in the vertical position; the second operator measures and records penetration depth. The test starts by allowing the hammer to free fall from the pre-established height causing the cone to penetrate the soil on impact. Figure 5 shows the dynamic cone being driven into the ground. Both operators need to keep track of the number of hammer drops (blows) between measurements. It is recommended that the set of hammer blows between measurements be 20, 10, 5, 3, 2 or 1 depending on the soil strength (Humboldt 2003). A typical penetration rate vs. depth profile is presented in Figure 6.

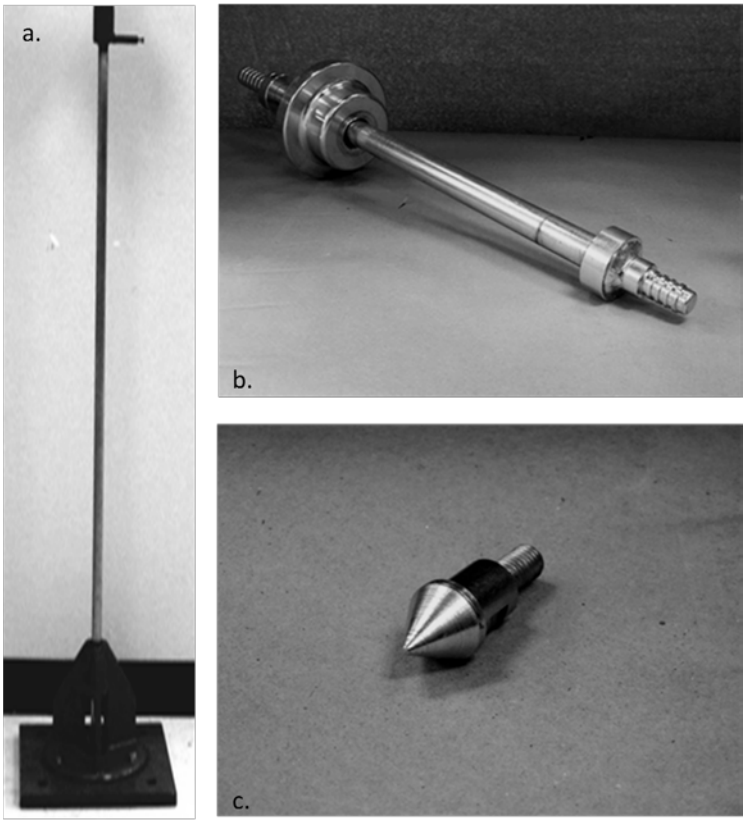


Figure 4. Dynamic cone apparatus: a. shaft and rigid guiding base to ensure vertical penetration; b. falling weight hammer; c. standard cone tip.

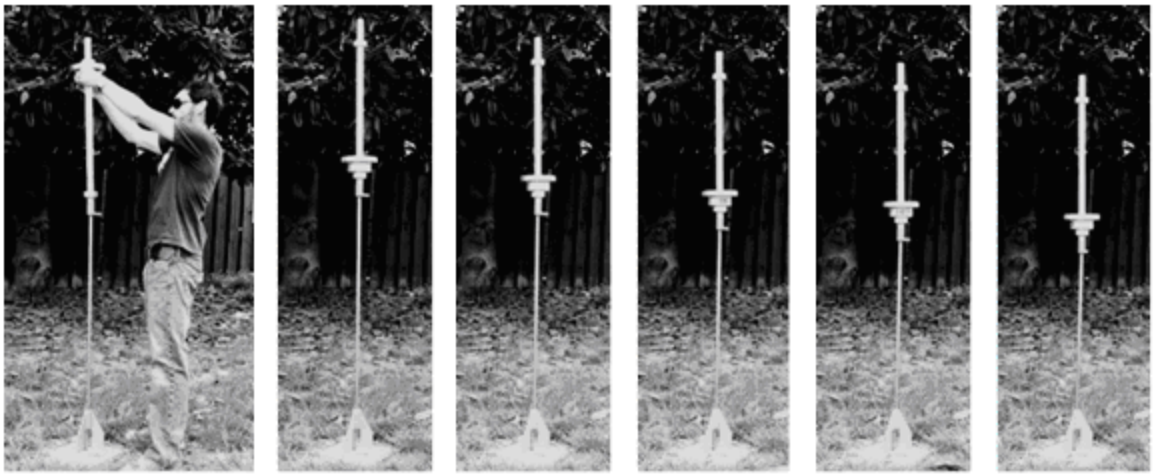


Figure 5. Dynamic cone penetrometer being driven into the ground. Each picture in the sequence is taken after 10 blows.

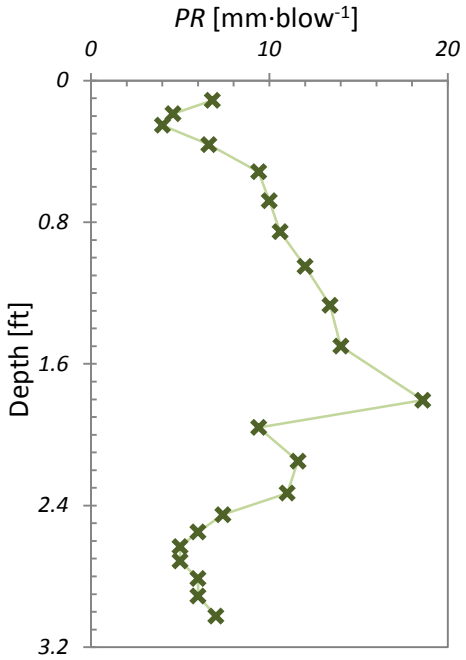


Figure 6. Typical penetration rate vs. depth profile.

Analysis of results

The strength values derived from cone penetration resistance can be used to estimate California bearing ratio (CBR) and soil support values (SSV). The most common available correlations are in terms of CBR~PR, and almost invariably show linear dependence in logarithmic scale. A summary of published correlations is presented in Table 2. While valuable for preliminary estimates, these correlations do not take into account any of the soil properties that define the behavior of granular materials (i.e. moisture content, particle size, particle shape, void ratio, friction angle. etc.). Furthermore, the correlations attempt to compare soil behavior that occurs under different strain regimes; CBR measures soil performance in the elastic regime while in DCP the sediment is driven to failure (Burnham and Johnson 1993). To avoid misleading results, it is preferable to analyze the dynamic cone penetration test results in terms of penetration rate

rather than using the correlated values of CBR or SSV. Additional correlations are available for falling weight deflectometer modulus, plate load test modulus, and resilient modulus.

Table 2. Summary of dynamic cone penetration (DCP) test correlations

Correlation	Reference
$Log(CBR) = 2.56 - 1.15 Log(PR)$	(Smith and Pratt 1983)*
$Log(CBR) = 2.81 - 1.32 Log(PR)$	(Harison 1989)*
$Log(CBR_{lab}) = 2.465 - 1.12 Log(PR)$	(Webster et al. 1992)*
$Log(CBR_{field}) = 2.53 - 1.14 Log(PR)$	*Piedmont residual soils* (Coonse 1999)*
$CBR = \frac{5.1}{PR^{0.2} - 1.41}$	(Abu-Farsakh et al. 2004)
$M_R = a_0 (PR)^{a_1} \left[\gamma_{dr}^{a_2} + \left(\frac{LL}{w_c} \right)^{a_3} \right]$	(George and Uddin 2000)
$M_R = 532.1 \cdot PR^{-0.492}$ ~ Fine-grained $M_R = 235.3 \cdot PR^{-0.475}$ ~ Coarse-grained	(George and Uddin 2000)
$\ln(M_{FWD}) = 2.35 + \frac{5.21}{\ln(PR)}$	(Abu-Farsakh et al. 2004)
$E_{back} = 338 \cdot (PR)^{-0.39}$	(George and Uddin 2000)
$E_{PLT(i)} = \frac{9770}{PR^{1.6} + 36.9} - 0.75$ $E_{PLT(r)} = \frac{4374.5}{PR^{1.4} + 14.9} - 2.16$	(Abu-Farsakh et al. 2004)

* collected by (Wu and Sargand 2007).

PR= DCP penetration rate [mm·blow^{-1}]

CBR= California Bearing Ratio [ ]

E_{back} = Back-calculated elastic modulus [MPa]

M_{FWD} = Falling weight deflectometer modulus [MPa]

$E_{PLT(i)}$ = Initial plate load test modulus [MPa]

$E_{PLT(r)}$ = Reloading plate load test modulus [MPa]

M_R = Resilient modulus [MPa]

γ_{dr} = Field density / max dry density [ ]

LL = Liquid limit [ ]

w_c = water content [ ]

a_0 = 27.86; a_1 = -0.114; a_2 = 7.82; a_3 = 1.925

Summary

The dynamic cone penetration test does not require specialized operators. Field crews can be readily trained to perform the test which can be conducted very rapidly. Small size of the apparatus and the absence of electronic components make it easy to transport and use in remote locations. The ability to conduct in-house DCPT soundings and the low cost of mobilization and servicing, make the test economic, versatile and well suited for routine subgrade exploration. Furthermore, the low capital costs of equipment (under $1,000) make it affordable to acquire multiple units for simultaneous use in the same or different projects.

2.3.1.4 Helical Probe Test (HPT)

The helical probe consists of an auger bit welded to a 4 to 6 ft long shaft. The probe is advanced into the ground by rotating it clockwise in the absence of vertical forces other than the device self-weight. The torque measured while the probe advances into the soil is a function of the force required to penetrate the soil, open a cavity in front of the probe, and overcome frictional resistance against the soil mass. Upon withdrawal, the measured torque is just a function of the frictional resistance at the interface between the soil and the retracting probe (Yokel and Chung 1986). Correlations between the helical probe test and conventional geotechnical in-situ tests make it possible to use the helical probe for shallow soil surveys. Similarly to dynamic cone data, the empirical nature of available correlations leads to uncertainties in the interpretation of results. Helical probe data reflect the shear strength and stiffness of the subgrade and the measured torque should preferably be analyzed in its own virtue.

Test Procedure

The probe is advanced into the ground by rotating a torque-meter wrench positioned at the end of the shaft. The driving rate is maintained at about 8 seconds per revolution, and readings are taken at 6 inch penetration intervals. After attaining the desired depth for the survey the probe is retracted by turning it in the counter clockwise direction while recording the withdrawal torque (Yokel and Mayne 1988). The complete apparatus, including the helical probe and the torque-meter wrench, are presented in Figure 7. Typical data from an HPT sounding are presented in Figure 8.

Helical probe test results correlate well with cone penetration test results when provisions are made to account for the soil particle size (Yokel and Mayne 1988). HPT test results also correlate with density and relative compaction (Cortes 2010). Helical Probes are as portable as dynamic cone penetrometers. Advancing the probe into the ground does not require a hammer, which makes the test safer for the operator. While two operators are recommended to conduct and record test results, one can easily be replaced by a voice recording device. The main advantage of the helical probe over the dynamic cone penetrometer is that it offers the capability of collecting a continuous torque vs. depth profile, limited only by the manner in which data is acquired.

Figure 7. Helical probe apparatus: a. torque-meter wrench; b. helical probe tip. Inserts c, d, and e show the helical probe while being driven into the ground.

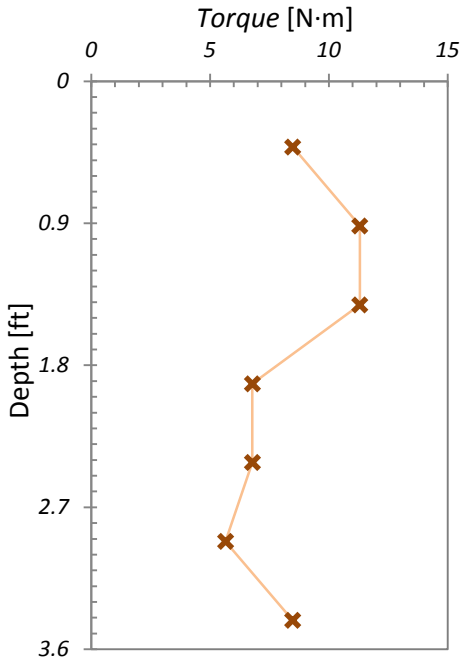


Figure 8. Typical torque vs. depth profile.

2.4 LABORATORY INVESTIGATION

The base level of testing for instability assessment and stabilizer selection consist of grain-size distribution and Atterberg limits. However, further insight into the soil compatibility with a particular stabilizer can benefit from specific surface area testing. In instances where calcium-based chemical stabilizers are under consideration, water-soluble sulfate concentrations must be determined to rule out potential issues associated to post construction heave.

2.4.1 Grain size distribution

Grain size distribution is defined as the relative amount (by mass) of particle sizes present in the soil and is typically presented in a plot of percent finer by weight versus particle diameter in millimeters. For every soil, a grain size distribution can be defined in terms of gravel, sand, silt and clay size percentages. The percentage of particles smaller than 75 μm (i.e., the silt and clay fractions combined) is known as the fines content (f_c). Soils with $f_c > 50\%$ are classified as fine-grained soils whereas soils with $f_c < 50\%$ are classified as coarse-grained soils. The grain-size distribution can be used to infer soil properties such as: permeability, water absorption, stiffness and strength (Lambe and Whitman 1969). In coarse-grained soils, the grain-size distribution controls the packing density and the shear strength. Well-graded soils exhibit higher average coordination number; thus, when cemented, can attain higher strength and stiffness. Despite the preponderance of grain-size distribution on coarse-grained soil behavior, fines contents as small as 5% are known to govern hydraulic conductivity, and fines contents above 12% can influence significantly the soil mechanical properties (Mitchell and Soga 2005; Santamarina et al. 2001). Under these circumstances the most relevant soil properties for analysis are the Atterberg limits.

2.4.2 Atterberg limits

The Atterberg limits of a fine-grained soil are defined as the values of water content at which the soil exhibits marked changes in behavior. As the water content of a soil increases, its state changes from that of a solid to a semi-solid, to a plastic, and finally to that of a liquid. The plastic limit (PL) of a soil is defined as the water content at which the soil transitions from a semi-solid state to a plastic state. The liquid limit (LL) is the water content at which the soil transitions from a plastic state to a liquid state. The plasticity index (PI), defined as PI = LL – PL, conveys the range of water contents over which the soil exhibits plastic behavior. Both the plasticity index and the liquid limit are used for classification of fine-grained soils, and as index properties to estimate their mechanical response. Table 3 shows typical ranges of values for Atterberg limits of selected clays.

Table 3. Properties of selected clays (Mitchell and Soga 2005; Santamarina et al. 2001).

	Montmorillonite	**Illite**	**Kaolinite**	**Attapulgite**
Length, L_p (nm)	1-500	100-200	300-3000	4000-5000
Aspect ratio	100	10	3-10	400-1000
Specific surface, S_a [$m^2 \cdot g^{-1}$]	400-800	80-100	10-20	140-170
Liquid Limit, *LL* [%]	100-950	60-120	30-110	160-230
Plastic Limit, *PL* [%]	50-100	35-60	25-40	100-140

The plasticity index and liquid limit are a reflection of the interactions between the fine-grained soil fraction and the pore fluid (Santamarina et al. 2001). Therefore, the Atterberg limits offer a good index to estimate potential for subgrade swelling and shrinkage in response to fluctuating environmental conditions.

Grain size distribution and Atterberg limits are the base level of testing necessary to assess potential subgrade instability, and serve as the basis of stabilizer selection (DOA 1984; Jones et al. 2010; Little and Nair 2009). However, these tests do not lend themselves for easy field implementation. Direct determination of the soil specific surface area can offer complementary information in the laboratory and allow for rapid determination of potential instability in the field.

2.4.3 Specific Surface Area

Specific surface area (S_a) is defined as the ratio between the surface area of an object and its mass, and is usually given in $m^2 \cdot g^{-1}$ (Mitchell and Soga 2005). The value of specific surface area depends on the amount of exposed particle surface (Cerato and Luteneggerl 2002); therefore, in the absence of internal porosity, larger particles exhibit low values of S_a and small particles exhibit high values of S_a. The specific surface is also a function of particle shape (Santamarina et al. 2001). Spherical particles have the smallest surface area to volume ratio, but when deformed into thin plates or discs, the specific surface area increases dramatically (Cerato and Luteneggerl 2002).

Determination of specific surface area

There are various techniques available to measure specific surface area. The most common being: gas and methylene blue adsorption. The gas adsorption method is based on the condensation of gas molecules onto mineral surfaces. The measurement is made by determining the relationship between the injected gas volume and the pressure required to derive it. In the methylene blue test, S_a is determined based on the amount of aqueous methylene blue molecules absorbed onto the soil particles surface. Several other methods have also been historically used to evaluate specific surface area of soils, but due to time constraints, cost or the need for specialized equipment, have not become widely used.

The methylene blue test offers simplicity and does not require expensive laboratory equipment which makes it an excellent candidate for routine geotechnical characterization testing. *The test is used as a reliable, simple, laboratory technique for determining specific surface area of soils, but can also be readily modified to measure threshold specific surface areas in the field, see Appendix B.* Since methylene blue adsorbs to the negatively charged particle surfaces, the specific surface of the soil can be determined from the number of methylene blue molecules required to saturate the exposed soil surface area. The step-by-step procedure is given as follows (Santamarina et al. 2002): (i) prepare the methylene blue solution by mixing 1.0 g of dry methylene blue powder with 200 mL of deionized water; (ii) prepare the soil slurry by mixing 10 g of oven-dry soil with 30 mL of deionized water; (iii) add the methylene blue solution to the soil suspension in 0.5 mL increments; (iv) for each addition of methylene blue, mix the soil suspension for 1 minute, remove a small drop of the suspension, and place it on Fisher brand filter paper type P5; (v) if the unadsorbed methylene blue forms a permanent light blue halo around the soil aggregate spot, the “end point” has been reached (the methylene blue has saturated all the mineral surfaces of the soil particles); and (vi) determine the

specific surface from the amount of methylene blue required to reach the end point. The relationship between the methylene blue adsorbed and the specific surface area is given by:

$$S_a = \frac{1}{319.87} \cdot \frac{1}{200} \cdot (0.5N) \cdot A_v A_{MB} \cdot \frac{1}{10} \qquad [1]$$

where N is the number of 0.5-mL methylene blue solution increments added to the soil suspension, A_v is Avogadro's number (6.02×10^{23} mol^{-1}), and A_{MB} is the area covered by one methylene blue molecule ($A_{MB} \approx 130$ $Å^2$).

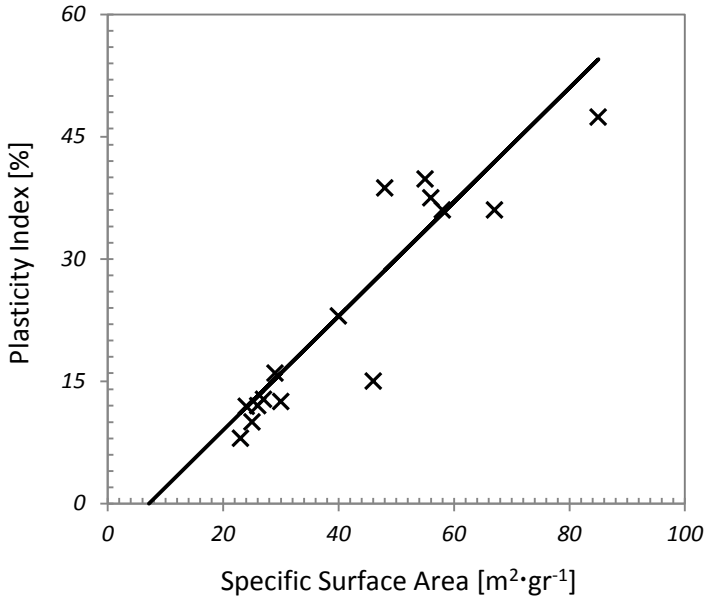


Figure 9. Linear model of the relationship between plasticity index and specific surface area for Canadian clays (Locat et al. 1984)

The specific surface area of a fine-grained soil can be used as an index to estimate its plasticity index. A relationship between specific surface area and plasticity index modeled for various soil types is shown in Figure 9. The PI vs. S_a data trend can be fit using a linear relationship given by:

$$PI = (7x10^{-4} kg \cdot m^{-2}) \cdot S_a + 5 \qquad [2]$$

2.4.4 Water-soluble sulfate

The concentration of water soluble sulfates in a soil is determined from a representative air dried sample sieved, typically using a No. 40 sieve. Soluble sulfate ions are extracted using deionized water at dilution ratios that range from 1:3 (i.e., 3 parts of water for 1 part of soil) to 1:80 depending on the test protocol followed, see Table 4. The soil and water are then agitated by one of several means, for periods of time that range from one to several hours. The slurry is left idle to allow for the sulfate in the soil to dissolve. The pore fluid is then separated from the soil by filtration, and the concentration of sulfate ions is determined by one of three methods (Little and

Nair 2009): (i) inductively coupled plasma (ICP), (ii) atomic emission spectroscopy, or (iii) gravimetric or colorimetric methods. Table 4 summarizes key differences between some of the test protocols available for the determination of water soluble sulfates. The preferred standard followed by the state laboratory at New Mexico DOT is AASHTO T 290.

The results of water soluble tests are analyzed in the context of thresholds that indicate the risk associated to ettringite formation, see Table 5. Test results are used to determine the suitability of a stabilizer and whether or not special construction provisions are required (details are presented in Chapter 3). However, *the use of different test protocols on the same soil specimen can yield different values of soluble sulfates, and lead to the assignment of different risk levels*. In certain cases the difference can be high enough to classify a soil as a low or moderate risk with one test protocol, and as a high or even unacceptable risk with another (Hayes 2007).

Table 4. Comparison of test protocols used in the determination of soil water-soluble sulfates (Hayes 2007; Little and Nair 2009).

Test Protocol	*Sample preparation*		*Extraction*		Measurement
	Drying	Sieving	Dilution	Mixing	
AASHTO T 290-95 (A)	-	No 10 (2mm)	1:3	12 hr @ near boiling	Gravimetric
AASHTO T 290-95 (B)	-	No 10 (2mm)	1:3	12 hr @ room temp.	Turbidimetric
ASTM C1580	18 ~ 24 hr @ 110°C	No 30 (.6mm)	1:8 and 1:80	agitated for 1hr	Turbidimetric
Caltrans 417	Not stated	No 10 (2mm)	1:3	agitated for 15min	Turbidimetric/Gravimetric
CP-L-2103 (CDOT)	-	No. 40 (.425mm)	1:10	16hr @ 60 °C	Turbidimetric
CSA A23.2-3B (Canada)	Air dry	No 50 (.05mm)	variable	agitated for 6hr	Gravimetric
Tex-145 (TxDOT)	Air dry	No. 40 (.425mm)	1:20	12hr @ room temp.	Turbidimetric
Tex-146 (TxDOT)	Air dry	No. 40 (.425mm)	1:20	12hr @ room temp.	Resistivity
Tex-620J (TxDOT)	Air dry	No. 40 (.425mm)	1:10	24 hr @ near boiling	Gravimetric
USBR (1973)	Air dry	No 10 (2mm)	1:10	agitated > 6hr	Resistivity/Gravimetric

Table 5. Levels of risk associated to calcium-based chemical stabilization of sulfate bearing soils (Little and Nair 2009).

Risk Involved	Soluble Sulfate Concentrations	
	Parts Per Million	Percent dry weight
Low Risk	Below 3,000 ppm.	Below 0.3%
Moderate Risk	Between 3,000 and 5,000 ppm	Between 0.3% and 0.5%
Moderate to High Risk	Between 5,000 and 8,000 ppm	Between 0.5% and 0.8%
High to Unacceptable Risk	Greater than 8,000 ppm	Greater than 0.8%
Unacceptable Risk	Greater than 10,000 ppm	Greater than 1.0%

3. SUBGRADE PERFORMANCE IMPROVEMENT

3.1 INTRODUCTION

It is common for roadway alignments to cross areas with subgrade soils that exhibit poor engineering performance. In such cases, construction can proceed only by avoiding or mitigating potential soil problems. While effective, soil excavation, removal and replacement may not always be the most feasible alternative. It can in fact be quite expensive and wasteful considering the need for disposal of the excavated soils and transportation of a competent replacement (Prusinski and Bhattacharja 1999). Thus, avoidance is not always an option. Mitigation through improvement of the soil weaknesses can in most cases be the more practical and economical solution.

Soil performance improvement can be categorized into *subgrade modification*, *subgrade stabilization*, and *base stabilization*. In general, all these are defined as the alteration of any property of a soil in the improvement of its engineering performance (Lambe and Whitman 1969). However, each one refers to a certain target level of performance expected of the improved soil properties. The main objective of a subgrade modification is to temporarily (i.e., hours to days) alter soil properties to allow for the construction of a working platform. The improvement is therefore more operational, in the sense that it is meant to allow for construction to continue. Performance criteria for subgrade modification typically include reductions in plasticity index and/or swelling capacity. In the case of subgrade stabilization, soil properties are improved over a longer lasting periods of time (i.e., months to years), so that it may serve as a more competent pavement foundation. The improvements associated to subgrade stabilization can positively impact pavement design by allowing for the reduction in thickness of bases and even surface layers. Stabilized subgrade performance is measured in terms of strength and durability; however, expected performance is rather low (e.g., unconfined compressive strengths below 200 psi). Higher levels of performance are typically associated to base stabilization works which allow for more significant reductions in base and surface layer thicknesses. The specific soil properties to be altered and the magnitude of the improvement depend on the goals of each project. In the case of roadway works, properties of interest include the soil strength, stiffness, compressibility, plasticity, and swelling capacity.

The shear strength, stiffness, and compressibility of a soil are a reflection of its internal structure. In coarse-grained soils (particle size > 75μm) this structure is referred to as the *granular skeleton*. In fine-grained soils (particle size < 75μm) the internal structure is often referred to as *soil fabric*. Typically, a soil will exhibit a wide range of particle sizes, and its structure will be defined by the arrangement of and interactions among these. An important parameter to describe such a soil is the *fines content* (i.e., the mass fraction of particles smaller than 75μm). The behavior of a soil is governed by the granular skeleton at low fines contents, and by soil fabric at high fines contents, see Figure 10.

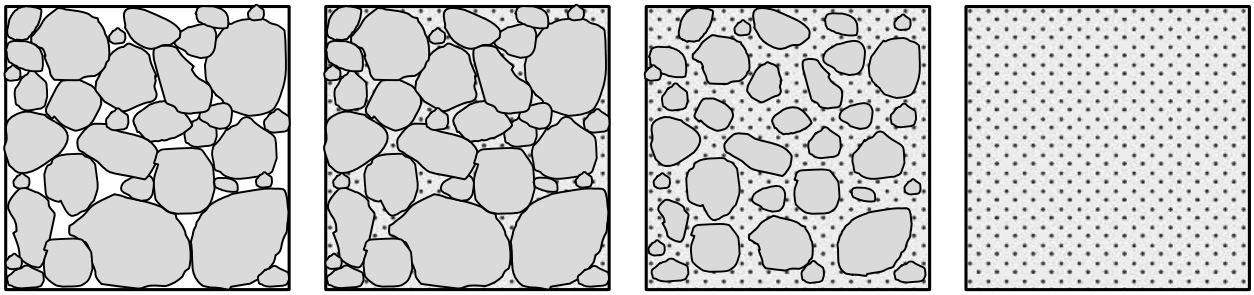


Figure 10. Progression from a coarse-grained granular skeleton (a) to a fine-grained soil fabric (d) as the fines content increases (b) and (c).

In coarse grained soils, the grain-size distribution and the particle shape of the soil determine the maximum and minimum densities it can theoretically attain (Cho et al. 2006; Mitchell and Soga 2005; Narsilio and Santamarina 2008). Uniform or poorly graded soils exhibit low density granular skeletons. The presence of multiple particle sizes within a soil results in higher packing densities, as smaller particles fill in the voids left by larger particles (Furnas 1931; Graton and Fraser 1935). Thus, well graded soils tend to form denser packings. The small strain shear strength, stiffness, and compressibility of a coarse-grained soil is directly related to its density (Salgado et al. 2000; Vanapalli et al. 1996); thus, the engineering performance of the soil can be improved through densification. Classical mechanical stabilization of subgrades relies precisely on increasing the packing density of the soil through compaction and soil blending.

Plasticity and swelling are characteristic of fine-grained soils only, the behavior of which is dominated by the interactions between the soil and the pore fluid (i.e., water). At low water contents these soils behave as semi-solids due to the high suction developed at capillary menisci. In the semi-solid state, a fine-grained soil exhibits high strength, high stiffness, and very low compressibility. If the water content increases, suction diminishes, causing the soil to behave as a plastic-solid: the strength and stiffness of the soil decrease while its volume expands. Finally, at high water contents the soil behaves as a viscous fluid, with very low residual shear strength. These changes in behavior are rooted on the evolution of soil fabric, which reflects the interplay between capillary forces, diffused double layer repulsion and van der Waals attraction (Santamarina 2002). Because of the strong dependency of fine-grained soil properties on water content, mechanical densification can only improve their performance temporarily, as any natural fluctuation in water content would result in loss of performance. Therefore, the stabilization of fine-grained soils requires the introduction of materials that react with soil and water to alter the soil fabric and limit its response to variations in water content over a longer period of time (i.e., chemical stabilization).

Central to subgrade improvement is the compatibility between the alteration method and the soil to be improved. As previously hinted, the improvement of coarse-grained soils can be achieved by purely mechanical methods (i.e., compaction and soil blending). Conversely, the improvement of fine-grained soils requires, in addition, the introduction of chemical reactants (e.g., cement, lime, fly ash). A review of improvement methods and the soils they are most compatible with is offered in the following sections.

3.2 MECHANICAL SUBGRADE IMPROVEMENT

Mechanical improvement encompasses a variety of methods intended to increase soil performance through enhanced particle interaction. In the case of coarse-grained soils, mechanical improvement is primarily accomplished through compaction and soil blending. For fine-grained soils, the positive effects of compaction and blending alone are only temporary; however, an improvement in long term performance (months to years) can be obtained through reinforcement of the soil with geosynthetic materials (e.g., geogrids and geotextiles). Mechanical improvement methods are compatible with any type of soil. For instance, compaction always accompanies chemical improvement methods used in fine-grained soils.

3.2.1 Compaction

The optimum mechanical performance of a soil is attained by maximizing its packing density. Compaction rearranges soil particles, moving them closer together into a denser arrangement. Increasing the density of the soil leads to a reduction in porosity, an increase in coordination number, and an increase in particle interlocking and rotational frustration (Santamarina et al. 2001). As a result, compaction normally increases the soil shear strength, and reduces its compressibility (Lambe and Whitman 1969).

If a soil is compacted with a given type and magnitude of compactive effort at multiple water contents, a curve similar to those shown in Figure 2 can be constructed. In general, compaction curves suggest that as the molding water content increases, the dry density of the compacted soil increases reaching a peak value, and decreasing thereafter. The density and water content at peak are, respectively, the *maximum dry density* and the *optimum compaction water content*. These two parameters depend not only on the soil, but also on the type and magnitude of the compactive effort (see Figure 11). Thus, the objective of a compaction design is to identify the optimum compaction water content and the maximum dry density attainable in the field with available compaction equipment. Field-scale compaction tests can be expensive and lengthy, which makes them unfeasible for routine design work. Laboratory-scale compaction tests overcome these difficulties, yet it is hard to ensure perfect agreement between laboratory-scale and field-scale compaction test results. Therefore, most compaction specifications are prescribed based on laboratory compaction test results, but contain built-in tolerances to accommodate for discrepancies between field and laboratory conditions. Most state specifications require subgrade soils to be compacted to at least 95 percent of the laboratory-determined maximum dry density. However, small increases in strength can be attained by increasing the compactive effort, i.e., using a heavier roller and/or prescribing additional passes (Jones et al. 2010).

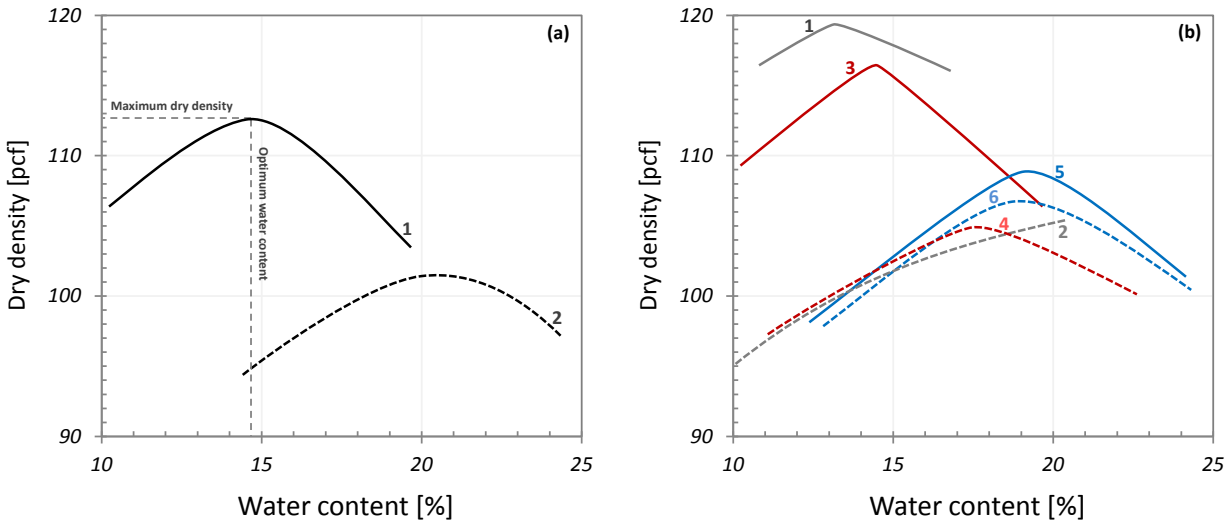


Figure 11. Influence of (a) soil type, and (b) the type and magnitude of the compactive effort on soil compaction curves. Well graded coarse-grained soils (a-1) typically attain higher maximum dry density at lower optimum water content than fine-grained soils (a-2). Part (b) depicts the effect of type and magnitude of the compactive effort: (b-1) 2000 psi static compaction, (b-2) 200 psi static compaction, (b-3) modified proctor compaction, (b-4) standard proctor compaction, (b-5) field compaction with 6 passes of a rubber tire compactor, and (b-6) field compaction with 5 passes of a sheepsfoot roller compactor (modified from Lambe and Whitman 1969).

The maximum dry density and optimum moisture content are typically determined using a standard (ASTM D698 or AASHTO T99) or modified (ASTM D1557 or AASHTO T180) laboratory Proctor compaction test (AASHTO 2010; AASHTO 2010; ASTM 2012; ASTM 2012). The Unconfined Compressive Strength test (UCS) or the California Bearing Ratio test (CBR) can be used for comparing shear strengths at different densities. The results of these performance tests are used to determine whether compaction alone can satisfy target stabilization performance.

3.2.1.1 Construction considerations

The soil type and the prescribed compaction depth dictate the type of equipment needed for compaction. Fine-grained soils are best compacted with sheepsfoot rollers, while coarse-grained soils are best compacted using smooth-drum vibratory and/or rubber-tired rollers. The maximum thickness for each compaction lift is governed by the static weight of the compaction equipment; recommended maximum thicknesses are presented in Table 6.

Table 6. Recommended maximum compaction layer thickness as a function of the equipment static weight, after (Jones et al. 2010).

Max layer thickness [in]	Static weight [tons]
< 6	12
6 - 8	15
8 - 10	18
10 - 12	20
> 12	25

The field water content must be determined and accounted for in the calculation of optimum compaction water content (Jones et al. 2010). Depending on field moisture, attainment of the optimum compaction water content may require wetting or drying (Lambe and Whitman 1969). Fine-grained soils are typically compacted slightly above optimum (i.e., < +2%). Higher moisture contents may render fine-grained soils unworkable. Coarse-grained soils are usually compacted very near the optimum compaction water content (Jones et al. 2010).

Several passes of the compactor may be needed before reaching a target compaction level. Field compaction is typically assessed through density measurements (e.g. nuclear density gauge, sand cone, balloon test). Issues related to heterogeneous compaction may arise when attempting to compact a layer thicker than recommended, and/or when inappropriate compaction equipment is utilized. The zone of stress influence within an elastic medium beneath static strip and square loads are given in Figure 12. Despite the high stress concentration at the surface, the stresses within the material decrease rapidly with depth. In the case of strip loads (i.e., a roller compactor) the vertical stresses are reduced to 20% of the applied surface stress at a depth of just 3 times the load thickness (B). In an excessively thick lift, only the soil exposed to high stresses attains high density. The soil deeper within the lift, where the stress influence is low, remains unchanged. In such cases the upper part of the lift may attain satisfactory compaction while the lower portion remains under-compacted.

Heterogeneously compacted lifts fail to perform as designed under service loads; thus, it is important to identify them during construction, when timely corrective actions can be taken to address the problem. Homogeneity can be assessed though the use of density probes that can sample the full depth of the compacted lift, or by advancing strength probes while monitoring variations in penetration resistance as a function of depth (e.g., DCP and HPT).

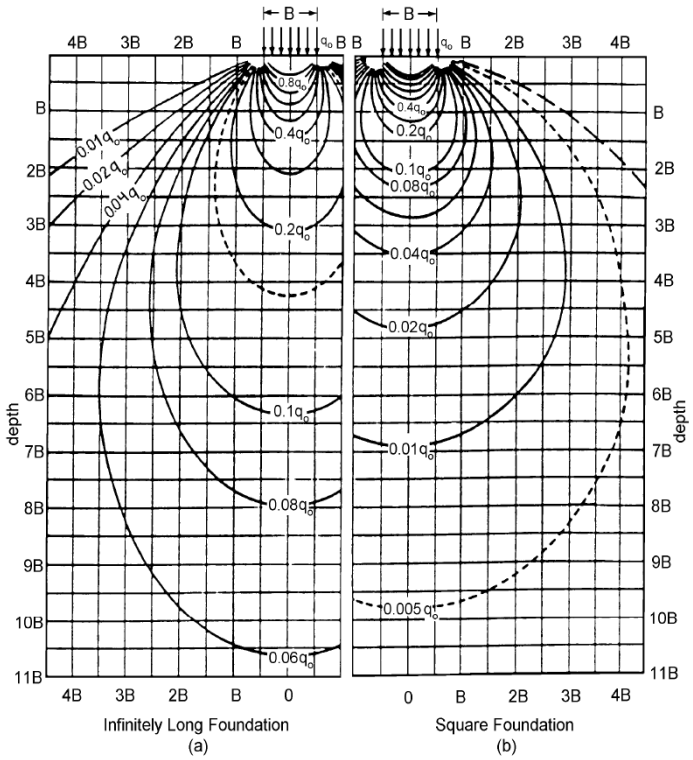


Figure 12. Boussinesq vertical stress influence contours for (a) strip loads and (b) square loads (AASHTO 2012).

3.2.1.2 Performance compliance assessment

Performance compliance assessment methods applicable to soil compaction include laboratory and in-situ material testing. Laboratory testing includes grain size distribution, unit weight and water content (i.e. compaction curve), which serve to confirm design parameters based on limited pre-construction soil sampling. In-situ tests are used to confirm compaction targets are achieved in terms of compacted density (e.g., sand cone and/or nuclear density gauge). Field density measurements should be representative of the entire lift; thus, penetration type sampling is recommended. Surface based measurements should be used with great care to ensure measurements are representative of the entire lift, and not hardened crusts with under-compacted material underneath. *Penetration type measurements (e.g., DCP and HPT) can, and ought to, be used complementarily to confirm layer homogeneity and help identify deficiently compacted lifts.*

Mechanistic design methods will eventually require compacted layer performance inputs such as strength and stiffness. Therefore, emerging performance compliance assessment procedures will need to incorporate resilient modulus, penetration resistance, and plate load testing.

3.2.2 Soil blending

Soil blending involves the mixing of multiple soils of different gradations to attain a well graded soil blend which exhibits improved strength and stiffness while optimizing compaction (i.e., attainment of a higher maximum dry density at the same compactive effort). As previously discussed, the compacted density of a soil is in great part governed by its grain-size distribution. The lower densities typical of uniform or poorly graded soils can be increased by selectively incorporating particle sizes missing in the original soil gradation through blending. In the majority of cases, a fine-grained soil is improved by the addition of coarse aggregates (Jones et al. 2010); very rarely should a fine-grained soil be used to improve the mechanical properties of a coarse-grained soil. When properly compacted, a soil that exhibits a continuous grain-size distribution (i.e. a sand and gravel skeleton containing fine sand, silt, and clay particles to fill the voids left by the coarser particles), creates a dense, high load bearing, and water impermeable layer (Kézdi 1979). Figure 13 depicts the ideal gradation for a load bearing granular layer.

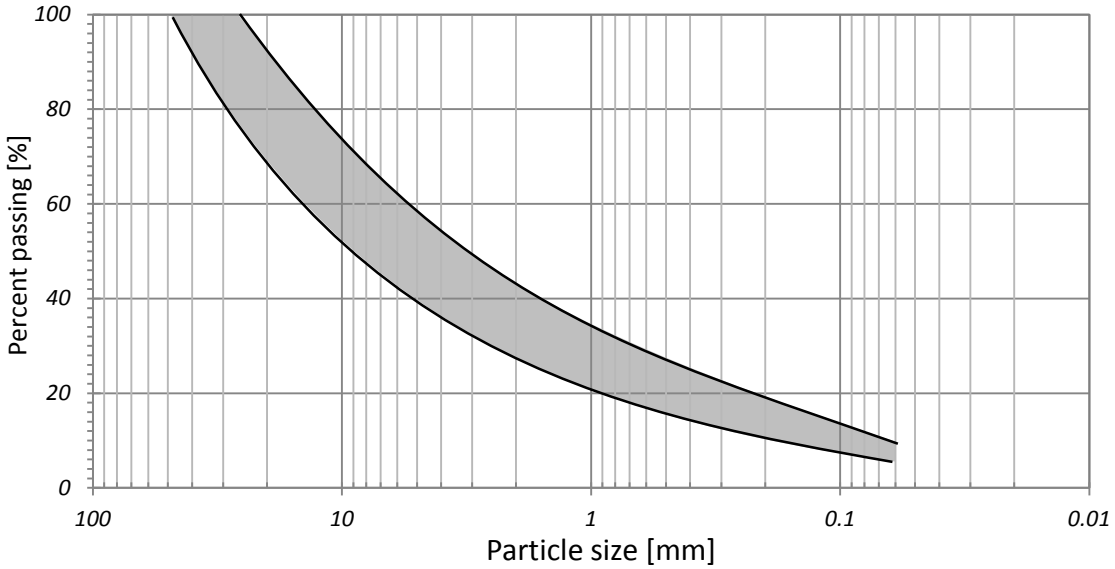


Figure 13. Most favorable grain-size distribution in a load bearing granular layer (modified from Kézdi 1979).

Multiple mathematical tools are available to determine the grain-size distribution and mixing ratio of the soil required to attain a target gradation curve. Thus, for a given soil, one can determine a complimentary 'manufactured' soil which, when blended at an appropriate ratio, yields a uniformly graded soil. While technically sound, manufacturing a complimentary gradation is not often practical, as the costs associated with production and delivery would make the procedure economically unfeasible. A more attractive alternative is to utilize suitable materials from cut sections of the roadway alignment or nearby aggregate sources. While in this case it is difficult to attain a perfectly uniform grain-size distribution, the properties of the blended soil can nonetheless be significantly improved (Jones et al. 2010; Kézdi 1979). Determining the mixture proportions can be accomplished using the arithmetic method. Given two soils, A and B, with mass fraction of particles of size *i* given by A*i* and B*i* respectively, the mixing ratios $a \cdot \mathrm{A} + b \cdot \mathrm{B}$ which make the blended gradation curve as close to the target gradation X as possible are given by:

$$a = \frac{\sum |X_i - B_i|}{\sum |X_i - A_i| + \sum |X_i - B_i|} \qquad [3]$$

$$b = 1 - a$$

Soil blending typically requires laboratory determination of the soil grain-size distribution (ASTM C136) as well as the Atterberg limits (ASTM D4318). Poorly graded soils are good candidates for stabilization by blending and compaction; however, the presence of medium to high plasticity fines in the soil may render its stabilization temporary. In such cases, a more permanent improvement in performance may be attained by combining chemical improvement methods with soil blending and compaction. Soil blending is not a stand-alone improvement method, and will always be accompanied by compaction; thus, the performance of a blended soil can in part be assessed by determining the maximum dry density of the blend. As

with other compacted soils, CBR and UCS measurements can be conducted to draw correlations between density and strength, and to determine if the target performance levels can be met.

3.2.2.1 Construction considerations

The main construction issues related to soil improvement by blending are associated to segregation when improperly mixed. The use of a rotary mixer (see Figure 14) in combination with a water tanker have been found to be effective for in-situ soil mixing and for blending materials in general (Jones et al. 2010). The imported soil is typically spread over to the thickness required to achieve the target blending ratio. The combination of high temperatures and low atmospheric moisture content that characterize the sate climate can lead to rapid drying and cluster formation in clays. Under these circumstances, attaining a homogeneously mixed soil becomes very difficult and the potential for destabilization after construction can be very high. Considering state climatic conditions and stock pile management practices, the use of soil blending should be limited to the addition of coarse-grained soils (i.e., sands and gravels) to improve the properties of a fine-grained subgrade (i.e., choking).

Figure 14. Dual purpose pavement recycler and soil mixer equipment examples. CAT RM500 Road Reclaimer (left) and BOMAG MPH 125 Soil Stabilizer/Asphalt Recycler (right), images courtesy of http://www.bomag.com/us/ and http://www.cat.com/en_US.html.

3.2.2.2 Performance compliance assessment

Performance compliance assessment methods applicable to soil blending include laboratory and in-situ material testing. Laboratory testing includes grain size distribution, unit weight, water content, which serve to confirm design parameters based on limited pre-construction soil sampling. In-situ tests are used to confirm homogeneous blending and depth of penetration. Since soil blending is typically followed by compaction, performance compliance assessment procedures used in compaction must also be followed.

3.2.3 Geosynthetics

Geosynthetics are a family of polymeric products developed in response to the needs of civil engineering projects. Numerous types of geosynthetics have emerged over the years, including geotextiles, geonets, geogrids, geomembranes, geosynthetic clay liners, geofoams, geocells and geocomposites. Geogrids and geotextiles are the main types of geosynthetics used in subgrade improvement, their use dates back to the 1970's (Giroud and Han 2004). Geogrids are typically made of polyethylene or polypropylene that is either extruded, stretched, woven, or knitted to form a grid with relatively large openings (i.e., one or multiple gravel size particles can fit within the openings). Geotextiles are permeable fabrics made typically from polyester or polypropylene,

and are either woven or nonwoven (Shukla 2012). The pores within geotextiles are typically small which makes them useful for separation and filtration purposes.

Geosynthetics are typically used to provide *separation* between the sub-base and subgrade layers, and/or to *reinforce* the sub-base layer. The long-term performance of a pavement structure depends on each layer functioning as designed. Over time, under service loads, intermixing occurs between the layers, altering their grain-size distribution and their mechanical properties (i.e., strength and drainage), and compromising the ability of the pavement structure to accommodate traffic loads. Geotextiles are highly effective at maintaining pavement layers separate, avoiding the 'contamination' of the base and sub-base layers by migrating subgrade particles. Separating the materials ensures that the integrity of each layer is maintained, allowing them to function as intended (Holtz et al. 1998; Zornberg et al. 2008).

Geogrids are often employed as reinforcement for base and sub-base layers. While the properties of the subgrade soil are not altered directly, geogrid reinforcement alters the mechanical response of the pavement structure resulting in the following:

Influence on the reinforced layer

The geogrid interacts with the coarse-grained soil particles of the base and sub-base layers through interlocking. Particles in the vicinity of the grid fill its openings allowing the grid ribs to offer confinement and provide lateral restraint upon loading, see insert in Figure 15-a. The lateral restraint results in a reduction in surface rutting while the interlocking effect leads to an increase in stiffness which aids in stress distribution, reducing the magnitude of the vertical stresses on the subgrade. The presence of the grid further reduces the shear stress transmitted to the subgrade which results in an apparent increase in its bearing capacity (Perkins 1999).

Influence on the subgrade soil

The geogrid can improve the subgrade performance through four mechanisms (Giroud and Han 2004; Shukla 2012; Zornberg 2011): (i) prevention of local subgrade shearing, (ii) redistribution of stresses, (iii) reduction or reorientation of shear stresses, and (iv) tensioned membrane effect. When the vertical stress on the subgrade reaches the elastic limit of the soil, it fails and local permanent shear occurs as the sub-base punches into the subgrade. Shear zones grow under repeated loading, causing deterioration of the sub-base and an increase in the magnitude of the vertical stresses on the subgrade, until eventually the bearing capacity of the subgrade is reached and complete failure occurs. The presence of geogrids between the sub-base and the subgrade prevents the development of local shear zones (Giroud and Noiray 1981), see Figure 15-b. Since the geogrid increases the stiffness of the sub-base layer, the magnitude of the vertical stresses on the subgrade decreases, as the surface load is spread over a larger area. Furthermore, the interlocking between the geogrid and the sub-base protrudes into the subgrade creating a rough surface that resists lateral movement within the subgrade. This 'inward shear' resistance increases the subgrade bearing capacity (Perkins 1999). Finally, when rut develops over the surface and across the sub-base-subgrade interface, the deformed shape of the geogrid causes an increase in confinement in the subgrade and a reduction in the vertical stresses under the load (Giroud and Noiray 1981), see Figure 15-c.

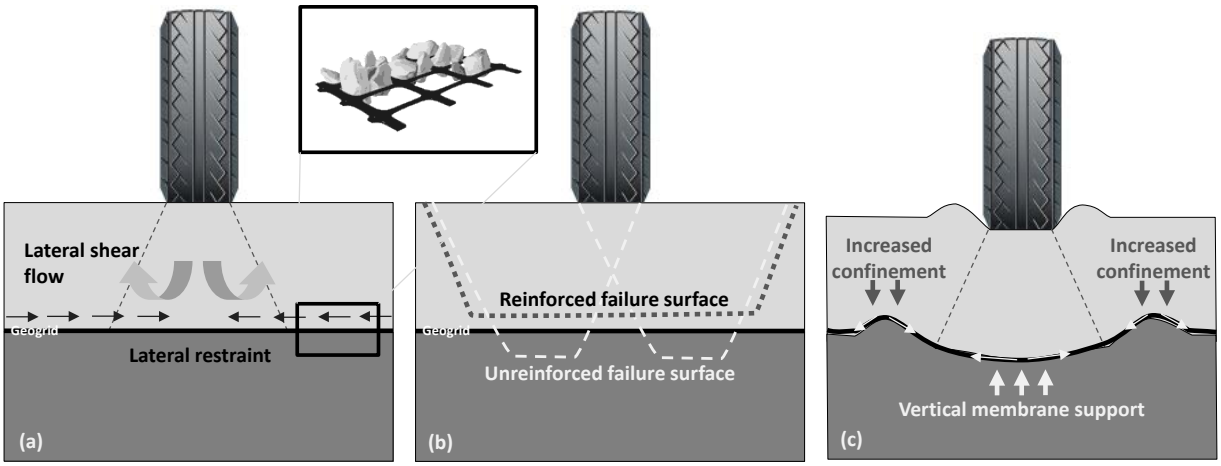


Figure 15. Geogrid subgrade stabilization mechanisms: (a) lateral restraint and interlocking, (b) reduction and reorientation of shear stresses, and (c) tensioned membrane effect. After (Haliburton et al. 1981).

Geogrid reinforced working platforms accumulate plastic deformations at a much lower rate than unreinforced working platforms, and deformation ceases after a certain number of cycles. As a result, total deformation is reduced in the presence of the geosynthetic (Maxwell et al. 2005). Geogrids have also been used to stabilize roads crossing over expansive clays. The presence of a paved (i.e., impervious) surface over an existing clay deposit alters the shape of wetting and drying fronts. Water from precipitation events enters the expansive soil deposits through pervious shoulders causing the soil underneath to expand. Since the hydraulic conductivity of the clay is low, the wetting front is confined to the area under the shoulder resulting in differential volume expansion. Over time, these differential volumetric changes wear the pavement to the point of cracking. Introducing a geogrid reinforcement at the interface between the sub-base and the subgrade has been found to eliminate the longitudinal cracking in the pavement surface (Zornberg and Gupta 2009).

Despite available data, there still is no consensus on the geosynthetic material properties that most directly affect the performance of the subgrade. However, the tensile strength in the cross-machine direction especially at 2% strain appears to play a major role in rut control. Data from multiple test sections indicates that welded, woven, and stronger integrally formed geogrids perform significantly better than geotextiles and weaker integrally formed geogrids (Cuelho and Perkins 2009).

3.2.3.1 Uses

Geosynthetics have been found to be useful in handling the following subgrade conditions in both temporary and permanent road applications (Holtz et al. 1998):

i. Poor soils such as: clayey sands, low plasticity silts and clays, high plasticity silts and clays, and organic soils.
ii. Soils that exhibit low undrained shear strength (< 90kPa), low CBR (< 3), and or low resilient modulus (< 30 MPa).
iii. Areas with high water table.
iv. Subgrades that exhibit high sensitivity.

Geosynthetics can be used to reinforce sites where available soils are too weak to allow access to construction equipment. Even if the finished roadway can be supported by the existing subgrade, construction in certain soils can become very challenging. In such cases, the development of a working platform may require subgrade improvement (mechanical or chemical) or excavation and replacement. If the problems are known a priori (i.e., during preliminary site exploration), multiple subgrade improvement methods can be compared so that the most cost effective be selected and implemented. However, in cases were the weak soils are found as 'patches' that were not identified during preliminary samplings, the time necessary to assess site conditions and to select a suitable improvement method could result in costly interruptions to construction. Under these circumstances, geosynthetics can be deployed rapidly to offer a cost-effective alternative. Provided soil conditions are not severe enough to require further analysis to either identify a suitable improvement method or to modify the pavement structure to accommodate the subgrade conditions.

3.2.3.2 Design

The size of aggregate in the sub-base layer determines the opening size needed to enhance the interaction between the soil and the geogrid. In general, each manufacturer provides a design guide for their product. In the case of geogrids, properties such as the tensile strength and junction efficiency need to be selected based on the expected design loads (Zornberg et al. 2008). If a geotextile is being used, a woven or nonwoven fabric will be chosen depending on whether it is being purposed for filtration or drainage, and the size of the particles being filtered. Geosynthetic reinforcement can serve two purposes: increase the service life of a given pavement section, or allow for a reduction in the reinforced layer thickness (base or sub-base) to satisfy a target service life. In order to make a fair comparison with alternative subgrade improvement methods, it is recommended to take into consideration potential thickness reductions in sub-base and base layers when considering the economic feasibility of geosynthetic reinforcement.

In the majority of cases, stresses induced by construction equipment exceed the expected maximum stresses induced by traffic loads; hence, the selection of geosynthetic reinforcement is governed by maximum expected construction stresses. The geotextile must survive construction in order to meet its post-construction target performance requirements (Holtz et al. 1998). Geotextile performance requirements to meet the most severe conditions anticipated during construction are listed in Table 7. Geotextiles that meet or exceed the tabulated values can be considered acceptable for most projects (Holtz et al. 1998). The construction of trial sections is common practice to evaluate the effect of construction equipment at the site. After completion of the trial section the geosynthetics are exhumed to assess their tolerance to imposed construction stresses. In case of poor performance the following actions may be taken: (i) increase lift thickness, (ii) modify the construction equipment, and/or (iii) select a different geosynthetic (Holtz et al. 1998).

Table 7. Geotextile performance requirements, after AASHTO (1997).

Property	Requirement		Testing Standard
	< 50% elongation	>50% elongation	
Grab strength	1400 N	900 N	ASTM D4632
Sewn Seam Strength	1200 N	810 N	ASTM D4632
Tear Strength	500 N	350 N	ASTM D4533
Puncture Strength	500 N	350 N	ASTM D4833
Burst Strength	3500 kPa	1700 kPa	ASTM D3786
UV Stability (Retained Strength)	50% after 500 hr exposure		ASTM D4355

Design guidelines for temporary and permanent roadways are provided by FHWA in HI-95-038. In both cases, the geosynthetic is assumed to provide no structural support; thus, no reduction in the aggregate base thickness is permitted. In reality, the presence of the geosynthetic does increase the section structural capacity. Giraud and Han (2004) refined a method for determining the thickness of the base required to limit rut depth in geosynthetic reinforced unpaved roads:

$$h = \frac{1.26 + \left(0.96 - 1.46J^2\right)\left(\frac{r}{h}\right)^{1.5} \log N}{1 + 0.204\left[\min\left(\frac{E_{bc}}{E_{sg}}, 5\right) + 1\right]}\left[\sqrt{\frac{P}{\pi r^2 m N_c c_u}} - 1\right] r \qquad [4]$$

where:

h: required base thickness,
N_c: bearing capacity factor (N_c = 5.14 for geotextiles and N_c = 5.71 for geogrid reinforcement),
c_u: subgrade soil undrained shear strength,
m: bearing capacity mobilization coefficient ($0 < m \leq 1$),
r: equivalent circular contact area radius,
P: wheel load (force),
J: aperture stability modulus of geogrid,
N: number of load repetitions,
E_{bc}: base resilient modulus, and
E_{sg}: subgrade resilient modulus.

This expression has been derived assuming a minimum base course thickness of 4 inches with a single layer of reinforcement placed at the interface between the subgrade and the base. Furthermore, the subgrade soil has been assumed to be homogeneous within the depth of influence of the load (typically less than 5 ft), allowing for the development of subgrade failure without the influence of deeper soil layers (Giroud and Han 2004). Similar equations are incorporated in proprietary design methods freely available from geosynthetics manufacturers.

3.2.3.3 Performance compliance assessment

There are a variety of performance compliance assessment methods applicable to geosynthetic reinforced pavements. The selection of specific assessment protocols depends on project specific

conditions, design method and desired level of performance. Material specific performance testing for geosynthetics includes: mass per unit area, opening size, tensile strength, burst resistance, puncture resistance, tear strength, overlap efficiency, interfacial shear strength, UV resistance and permittivity. These tests are intended to confirm material properties and to ensure the selection of a geosynthetic that can meet project specific needs. The tests involved require specialized laboratory equipment and must conform to established standard testing protocols defined by ASTM and AASHTO. In addition to material testing, the pavement section as a hole must undergo performance evaluation; thus, base course and subgrade properties must be obtained to assess compaction and mechanical performance. Sand cone and/or nuclear density measurements can be used to evaluate compaction level while static and dynamic plate load tests can be used to assess the mechanical performance of individual pavement layers as well as of the entire pavement section (Schaefer and Berg 2014).

3.3 CHEMICAL SUBGRADE IMPROVEMENT

Chemical improvement encompasses a variety of methods intended to alter the physical properties of the subgrade soil, and its response to fluctuations in environmental conditions. Chemical improvement is typically associated to fine-grained soils of medium to high plasticity (i.e., PI > 20), and consists primarily of a combination of admixtures that interact with soil particles through cation-exchange and pozzolanic reactions. Among these, the most commonly used are portland cement, lime and fly ash. The incorporation of these admixtures can be effectively used to limit swelling and frost heave, and to improve the strength and stiffness of the soil (Christopher et al. 2006; Cokca 2001; Kézdi 1979; Prusinski and Bhattacharja 1999). In this section, the three most common types of chemical admixtures: (i) cement, (ii) lime, and (iii) fly ash are introduced in detail. Discussions regarding performance comparison for economic feasibility, and limitations associated to the use of these admixtures in soils containing soluble sulfates are presented towards the end of the section.

3.3.1 Cement

The addition of cement and water to a fine-grained soil followed by compaction results in an improvement of the soil strength and stiffness under varying environmental conditions (e.g., fluctuations in water content, ionic concentration, and temperature). Cement stabilization alters subgrades primarily in two ways: (1) it binds soil particles together increasing strength and providing resistance to tensile stresses, and (2) it causes partial to complete alteration of clay minerals effectively reducing soil plasticity (Jones et al. 2010). Thus, cement can be effectively used to improve the performance of both coarse and fine grained soils.

This section starts with a review of the mechanisms by which cement alters soil properties, providing an in-depth examination of the variables controlling strength. Available mixture design procedures are overviewed and followed by an assessment of their merit and potential for improvement. The review covers performance assessment methods and available testing standards.

3.3.1.1 Parameters that Govern Soil-Cement Compressive Strength

In the absence of organic matter and in low electrolyte concentrations, the main variables controlling the strength of cement-stabilized soil are: grain-size distribution, soil plasticity, cement content, moisture content, compacted dry density, and cement hydration time.

In coarse grained soils, the *grain-size distribution* controls packing density and shear strength. Well graded soils exhibit higher average coordination number; thus, when cemented yield higher strength and stiffness. While the compressive strength of cement stabilized soils increases with the sand and gravel fractions, the optimal strength is attained at fines content (f_c) greater than zero (Kwon et al. 2010). Therefore, the presence of silts and clays in the cemented soil facilitates the attainment of higher strength. There is however a value of f_c beyond which strength decreases (Houben and Guillaud 1994). Strength sensitivity to f_c depends primarily on the fines' plasticity and swelling capacity (Reddy and Jagadish 1995). The optimal f_c for non-expansive fines is in the range of 12 to 15% (Reddy et al. 2007).

Plasticity index (*PI*) and *liquid limit* (*LL*) are a reflection of the interactions between the fine-grained soil fraction and water. For soils sharing similar gravel, sand, and fines contents,

those with higher *PI* fines exhibit lower compressive strength (Kwon et al. 2010). The *PI* has a dramatic influence on the amount of cement and water required to attain a target compressive strength (Rahman et al. 2010; Reddy et al. 2007).

Cement content (C_c), defined as the ratio of the mass of cement (m_c) to the mass of dry soil (m_s), is considered to be the most important aspect of mixture design. At a given water content, an increase in C_c results in an increase in both dry and wet compressive strengths (Haralambos 2009; Jayasinghe and Kamaladasa 2007); however, this is only valid for a narrow range in C_c, the active zone. In this zone, the water content is higher than required for complete cement hydration, and an increase in the cement fraction yields a higher amount of cementitious products (Horpibulsuk et al. 2010).

The *water content* (w_{mix}) of a soil-cement mixture, defined as the ratio of the mass of water (m_w) to the mass of dry solids, controls the short term strength gain (compaction) and the long term strength gain (cement hydration). Considering the chemical reactions involved in the hydration of typical Type-I Portland cement, it takes about 4.2 moles of H_2O to fully react 1 mol of cement. This results in a theoretical water-to-cement ratio ($W/C = m_w \cdot m_c^{-1}$) of 0.31. At low W/C the relative spatial distribution of water and cement molecules limit their interactions; thus, additional water is required to overcome spatial constraints, $W/C_{min} \approx 0.42$ (Mindess and Young 1981). Reported experimental results indicate that in soil-cement mixtures the W/C at which the maximum strength is attained is not only significantly higher than 0.42, but also depends on the type of soil and the compactive effort, see Figure 16.

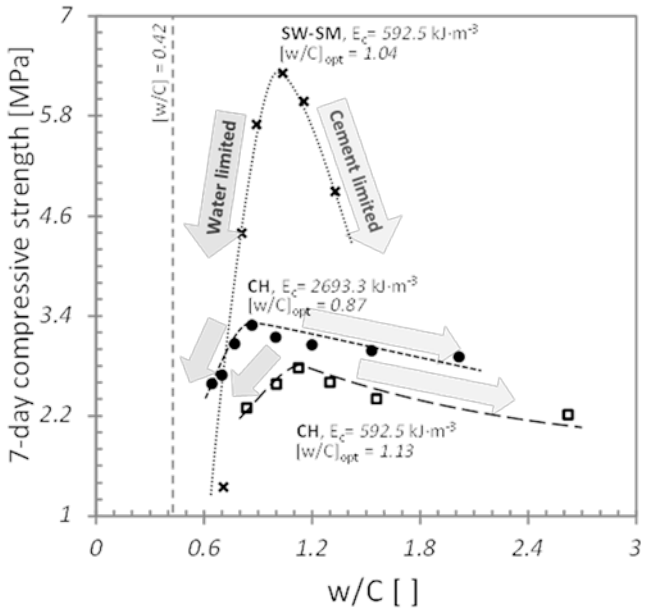


Figure 16. 7-day compressive strength of soil-cement mixtures at various water-to-cement ratios modified after Horpibulsuk et al. (2010).

The strength and durability of cement stabilized soils increase with the *compacted dry density* (Kwon et al. 2010; Spence 1975). The maximum dry unit weight of a compacted soil-cement specimen is higher than that of the untreated soil, whereas the optimum compaction water content remains practically the same (Horpibulsuk et al. 2010; Kézdi 1979). Changes in the compaction curve are associated to soil-fabric changes brought about by the release of calcium and hydroxide ions, which are produced during initial cement hydration reactions. The optimum

compaction water content ($w_{c\text{-}opt}$) and the optimum strength water content ($w_{s\text{-}opt}$) are not the same. Despite the gain in strength derived from a denser specimen compacted at $w_{c\text{-}opt}$, the strength of the soil-cement mixture reaches a peak at $w_{s\text{-}opt} > w_{c\text{-}opt}$ (Reddy and Kumar 2011). Water content not only influences the volume of voids, but also the amount of hydration products. Compacting wet of optimum results in an increase in cementitious products as evidenced by higher amounts of crystalline $Ca(OH)_2$ (Horpibulsuk et al. 2010). However, for values of $w > w_{s\text{-}opt}$, the concentration of hydration products diminishes and thereby the strength.

Marked changes in material properties take place over the *hydration time*. The growth in cementitious products results in changes in porosity. The volume of small pores (<0.1μm) decreases significantly during the first 7 days, after which reduction in the large pore space volume occurs. Thus, the overall porosity decreases as curing progresses (Horpibulsuk et al. 2010). The increase in cementitious products and associated reductions in pore space result in a progressive strength gain with time. Hydration time can however negatively influence strength if compaction is delayed (Reddy and Kumar 2011).

3.3.1.2 Available Mixture Design Guidance

Mixture design requires knowledge of soil and cement physical characteristics. Soil properties of significance include:

(1) grain-size distribution,
(2) liquid and plastic limits,
(3) in-situ water content, and
(4) optimum compaction water content.

The following chemical properties of the soil must also be determined in order to assess compatibility with cement stabilization (Kézdi 1979):

(1) sulphate content,
(2) magnesium content,
(3) calcium content,
(4) organic matter content, and
(5) pH.

Cement is typically used to improve soils that exhibit a plasticity index lower than 35; however, higher plasticity soils can and have been successfully treated with cement (Jones et al. 2010; Prusinski and Bhattacharja 1999). Sands, silty sands, silts, silts containing organics, and lean clays, are in general good candidates for cement stabilization. Sulfate reach subgrade soils (i.e. sulfate content > 3000 ppm), soils with high organic content, or soils that exhibit low pH require special considerations for treatment. The methods used for performance assessment are covered in the standards outlined in Table 8.

Table 8. Cement stabilization mixture performance assessment standards.

Performance Parameter	Standard Test Method	Comments
Plasticity index (PI)	ASTM D4318	Assessment of plasticity reduction (ASTM 2010).
Compressive strength	ASTM D1633	Assessment of compressive strength of stabilized soil (ASTM 2007).
Durability (wet-dry)	ASTM D559	Assessment of durability under wetting-drying cycles (ASTM 2003).
Durability (freeze-thaw)	ASTM D560	Assessment of durability under freeze-thaw cycles (ASTM 2003).
CBR	ASTM D1883	Alternative strength assessment method (ASTM 2007).

While the aforementioned standards provide guidance on performance testing for a given soil-cement mixture, none offer assistance for the selection of optimum cement and water contents. Available mixture design guidelines recommend minimum cement contents based on soil type, trial batches are then mixed, compacted, cured and tested to assess performance, and the process is repeated iteratively until a target performance is met (Kézdi 1979). In this section, three such empirical methods, used to determine cement and water contents, are presented.

Portland Cement Association Method

The Portland Cement Association (PCA) provides a graphical method for the determination of cement content (PCA 1992). The method is based on statistical analysis of experimental results involving approximately 2,500 different soils (Kézdi 1979), and is *applicable primarily to sandy soils*. The PCA method is only valid for soils in which the weight percent of particles smaller than d = 50 μm does not exceed 50%, no more than 20% of the particles are smaller than d = 5 μm, and the soil organic content is low. Therefore, its use is limited to coarse-grained soils with very specific fines content and activity. If the maximum particle size is d < 4.75 mm (passing sieve #4) procedure A is followed; otherwise, procedure B is applies.

Procedure A:

Soil-cement specimens are prepared and tested under standard proctor compaction to determine the optimum water content and corresponding maximum bulk density. Once the maximum bulk density is determined, the minimum cement content can be estimated using Figure 17. A soil-cement mixture batch is then prepared using the minimum estimated cement content at the optimum compaction water content. Three cylindrical specimens (100-mm in diameter and 115-mm in height) are cast from the batch to determine the compressive strength. The specimens are cured in a 100% humidity atmosphere for 7-days. Four hours prior to testing, the specimens are submerged in water. The test results are then plotted and compared to the curve shown in Figure 18. If the strength values plot above the curve, the cement content is considered to be satisfactory; however, if the value plots below the curve, the cement content is considered to be insufficient for the mixture and must be increased. The entire series of tests must be repeated at a higher cement content.

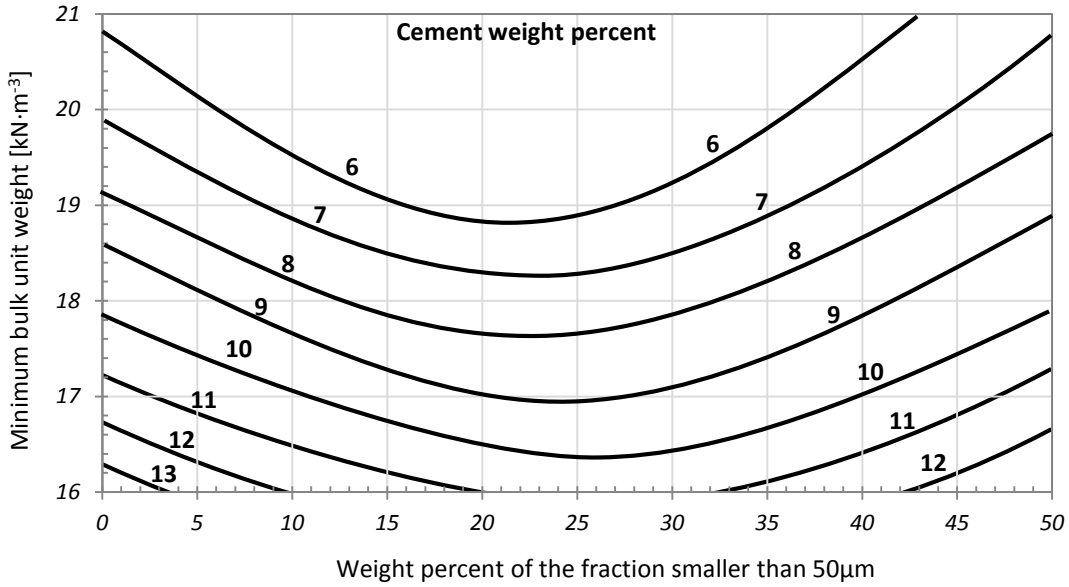


Figure 17. Graph for the approximate determination of the cement content; modified from Kézdi (1979).

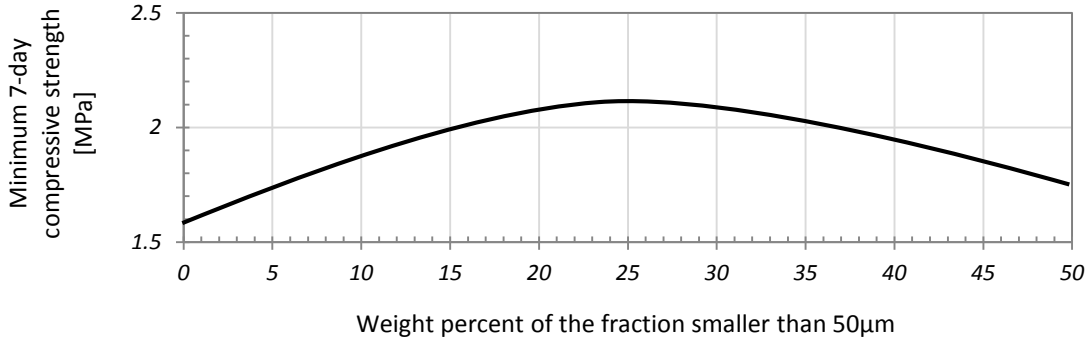


Figure 18. Minimum required value of the 7-day strength in soils containing particles no larger than 4.75 mm; modified from Kézdi (1979).

Procedure B:

The procedure for soils containing particles larger than d = 4.75mm (retained in sieve #4) requires also the determination of the optimum water content and maximum bulk density through standard proctor compaction tests. After the maximum bulk density is determined, the required amount of cement can be estimated based on Figures and nomographs (PCA 1992). A soil-cement mixture is then prepared at the optimum compaction water content and three cylindrical specimens are cast to determine the 7-day compressive strength. The specimen curing protocol is the same as that used in procedure A.

Methods based on soil classification

A simple procedure followed by multiple state departments of transportation (INDOT 2008) consist of selecting a first trial cement content based on the AASHTO soil classification, see Table 9.

Table 9. Cement requirement for stabilized soils (INDOT 2008).

AASHTO soil classification	Typical cement content range
A-1-a	3-5
A-1-b	5-8
A-2	5-9
A-3	7-10

Typically the cement content at the lower end of the range is selected for the first trial. Soil and cement are mixed and a standard proctor test is conducted on the mixture to obtain a value for the maximum dry density and the optimum compaction water content. The final step is to conduct compressive strength and California Bearing Ratio (CBR) tests on specimens compacted at 95% of the maximum dry density. Depending on the strength test results, the cement content is adjusted until the target performance is attained. Thus, the procedure may require several iterations of trial-and-error laboratory testing before a suitable cement content can be found.

NCHRP 144

The Texas Transportation Institute (TTI) recommends the selection of cement content based on the AASHTO soil classification and complementary pH tests. First trial cement contents can be selected using recommended ranges, see Table 10. The information in this table originates from a database of empirical evidence of soil-cement mixtures that exhibit satisfactory durability and performance (Little and Nair 2009). The pH test is conducted in a solution containing ten parts soil to one-part cement. If after 15 minutes the pH in the solution decreases below 12.0, the cement content must be increased. Reductions in pH are typically associated to the presence of organic matter in the soil, which can interfere with cement hydration. The maximum dry density of the mixture and the optimum compaction water content are determined according to the standard proctor compaction test at the selected cement content. The performance of the soil cement mixture is measured in terms of compressive strength and durability (wet-dry and freeze-thaw conditions). Durability tests are conducted following ASTM D559 (wet-dry) and ASTM D560 (freeze-thaw). These tests are highly subjective and their results typically exhibit high variability. Furthermore, it is hard to assess how accurately these tests reproduce field conditions. In order to overcome these shortcomings many state departments of transportations recommend minimum unconfined compressive strengths in lieu of durability tests (Little and Nair 2009). Note that both ASTM D559 and ASTM D560 standards have been withdrawn as of 2012 (ASTM 2003; ASTM 2003).

Table 10. Recommended trial cement contents (Little and Nair 2009).

AASHTO soil groups	Usual range in cement requirement		Estimate cement content, percent by weight
	Percent by volume	Percent by weight	
A-1-a	5-7	3-5	5
A-1-b	7-9	5-8	6
A-2	7-10	5-9	7
A-3	8-12	7-11	9
A-4	8-12	7-12	10
A-5	8-12	8-13	10
A-6	10-14	9-15	12
A-7	10-14	10-16	13

3.3.1.3 Assessment of Current Practices for Mixture Design

Cement soil improvement has been used in the U.S. for over 80 years and yet the process of mixture design remains in essence empirical. As presented earlier, available guidelines recommend minimum cement contents based on soil type, trial batches are then mixed, compacted, cured and tested to assess performance, and the process is repeated iteratively until target requirements are met (Kézdi 1979; Little and Nair 2009). This trial and error process fails to guarantee the determination of the optimum material proportions. Considering an average subgrade stabilization project spanning 30 ft in width and 1 ft in depth, a single percentile reduction in cement content translates into savings of 83 tons of cement per mile of road. Identifying the optimum mixture proportions for a desired performance level translates into significant economic benefits. Particularly considering that cement stabilization is expected to gain popularity with the advent of unconventional pavement structures, which rely on performance enhanced bases and subgrades as structural elements to reduce the thickness of the more expensive surface layers (Cortes and Santamarina 2012; Cortes et al. 2012).

In the absence of theoretically or phenomenologically based mixture design guidelines, the stabilization of plastic soils remains a mayor challenge for state transportation agencies. Despite knowledge of the variables that affect the performance of cement-stabilized soils, no mixture design method exists that can be applied universally i.e., to any soil. Thus, it seems there still is a missing component in the analysis of the problem. Here, we explore the key role of specific surface area on the behavior of soil-cement mixtures.

The Role of Specific Surface Area

Many of the observations presented in the literature can be readily understood in the context of surface area. Specific surface area (S_a) is the ratio of a particle's surface area to its mass. As the particle size (d) decreases, the specific surface area increases; thus, the specific surface of large particles i.e., sands and gravels, is comparatively small ($S_a < 0.04$ $m^2 \cdot g^{-1}$). In gap or well graded soils containing clayey fines ($S_a > 1$ $m^2 \cdot g^{-1}$), the relative contribution of coarse particles to the specific surface area of the soil ($S_{a\text{-}soil}$) is negligible, so $S_{a\text{-}soil}$ can be approximated by:

$$S_{a\text{-}soil} = f_c \cdot S_{a\text{-}fines} \quad [5]$$

where $S_{a\text{-}fines}$ is the specific surface of the fine grained soil fraction ($d < 75$ μm).

The nature of the physical processes that govern soil behavior changes when S_a exceeds about $1 m^2 \cdot g^{-1}$. While at low S_a, the behavior of soils is significantly impacted by self-weight and skeletal forces, at higher S_a, electrical and capillary forces gain prevalence (Santamarina et al. 2002). Both, soil-water interaction and cement hydration are mediated by surface level interactions.

Soil Hydration: dry clay particles have negatively charged surfaces, which attract cations to satisfy electroneutrality. When exposed to water, the cations adsorbed on the particle surface tend to diffuse away from the particle, in an effort to equalize the ionic concentration with the pore fluid (Mitchell and Soga 2005; Santamarina et al. 2001). The interactions between the electric field, generated by the particle surface, hydrated ions, and dipolar water molecules, lead to the formation of the diffuse double layer. The activity of water molecules within the double layer differs from that of free water molecules in the bulk fluid. Yet, as the distance from the particle surface increases, the electric field weakens, and activity increases. The water content associated to particle hydration (w_ϑ) is a direct function of the specific surface area of the soil (Santamarina et al. 2001):

$$w_\vartheta = S_a \cdot \vartheta \cdot \rho_w \quad [6]$$

where ϑ is the double layer thickness, and ρ_w is the density of water. It follows that the mass of water in the double layer is given by $m_{w\vartheta} = m_s \cdot w_\vartheta$. In coarse grained soils w_ϑ can be as low as 0.001% while in high specific surface clays it may reach values as high as 800%, see Table 11.

Table 11. Material properties for typical soils and cement (Cerato and Luteneggerl 2002; Mitchell and Soga 2005; Santamarina et al. 2001).

Material	*Diameter* d_g [m]	Aspect ratio $d_g \cdot t_g^{-1}$ [ ]*	*Specific surface* S_a [$m^2 \cdot g^{-1}$]	*Hydration water* w_ϑ [%]**
Montmorillonite	$1\times10^{-7} \sim 5\times10^{-7}$	100	400 ~ 800	600
Attapulgite	$4\times10^{-6} \sim 5\times10^{-6}$	400 ~ 1000	330	330
Illite	$1\times10^{-7} \sim 2\times10^{-6}$	10	80 ~ 100	90
Kaolinite	$3\times10^{-7} \sim 3\times10^{-6}$	3 ~ 10	10 ~ 20	15
Silt	$2\times10^{-6} \sim 6\times10^{-5}$	< 5	0.04 ~ 1.1	0.6
Sand	$6\times10^{-5} \sim 2\times10^{-3}$	< 5	0.001 ~ 0.4	0.2
Gravel	$> 2\times10^{-3}$	< 5	< 0.001	<0.001
Cement	$1\times10^{-6} \sim 5\times10^{-5}$	-	0.05 ~ 2.0	42

* The aspect ratio is the ratio of the particle diameter d_g to the particle thickness t_g.

** The hydration water content is computed according to $w_\vartheta = S_a \cdot \vartheta \cdot \rho_w$; where ϑ is the double layer thickness (assumed 100 Å), and ρ_w is the density of water (1000 kg·m^{-3}).

Cement hydration: upon exposure to water, calcium and hydroxide ions are released from the exposed surfaces of tricalcium and dicalcium silicate grains; resulting in the formation of a very alkaline pore fluid solution (pH > 12). When Ca^{2+} and OH^- ions reach a critical concentration, calcium hydroxide and calcium silicate hydrate (C-S-H) form. While calcium hydroxide crystallizes from solution, C-S-H forms a coating covering the cement grain. As hydration progresses the thickness of the hydrate layer increases, shielding the unreacted cement grain interior. At this point, access to water molecules takes place by diffusion through the C-S-H barrier. Hydration of the tricalcium aluminate and the ferrite phases follows a similar course, with ettringite playing an analogous role to C-S-H. The transition to diffusion-controlled hydration is characterized by a dramatic reduction in the reaction rate. Thus, cements with higher specific surface area (higher fineness) experience an increase in the rate of hydration and gain strength faster (Mindess and Young 1981).

Soil-cement hydration: upon initial exposure, water in a soil-cement mixture can be encountered in three distinct locations: hydrating soil particles (i.e., double layers), hydrating cement grains, and in the pore space between soil and cement particles. The total water content of the mixture (w_{mix}) can be defined as:

$$w_{mix} = m_w \cdot (m_s + m_c)^{-1} \qquad [7]$$

Over time, hydration of cement grains requires a continual supply of water molecules from the pore space; thus, it is reasonable to assume that water in the pore fluid will at some point be associated to cement hydration. The water-to-cement ratio would then be given by:

$$W/C = m_{wc} \cdot m_c^{-1} = C_c^{-1}[w_{mix}(C_c + 1) - w_\vartheta] \qquad [8]$$

where m_{wc} is the mass of water available for cement hydration, ($m_{wc} = m_w - m_{w\vartheta}$). Note that when w_ϑ is negligibly small W/C reduces to $W/C = m_w \cdot m_c^{-1}$, which implies that all the water in the mixture is available for cement hydration. This is clearly the case for coarse grained soils, such as the aggregates used in concrete ($S_a < 0.4$ m^2·g^{-1}); however, as the specific surface area of the soil increases, W/C decreases. Thus, the presence of silt and clay size particles effectively reduces the W/C. Depriving cement grains of access to water molecules hinders hydration and strength development, and prevents the mixture from attaining its full potential (Mindess and Young 1981).

Accommodating for the effects of w_ϑ is quite challenging because the thickness of the soil double layer is affected by the products of cement hydration reactions. Released Ca^{2+} interacts with clay surfaces through cation-replacement. At high ionic concentrations, Ca^{2+} can replace not only lower but also higher valence cations, resulting in either a decrease or an increase in ϑ. The high pH pore fluid environment, brought about by the release of OH^-, leads to deprotonation, which increases the negative surface electric charge density in clay particles (Stumm 1992). The high pH also promotes dissolution of silica (SiO_2) and alumina (Al_2O_3) in the clay, and alters the soil fabric surrounding the hydrating cement grains (Mitchell and Soga 2005).

3.3.2 Lime stabilization

The addition of lime to a fine-grained soil reduces its plasticity, increasing the soil strength while decreasing the extent of volumetric changes associated to fluctuations in water content. Lime is add-mixed to the soil in either powder or liquid form as either calcium oxide (CaO - quicklime) or calcium hydroxide ($Ca[OH]_2$ - hydrated lime). Both of these are high calcium lime, containing less than 5% magnesium oxide (MgO) or hydroxide ($Mg[OH]_2$). In some cases, dolomitic lime (containing up to 46% MgO or $Mg[OH]_2$) may be used (NLA 2004). The interaction between lime and soil particles is mediated by water. The byproducts of lime hydration alter the soil behavior in the short-term, so that a higher maximum dry density can be attain at lower optimum water content during compaction.

Lime is typically used in clayey soils of medium to high plasticity. Soils with a fines content of at least 25% and with a plasticity index of 10 or greater, are good candidates for lime stabilization (Little 1999). The presence of soluble sulfates (SO_2^{4-}) or organic matter (OM) often results in performance issues. Treatment of soils with soluble sulfate contents as low as 3000 ppm (0.3% by weight in a ten-to-one water to soil solution [Little 1999]) can result in long term swelling (for a detailed discussion on the treatment of sulfate bearing soils refer to section 3.3.5). Similarly, when the soil organic content exceeds 1% by weight, it becomes very difficult to attain target unconfined compressive strengths through lime stabilization (NLA 2006).

Performance enhancements through lime stabilization are attributed to the chemical and physical processes that take place once lime and soil are exposed to water. These processes result in immediate and long-term effects (Celauro et al. 2012). While immediate effects alter the soil fabric, long-term effects involve pozzolanic reactions that render soil particles cemented.

Soil-lime hydration: when CaO is exposed to water, it hydrates to form $Ca(OH)_2$, known as hydrated or slaked lime. The reaction is exothermic, which can result in evaporation of free water in the soil. Dissociation of $Ca(OH)_2$ into Ca^{2+} and $2(OH^-)$ takes place and the released ions begin to interact with the soil particles through ion exchange. Calcium ions replace adsorbed ions on the surface of clay particles, most commonly sodium (Na^+). This alters the thickness of the diffused double layer, reducing inter-particle repulsion and allowing the soil to flocculate, changing the soil fabric. Further changes in the soil are brought about by hydroxide ions. The increase in concentration of OH^- causes the pH of the pore fluid to increase, triggering the partial dissolution of silicates and aluminates from clay particles. Dissolution products then slowly combine over time with free calcium ions to form calcium-silicate-hydrates and calcium-aluminate-hydrates. These cementitious products increase the strength and stiffness of the soil in the long-term (Kézdi 1979).

Lime stabilized soil will exhibit a reduction in plasticity index, an increase in strength, and a reduction in its swelling capacity. Compaction parameters will also change. The mixture will exhibit an increase in the maximum dry density and lower optimum compaction water content. Thus, the soil becomes more workable and it is easier to compact. Short term gains in strength are associated to changes in soil fabric (flocculation), which lead to an increase in shear strength. Pozzolanic reactions provide the long-term strength gains and the development of cohesion. The stabilized soil becomes less sensitive to fluctuations in moisture content, i.e. swelling and shrinking. However, the effect is not permanent, and can be reverted over time. Available data indicates that effects are still present even after 12 years in service which is in the order of magnitude of a pavement service life (Prusinski and Bhattacharja 1999).

3.3.2.1 Mixture design

The performance of lime stabilized soils depends on the soil grain-size distribution and mineralogy, the lime content, and the water content. The ultimate goal of mixture design is to identify the proportions of lime and water required to meet the target soil performance. A mixture of lime and water adequately proportioned allows the lime to completely hydrate while maintaining the soil at or slightly above the optimum compaction water content. Sufficient lime must be added to increase the pore fluid pH to a level such that silicate and aluminate minerals can dissociate from the soil particles for pozzolanic reactions to proceed. The National Lime Association (NLA) recommends the following procedure: (1) initial soil evaluation, (2) approximate lime demand determination, (3) determination of the maximum dry density and optimum compaction water content, and (4) lime stabilized soil performance assessment (unconfined compressive strength and swelling).

First, the soil is classified according to: grain-size distribution (ASTM C136), plasticity index (ASTM D4318), organic content, and sulfate content (ASTM 2006; ASTM 2010). Once it has been determined if the soil is suitable for lime stabilization, the minimum lime content is determined. The minimum lime content is the smallest amount of lime that causes the pore fluid pH to reach 12.4 (NLA 2006) as per ASTM D6276 (ASTM 2006). Minimum lime contents typically range between 3 and 6% lime by weight (Zukri 2012).

After the lime content has been established, the soil-lime mixture is subjected to compaction testing (ASTM D698, AASHTO T99). Since the addition of lime alters the soil maximum dry density and optimum water content, it is necessary to determine the soil-lime mixture compaction parameters at the minimum lime content. A key deviation from the standard procedure is that the soil-lime-water mixture must be allowed to 'mellow' between 1 and 24 hours in an air tight, moisture proof bag prior to compaction testing (NLA 2006). Note that changes in plasticity index can also be measured using the soil after the mellowing period (Little 2000).

A minimum of two compacted specimens are then fabricated at three water contents: optimum, 1% above optimum, and 1% below optimum (Little 1999). The compacted soil-lime specimens are then sealed in an air tight, moisture proof packing, and allowed to cure for 7 days at 40 ºC (104 ºF). The recommended curing period and temperature have been selected to provide an accelerated representation of long-term cure under ambient field conditions (Little 2000). Following the curing process, the specimens are subjected to capillary soaking to evaluate its properties under reasonable field conditions. The specimens are placed on top of a saturated porous stone and wrapped with a wet absorptive fabric. The specimens are then placed in a water bath for 24 hours. The water level should reach the top of the porous stone but must not come in contact with the specimen itself (NLA 2006). Finally any volumetric expansion in the specimens is recorded and the unconfined compressive strength of each specimen determined according to ASTM D5102 (ASTM 2009).

3.3.2.2 Alternative mixture design considerations

The ability of the soil to react to the lime is key to a successful stabilization. In case of a low reactive soil (i.e., negligible dissolution of silicates and aluminates at the target pH) there are additives available to supply the silicates and aluminates required for pozzolanic reactions to proceed, including volcanic materials rich in amorphous silicate and aluminate particles, fly ash,

and lime kiln dust (NLA 2004). While lime remains the most popular calcium based stabilizer, substances rich in calcium oxides can be used as partial lime substitutes (Kamon and Nontananandh 1991). An example is calcium carbide residue, which contains not only nearly the same amount of calcium oxide as lime, but is also rich in pozzolans (Kampala and Horpibulsuk 2013).

3.3.2.3 Construction considerations

The selection of lime type depends on subgrade field moisture conditions. If the soil is significantly above the optimum compaction water content, using quicklime in powder form will aid in absorbing the excess water as the lime hydrates. If the soil is near the optimum compaction water content then hydrated lime should be used in order to prevent water depletion upon hydration. If the soil is significantly below optimum moisture content then lime slurry can be used to provide excess water. Climatic conditions during construction must also be considered in the selection of lime type. Construction under strong wind gusts may require the use of slurries since blown quick and hydrate lime powders can become a health and safety hazard.

The application of lime in the field depends on the type of lime being delivered. Lime should be spread uniformly throughout the stabilization depth and in the predetermined dosage. Scarifying the subgrade surface is not always necessary, but helps in the spreading process. Once the lime is placed on the subgrade it must be thoroughly mixed using a disk plough, a pulvimixer or similar equipment. Compaction should be delayed to allow for the soil-lime to mellow. Mellowing times can vary from hours to days depending on the presence and concentration of soluble sulfates in the soil.

3.3.3 Fly ash stabilization

Fly ash is one of the residues generated in the combustion of coal, most often for electric power generation. Fly ash consists of small hollow spheres of silicon, aluminum, and iron oxides and unoxidized carbon that rise with flue gases (Cokca 2001). As a geomaterial, it can be regarded as a non-plastic silt. Fly ash composition can vary considerably depending on the type of coal burned and the manner in which the power plant is operated (Cabrera and Woolley 1994). The use of fly ash in subgrade stabilization dates back to the 1950's, when its non-self-cementing version (class F) was frequently used in conjunction with lime (Terrel et al. 1979). Over time, the use of fly ash shifted, and the self-cementing class C fly ash gained preponderance; particularly in the mid-west and western states (White et al. 2005). The addition of fly ash to a fine-grained soil causes changes in swelling potential, plasticity characteristics and strength. The effects are similar to those of cement and lime, particularly in the short-term. Calcium, aluminum and iron ions released during the hydration of fly ash interact with the soil through cation replacement, reducing the double-layer thickness which results in a reduction in liquid limit and plasticity index. The relative change in plasticity and swelling induced by fly ash is less pronounced than that of lime because the concentration of calcium ions in fly ash is lower (Cokca 2001; Ferguson 1993), see Table 12. In the long-term, strength gain and cementation depend primarily on curing conditions and the concentration of free lime in the fly ash. Typical 7-day unconfined compressive strengths of class C fly ash stabilized soils is in the order of 100 psi but may be as high as 500 psi (Ferguson and Levorson 1999; Ferguson 1993). The optimal amount of stabilizer

required to induce a desired change in properties depends on the specific chemical composition of the fly ash used (Ferguson and Levorson 1999).

Fly ash is classified based on the amounts of calcium, silica, alumina, and iron present in the material. The relative amount of calcium to silicon, aluminum, and iron, oxides (hydration modulus) can be used as an index to predetermine the extent to which cementation will occur over time (Kamon and Nontananandh 1991). Class C fly ash (self-cementing) exhibits hydration moduli as high as 1.4 while the hydration modulus in class F fly ash rarely exceeds 0.3. Ash storage practices can also alter its properties. In addition to dry storage, fly ash is commonly found as hydrated fly ash (HFA) or conditioned fly ash (CFA). HFA is spread in thin lifts, watered, and compacted whereas CFA is mixed with water in a pug mill and then stockpiled (White et al. 2005). Available data from laboratory experimental studies on subgrade stabilization with self-cementing fly ash reveal that (White et al. 2005):

1. Typically used concentrations range between 10 and 20% fly ash by weight of dry soil.
2. The addition of fly ash leads to an increase in compacted dry density and lower optimum compaction water content.
3. The addition of fly ash decreases the swelling potential of the soil.
4. Long-term strength gain depends on compaction energy, compaction delay, curing time and temperature.
5. Sulfate bearing soils stabilized with self-cementing fly ash are susceptible to ettringite formation and swelling. *Note that certain ashes may contain sufficiently high concentrations of soluble sulfate for ettringite to form even when no sulfate is present in the soil*.
6. The use of hydrated fly ash or conditioned fly ash requires an increase in the stabilizer content and results in performance levels below those of dry fly ash.

3.3.4 Comparative performance of chemical stabilizers

Cement, lime and fly ash share a remarkably similar chemical composition; however, the relative mass fractions present in each are significantly different, see Table 12. This subtle difference results in profound changes in their interaction with soil particles, and governs both mixture design and level of performance of the stabilized subgrade. Key differences include: rate of change in mechanical performance (i.e., strength and/or stiffness) and attainable ultimate mechanical performance. Besides performance differences, the costs associated to each stabilizer can also be substantially different and vary over time. Under ideal circumstances, a pavement or geotechnical engineer should be able to determine the required mass fraction of each stabilizer needed to meet a desired level of performance, determine the costs associated to each alternative, and select the most economically favorable. During construction, the particulars of each stabilizer should govern the tolerances on water content, allowable compaction delays, curing times, etc., so that the target performance can be achieved.

Unfortunately, the empirical nature of available mixture design procedures for cement, lime, and fly ash stabilized subgrades forbid the use of optimization and deny the determination of optimum stabilizer mass fractions for comparison. Despite of this limitation, data available in the literature is presented to offer a very rough basis for comparison between the different

stabilizers. All information is presented in the context of two performance levels: modification and stabilization.

Table 12. Range in chemical composition for cement, lime, and fly ash (Cokca 2001; Kosmatka et al. 2002; Solanki and Zaman 2012).

Chemical Compound	Percent by weight [%]			
	Portland Cement	Lime	Class C Fly Ash	Class F Fly Ash
Calcium oxide [CaO]	62.8 - 72.2	72 - 74	15 - 40	1 - 12
Silica [SiO_2]	24.8 - 32.5	< 1	15 - 45	20 - 60
Alumina [Al_2O_3]	1.24 - 2.13	< 1	10 - 25	5 - 35
Ferric oxide [Fe_2O_3]	0.94 - 1.03	< 1	4 - 15	10 - 40
Hydration Modulus	2 - 2.3	>> 2.5	0.2 - 1.4	0.01 - 0.3

Modification (short-term effects)

Calcium oxides are present in cement, lime and fly ash. Calcium ions are released into solution upon hydration, triggering cation replacement in the diffused double layer of fine-grained soil particles. Cation exchange promotes absorption of calcium and the release of sodium and potassium which result in a decrease in the diffused double layer thickness and associated reduction in plasticity and swelling potential of the soil. The extent of the reduction in plasticity and swelling potential depends on the soil, and the type and amount of stabilizer used. In Figure 19, the change in performance of a high plasticity clay (PI = 52) for increasing amounts of stabilizer (cement, lime and two types of fly ash) is presented.

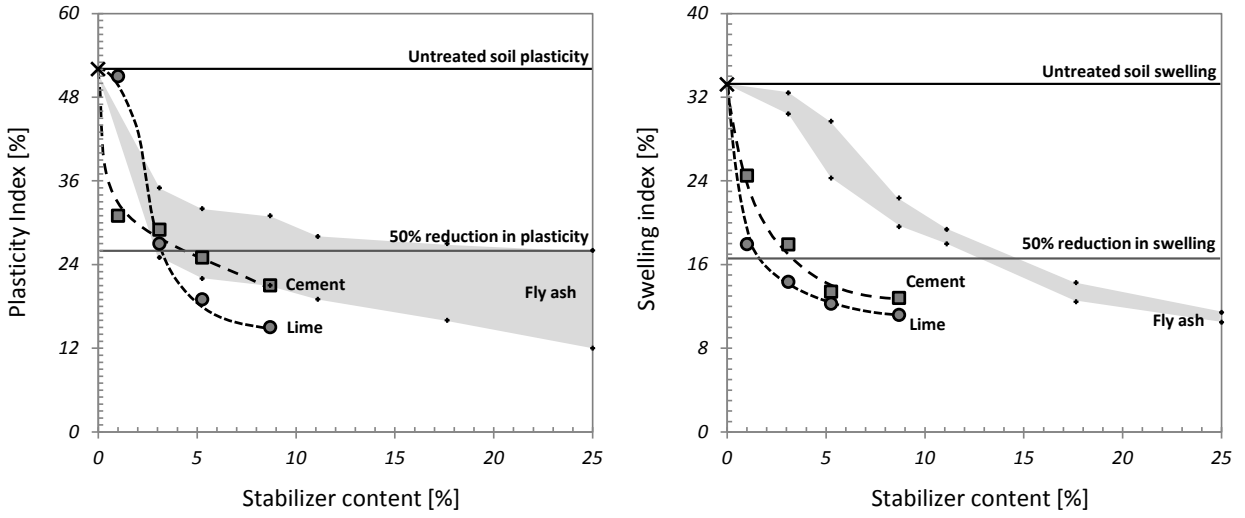


Figure 19. Changes in plasticity index and swelling potential induced by different stabilizers as a function of the mass fraction added to the soil; data from Cokca (2001).

It can be observed that a 50% reduction in plasticity (to PI < 26) can be readily attained using a relatively low mass fraction of stabilizer, about 3% stabilizer by dry mass of soil. Furthermore, the required amount of cement and lime is nearly the same while the required amount of fly ash can be significantly different (either 3% or 20%). Changes in the swelling index, defined as the

percentage of volume expansion, vary significantly depending on the choice of stabilizer. From the figure, a 50% reduction in swelling index would require either 1.5% lime, 3.5% cement, or 14% fly ash. Note that while the required mass fractions may only be relevant to the tested soil, the results highlight the fact that *different mass fractions of each stabilizer are needed to attain the same level of performance*. Also, note that for the given example, fly ash would have to be at least ten times cheaper than lime in order to offer an equivalent cost-benefit. Changes in plasticity and swelling capacity driven by cation exchange generally occur rapidly within an hour from exposure of the soil-stabilizer mixture to water.

The development of shear and tensile strength (i.e., flocculation and cementation) in soils stabilized with cement, lime, and fly ash varies very significantly. The mechanical performance of the soil is improved as a result of changes in soil fabric brought about by changes in the ionic concentration and pH of the pore fluid (Mitchell and Soga 2005; Santamarina et al. 2001). Flocculation and agglomeration change the texture of the fine-grained soil from that of a plastic to more of a coarse-grained soil (Prusinski and Bhattacharja 1999). In the process the plasticity index is further reduced, and the soil develops higher shear strength. The effects of flocculation and agglomeration are readily seen within several hours from exposure to water and are common to all three stabilizers.

Stabilization (long-term effects)

Further improvements in mechanical properties are realized through pozzolanic reactions that render the soil cemented. It is here where the effects of the stabilizers vary the most. Cement provides the highest gains in early strength (within a day), and continue to develop over months and even years (Prusinski and Bhattacharja 1999). The rapid gain in strength requires construction to proceed swiftly. Delay in compaction can significantly hamper ultimate performance and should be limited to under 2 hr. 24hr delays in compaction for cement treated soils can reduce 28th day unconfined compressive strength by over 40% (Christensen 1969). Strength development due to pozzolanic reactions in lime treated soils takes place at a much slower rate. Since lime is depleted of silica and alumina (see Table 12), it relies on partial dissolution of soil particles at high pH to provide the ingredients needed to form cementitious products. For this reason, the rate of strength gain can be slow and highly dependent on soil mineralogy and specific surface area. Delays in compaction do not affect lime-treated soils as much as they do cement-treated soils. In certain cases, no measurable changes in 28th day unconfined compressive strength could be observed for 24 hr delays in compaction (Christensen 1969). The wider range in chemical composition exhibited by fly ash makes it more difficult to anticipate the rate of change in strength or the tolerances in delayed compaction; however, maximum compaction delay times of 4 hours are recommended for class F fly ash (Ferguson and Levorson 1999; Terrel et al. 1979). Class C fly ash can react very strongly upon hydration; thus, it is recommended that compaction follows immediately after mixing (White et al. 2005).

Perhaps the most significant difference in performance between the calcium-based chemical stabilizers is in the ultimate performance achievable at 7 and 28 days. Lime has long been considered the stabilizer of choice for stabilization of high plasticity fine-grained soils; however, cement can outperform lime when appropriately dosed, compacted and cured. Bhattacharja and Bhatty (2003) treated two soils with plasticity indexes of 25 and 42 using cement and lime at 3, 6, and 9% stabilizer contents. The results of 7 and 28-day unconfined compressive strengths are shown in Figure 20. Cement outperformed lime at all stabilizer content ratios in the treatment of the 25 PI soil, in the extreme cases by as much as 6 times the 7-day

unconfined compressive strength and 4 times the 28-day. In the case of the higher PI soil, lime outperformed cement at 3% stabilizer only by 1.2 times the 7-day unconfined compressive strength and 1.4 times the 28-day. At higher stabilizer contents, cement outperformed lime in the high PI soil. Similar results have been reported by others (Christensen 1969; Prusinski and Bhattacharja 1999).

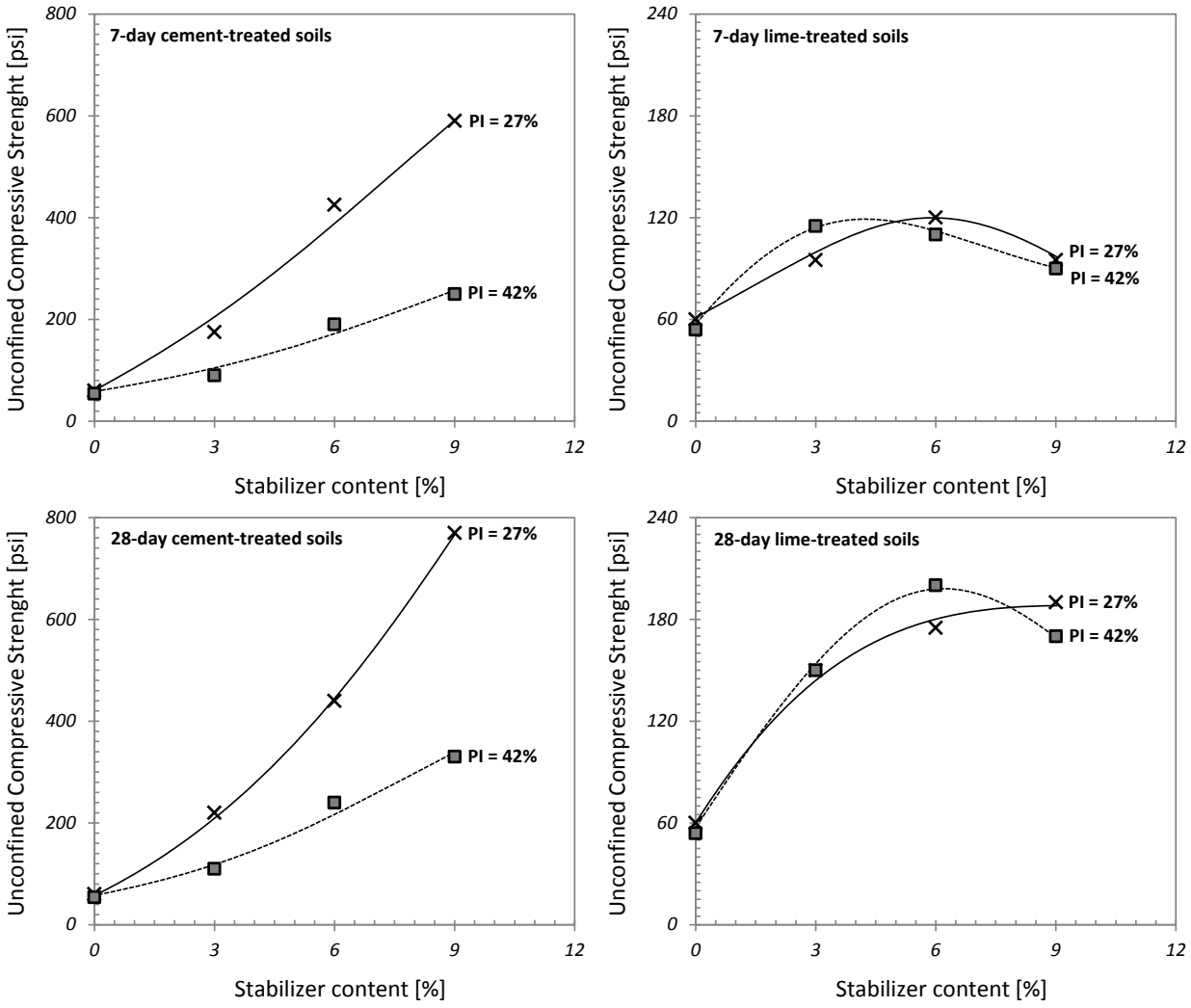


Figure 20. Evolution of unconfined compressive strength over time for cement and lime-treated soils of medium and high plasticity, data from (Bhattacharja and Bhatty 2003).

Despite limited information on stabilizer performance after several years in service, available data suggests that the effect of lime and cement on soil plasticity can remain even after 12 to 45 years in service. While cementation may be lost due to fragmentation of the stabilized subgrade, the somewhat permanent reduction in plasticity can limit swelling and shrinking of the soil in response to fluctuations in water content (Prusinski and Bhattacharja 1999; Roberts 1986).

3.3.5 Chemical stabilization of sulfate bearing sediments

In the presence of soluble sulfates, the chemical reactions triggered by the introduction of calcium oxides (i.e., lime, cement, and fly ash) and water can lead to the formation of ettringite and thaumasite. Ettringite and thaumasite crystals swell upon formation. When these minerals form, the stabilized soil undergoes significant volumetric changes. The effect of the expansive minerals formation is aggravated when crystals grow in concentrated regions as supposed to homogeneously distributed throughout the stabilized soil layer. In such cases differential volume expansion occurs, altering the curvature of the pavement surface and requiring immediate corrective action, which typically implies pavement demolition and reconstruction. The presence of soluble sulfates is only evidenced during long term pozzolanic reactions. Cation exchange, flocculation, and carbonation occur quickly (minutes to hours) and produce the effects expected of an appropriately stabilized soil. Thus, no immediate indication of potential heave are observed during construction or within months from opening to traffic (Hunter 1988).

3.3.5.1 Ettringite

Ettringite is a hydrous calcium alumino-sulfate mineral that precipitates in high pH environments. The balanced chemical reaction that describes ettringite formation is (Myneni et al. 1998):

$$6Ca^{2+} + 2Al^{3+} + 3SO_4^{2-} + 12OH^- + 26H_2O \leftrightarrow Ca_6\left[Al(OH)_6\right]_2(SO_4)_3 \cdot 26H_2O \qquad [9]$$

A common misconception regarding ettringite formation is that lime is the only stabilizer that causes it. Calcium ions (Ca^{2+}) can originate from the hydration of cement, lime, and/or fly ash; whereas aluminum (Al^{3+}) can originate from the tricalcium aluminate and the ferrite phases in Portland cement, amorphous aluminum oxides in fly ash, or the dissolution of clay minerals in the soil. Thus, *any calcium-based stabilizer, under certain circumstances, can trigger the formation of ettringite*. Common sources of sulfate (SO_4^{2-}) in soils include gypsum, anhydrite, and sulfides undergoing oxidation (i.e., pyrite).

The stability of ettringite is governed by pH, temperature, and the concentrations of dissolved CO_2 and H_2O. Ettringite can exist at pH values lower than 10.7 in association with gypsum and aluminum hydroxide yet it fully dissolves near pH = 7 (Myneni et al. 1998). The high pH values required to trigger ettringite formation are attained by the dissolution of calcium oxides and hydroxides from lime, cement, or fly ash. High temperatures (50~80 ºC) favor monosulfoaluminate over ettringite (Damidot and Glasser 1992). High CO_2 and low H_2O activity lead to the decomposition of ettringite into aragonite (Nishikawa et al. 1992). Note that the available literature reports on studies conducted in the quaternary system CaO-Al_2O_3-$CaSO_4$-H_2O. The introduction of additional chemical species, upon interaction with the soil in a natural system, can result in the formation of additional minerals and/or changes in the stability conditions.

The kinetics of formation (i.e., rate at which ettringite forms) depend on the availability of reactants. In the case of cement where the reactants come from structurally amorphous, uniformly blended, high specific surface area particles, the reactions take place rapidly and the amount of ettringite formed is only limited by the initial amount of reactants. When a fine-grained soil is treated with a calcium-based chemical stabilizer, reactant availability is governed by the mineralogy and dissolution properties of the soil. Soil minerals are not amorphous, and are not necessarily homogeneously distributed; thus, ettringite formation in cement, fly ash, and lime-treated soils can take place well after stabilizer hydration. Ettringite contributes positively

to a cemented material performance when it forms rapidly during hydration, because it becomes part of the cemented structure. Secondary ettringite formation, which occurs after the cemented structure has formed, disrupts the cemented matrix and impacts negatively its performance (Little and Nair 2009).

3.3.5.2 Thaumasite

Thaumasite is a calcium carbonate silicate sulfate hydrate mineral believed to form from as an isostructural alteration of ettringite in the presence of carbonates and soluble silica (Little and Nair 2009):

$$Ca_6\left[Al(OH)_6\right]_2(SO_4)_3 \cdot 26H_2O + 2(H_2SiO_4)^{2-} + 2CO_3^{2-} + O_2 \rightarrow$$
$$Ca_6\left[Si(OH)_6\right]_2(SO_4)_3(CO_3)_2 \cdot 24H_2O + 2Al(OH)_4^- + SO_4^{2-} + 4OH^- + 2H_2O \qquad [10]$$

Thaumasite may form in lime, cement and fly ash treated soils as a result of low temperature (T ~ 15 °C) sulfate reaction and intensive carbonation (Bensted 2000). Thaumasite may occupy only 45% of the volume of ettringite; thus, isostructural formation of thaumasite from ettringite can result in softening or collapse of the original structure (Warren and Reardon 1994). This is in agreement with field observations (Little and Nair 2009).

3.3.5.3 Soluble sulfates

Sulfur can be found in soils in the form of sulfide (S^{2-}) bearing minerals such as pyrite, marcasite and greigite, and in the form of sulfate (SO_4^{2-}) bearing minerals such as gypsum, anhydrite, barite, and jarosite (Burkart et al. 1999; Czerewko et al. 2003). Sulfate can originate from the dissolution of sulfate bearing minerals or from the oxidation of sulfide bearing minerals, both of which depend on environmental conditions such as pH, temperature, and pressure (Little and Nair 2009). Therefore, the mere presence of these minerals does not imply the availability of aqueous sulfate for ettringite/thaumasite formation.

Among the multiple naturally occurring sulfate sources, gypsum is the most commonly observed in natural sediments (Czerewko et al. 2003). Its solubility under normal atmospheric conditions is 2.58 grams per litter of water (Burkart et al. 1999). Because of its low solubility, the availability of water is critical in determining the extent of gypsum dissolution. In dry arid climates, the extent and duration of precipitation events becomes a controlling factor. It is unlikely that a gypsum deposit will be completely dissolved under a single rain event, releasing all available sulfate into the sediment. More likely, the release of sulfate ions will take place over time in small quantities following multiple precipitation events.

Because of its weak adsorption to soil minerals and the solubility of gypsum and other evaporites, soluble sulfate is relatively mobile in the environment (Little and Nair 2009). Two hydrologic processes account for the majority of sulfate migration in aqueous systems: (1) surface/subsurface runoff and (2) the upward migration of water from shallow water tables under the effect of capillary suction. In dry arid climates, sulfate deposits typically concentrate in the near surface as a result of evaporation processes. However, in rolling terrains, sulfate can accumulate in larger quantities along low-lying areas due to surface runoff (Little and Nair 2009).

3.3.5.4. Treatment

The stabilization of sulfate bearing sediments can be accomplished by regulating reactant concentrations and environmental conditions that favor ettringite/thaumasite formation. Calcium, sulfate, alumina, water, and high pH are all required for ettringite to form. In the case of thaumasite silica and carbonate are also essential. Controlling the relative concentrations of any of these constituents can limit the formation of the expansive minerals. Lime is typically proportioned in high quantities to maintain the high pH required to drive pozzolanic reactions in treated soils. Reducing the amount of lime used as to keep the pH below that required for ettringite formation can effectively control swelling; however, it can also negatively impact the long-term strength gain in the treated soil (Little and Nair 2009). Since the soil itself serves as a source for silica, alumina and sulfate it is impractical to control these reactants. Besides maintaining a low pH, an alternative method is to limit the amount of water. Ettringite requires copious amounts of water to form and grow. Limiting water intake during and after construction, through effective drainage, can also control swelling but may prove more challenging.

Alternatively, post-construction swelling can be minimized by doing the complete opposite, creating all the conditions to foster rapid ettringite formation. NCHRP 145 "Stabilization of sulfate rich soils" advocates for the addition of water beyond of what is required for optimum compaction, followed by mixing and a time allowance for mellowing (i.e., enough time for ettringite to form and for the concentration of soluble sulfate to drop below safe levels). The soluble sulfate content in a soil is the determinant factor in selecting a treatment option. Soluble sulfate contents below 3000 ppm (0.3% by weight in a ten-to-one water-to-soil solution [Little 1999]) rarely cause performance issues, except if additional sulfate continues to migrate and accumulates in the soil. Soils with SO_2^{4-} concentrations of up to 8000 ppm (0.8% by weight) can still be treated using lime, provided special construction procedures are followed. However, the use of lime in soils with soluble sulfate concentrations exceeding 8000 ppm is not recommended (Jones et al. 2010; Little and Nair 2009; NLA 2000; NLA 2006). The following treatment alternatives are recommended in NCHRP 145 for these three different cases (Little and Nair 2009):

1. *Traditional lime treatment* (soluble sulfates < 3000 ppm)
   If the total soluble sulfate concentrations are below 3000 ppm or 0.3% by dry weight of soil, lime stabilization should not pose a significant concern. While there is still potential for swelling, its magnitude can be managed by adequate moisture treatment as long as external sulfate migration can be prevented. *Whenever soluble sulfates are detected, lime slurry should be used instead of calcium oxide in order to ensure complete lime hydration at the time of construction*.

2. *Modified treatment* (3000 < soluble sulfates < 8000 ppm)
   In this range of soluble sulfates contents the soil exhibits a moderate risk of deleterious swelling caused by ettringite formation. Lime stabilization can be attempted only when accompanied by special mixture designs and construction practices. Lime slurry must be used and the moisture content needs to be increased above the optimum for compaction in order to foster ettringite formation. The soil-lime mixture must be allowed to 'mellow' for a period of time sufficiently long for ettringite to form prior to compaction. Both the mellowing time and the water content are determined by monitoring the soluble sulfate concentration over time. As ettringite forms, sulfate is consumed. The mellowing time at a given water content is that at which the sulfate content falls below 3000 ppm.

3. *Alternative treatment* (soluble sulfates > 8000 ppm)
   At high sulfate contents the risk for deleterious swelling is very high. *Successful stabilization of these soils is very challenging and should be attempted only with the outmost care*. Alternative treatments:
   a. Remove the sulfate bearing soil and replace it with a soil that does not require a chemical stabilization treatment to meet performance requirements.
   b. Blend the sulfate bearing soil with more competent material (i.e., mechanical stabilization).
   c. Dilute the sulfate concentration by mixing in a soil with no sulfates and stabilize using lime. Proportions should be determined based on performance testing to guarantee volume expansion does not exceed 2%.
   d. Additives are currently an active area of research to identify materials that could complement lime treatment.

Despite the detrimental effects of swelling associated to ettringite formation, the presence of sulfate bearing minerals is not inherently adverse to roadway construction, particularly when they are present in non-plastic soils. Tests conducted in the State of New Mexico have demonstrated that gypsum-anhydrite bearing silty fine sands in Carlsbad can be successfully used as engineered fills for subgrades and embankments (Lommler 2006). In the absence of the calcium ions and high pH produced by the addition of lime, no expansive minerals form. Properly placed, graded, and watered gypsum-anhydrite has been found to develop a crust that aids exposed areas in resisting wind and water erosion (Lommler 2006). The main challenge in using gypsum-anhydrite bearing soils as an engineered subgrade is controlling water content during compaction. Gypsum contains water molecules in its lattice structure; thus, when water is added some remains as structural water and does not contribute to pore or free water (i.e., compaction water). Accounting for the relative amounts of structural and free water is challenging and requires conducting water content testing at temperatures below recommended standards (60 °C as supposed to 110 °C). Performance compliance assessment of field compaction using density measurements affected by moisture (i.e., nuclear densometer) require special calibration, and it is recommended they be accompanied by mechanical performance measurements such as strength and stiffness tests (Lommler 2006).

3.3.6 Performance compliance assessment

Chemical stabilization lacks formal performance compliance assessment protocols. However, it is possible to instate available procedures, measurements, and observations to ensure that construction satisfies the requirements imposed by project designs and specifications. Quality control checks can be used to confirm material specifications such as: grain-size distribution, density, water content, plasticity, strength, and stiffness; or process specifications such as: percent additive used, mixing rate, spreading and compaction timing, and curing time (Schaefer and Berg 2014). Similarly, quality assurance checks can be implemented to monitor design and construction records, and to confirm performance in terms of thickness, density, strength and stiffness (Schaefer and Berg 2014). The most relevant performance criteria for chemical

stabilization include: the short-term reduction in plasticity index and swelling index, the long-term strength and stiffness gain as a function of time (e.g., unconfined compression and resilient modulus at 7 and 28 days) and volumetric changes in response to temperature and moisture cycles. These can be readily evaluated using a combination of laboratory and field tests performed during and after completion of construction as outlined below (Schaefer and Berg 2014):

Prior to construction

The stabilizer and the soil should be sampled and tested to confirm compliance with material standards in the case of the stabilizer, and to confirm mixture design assumptions in the case of the soil.

During construction

Monitoring the effectiveness of stabilizer spreading and mixing is a key aspect of quality control. For this purpose, stabilizer contents should be measured along transverse sections of the road and at multiple depths. Quantitative analysis of stabilizer content requires sophisticated laboratory tools which makes it expensive and time consuming (Qubain et al. 2006). The presence of lime and cement can be indirectly monitored in the field by conducting pH measurements (e.g., phenolphthalein test). While these measurements cannot confirm stabilizer content, they are helpful in confirming target depths of improvement (DOD 1994). Visual inspection of dug trenches can also be used to assess uniformity of mixture across the road and with depth (PCA 1992).

Water content is a key variable controlling the performance of chemically stabilized soils; therefore, monitoring of water content is fundamental to controlling and ensuring performance. Water content is typically measured by the oven-dry or nuclear gauge methods.

Post construction

Field density must be determined immediately after compaction to confirm that targets have been achieved. The field performance of the stabilized subgrade over time can be assessed via penetration type testing (e.g., DCP and HPT) or deflection type testing (e.g., dynamic plate load test) depending on target performance levels (Qubain et al. 2006; Vennapusa and White 2009; White et al. 2005). In addition, core samples can be collected for unconfined laboratory compression and resilient modulus testing.

4. NEEDS ASSESSMENT

The improvement of unstable subgrade soils not only presents technical challenges, but also operational challenges at multiple levels of decision making. Even when technical aspects of improvement methods are sufficiently understood to allow for their use with an acceptable degree of confidence in performance, operational aspects of implementation can still result in suboptimal stabilizer selection (higher cost for a given performance level) and in extreme cases pavement failure (stabilizer incompatibility or misuse). While in certain cases failure may arise from unknown factors which make it unavoidable; in other, failures can be attributed to misuse, and are therefore preventable.

In this section, available NMDOT specifications for subgrade preparation and soil improvement are critically reviewed at the light of findings from previous chapters. Shortcomings or issues are discussed and needs highlighted for review, and to facilitate the development of improved guidelines and specifications.

4.1 SECTION 207: SUBGRADE PREPARATION (2014)

Section 207 covers specifications for finishing subgrades prior to construction of pavement sub-base or base layers. The specification requires attainment of 95% compaction on the top 6 inches of the roadbed. Target performance is defined in terms of maximum dry density (AASHTO T180 [modified Proctor], Method D [TTCP Modified], and AASHTO T224), and R-value:

> Maximum dry density of all soil types encountered or used will be determined in accordance with AASHTO T 180 (Modified Proctor), Method D (TTCP Modified) and AASHTO T 224.
>
> Ensure the top two (2) ft of the finished Subgrade contains material with the design R-value. For Subgrade material not meeting design R-value, perform subexcavation in accordance with section 203.3.6.
>
> Compact the top six (6) inches of the Roadbed to 95% of maximum density.
>
> Ensure the soil moisture content (at the time of compaction) is from optimum to optimum minus five percent (5%). For soils with a PI of 15 or greater, ensure the moisture content of the soil at the time of compaction is from optimum moisture to optimum moisture plus four percent (4%).
>
> Conduct field density tests at locations according to Minimum Testing Requirements, in accordance with AASHTO T 310, or by other Department-approved methods.

It is presumed that the R-value of the soil at maximum compacted dry density is considered as the performance metric in the specification. However, only compaction testing is prescribed, and it is unclear if R-values are actually measured or estimated. The target R-value is established on a project-by-project basis by the pavement designer. If any soil within two feet from the surface fails to meet the design R-value a ‘subexcavation’ is required in accordance to section 203.3.6. Performance compliance assessment is conducted using field density testing using the nuclear gauge density test (AASHTO T310).

The specification does not necessarily address issues related to heterogeneous compaction which may arise when attempting to compact a layer thicker than recommended,

and/or when inappropriate compaction equipment is utilized. In such cases the upper part of the lift may attain satisfactory compaction while the lower portion remains under-compacted. Complimentary field testing with penetration strength probes (e.g., DCP and HPT) could be required to help identify deficiently compacted lifts during construction, when timely corrective actions can be taken. Mechanistic design methods require compacted layer performance inputs such as strength and stiffness. Therefore, emerging performance compliance assessment procedures will eventually need to incorporate resilient modulus, penetration resistance, and plate load testing.

Finally, Section 207 should only apply to projects or sections of projects were no subgrade performance issues had been observed in preliminary soil explorations. Soil improvement method-specific specifications (i.e., blending, cement, or lime) should be made available for projects where the need for improvement is known prior to the beginning of construction.

4.2 SECTION 203.3.6 SUBEXCAVATION

Section 203.3.6 requires removal of soils with R-value below the target performance specified by the pavement designer within 2 ft from the final subgrade surface.

> Remove unsuitable Materials from the Subgrade unless an alternative treatment is agreed upon by the Project Manager

By specifying the default action as removal and replacement at the cost of the Department, section 203.3.6 effectively discourages contractors from attempting the use of alternative subgrade improvement methods. When underperforming or unstable soils are found, Section 207 should not directly require removal and replacement (which in the end is just another improvement method), but rather trigger assessment of instability and determination of a suitable, cost effective method for improvement. A compilation of rapid deployment methods (including subexcavation) can even be pre-tailored to multiple levels of performance so that construction is not disrupted for extended periods of time.

4.3 SECTION 203.3.7 UNSTABLE SUBGRADE STABILIZATION

Section 203.3.7 offers no guidance on soil improvement other than an ambiguous assessment of short-term performance (proof rolling) and a requirement to meet an equivalent R-value approved by the State Pavement Design Engineer.

> Areas that have been proof rolled and lack sufficient stability in the opinion of the Project Manager shall be treated as unstable Subgrade.
>
> Prior to placement of the base material, the stabilized Subgrade shall be proof rolled with a roller having a minimum weight of 30 tons and shall exhibit no displacement when proof rolled.
>
> The Contractor shall stabilize unstable Subgrade using acceptable chemical or mechanical methods meeting an equivalent R-value as approved by the State Pavement Design Engineer.

As evidenced by the literature collected in Chapter 3, subgrade stabilization is not at all trivial. A specification for the construction of subgrade stabilization projects needs to provide sufficient detail to offer unequivocal guidance to contractors, as well as clear performance targets and performance compliance assessment methods to ensure an acceptable degree of confidence in the final performance of the stabilized subgrade. Unstable subgrade stabilization should be removed from section 203, and assigned a suitable 200 section of its own. A draft specification that combines Section 306 and Special Provision 306-B (Unstable Subgrade Stabilization) can speed up the development of such section.

4.4 SECTION 306-B: SPECIAL PROVISIONS FOR UNSTABLE SUBGRADE STABILIZATION

The intent of special provision 306-B is to cover the requirements for correcting unstable subgrades found in non-borrow sections. Unstable subgrades are defined as "subgrade that is soft, gummy, pumping, and/or displaces with applied loading." While descriptive, this definition is rather ambiguous and lacks objectivity. Instability needs to be defined in terms of measurable material properties that are associated to unacceptable mechanical performance i.e., plasticity index, fines content, swelling index.

If the soil is deemed stable (by the section 306-B definition), the presence of material with an R-value bellow design requirements triggers once again removal and replacement:

> When the top 2 feet of the subgrade is stable, but is below Design R-Value, sub-excavation and backfill with Design R-Value material or better shall conform to SECTION 203 - EXCAVATION, BORROW, & EMBANKMENT unless an alternative treatment is specified in the Contract or agreed upon by the Project Manager.

As mentioned earlier, when discussing section 203.3.6, this statement effectively discourages the use of alternative stabilization methods. The special provision goes on to specify that even if a higher R-value is attained after stabilization, the increased performance of the subgrade should not be used to alter the original pavement design:

> Subgrade modified by this specification is for stabilization only and is not considered in the structural design of the pavement structure; thus no modification of the pavement structure shall be made.

While conservative, this provision limits the true potential of subgrade stabilization, and gives an unfair advantage to certain stabilization methods. For example, an increase in R-value for a certain soil can be attained by blending and compaction, or by the addition of cement. Blending and compaction may prove cheaper than cement upfront; however, the use of cement can result in a better performing subgrade that allows for reductions in the thicknesses of base and surface layers. Thus, if the cost savings associated to reductions in more expensive pavement layers are factored into the cost-benefit analysis, cement may become the more advantageous option. Alternatively, if the original pavement design must be maintained, the enhanced subgrade performance can be input to estimate the increase in estimated pavement service life derived from the improved subgrade. If a dollar value is assigned to the additional service life, a fairer comparison between the multiple improvement alternatives can be made.

The special provision allows contractors to select a subgrade improvement method under special circumstances:

> The Contractor may choose any of the options contained in this specification unless otherwise indicated on the plans or in the contract documents and at the contractors option, change to the other option during that project at no additional cost to the Department.

The alternatives are chemical stabilization (option A) or mechanical stabilization (option B):

> Option A - Chemically Stabilized Subgrade - This work shall consist of Chemical Stabilization to stabilize the subgrade. Chemically Stabilized Subgrade includes, but is not limited to, Portland Cement, Lime, etc. or combinations thereof. Chemical testing shall include sulfate content of the subgrade by a testing lab approved by NMDOT Materials Lab. Where sulfate content of the subgrade reaches 2,000 or more parts per million (ppm), chemical stabilization shall not be utilized. Results of the sulfate content lab testing and mix design shall be provided to the NMDOT prior to utilization of the chemical stabilized subgrade alternative. The contractor shall allow the NMDOT 3 working days to review and comment on the sulfate content lab testing and mix design prior to commencing the chemical subgrade stabilization.
>
> Option B – Mechanically Stabilized Subgrade - This work shall consist of Mechanical Stabilization to stabilize the subgrade. Mechanically Stabilized Subgrade includes, but is not limited to, ripping/drying/replacing, excavation and replacement (replacement material shall meet the design R-value or better within the top 2 feet of subgrade), aggregates, rock, underdrains, geotextiles, and/or reinforcement materials, or combinations there of.

Neither option offers sufficient guidance for construction, minimum performance levels, and performance compliance assessment methods. In the case of chemical stabilization, the focus on sulfate bearing soils reflects negative experiences with recent failures. However, other key aspects of chemical stabilizer implementation are missing. For instance, restrictions on environmental conditions at the time of placement (i.e., temperature, precipitation, wind, etc.), stabilizer application method (powder, slurry), maximum allowable compaction delay times, curing times and conditions, among others. By neither providing clear step by step guidance, or unambiguous performance metrics and assessment methods for acceptance, the special provision creates an environment of uncertainty that can easily discourage all but few specialized contractors from attempting subgrade stabilization by any means other than removal and replacement (i.e. subexcavation).

The acceptance criterion fails to capture the essential differences between available stabilization methods:

> The stabilized subgrade shall meet the requirements of SECTION 303 - BASE COURSE (QLA), subsection 303.3.1 Preparation of Subgrade. Prior to placement of the base material, the stabilized subgrade shall be proof rolled with a roller having a minimum weight of 27 tons and shall exhibit no displacement when proof rolled. Stabilized subgrade locations that continue to exhibit displacement are to be corrected at no additional cost to the Department

Section 303.3.1 requires removal of loose and deleterious material from the subgrade, requires minimum density within the upper 6 inches of the subgrade, and specifies proof rolling to assess performance. All of these can be attained, under favorable environmental conditions, without the

subgrade being actually stable. Better acceptance criteria must be developed on a method specific basis.

4.5 DEVELOPMENT OF GUIDANCE DOCUMENTS AND DRAFT SPECIFICATIONS FOR SUBGRADE IMPROVEMENT

Rapid deployment of best practices in subgrade improvement can benefit the state of New Mexico tremendously in terms of economic savings both up front, and in avoidable early maintenance and reconstruction. However, technical knowledge must be transformed into straight forward guidance for selection, design, construction, and inspection of subgrade improvement projects. Each of these four phases is key to the successful implementation of any of the improvement methods discussed in Chapter 3. Guidance needs to be accompanied by clear, technically-sound specifications to enforce them and to establish expected levels of performance and performance compliance assessment methods.

It is also important to understand that the stakeholders in each phase do not necessarily share a common set of technical skills and/or project priorities. For instance, stabilizer-soil compatibility may be irrelevant to a construction crew already assigned to an improvement project; however, detailed procedural implementation information is a basic necessity. Developing a single volume guide for subgrade improvement, while comprehensive, may result in information overloading and limit its usefulness. A much better alternative would be the parallel development of a selection guide, a design guide, a construction guide, and an inspection guide. Each of which contains information only relevant to the task at hand; yet, share a common core so that all are consistent and technically sound.

5. STABILIZATION SELECTION GUIDE

The stabilization selection guide is a tool developed for decision makers to assess the compatibility of soil stabilization methods with a particular soil of known plasticity index (PI) and fines content (f_c). The guide is an interactive M.S. Excel ® spread sheet which receives as input the soil properties (PI and f_c) and the coordinates of the project location, and automatically assesses stabilization compatibility, potential performance, and costs. Thus, with minimal work, decision makers can readily identify the best performing alternatives for the money.

The selection guide is not meant to be used for design, only for feasibility analysis. There are multiple assumptions made to simplify the analysis and comparison of treatment alternatives that are inherently different from each other. Therefore, this section is dedicated to stating all assumptions made in the development of the decision guide.

5.1. COMPACTION

Compaction is always recommended when the subgrade soil exhibits plasticity index lower than 10 or fines content lower than 12%. The PI selection criterion marks the boundary between low and medium plasticity soils. Higher plasticity soils, especially those with PI>20, are prone to exhibit swelling and shrinking, which over time degrades the mechanical performance of the compacted subgrade. Therefore, it is recommended that compaction be accompanied by another stabilization method when dealing with fine-grained soils that exhibit PI>10. The fc selection criterion draws from the unified soil classification system which recognizes that at fines contents higher than 12% the fine-grained soil fraction may exert control over the mechanical performance of the soil. Conversely, at lower fines contents, the mechanical performance of the soil is governed by the coarse-grain soil skeleton regardless of the properties (e.g., plasticity index) of the fines. While compaction alone may still be a suitable alternative in soils that exhibit PI > 10 and 12% < fc < 50% it should be evaluated on a case by case basis.

5.1.1. Estimated performance level

Performance estimates are given in terms of the resilient modulus of the compacted subgrade. NMDOT tabulated R-values (Lenke et al. 2006) are used as the primary performance estimate for compacted soils, see Table 1. Tabulated R-values are converted to resilient modulus M_r using the correlation proposed in the AASHTO Guide for Design of Pavement Structures (1993):

$$M_r\left(psi\right) = 1000 + 555R \qquad [11]$$

Maximum and minimum values presented in Table 13 are used as the estimated upper and lower bounds for performance of the compacted soil. The values in the table have been reduced by a factor of 0.6.

Note that the estimated performance is offered to provide an order of magnitude type estimate so that multiple stabilization alternatives can be compared. The values presented are not to be used in design documents in lieu of actual laboratory measured values.

Table 13 Resilient modulus estimates based on NMDOT R-values.

PI	Resilient modulus estimates [psi]											
	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7	A-3	A-4	A-5	A-6	A-7-5	A-7-6
0	25,000	23,983	19,580					16,192	15,853			
1	24,661	23,654	18,563					15,515	14,837			
2	24,661	22967	17,886					14,498	13,482			
3	24,322	22290	16,870					13,821	12,466			
4	24,322	21612	16,192					12,805	11,450			
5	24,322	21273	15,176					12,127	10,433			
6	23,983	20,596	14,498					11,111	9,417			
7			13,482					10,095	8,401			
8			12,805					9,417	7,385			
9			11,788					8,401	6,369			
10			11,111					7,723	5,352			
11					10,433	11,788				4,675	3,660	2,981
12					10,095	11,450				4,675	3,660	2,981
13					9,756	11,111				4,336	3,660	2,981
14					9,417	10,772				4,336	3,660	2,981
15					9,078	10,095				4,336	3,320	2,981
16					8,740	9,756				3,997	3,320	2,981
17					8,401	9,417				3,997	3,320	2,642
18					8,062	9,078				3,997	3,320	2,642
19					7,723	8,740				3,659	3,320	2,642
20					7,385	8,062				3,659	2,981	2,642
21					7,046	7,723				3,320	2,981	2,642
22					6,707	7,385				3,320	2,981	2,642
23					6,369	7,046				3,320	2,981	2,642
24					6,030	6,707				2,981	2,981	2,642
25					5,691	6,030				2,981	2,642	2,642
26					5,352	5,691				2,981	2,642	2,642
27					5,012	5,352				2,642	2,642	2,642
28					4,675	5,012				2,642	2,642	2,642
29					4,336	4,336				2,642	2,642	2,642
30					3,997	3,997				2,304	2,304	2,642
31					3,659	3,659				2,304	2,304	2,642
32					3,320	3,320				1,965	2,304	2,642
33					2,981	2,981				1,965	2,304	2,642
34					2,642	2,304				1,965	2,304	2,642
35					2,304	1,965				1,626	1,965	2,304
36					1,965	1,626				1,626	1,965	2,304
37					1,626	1,287				1,626	1,965	2,304
38					1,287	950				1,287	1,965	2,304
39					950					1,287	1,965	2,304
40										1,287	1,626	2,304

No correlations available

No correlations available

5.2. BLENDING

Coarse-grained material can be blended into a soil to reduce its fines content and mitigate the effect of the fines fraction on the performance of the blended mixture. Performance can be optimized by selecting a coarse-grained soil with a grain-size-distribution that complements that of the original soil, as to yield a well-graded blend. While theoretical means of identifying a complementary soil are available, it is typically unfeasible to tailor a bled on a project-by-project basis. Improvements in performance can none the less be obtained by blending coarse-grained material ($f_c = 0$) of any gradation to reduce the fines content of the blend. If sufficient material is added to bring $f_c \leq 12\%$, the blended soil can be stabilized through compaction alone.

5.2.1. Estimated performance level

Performance estimates are given in terms of the resilient modulus of the compacted blended soil. The blended mixture is characterized according to the AASHTO soil classification system and the performance is estimated using Table 13.

Note that the estimated performance is offered to provide an order of magnitude type estimate so that multiple stabilization alternatives can be compared. The values presented are not to be used in design documents in lieu of actual laboratory measured values.

5.2.2. Cost estimate

The mass of coarse-grained soil needed to reduce the fines content of a blend to 12% can be estimated using the following equation:

$$m_{cs} = \frac{f_c - 0.12}{0.12} \times \frac{\gamma_{cs} \times V_T}{g} \qquad [12]$$

m_{cs} = mass of clean, dry, coarse-grained soil [tons],
f_c = fines content [ ],
γ_{cs} = dry unit weight of clean coarse-grained soil [$kN \cdot m^{-3}$],
V_T = treatment volume (length × width × depth) [m^3],
g = acceleration due to gravity [$9.81 m \cdot s^{-2}$].

The costs associated to blending treatment can be estimated using the following equation:

$$C_B = m_{cs}.c_{cs} + m_{cs}.c_t.x_{cs} + c_m.x_m + c_h.t \qquad [13]$$

C_B = total cost [$],
m_{cs} = mass of clean, dry, coarse-grained soil [tons].
c_{cs} = cost of aggregate [$\$ \cdot ton^{-1}$],
c_t = transportation cost [$\$ \cdot ton^{-1} \cdot mile^{-1}$],

x_{cs} = distance to aggregate source [miles],
c_m = blending equipment mobilization cost [$·mile^{-1}],
x_m = distance to equipment provider [miles],
c_h = hourly equipment operation cost [$·hr^{-1}],
t = operation time [hr].

5.3. CEMENT

Cement can reduce the plasticity of a soil upon mixing and increase significantly the soil strength and stiffness after curing. Cement treatment can be effectively applied to soils ranging from sands and silty sands, to silts, and lean clays. Typical cement contents used in soil stabilization projects vary between 3% and 10%

5.3.1. Estimated performance level

The mechanical performance of soil-cement mixtures is often evaluated through laboratory testing on a case by case basis. Because of the unavailability of performance predicting models, it is difficult to estimate performance without actually testing the material. However, data reported in the literature can be used to provide rough estimates of performance in terms of unconfined compressive strength, see Figure 21.

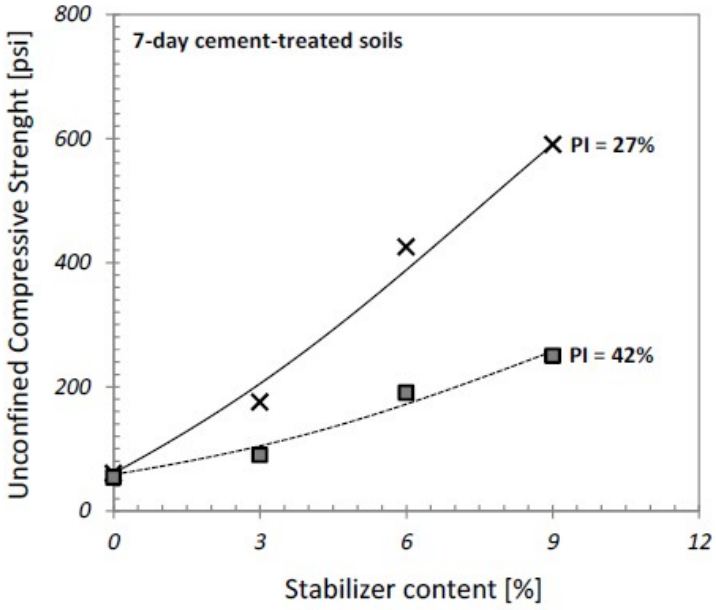


Figure 21. 7-Day unconfined compressive strength of high and very high plasticity soils (Bhattacharja and Bhatty 2003).

From the data presented in Figure 21, it is possible to draw ranges in performance for soils with PI < 27%, 27% ≥ PI < 42% and PI ≥ 42%. For instance, treating a soil of PI < 27% with 6% cement would be expected to yield a 7-day unconfined compressive strength above 380 psi. If the soil exhibited PI between 27% and 42%, the addition of 6% cement would yield an

unconfined compressive strength between 160 and 380 psi, and below 160 psi if it exhibited PI > 42%. To allow for performance comparison with the previous treatment options, the 7-day unconfined compressive strengths are used to estimate the resilient modulus of the soil-cement mixture (Thompson 1986):

$$M_r\left(psi\right) = 440 q_{u-7day} + 0.28\left(q_{u-7day}\right)^2 \quad [14]$$

Mr = resilient modulus [psi],
$q_{u\text{-}7day}$ = 7-day unconfined compressive strength [psi].

Note that because of the limited amount of data used for the unconfined compressive strength estimates and the empirical nature of the correlation between M_r and q_u, these values should only be used as order of magnitude estimates and never for design in lieu of laboratory test results.

Test results conducted on fine grained soils with plasticity indexes of 10% and 47% are presented in Table 14:

Table 14 Measured small strain stiffness of soil-cement mixtures

PI [%]	Cc [%]	M_{max} [ksi]	E_{max}* [ksi]
10	3	725	431
	6	897	534
47	3	574	342
	6	880	524

**** Assuming a Poisson's ratio of 0.36***

The values of constrained modulus (M_{max}) are recovered from p-wave velocity measurements, which capture the behavior of soil-cement mixtures in the small strain regime. Young's modulus values (E_{max}) are estimated from the constrained modulus measurements and assuming a Poisson's ratio of 0.36. As the strain level increases, the material experiences structural degradation and the stiffness decreases. Therefore, the values presented are representative of the soil-cement mixture response at low stresses (typical of conventional flexible and rigid pavement subgrades). At low strains, the mechanical response of soil-cement mixtures is linear-elastic and the resilient modulus is equal to the maximum Young's modulus. For more details see Appendix A.

5.3.2. Cost estimate

The mass of cement needed for treatment will vary depending on the desired level of performance. The mass of cement depends on the choice of cement content as follows:

$$m_c = C \times \frac{\gamma_{s-c} \times \mathrm{V_T}}{g} \quad [15]$$

m_c = mass of cement [tons],

C = cement content [ ],
$\gamma_{s\text{-}c}$ = dry unit weight of the compacted soil-cement mixture [$kN \cdot m^{-3}$],
V_T = treatment volume (length × width × depth) [m3],
g = acceleration due to gravity [$9.81 m \cdot s^{-2}$].

The costs associated to cement treatment can be estimated using the following equation:

$$C_c = m_c.c_c + m_c.c_t.x_c + c_m.x_m + c_h.t \quad [16]$$

C_c = total cost [$],
m_c = mass of cement [tons],
c_c = cost of cement [$\$ \cdot ton^{-1}$],
c_t = transportation cost [$\$ \cdot ton^{-1} \cdot mile^{-1}$],
x_c = distance to cement source [miles],
c_m = blending equipment mobilization cost [$\$ \cdot mile^{-1}$],
x_m = distance to equipment provider [miles],
c_h = hourly equipment operation cost [$\$ \cdot hr^{-1}$],
t = operation time [hr].

5.4. LIME

Lime can also reduce the plasticity of a soil upon mixing and increase its soil strength and stiffness. However, strength gains in lime treated soils take place over longer periods of time. Unlike cement, lime is only effective at treating fine-grained soils with PI > 20. Typical lime contents used in soil stabilization projects vary between 3% and 15%.

5.4.1. Estimated performance level

The mechanical performance of soil-lime mixtures is often evaluated through laboratory testing on a case by case basis. The unavailability of performance predicting models makes it difficult to estimate performance without actually testing the material; however, data reported in the literature can again be used to provide rough estimates of performance in terms of unconfined compressive strength, see Figure 22.

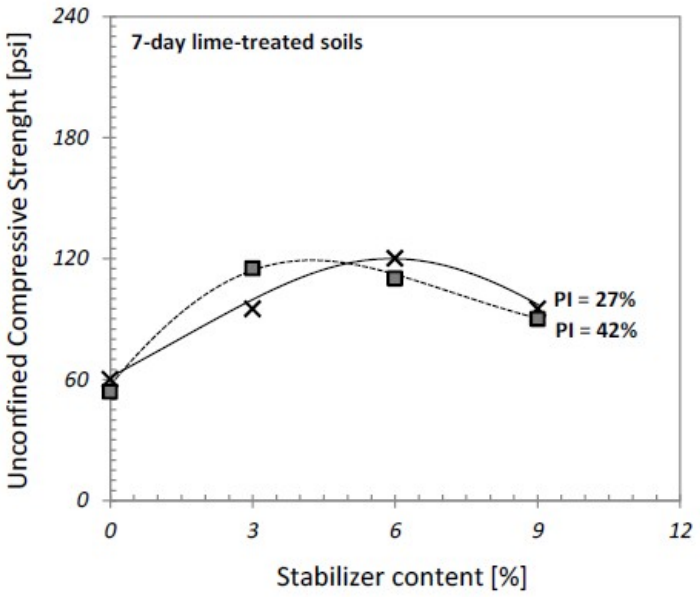


Figure 22. 7-Day unconfined compressive strength of high and very high plasticity soils treated with lime (Bhattacharja and Bhatty 2003).

To allow for performance comparison with the previous treatment options, 7-day unconfined compressive strengths are used to estimate the resilient modulus of the soil-cement mixture (Little 2000):

$$M_r(psi) = 124 q_{u-7day} + 9980\, psi \qquad [17]$$

M_r = resilient modulus [psi],
$q_{u\text{-}7day}$ = 7-day unconfined compressive strength [psi].

Note that because of the limited amount of data used for the unconfined compressive strength estimates and the empirical nature of the correlation between Mr and qu, these values should only be used as order of magnitude estimates and never for design in lieu of laboratory test results.

5.4.2. Cost estimate

The mass of lime needed for treatment will vary depending on the desired level of performance. The mass of lime depends on the choice of stabilizer content as follows:

$$m_L = L \times \frac{\gamma_{s-L} \times V_T}{g} \qquad [18]$$

m_L = mass of lime [tons],
L = lime content [ ],
$\gamma_{s\text{-}L}$ = dry unit weight of the compacted soil-lime mixture [$kN \cdot m^{-3}$],

V_T = treatment volume (length × width × depth) [m^3],
g = acceleration due to gravity [9.81m·s^{-2}].

The costs associated to cement treatment can be estimated using the following equation:

$$C_L = m_L.c_L + m_L.c_t.x_c + c_m.x_m + c_h.t \quad [19]$$

C_L = total cost [$],
m_L = mass of lime [tons],
c_L = cost of lime [$·$ton^{-1}$],
c_t = transportation cost [$·$ton^{-1}$·$mile^{-1}$],
x_s = distance to lime source [miles],
c_m = blending equipment mobilization cost [$·$mile^{-1}$],
x_m = distance to equipment provider [miles],
c_h = hourly equipment operation cost [$·$hr^{-1}$],
t = operation time [hr].

5.5. REMOVAL AND REPLACEMENT

Removal of unstable soil and replacement with a better performing material can be an effective means of solving subgrade performance issues. This is recognized in the 2014 NMDOT specifications (section 203.3.6). However, under certain circumstances, removal and replacement, or subexcavation, can result in considerable costs; particularly when sources of replacement material cannot be found nearby. The spread sheet can be used to compare the costs and expected performance of subexcavation jobs against alternative stabilization methods in order to ensure the selection of the optimal alternative.

5.5.1. Estimated performance level

The mechanical performance of the replacement soil is given in terms of the compacted soil resilient modulus. Estimates are obtained using the AASHTO soil classification of the replacement soil. M_r values are presented in Table 13.

5.5.2. Cost estimate

The mass of soil for removal and the mass of soil required for replacement are estimated according to:

$$m_{out} = \frac{\gamma_{d-field} \cdot V_T}{g} \quad [20]$$

$$m_{in} = \frac{\gamma_{d-comp} \cdot V_T}{g} \quad [21]$$

m_{out} = total mass of soil for removal [tons],

m_{in} = total mass of replacement soil [tons],
$\gamma_{d\text{-}field}$ = dry unit weight of removal soil in-situ [$kN \cdot m^{-3}$],
$\gamma_{d\text{-}comp}$ = dry unit weight of compacted replacement soil [$kN \cdot m^{-3}$],
V_T = treatment volume (length × width × depth) [m^3],
g = acceleration due to gravity [$9.81 m \cdot s^{-2}$].

The unit weights of the replacement soils are estimated based on soil classification. These values should not be used for design or budgeting in lieu of laboratory measurements.

The costs associated to removal and replacement can be estimated using the following equation:

$$C_R = m_{out}.c_r + m_{out}.c_{rt}x_r + m_{out}.c_d + m_{in}.c_{rs} + m_{in}c_t.x_s \quad [22]$$

C_R = total cost [$],
c_r = cost of soil excavation [$\$ \cdot ton^{-1}$],
c_{rt} = transportation cost to disposal location [$\$ \cdot ton^{-1} \cdot mile^{-1}$],
x_r = distance to disposal location [miles],
c_d = disposal/storage cost [$\$ \cdot ton^{-1}$],
c_{rs} = cost of replacement soil [$\$ \cdot ton^{-1}$],
c_t = aggregate transportation cost [$\$ \cdot ton^{-1} \cdot mile^{-1}$],
x_s = distance to aggregate source[miles].

5.6. GEOGRID

The introduction of a geogrid beteween the base or subbase layer and the subgrade results in an improvement in the pavement performance; however, the subgrade itself experiences no physical changes. Thus, the mechanical performance of a subgrade in a geogrid reinforced pavement section is almost the same as in an unreinforced section. Geogrid designers relay on aggregate base thickness reduction factors to compare the performance of reinforced and unreinforced pavement sections. This method can be used in conjunction with the AASHTO 1993 flexible pavement design equations to estimate the 'effective improvement' in the subgrade resilient modulus. The effective change in resilient modulus can be obtained by solving the design equation iteratively considering the change in the base structural number caused by the thickness reduction.

5.6.1. Estimated performance level

A parametric study was conducted to estimate the effective change in subgrade performance based on a reduction in the aggregate base layer. Three different pavement sections were evaluated for three different initial subgrade resilient moduli. The pavement sections were selected to represent low (300,000 ESAL), medium (3,000,000 ESAL), and high (30,000,000 ESAL) traffic volume. Subgrades with resilient modulus of 1,500, 3,000 and 4,500 psi were selected for each pavement section. Thickness reductions of 5, 10, 20 and 50% were simulated to estimate effective changes in the subgrade resilient modulus.

The pavement structures analyzed consisted of three layers placed over a subgrade: hot nix asphalt (HMA), base, and sub-base. Design parameters used included: R = 95%, S_o = 0.35, ΔPSI = 2.1, E_1 = 400,000 psi, E_2 = 30,000 psi, E_3 = 11,000 psi, a_1 = 0.42, a_2 = 0.14, a_3 = 0.08, m_2 = 1.2, m_3 = 1.2. Where R is the design-performance reliability, S_o is the combined standard error of the traffic prediction and performance prediction, ΔPSI is the difference between the initial and the terminal design serviceability index, E_1 is the HMA modulus of elasticity, E_2 is the base modulus of elasticity, E_3 is the sub-base modulus of elasticity, a_1 is the HMA relative strength layer coefficient, a_2 is the base relative strength layer coefficient, a_3 is the sub-base relative strength layer coefficient, m_2 is the base drainage coefficient, and m_3 is the sub-base drainage coefficient.

A total of 36 simulations were conducted and the results are presented in Table 15. In each data cell, the sub base thickness reduction fraction assumed is accompanied by the computed effective subgrade resilient modulus in psi and the percent increase from the unreinforced pavement structure. For instance, the increase in effective resilient modulus assuming a 50% sub base thickness reduction in a medium volume pavement with an initial subgrade resilient modulus of 3,000 psi is 31.41%. Considering all cases analyzed, the minimum and maximum changes in effective subgrade resilient modulus were 0.46% (at 5% thickness reduction) and 41.10% (at 50% thickness reduction).

Table 15 Parametric study results.

	Low Volume Traffic Road (ESAL = 300,000)			Medium Volume Traffic Road (ESAL = 3,000,000)			High Volume Traffic Road (ESAL = 30,000,000)		
	Base Reduction [%]	Subgrade MR [psi]	Increment [%]	Base Reduction [%]	Subgrade MR [psi]	Increment [%]	Base Reduction [%]	Subgrade MR [psi]	Increment [%]
Very Week Subgrade (MR = 1500)	5	*1526.48*	1.77	5	*1523.65*	1.58	5	*1514.32*	0.95
	10	*1572.02*	4.80	10	*1559.03*	3.94	10	*1555.61*	3.71
	20	*1655.87*	10.39	20	*1632.92*	8.86	20	*1649.98*	10.00
	50	*1785.57*	19.04	50	*1893.84*	26.26	50	*1932.86*	28.86
Medium Week Subgrade (MR = 3000)	5	*3027.11*	0.90	5	*3020.75*	0.69	5	*3052.14*	1.74
	10	*3134.48*	4.48	10	*3104.08*	3.47	10	*3151.45*	5.05
	20	*3363.86*	12.13	20	*3302.34*	10.08	20	*3380.83*	12.69
	50	*4112.14*	37.07	50	*3942.30*	31.41	50	*4087.30*	36.24
Stiff Subgrade (MR = 4500)	5	*4520.64*	0.46	5	*4569.00*	1.53	5	*4562.85*	1.40
	10	*4652.97*	3.40	10	*4708.92*	4.64	10	*4727.23*	5.05
	20	*5030.25*	11.78	20	*5005.18*	11.23	20	*5109.49*	13.54
	50	*6349.59*	41.10	50	*6094.07*	35.42	50	*6308.25*	40.18

The parametric study offers a very rough estimate of the 'equivalent' subgrade resilient modulus improvement that results from the geogrid reinforcement. While the data allows for a crude improvement comparison in terms of resilient modulus, it should be noted that the geogrid reinforcement offers additional improvements in the mechanical performance of the pavement section as a whole. ***Note that the 1993 AASHTO design equations are empirical in nature and were never meant to be used in the analysis of geogrid reinforced pavements; therefore, the proposed method offers a very rough estimate and should not be used to judge the full benefits of geogrid reinforcement.***

5.6.2. Cost estimate

The costs associated to geogrid reinforcement can be estimated using the following equation:

$$C_G = a_g.c_g + a_g.c_{tg}.x_g + c_h.t \quad [23]$$

C_G = total cost [$],
a_g = required geogrid [ft^2],
c_g = cost of geogrid [$\$\cdot ft^2$],
c_{tg} = transportation cost for geogrid [$\$\cdot ton^{-1}\cdot mile^{-1}$],
x_g = distance to geogrid provider [miles],
c_h = hourly equipment operation cost [$\$\cdot hr^{-1}$],
t = operation time [hr].

6. DESIGN GUIDE

6.1 COMPACTION

The optimum mechanical performance of a soil is attained by maximizing its packing density. Compaction rearranges soil particles, moving them closer together into a denser arrangement. Increasing the density or unit weight of the soil leads to a reduction in porosity, an increase in coordination number, and an increase in particle interlocking and rotational frustration (Santamarina et al. 2001). As a result, compaction increases the soil shear strength, and reduces its compressibility (Lambe and Whitman 1969).

The design of compaction jobs starts with the laboratory determination of maximum dry unit weight and optimum compaction water content, and is completed with the specification of a minimum acceptance field dry unit weight and tolerances for field moisture content.

6.1.1. Compacted dry unit weight and water content

Particle rearrangement and densification is attained through the introduction of mechanical energy. In the laboratory, mechanical or compaction energy is transferred to the soil by the impact of a falling weight. In the field, compaction energy is transferred to the soil by the moving weight and vibration of heavy compaction equipment. At a constant compaction energy, or compactive effort, the maximum attainable dry unit weight of a soil is a function of the soil water content. The maximum dry unit weight and optimum compaction water content are determined from the results of Proctor compaction tests. Figure 23 shows typical results of Proctor compaction tests conducted on different soils.

As shown in Figure 23, the maximum dry unit weight of the soil is only attained at a very specific water content, ***the optimum compaction water content***. Any change in the water content results in a lower dry unit weight and diminished material performance. The sensitivity of the maximum dry unit weight to a change in water content varies depending on soil type. Soils can range from relatively insensitive as shown in Figure 23-c to very sensitive as shown in Figure 23-d. Soils that are relatively insensitive to changes in compaction water content offer an advantage in terms of construction tolerances because target field densities can be attained in a wider range of water contents. The more sensitive a soil is to changes in compaction water content, the tighter the tolerances on field water content, which makes it more challenging to attain target field densities.

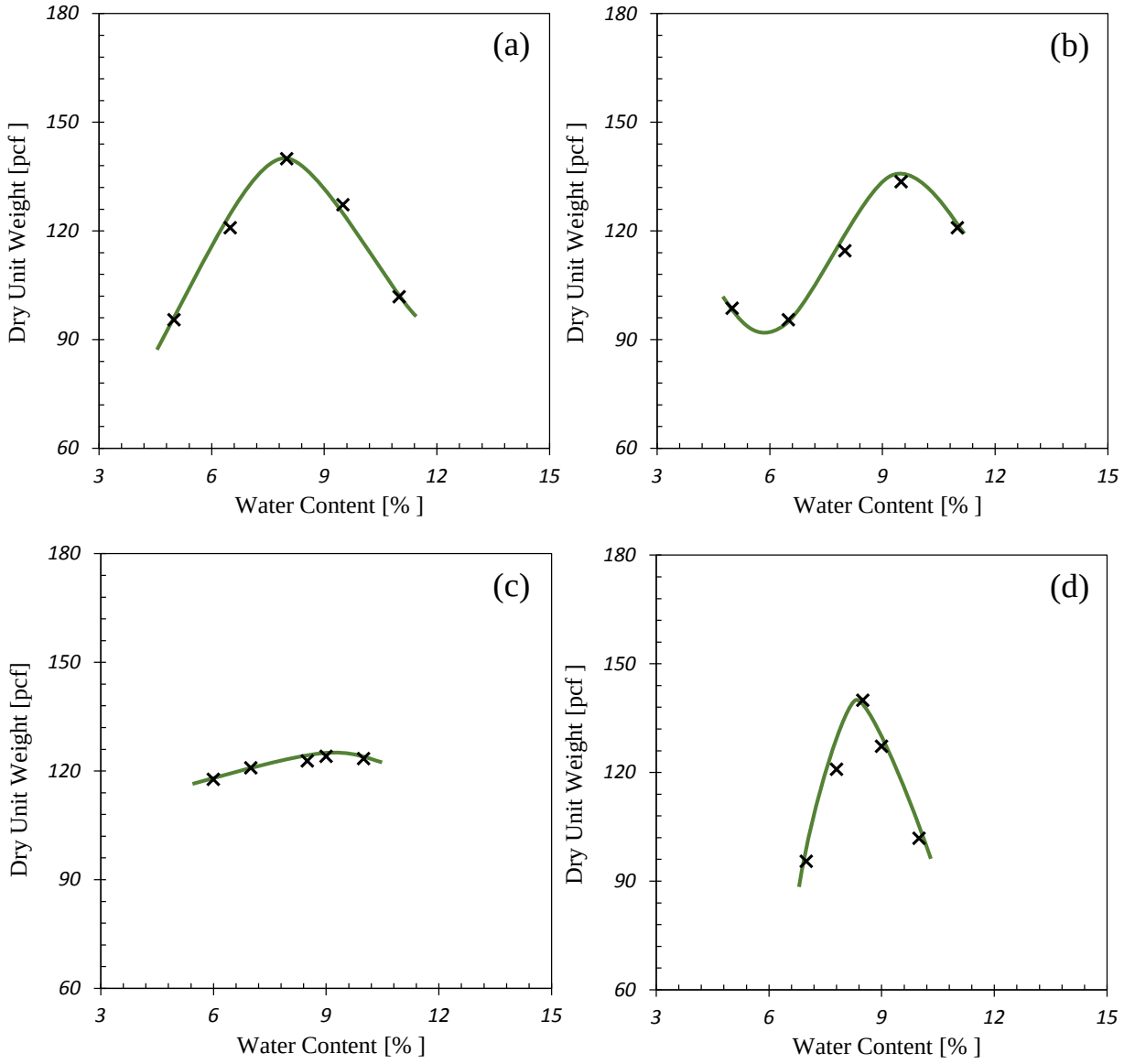


Figure 23. The compacted dry unit weight of a soil depends on the compaction water content. (a) Shows the classical example of a compaction curve. Compaction curves can however vary in shape as in (b), and in sensitivity as shown in (c) and (d).

6.1.2. Compacted dry unit weight and compactive effort

The maximum dry unit weight and the optimum compaction water content of the soil also depend on the compactive effort. An increase in the compaction energy transferred into the soil generally results in the attainment of a higher maximum dry unit weight at a lower moisture content. Higher compactive efforts are typically associated to more efficient compaction equipment. In the laboratory, the effects of an increase in compaction effort can be determined by conducting ***Standard and Modified Proctor Compaction Tests***. Figure 24 shows a comparison of the results of a standard and a modified proctor compaction test conducted on the same soil.

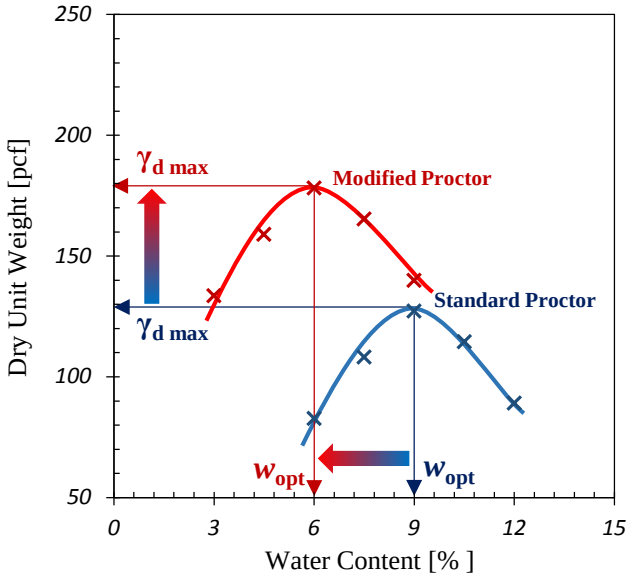


Figure 24. An increase in the compactive effort results in the attainment of a higher maximum dry unit weight at a lower optimum compaction water content. The compactive effort of a modified Proctor compaction test is over four times higher than that of a standard Proctor compaction test.

As shown in Figure 24, an increase in compactive effort results in an increase in the maximum dry unit weight and a decrease in the optimum compaction water content.

The standard test for determination of maximum dry unit weight and optimum compaction water content established by the Department in the 2014 Standard Specifications for Highway and Bridge Construction is the Modified Proctor Compaction Test. Requiring contractors to compact the soil with a higher compactive effort results in improved performance through densification as well as reductions in costs associated to transportation and volume of water used in compaction jobs. ***It should be noted that failure to provide a sufficient field compactive effort would result in a drastic reduction in the attainable field dry unit weight***.

6.1.3. Compaction Specifications

Compaction specifications include the minimum acceptable field compacted dry unit weight and the tolerances for field compaction water content. The minimum acceptable field compacted dry unit weight is often specified in terms of the **degree of compaction** (**DC**). The degree of compaction is defined as the ratio of the field measured dry unit weight ($\gamma_{dry\text{-}field}$) to the maximum laboratory dry unit weight ($\gamma_{dry\text{-}max}$).

$$DC = \frac{\gamma_{dry-field}}{\gamma_{dry-\max}} \qquad [24]$$

Where $\gamma_{dry\text{-}field}$ is the field measured dry unit weight after compaction and $\gamma_{dry\text{-}max}$ is the maximum dry unit weight determined from the results of the Modified Proctor Compaction Test, see Figure 25. The minimum acceptable degree of compaction for roadbeds and roadbed embankments is 95%. Thus, the minimum acceptable field dry unit weight is given by:

$$\gamma_{dry-field} \geq (0.95) \cdot \gamma_{dry-\max} \qquad [25]$$

This is shown graphically in Figure 25. The tolerances in compaction water content are determined by drawing a horizontal line at the minimum acceptable field dry unit weight. The two points at which the horizontal line intersects the compaction curve are used to identify the minimum and maximum allowable water contents.

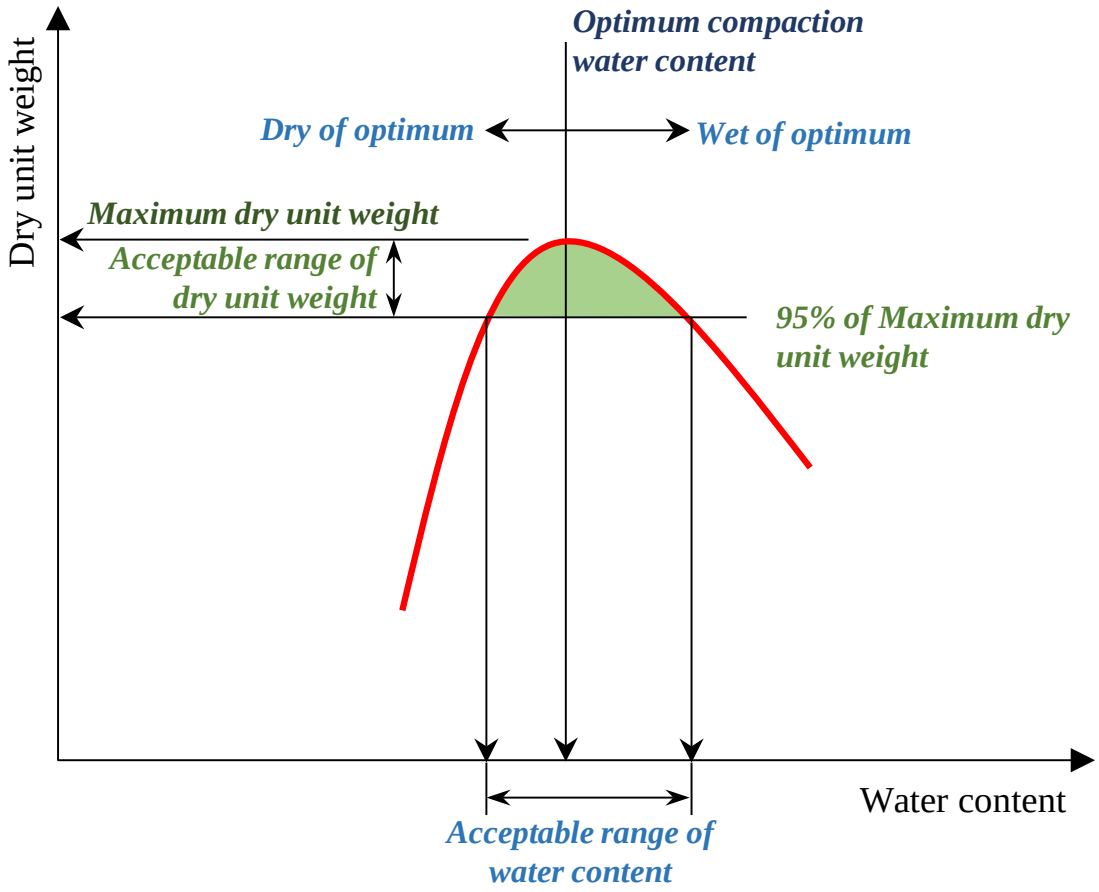


Figure 25. Analysis of compaction test results. The soil compaction curve is used to determine the maximum dry unit weight. Acceptance of a subgrade compaction job requires the field dry unit weight to be at least 95% of the laboratory determined maximum dry unit weight (green shaded area). Tolerances on compaction water content are obtained from the intersections of the compaction curve and the 95% dry unit weight line.

6.1.4. Test procedures and example calculations

Laboratory proctor compaction testing is used to determine the maximum dry unit weight and the optimum compaction water content. Two test protocols are available for compaction testing, the standard Proctor test (AASHTO T99) and the modified Proctor test (AASHTO T180). The two protocols are intended to capture differences in the ability of construction equipment to transfer compaction energy into the soil. When higher compaction energy or compactive effort is transferred into the soil, higher maximum dry unit weight is attained at lower optimum

compaction water content. The standard test for determination of maximum dry unit weight and optimum compaction water content established by the Department in the 2014 Standard Specifications for Highway and Bridge Construction is the Modified Proctor Compaction Test.

Modified Proctor Compaction Test

Equipment check list:

- ✓ Compaction mold,
- ✓ No. 4 sieve,
- ✓ Modified proctor hammer (10 lb weight and vertical fall of 18 in)
- ✓ Balance sensitive to 0.01 lb,
- ✓ Large flat plan,
- ✓ Extrusion jack,
- ✓ Steel straightedge,
- ✓ Moisture cans,
- ✓ Drying oven,
- ✓ Plastic squeeze bottle with water.

Summarized procedure:

1. Collect 10 lb (4.5 kg) of air dry soil,
2. Sieve the soil using the No. 4 sieve and collect at least 6 lb (2.7 kg) of material,
3. Add enough water to increase the water content to approximately 5%,
4. Record the weight of the mold and base plate (without extension),
5. Attach the extension to the mold,
6. Pour soil into the mold to about a third of its height and compact it by dropping the hammer 25 times,
7. Repeat the procedure as to obtain 5 compacted layers,
8. Remove the mold extension,
9. Using a straightedge bar, trim the excess soil at the top of mold,
10. Record the weight of the mold, base plate and compacted moist soil,
11. Remove the compacted soil from the mold using the extrusion jack,
12. Collect a 20 to 50 gr soil sample and determine its moisture content,
13. Break the remaining soil and mix it with the leftover moist soil,
14. Add more water and remix the soil,
15. Repeat 6 through 13 for at least four more increments in water content,
16. Record the masses of the compacted soil samples and the corresponding water contents.

Example calculations:

Modified Proctor compaction tests conducted in the laboratory at six different compaction water contents are shown in table below. The mold used for testing was the standard 4 inch mold.

Specimen No.	1	2	3	4	5	6
Water content (%)	8.15	11.78	13.29	16.81	20.40	22.84
Weight (lb)	3.49	3.77	4.19	4.37	4.14	4.06

Based on the results, determine the maximum dry unit weight, the optimum compaction water content, the minimum acceptable field unit weight for a 95% degree compaction, the maximum allowable field water content and the minimum allowable field water content.

1. Determine the bulk unit weight dividing the weight of the specimen by the volume of the compaction mold. The volume of a standard 4 inch mold is 1/30 ft^3, and the volume of a standard 6 inch mold is 1/13.3 ft^3. For the first specimen:

$$\gamma_{bulk} = \frac{W_T}{V_T}$$

γ_{bulk} = Bulk unit weight [pcf],
W_T = Total weight [lb],
V_T = Total volume [ft^3].

For the first specimen:

$$\gamma_{bulk} = \frac{3.49 lb}{\left(1/30\ ft^3\right)} = 104.77\, pcf$$

Specimen No.	1	2	3	4	5	6
Water content (%)	8.15	11.78	13.29	16.81	20.40	22.84
Weight (lb)	3.49	3.77	4.19	4.37	4.14	4.06
Bulk unit weight (pcf)	104.77	113.18	125.80	131.20	124.29	121.89

2. Determine the dry unit weight according to the following equation:

$$\gamma_{dry} = \frac{\gamma_{bulk}}{1+w}$$

γ_{dry} = Dry unit weight [pcf],
w = Water content [ ].

For the first specimen:

$$\gamma_{dry} = \frac{104.77\, pcf}{\left[1+\left(\frac{8.15\%}{100\%}\right)\right]} = 96.88\, pcf$$

Specimen No.	1	2	3	4	5	6
Water content (%)	8.15	11.78	13.29	16.81	20.40	22.84
Weight (lb)	3.49	3.77	4.19	4.37	4.14	4.06
Bulk unit weight (pcf)	104.77	113.18	125.80	131.20	124.29	121.89
Dry unit weight (pcf)	96.88	101.25	111.04	112.32	103.23	99.23

3. Determine the zero air voids line using the following equation:

$$\gamma_{zav} = \frac{G_s}{[1 + wG_s]}\gamma_w$$

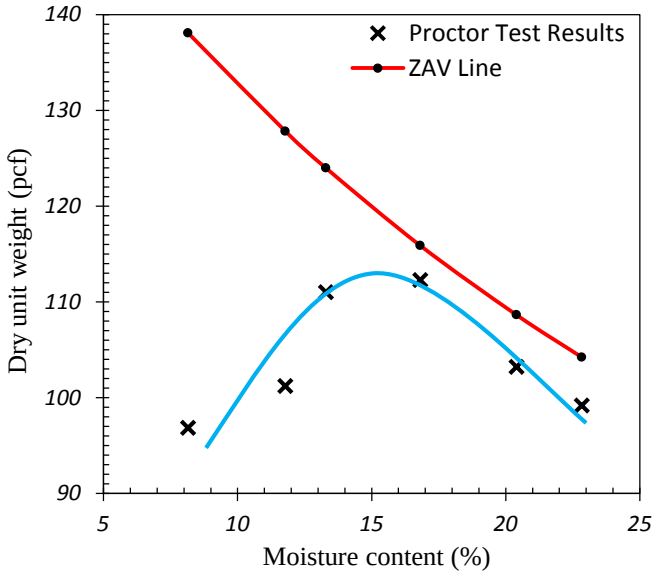
γ_{zav} = Zero air voids dry unit weight [pcf],
γ_w = Unit weight of water [pcf],
G_s = Specific gravity of solids [ ].

For the first specimen:

$$\gamma_{zav} = \frac{2.7}{\left[1 + \left(\frac{8.5\%}{100\%}\right) \times 2.7\right]} \times 62.4\,pcf = 138.09\,pcf$$

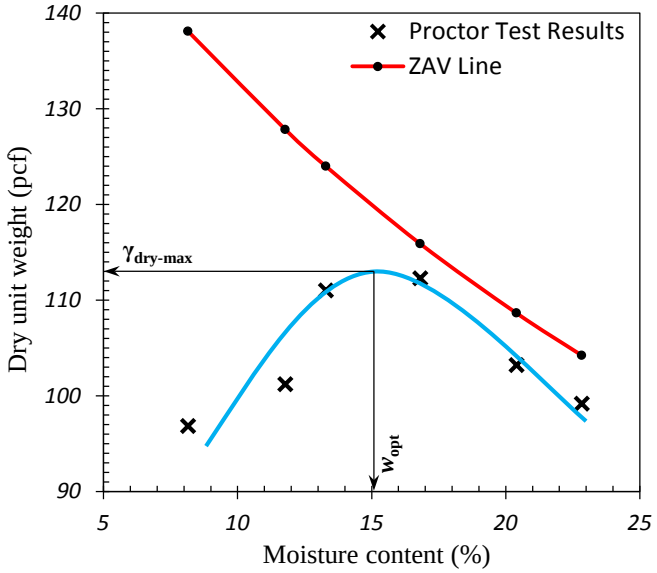
Specimen No.	1	2	3	4	5	6
Water content (%)	8.15	11.78	13.29	16.81	20.40	22.84
Weight (lb)	3.49	3.77	4.19	4.37	4.14	4.06
Bulk unit weight (pcf)	104.77	113.18	125.80	131.20	124.29	121.89
Dry unit weight (pcf)	96.88	101.25	111.04	112.32	103.23	99.23
ZAV Dry unit weight (pcf)	138.09	127.82	123.98	115.89	108.64	104.22

4. Plot the dry unit weight vs. water content and the ZAV dry unit weight vs. water content in the same graph. Draw a fitting line on the proctor compaction dry unit weight data (blue line) as shown in the figure below.



Under no circumstance should the compaction test curve intersect the ZAV. Whenever it does, it is an indication of faulty test results.

5. Using the graph determine the maximum dry unit weight and the optimum compaction water content as shown in the figure below.

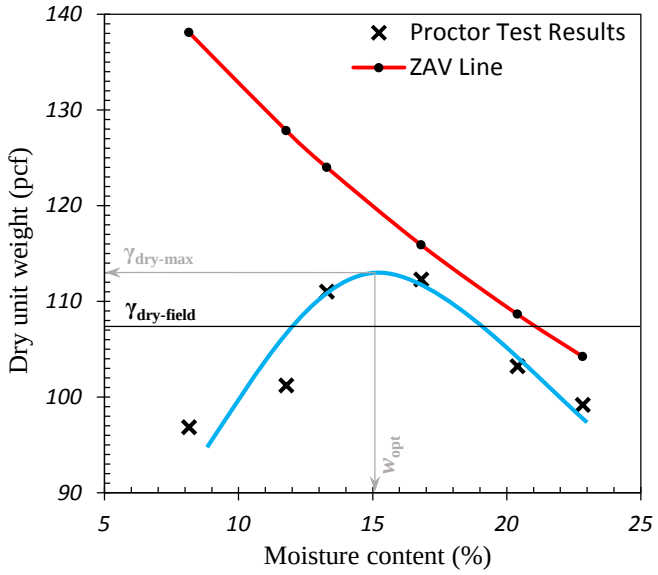


From the graph, the maximum dry unit weight $\gamma_{dry\text{-}max}$ = 113pcf, and the optimum compaction water content w_{opt} = 15%.

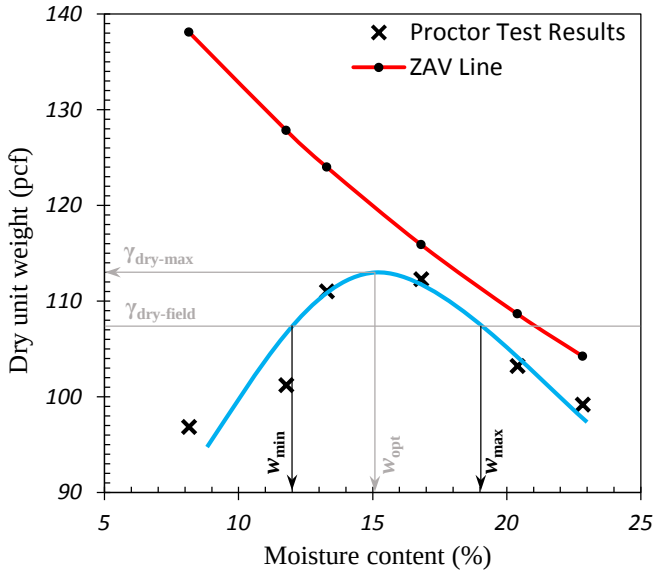
6. Proceed to determine the minimum dry unit weight for a degree of compaction of 95%.

$$\gamma_{dry-field} \geq (0.95) \times \gamma_{dry-\max} \geq (0.95) \times 113 pcf = 107.35 pcf$$

7. Draw a horizontal line at the level of the minimum field dry unit weight.



8. Determine the water contents at which the horizontal line intersects the compaction curve.



From the graph, the maximum compaction water content w_{max} = 19 %, and the minimum compaction water content w_{min} = 12%.

6.1.5. Deviations from standard design procedures

Deviations from standard test procedures are ***required*** when dealing with special types of soils found in the state of New Mexico. Shale and gypsum soils can be found in certain areas of the

state. The determination of compaction specifications for these soils requires modified procedures as discussed next.

Crushed shale

Shales are in the borderline between soils and rocks. Often described as soft rocks, shales can alternatively be thought of as very heavily over consolidated clays. Shale outcrops can be encountered in all six districts and can be classified in three categories: '***bad shales***', '***good shales***' and '***problem shales***'. The qualifiers are given to the material based on the stability, or resistance, of the shale to weathering and erosion. Upon weathering, shales can revert back to the clays they originated from. The properties of the weathered material can in many cases resemble those of high plasticity expansive clays, which can result in surface heaving and pavement failure.

When bad shales are found, the material should be excavated and discarded. It should never be used in compacted fill sections of the roadway. Good shales, on the contrary, are suitable for compacted fill sections. The distinction between good and bad shales is therefore extremely important as it has significant impacts on the economic feasibility and long term performance of compacted fills. The jar-slake test is a simple field screening test proposed on the Shale Use Policy Report (Lommler 2009) for evaluating the weathering resistance of shales. In the test, shale particles are immersed in water inside a glass jar. The reaction between the shale and the water is monitored visually for up to 24 hours. A description of the reaction is recorded immediately after immersion, at 30 minutes after immersion, and at regular intervals thereafter. Observations are then compared to the reactions described in Table 16, from which the Slake Index and Weathering Grade can be estimated.

Table 16. Jar-slake index (Lommler 2009).

Slake Index & Weathering Grade	Time = 0 min	Time = 30 min
1	Degrades into a pile of flakes.	Completely disintegrates and turns into mud. Water is very foggy.
2	Breaks rapidly and/or forms many chips.	Disintegrates with some fragments barely intact. Water is foggy.
3	Breaks rapidly and/or forms many chips.	Partially intact. Water is foggy.
4	Breaks slowly and/or forms several fractures.	Moderately intact. Water is cloudy.
5	Breaks slowly and/or forms few fractures.	Remains intact. Water is slightly cloudy.
6	Remains intact, no change.	Remains intact. Water is clear.

In addition to the jar-slake test, the shale should be subjected to laboratory slake durability testing. Once a shale has been cleared and classified as a good shale, conventional compaction testing and analysis can be conducted. For detailed information related to slake tests and shale formations found in the state of New Mexico refer to the Shale Use Policy Report.

Gypsum and Anhydrite

Gypsum and anhydrite rich soils can be found in the State of New Mexico and pose particular compaction design challenges. In gypsum-anhydrite rich soils, water is found not only within pores (free water) but also as part of the solid mineral structure itself (structural water). Standard compaction design procedures are applicable to soils in which only free water is present, and therefore need to be modified in order to capture the effect of structural water. A study commissioned by the state to assess the performance of compacted gypsum fill concluded that

suitable performance can be attained through well-controlled placement, processing, and compaction similar to standard earthwork construction.

Compaction water content is related to the free water content of the soil. If a subgrade is made out entirely of gypsum ($CaSO_4 \cdot 2H_2O$), even in a dry state (free water = 0) the soil has a structural water content of 26.4%. While seemingly high, this water is chemically bound to the mineral and has almost no effect on the compaction of the material (Lommler 2006). Thus, additional water must be added to facilitate the soil densification. Anhydrite ($CaSO_4$) contains no structural water; however, in the presence of water it 'absorbs' it into its molecular structure thereby converting into gypsum. In the process, free water becomes chemically bound and the material experiences cementation.

Conventional standard procedures for the determination of water content in soils require the drying temperature to be 110 degrees centigrade. However, at this temperature both free and structural water are removed from gypsum rich soils and there is no way of determining how much of each is present in the soil (Lommler 2006). ***The critical difference to the standard procedure is to determine the water content at a temperature of 60 degrees centigrade instead, to drive off free water without removing structural water***. The graphs of dry unit weight vs. compaction water content should be drawn using the free water content only!

In addition to the procedural differences in the determination of compaction water content, gypsum-anhydrite rich soils will also have a wider variation in the specific gravity of solids. In the case of pure gypsum $2.31 < Gs < 2.33$, and in the case of pure anhydrite $2.9 < Gs < 3.0$; therefore, for practical purposes the specific gravity can vary between 2.3 and 3.0.

6.2. BLENDING

Soil blending involves the mixing of multiple soils of different gradations to attain a well graded soil blend which exhibits improved strength and stiffness while optimizing compaction (i.e., attainment of a higher maximum dry unit weight at the same compactive effort). The compacted density of a soil is in great part governed by its grain-size distribution. The lower densities typical of uniform or poorly graded soils can be increased by selectively incorporating particle sizes missing in the original soil gradation through blending.

6.2.1 Simplified Blend Design

For practical purposes, blending is herein defined as the addition of coarse-grain soil material to a fine-grained soil. While the density and performance of a subgrade can be tailored by selecting a suitable coarse-grain blend, in practice it is often times economically unfeasible to procure a very specific gradation. Often times it is only economically advantageous to use unprocessed available coarse-grained material from nearby sources or from cut sections of the same road alignment. The ultimate goal of blending is to reduce the fines content of the subgrade to a maximum value of 12%. The unified soil classification system recognizes that a fines content of 12% marks the threshold where behavior of the soil begins to be strongly influenced by the behavior of the fines. Thus, if the fines content can be reduced below 12%, the behavior of the soil will be governed by the coarse-grain skeleton. The mass of coarse-grained soil needed to reduce the fines content of a blend to 12% can be estimated using the following equation:

$$m_{cs} = \frac{f_c - 12\%}{12\%} \times m_s \qquad [26]$$

m_{cs} = mass of clean, dry, coarse-grained soil [tons],
f_c = fines content [ ],
m_s = total dry mass of subgrade soil [tons].

The required mass of coarse-grained soil should be compared with the available mass of coarse-grained soil to determine if blending is a feasible alternative for stabilization. Note that the equation assumes that the coarse-grained soil contains no fines.

Ultimately the mechanical performance of the soil will be dictated by the blended grain-size distribution and the degree of compaction attained. Therefore compaction and R-value testing need to be conducted in a sample of the blended soil to determine if target project performance can be satisfied through blending and compaction alone.

6.2.2. Test procedures and example calculations

Check list:

- ✓ Subgrade soil grain size distribution (fines content),
- ✓ Blended soil grain-size distribution,
- ✓ Blended soil compaction curve, maximum dry unit weight and optimum compaction water content,
- ✓ Blended soil compacted R-value.

Example calculations:

A 500 ft stretch of a 2-lane road cuts across a soil with a fines content of 40% and a plasticity index (PI) of 30. 4,000 tons of clean sand (fines content = 0) are available in stockpiles collected from cut sections of the same road. Determine if the available sand stockpiles are sufficient to treat the soil by blending alone.

1. Estimate the total dry mass of subgrade soil for treatment. Calculate the treatment volume by multiplying the width of the roadway by its length by 3 ft of depth:

$$V_T = 24\,ft \times 500\,ft \times 3\,ft = 36{,}000\,ft^3$$

convert the volume from ft^3 to m^3:

$$V_T = 36{,}000\,ft^3 \times \left(0.0283 \tfrac{m^3}{ft^3}\right) = 1018.8 m^3$$

Compute the mass by multiplying the volume by the dry density of the subgrade soil. If the density of the subgrade soil is not known use an upper and lower estimate as follows:

$$m_s = V_T \times \left(1.5 \tfrac{tons}{m^3}\right) = \left(1018.8 m^3\right) \times \left(1.5 \tfrac{tons}{m^3}\right) = 1{,}528 tons \qquad \textbf{\textit{Upper bound}}$$

$$m_s = V_T \times \left(1.1 \tfrac{tons}{m^3}\right) = \left(1018.8 m^3\right) \times \left(1.1 \tfrac{tons}{m^3}\right) = 1{,}121 tons \qquad \textbf{\textit{Lower bound}}$$

2. Estimate the total mass of coarse grained soil required for blending:

$$m_{cs} = \frac{f_c - 12\%}{12\%} \times m_s = \frac{40\%_c - 12\%}{12\%} \times 1{,}528 tons = 3{,}566 tons \qquad \textbf{\textit{Upper bound}}$$

$$m_{cs} = \frac{f_c - 12\%}{12\%} \times m_s = \frac{40\%_c - 12\%}{12\%} \times 1{,}121 tons = 2{,}615 tons \qquad \textbf{\textit{Lower bound}}$$

3. Since the estimated mass of material is less than what is available in the stock piles, blending is a feasible alternative.

The next step is to collect a subgrade soil sample from the field, blend it, compact it and determine the R-value.

1. Collect a 20lb sample of dry subgrade soil.

2. Determine the required amount of coarse-grained soil for blending according to:

$$m_{cs} = \frac{f_c - 12\%}{12\%} \times m_s = \frac{40\% - 12\%}{12\%} \times 20lb = 47lb$$

3. Blend in the 20lb of subgrade soil with the 47lb of coarse-grained soil ensuring uniform mixing and no segregation.

4. Use the blended soil to conduct modified proctor compaction tests and determine the optimum compaction water content and the maximum dry unit weight.

5. Determine the R-value of the compacted blended soil at optimum compaction water content.

6.3. CEMENT

The addition of cement and water to a fine-grained soil followed by compaction results in an improvement of the soil strength and stiffness under varying environmental conditions (e.g., fluctuations in water content, ionic concentration, and temperature). Cement stabilization alters subgrades primarily in two ways: (1) it binds soil particles together increasing strength and providing resistance to tensile stresses, and (2) it causes partial to complete alteration of clay minerals effectively reducing soil plasticity (Jones et al. 2010). Thus, cement can be effectively used to improve the performance of both coarse and fine grained soils.

6.3.1. Soil-cement mixture proportioning for fine-grained soils

Mixture design requires knowledge of soil and cement physical characteristics. Soil properties of significance include:

(1) grain-size distribution,
(2) liquid and plastic limits,
(3) in-situ water content, and
(4) optimum compaction water content.

The following chemical properties of the soil must also be determined in order to assess compatibility with cement stabilization (Kézdi 1979):

(1) sulphate content,
(2) magnesium content,
(3) calcium content,
(4) organic matter content, and
(5) pH.

Cement is typically used to improve soils that exhibit a plasticity index lower than 35; however, higher plasticity soils can and have been successfully treated with cement (Jones et al. 2010; Prusinski and Bhattacharja 1999). Sands, silty sands, silts, silts containing organics, and lean clays, are in general good candidates for cement stabilization. Sulfate reach subgrade soils (i.e. sulfate content > 3000 ppm), soils with high organic content, or soils that exhibit low pH require special considerations for treatment.

The success or failure of a soil-cement mixture involving fine grained soils is very closely related to the project execution during construction. In addition to being affected by the cement content and water content, the mechanical performance of a soil cement mixture is also affected by the compactive effort and the degree of mixing. Hence, laboratory mixture design must capture field construction procedures.

The design of soil-cement mixtures can be reduced to five steps:

1. Soil-cement compaction,
2. Water content selection,
3. Specimen casting,
4. Specimen curing,
5. Performance determination.

1. Soil-cement compaction

Soil compaction tests (i.e., AASHTO T99 or T180) must be conducted on the soil-cement mixture. The addition of cement to a wet soil can alter its behavior; this is more pronounced in fine-grained soils that exhibit plastic behavior. Therefore, the optimum compaction water content needs to be determined in the presence of cement. It is recommended that the test be conducted at cement content between 3% and 6% by weight. The results of the test should yield a curve of dry unit weight versus water content similar to that shown in Figure 25; however, the calculations are slightly modified to capture the presence of cement as follows:

$$\gamma_{dry} = \frac{m_T \cdot g}{[1 + C_c + w_{mix}]V_T}$$

γ_{dry}	=	Dry unit weight of the soil-cement mixture,
m_T	=	Total mass of the mixture,
g	=	Acceleration due to gravity,
C_c	=	Cement content (mass of cement divided by the mass of dry soil),
w_{mix}	=	Water content (mass of water divided by the mass of dry soil),
V_T	=	Total volume,

Similarly, the zero air voids curve is given by:

$$\gamma_{zav} = \frac{G_s \cdot G_c(1 + C_c)}{G_c(1 + w_{mix} G_s) + C_c G_s} \cdot \gamma_w$$

γ_{zav}	=	Zero air voids dry unit weight [pcf],
γ_w	=	Unit weight of water [pcf],
G_s	=	Specific gravity of soil solids [ ],
G_c	=	Specific gravity of cement [ ].

Figure 26 presents an example of a compaction curve for a high plasticity soil-cement mixture at 6% cement content compacted according to ASTM C109 Standard test method for compressive strength of hydraulic cement mortars.

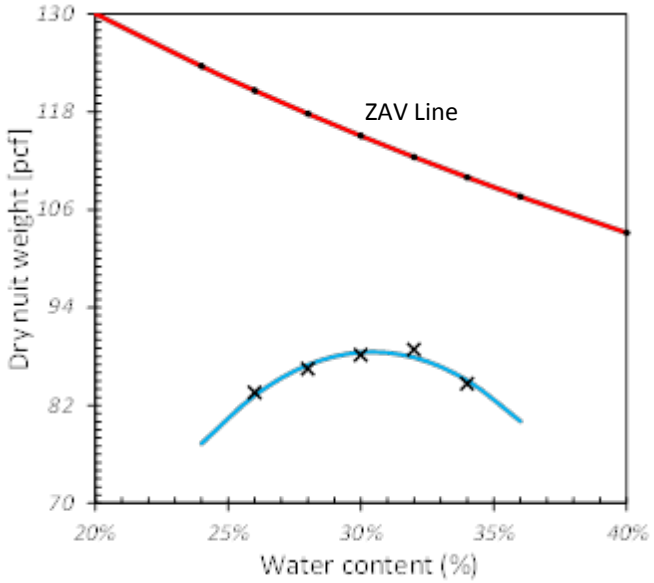


Figure 26. Compaction curve of a soil-cement mixture.

Note that testing using the standard proctor molds (i.e., 4 or 6 inch cylinders) requires a minimum soil mass of about 20 to 50 pounds. In fine grained soils with no material retained in sieve #4, it is possible to use an alternative cubic mold of 2 inches in size. This size reduction can limit the required mass of soil to about 8 pounds which in turn allows for testing of duplicate

specimens. However, the compactive effort must be adjusted so that it is equivalent to either the standard or modified proctor compactive effort depending on the construction specifications. ***It is extremely important to conduct the test at a compactive effort that is representative of the expected field construction. Discrepancies in the compactive effort between the laboratory and field compacted material can yield substantial differences in performance and may render the mixture design inadequate.*** See Appendix A for more details.

2. Water content selection

The soil-cement mixture compaction data is used to guide the selection of water contents for performance testing. In order to determine optimum strength water content, the compaction data needs to be converted from dry unit weight in pounds per cubic foot to unit less void ratio as follows:

$$e = G_s \frac{\gamma_w}{\gamma_{dry}} - 1$$

Figure 27 shows the void ratio vs. water content graph that corresponds to the compaction data presented in Figure 26. The void ratio data is then fitted to a quadratic equation using a least squares error minimization algorithm. This process is automated in Excel ®. Curve fitting should yield a quadratic equation of the form $e = Aw^2 + Bw + C$, where A, B, and C are the curve fitting parameters. In the example from Figure 27: A = 61.025, B = -37.128, and C = 6.5137.

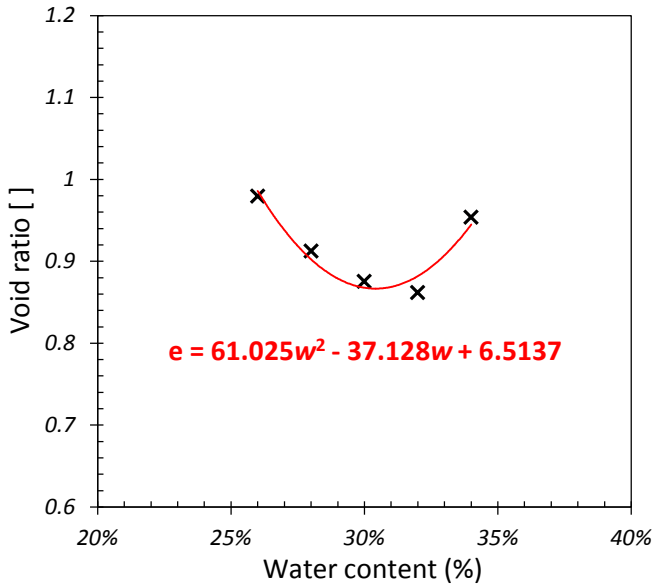


Figure 27. Void ratio vs. water content. The red curve shows graphically the best fit quadratic equation.

Using the quadratic equation, three water contents are found as follows:

$$w_{mix1} = \frac{-B}{2A}$$

$$w_{mix2} = \sqrt{\frac{C}{A}}$$

$$w_{mix3} = \frac{w_{mix1} + w_{mix2}}{2}$$

In the example, w_{mix1} = 30.4%, w_{mix2} = 32.7%, and w_{mix3} = 31.5%.

w_{mix1}, w_{mix2} and w_{mix3} offer a reduced range in water content from which the optimal mechanical performance is expected to be found. Note that in lower plasticity soils $w_{mix3} \rightarrow w_{mix1}$, in which case the optimum performance water content is attained at $w_{mix1.}$

3. Specimen casting

Once w_{mix1}, w_{mix2} and w_{mix3} have been determined, 3 soil-cement specimens should be mixed and compacted at each of the three water contents for a total of 9 specimens. Using three specimens provides triplicates which facilitate the determination of variability in the results. If w_{mix2} is within 1% of w_{mix1}, prepare only 3 specimens at w_{mix1}.

Note that testing using the standard proctor molds (i.e., 4 or 6 inch cylinders) requires a minimum soil mass of about 40 to 100 pounds. In fine-grained soils with no material retained in sieve #4, it is possible to use an alternative cubic mold of 2 inches in size. This size reduction can limit the required mass of soil to about 15 pounds. However, the compactive effort must be adjusted so that it is equivalent to either the standard or modified proctor compactive effort depending on the construction specifications. ***It is extremely important to cast the specimens at a compactive effort that is representative of the expected field construction. Discrepancies in the compactive effort between the laboratory and field compacted material will yield substantial differences in performance and may render the mixture design inadequate.***

4. Specimen curing

Curing of the compacted soil-cement specimens should be done by eliminating moisture loss to the environment. This can be achieved using a moist room or by covering the specimens in an air tight plastic wrap and storing them at room temperature for 7 days.

5. Performance determination

At 7 days of curing, the specimens should be removed from their plastic wraps or moist room and tested for p-wave velocity in order to determine the small strain elastic modulus. After that, the specimens need to be submerged in water for at least 4hr prior to testing for unconfined compressive strength.

The optimum water content is selected from the averaged results of the three specimens. If the mechanical performance at optimum satisfies the specified subgrade performance for the project, the mixture design is completed. If the optimum performance is below specifications, increase the cement content by 1% while maintaining the same optimum water content and repeat steps 3 and 4. If there is a need to increase the cement content by more than 3% of the initial cement content, then steps 1 through 4 should be repeated.

Note: ASTM Standard C109 can be used for general guidance to test soil-cement mixtures; however, compaction of the specimens should be altered to capture the field compactive effort. Do not use the specimen compaction method outlined in ASTM C109.

6.4. LIME

The addition of lime to a fine-grained soil reduces its plasticity, increasing the soil strength while decreasing the extent of volumetric changes associated to fluctuations in water content. Lime is add-mixed to the soil in either powder or liquid form as either calcium oxide (CaO - quicklime) or calcium hydroxide ($Ca[OH]_2$ - hydrated lime). Both of these are high calcium lime, containing less than 5% magnesium oxide (MgO) or hydroxide ($Mg[OH]_2$). In some cases, dolomitic lime (containing up to 46% MgO or $Mg[OH]_2$) may be used (NLA 2004). The interaction between lime and soil particles is mediated by water. The byproducts of lime hydration alter the soil behavior in the short-term, so that a higher maximum dry unit weight can be attain at lower optimum compaction water content.

Lime is typically used in clayey soils of medium to high plasticity. Soils with a fines content of at least 25% and with a plasticity index of 10 or greater, are good candidates for lime stabilization (Little 1999). The presence of organic matter (OM) or soluble sulfates (SO_2^{4-}) often results in performance issues. When the soil organic content exceeds 1% by weight, it becomes very difficult to attain target unconfined compressive strengths through lime stabilization (NLA 2006). Similarly, when soluble sulfates are present in the soil, lime treatment can result in long term swelling. ***For a detailed discussion on the treatment of sulfate bearing soils refer to Section 6.3.2. Deviation from Standard Procedures***.

Performance enhancements through lime stabilization are attributed to the chemical and physical processes that take place once lime and soil are exposed to water. These processes result in immediate and long-term effects (Celauro et al. 2012). While immediate effects alter the soil fabric, long-term effects involve pozzolanic reactions that render soil particles cemented.

Lime stabilized soil will exhibit a reduction in plasticity index, an increase in strength, and a reduction in its swelling capacity. Compaction parameters will also change. The mixture will exhibit an increase in the maximum dry unit weight and lower optimum compaction water content. Thus, the soil becomes more workable and it is easier to compact. Short term gains in strength are associated to changes in soil fabric (flocculation), which lead to an increase in shear strength. Pozzolanic reactions provide the long-term strength gains and the development of true cohesion. The stabilized soil becomes less sensitive to fluctuations in moisture content, i.e. swelling and shrinking. However, the effect is not permanent, and can be reverted over time. Available data indicates that effects are still present even after 12 years in service which is in the order of magnitude of a pavement service life (Prusinski and Bhattacharja 1999).

6.4.1. Soil-lime mixture proportioning

When CaO is exposed to water, it hydrates to form $Ca(OH)_2$, known as hydrated or slaked lime. The reaction is exothermic, which can result in evaporation of free water in the soil. Dissociation of $Ca(OH)_2$ into Ca^{2+} and $2(OH^-)$ takes place and the released ions begin to interact with the soil particles through ion exchange. Calcium ions replace adsorbed ions on the surface of clay particles, most commonly sodium (Na^+). This alters the thickness of the diffused double layer, reducing inter-particle repulsion and allowing the soil to flocculate, changing the soil fabric. Further changes in the soil are brought about by hydroxide ions. The increase in concentration of OH- causes the pH of the pore fluid to increase, triggering the partial dissolution of silicates and aluminates from clay particles. Dissolution products then slowly combine over time with free

calcium ions to form calcium-silicate-hydrates and calcium-aluminate-hydrates. These cementitious products increase the strength and stiffness of the soil in the long-term (Kézdi 1979).

The performance of lime stabilized soils depends on the soil grain-size distribution and mineralogy, the lime content, and the water content. ***The ultimate goal of mixture design is to identify the proportions of lime and water required to meet the target soil performance***. A mixture of lime and water adequately proportioned allows the lime to completely hydrate while maintaining the soil at or above the optimum compaction water content. Sufficient lime must be added to increase the pore fluid pH to a level such that silicate and aluminate minerals can dissociate from the soil particles for pozzolanic reactions to proceed. The National Lime Association (NLA) recommends the following procedure:

1. Initial soil evaluation,
2. Lime content determination,
3. Determination of the maximum dry unit weight and optimum compaction water content,
4. Lime-stabilized soil performance assessment (unconfined compressive strength and swelling).

6.4.1.1. Initial soil evaluation

First, the soil is classified according to: grain-size distribution, plasticity index, organic content, and sulfate content. Grain-size distribution is determined using the sieve analysis in accordance with ASTM C136 (ASTM 2006). Plasticity index is determined according to ASTM D4318 (ASTM 2010). Organic content is determined according to ASTM D2974 (ASTM 2014). ***There is a variety of test protocols for the determination of soluble sulfate content (Hayes 2007). Test results have been found to depend on the test protocol followed; therefore, it is extremely important to specify a test protocol***. The latest version of ASTM C1580 (ASTM 2009) is recommended as the preferred alternative (Hayes 2007). To determine if lime is a suitable treatment for the subgrade soil, check the following conditions:

- ✓ The fines content is greater than 25%,
- ✓ The plasticity index is greater than 10,
- ✓ The organic content is below 1%,
- ✓ The soluble sulfate content is below 3000 ppm.

If all conditions are met, lime stabilization is a suitable alternative for the subgrade soil treatment.

6.4.1.2. Lime content determination

The minimum lime content is the smallest amount of lime that causes the water pH in the soil to reach 12.4 (NLA 2006) as per ASTM D6276 (ASTM 2006). In the test, the soil is sieved passed the #40 sieve (0.425mm) and dried at 60 degrees centigrade. 25-gram soil specimens are separated and mixed with lime at 2, 3, 4, 5, and 6% lime by weight of dry soil. 100mL of water are added to each soil-lime mixture. The mixtures are shaken for 30 seconds every 10 minutes for 1 hour. The pH of the slurry is measured with a precision of 0.01 pH. The lowest lime content that results on a pH of 12.4 or higher is the minimum lime content for the soil-lime mixture. Minimum lime contents typically range between 3 and 6% lime by weight of dry soil (Zukri 2012).

6.4.1.3. Determination of the maximum dry unit weight and optimum compaction water content

After the lime content has been established, the soil-lime mixture is subjected to compaction testing using ***the Standard Proctor Compaction Test (AASHTO T99)***. Since the addition of lime alters the soil maximum dry unit weight and optimum water content, it is necessary to determine the soil-lime mixture compaction parameters at the minimum lime content. A key deviation from the standard procedure is that the soil-lime-water mixture must be allowed to 'mellow' between 1 and 24 hours in an air tight, moisture proof bag prior to compaction testing (NLA 2006). Note that changes in plasticity index can also be measured using the soil after the mellowing period (Little 2000).

6.4.1.4. Lime-stabilized soil performance assessment (unconfined compressive strength and swelling).

A minimum of two compacted specimens are then fabricated at three water contents: optimum, 1% above optimum, and 1% below optimum (Little 1999). The soil-lime-water mixture must be allowed to mellow between 1 and 24 hours in an air tight moisture proof bag prior to compaction(NLA 2006). The compacted soil-lime specimens are then sealed in an air tight, moisture proof packing, and allowed to cure for 7 days at 40 ºC (104 ºF). The recommended curing period and temperature have been selected to provide an accelerated representation of long-term curing under ambient field conditions (Little 2000). Following the curing process, the specimens are removed from their curing packing, their dimensions are measured, and their volume recorded. The specimens are then placed on top of a saturated porous stone, wrapped with a wet absorptive fabric, and placed in a water bath for 24 hours for capillary soaking. The bath water level should reach the top of the porous stone on which the specimen rests and come in contact with the absorptive wrapping fabric, but must not come in contact with the specimen itself (NLA 2006). After capillary soaking the volume of the specimens is measured again, and their unconfined compressive strength determined according to ASTM D5102 (ASTM 2009).

Calculate the volumetric strain using the following equation:

$$\varepsilon_v = \frac{V_f - V_o}{V_o} \times 100\% \qquad [27]$$

ε_v = volumetric strain,
V_f = volume of the specimen after capillary soaking,
V_o = volume of the specimen before capillary soaking.

Volumetric strains of 2% or lower are considered acceptable. If the volumetric strain exceeds 2%, fabricate additional specimens at lime contents 1 and 2% higher, and repeat the test. If the volumetric strain remains above 2% in these specimens it is necessary to conduct expansion, shrinkage, and uplift pressure tests according to ASTM D3877 (ASTM 2008). If the volumetric strain increases substantially this could be an indication of higher sulfate content than expected.

The minimum acceptable unconfined compressive strength of the soil-lime mixture is to be determined by the pavement design engineer. As a guideline, recommended values of unconfined compressive strength are presented in Table 17.

Table 17. Soil-lime mixture recommended unconfined compressive strengths (NLA 2006).

Pavement section	8-day extended soaking [psi]	Cyclic Freeze-Thaw **		
		3-cycles [psi]	7-cycles [psi]	10-cycles [psi]
Rigid Pavement	50	50	90	120
Flexible Pavement (> 10 in) *	60	60	100	130
Flexible Pavement (8 – 10 in) *	70	70	100	140
Flexible Pavement (5 – 8 in) *	90	90	130	160

* Total pavement thickness over the treated subgrade
** Number of expected freeze-thaw cycles in the treated subgrade during the first winter.

If target compressive strengths are not met, increase the lime content by 1 or 2% and repeat steps 3 and 4.

6.4.2. Deviation from Standard Procedures

In the presence of soluble sulfates, the chemical reactions triggered by the introduction of lime and water can lead to the formation of ettringite and thaumasite. Ettringite and thaumasite crystals swell upon formation. When these minerals form, the stabilized soil undergoes significant volumetric changes. The effect of the expansive minerals formation is aggravated when crystals grow in concentrated regions as supposed to homogeneously distributed throughout the stabilized soil layer. In such cases differential volume expansion occurs, altering the curvature of the pavement surface and which requires immediate corrective action. Correction typically implies pavement demolition and reconstruction. The presence of soluble sulfates is only evidenced during long term pozzolanic reactions. Cation exchange, flocculation, and carbonation occur quickly (minutes to hours) and produce the effects expected of an appropriately stabilized subgrade soil. Thus, no immediate indication of potential heave may be observed during construction or even within months from opening to traffic (Hunter 1988).

Sulfur can be found in soils in the form of sulfide (S^{2-}) bearing minerals such as pyrite, marcasite and greigite, and in the form of sulfate (SO_4^{2-}) bearing minerals such as gypsum, anhydrite, barite, and jarosite (Burkart et al. 1999; Czerewko et al. 2003). Sulfate can originate from the dissolution of sulfate bearing minerals or from the oxidation of sulfide bearing minerals, both of which depend on environmental conditions such as pH, temperature, and pressure (Little and Nair 2009). Therefore, the mere presence of these minerals does not imply the availability of aqueous sulfate for ettringite/thaumasite formation.

Among the multiple naturally occurring sulfate sources, gypsum is the most commonly observed in natural sediments (Czerewko et al. 2003). Its solubility under normal atmospheric conditions is 2.58 grams per litter of water (Burkart et al. 1999). Because of its low solubility, the availability of water is critical in determining the extent of gypsum dissolution. In dry arid climates, the extent and duration of precipitation events becomes a controlling factor. It is unlikely that a gypsum deposit will completely dissolve and release all available sulfate into the sediment after a single rain event. More likely, the release of sulfate ions will take place over time in small quantities following multiple precipitation events.

The stabilization of sulfate bearing sediments can be accomplished by regulating reactant concentrations and environmental conditions that favor ettringite/thaumasite formation. Calcium, sulfate, alumina, water, and high pH are all required for ettringite to form. In the case of thaumasite silica and carbonate are also essential. Controlling the relative concentrations of any of these constituents can limit the formation of the expansive minerals. Lime is typically proportioned in high quantities to maintain the high pH required to drive pozzolanic reactions in treated soils. Reducing the amount of lime used as to keep the pH below that required for ettringite formation can effectively control swelling; however, it can also negatively impact the long-term strength gain in the treated soil (Little and Nair 2009). Since the soil itself serves as a source for silica, alumina and sulfate it is impractical to control these reactants. Besides maintaining a low pH, an alternative method is to limit the amount of water. Ettringite requires copious amounts of water to form and grow. Limiting water intake during and after construction, through effective drainage, can also control swelling but may prove more challenging.

Alternatively, post-construction swelling can be minimized by doing the complete opposite, creating all the conditions to foster rapid ettringite formation. NCHRP 145 "Stabilization of sulfate rich soils" advocates for the addition of water beyond that required for optimum compaction followed by mixing and a time allowance for mellowing. The soluble sulfate content in a soil is the determinant factor in selecting a treatment option. Soluble sulfate contents below 3000 ppm (0.3% by weight in a ten-to-one water-to-soil solution) rarely cause performance issues, except if additional sulfate continues to migrate and accumulates in the soil. Soils with SO24- concentrations of up to 8000 ppm (0.8% by weight) can still be treated using lime, provided special construction procedures are followed. However, the use of lime in soils with soluble sulfate concentrations exceeding 8000 ppm is not recommended (Jones et al. 2010; Little and Nair 2009; NLA 2000; NLA 2006). The following treatment alternatives are recommended in NCHRP 145 for these three different cases (Little and Nair 2009):

6.4.2.1. Traditional lime treatment (soluble sulfates < 3000 ppm)

If the total soluble sulfate concentrations are below 3000 ppm or 0.3% by dry weight of soil, lime stabilization should not pose a significant concern. While there is still potential for swelling, its magnitude can be managed by adequate moisture treatment as long as external sulfate migration can be prevented. ***Whenever soluble sulfates are detected, lime slurry should be used to ensure complete lime hydration at the time of construction***.

6.4.2.2. Modified treatment (3000 < soluble sulfates < 8000 ppm)

In this range of soluble sulfates contents the soil exhibits a moderate risk of deleterious swelling caused by ettringite formation. Lime stabilization can be attempted only when accompanied by special mixture designs and construction practices. Lime slurry must be used and the moisture content needs to be increased above the optimum compaction water content in order to foster ettringite formation. The soil-lime mixture must be allowed to 'mellow' for a period of time sufficiently long for ettringite to form prior to compaction. Both the mellowing time and the water content are determined by monitoring the soluble sulfate concentration over time. As ettringite forms, sulfate is consumed. The mellowing time at a given water content is the time needed for the sulfate content to fall below 3000 ppm.

6.4.2.3. Alternative treatment (soluble sulfates > 8000 ppm)

At high sulfate contents the risk for deleterious swelling is very high. ***Successful stabilization of these soils is very challenging and should be attempted only with the outmost care***. Alternative treatments:

- ✓ Remove the sulfate bearing soil and replace it with a soil that does not require a chemical stabilization treatment to meet performance requirements.
- ✓ Blend the sulfate bearing soil with more competent material.
- ✓ Dilute the sulfate concentration by mixing in a soil with no sulfates and stabilize using lime. Proportions should be determined based on performance testing to guarantee volume expansion does not exceed 2%.

6.5. GEOGRID REINFORCEMENT

Geogrids are the main type of geosynthetics used in subgrade reinforcement, their use dates back to the 1970's (Giroud and Han 2004). Geogrids are typically made of polyethylene or polypropylene that is either extruded, stretched, woven, or knitted to form a grid with relatively large openings (i.e., one or multiple gravel size particles can fit within the openings). Geogrids are often employed as reinforcement over the subgrade for base and sub-base layers. While the properties of the subgrade soil are not altered directly, geogrid reinforcement alters the mechanical response of the pavement structure. The geogrid can improve the subgrade performance through four mechanisms (Giroud and Han 2004; Shukla 2012; Zornberg 2011): (i) prevention of local subgrade shearing, (ii) redistribution of stresses, (iii) reduction or reorientation of shear stresses, and (iv) tensioned membrane effect.

When the vertical stress on the subgrade reaches the elastic limit of the soil, it fails and local permanent shear occurs as the sub-base punches into the subgrade. Shear zones grow under repeated loading, causing deterioration of the sub-base and an increase in the magnitude of the vertical stresses on the subgrade, until eventually the bearing capacity of the subgrade is reached and complete failure occurs. The presence of geogrids between the sub-base and the subgrade prevents the development of local shear zones (Giroud and Noiray 1981), see Figure 28-b. Since the geogrid increases the stiffness of the sub-base layer, the magnitude of the vertical stresses on the subgrade decreases, as the surface load is spread over a larger area. Furthermore, the interlocking between the geogrid and the sub-base protrudes into the subgrade creating a rough surface that resists lateral movement within the subgrade. This 'inward shear' resistance increases the subgrade bearing capacity (Perkins 1999). Finally, when rut develops over the surface and across the sub-base-subgrade interface, the deformed shape of the geogrid causes an increase in confinement in the subgrade and a reduction in the vertical stresses under the load (Perkins 1999), see Figure 28-c.

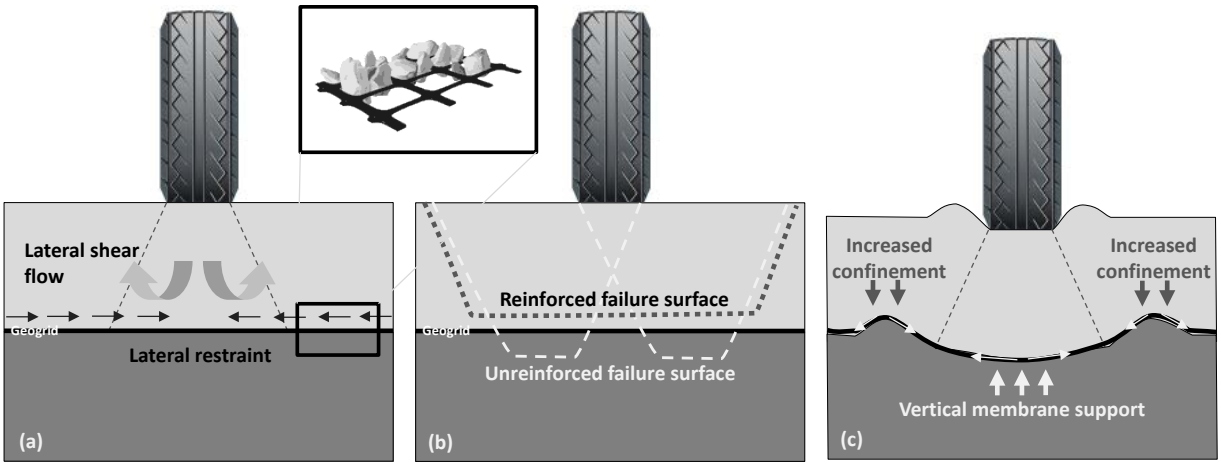


Figure 28. Geogrid subgrade stabilization mechanisms (Haliburton et al. 1981): (a) lateral restraint and interlocking, (b) reduction and reorientation of shear stresses, and (c) tensioned membrane effect.

Geogrid reinforced working platforms accumulate plastic deformations at a much lower rate than unreinforced working platforms, and deformation ceases after a certain number of cycles. As a result, total deformation is reduced in the presence of the geosynthetic (Maxwell et al. 2005). Geogrids have also been used to stabilize roads crossing over expansive clays. The presence of a paved (i.e., impervious) surface over an existing clay deposit alters the shape of wetting and drying fronts. Water from precipitation events enters the expansive soil deposits through pervious shoulders causing the soil underneath to expand. Since the hydraulic conductivity of the clay is low, the wetting front is confined to the area under the shoulder resulting in differential volume expansion. Over time, these differential volumetric changes wear the pavement to the point of cracking. Introducing a geogrid reinforcement at the interface between the sub-base and the subgrade has been found to eliminate the longitudinal cracking in the pavement surface (Zornberg and Gupta 2009).

Geogrids have been found to be useful in handling the following subgrade conditions in both temporary and permanent road applications (Holtz et al. 1998):

- ✓ Poor soils such as: clayey sands, low plasticity silts and clays, high plasticity silts and clays, and organic soils.
- ✓ Soils that exhibit low undrained shear strength (< 90kPa), low CBR (< 3), and or low resilient modulus (< 30 MPa).
- ✓ Areas with high water table.
- ✓ Subgrades that exhibit high sensitivity.

Geogrids can also be used to reinforce sites where available soils are too weak to allow access to construction equipment. Even if the finished roadway can be supported by the existing subgrade, construction in certain soils can become very challenging. In such cases, the development of a working platform may require subgrade improvement (mechanical or chemical) or excavation and replacement. Geosynthetics can be deployed rapidly to offer a cost-effective solution in such cases.

7. CONSTRUCTION GUIDE

7.1 COMPACTION

Compaction construction is a 5 step process: equipment selection, determination of layer thickness, modification of field moisture content, and actual compaction. Preconstruction work starts with the selection of a compactor type suitable for the subgrade and embankment fill soils. Specific equipment characteristics are then used to determine the maximum compaction lift thickness allowed. Construction starts with the modification of field moisture content and mixing of the soil to achieve the moisture content prescribed in design documents. Once the correct moisture content is achieved, compaction equipment moves in. Construction is accompanied by periodic field density testing to ensure the target densities are met.

7.1.1. Equipment selection

The soil type and prescribed fill lift thickness dictate the type of equipment needed for compaction. Coarse-grained soils are best compacted using smooth-drum vibratory and/or rubber-tired rollers while fine-grained soils are best compacted with sheepsfoot or pad foot rollers. Pneumatic tire rollers are classified according to tire size, tire pressure and wheel loads, and work better in sandy soils (Figure 29-a). Vibratory drum compactors are classified based on drum weight and width, dynamic force, and operating frequency (Figure 29-b).). Smooth steel wheel rollers, 10 tons or heavier, are very commonly used (Figure 29-c). Sheepsfoot rollers are better for clays as they compact the layer from the bottom of the lift upward towards the top (Figure 29-d). If the contractor decides to use a compactor that is not recommended for the particular type of fill soil, a test strip should be compacted to assess the suitability of the equipment. Whenever issues arise in attaining target compaction, an experienced geotechnical engineer should be sought for consultation.

7.1.2. Compaction lift thickness

The maximum thickness for each compacted lift is determined by the static weight of the compaction equipment used. Recommended maximum thicknesses are presented in Table 18. Note however that NMDOT specifies a maximum un-compacted lift thickness of 8 in (506.3.5.3). Issues related to heterogeneous compaction may arise when attempting to compact a layer thicker than recommended, and/or when inappropriate compaction equipment is utilized. In an excessively thick lift, only the soil exposed to high stresses may attain adequate density. The soil deeper in the lift where the stress influence is low remains unchanged.

Figure 29 Common types of compactors: (a) pneumatic tire roller, (b) vibratory drum compactor, (c) smooth steel wheel roller, and (d) pad foot roller, images courtesy of http://www.cat.com/.

Table 18. Recommended maximum compaction layer thickness as a function of the equipment static weight, after (Jones et al. 2010).

Max layer thickness [in]	Static weight [tons]
< 6	12
6 - 8	15
8	> 18

7.1.3. Modification of field moisture content

Control over the field moisture content is essential in any successful compaction job. Water content determines the maximum dry density that can be attained from a given compactive effort. Design documents should clearly state the optimum compaction water content as well as the acceptable tolerances above and below it. ***By NMDOT specifications (203.3.8), soils with plasticity index above 15 should be compacted at water content between optimum and 4% above optimum. Otherwise, field water content must be between 5% below optimum and optimum***.

Depending on the actual field moisture, attainment of an acceptable water content may require wetting or drying (Lambe and Whitman 1969). ***In wet soils***, where the field water content exceeds optimum, compaction becomes difficult, target densities cannot be attained and the fill soils' strength decreases. Even slight excess water above tolerances will result in pumping of the

fill surface, further increase in water content will cause rutting. Reductions in field water content are best attained by plowing and disking or loosening the soil to expose it to dry moving air and sun. A disc harrow or a soil mixer offers good results. ***In dry soils***, where the field water content is below acceptable tolerances, compaction becomes more difficult, and will require additional passes of the compactor. Even with the additional effort, target densities may not be attained. In such cases, water must be added to the fill soil, typically using a water tanker equipped with a sprayer nozzle. Flooding should be avoided because it would lead to excess moisture and delay compaction work.

7.1.4. Compaction

Compaction should proceed swiftly as long as the proper equipment and layer thickness have been selected. Furthermore, a well-mixed fill soil at the appropriate field water content will ensure that target densities can be attained with a minimal number of passes of the compactor. In certain occasions, several passes of the compactor may be needed before reaching a target compaction level. Field compaction is typically assessed through density measurements (e.g. nuclear density gauge, sand cone, or balloon test). Density testing is extremely important and should be conducted in a manner that tests the entire lift thickness. In certain cases the compactor may only alter the upper portion of the lift and create a hard crust on the surface while the rest of the lift below it remains uncompacted. ***Partially compacted lifts fail to perform as designed under service loads; thus, it is important to identify them during construction, when timely corrective actions can be taken***. The density of the lift should be higher than the target density specified in the compaction design documents. In case of difficulties:

Problem	Solution
An excessive number of passes are required to attain target compaction	1. Reduce the thickness of the lift 2. Use heavier compaction equipment 3. Adjust the fill's moisture content closer to optimum
Pumping is observed as the compactor moves over the fill	1. Make sure the compactor is a sheepsfoot roller 2. Loosen the soil and allow it to dry, then attempt compaction again 3. If conditions persist mark the area as unstable soil
Cannot achieve specified compaction, or density test results exceed 102% or are below 88%	1. Do a field "one-point" compaction test 2. If the "one-point" compaction test is not close to expected, or if soil appears different throughout the lift, re-sample the soil and submit for compaction testing.

Proof rolling

Proof rolling consists of driving heavy construction equipment (proof roller) over the subgrade while monitoring the deformation of the road. Close inspection of the subgrade response throughout the procedure allows for qualitative evaluation of the compacted fill performance. Locations where deflection, cracking or rutting of the surface of the fill are observed indicate instability and should be marked for correction or investigation. The extent of the unstable areas

dictates the type of corrective actions best suited for the problem. Small contained defects are better dealt with by removal and replacement (subexcavation) or geogrid reinforcement. Rapid field tests can be used to screen for deleterious soil conditions. If there is no indication of such conditions, construction can proceed without causing significant delays. In larger areas with more frequent defects, or where field tests indicate deleterious soil conditions, soil samples need to be collected and submitted for laboratory testing.

Proof rolling should be conducted immediately after the subgrade compaction operation. This minimizes the likelihood of changes in the field water content affecting the proof rolling evaluation. If the compacted fill becomes wet, the material will displace and rut during proof rolling. If the compacted fill dries out, a surface crust may form which can carry the proof roller over soft spots, effectively hiding defective areas of the fill. It is also very important to select a proof roller appropriate for the embankment's soil type. An excessively heavy proof roller may cause a well compacted fill to become unstable. Conversely, a light proof roller may help to hide soft areas (Ohio DOT). For soils classified as A-3, A-6, and A-7, it is recommended to use a rubber-tired 35 ton roller with a tire pressure of 120 psi. For coarse-grained soils the recommended roller weight is 50 tons at a tire pressure of 150 psi (ODOT 2013).

7.1.5. Environmental effects

In freezing weather compaction must be halted. At temperatures below 32 degrees Fahrenheit there is a risk for ice to form during compaction. In such cases, subsequent melting of the ice leaves a porous structure in the subgrade prone to collapse and failure. In cold weather, check the air and ground temperature periodically as compaction proceeds to ensure it is above freezing. Compaction tests should be conducted more frequently whenever temperature falls below 40 degrees Fahrenheit. Throughout the state, these adverse conditions should be expected between November and February.

7.2. BLENDING

Construction procedures for soil blending consist of placement, mixing, moisture conditioning and compaction. In a soil blending project, coarse-grained soil is admixed into the subgrade and/or the fill to alter its grain-size distribution. Therefore, it is of outmost importance to ensure that the soil added does not contain fine material (aggregates passing the #200 sieve). In certain cases fine material may disguise itself as coarse grains; this is particularly the case in clays that have experienced desiccation. Careful inspection of the additive material must be conducted prior to and during placement, a slaking test can be readily conducted on a coarse grain soil to determine if it contains desiccated clays. The jar-slake test is a simple field screening test proposed in the NMDOT Shale Use Policy Report (Lommler 2009) for evaluating the weathering resistance of shales. The test can be modified for use as a screening test for coarse grained material suspected of containing clay. In the test, coarse grained particles are immersed in water inside a glass jar. The reaction between the material and the water is monitored visually for a few minutes. If the material disintegrates into a pile of flakes and the water turns into mud, the blending material contains fine grained soil and it is unsuitable for blending. ***Using material***

that contains fines will result in failure not only of the subgrade stabilization project, but also of the entire pavement.

7.2.1. Placement, mixing and moisture conditioning

Spreading, moisture conditioning, and mixing are extremely important for the success of a stabilization project. The coarse grained material must be spread evenly over the subgrade and mixed into it down to the prescribed treatment depth using approved equipment. In dry subgrade conditions, moisture conditioning should be conducted preferably during mixing to avoid the generation of dust. In wet subgrades, where the field water content exceeds optimum, reductions in water content are best attained by plowing disking and turning over the subgrade soils to expose them to dry moving air and sun.

7.2.2. Compaction

Once the blended material has been mixed into the subgrade resulting in a homogeneous layer, the material can be compacted following the guidelines prescribed in section 7.1.

7.3. CEMENT

Construction procedures for cement stabilization are very similar to those followed for the compaction of road subgrades. The main differences lie in the timing and order of the procedures which must be adapted for weather conditions (wind, temperature, rain) and field water content. The ultimate goal is to obtain a dense homogeneous layer compacted within the allowed tolerances in water content.

Construction of soil-cement compacted subgrades involves: equipment selection (both compaction and mixing), cement spreading, moisture conditioning and mixing, compaction, and curing. Figure 30 depicts the different steps involved in the construction of cement stabilized subgrades. Cement spreading must be controlled to ensure even distribution and homogeneous cement content (Figure 30-a). Mixing and moisture-conditioning the soil-cement can be conducted simultaneously using a soil reclaimer by injecting water into the mixing drum (Figure 30-b). Compaction must follow immediately after mixing as the chemical reactions that cause cementation start the moment cement comes in contact with water (Figure 30-c and 30-d).

Cement should be brought shortly before placement and preferably not be stored in the field. Whenever it does need to be stored, it should be done in air tight containers that can protect it from moisture.

7.3.1. Equipment selection

Compaction equipment

Compaction equipment selection depends on the type of subgrade. In the majority of subgrade stabilization projects the soil to be treated will be fine-grained. Therefore, compactors that are

capable of exerting kneading action must be used, see an example in Figure 30-d. Sheepsfoot or pad foot rollers are typically the best alternative available for compaction of the cement treated layer. Steel-wheel or pneumatic tire rollers should be used to finish the surface of the compacted layer (ADOT 2008). Unlike regular compaction jobs, for cement stabilization it is preferable that the layer be compacted in a single lift. For a typical 12 inch stabilization depth, the minimum compactor weight required is 25 tons.

Figure 30 Construction of cement stabilized subgrade: (a) cement spreading, (b) soil-cement mixing and water conditioning, (c and d) compaction (White et al. 2013).

Mixing equipment

The choice of mixer depends primarily on the stabilization depth. Equipment capable of handling mixing and moisture conditioning simultaneously should be favored over alternative options.

7.3.2. Cement spreading, moisture conditioning and mixing

Cement spreading, moisture conditioning, and mixing are extremely important for the success of a cement stabilization project. Specific procedures depend on field water content and environmental conditions at the time of mixing. Under no circumstances should cement stabilization be attempted if the temperature is below 40º Fahrenheit in the shade, or if it is

anticipated that temperature will fall below 40º Fahrenheit within 24 hours (ADOT 2008). The following alternatives take into account a combination of factors that should be considered.

Dry placement

Weak to no wind

Field water content lower than the design water content

Dry placement is the preferred alternative for cement placement. Do not apply cement in windy conditions which could cause cement blowing. Blown cement reduces the amount of material available for the stabilization job and can negatively impact traffic, workers, and neighbors. Cement should only be placed over an area where mixing, compaction and finishing can be achieved during the same working day. Moisture conditioning can be done before or during mixing (TXDOT 2004):

Before mixing, condition the subgrade by adding water and mixing the material as to get the field water content as close the design water content as possible. Proceed to distribute the required quantity of cement uniformly over the subgrade and mix it into the subgrade using approved equipment.

During mixing, distribute the required quantity of cement specified in design documents over the subgrade. Proceed to mix the cement into the soil using a soil mixer equipped with water sprayers inside the mixing chamber. Enough water should be added during mixing to attain but not exceed the design water content.

In both cases, compaction must proceed swiftly after mixing in order for the material to attain target performance levels.

Slurry placement

Medium to high wind

Field water content lower than the design water content

Slurry placement can be a good method to eliminate cement blowing in windy conditions; however, mixing must be conducted very rapidly. Within two hours of adding water to the cement, the slurry should be placed, mixed, and compacted to ensure the cement cures and hardens within the subgrade (ADOT 2008; TXDOT 2004). ***Failure to spread, mix in the slurry, and compact the material within the allowable time will lead to unsatisfactory material performance***.

7.3.3. Compaction

The thickness of the treated compacted layer should not exceed 12 inches, water content should be within 2% of the design water content, and compaction should yield at least 95% of the design dry density. Compaction should be started using sheepsfoot rollers to attain the target field density and finished using steel-wheel or pneumatic rollers to finish the subgrade surface. No more than two and a half hours should elapse between the time cement is exposed to water and the time compaction is completed (ADOT 2008).

The compacted subgrade should exhibit a uniform surface reasonably smooth and in conformity to the lines, grades, dimensions, and cross sections prescribed in design documents (ADOT 2008).

7.3.4. Curing

Water in the compacted layer is susceptible to evaporation particularly in dry, hot, windy conditions. Loss of water affects the curing process of the cement treated subgrade and lead to significant reductions in performance. A bituminous curing seal should be laid over the compacted cement-treated subgrade within a few hours of compaction completion, as soon as it is practical. The sealed subgrade should be kept free of heavy equipment traffic for at least 72 hours. Construction of subsequent pavement layers can resume no earlier than 10 days after sealing.

7.3.5. Environmental effects

The weather plays a fundamental role in the construction of cement stabilized subgrades. Cold weather (temperatures below 40º F) will slow the reaction of the cement and is not recommended. These circumstances occur mainly between November and February throughout the state. Freezing will deprive the soil-cement mixture of the water needed for cement hydration and for compaction. Rain can result in excess water which could affect the hydration process (dilution), wash out the stabilizer, and make it difficult to attain compaction within the allowed time tolerance. These conditions are prevalent in the months of July and August. Windy conditions may cause safety and environmental problems when the project plans call for dry cement mixing. Windy conditions are mostly concentrated in the spring and fall seasons.

7.4. LIME

Lime stabilization construction procedures are very similar to those followed for cement stabilization of road subgrades, although lime stabilization is only used for fine-grained, clayey subgrade soils. The primary differences lie in the time tolerances. Lime reacts at a much slower rate than cement when exposed to water. Therefore, there is more time allowed for mixing and compacting. As in the case of cement stabilization, the timing and order of the procedures must be adapted for weather conditions (wind, temperature, rain) and field water content. Lime is add-mixed to the soil in either powder or liquid form as either calcium oxide (CaO – also known as quicklime) or calcium hydroxide ($Ca[OH]_2$ – also known as hydrated lime). Quicklime and hydrated lime cannot be mixed together, because water demand, curing time, and general handling procedures are different for each (ADOT 2005). Whatever kind of lime is used, it must be kept protected from moisture and stored in the field for as short a time as possible.

Construction of soil-lime compacted subgrades involves: equipment selection (both compaction and mixing), lime spreading, moisture conditioning and mixing, compaction, and curing.

7.4.1. Equipment selection

Compaction equipment

Only compactors capable of exerting kneading action should be used for the compaction of lime stabilized subgrades, see an example in Figure 30-d. Sheepsfoot or pad foot rollers are typically the best alternative available for compaction of the lime treated layer. Steel-wheel or pneumatic tire rollers should only be used to finish the surface of the compacted layer, never as the primary means of compacting the entire lift. It is preferable that the layer be compacted in a single lift. For a typical 12 inch stabilization depth, the minimum compactor weight required is 25 tons.

Mixing equipment

The choice of mixer depends primarily on the stabilization depth. Equipment capable of handling mixing and moisture conditioning simultaneously should be favored over alternative options.

7.4.2. Lime spreading, moisture conditioning and mixing

Lime spreading, moisture conditioning, and mixing are extremely important for the success of a stabilization project. Specific procedures depend on field water content and environmental conditions at the time of mixing. Under no circumstances should lime stabilization be attempted if the temperature is below 40º Fahrenheit in the shade, or if it is anticipated that temperature will fall below 40º Fahrenheit within 24 hours. Placement and mixing of dry lime in powder form should be conducted only on windless days. Lime, quicklime in particular, can pose serious safety and environmental issues when blown by wind. The following placement alternatives take into account a combination of factors that should be considered.

Dry placement

Weak to no wind

The dry method is recommended only for placement of hydrated lime and pelleted quick lime. Dry placement is not advised for use with quicklime powder. Distribute the required amount of lime evenly over the treatment area. Lime should only be placed over an area where mixing, compaction and finishing can be achieved during the same working day. Moisture conditioning can be done before or during mixing (TXDOT 2004):

Before mixing, condition the subgrade by adding water and mixing the material as to get the field water content as close to the design water content as possible. Proceed to distribute the required quantity of cement uniformly over the subgrade and mix it into the subgrade using approved equipment.

During mixing, distribute the required quantity of cement specified in design documents over the subgrade. Proceed to mix the lime into the soil using a soil mixer equipped with water sprayers inside the mixing chamber. Enough water should be added during mixing to attain but not exceed the design water content.

Compaction must follow within the time prescribed in design documents. Compacting the lime stabilized soil too early can be even more detrimental than delaying it for too long. The attainment of target performance depends very strongly on strict adherence to design specifications.

Slurry placement

Medium to windy

Field water content lower than the design water content

Slurry placement can be a good method to eliminate the risk for lime blowing whenever windy conditions are prevalent. The slower rate of reaction with lime allows for a larger tolerance in the time of placement. Lime slurry should be placed and mixed the same day lime is exposed to water. ***Failure to spread, mix in the slurry, and compact the material within the allowable time window will lead to unsatisfactory material performance***.

7.4.3. Compaction

The thickness of the compacted layer should not exceed 12 inches, water content should be within 2% of the design water content, and compaction should yield at least 95% of the design dry density. Compaction should be started using sheepsfoot or pad foot rollers to attain the target field density and finished using steel-wheel or pneumatic rollers to finish the subgrade surface. Compaction must be completed within the allowable time window, no earlier and no later than prescribed in design documents. The compacted subgrade should exhibit a uniform surface reasonably smooth and in conformity to the lines, grades, dimensions, and cross sections prescribed in design documents (ADOT 2008).

7.4.4. Curing

Water in the compacted layer is susceptible to evaporation particularly in dry, hot, windy conditions. Loss of water affects the curing process of the lime treated subgrade and lead to significant reductions in performance. A bituminous curing seal should be laid over the compacted lime-treated subgrade within a few hours of compaction completion, as soon as it is practical. The sealed subgrade should be kept free of heavy equipment traffic for at least 72 hours. Construction of subsequent pavement layers can resume no earlier than 10 days after sealing.

7.4.5. Environmental effects

The weather also plays a fundamental role in the construction of lime stabilized subgrades. Cold weather (temperatures below 40º F) will slow chemical reaction rates which is detrimental to the stabilization process. These circumstances occur mainly between November and February throughout the state. Rain can result in excess water which could affect the hydration process (dilution), wash out the stabilizer, and make it difficult to attain compaction within the allowed time tolerance. These conditions are prevalent in the months of July and August. Windy conditions may cause safety and environmental problems when the project plans call for dry cement mixing. Windy conditions are mostly concentrated in the spring and fall seasons.

7.5. GEOGRID

Construction of geogrid reinforced subbases and bases is a two steps process. First, the geogrid needs to be installed, and then the aggregate fill is placed and compacted. The simple process can be accomplished rapidly and without the need for specialized equipment. Only geogrid approved by the design engineer should be used in the project. Rolls should be furnished with protective wrapping effective against ultraviolet exposure. They should be individually labeled to provide product identification including: product type, manufacturer, quantity, lot number, roll number, date of manufacturing, date of shipping, and the project to which it is assigned (ADOT 2008). If stored outdoors, geogrid rolls must be placed above ground on an elevated platform and protected with a light colored, opaque, waterproof cover. Geogrid must not be exposed to ultraviolet light for a period exceeding 14 days, stored in temperatures below 0 ºF, or in extreme heat.

7.5.1. Geogrid installation

Geogrid installation is also susceptible to weather conditions. The geogrid shall not be placed at times of wet and snowy conditions, heavy rainfall, or extreme cold or heat. First clear the installation area by removing sharp objects, large stones, or any debris from the surface of the compacted subgrade. If the grid is to be used in the construction of a working platform, excavate and remove the topsoil leaving a firm and smooth surface, free of unsuitable materials. Geogrid rolls should be rolled along the alignment in the direction of advancing construction. Torn, damaged, or defective geogrid must be rejected. If multiple rolls are required to cover the reinforcement area, leave an overlap between them as prescribed in design documents. Typical overlaps are between 2 and 3 feet depending on the R-value of the underlying soil. At transverse joints, the preceding roll must lay over the following roll. Wrinkles and folds must be removed prior to securing the grid in place with pins. Typical pins consist of 4 in long $3/16^{th}$ in diameter steel bars with either 1.5 inch diameter washers or u-shaped endings.

The geogrid should be hand tensioned prior to anchoring it to the ground with the pins. Anchoring locations are selected at the edges, including overlaps, and along the center at 30 ft intervals. Overlaps along curves can be made by folding (Figure 31) or cutting (Figure 32) the roll. The use of anchoring pins may be reduced or even eliminated at direction of the supervising engineer as long as it can be shown that the geogrid can be adequately tensioned by hand and anchored by placing aggregate in a progressive installation process (ADOT 2008). After placement the grid must be inspected for damage. Any holes, rips or tears present must be fixed by placing an overlay of geogrid directly over the damaged area leaving an overlap no less than 3 feet. Excessively large damages must be fixed by replacing the grid over the affected area.

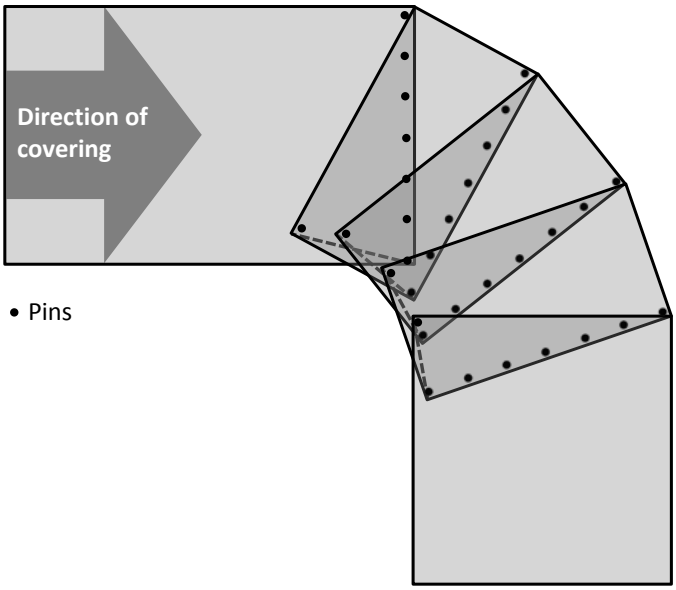


Figure 31 Geogrid placement over curves using folds.

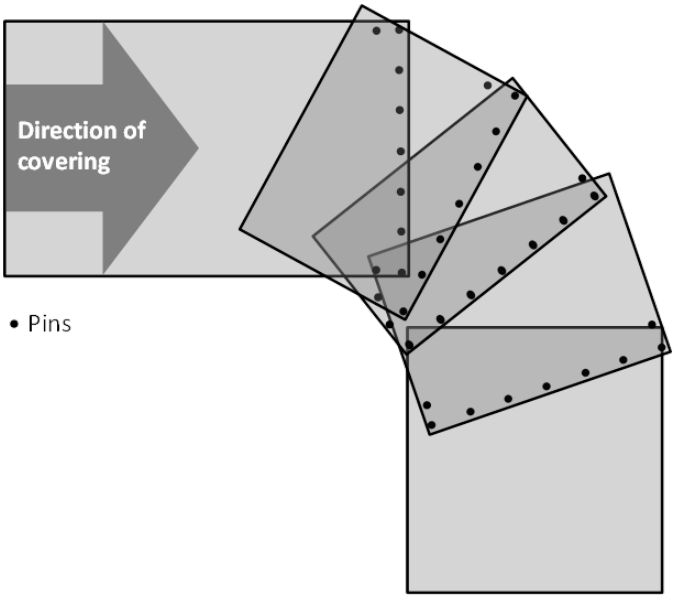


Figure 32. Geogrid placement over curves using Cuts.

7.5.2. Aggregate fill placement and compaction

The aggregate layer must be placed and spread before compaction. The minimum recommended thickness for the unbound aggregate layer is 4 inches. The aggregate must be back-dumped and bladed onto the geogrid by gradually raising the blade as it moves forward (Figure 33 b and c). End and belly type dump-truck-traffic directly over the grid should only be allowed if the subgrade can offer sufficient support. Trucks should drive no faster than 5 mph and dump the aggregate as they go. Sudden stops and/or turning should be avoided until there is a minimum of

6 in of aggregate over the geogrid. If surface deformations develop during spreading or compaction, fill in with additional aggregate instead of blading from surrounding areas.

Compaction should follow only after a sufficiently thick layer of aggregate has been laid over the geogrid (minimum 6 in). Material mixing and moisture conditioning should be carried out prior to dumping it over the geogrid. ***No material mixing or processing can be attempted once it is on the geogrid***.

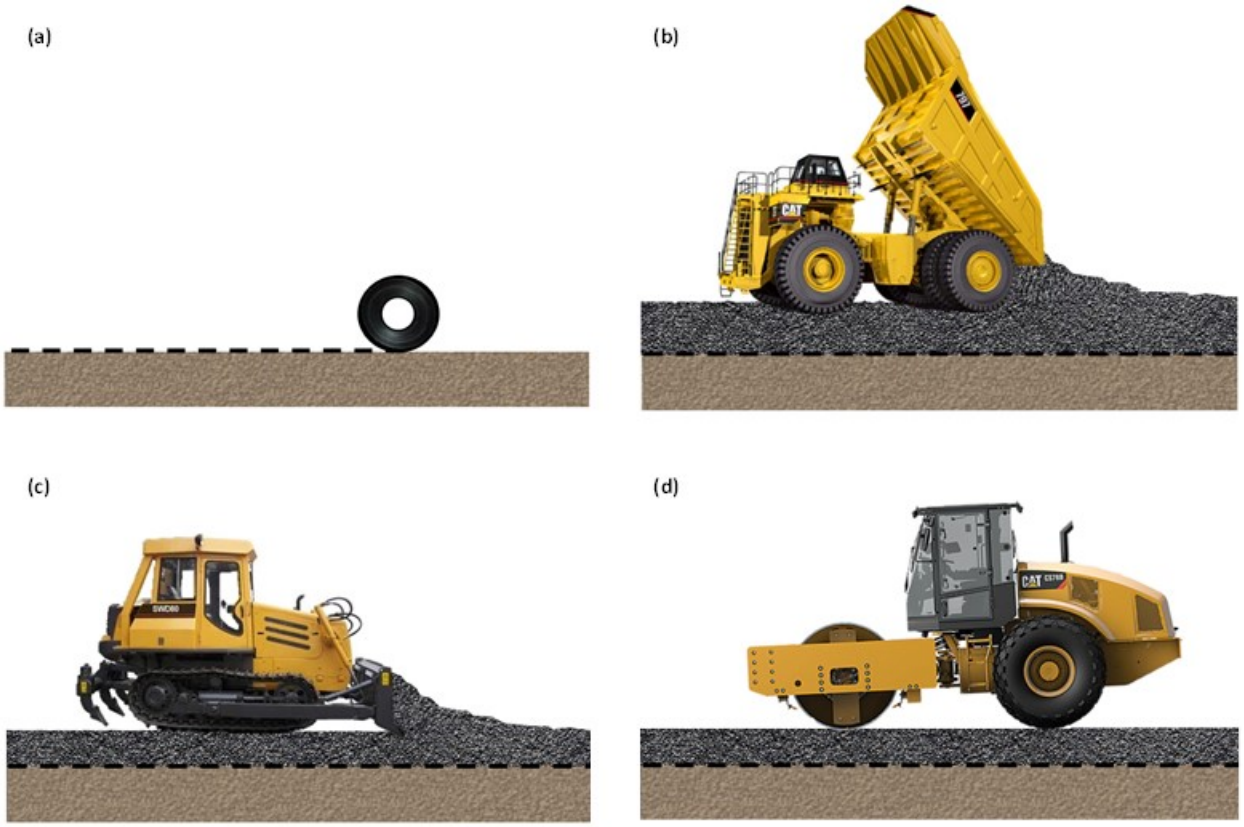


Figure 33. Aggregate fill placement and compaction.

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