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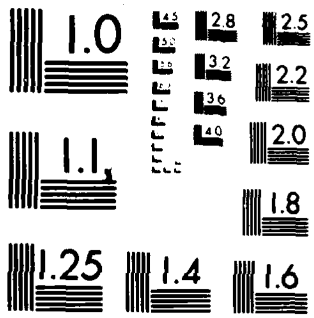
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CHEMICAL STABILIZATION OF SUBGRADE SOIL
FOR THE
STRATEGIC EXPEDITIONARY LANDING FIELD

A Special Research Problem

Presented to

The Faculty of the School of Civil Engineering

By

Michael H. Conaway

In Partial Fulfillment

of the Requirements for the Degree

Master of Science of Civil Engineering

N66324-70-A-0067

June 1983

GEORGIA INSTITUTE OF TECHNOLOGY
A UNIT OF THE UNIVERSITY SYSTEM OF GEORGIA
SCHOOL OF CIVIL ENGINEERING
ATLANTA, GEORGIA 30332

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Approved:

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Dr. Q. L. Robnett, Date

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ABSTRACT

The Strategic Expeditionary Landing Field (SELF) is a military expeditionary-type airfield with an aluminum matted surface that is designed for sustained tactical and cargo airlift operations in an amphibious objective area. Because of the operational traffic parameters such as loads of the various types of aircraft, tire pressures and volume of traffic, a base layer must be constructed over subgrade soil support conditions which may be only marginal. The base layer could be constructed with conventional soil construction techniques (compaction) and yield the required strength. It would be difficult, however, to maintain this strength for the required one-year service life under many climatic conditions due to the degrading effects of water on the support capacity of many soils. Chemical soil stabilization with lime, portland cement and asphalt stabilizing agents could be used to treat the soil. These additives, when properly mixed with certain types of soils, initiate reactions which will increase soil support strength and enhance durability (resistance to the degrading effects of water). Technically, this procedure is quite viable but logistically, it may not be feasible.

CHAPTER I

INTRODUCTION

Throughout their history, the U.S. Marine Corps and Navy have been called upon to support the nation in the implementation of foreign policy by providing expeditionary-type forces, usually amphibious in nature, in all parts of the world. The Marine Amphibious Force (MAF) is a highly mobile contingent of ground forces which is directly supported by elements of the Marine Air Wing (MAW). Modern amphibious warfare doctrine requires that advancing ground forces be given continual close air support which is a vital element of battle in the Amphibious Objective Area (AOA). This requirement to project tactical air power ashore became very apparent in the early stages of Marine aviation support of combat operations in Vietnam [57].

When the MAF initiates its amphibious assault, air support is provided by aircraft carrier-based attack and fighter aircraft. Because of the Marine Corps' seagoing mission, seldom are existing airfields encountered close enough to the objective area from which tactical operations may be conducted. When executing close air support sorties, the critically important aircraft carriers must remain in proximity to the AOA. This greatly restricts their mobility, thereby increasing the susceptibility to attack. It is imperative, therefore, that expeditionary-type airfields be rapidly established ashore so that the land-based MAW may relieve the carriers of the support duties.

Construction of expeditionary-type airfields is just one facet of a much larger system of support facilities which are crucial to the success of an amphibious assault. The Amphibious Logistic Support Ashore (ALSA) System provides a consistent and efficient flow of materials, equipment, services, and supplies to combat troops. It encompasses engineering, construction, maintenance, transportation and service functions of six component subsystems, which provides the required airfield(s) as well as other facilities such as supply roads, ammunition storage areas, fuel storage areas and many more. A more detailed discussion of this system can be found in Reference [26].

In the 1950's, the Marine Corps adopted a "vertical envelopment" concept [57]. Movement of troops and supplies ship-to-shore by helicopter following the securing of the beachhead by amphibious troops is now rudimentary to modern Marine warfare. Later, the advent of the vertical/short takeoff and landing (V/STOL) aircraft, the AV-8A Harrier, further supported the "vertical envelopment" concept. Both the helicopter and V/STOL aircraft can provide almost immediate logistical and tactical support to the operational commander. This, however, is contingent upon rapid construction of matted landing pads and short runways. Since the facility requirements are far less for the Harrier than for more conventional tactical aircraft and can be constructed relatively fast, the aircraft carrier-based squadrons can be augmented with Harrier aircraft within days of the amphibious landing. But until more advanced facilities are constructed, the carrier planes cannot be completely relieved as more than half of the tactical aircraft in the MAW are conventional, high performance attack and fighter planes.

Current AOA doctrine calls for a building block expansion of an initial 72 feet square matted vertical takeoff and landing (VTOL) pad, through several interim enlargements, until a 5,200 feet long by 96 feet wide matted Expeditionary Airfield (EAF) is constructed. Each phase of this expansion is designed to handle increasing numbers of aircraft of varying types that require greater support facilities. Consequently, each phase demands more construction effort than its predecessor. Theoretically, because the runway and parking surfaces are covered with prefabricated aluminum matting sections which piece together, and all airfield appurtenances such as lighting, communication and navigational aid systems are portable, the airfield could be relocated as necessitated by tactical developments. These facilities are designed and constructed rapidly to an expected service life of a few days for the VTOL pad, up to several months for the EAF.

Should the tactical situation warrant it, a Strategic Expeditionary Landing Field (SELF) may be constructed. This entails an 8,000 feet long by 96 feet wide runway, a parallel taxiway 78 feet wide, large parking and maintenance aprons, aircraft arresting gear, lighting, communications, navigational aids, and other support facilities. It, like the EAF, is surfaced with AM2 aluminum matting which, along with airfield appurtenances, are containerized and prepositioned for rapid deployment to the proposed site. Unlike the EAF and its predecessors, the SELF is designed to provide strategic airlift and tactical operations of a more permanent nature for up to a year. It is required to support one or more Marine Air Groups (MAGs), an element of MAW. A MAG consists of 96 aircraft as follows: 3 F4 Phantom or F18 Hornet fighter squadrons of 12 aircraft each; 2 AV4 Skyhawk or AV8 Harrier attack squadrons of 20 aircraft each; 1 A6 Intruder attack squadron of 12 aircraft each; and 1 KC-130 Hercules tanker detachment of 8 aircraft. The SELF must also provide transient parking and cargo handling facilities for 3 cargo aircraft, either the C141 Starlifter or C5 Galaxy from the Military Airlift Command, or an aircraft such as the DC-8 or DC-10 from the Civil Reserve Air Fleet.

The AM2 matting consists of 12 feet long by 2 feet wide extruded aluminum sections with a solid top and bottom. With an antiskid compound applied, it weighs approximately 6.8 pounds per square foot (psf). It is configured with underlap and overlap connections at the ends and hinge joint connections at the sides for relative ease of joining to adjacent mats. It was designed to withstand heavy static and dynamic gear loads for limited aircraft volume under marginal soil support conditions [52]. The AM2 is classified as a medium-duty mat [38]; accordingly, it is designed to withstand 1,000 coverages of a 25 kip (kilo pound equal to 1000 pounds) wheel load at a tire pressure of 250 pounds per square inch (psi) [52]. A coverage is defined when each point of the pavement within the design traffic width receives one load application [64]. It may be laid on an in-situ soil with a California Bearing Ratio (CBR) as low as 4 [52].

Because the SELF concept mandates sustained tactical and strategic airlift operations for a relatively long duration, a higher CBR value than 4 is required. Traffic volumes, tire pressures, wheel loads and configurations, anticipated service life, and the intended use of any pavement will dictate the required strength. Myriad soil support conditions could be encountered by the MAF. Most of these will probably compact, with proper construction techniques, equipment and moisture conditions, to a level which will yield the required CBR value. To maintain this strength over a period of one year, however, may prove to be quite difficult due to the degrading effects of water on the support capacity of most fine-grained soils. Water easily infiltrates the subgrade through the joints in the matting. Without taking some measure to prevent the deteriorating effects of surface water percolating into the subgrade, a one year service life for the SELF is highly questionable.

The purpose of this paper is to evaluate the use of chemical soil stabilization techniques to alleviate the nonconstant subgrade stability problem. This study will be restricted to the use of lime, portland cement, and asphalt stabilizing agents.

These stabilizers, when properly blended with varying types of soils, will often yield elevated CBR values and maintain them at acceptable levels under near saturated conditions. There are many variables which will affect the results of the stabilization process. These will be explored.

CHAPTER II

SELECTION OF STABILIZER

Introduction

Chemical Soil Stabilization offers many engineering and construction benefits and advantages over unstabilized soils [44, 45]:

a. function as a working platform (construction expedient)

b. reduce dusting

c. waterproof the soil

d. upgrade marginal aggregates or soils

e. improve strength

f. improve durability

g. control volume changes of soils

h. improve soil workability

i. dry wet soils

j. reduce pavement thickness requirements

k. conserve aggregates

l. reduce construction and haul costs

m. conserve energy

n. provide a temporary or permanent wearing surface

In the application of soil stabilization to the SELF, only a few of these are important, although most of them could provide some benefit. The primary objective is to construct this facility as rapidly as possible without sacrificing the very basic engineering principles that are elementary to its performance. Since a support CBR value of about 7 will be required, improving strength is certainly an important factor for many subgrade conditions. This could be accomplished during construction, depending upon the soil type and moisture conditions, without stabilization through compaction; however, as discussed in Chapter I, resistance to the deteriorating effects of moisture could prove crucial to providing a one-year service life under many climatic conditions. Therefore, improving durability is the most important characteristic. The benefit of reducing thickness requirements is directed primarily toward the construction of subbase and base course layers in flexible and rigid pavements. Reducing the thickness of the strengthened subgrade layer under the SELF's matting, however, could reduce construction time. This will be explored in Chapter III. Stabilized soil can function as a working platform to expedite construction in areas where excessively wet subgrades are encountered. Even if conventional construction techniques (compaction) were to be employed to strengthen the subgrade, it may prove to be difficult, if not impossible, to begin construction operations with heavy equipment if an in-situ soil is extremely wet. Because of the "drying" effect that lime has on certain types of soils, some wet subgrades could be converted to a firm, dry surface in a short amount of time. This technique was successfully employed in the Mekong Delta of South Vietnam [44].

Proper mixing of a stabilizer and soil is imperative to gain the best results. Many clay soils, because of their composition and relatively high plasticity, are difficult to pulverize, i.e., to reduce to very fine particles, which is a key element to successful mixing. Lime can reduce the plasticity index of a soil or make it nonplastic, thereby altering its properties and creating a very friable condition.

The benefits of chemical stabilization described heretofore are but a few of the advantages that can be achieved over unstabilized soil construction. Because of the specific objectives of immediate strength improvement and improved durability for application to the SELF, these two benefits will be concentrated on. Other properties and characteristics of soil-stabilized mixtures will be mentioned where appropriate.

Selection of the Stabilizer

The first priority in planning a soil stabilization project is to select the best stabilizing agent for the given soil that will meet the design objectives. Because of the effects that lime, portland cement, and asphalt products have on various types of soils, the planning engineers must have some guideline to assist in selection of the proper stabilizer. Figure 1 provides a very basic flow chart which will assist in the selection. Notice that several engineering properties of the soil, namely gradation and Atterberg limits, must be known before this chart can be utilized. It will be assumed in the context of this paper that all required engineering data is available or can be reasonably estimated. Once these properties are known, the best soil stabilizer may be selected. Where flow along a specific path leads to a choice between two stabilizers, the top one in the chart is generally the better choice of the two.

At this point, the engineer has a good idea of which stabilizer to employ. To project how the soil will react with the stabilizer,

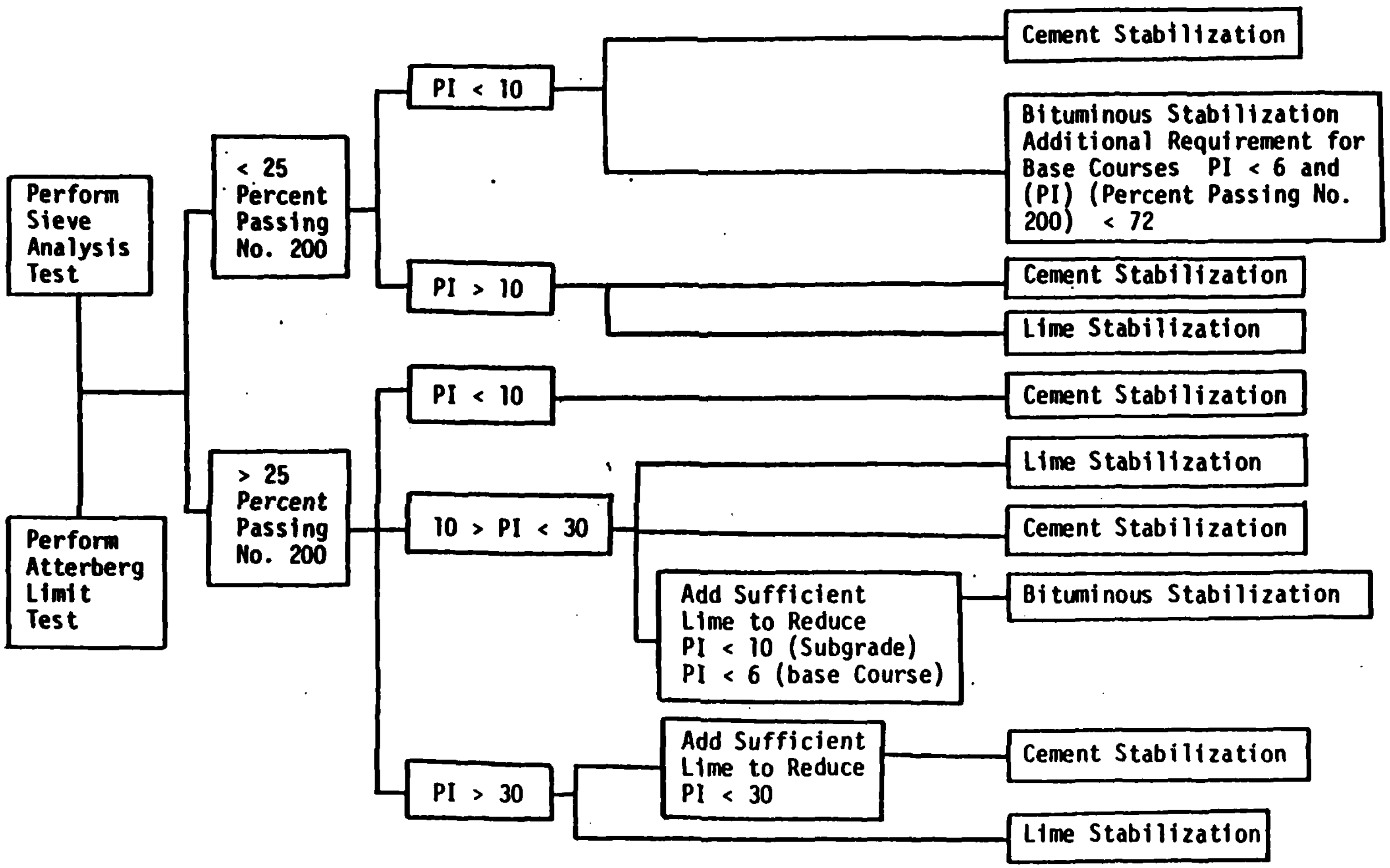


Figure 1. Selection of Stabilizers [44].

more specific engineering data is required. The soil should be classified under the Unified Soil Classification System (USCS). Figure 2 provides a USCS chart for referral.

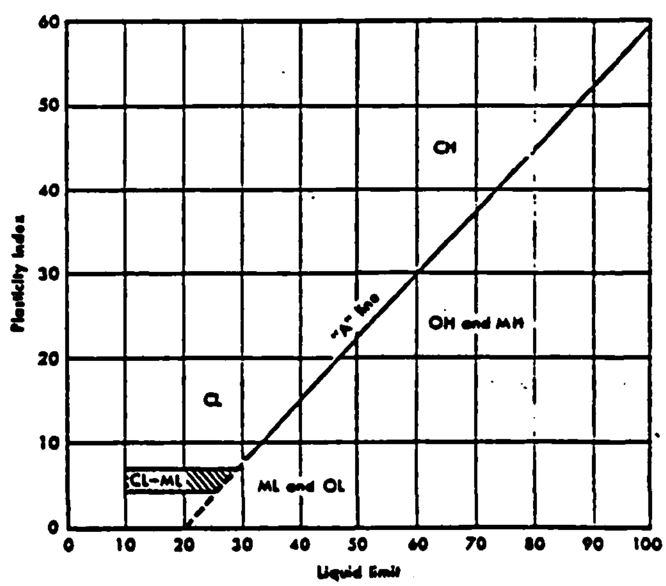
Three types of stabilizers will be evaluated: lime, portland cement, and asphalt products. Possible combinations of these agents where beneficial to the specific conditions will also be discussed.

Lime Stabilization

Lime, as used in soil stabilization processes, refers to oxides and hydroxides of calcium and magnesium. There are various types of lime that are commercially available. Calcitic quicklime and dolomitic quicklime can be used but they are caustic and can be dangerous to handle. These are produced by calcining calcite and dolomite limestone, respectively, and are used more in Europe than in the United States. By slaking quicklime, three forms of hydrated lime can be produced: high-calcium, monohydrated dolomite and dihydrated dolomitic. The first two are the most commonly used lime products for stabilization purposes [44]. Lime can also be obtained as a by-product of two industrial processes: (1) flue dust from the calcining process in lime production; and (2) from acetylene gas production from calcium carbide. By-product lime, however, may lack quality and should be evaluated before it is used.

Lime generally produces beneficial engineering effects in fine-grained soils. Several reactions occur when lime is introduced in these soils. Cation exchange and flocculation-agglomeration reactions take place rapidly and almost immediately produce changes in soil plasticity, workability and uncured strength.

Major divisions			Group symbols		Typical names	Laboratory classification criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 per cent ..... GW, GP, SW, SP; More than 12 per cent ..... GM, GC, SM, SC; 5 to 12 per cent ..... Borderline cases requiring dual symbols**	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM*	d	Silty gravels, gravel-sand-silt mixtures		Atterburg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterburg limits above "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM*	d	Silty sands, sand-silt mixtures		Atterburg limits below "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
				u				
			SC		Clayey sands, sand-clay mixtures		Atterburg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			



Plasticity Chart

Figure 2. Unified Soil Classification System.

*Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

**Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

Cation exchange is a very complex chemical phenomenon. Essentially, excess positively charged calcium ions contained in lime replace dissimilar cations in the exchange complex of the soil. Flocculation and agglomeration produce an apparent change in soil texture. The clay particles flocculate, i.e., form loose, fluffy lumps, and then gather in a cluster or larger-sized aggregations. Consequently, a "clayey" soil is changed to a "silty" texture. The cation exchange and flocculation-agglomeration reactions reduce the soil's plasticity which in turn makes it more workable from a construction aspect. These reactions occur in nearly all fine-grained soils [53].

Another reaction that takes place when lime is mixed with certain soils is the pozzolanic reaction. This occurs when the calcium from the lime reacts chemically with the silica and alumina minerals of the soil in the presence of water to form a cementing-type material. When a sufficient quantity of lime is added, the pH of the lime-soil mixture is elevated to approximately 12.4, the pH of saturated lime water. The solubility of the soil silica and alumina compound is greatly increased at elevated pH levels [44, 45]. Thus, the reaction is catalyzed. The cementing products are similar to those produced by the hydration of portland cement. The formation of these cementing agents effect substantial strength increases with reactive soils. The extent to which the cementitious material is formed is dependent upon the inherent properties and characteristics of the soil. These include soil pH, organic carbon content, natural drainage, presence of excessive quantities of exchangeable sodium, clay mineralogy, degree of weathering, presence of carbonates, extractable iron, silica-sesquixide ratio and silica-alumina ratio.

As previously mentioned, lime stabilization works best with fine-grained cohesive soils. To gain the best reactivity, in general, the soil should have a minimum clay content of 10 percent and a plasticity index greater than 10 [45]. Additionally, the percentage by weight of soil that should pass the number 200 sieve should be between 30 and 40. Benefits have been noted, however, in soils that do not fall into these categories [45]. Also, the type of lime and the quantity that is added to the soil will affect the reaction.

Curing time is a major factor since the strength continues to develop with time if proper temperature exists. Temperature exerts a major influence on the pozzolanic reaction: the higher the temperature, the faster the reaction will occur. Conversely, with lower temperatures, the reaction is retarded and virtually ceases at temperatures less than 40°F (4.4°C). Moisture must be present for the pozzolanic reaction to occur. Only a small amount of water, however, is required for hydration, and thus, optimum compaction moisture is retained or maintained if sufficient.

Unlike the cation exchange and flocculation-agglomeration reactions, pozzolanic reactions do not necessarily occur in most fine-grained soils. If cementing occurs, the soil is said to be "reactive." If no pozzolanic strength increase occurs, or it is relatively low, the soil is classified "nonreactive."

Carbonation is another reaction which may occur in lime-soil mixtures. This, unlike the others discussed herein, is highly undesirable. Lime reacts with carbon dioxide to form a carbonate. Chapter IV will discuss ways to minimize this reaction during the construction process.

Now that the various soil-lime reactions have been discussed, the next step is to focus on those engineering and construction advantages that are applicable to the SELF. Should a fine-grained, cohesive soil be encountered, the plasticity reduction aspects of lime-soil mixtures would be beneficial. This will greatly increase the workability or ease of manipulation of the soil, thus reducing construction effort. This benefit can be expected of nearly all fine-grained soils. Typical effects of lime on plasticity reduction can be seen in Table 1.

To discuss the pozzolanic reaction benefits of lime-reactive soils, the soil-lime mixture will be classified as uncured or cured. "Uncured" simply means the immediate effects. "Cured" means that the mixture has had time to develop increased strength as the pozzolanic reaction continues over time. Much of the data which will be presented shows laboratory curing parameters of 48 hours at 120°F (48.9°C). This is the time that the compacted soil specimens are kept in a drying oven at the specified temperature. This is a common practice for soil-lime mixture testing and these parameters approximately equate to field curing for 28 days at 70°F (21.1°C) [53].

Uncured mixtures experience immediate strength increases and moisture-density relationship changes. Immediate strength increases in terms of CBR are very important to the SELF. As discussed in Chapter I, because of the increased traffic volume, longer service life and other factors, the CBR of the subgrade must be a minimum of 7 which is higher than 4 for which the AM2 matting was designed. Figure 3 shows the immediate CBR increases of a USCS CL soil with the addition of 3% and 5% lime by dry weight of soil. At the moisture content range between 14 and

Table 1. Atterberg Limits for Natural and Lime-Treated Soils [44].

Soil	Unified Classification	Natural Soil LL	Natural Soil PI	3% Lime LL	3% Lime PI	5% Lime LL	5% Lime PI
Bryce B	CH	53	29	48	21	NP	
Clay Till	CL	49	27	51	12	59	11
Cowden B	CH	54	33	47	7	NP	
Drummer B	CH	54	31	44	10	NP	
Fayette C	CL	32	10	NP			
Hosmer B_2	CL	41	17	NP			
Piasa B	CH	55	36	48	11	NP	
Illinoian Till	CL	26	11	27	6	NP	

LL - Liquid Limit

NP - Nonplastic

PI - Plasticity Index

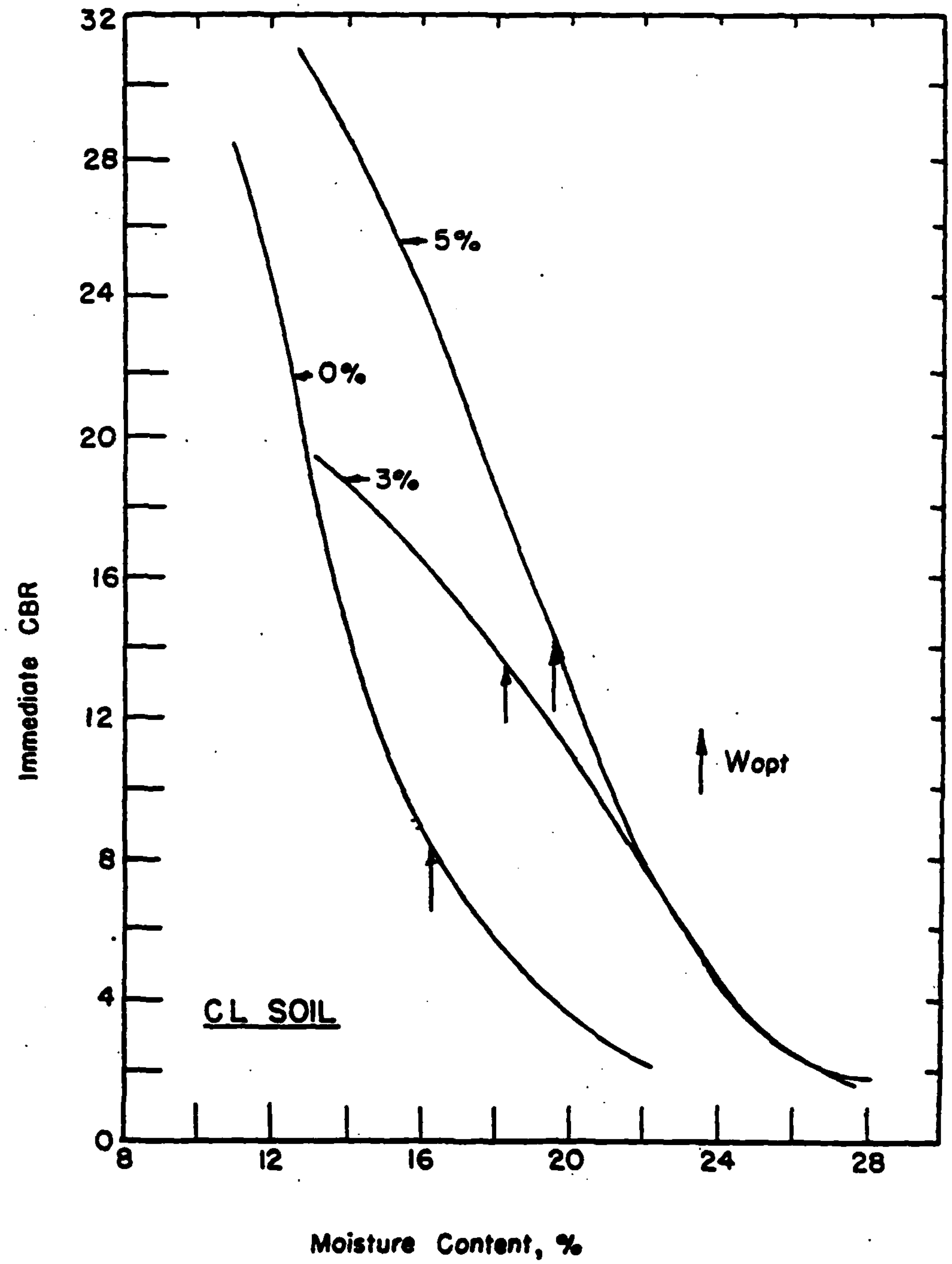


Figure 3. CBR-Moisture Content Relations for Natural and Lime-Treated (3%, 5%) CL Soil (AASHTO T-99 Compaction)[44].

20 percent, significant increases were noted. Even though the strength of the soil-lime mixture will increase with time, the immediate strength increase will permit aircraft operations at heavy loads in great volume as soon as the matting is laid. This may be highly desirable from an operational standpoint; there would be no concern by the operational commander over limiting loads, coverages or tire pressures as would be the case with a medium-duty mat such as the AM2 over a CBR of 4.

Compaction of the soil-stabilizer mixture will still be required which will be discussed in Chapter IV. Notice that the data presented in Figure 3 was compacted to AASHTO (American Association of State Highway and Transportation Officials) T-99 specifications. Soil compaction is characteristically required to be 95-100% of the maximum dry density achieved in the laboratory through AASHTO or ASTM (American Society for Testing and Materials) procedures. Another characteristic of soil-lime mixtures is a reduction of the maximum dry density of the soil and an increase in optimum moisture content. Maximum dry density reductions of 3-5 pounds per cubic foot (pcf) and optimum moisture content increases of 2-4% are common [45]. Figure 4 shows these changes for a USCS CL soil.

Cured lime-soil mixtures also exhibit enhanced engineering properties. As a function of time, the pozzolanic reaction continues if kept at the proper temperature. The CBR that can be developed in cured lime-fine-grained soil can exceed 100. Table 2 shows CBR test results for 15 different fine-grained reactive soils in their natural, untreated state, with lime added in the uncured state and cured under laboratory conditions for 48 hours at 120°F (48.9°C). Nearly all of the soils

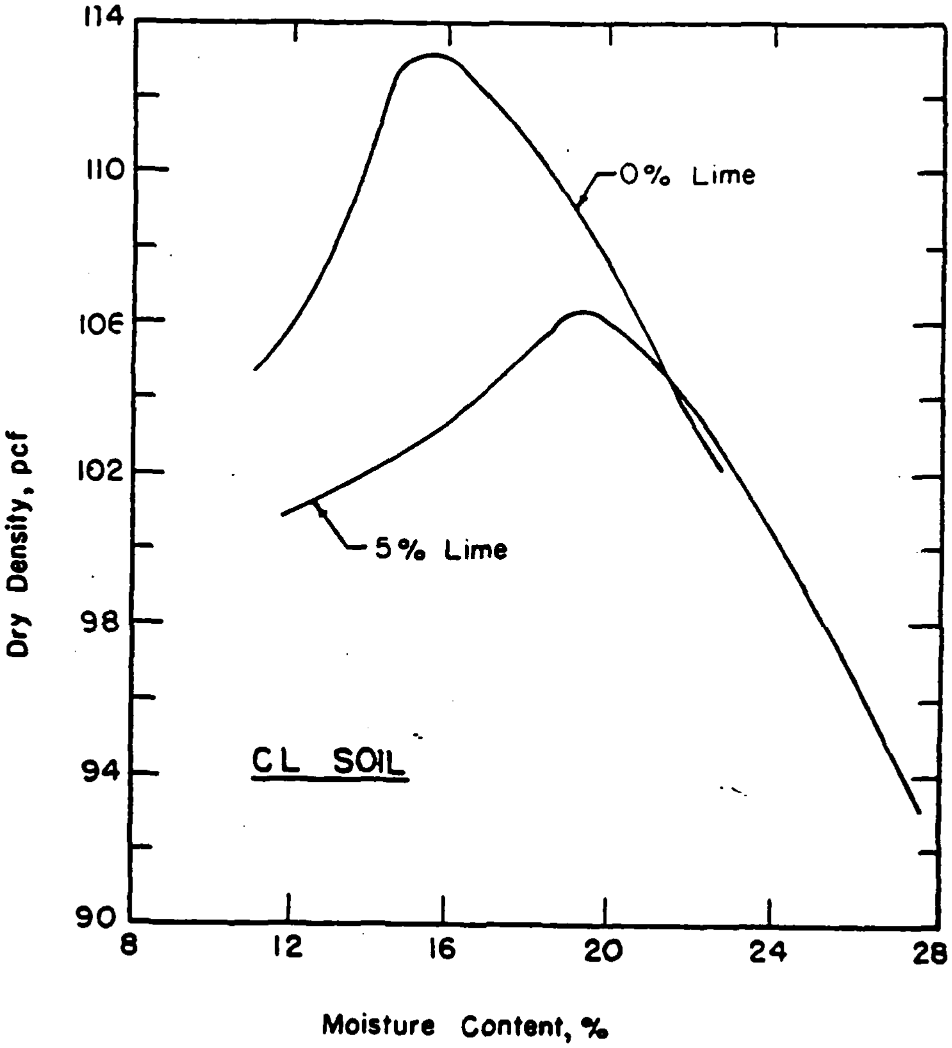


Figure 4. Moisture-Density Relations for a Natural and Five Percent Lime-Treated CL Soil (AASHTO T-99 Compaction)[44].

Table 2. CBR Values for Natural and Lime-Treated Soils [44].

Soil	Unified Classification	Natural Soil		% Lime	Soil-Lime Mixtures			
					No Curing[1)]		48 Hours Curing @ 120°F	
		CBR, %	Swell, %		CBR, %	Swell, %	CBR, %	Swell, %
Reactive Soils								
Accretion Gley 2	CL	2.6	2.1	5	15.1	0.1	351.0	0.0
Accretion Gley 3	CL	3.1	1.4	5	88.1	0.0	370.0	0.1
Bryce B	CH	1.4	5.6	3	20.3	0.2	197.0	0.0
Champaign Co. Till	CL-ML	6.8	0.2	3	10.4	0.5	85.0	0.1
Cisne B	CH	2.1	0.1	5	14.5	0.1	150.0	0.1
Cowden B	CH	7.2	1.4	3	--	--	98.5	0.0
Cowden B	CH	4.0	2.9	5	13.9	0.1	116.0	0.1
Cowden C	CL	4.5	0.8	3	27.4	0.0	243.0	0.0
Darwin B	CH	1.1	8.8	5	7.7	1.9	13.6	0.1
East St. Louis Clay	CH	1.3	7.4	5	5.6	2.0	17.3	0.1
Fayette C	CL	1.3	0.0	5	32.4	0.0	295.0	0.1
Illinoian B	CL	1.5	1.8	3	29.0	0.0	274.0	0.0
Illinoian Till	CL	11.8	0.3	3	24.2	0.1	193.0	0.0
Illinoian Till	CL	5.9	0.3	3	18.0	0.9	213.0	0.1
Sable B	CH	1.8	4.2	3	15.9	0.2	127.0	0.0
Non-Reactive Soils								
Fayette B	CL	4.3	1.1	3	10.5	0.0	39.0	0.0
Miami B	CL	2.9	0.8	3	12.7	0.0	14.5	0.0
Tama B	CH	2.6	2.0	3	4.5	0.2	9.9	0.1

[1)] Specimens were placed in 96 hour soak immediately after compaction.

exhibited significant immediate CBR increases and substantial cured CBR increases.

It has been stated that CBR values of 100 or more have little practical meaning [45]. The test essentially measures the soil's resistance to penetration and compares it to the resistance of crushed stone. The CBR is a ratio expressed as a percentage of the resistance to penetration of a soil as compared to a well-graded, crushed limestone which serves as the CBR = 100 material. Therefore, a number in excess of 100 is essentially meaningless. However, a CBR of 100 or more in a soil-lime mixture does indicate that the material has at least achieved the same ability to resist penetration as the limestone due to the pozzolanic cementing action. It has been further suggested that compressive and tensile strengths are better indications of actual strengths achieved [45, 53]. Many agencies, including military organizations who design expeditionary-type airfields, use CBR as the design parameter for pavements. It will be used as such in the context of this paper.

Notice that results for three "nonreactive" soils were evaluated in Table 2. Even these displayed increased uncured and cured CBR values, but not to the degree that the reactive soils did.

Another aspect of a cured soil-lime mixture, which is perhaps more important than increased strength, is improved durability. As stated heretofore, the detrimental effects of moisture on soil in an engineering application is of prime concern in design of the SELF. According to researchers [44, 45, 53, 56] who have done extensive study of soil-lime mixtures, prolonged exposure to water only produces slight detrimental effects. The ratio of soaked (nearly saturated) to unsoaked compressive strengths of the mixtures has been approximately .7 to .85 which is quite high [45, 53, 56]. Notice that the data in Fig. 6 for the uncured state had been taken after the specimens had soaked for 96 hours. Even in a soaked state, the specimens still exhibited increased CBR values.

The ability of lime-soil mixtures to resist the effects of water have been further supported by the successful use of lime in underwater stabilization. Many cases have been recorded where lime has been used in irrigation canals, reservoir bottoms, levees, and earth dams and has prevented softening of the soil, reduced leakage and even resisted erosion from flowing or percolating water due to the development of pozzolanic strength [22].

In addition to moisture exposure, the detrimental effects of freeze-thaw action should be considered if the SELF were to be constructed in an area where freezing temperatures occur. The damage is generally characterized by increased volume and reduced strength [56].

There are two basic types of freeze-thaw or frost action: heaving and cyclic freeze-thaw. Heaving results in a bulge in the surface of the pavement that can cause damage and make it unusable. This occurs in soils beneath that are frost-susceptible when ice lenses form and expand in a static frost condition (soil remains frozen) [56]. Most coarse-grained soils are not frost-susceptive, so there is little concern with them. Many fine-grained soils, however, are susceptive to heave. As an example, if lime is used to reduce the plasticity of a highly plastic clay (which is already somewhat frost susceptible), the more silty texture that is obtained makes the soil extremely susceptive to heave. Also, because of the high moisture condition and loss in soil density from the ice lense formation, soil strength is decreased. Sufficient pozzolanic strength must be developed to reinstate, and exceed, the heave resistance lost from the cohesive state. A minimum cured unconfined compressive strength of 200 psi (in excess of CBR = 20 [28]) of the lime-soil mixture must be obtained to minimize the volume change during heave to about 2% [56]. Therefore, even though a CBR of about 7 is required in the stabilized-soil layer under the matting for support purposes, a value in excess of 20 would be required to essentially prevent heave in a frost-susceptive soil. Insofar as strength goes, sustained freezing of a quality soil-lime mixture does not cause strength reduction [15]. Once it thaws, though, some strength reduction will most likely occur. The Corps of Engineers through research have classified soils as to their relative susceptibility to frost action [64].

Cyclic freeze thaw occurs when a frost line moves through a soil, causing a freeze and subsequent thaw. This is common in many regions, for example, when a soil repeatedly freezes at night and thaws during the day at certain times of the year. Repeated freeze-thaw cycles reduce the resilient modulus (weaken) most fine-grained soils [53, 56]. In a cured soil-lime mixture, enough strength is developed to substantially increase the resilient modulus, thus offsetting any reduction from freeze-thaw action.

Before the use of lime is implemented once it is chosen as the best stabilizer for a given soil, the engineer must insure that certain climatic conditions are taken into account; the soil should not be frozen when operations are initiated, the air temperature should be at least 40° F (4.4° C) and rising, and there should be at least two weeks of warm or hot weather prior to cooling temperatures. Once the soil-lime mixture has had a few weeks to cure and develop pozzolanic strength, cooling or freezing temperatures will cause a temporary cease in the strength gain process. There will be no further increase in strength but what has been achieved will not be lost.

In summary, lime stabilization works best with fine-grained soils. Medium and moderately fine soils may also benefit from lime stabilization. A decrease in plasticity, increased workability and increased strength can be expected. The following soils as classified under USCS should be considered for lime stabilization [44]: CH, CL, MH, SC, SM, GC, SW-SC, SP-SC, SM-SC, GW-GC, GP-GC, GM-GC, OL, and OH.

Cement Stabilization

Portland cement is a hydraulic cement made by calcining limestone with chalk or other substances. Its chemical composition includes calcium oxide (quicklime) and elements of silica and alumina. When portland cement is hydrated, calcium silicate and aluminate hydrate becomes the predominant cementing compound.

All types of portland cement have been successfully used in soil stabilization. Type I portland cement, which is considered the standard type, and Type IA, which is air-entraining cement, have been used extensively and have yielded similar results [44]. Type II seems to be the preferred type today because of its greater resistance to sulfate attack which can have a detrimental effect on hardened portland cement-soil mixtures. High early strength cement, Type III, is reported to have yielded higher strengths than the other cements in some soils [44].

Cement and lime soil stabilization are similar in many ways. Where lime had to derive silica and alumina from the soil to achieve the pozzolanic cementing action, cement contains these compounds in its chemical makeup and begins hydration and strength formation as soon as water is added. The reactions are essentially the same. The strength formation in coarse-grained soils is due to surface adhesion forces between the cement material, which is in a gel form once hydration begins, and the surface of the soil particles. This is very similar to the cementing action in portland cement concrete. In sand, the aggregates or particles only become cemented at the points of contact between grains in typical soil-cement mixtures [47]. The cementing action will be at its greatest in a well-graded (many sizes) soil where there are minimal voids and numerous contact points and large contact areas. On the other hand, a uniformly graded (one size) sand requires a fairly high cement content to gain strength due to a minimum amount of contact area between grains.

In fine-grained silty and clayey soils, the cementing action bonds the mineral aggregates and soil particles to form a "floating aggregate matrix" that essentially encases the soil aggregates. The clay particles do little to enhance the strength; the matrix forms a honeycomb-type structure which becomes the strength element [47]. The effect cement has on the surface chemistry of the particles reduces their affinty to water. This, combined with the added strength from the matrix, prevents the soil from significant softening when exposed to moisture, thus increasing its durability. Additional strength may be achieved through a lime-soil reaction. Approximately 4% of the calcium oxide (which constitutes about 63% of the total chemical composition) in portland cement is free, i.e., it is not "tied up" with other chemicals. After hydration, this free lime plus calcium hydroxide formed during the hydration process react with the silicas and aluminas in the soil in a pozzolanic reaction. Therefore, with fine-grained soils, essentially two cementing actions may occur. Also, because of the free lime, some cation exchange and flocculation-agglomeration reactions occur although, in general, not to the extent as with lime-soil mixtures. This will cause some plasticity reduction but not to the degree that lime-soil mixtures do. One basic difference between lime and cement stabilization with soils is that the hydration process is more rapid than the pozzolanic reaction.

A very wide range of soils may benefit from cement stabilization. According to the Portland Cement Association, any soil may be stabilized with cement. The use of cement with sands, sandy and silty soils, and clayey soils of low to medium plasticity provides the best effectiveness and economy in airfield construction when compared to other stabilizers [44]. Should highly plastic soil (plasticity index greater than 30) be encountered, it will be most difficult to pulverize and mix the cement into the soil. In this case, the addition of lime first can reduce the plasticity so that the soil may be easily pulverized and yield a much more homogeneous mixture with the cement.

The presence of some finely divided organic matter in a soil may impair the hydration process and cause reduction in strength over what would normally be expected. Therefore, soils with high organic matter should be avoided. Also, sulfate attack can affect some soil-cement mixtures. Deterioration of fine-grained soil-cement mixtures has been noted due to sulfate-clay reactions. Coarse-grained soil-cement mixtures, however, do not seem to be susceptible to sulfate attack [44].

When discussing the engineering properties of soil-cement mixtures, normally they are divided into two groups of soil types: (1) coarse-grained or granular, cohesionless soils (USCS: G_ and S_); and (2) fine-grained, cohesive soils (USCS: C_ and M_).

The degree to which cement enhances a soil is contingent upon many factors: the nature of the soil, density obtained through compaction, water content, confining pressure, cement content, curing time and conditions, and the deleterious effects of past loadings and weathering on the soil. It is difficult to predict just how a soil will react to cement treatment because of these factors, many of which cannot be controlled. In achieving density through compaction, generally the cement will alter the maximum dry density and optimum water content of the soil but the direction of these changes is unpredictable. The flocculating action of the cement tends to cause similar changes as lime; a slight increase in optimum moisture content and a slight decrease in maximum dry density might be expected. On the other hand, the high specific gravity of unhydrated cement in relation to the soil tends to result in a slightly higher density. For example, a reduction of as much as 2% in optimum moisture has also been observed.

Compressive strength measured in pounds per square inch (psi) is the most widely used measure of the effectiveness of cement-treated soils. Depending on the cement content, this may range as low as 20 or 30 psi up to 2,000 psi with some granular soils. Normally, the high strength will be achieved with the coarse-grained, cohesionless soils. The cement content required to achieve a desired strength level varies from soil to soil. A linear relationship has been used to provide an estimate of compressive strength of a given soil based on the percent of cement used [44]. Figure 5 shows this relationship where UC is unconfined compressive strength in psi and C is cement content by percent of dry weight of soil. Table 3 provides the usual range of cement requirements for varying types of soils. In general, the finer the soil is, the more cement required. This is due to the increased surface area per unit volume of a fine soil compared to one that is more coarse. There are simply more particles to cement together. Table 3 may be used to estimate cement requirements for a given soil type.

A relationship has been developed betwen strength and curing time for a given soil-cement mixture [44]:

$$(uc)_d = (uc)_{d_o} + K \log \left(\frac{d}{do}\right) \qquad (1)^*$$

where

$(uc)_d$ = unconfined compressive strength at an age of d days, in psi

$(uc)_{d_o}$ = unconfined compressive strength at an age of d_o days, in psi

K = 70 C for granular soils and 10 C for fine-grained soils

C = cement content, in percent by weight of soil

For estimating purposes, it can be anticipated that the 28-day strength will be about 1.5 times the 7-day strength.

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*Refers to equation number.

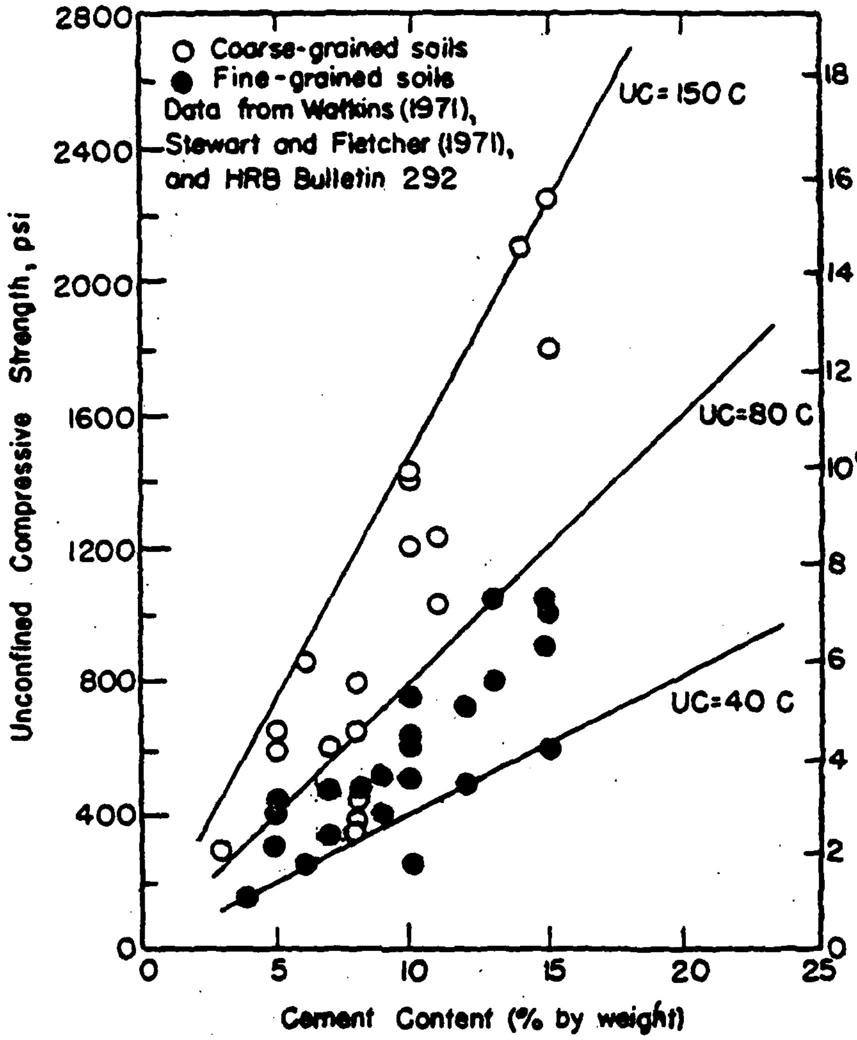


Figure 5. Relation Between Cement Content and Unconfined Compressive Strength for Soil and Cement Mixtures. (Equations give strength in psi.) [44]

Table 3. Cement Requirements for Various Soils [44].

AASHO Soil Classification	Unified Soil Classification*	Usual Range in Cement Requirement**		Estimated Cement Content and That Used in Moisture-Density Test, Percent by Weight	Cement Contents for Wet-Dry and Freeze-Thaw Tests, Percent by Weight
		Percent by Volume	Percent by Weight		
A-1-a	GW, GP, GM, SW, SP, SM	5 - 7	3 - 5	5	3 - 5 - 7
A-1-b	GM, GP, SM, SP	7 - 9	5 - 8	6	4 - 6 - 8
A-2	GM, GC, SM, SC	7 - 10	5 - 9	7	5 - 7 - 9
A-3	SP	8 - 12	7 - 11	9	7 - 9 - 11
A-4	CM, ML	8 - 12	7 - 12	10	8 - 10 - 12
A-5	ML, MH, CH	8 - 12	8 - 13	10	8 - 10 - 12
A-6	CL, CH	10 - 14	9 - 15	12	10 - 12 - 14
A-7	OH, MH, CH	10 - 14	10 - 16	13	11 - 13 - 15

*Based on correlation presented by Air Force.

**For most A horizon soils the cement should be increased four percentage points, if the soil is dark grey to grey, and six percentage points if the soil is black.

Since the strength criterion needed for the SELF is CBR, a conversion from unconfined compressive strength would be desirable. Figure 6 provides this relationship for coarse- and fine-grained soils. As discussed in the lime stabilization portion of this chapter, the meaning of CBR values greater than 100 is not known. However, as will be seen in Chapter III, CBR requirements for the SELF will be far below 100. Therefore, no further exploration into this area will be necessary.

As with lime-soil mixtures, perhaps the most important requirement of a soil-cement mixture is its ability to maintain its strength while exposed to the elements. Certainly strength is important, but most soil-cement mixtures that possess adequate resistance to wetting will also have adequate strength [44]. The converse of this, however, is not necessarily true.

Unlike the treatment of fine-grained, cohesive soils with lime, cement mixed with fine-grained soils does not normally produce the immediate strength increases. This may or may not be a problem in application to the SELF depending on the CBR value of the natural soil and how soon traffic will be expected after the cement treatment has been completed (function of mat laying time).

In summary, most any type of soil may benefit from cement stabilization. To gain the best effectiveness, sands, sandy and silty soils, and clayey soils of low to medium plasticity should be considered.

Asphalt Stabilization

Asphalt is one of two groups of bituminous materials, the other being tar. The primary source of asphalt in the U.S. is through the fractional distillation of petroleum crude oil. Asphalt is essentially

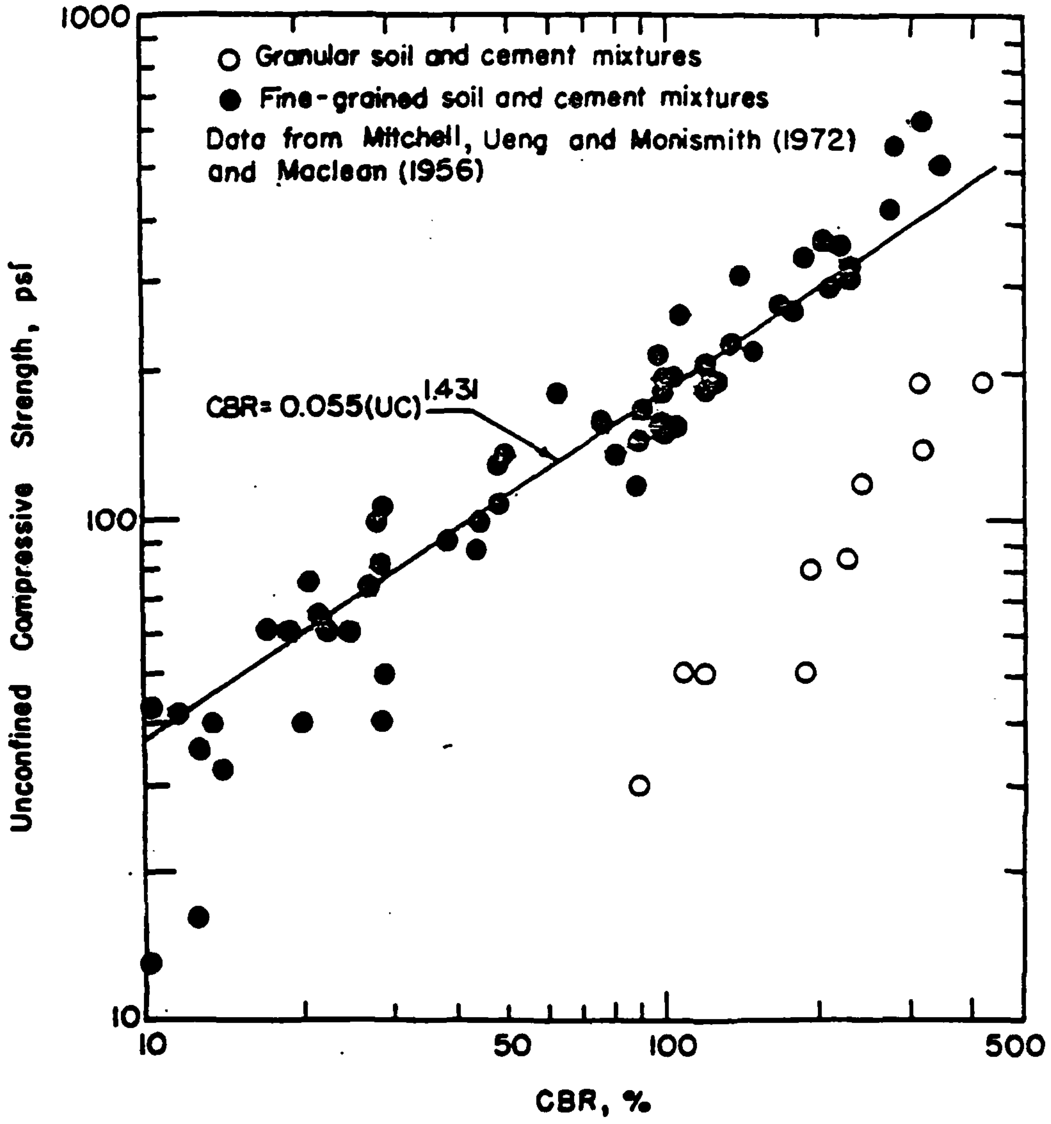


Figure 6. The Relation Between CBR and the Unconfined Compressive Strength of Soil and Cement Mixtures [44].

the residual material from the distillation process after gasoline, kerosene, diesel fuel, and lubricating oils are removed from the crude. Asphalt is also available from natural deposits such as rock asphalt.

Asphalt cement is the basic refined material. It is a hard, high-molecular weight material which is a semi-solid at ambient temperatures. Its initial high viscosity is reduced with temperature increases. It is graded on the basis of consistency and viscosity.

Liquid asphalt products are derived from asphalt cement. The most common liquid asphalts are cutbacks and emulsions. Cutback asphalts are formed by mixing asphalt cement with a nonvolatile oil and a solvent like gasoline or kerosene, depending on the rate of cure desired. The viscosities are low enough that these products can be mixed and sprayed at relatively low temperatures. They are graded based on curing time, nature of the residue and consistency. The curing time is controlled by the amount and type of solvent used: for a rapid cure (RC), gasoline or naptha is used; for a medium cure (MC) rate, a kerosene-type solvent is used; and for a slow cure (SC), a low volatile oil is employed. The greater the volatility of the solvent, the faster the cure due to higher evaporation rates. The RC grade has a harder residue than the MC which, in turn, has a harder residue than the SC. Typically, harder residues produce stronger compacted mixtures and are less susceptible to temperature changes. But, harder residues tend to be less flexible and cannot withstand deformations like softer residues can. These factors must be considered. Typical grade designations, which are based on kinematic viscosity at 140°F (60°C), are as follows:

Rapid Cure	Medium Cure	Slow Cure
--	MC-30	--
RC-70	MC-70	SC-70
RC-250	MC-250	SC-250
RC-800	MC-800	SC-800
RC-3000	MC-3000	SC-3000

The number provided is the lower range of the viscosity of the material in centistokes at 140°F (60°C). The upper viscosity limit is twice that of the lower limit. For example, an MC-800 has a viscosity range of 800-1600 centistokes at 140°F. As the viscosity increases, the resistance to flow will increase. The amount of solvent added controls the viscosity. For example, an RC-70 would have approximately 40% solvent and 60% asphalt cement: an RC-3000 would have approximately 15% solvent and 85% asphalt cement [59]. The viscosity controls the consistency, or workability, which is especially important at construction.

Emulsions are mixtures of asphalt cement, water and an emulsifying agent. Since asphalt cement will not dissolve in water, an emulsifier, which dissolves in the aqueous phase, suspends the asphalt cement globules in the water medium and prevents them from coalescing. Soaps are commonly used as an emulsifier. When an emulsion cures, it is said to break, i.e., the asphalt cement globules coalesce, causing the water to "squeeze out" and evaporate. Emulsions are manufactured with both positively (cationic) and negatively (anionic) charged emulsifying agents which control the net charge of the mixture. Since certain types of aggregate and soil particles have a greater affinity to water than asphalt, by selecting an emulsion with the opposite charg of the aggregate or soil, better coating of the particles is achieved through electrical bond. This is highly desirable because the lack of a strong bond between asphalt and aggregate or soil particles due to the presence of water can result in stripping, the loss of bond between the asphalt and soil or aggregate. Stripping will cause a loss of strength in the asphalt treated soil/aggregate mixture. Like cutbacks, emulsions are graded for varying curing or setting characteristics, the grade being controlled by the type and amount of emulsifying agent. Rapid setting (RS), medium setting (MS), and slow setting (SS) cationic and anionic emulsions are manufactured. The following grades are available:

Cationic	Anionic
CRS-1	RS-1
CRS-2	RS-2
CMS-2s	-
CMS-2	MS-2
CSS-1	SS-1
CSS-1h	SS-1h

The cationic type is distinguished from the anionic with a "C" before the grade designation.

The soil stabilization mechanisms with asphalt products are very much different than those with lime or cement. When lime and cement are employed, chemical interactions occur between the stabilizer and soil which actually alter physical properties of the soil. Asphalt stabilization does not work in this way.

In fine-grained soils, the stabilization occurs through a waterproofing effect. The soil particles or aggregates of particles are coated with an asphalt membrane that prevents or impairs the infiltration of water. This enhances stability and durability because the detrimental effects of water, which normally cause a decrease in shear and compressive strengths, are repelled by the asphalt film. Although there is no appreciable increase in strength with asphalt stabilization of fine-grained soils, the inherent strength of the soil through high cohesion at low water content can be maintained through the waterproofing phenomenon.

In coarse-grained, cohesionless soils, two basic mechanisms occur: waterproofing and adhesion. The waterproofing action is essentially the same as with fine-grained soils; the asphalt forms a film which prevents or hinders water infiltration and its deleterious effects. Adhesion occurs because the soil particles adhere to the asphalt, thus binding them in a cementing action. This increases shear and compressive strengths by increasing cohesion.

Nearly all soil types may benefit from soil stabilization; however, some types yield better results than others. Table 4 provides a general guideline for selecting the soils which are the most suitable for asphalt stabilization. The column headed "Soil-Bitumen" represents fine-grained soils. The other two columns are "Sand-Bitumen" (S_) and "Sand-Gravel Bitumen" (G_) which are the coarse-grained soils. Because of the great surface area in fine grain soils, it is virtually impossible to coat each and every particle with a film of asphalt. Enough asphalt is added, however, to substantially coat aggregations of particles to gain sufficient waterproofing benefit. Any cohesionless soils, other than the ones

Table 4. Engineering Properties of Materials Suitable for Bituminous Stabilization [44].

Percent Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel Bitumen
1 - 1 1/2"			100
1"	100		
3/4"			60-100
No. 4	50-100	50-100	35-100
No. 10	40-100		
No. 40		35-100	13-50
No. 100			8-35
No. 200	5-12	Good - 3-20 Fair - 0-3 & 20-30 Poor - >30	
Liquid Limit		Good - <20 Fair - 20-30 Poor - 30-40 Unusable - >40	
Plasticity Index	10	Good - <5 Fair - 5-9 Poor - 9-15 Unusable - >12-15	10

represented in this table, that are identified by ASTM, AASHTO or federal/state/local agencies as suitable for hot mix asphalt concrete, are generally acceptable for asphalt stabilization [44].

The two properties of asphalt stabilized soils that will be applicable to the SELF are stability and durability. Stability is essentially the strength of the mixture when used in the context of soil-bituminous mixtures. The most widely used tests for stability are the Hveem, Marshall and unconfined compression tests, but CBR can be used [44].

Combination Stabilizers

There are advantages to using lime, portland cement and asphalt products in combination with one another. Because of the great variability of the effectiveness of these stabilizers when used with different types of soils, the use of one stabilizer in combination with another may compensate for the lack of effectiveness of the other. For example, as previously discussed in the Cement Stabilization section of this chapter, lime may reduce the plasticity of a highly plastic soil, thus increasing workability so that cement may be more readily and properly mixed. In this case, the lime makes up for the cement's inability to facilitate pulverization and mixing problems due to high plasticity, yet the cement may provide the required strength with a soil that is nonreactive with lime.

Lime may also be used in concert with asphalt. It will act as an antistripping agent, thus increasing the effectiveness of asphalt stabilization. It can also enhance water resistance.

If an asphalt emulsion is employed, the addition of lime or cement may catalyze the curing rate which is a key factor. This may be especially useful in cool, damp weather where evaporation of the water medium may be retarded. Moisture resistance is also enhanced in emulsified asphalt-soil mixtures with a pretreatment of lime or cement [44].

Figure 7 can be used to assist in the selection of combination stabilizers. The amount of the primary stabilizer would be the same as if it were not used in combination. In a lime-cement combination, a 1 to 3% lime pretreatment can be expected, followed by a 3-10% cement content. In a lime-asphalt or cement-asphalt combination, 1-3% lime or cement pretreatment can be anticipated, followed by 4-7% asphalt. Lime in slurry form is best with emulsions; pulverized, dry lime is best with asphalt cement and cutbacks [44].

The use of combination stabilizers may be applicable to the SELF, depending on the exact soil and climatic conditions. Because of the added logistical burdens, its feasibility may be questioned, but from a purely technical aspect, combination stabilizers could be highly beneficial.

In the interest of maintaining the time-constrained scope of this project, no further discussions of combination stabilizers will be made.

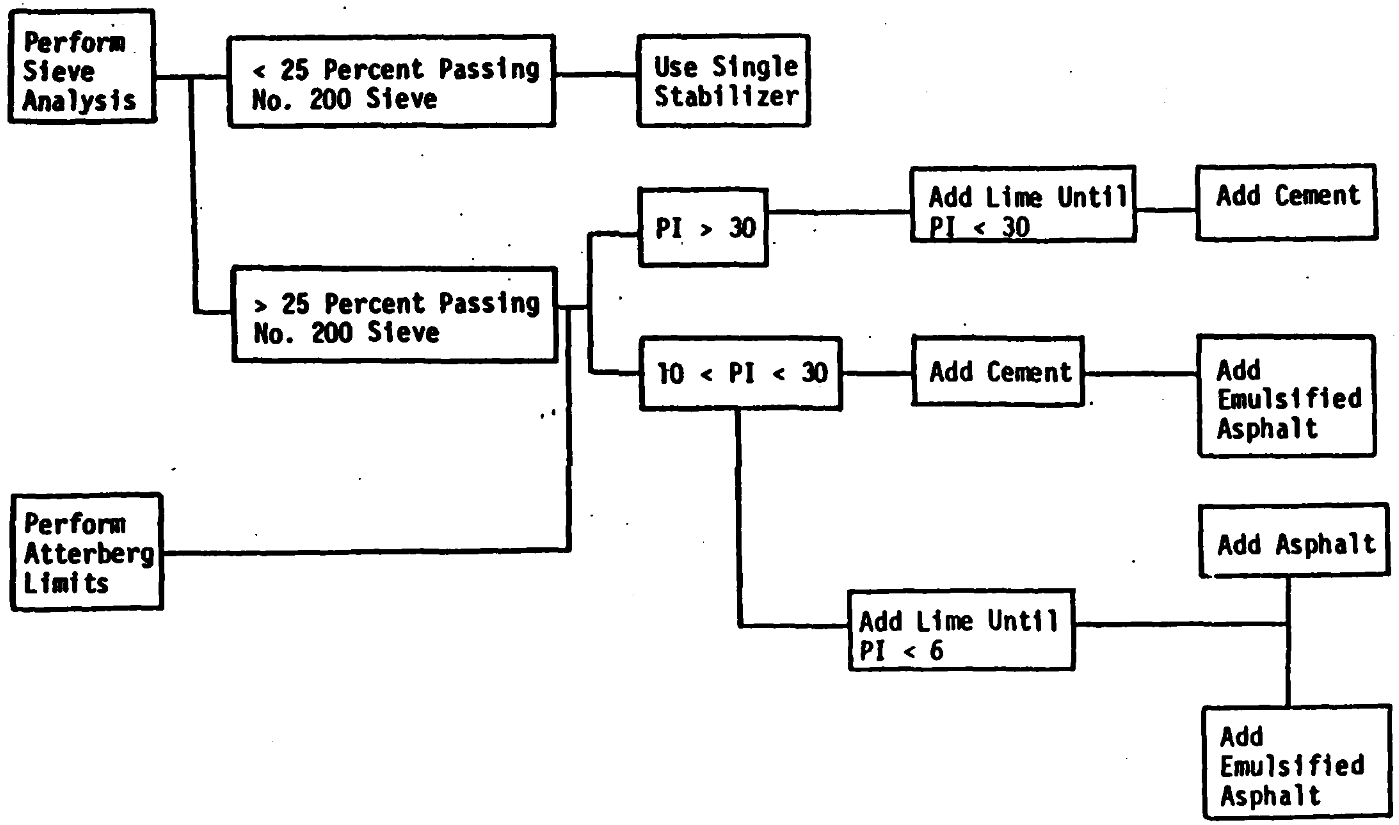


Figure 7. Selection of Combination Stabilizers [44].

CHAPTER III

THICKNESS OF THE STABILIZED-SOIL LAYER

Introduction

The thickness of any pavement system and the strength of its layers are dictated by the operational traffic parameters such as weight of the vehicles, tire pressures and configurations, and volume of traffic over the service life. Strength of the subgrade soil over which the pavement will be constructed is also an important factor. In design of the SELF, the strength of the stabilized-soil layer (referred to herein as base) will be controlled by the applied traffic independent of the strength of the soil beneath (referred to herein as subgrade). The thickness of the base will be such to prevent overstress of the subgrade which is a direct function of subgrade strength. The strength of the stabilized-soil layer should not be too great as this can cause cracking under load in the base induced by excessive radial stresses at the bottom of the base layer.

The emphasis of this paper is on the enhanced durability of a stabilized soil compared to an untreated material. No attempt will be made to specifically develop an exact thickness/strength design of the soil-mat pavement system. This would entail research and analysis which is beyond the scope of this paper. Research work that closely relates to stabilized-soil-mat systems, which has been extracted from literature will be presented. This will provide a general framework for the strength and thickness requirements.

U.S. Army Contingency Planning Method

The Army's Technical Manual TM5-330 [38], which is a planning and design guide for facilities in the theater of operations, provides some guidelines on strength and thickness of stabilized-soil-mat systems for airfields. The manual segregates its design data according to the type of airfield which is determined by its location and mission. The Rear Area Heavy Lift airfield and the Rear Area Tactical airfield closely relate to the SELF. These airfields are designed to handle the same or similar tactical and cargo aircraft as the SELF for a duration of six to twelve months. The manual does not outline the operational data such as loads, tire pressures, etc. on which the design figures were based. It does, however, state that operational traffic parameters were used in developing the designs for each airfield and its intended use.

The required soil strengths that are provided in the manual are in terms of airfield index (AI). This unit of strength measurement is taken from an expedient testing device called the airfield cone penetrometer which is used in the field when testing facilities are not available. A correlation has been developed between AI and CBR. Figure 8 provides the correlation.

Table 5 provides the design requirements for the aforementioned airfield. This includes typical aircraft which would use each airfield, the minimum strength of the base and the thickness of the base layer. No other ranges of subgrade AI were provided in the manual.

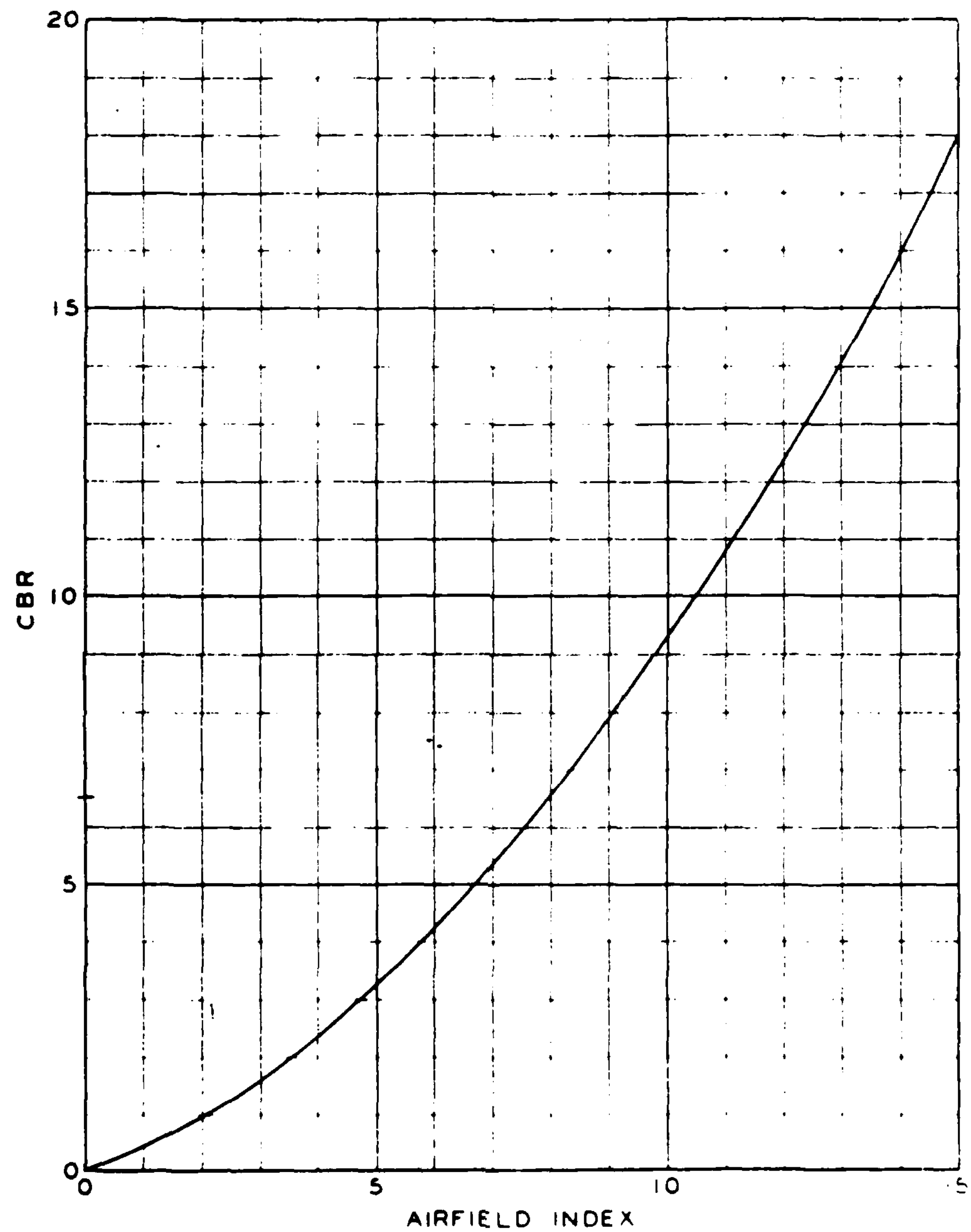


Figure 8. Correlation of CBR and Airfield Index.

Table 5. Army Airfield Design Requirements [38].

Type of Airfield	Typical Aircraft	Minimum Strength of Stabilized Base in Terms of		Thickness of Stabilized Base in Inches for Subgrade Strength of	
		AI	CBR	AI = 5 - 6 (CBR=3.2-4.2)	AI = 6 - 8 CBR=(4.2-6.5)
Rear Area Heavy Lift	C130 C141 C5	8	6.5	33	22
Rear Area Tactical	F4	8	6.5	7	5

It is obvious from this data that the cargo aircraft would be the controlling design aircraft type if the two airfields were combined.

The TM5-330 describes a thickness design procedure which takes into account the fact that subgrade soil strength may not be constant with depth. This could be used to tailor the thicknesses presented in Table 5 to fit the specific subgrade conditions as revealed by the airfield cone penetrometer. Unfortunately, the manual fails to provide the specific information for these two airfields which is necessary to use the design procedure. One must rely on the information in Table 5 as the design guideline when using this source.

U.S. Navy Design Method

A thickness design technique, which was employed by Brownie [12] and Howell [26], is based on the use of flexible pavement design curves from the Navy's Design Manual for Airfield Pavements, NAVFAC DM-21.3 [20]. For a given subgrade CBR strength, gross aircraft weight and the number of passes of the aircraft, a flexible pavement thickness is determined. This is then adjusted with a thickness reduction factor (TRF) to relate the flexible pavement system to the soil (unstabilized)-mat system.

The following example is provided to illustrate this design method. Assume that it is desired to design the thickness of the base under an AM2 matted runway with the following parameters: the subgrade strength is CBR 9; and the design aircraft is an F4 Phantom with a gross weight of 50 kips, tire pressures equal 400 psi and 300,000 aircraft passes are expected. A pass is the number of times the runway is traversed by the aircraft. Accordingly, a takeoff equals one pass and a landing equals one pass. The DM-21.3 would be consulted to establish the required total thickness of the flexible pavement system (surface layer of asphalt concrete, a base course layer and a subbase layer) which would support the given design parameters. Figure 9 is the DM-21.3 design curve for the F4 Phantom. The top of the figure is entered with a subgrade CBR of 9. Follow this value downward until the gross weight of 50 kips curve is intersected. Upon reaching the intersection, go horizontally to the right until the 300,000 aircraft passes curve is intersected, then turn downward and read the required total pavement thickness along the bottom of the graph. A value of approximately 18 inches should be read.

The next step is to determine the TRF which correlates the flexible pavement system to the soil-mat system. Figure 10 is used to establish the TRF. In order to use this graph, a single or equivalent-single-wheel load in kips must be known. This is quite simple with the F4 aircraft as it has only two wheels in its main landing gear and a single nose wheel. As will be seen later, establishing the equivalent-single-

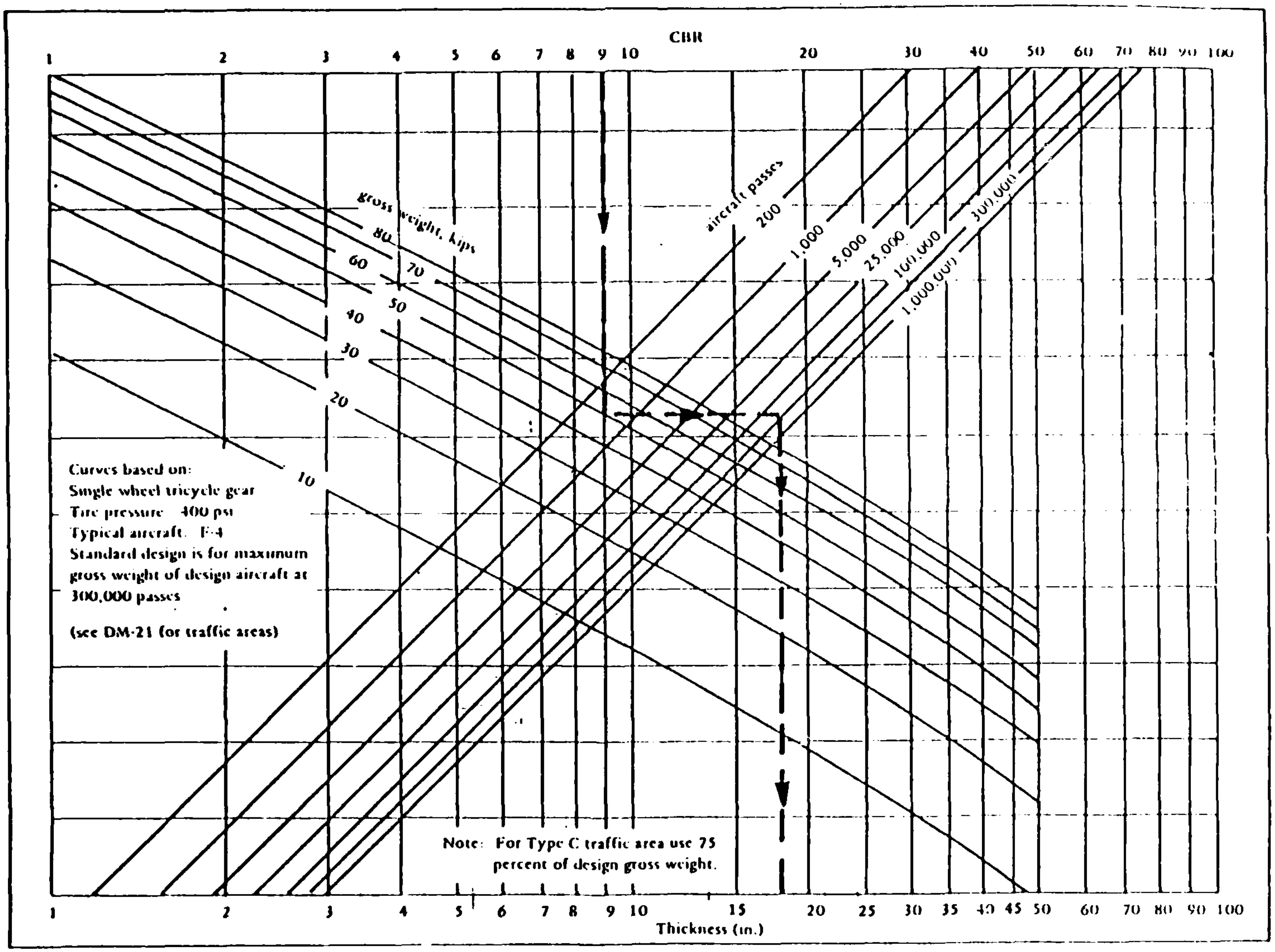


Figure 9. Flexible Pavement Design Curves Navy and Marine Corps Single-Wheel Aircraft, 400-psi Tire Pressure, Type B and C Traffic Areas.

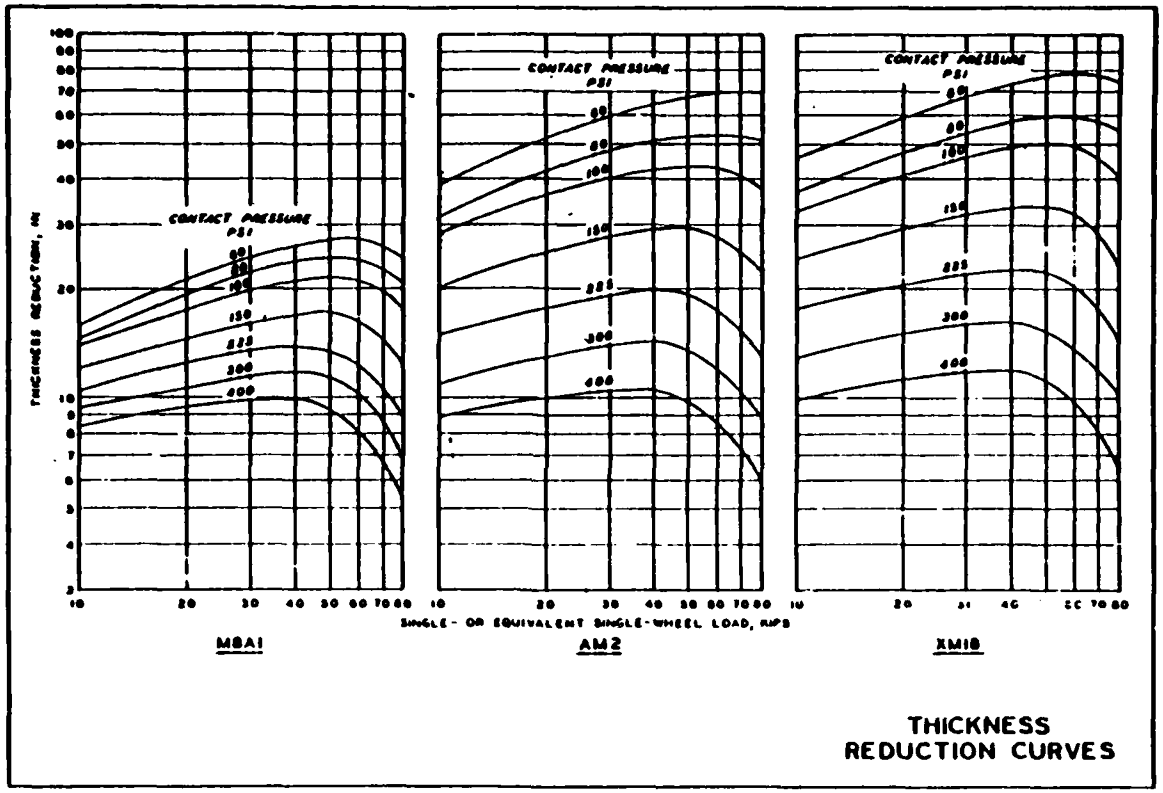


Figure 10. Thickness Reduction Curves. [58]

wheel load for an aircraft with multiple wheel gear configurations is a more complicated process. According to Reference [24], a maximum of 87.7% of the aircraft weight is on the main gear which means the remaining 12.3% of the weight is supported by the nose wheel. Since the main gear supports most of the weight, it is used to establish the single wheel load; 87.7% of 50 kips is 43.85 kips. This figure is divided by two (two wheels in main gear) to get a single wheel load of 21.9 kips. The bottom horizontal scale of the AM2 matting graph of Figure 10 is entered with a single wheel load of approximately 22 kips. Read upward until the 400 psi contact pressure is intersected (in pavement design, the tire pressure of the vehicle is assumed to be equal to the contact pressure on the pavement). Go to the left horizontally and read the TRF in inches along the vertical scale of the M8A1 matting graph. A value of 10 should be read.

Once the required flexible pavement thickness and the TRF are known, it is quite simple to complete the procedure. The required base thickness under the AM2 matting is the difference between the two values. In this example, the required base thickness is eight inches (18 inches of flexible pavement minus 10 inches of TRF = 8 inches of base).

From this method, a reasonable design thickness of the base layer under the matting may be established. This procedure, however, does not address the required strength of the base layer. Insofar as this paper is concerned, it will be assumed that a minimum strength of CBR 7 will be required. This is based on the data provided in the Army TM5-330 for stabilized-soil-mat pavements.

In most flexible pavement design methods, if stabilized base or subbase layers are used, the thickness may be reduced somewhat over what would be required if unstabilized materials were employed. For example, in the Navy DM-21.3, one inch of lime or cement-stabilized subbase may be substituted for 1.2 inches of unstabilized subbase materials; one inch of cement or asphalt-stabilized base may be substituted for 1.5 inches of unstabilized base course material. This reduction in thickness is due, in part, to the higher moduli of elasticity of stabilized pavement materials. The cementing actions discussed in Chapter II stiffen the materials, thereby increasing their structural capacity, reducing the subgrade stress and, consequently, reducing the thickness required (note: The strength should not be too high as tensile stresses which can cause cracking may develop as the material stiffens). Whether or not this principle may be applied to reducing the base thickness under the matting is not known. Without supporting research, it must be conservatively assumed that it cannot. Therefore, this thickness design procedure will be the same, regardless of the nature of the base layer (stabilized or unstabilized).

Table 6 provides a summary of required base thickness at varying subgrade strengths for the example aircraft (50 kips F4, 400 psi tire pressure and 300,000 expected passes). It can be clearly seen that the strength of the subgrade has a marked influence on the required base thickness. This should immediately alert the designers and planners of the SELF to select a site which has soil support conditions as good as possible as this will certainly impact construction time.

Table 6. Required Base Thickness for Varying Subgrade Strengths for an F4-Navy Method (50 kips, 400 psi Tire Pressure, 300,000 Passes).

Subgrade CBR	Flexible Pavement Thickness (in)	TRF (in)	Required Base Thickness (in)
2	38	10	28
3	32	10	22
4	28	10	18
6	23	10	13
7	20	10	10
9	18	10	8
10	17	10	7

This design method will now be applied to the design of the SELF. The same procedure will be followed for each MAG aircraft group as was provided in the previous example. A subgrade CBR of 4 will be assumed. The maximum gross aircraft loads, tire pressures and gear configurations were taken from References [20, 24, 26, 38]. As stated previously, to determine the equivalent-single-wheel load for aircrafts whose main gear are composed of multiple wheels in various configurations is not a simple process. There are many accepted methods to do this. A table has been provided in Reference [24] which outlines this information.

The key element to this design procedure is estimating the expected number of aircraft passes for each category of aircraft. Since the SELF should be designed for a service life of one year, 365 days will be used as the basis for the design. It is estimated that the fighter and attack aircraft (F4/F18 and A6/AV4/AV8) will conduct three sorties each in a 24-hour day. It is estimated that the tanker aircraft (KC-130) will conduct two sorties each per day. The cargo aircraft (C5/C141/ DC 8 or 10), since they are transient, are estimated to arrive at a rate of one per day. These estimates are arbitrary and can be adjusted should more accurate estimates become available. With these estimates, the total number of aircraft passes within each group may be estimated. The calculations for the F4/F18 group are provided as an example: 36 fighters in the MAG x 3 sorties per day each x 365 days x 2 passes per sortie = 78,840 passes rounded up to 80,000 per year to better fit the design curves. Similar calculations are conducted for each group.

Table 7 provides a summary of the thickness design data and calculations for the MAG aircraft. Since the cargo aircraft are only expected to arrive at a rate of one per day, the C5 and C141 split the time in Table 7 (183 days for the C5 and 182 days for the C141).

In evaluating the required base thickness for the various aircraft types in Table 7, it can be clearly seen that the F4/F18 aircraft is the critical aircraft, around which the design should be centered. This is due to the extremely high tire pressures (400 psi) which cause very high stresses in the upper part of the pavement. Therefore, the base thickness must be increased to preclude excessive stresses in the subgrade which could cause a pavement failure.

Ordinarily, in flexible pavement design, once the critical or design vehicle or aircraft is known, the effects of the mixed traffic (different vehicle types) are considered. The other aircraft are evaluated in terms of the design aircraft, i.e., an equivalent-wheel-load factor is

Table 7. Summary of Thickness Design Data and Calculations for the SELF-Navy Method (Subgrade = CBR 4).

Aircraft Type (# in MAG)	Max. Gross Aircraft Wt. (Kips)	Gear Configu-ration	Total # Wheels in Main Gear	# of Wheels in One Gear Main Gear Cluster	[a]Equivalent Single-Wheel Load (Kips)	Max. Contact Pressure (psi)	Total # of Passes	Flexible Pavement Thickness (inches)	Thickness Reduction Factor (inches)	Required Base Thickness (inches)
F4/F18 [36]	80	Single Tricycle	2	1	27.0	400	80,000	31	11	20
A6/AV4/ AV8 [52]	80	Single Tricycle	2	1	27.0	150	120,000	34	27	7
KC-130 [8]	200	Single Tandem Tricycle	4	2	44.4	115	20,000	38	38	0
C5	850	Dual Delta Tandem	24	6	36.1	115	500	40	38	2
C141	325	Dual Tandem Tricycle	8	4	52.4	180	500	34	24	10

[a]Specifically designated for landing mat surfaces [24].

used to correlate the damage caused per pass of an aircraft to a given pavement system relative to the damage caused by the design or standard aircraft to the same pavement system [64]. There are several established methods which can be used to accomplish this correlation for flexible pavements. None of them, however, could be adapted to the soil-mat system because of the effects of the thickness reduction factor. Therefore, a very conservative estimate of 200,000 passes for the F4/F18 aircraft will be used. This includes the 80,000 estimated passes for the design aircraft and an additional 120,000 passes to account for the damage caused by the other aircraft. This conservative estimate will also make up for any deficiencies in the initial arbitrary number of expected passes for each aircraft group.

Using Fig. 9 again for an 80-kip F4 Phantom with 200,000 passes on a CBR 4 subgrade will yield a flexible pavement thickness of 33 inches. With the 11-inch thickness reduction factor from Table 7, a net 22 inches of base is required. One must bear in mind that as the subgrade strength varies, so too does the required base thickness. This is exhibited in Table 8.

Table 8. Required Base Thickness for Varying Subgrade Strengths for an F4-Navy Method (80 kips, 400 psi Tire Pressure, 200,000 Passes).

Subgrade CBR	Flexible Pavement Thickness (in)	TRF (in)	Required Base Thickness
2	47	11	36
3	39	11	28
4	33	11	22
6	27	11	16
7	25	11	14
9	23	11	12
10	22	11	11

U.S. Army Corps of Engineers Method

In 1971, a study was conducted at the U.S. Army Corps of Engineers Waterway Experiment Station to develop a method for determining thickness requirements for landing mat-surfaced airfields [58]. The AM2 matting was tested, along with several other types, to investigate the effects of load, tire pressure and soil strength on its performance. The AM2 matting test section consisted of a CBR 3.7 clay soil subgrade; the same material was used as the base but at CBR 5. The subgrade and base for the other types of matting were of the same clay material as the AM2 section and their strengths ranged from CBR 1.3 to 3.7 and CBR 3.0 to 8.0, respectively. All test data was combined and graphed. A conservative line through the data points was established which yielded the following equation:

$$t_{um} = (0.2875 \log C + 0.1875) \sqrt{\frac{P}{8.1\ CBR} - \frac{A}{\pi}} - TR \qquad (2)$$

where

t_{um} = total thickness of strengthening soil under the mat, inches (in.)

C = number of aircraft coverages (to be defined)

P = single or equivalent-single-wheel load, pounds (lb.)

A = tire contact area, square inches (sq. in.)

TR = TRF = mat thickness reduction factor from Fig. 10

CBR = CBR of subgrade soil

Since this equation was derived from all data including matting systems other than the AM2, it is assumed that the equation is valid for the entire range of CBR values for the AM2 matting (subgrade: 1.3-3.7 and base: 3.0-8.0).

In order to use the equation, two parameters not previously introduced must be employed: tire contact area and coverages. Tire contact areas for the MAG aircraft were extracted from the TM5-330 [38]. A coverage is defined as a significant number of aircraft passes in adjacent longitudinal tire paths to cover a given width of pavement one time [24]. Since an aircraft does not travel over the exact same section of the pavement, especially when landing, it takes more than one takeoff or landing to travel over each longitudinal path of the pavement within a prescribed width. The aircraft traffic on the SELF is assumed to be channelized, i.e., most of the traffic on the runway and taxiway are very close to the centerline and do not usually wander outside of an estimated width of pavement (37.5 feet for runways and 7.5 feet for taxiways according to the Corps of Engineers [64]). Tables in Reference [24] provide coverage figures for MAG aircraft in terms of cycles per coverage where a cycle is defined as one takeoff and one landing.

Table 9 provides the design parameters and data for the MAG aircraft groups. A subgrade strength of CBR 4 is assumed. Equation (2) is used for the thickness design. The required thicknesses of base (t_{um}) as shown in Table 9 are relatively close to those summarized in Table 7 (Navy Method) for the F4/F18, A6/AV4/AV8 and C141. Where the Navy Method yielded a 0 base thickness for the C130, the Corps of Engineers method requires a 6-inch base. The Navy Method yielded a 2-inch base for the C5 and the Corps Method yielded a -11 (negative value means no base is required). Again, the F4/F18 is the critical design aircraft. Taking into account the mixed traffic and estimated 200,000 passes of the F4 as was done in the Navy Method, a revised base thickness of 28 inches is required.

Table 9. Summary of Thickness Design Data and Calculations for MAG Aircraft--Corps of Engineers Method.

Aircraft Type (# in MAG)	A Contact Area (sq. in.)	P Single or Single-Equivalent-Wheel Load (lb.)	# of Passes	# of Passes Per Coverage	C Coverages	TR Thickness Reduction Factor (in.)	t_{um} Thickness of Base (in.)
F4/F18 [36]	102	27,000	80,000	14.72	5,435	11	25
A6/AV4/AV8 [52]	100	27,000	120,000	14.72	8,152	27	10
KC-130 [8]	440	44,400	20,000	4.24	4,717	38	6
C5	290	36,100	500	2.70	185	38	-11
C141	208	52,400	500	4.50	111	24	7

Table 10 summarizes the required base thicknesses for the design aircraft using the Corps of Engineers methods.

Table 10. Required Base Thickness for Varying Subgrade Strengths for an F4--Corps of Engineers Method.

Subgrade CBR	Required Base Thickness (in.)
2	45
3	34
4	28
6	20
7	18
9	14
10	13

Summary

This chapter has presented three methods used by the military to establish the required base thickness under AM2 landing mat airfields. The Army Contingency Planning Manual, the TM5-330, does not appear applicable to the SELF. The guidelines that it provides are essentially opposite of the calculated values of the other two methods. It required a relatively thick base layer for the cargo airfield and a thin layer for the tactical field. Since the manual did not provide the specific traffic parameters that were used in developing the thickness design, it is difficult to determine the reasons for the disparity. More than likely, though, the Rear Area Heavy Lift airfield was designed for an extremely high volume of heavy traffic to warrant the thick base layer. This is not surprising considering that it is the primary logistic link to a very large combat force. On the other hand, the relatively thin base layer under the

Rear Area Tactical field is apparently designed for low volumes of traffic and much lower tire pressures than for the MAG aircraft.

Both the Navy and Army Corps of Engineers methods seem applicable to design of the SELF. The computed results are consistent with one another and what would be reasonably expected given the aircraft loads, tire pressures and traffic volume. Either method could be used with confidence by SELF planners to determine the thickness design of the base layer.

There is one void, however, in these methods. Neither of them specifically address the required strength of the base layer. The Corps of Engineers method does give the range of values used (CBR 3 to 8) during the testing. These are consistent with the minimum base strength specified in the TM5-330 (CBR about 7). The Navy method does not address base strength at all. It can be concluded that a base strength of 3 to 8 over a subgrade of 2 to 3 would be adequate. It is quite likely that the strength of the base will be much higher because of the amount of stabilizer that must be added to gain durability. Chapter II pointed out the increased CBR values that resulted from stabilization.

CHAPTER IV

CONSTRUCTION OF STABILIZED LAYER

Mix Design

Before construction procedures can be discussed and construction time can be estimated, there must be a brief discussion on mix design, i.e., the amount of stabilizer to add to the soil to achieve the design objectives. Laboratory testing and analysis of stabilized-soil mixtures, using soil from the proposed site, will yield an exact design to meet strength and/or durability requirements. Ideally, this would provide the SELF planners with the optimum design so that logistical burdens of transporting the stabilizing agent(s) would be minimized. In an actual contingency situation, however, the SELF planners may not be able to obtain soil samples or have the time and resources to conduct the laboratory tests that would be necessary to develop the actual design. Therefore, this paper will not delve into the many laboratory testing procedures that would be used, but present information which will provide guidelines to select the amount of stabilizer to be used. References [44] and [45] provide excellent discussions of laboratory testing procedures for the various types of stabilizers and lists of ASTM/AASHTO testing specifications that are applicable to each.

Table 11 outlines the normal range of quantities of lime, cement, and asphalt stabilizing agents that would be used with various types of soil. Notice that as the soil becomes finer (moves from gravels

Table 11. Summary of Soil Stabilizers for Strength Improvement Function [38].

(1) Material	(2) Form of Material	(3) Applicable Soil Range	(4) Estimated Range of Quantity Requirements (%)†	(5) Minimum Curing Time Requirements
Portland cement	Powder	Gravels	3-4	24 hours
		Sands	3-5	
		Silts-clayey silts	4-6	
		Clays	6-8	
Lime				
1. Hydrated	Powder	Clayey gravels	2-4	7 days
		Silty clays	5-10	
		Clays	3-8	
2. Quicklime	Powder	Clayey gravels	2-3	4 hours
		Silty clays	3-8	
		Clays	3-6	
Bituminous material				
1. Asphaltic cutbacks				
a. RC-70 to RC-800	Liquid	Sands	5-7††	1-3 days
		Silty sands	6-10	
		Clayey sands	6-10	
b. MC-70 to MC-800	Liquid	Sands	5-7	3-5 days
		Silty sands	6-10	
		Clayey sands	6-10	
2. Asphaltic emulsions	Liquid	Sands	5-7	1-3 days
		Silty sands	6-10	
		Clayey sands	6-10	

† Based on dry density of existing soil.
†† All quantities listed for asphalts are actual bitumen requirements, exclusive of volatiles.

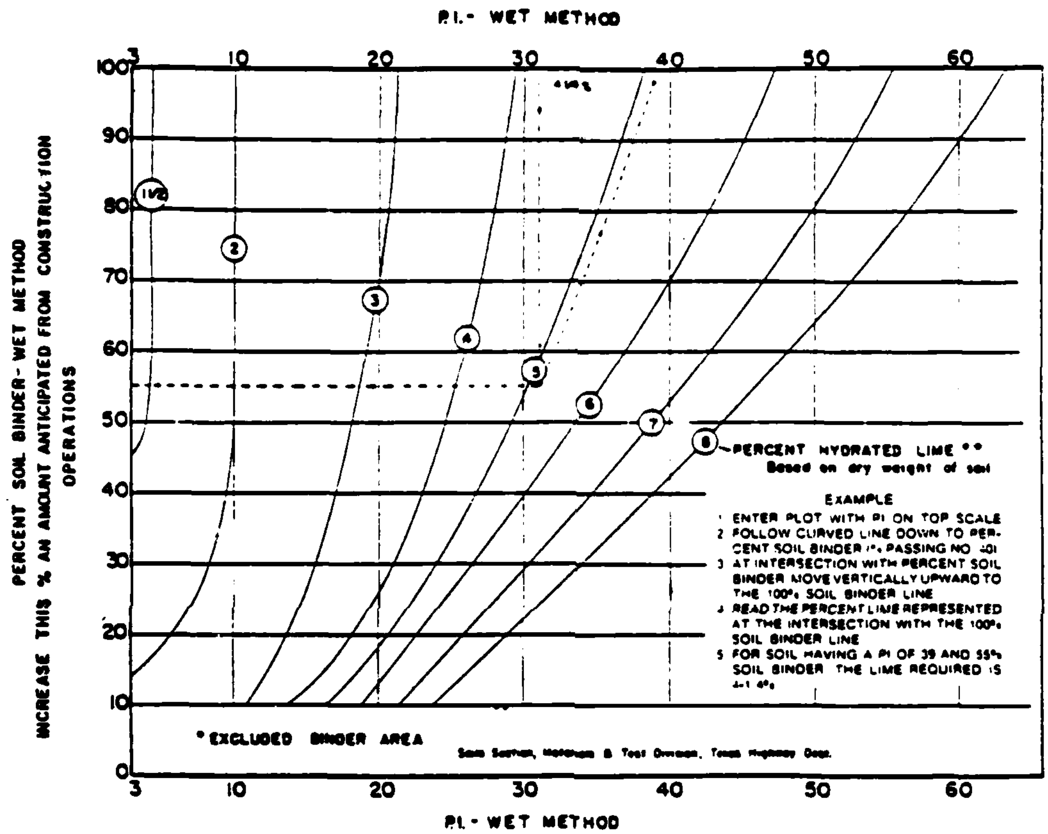
and sands to silts and clays), greater quantities of stabilizers are required. As discussed in Chapter II, this is due to the larger surface area of the finer soils which necessitates more stabilizer to coat or cement the many particles or aggregations of particles.

The cure time that is suggested is an important factor in developing the required initial strength and stability of the mixture before traffic is applied. Since the AM2 matting must be laid on the base layer once stabilization, compaction and finished grading are completed, the stabilized soil will have adequate time for initial cure due to significant time required to lay over 2.6 million square feet of matting that are required.

Since Table 11 is a general summary of the quantities of stabilizers that would be required, more detailed procedures will be presented that, when used, will yield much more accurate estimates of stabilizer requirements.

Lime. Figure 11 provides a chart whereby the percent of lime required to stabilize fine-grained soils may be estimated. The figure is entered with the plasticity index (PI) of the soil and the percent soil binder (percent of the soil passing the number 40 sieve). This procedure will yield the percent of hydrated lime required based on dry weight of the soil. An example is provided in the lower right-hand corner of the figure.

Cement. Table 3, which is found in Chapter II, provides a detailed guideline to select the quantity of cement required for soils that have been classified under the USCS system. To better estimate the amount of cement required, Tables 12 and 13 may be used for coarse-grained and fine-grained soils respectively. To use Table 12, the percent of soil retained on the number 4 sieve and the percent of soil smaller than



* Exclude use of chart for materials with less than 10% - No. 40 and cohesionless materials (PI less than 3)

** Percent of relatively pure lime usually 90% or more of Ca and/or Mg hydroxides and 85% or more of which pass the No 200 sieve. Percentages shown are for stabilizing subgrades and base courses where lasting effects are desired. Satisfactory temporary results are sometimes obtained by the use of as little as 1/2 of above percentages. Reference to cementing strength is implied when such terms as "Lasting Effects" and "Temporary Results" are used.

Figure 11. Selection of Lime Stabilizer Quantity [20].

Table 12. Average Cement Requirements for Granular and Sandy Soils [20].

Material Retained on No. 4 Sieve percent	Material Smaller Than 0.05 mm percent	Cement Content, Percent by Weight Maximum Dry Density, lb/cu ft (Treated Material) 116-120	121-126	127-131	132-137	138-142	143 or more
0-14	0-19	10	9	8	7	6	5
	20-39	9	8	7	7	5	5
	40-50	11	10	9	8	6	5
15-29	0-19	10	9	8	6	5	5
	20-39	9	8	7	6	6	5
	40-50	12	10	9	8	7	6
30-45	0-19	10	8	7	6	5	5
	20-39	11	9	8	7	6	5
	40-50	12	11	10	9	8	6

Note: Base course goes to 70 percent retained on the No. 4 sieve.

Table 13. Average Cement Requirements for Silty and Clayey Soils [20].

Group Index[a]	Material Between 0.05 and 0.005 mm percent	Cement Content, Percent by Weight Maximum Dry Density, lb/cu ft (Treated Material) 99-104	105-109	110-115	116-120	121-126	127-131	132 or more
0-3	0-19	12	11	10	8	8	7	7
	20-39	12	11	10	9	8	8	7
	40-59	13	12	11	9	9	8	8
	60 or more	--	--	--	--	--	--	--
3-7	0-19	13	12	11	9	8	7	7
	20-39	13	12	11	10	9	8	8
	40-59	14	13	12	10	10	9	8
	60 or more	15	14	12	11	10	9	9
7-11	0-19	14	13	11	10	9	8	8
	20-39	15	14	11	10	9	9	9
	40-59	16	14	12	11	10	10	9
	60 or more	17	15	13	11	10	10	10
11-15	0-19	15	14	13	12	11	9	9
	20-39	16	15	13	12	11	10	10
	40-59	17	16	14	12	12	11	10
	60 or more	18	16	14	13	12	11	11
15-20	0-19	17	16	14	13	12	11	10
	20-39	18	17	15	14	13	11	11
	40-59	19	18	15	14	14	12	12
	60 or more	20	19	16	15	14	13	12

[a] Taken from figure 12.

0.05 millimeters (mm) must be known. To use Table 13, the Group Index and the percent of soil between the 0.05 and the 0.005 mm sizes must be known. The Group Index may be found from Fig. 12 by knowing the liquid limit (LL), PI and percent passing the number 200 sieve of the soil in question.

Asphalt. The Asphalt Institute has adopted methods for determining the percent asphalt in cutbacks and emulsions that are required for stabilization purposes [44, 45]. These are presented in the following equations:

Cutbacks:

$$p = 0.02\,(a) + 0.07\,(b) + 0.15\,(c) + 0.20\,(d) \tag{3}$$

where,

p = percent of residual asphalt by weight of dry aggregate or soil

a = percent of soil or aggregate retained on the No. 50 sieve

b = percent of soil or aggregate passing the No. 50 sieve and retained on the No. 100 sieve

c = percent of soil or aggregate passing the No. 100 sieve and retained on the No. 200 sieve

d = percent of soil or aggregate passing the No. 200 sieve

Emulsions:

$$p = 0.05\,(a) + 0.1\,(b) + 0.5\,(c) \tag{4}$$

where,

p = percent by weight of asphalt emulsion based on dry weight of soil or aggregate

a = percent of aggregate or soil retained on the No. 8 sieve

b = percent of aggregate or soil passing the No. 8 sieve and retained on the No. 200 sieve

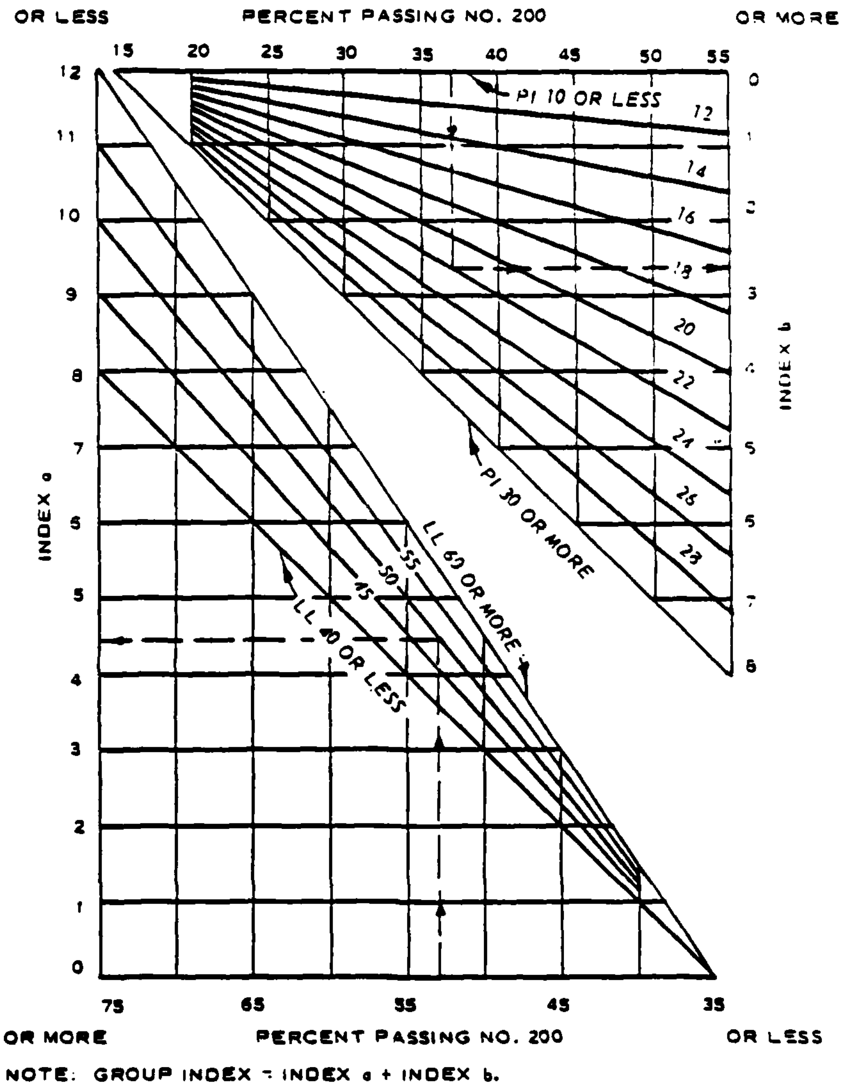


Figure 12. AASHTO Group Index for Silty and Clayey Soils [42].

c = percent of soil or aggregate passing the No. 200 sieve

In addition, Table 14 may be used to estimate the required amount of asphalt emulsion.

Adjustment to the Mix Design. Once the mix design has been developed, it is common design practice to add some percent additional stabilizer to account for the variability of the construction process. The percentages of stabilizer recommended in these tables and figures are based on laboratory designs where variables such as proper soil pulverization, adequate mixing and blending of soil and stabilizer, or the moisture content of the soil can be closely controlled. It is much more difficult to control quality of the mixture during construction. To offset this, some adjustment to the mix design should be made. An additional 2% of stabilizer is a good working value. So, if the mix design yielded 6% lime, for example, 8% should be specified.

Quantity of Stabilizer for the SELF

Once the mix design has been established, along with the required thickness of base, SELF planners, for logistical purposes, may want to estimate the total quantity of stabilizer needed. The TM5-330 [38] provides the following equations to facilitate this estimate:

a. For cement or lime:

$$Q = A \times \frac{T}{12} \times D \times \frac{S}{100} \qquad (5)$$

where,

Q = total quantity of stabilizer, lbs.

A = area to be treated, sq. ft.

Table 14. Emulsified Asphalt Requirements [44].

Percent Passing No. 200 Sieve	Pounds of Emulsified Asphalt per 100 Pounds of Dry Aggregate at Percent Passing No. 10 Sieve					
	≤ 50	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

T = required thickness of stabilized layer, in.

D = dry density of soil, lbs. per cu. ft. (cubic foot)

S = percent treatment specified

b. For liquid asphalts (cutbacks and emulsions):

$$Q = A \times \frac{T}{12} \times D \times \frac{S}{100} \times \frac{100}{R} \tag{6}$$

where,

R = percent residual asphalt cement in cutback or emulsion form

All other values are the same.

Table 15 provides the approximate percent residual asphalt cement for several selected liquid asphalts commonly used in soil stabilization [38]:

Table 15. Typical Percent Residual Asphalt Cement in Selected Liquid Asphalts.

Type and Grade	R(%)
MC-70	50
MC-250	60
MC-800	70
RC-70	55
RC-250	65
RC-800	75
SS-1	60

To illustrate the use of these equations, it is assumed that it is desired to estimate the amount of lime and cutback asphalt (MC-70) that would be required for a 6% adjusted stabilization treatment of a 16-inch base of the SELF's runway (8000 ft. x 150 ft.) for a soil whose dry density was 100 lbs. per cu. ft. Using Eq. (5) for the lime:

$$Q = (8000 \text{ ft.} \times 150 \text{ ft.}) \times (\frac{16}{12}) \times (100) \times (\frac{6}{100})$$

$$Q = 9{,}600{,}000 \text{ lbs. or } 4{,}800 \text{ tons of lime}$$

Using Eq. (6) for the cutback asphalt:

$$Q = (8000 \text{ ft.} \times 150 \text{ ft.}) \times (\frac{16}{12}) \times (100) \times (\frac{6}{100}) \times (\frac{100}{50})$$

$$Q = 19{,}600{,}000 \text{ lbs. or } 9{,}800 \text{ tons of cutback asphalt}$$

Stabilization Equipment

In a Marine amphibious operation, the MAF has Engineer Battalions which have the capability to accomplish horizontal construction projects. Because of the enormous construction effort that is required to construct a SELF, it would be necessary to employ units from the Naval Construction Force (NCF) or Seabees in the form of Naval Mobile Construction Battalions (NMCBs). These 750-man units, with their normal table of allowance (TOA) and augment heavy construction equipment could accomplish construction of a SELF, including chemical soil stabilization of the base layer. It is quite possible that the MAF Engineer Battalions would be required closer to the advancing combat forces and would not be available to assist in construction of the SELF.

It is assumed in the context of this paper that the assets of three NMCBs will be available and committed to the SELF construction with no MAF Engineer Battalion support.

Table 16 summarizes the equipment that would be available to each NMCB, either from the regular TOA (referred to herein as P-25) or from the NCF Support Unit of augment equipment (referred to herein as P-31). Only the equipment that relates directly to the stabilization process

Table 16. NCF Stabilization Equipment.

Equipment	No. in P-25	No. in P-31	Total Number
Mixer/pulverizer	0	2	6
Lime/cement spreader	0	2	6
Semi-trailer, bulk cement/lime	0	1	3
Water distributor	4	6	30
Asphalt distributor	1	4	15
Rubber-Tired compactor	2	0	6
Sheepsfoot compactor	0	6	18
Smooth drum, vibratory compactor	3	7	30

is listed. The last column is the total number of pieces of equipment for the three NMCBs; the other columns reflect the assets of one NMCB.

The mixer/pulverizer will govern the production of the stabilization process. The production of the stabilizer spreader/distributor equipment is such that this operation will proceed just ahead of the mixing operation so that the stabilizing agent is not subjected to prolonged climatic exposure before it is mixed into the soil.

There are two models of mixer/pulverizer equipment used by the NCF: the Pettibone Wood Model 750C "Speedmixer"; and the Buffalo-Springfield Model 733 "Soil Stabilizer." The Pettibone Wood unit is limited to a working depth of 16 inches or less. This immediately restricts the base layer thickness to 16 inches. Although a thicker layer could be constructed in two or more lifts, it would entail a prohibitive amount of construction time and is, therefore, not practically applicable to the SELF concept. The working depth of the Buffalo-Springfield unit was not available. Even if it could mix at a greater depth, there is no guarantee that this model, vice the Pettibone Wood model would be assigned to the construction units.

Construction Procedures

The objective in constructing a stabilized-soil layer is to obtain a thorough, uniform mixture of the correct quantity of stabilizer and soil with the correct moisture content to facilitate mixing and compaction. The procedures that are used to construct a stabilized-soil layer with different types of stabilizing agents are quite similar, although there are some differences. This section will outline the general construction procedures that would be followed in constructing the base layer of the SELF with the NCF equipment resources. Where there are significant differences between stabilizing agents at any phase of construction, they will be mentioned.

It is assumed in the context of this paper that only mixed-in-place construction will be used, i.e., the stabilizer is laid directly on the soil to be treated and a mixing machine churns the soil and stabilizer as it travels by. This is the normal technique that is used for subgrade stabilization. Central plant processing can be used to mix a soil and stabilizer which is then hauled to the site, dumped and spread, but this is a time consuming endeavor that requires additional equipment and a borrow source of soil. No real benefit would be achieved by this process in application to the SELF.

There are five basic phases of construction of a stabilized-soil: soil preparation, addition of stabilizer, mixing and blending, compaction, and finish grading and curing. These will be discussed separately in the following subsections.

Soil Preparation. After all clearing, cut and fill work, and rough grading to the proper geometric specifications have been completed, the stabilization process may begin. If the soil is in a hardened condition, it must first be loosened or scarified. Graders with scarification teeth and bulldozers with ripper blades can accomplish this efficiently. Any large debris such as boulders or tree stumps should be removed.

In order to gain an intimate stabilized-soil mixture, the soil must be properly pulverized. Most soils, especially fine-grained soils, will require pulverization, and multiple passes of the pulverization equipment may be required. A good check to see if the soil is sufficiently pulverized is to run a sieve analysis. Approximately 60 to 80% of the soil should pass the No. 4 sieve. If the soil is extremely wet, it may require some aeration before beginning the pulverization process. If the soil is quite dry, the addition of some water with the water distributors will aid the pulverization.

As discussed in Chapter II, if the soil is highly plastic, a lime pretreatment, regardless of the primary stabilizing agent that has been chosen, may have to be performed first to reduce the PI, so that pulverization of the soil may be obtained. Without this, pulverization of the soil may be impossible.

Except for the highly plastic soils, pulverization may be accomplished relatively easily. The coarse-grained gravels and sands may require some pulverization but possibly none at all. In this case, the pulverization may be done as the stabilizer is mixed into the soil.

Once the soil has been processed to the proper pulverized state, the soil must be brought to the correct moisture content to facilitate uniform mixing with the stabilizer, but more importantly compaction of the stabilized-soil mixture. In order to do this, laboratory compaction tests must be performed to determine the optimum moisture content which coincides with maximum dry density of the soil. ASTM D-698 or D-1557 or AASHTO T-99 specifications are commonly used guidelines in conducting these tests. The moisture content of the soil should be slightly dry or wet of optimum depending upon the type of soil and compaction equipment to be employed. This procedure will be disucssed in more detail in the compaction subsection.

Addition of Stabilizer. The distribution of lime or cement will be accomplished in the dry, powdered form by the six spreaders. These will be fed directly from the bulk semi-trailers as the spreading proceeds. Since there are only three of these bulk trailers, three spreaders will be fed by dump trucks which will haul the cement or lime from a central point.

It is assumed that only bulk distribtuion will be used. The stabilizer could be distributed by standard 94-lb. bags of cement or 50-lb. bags of lime which are spread by hand. This would not be feasible on such a large project as the SELF. In addition, the spreader machines have a much better control over the spread rate so that no stabilizer is wasted.

If liquid asphalt is chosen as the stabilizer, the asphalt distributor trucks would be employed. These are designed with controlling devices so that the spread rate is uniform and in the quantity required.

The distribution process will proceed just ahead of the mixing operation. Since the mixer is limited to a working width of 75 in. (190.5 cm), and the spreaders and distributors have a much wider width of application, they will not constrain the production of the system. The stabilizer should not be distributed wider than 75 in. This will preclude excessive exposure to the environment prior to mixing, which is important, as dry lime or cement may be blown and the volatiles or water in the liquid asphalts may begin to evaporate.

It is possible that more than one application of the stabilizer could be required. If lime were being used with a highly plastic clay (PI > 50), it may be advantageous to apply the lime in two increments [53]. The first application of a portion of the total quantity required would be mixed, rolled and allowed to cure for a few days. This would mellow the clay and reduce the plasticity to a point that the soil could be further pulverized. Then the remaining lime would be added to complete the stabilization. It is also common that asphalt stabilization be accomplished in graduated steps to obtain the specified mixture [44].

Mixing and Blending. Once the stabilizer has been laid, the mixer/pulverizer machines are again employed. These travel along and mix the stabilizer and soil at varying speeds depending upon the depth of application. Obviously, the greater the depth of application, the slower the production. Additional pulverization is achieved during the mixing.

As discussed in the Mix Design portion of this chapter, the design quantity of stabilizer should be adjusted to counter the variability of the construction process. In-place mixing efficiency, as measured in terms of in-place stabilized-soil strength compared to laboratory tests, is commonly 60 to 80% of that which is obtained in laboratory samples [44]. Therefore, by increasing the stabilizer quantities by one or two percent, there is a much higher probability that the required amount of stabilizer will be uniformly mixed into the soil.

For soils that do not require initial pulverization, or require a minimum amount, mixing and pulverization are achieved simultaneously in the mixing and blending process.

For lime stabilization, mixing/pulverization should continue until 100% of the soil-lime mixture passes the 1-in. sieve and 60% passes the No. 4 sieve [44]. For cement stabilization, 100% passing the 1-in. sieve and 80% passing the No. 4 sieve, except for small gravel and stone, should be achieved.

Compaction. Compaction should commence as soon as possible after mixing has been completed. With lime stabilization, timely commencement of compaction will preclude excessive exposure to air which results in carbonation. This reaction can adversely affect strength development. Experience has shown that compaction of emulsified asphalt mixes should begin just before, or at the same time as, the emulsion starts to break, i.e., the asphalt globules begin to coalesce.

In horizontal construction, a pavement layer is normally required to be compacted to at least 95% of maximum dry density. The densification of the soil increases its strength and resistance to settlement and deformation under load. Also, it is common that layers be constructed in 6-inch lifts, each of which is compacted.

In application to the SELF, construction of the base layer should be done in one lift at the specified thickness to minimize construction time. In lime stabilization, it is more beneficial to construct in one thick lift than multiple lifts as carbonation is kept to a minimum [53]. By compacting in one thick lift, though, it will prove difficult to achieve 95% of maximum dry density, especially in fine-grained soils. Achieving 95% compaction in the upper 6 to 9 in. (15-23 cm) with something less in the lower part of the base is often acceptable in lime-soil mixtures [53]. This would seem to be reasonably applicable to other types of stabilization and to construction of the SELF's base.

In order to minimize the required compactive effort, it is critically important that the soil be at the proper moisture content and that the appropriate compaction equipment be utilized for a given soil type.

Sheepsfoot rollers provide the most efficient compaction of fine-grained soils. The greatest compaction efficiency (least number of passes to achieve the required density) is obtained when the moisture content is slightly dry of optimum (1-2%). This will still provide enough moisture to aid uniform mixing. The sheepsfoot will not compact the top few inches of the layer so a rubber-tired or steel wheel roller must be used to finish the compaction.

Coarse-grained, cohesionless soils compact best with a rubber-tired roller at a moisture content slightly wet of optimum. A steel wheel vibratory roller may also be used.

Once the best compaction moisture content has been determined, it should be further adjusted to compensate for expected evaporation, especially if it is a hot, dry climatic condition. An additional half percent should be sufficient.

Final Shaping and Curing. Once all compaction has been completed, final shaping may be required to insure that the proper drainage slope is achieved.

Curing is very important to strength development. Time, temperature and moisture are the key elements to proper curing. Temperature has been adequately addressed in Chapter II. The normal cure times are listed in Table 11. Since moisture is also important to curing of lime and cement stabilized soil, additional water may have to be sprinkled on, especially if the weather is hot and dry. With asphalt stabilization, no moisture should be added. Its curing depends upon evaporation of its liquid medium for strength gain.

Soon after final shaping and compaction have been completed, mat laying may commence. Since this operation is very labor intensive, curing will not be adversely affected by foot traffic. Equipment traffic, however, should be avoided during the initial curing periods as recommended in Table 11. When equipment is needed, it should be restricted to matted surfaces only.

Estimation of Construction Time

The primary point of the paper is to discuss the engineering advantages that stabilized soil has over unstabilized soil in providing a one year service life for the SELF. In order to gain these advantages, SELF planners must be willing to sacrifice additional construction time.

This section will attempt to provide a reasonable order of magnitude estimate of the additional time that would be required for stabilization of the base layer over and above that which would be required for unstabilized-soil construction.

Since the compactive effort is essentially the same whether or not stabilization is used, it will be included with the normal soil construction process and, therefore, not be estimated. SELF planners should expect some compactive effort reduction by using stabilization but this would be most difficult to estimate.

The SELF requires 2,607,720 sq. ft. of AM2 matting for its runway, taxiway and parking aprons [9]. Therefore, this same area will require stabilization at the specified depth. Normally, in airport pavement design, the pavement structure may be different at various areas at the facility. For example, the taxiways, aprons and ends of the runways are subjected to slow moving, often channelized traffic which induces the most pavement damage. On the other hand, the center third of the runway receives much less damage because of the high speeds of the aircraft and partial lift. Therefore, a thinner pavement may be used than in the other areas. This is normally referred to as traffic areas. This concept has not been applied in the context of this paper. The base layer has the same thickness throughout. SELF planners, however, might want to consider it to reduce construction time.

In developing estimates of construction time, it must be emphasized that a number of variables will affect the construction process. The type of soil will certainly dictate the production rates, as a cohesive soil is much more difficult to pulverize than a cohesionless one. If more than one pass of the pulverizer is required, quite obviously, additional time is required. The thicker the base layer is, the slower the production of the pulverizing/mixing machine, and so on.

An estimate will be developed for a range of reasonably expected construction times for a desirable soil condition to one that would not be desirable.

The Pettibone Wood Company has provided some production data for its model 750C "Speedmixer" [37] which will be used as the basis of the estimate. They suggest the following production rates for a USCS CL soil at different states of compaction:

a. wet, uncompacted - 1,067 cubic yards per hour (cy/hr.)

b. partially compacted - 948 cy/hr.

c. compacted - 439 cy/hr.

At these production rates, one pass of the fleet of six pulverizers/mixers over the entire area of the SELF for a 16-in. base would be as follows:

a. $2{,}607{,}720 \text{ sq. ft.} \times \frac{16 \text{ in}}{12 \text{ in/ft}} \times \frac{1 \text{ cy}}{27 \text{ cu ft}} \times \frac{1 \text{ hr}}{1{,}067 \text{ cy}} = 120.7 \text{ hrs.},$

$$\frac{120.7 \text{ hrs}}{6 \text{ machines}} = \underline{20.1 \text{ hrs.}}$$

b. Same equation but with a production of 948 cy/hr = <u>22.6 hrs.</u>

c. Same equation but with a production of 439 cy/hr = <u>48.9 hrs.</u>

So, if a desirable soil were encountered, e.g., a non-plastic GW, that required no initial pulverization, the stabilization time might equal one pass of the mixing fleet at the highest CL production rate of 1,067 cy/hr. Therefore, 20.1 hrs. would be required. An arbitrary 10% adjustment is added to this figure to account for moisture content adjustment with the water distributors. This would result in <u>22 hrs.</u> of construction time.

Now assume that a plastic clay is encountered. This may require as many as three passes of the pulverization fleet first, followed by addition of the stabilizer and mixing. In this case, each of three passes during pulverization would be at a continuing higher production rate due to the degree of pulverization achieved during the previous pass. Three passes would be followed by a mixing pass at the highest production rate. Therefore, the total time is as follows:

1st pass - 439 cy/hr. - 48.9 hrs.

2nd pass - 948 cy/hr. - 22.6 hrs.

3rd pass - 1,067 cy/hr. - 20.1 hrs.

Mixing pass - 1,067 cy/hr. - 20.1 hrs.

Total = 111.7 hrs. adjusted up 10% = 123 hrs.

To place these estimates into complete perspective, assume that no more than 12 productive hours of work per day can be expected. No night work can be undertaken as lights might draw enemy fire. Therefore, in terms of construction days:

a. For the desirable soil -

$$\frac{22 \text{ hrs.}}{12 \text{ hrs/day}} = 1.8 \text{ days}$$

b. For the undesirable soil -

$$\frac{123 \text{ hrs.}}{12 \text{ hrs/day}} = 10.3 \text{ days}$$

Now consider some other factors that are inherent to horizontal construction. The above estimates are based on a fleet of six pulverizers working productively for 60 minutes per hour without mechanical failures, day after day. A 60-minute productive hour is seldom consistently achieved in construction for any number of reasons; the operators need to take care of personal business or get a drink of water, to name a few. Often, horizontal construction is estimated on a 50 minute per hour basis. Applying this to the previous estimates, the following changed values are yielded:

a. Desirable soil -

$$1.8 \text{ days} \times \frac{60 \text{ min.}}{50 \text{ min.}} \text{ factor is } 2.2 \text{ days}$$

b. Undesirable soil -

$$10.3 \text{ days} \times \frac{60 \text{ min.}}{50 \text{ min.}} \text{ factor} = 12.3 \text{ days}$$

Construction equipment often experiences mechanical down time due to the wear and tear of heavy construction. A deadline (equipment out of service) of 10 to 30% is not uncommon, based on the author's experience. Consider the situation where only five pulverizers are in service at any one time, combined with a 50-minute productive hour. The estimates would change as follows:

a. Desirable soil-

$$2.2 \text{ days} \times \frac{6}{5} \text{ factor} = 2.6 \text{ days}$$

b. Undesirable soil-

$$12.3 \text{ days} \times \frac{6}{5} \text{ factor} = 14.8 \text{ days}$$

The above are just a few examples of the variability of the construction process. Many other factors and delays will affect the production and project time.

In summary, in the interest of providing an order of magnitude estimate for stabilization of the SELF's base layer, a working range of 3 to 21 days can be used, depending upon the many factors, such as soil type, which will ultimately control the construction time.

CHAPTER V

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The objective of this paper was to discuss the engineering advantages that stabilized-soil construction of the base layer of the SELF has over unstabilized soil construction. Because the SELF concept mandates sustained tactical and cargo airlift operations for up to one year, the base layer under the AM2 matted surface, if constructed without stabilization, may not be able to maintain its strength under many climatic conditions. This is due to the degrading effects of water, which easily infiltrates the soil through the mat's joints, on the support capacity of many soils.

The advantages of soil stabilization are many. The most important of these in application to the SELF are increased strength and enhanced durability. These improved engineering properties may be achieved by adding chemical stabilizing agents such as lime, portland cement or liquid asphalt products to the soil. Each of these works most effectively with certain types of soils so selection of the best stabilizer for a given soil type is important.

Lime stabilization works best with fine-grained soils. Some of these, especially many clays, may be highly plastic or cohesive. The addition of lime to these soils causes chemical reactions which reduce the plasticity and change the texture of the soil. This makes the soil much more workable from a construction perspective. Some soils, when treated with lime, will chemically form a cemented condition which can increase the strength of the soil. These improved soils will also resist the degrading effects of infiltrating water and maintain much of their increased strength. If the SELF were to be constructed in an environment where freezing temperatures may occur, lime stabilization of frost-susceptible soils, where sufficient strength has been achieved, will resist damaging frost heave and offset loss of strength by freeze-thaw action.

Cement stabilization is quite versatile as it can be used effectively with a broad range of soil types. The addition of cement to soils causes strength increases through a bonding of the soil particles. Once this cementing has occurred, the mixture will resist the degrading effects of moisture on the support capacity of the soil.

Liquid asphalt, i.e., cutbacks and emulsions, work best with coarse-grained, nonplastic soils. The addition of these products will waterproof the soil particles and bind them together. Some increase in strength and enhanced durability can be expected.

Whichever stabilizer is chosen, if it is the best one for a given soil condition and is properly used, the soil can be constructed to a strong, durable state and will provide a one-year service life for the SELF with minimum maintenance.

Because of the different types of aircraft that will use the SELF with varying loads, tire pressures and gear configurations, a specific thickness of stabilized base material must be constructed, depending upon the expected volume of this mixed traffic. When loads are applied to a pavement, stresses on the weaker subgrade soil (the soil beneath the stabilizea layer) that result from these loads must be absorbed or reduced by the base material. The thickness of the base layer will control these vertical subgrade stresses and, if properly designed and constructed, will preclude pavement failure. This paper provides several methods that can be used to properly design the base layer.

This paper also discussed the construction equipment held by the Naval Construction Forces that would be used should a stabilization project be undertaken. These equipment resources could realistically accomplish a soil stabilization project of the magnitude of the SELF.

The AM2 matting was not designed for a facility like the SELF where high volumes of mixed aircraft traffic must operate for a duration up to a year. It was designed for an airstrip that could be rapidly and easily constructed, without a base layer, on marginal soil support conditions for limited loads, tire pressures and aircraft volume. To bring this into perspective, if the 36 F4 aircraft from a MAG were to operate on an AM2 matted field with a subgrade CBR of 4, with no base layer which restricts the single wheel load to 17,100 lbs. (38,000 lb. gross aircraft load) and tire pressure to 250 psi, at three sorties each per day, the service life of the pavement would expire in 68 days (based on the Navy Design Method presented in this paper). Only through construction of a strengthened base layer through chemical stabilization could the SELF concept be reasonably fulfilled. The strength of the base could be achieved in many soils through unstabilized soil compaction. But to maintain this strength for one year under many climatic conditions would undoubtedly prove to be impossible without an exorbitant amount of disruptive maintenance.

Technically, soil stabilization of an AM2 matted SELF could turn a concept into a reality. Logistically, it may be deemed infeasible. As an example, to stabilize a 16-inch base layer under the 2.6 million plus square feet of AM2 matting, approximately 10,500 tons of lime or cement would be required for a 6% treatment. That is the equivalent weight of 840, 25,000-lb. pieces of construction equipment. This is quite a large logistical demand.

The current SELF concept mandates that the airfield be completed within 45 days [16]. Based on a review of Reference [26], the research connected with this paper, experience, knowledge of construction, and sound intuition, this would be an impossible task, even under almost ideal conditions. If stabilization were chosen, it would further compound the construction time problem, possibly by as much as several weeks.

It appears that if the SELF concept is to be realized, one of two courses of action must be taken. First, if a durable membrane could be developed that could be placed under the AM2 matting and made to withstand wear and tear, especially at construction, a one-year service life could be obtained through conventional soil compaction techniques. The membrane would prevent surface water from infiltrating the soil beneath it. Secondly, a matting system should be developed to replace the AM2 that would be strong and rigid enough to preclude the requirement of the base layer, over marginal soil support conditions, and support the required aircraft loads, tire pressures and volumes of traffic for one year. The matting would have to be logistically feasible and relatively easy to install without specialized equipment.

The Navy Civil Engineering laboratory has developed a fiberglass-reinforced rigid polyurethane expedient pavement [51, 52] which would potentially resolve the matting problem. Research and development has not progressed to a point that this system has been operationally adopted. Continued research and development of this matting and others like it could eventually fulfill the SELF concept.

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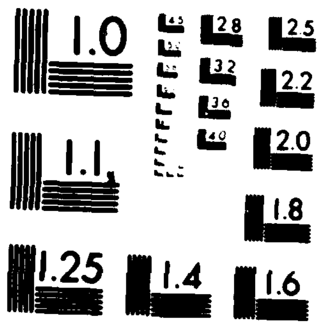

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