CERTAIN ENGINEERING ASPECTS OF SOIL STABILIZATION WITH LIME

A Thesis

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by

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PREFACE

This thesis is an investigation of the effects produced on the physical properties of several soils from Manitoba and Ontario, when admixtures of lime are used.

ACKNOWLEDGEMENTS

The author is indebted to all those who have contributed, encouraged or assisted in making this investigation possible. Special recognition is given to the following who played a major role in this research program.

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CHAPTER I

INTRODUCTION

The main purpose of this thesis was to determine what effects additives of lime had on stabilizing local Manitoba soils in the construction of base and subbase courses for highways, railways, city streets, and airfields. This thesis presents the results of laboratory investigations of lime stabilization on a variety of soils obtained from different parts of Manitoba, as well as two types from Ontario.

The use of admixtures of lime for soil stabilization has been a subject of considerable interest to highway and airfield engineers in recent years, particularly in the southern part of the United States. Structural engineers are quite familiar with the phrase, "The building is only as sound as its foundations". This is also applicable to highway and airfield engineering, particularly in this day and age when we must contend with every increasing traffic loads. Stable, durable road bases are a vital necessity for the construction of good roads.

Where abundant supplies of satisfactory base materials, such as gravel and crushed stone, are locally available, the construction of road beds is easily accomplished. However, in localities where there is a deficit of satisfactory base materials, the great need arises for an economical means of converting or upgrading natural soils and soil materials into satisfactory base materials. Highway and airfield engineers commonly refer to this method of improving the strength of this material as "soil stabilization".

CHAPTER II

Review of the Development of Lime Stabilization

Although the use of lime as a soil stabilizing agent has only recently received widespread recognition, the idea was born many centuries ago.

The ancient pyramid builders recognized the fact that lime had cementitious properties when mixed in with clayey soils. The pyramids of Shensi in the Tibetan-Mongolian Plateau, over 5000 years old, and still intact, were constructed out of compacted mixtures of clay and lime[1].

The historic Romans, famous for their advanced form of civilization centuries before the birth of Christ, are believed to have been the first users of lime in connection with road construction. Most of their principal roads radiating out of Rome were stabilized with lime.

The Romans were marvels at engineering feats. Even today we are still amazed when we think about the towering Colosseum, the mighty aqueducts, and the Appian Way. Unfortunately, their civilization was finally destroyed after a series of disastrous battles, however, the ruins of their works are still in evidence today.

According to studies made by archaeologists of ancient Roman civilization, the typical section of the Appian Way[2] was approximately four feet thick and consisted of five layers from the wearing surface to the subbase. The materials used in the road were a gradation of rock, sand, and lime. In three of these lifts admixtures of lime were used to give added stability.

Crude forms of lime stabilization have been used on the rural and village roads of China for years. Lime, serving mainly as a cementitious material, has also had application in the construction of earth dams in India and China [3].

A considerable amount of interest has been shown in the United States in all forms of soil stabilization for constructing roads. However, lime stabilization has been introduced only recently. Additives of lime in the hydrated form appeared to have been first used for stabilization purposes in 1924, in the State of Missouri, where short experimental test strips were laid. The U.S. Bureau of Public Roads in Iowa and Dakota followed with similar experiments. The results of these first test strips showed that the addition of from 3% to 6% of hydrated lime made earth roads on clay soils reasonably stable.

Although the test results obtained from these experiments were encouraging, it was not until the second world war that lime stabilization was used on a large scale. The first major application was in runway and taxi-strip construction on airfields in Texas. The enormous demands for satisfactory base course materials could not be met economically unless lower grades of gravels with high clay contents could be successfully stabilized. The use of lime to up-grade the plastic material proved successful, and was one of the first uses of this method in the United States.

Shortly after the war, the Texas State Highway Department commenced using lime stabilization on civil roads. At the beginning, they employed a locally available waste lime, however, later, they began using commercial hydrated lime ranging from 3% to 8%. The results were quite satisfactory and by 1953, 250 miles of road had been constructed in Texas using soil stabilized with lime [3].

Shortly after, British road builders, hearing of the success the Americans had achieved with lime stabilization, began conducting lime-soil laboratory tests. Their first actual application was in Worcestershire, where hydrated lime was successfully used in place of stone for road base construction, the soils stabilized in these roads being sands, or light clays.

Similarly, in other parts of the British Commonwealth, soil engineers in Northern Rhodesia and Nyasaland found that they were able to up-grade clayey gravels for road bases by stabilizing or actually neutralizing the excessive amounts of clay fines in the material by adding as little as 2 to 3% of lime. They found from laboratory experiments (Road Research Laboratory), and from actual field applications, that lime did have a marked effect, particularly in stabilizing cohesive soils such as heavy clays. In the early 1950's, British engineers [4], like Kerr and Brooke-Bradley, used hydrated lime in conjunction with Portland cement to reduce the plasticity, alter the texture, and make cohesive soils more workable.

According to published literature [4], it is believed that the first experiments in the Soviet Union, in connection with lime-soil mixtures for roadways, were made by Okhotina in 1926. These experiments were conducted on Cambrian clays, and it was found that an additive of 5% hydrated lime greatly reduced the plasticity of such clays. Okhotina's findings were later confirmed by other Russian workers on a variety of soils (Groditskaya and Ipatova, 1932; Bykovskii, 1937; Volkov, Gelmer Zasobin and Panteleev, 1948.) It is unknown whether the Soviet Union has any lime stabilized roads as yet, however, tentative specifications have been prepared for the construction of roads using lime-stabilized soil. The results of laboratory tests on Russian soils reveal that there is an optimum percentage of lime for successful stabilization. Although the principle by which this is judged is unknown, they have concluded that the range is within 5 to 12% by the weight of the soil [4].

CHAPTER III

Chemical and Physical Properties of Lime in Relation to Soil Stabilization

The widespread production, low cost, and abundant supply of lime has made it one of the most widely used chemicals in industry today.

The raw material, limestone, occurs in numerous surface deposits across Manitoba [5]. One of the largest areas lies in a great belt 100 miles wide, extending northwesterly across the province, from near Winnipeg to The Pas.

At present, however, there are only two major locations where lime is produced on a large scale. Stonewall [5], which is located approximately 25 miles northwest of Winnipeg, has long been an important centre for the production of white dolomitic lime. Spearhill [5], located approximately 120 miles northwest of Winnipeg, is known as the only location in Manitoba where high-calcium lime is produced.

The proper "burning" of lime is a fairly complex process. In general, however, lime is produced by the controlled calcination of limestone at temperatures in the vicinity of 2500°F. The limestone is prepared by first drilling and blasting the deposit. The product of the blast is then classified so that only the satisfactory size of stone is burned. During the burning process a careful control is kept on the temperatures in the hot zone, the flow of material through the kiln, and the removal of the end product from the bottom of the kiln to ensure the production of a pure homogeneous chemical.

Quicklime, the product of calcination, consists of the oxides of calcium and magnesium, and in this locality it is available in two forms:

High calcium quicklime, produced at Spearhill, usually contains over 97 per cent calcium oxide (CaO), the magnesium oxide (MgO) content varying from 0.5 to 2.5 per cent.

Dolomitic quicklime, a product of the Stonewall deposit, usually contains from 45 to 50 per cent of calcium oxide and from 35 to 40 per cent of magnesium oxide.

Both of these types of quicklime have a considerable affinity for water. High calcium hydrated lime contains generally from 72 to 74 per cent calcium oxide and from 23 to 24 per cent water. Under normal hydrating conditions only the calcium oxide fraction hydrates. The resulting chemical composition is as follows: 46 to 48 per cent calcium oxide, 33 to 34 per cent magnesium oxide, and 15 to 17 per cent water. Under steam and pressure it is possible to hydrate almost all of the magnesium oxide as well as all of the calcium oxide, however, according to published literature, pressure hydrated dolomitic lime has had no application in soil stabilization.

Chemical lime is a white solid having a crystal lattice structure. The product of calcination, quicklime, reacts highly with water, generating a considerable amount of heat in the hydration process. In the presence of moisture, the lime reacts slowly with the carbon dioxide of the air,forming water insoluble carbonates. Although commercially, quicklime is available in a variety of sizes ranging from lump to a finely pulverized powder, it tends to react more favourably and the most efficient utilization is made of the chemical when it is in the pulverized form. Quicklime is commercially available by the carload, in bulk, or in moisture proof paper bags. Hydrated lime can be purchased as a dry powder either in bulk or bagged. On some projects it has been found more convenient to obtain the lime in the form of a suspension in water or a slurry [8].

CHAPTER IV

Theory of Lime Stabilization

According to published literature, no specific answer has been given on exactly how lime reacts with a soil. The theory behind the behaviour of a mixture of soil and lime is rather complicated. It did appear, however, that the reaction was chemical in nature. The first apparent change in the soil's properties was noticed immediately after mixing a small quantity of lime with a moist soil. Particularly when the lime was mixed in with a clay, there was a sharply defined drop in the soil's stickiness. It appeared that an agglomeration of the finely divided clay particles occurred and the soil then exhibited properties similar to those of a silt or a sand.

Immediately after a soil had received an admixture of lime there was a pronounced change in the plasticity properties of the soil. (The plasticity properties illustrate the soils behaviour in relation to its moisture content). Generally, a reduction of the plasticity index occurred by either a decrease of the liquid limit, or an increase of the plastic limit. In highly plastic soils, a combination of both was experienced. With some soils, the liquid limit was actually raised, however, in general, the net effect was a lowering of the plasticity index. A study [6] of the effect of lime on the Atterberg Limits for a variety of soils with plasticity index values ranging from 15 to 50, showed that, with 3 percent lime, the plasticity index values were reduced to from 2 to 15 respectively. A lowering of the plasticity index signified that a lime addition to a fine grained soil makes it increasingly difficult for the soil to become plastic.

K. A. Gutschick [7], stated that the agglomeration of clay particles, and the plasticity index reduction are explained primarily by base exchange. The base exchange reaction involves the replacement of the smaller weaker sodium and potassium ions of the clay by the larger calcium ions from the lime. Gutschick stated that this base exchange in the soil had another significant effect, in that it reduced the thickness of the absorbed moisture films or envelopes surrounding the clay grains, thereby reducing the plasticity. Thick water films shield the particles, so that they do not come into intimate contact with their neighbours; the mass they form under this circumstance possesses low stability and a very plastic state prevails [7].

Test results [9] at the Texas Engineering Station on clayey soils showed that the reaction of a soil and lime increased as the plasticity index and the capacity to absorb ions increased. Expressing the reactivity in terms of clay minerals, montmorillonite reacted the best with lime, illite next, and kaolinite the poorest, since the base exchange capacity of these types is the largest for montmorillonite clays.

Dr. R. L. Handy [2] explained that the plasticity index of a soil is a function of the surface area of individual grains of soil. If the surface area of the soil grains is large, (for example, in the case of minute flaky clay particles), the capacity to hold water by the phenomenon of electrostatic attraction is extremely large. On the other hand, soils with larger grains, having a shape between that of cubical and spherical, have a lesser capacity to absorb water since there is less available surface for wetting. This force of electrical attraction is of tremendous magnitude when compared to the force used to compact a soil. Hence, a clay material, even though well compacted, will absorb water and become plastic. Ultimately, if enough water is absorbed by the clay particles, the water "chains" bonding the particles together become so extended they lose much of their strength.

The fact that the addition of lime to a clayey soil reduced the plasticity index means that the reaction between the lime and the soil causes the clay particles to flocculate, or to stick together and behave like a silt. The clay no longer exhibits its tremendous ability to hold water between particle cleavage planes, but now occurs in larger grains and of shapes near that of cubes or spheres.

Clare and Cruchley [4] obtained results in the calcium and pH determinations which proved that stabilization was associated with the phenomenon of base exchange. Their results were in general conformity with the suggested mechanism of base exchange. They stated that the variation in the rate at which calcium is removed from the solution in the mixture was probably a complex function of the initial surface area (or edge area) of the clay particles and the extent to which this was altered by reaction with the lime. Their report indicated that no investigations had yet been made on the shear strength development of soil-lime mixtures, however, they felt that it was unlikely that the measurement of base exchange capacity, and the pH of the soil or the determination of the plasticity characteristics of the freshly prepared soil-lime mixtures alone would prove satisfactory means for determining whether a soil could be stabilized with lime.

Another but slower reaction is believed to take place when a soil is stabilized with lime. This reaction causes an increase in the unconfined compressive strength of a lime stabilized soil. The unconfined compressive strength is determined in the laboratory with a test machine which subjects a soil specimen (without any lateral support) to a compressive stress. Soils treated with lime increase on the average from 30 p.s.i. to 100 p.s.i., and after one to two years the strength gain is considerable, usually in the range of from 300 to 1500 p.s.i. [6].

In the early portion of the road-building era in Rome, lime was used primarily as a mortar. This mortar usually consisted of two parts locally derived sand and one part lime. It was used either to make lime-concrete, when nine parts of gravel were added, or to fill in the voids around stone chips or slabs in the road.

However, about 150 B.C., the Romans found that they could vastly improve their mortars by using a volcanic ash additive from Pozzonli (a town near Naples). The word pozzolan, after this Roman town, was used to describe a material exhibiting properties of volcanic ash.

A pozzolanic material is defined in A.S.T.M. Specification 129 as " . . . a siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value but will, in finely divided form, and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. In this category are included such materials as fly ash, volcanic ash, heat-treated diatomaceous earths, and heat treated or raw shales and clays." [10]

A paper by Chester McDowell [1] stated that a second, slower chemical reaction than base exchange caused cementation when lime in the form of calcium hydroxide combines with various forms of available silica to form insoluble calcium silicate. These reactions cause slow cementing, which if not too slow for practical application, can be beneficial, in that generally the faster a mixture gains its strength, the greater are its chances of having shrinkage. Excessive shrinkage is objectionable, since this condition causes a reduction in the soil's ability to resist tensile and flexural stresses.

K. A. Gutschick's [7] opinion is that the formation of a "natural cement", composed of calcium-alumino-silicate complexes, is the most important reaction involved in lime stabilization. This cement binds the soil particles together into a concrete-like mass due to the formation of hydrated calcium silicates and aluminates. Most clayey soils contain sufficient pozzolanic material to react with lime. Silty and sandy soils, on the other hand, lack pozzolans; therefore, to stabilize these types of soils, it is necessary to add sufficient pozzalon, whether it be fly ash, natural pozzolan, clay or any material which has the tendency to react with lime to form a natural cement.

Lea [11] also reported that the cementing effect is due to the formation of hydrated calcium silicates and probably also aluminates, rather than to cation exchange effects, and equilibrium is reached only after long periods of time. The fact that pozzolans are more reactive than other siliceous materials towards hydrated lime, has been ascribed to their physical condition, which is said to be amorphous, or vitreous, whereas most natural silicates are crystalline.

The third suggested mechanism involved in lime stabilization is carbonation. Carbonation can be defined as the formation of calcium carbonate with cementing properties by reaction with atmospheric carbon dioxide.

Borisova, a Russian soils engineer, suggested a fourth mechanism. He reported [12] that the mechanical strength of soil-lime mixtures was due to the crystallization and subsequent carbonation of calcium hydroxide, the crystallization resulting from saturation of the soil-water solution with lime, as a result of drying of the soil. Crystals of calcium hydroxide were believed to interlock with each other, and with the soil particles, cementing the mix into a monolithic mass. The long-term gain in strength of soil-lime mixtures was ascribed to carbonation of the calcium hydroxide.

According to the investigations carried out by K.E. Clare and A. E. Cruchley [4], it was shown that the two latter suggested mechanisms could be largely discounted, since all the samples were stored in air-tight containers. Hence, they stated that any changes in properties during storage could hardly have been due either to carbonation of the lime or to crystallization of calcium hydroxide following a reduction in moisture content. They felt that recrystallization of the calcium hydroxide would be possible if the temperatures at which the specimens were stored fluctuated markedly but they concluded that this was considered to be unlikely, since the specimens were kept away from draughts or sunlight.

K. A. Gutschick [7] stated that carbonation seemed to be of little significance, since very little carbon dioxide would penetrate the base after it was compacted, cured and surface treated. He felt that carbonation should actually be avoided, particularly during the period of lime application and mixing, since lime's effectiveness to react chemically with the soil would be decreased. He felt that the theory of crystallization of the calcium hydroxide was a possibility, but also of little significance.

The following is a general summary from published literature of how the physical characteristics are changed when additives of lime in the range of 1 to 5 per cent are used;

(a) The plasticity index is reduced.

(b) The strength is increased for confined and unconfined compression and California Bearing Ratio tests .

(c) Shrinkage is reduced due to slow strength gains.

(d) A fairly constant moisture content is maintained through the years in road bases treated with lime.

(e) The soils porosity is increased by a reduction of the soil binder (clay) content by the agglomeration mechanism of base exchange.

(f) The swelling pressures of heavy clays are reduced [13].

Four different types of reactions have been suggested [4] which bring about the changes in the physical characteristics of the lime stabilized soil. They are:

(1) The formation of calcium silicates and/or aluminates with cementing properties by reaction with the clay.

(2) The flocculation of the clay particles, and substitution of calcium ions for the exchangeable cations on the clay particles.

(3) The formation of calcium carbonate with cementing properties by reaction with atmospheric carbon dioxide.

(4) The formation of crystalline calcium hydroxide having cementing properties, due to supersaturation of the soil solution.

CHAPTER V

Limitations and Precautions in the Use of Lime Stabilization

Intense laboratory and field studies have shown that lime stabilization is not a "cure-all". It is important that highway engineers have taken certain precautions in its use to insure successful soil stabilization with lime. Lime stabilization is just another tool with which to build better roads, and a fairly new one in this country; hence we must understand precisely its uses and also its limitations. The following is a summary of its limitations set out by roadbuilders with extensive experience in lime stabilization:

(1) Lime should not be used alone in conjunction with stabilization of non-plastic soils such as sands, pure silts and gravels [6]. It is essential that a certain amount of clay is present to promote the chemical reactions stated earlier. If a material should be lacking natural pozzolans, artificial ones such as fly ash or volcanic ash should be added.

(2) It has been found that not all plastic clayey soils will react with lime [6]. Hence this stresses the necessity for preliminary laboratory investigations.

(3) Construction or installation should be made during above freezing temperatures. American engineers [3] suggest the construction season should be approximately from April 15 to September 15 in the northern United States. For the shorter summers in Manitoba, it would follow that the construction season for lime stabilized roads would be much shorter, possibly between May 1 to September 1. The lime-soil reaction is most effective,

although slow, under mild and moist conditions.

(4) If lime is used in base course construction, it is essential that the soil to be treated has at least 50 per cent of over No. 10 mesh sizes. If it is used for subbase work, considerably less granular material is necessary, and in subgrade construction, no granular material is necessary [6].

(5) Experience has shown that lime stabilization should not be used in base construction of less than 6 inches thickness [3].

(6) Designs for the proper percentage of lime and the adequate thickness of the stabilized base should be determined in the laboratory prior to field application [6].

(7) Proper equipment and construction procedures should be followed [14]:

(a) The in-place material should first be sufficiently scarified to the proper depth (minimum of 6 inches).

(b) The loosened material should then be uniformly mixed and pulverized after the lime is placed in the soil. The mechanical rotary mixer has been used with most favourable results.

(c) The soil-lime mixture should then be compacted at the optimum moisture content. Usually three types of rollers are used; namely the sheepsfoot, the pneumatic and the flat wheel in consecutive order.

(d) It is recommended that the compacted lime stabilized base be moist cured for a period varying from 3 to 7 days. Moisture must be present to promote chemical reaction during the initial stages of curing.

(8) The stabilized base course must be covered with

some form of wearing surface to prevent abrasion [3,6,14]. This surface seal or film also prevents the base from drying out and enables the materials to react properly into a hardened mass.

CHAPTER VI

Previous Laboratory Investigations in Canada

A review of previous published laboratory work in lime stabilization in Canada revealed that this investigation was one of the first attempts to evaluate the effect of lime on Canadian soils.

In correspondence with the Technical Information Service of the National Research Council it was found that no technical papers or bulletins have yet been written or published describing any laboratory or field investigations on lime stabilization in Canada.

An undergraduate thesis [15] on lime stabilization was submitted to the Faculty of Civil Engineering, the University of Manitoba, in 1950. This thesis illustrated the effects of 6 different percentages of hydrated lime on the Atterberg Limits of a local soil.

The following is a summary of the main points of that thesis [15]:

The soil used was a silty clay obtained from a sewer excavation in Winnipeg at a depth of 6 to 8 feet. The soil consisted of 54% clay, 36% silt and 10% fine sand. A commercial hydrated lime was used, however, the source or composition was not disclosed. The results obtained are summarized in Table 1 and Figure 1.

Mixture	Liquid Limit	Plastic Limit	Plasticity Index
Soil with no lime	66.3	32.0	34.3
Soil with 2% lime	58.3	41.7	16.6
Soil with 6% lime	53.2	42.0	11.2
Soil with 10% lime	50.0	42.4	7.6
Soil with 14% lime	49.2	43.8	5.4
Soil with 18% lime	51.4	46.0	5.4
Soil with 22% lime	57.0	48.6	8.3

Table 1.[15] The Variation in the Liquid Limit, the Plastic Limit and the Plasticity Index of a local Winnipeg Silty Clay, when Treated with Different Percentages of Hydrated Lime.

Although only one soil was tested for the effect of lime on the Atterberg Limits, this investigation[15] brought out several significant factors which can be considered as the basis for the continued research on this topic:

(1) Lime reacted with this particular Winnipeg soil.

(2) The reduction of the plasticity index was in accordance with published results of similar lime-soil testing elsewhere.

(3) Any soil to be stabilized calls for an optimum or most economical percentage of lime. For this parti-

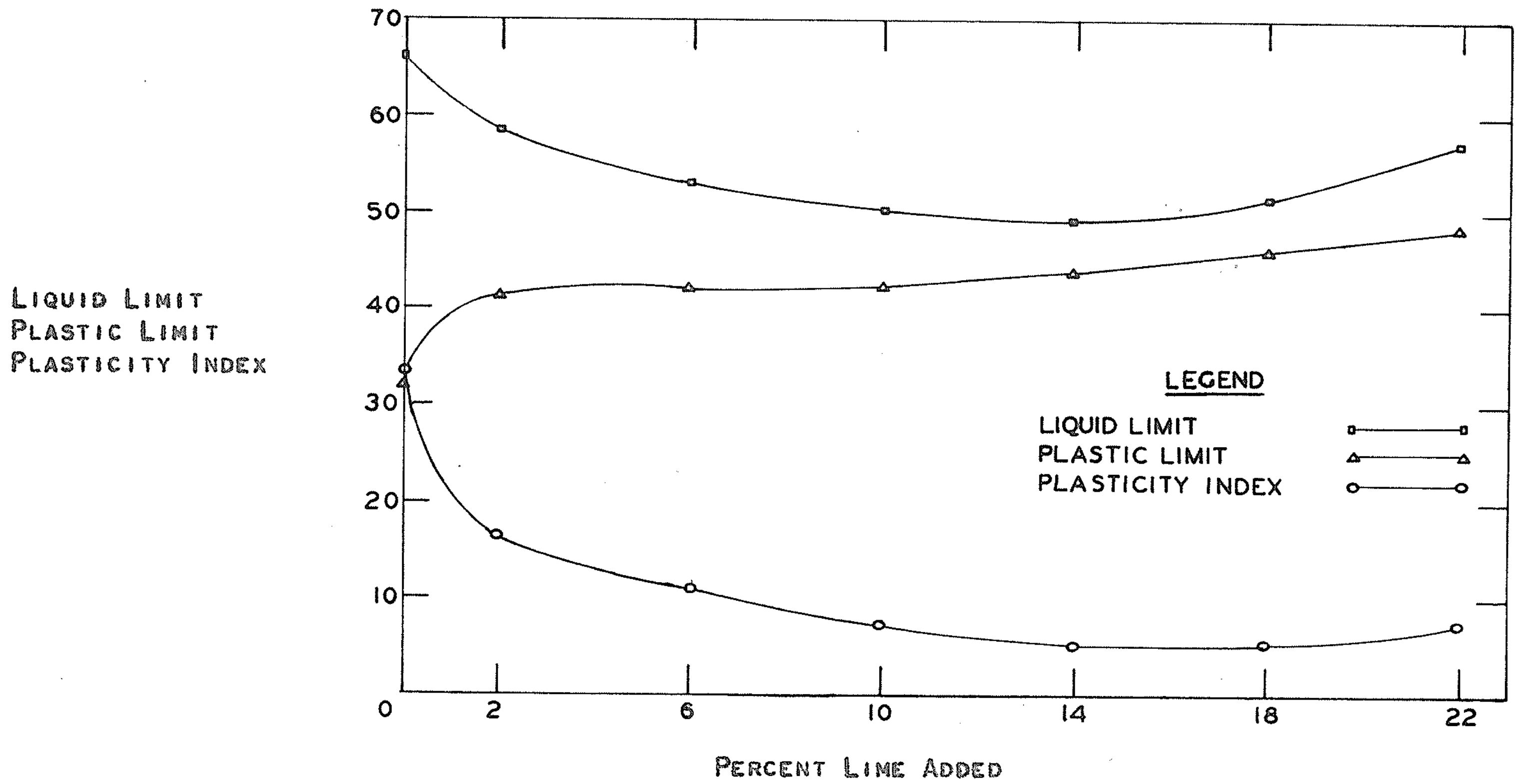

Figure 1 [15]. The Variation in the Liquid Limit, the Plastic Limit and the Plasticity Index of a Local Winnipeg Clay, when Treated with Different Percentages of Hydrated Lime.

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cular soil the plasticity index was reduced sharply at a low percentage (2%) and the reduction continued gradually until 14% was used. Higher percentages of lime than 14% caused the plasticity index to increase.

CHAPTER VI

Materials Used in the 1957-58 Lime-Soil Investigation at the University of Manitoba

As part of the thesis soil testing program undertaken for this investigation, the following materials were employed:

Dolomitic Hydrated Lime

A 50 pound bag of fresh, pulverized quicklime was obtained from the Stonewall Lime Plant. The material was recrushed further into a fine powder in the laboratory. A representative sample was hydrated and then oven dried. The hydrated lime was then again crushed until 100% passed a No. 20 sieve and a minimum of 85% passed a No. 100 sieve [16].

Dolomitic Quicklime

A representative sample from the 50 pound bag was crushed until the suggested specification fineness was achieved as mentioned above. Care was taken to prevent the quicklime from air-slaking due to the presence of moisture in the air. Both samples of lime were kept in labelled air tight containers and stored in a cool, dark place.

Laboratory Analysis [17]

Loss on Ignition	18.32%
Silica and Insoluble	0.22
Iron and Alumina (Al_2O_3, Fe_2O_3). . .	0.46
Lime (CaO)	48.36
Magnesium Oxide (MgO).	32.63
	99.99

Available Lime Index 45.13%

Probable Composition

Impurities	0.68%
Hydrated Lime ($Ca(OH)_2$).	63.89
Magnesia (MgO)	32.63
By difference, Carbonate mainly CO_2. .	2.79

Fly Ash

This is a waste material from coal burning utility plants and is used mainly in conjunction with lime to form a type of "synthetic" natural (or "Roman") cement. A 50 pound drum of specification fly ash was obtained from The Northern States Power Co., Minneapolis, U.S.A.

Chemical and Physical Characteristics of Fly Ash [18]

	%		%
Silica	40 to 50	Alkalies (Soda & Potash)	3 to 6
Alumina	15 to 24	Sulfur Trioxide	1 to 5
Iron Oxides	12 to 19	Free Carbon	½ to 3
Lime	4 to 8	Moisture less than	½
Magnesia	½ to 2	Loss on Ignition	½ to 5
Titania, less than	1	Reaction with Water - Alkaline	

Weight per cubic foot, 75 lbs.
Passing 325 mesh sieve, 88 to 95%
Specific Surface (Blaine), 2800 to 4500 Sq.Cm. per gram.

Soils Investigated

In all, 25 different soils were investigated, two of which originated from Ontario, the others being from Manitoba, mostly from the Carberry district. Exact analysis and locations of soil samples will be given later.

Laboratory Tests Performed

The following is a list of the soil tests performed. Testing procedures for all the tests, except Harvard

Compaction [21], were taken from "Soil Testing for Engineers",[20] by T. W. Lambe, of the Massachusetts Institute of Technology:

Grain Size - Sieve Analysis
- Hydrometer Analysis

Atterberg Limits - Liquid Limit
- Plastic Limit
- Plasticity Index

Standard Proctor Compaction

Unconfined Compression

Triaxial Compression

Harvard Compaction [21]

CHAPTER VII

Laboratory Results on Soil No. 1

A mixed, 50 pound sample was obtained by combining three samples from approximately the third points along a two mile secondary road, the link between the Trans Canada Highway and the town of Carberry, Manitoba. The individual samples were taken from the top 6 inches of the existing road base and the mixture was considered as representative of the two mile stretch. The samples were taken in this manner merely to first determine whether a lime additive would react with this particular type of soil prior to obtaining additional samples for a more detailed investigation.

Physical Properties of Soil No. 1

(a) The soil was described as an olive-grey silty and clayey sand. Its grain size distribution was determined by a Mechanical and Hydrometer Analysis and is shown below:

		Percent Passing Sieve	
Gravel	5%		
Sand	54	No. 10	95.1%
Silt	28	No. 40	93.6
Clay	13	No.200	44.5

(b) The Atterberg Limits were determined on the passing No. 40 material. The results are listed below:

Liquid Limit	31.2%
Plastic Limit	19.2
Plasticity Index	12.0

(c) Moisture-Density Relationship:

Maximum Dry Density 107.0 lbs per cu.ft.
Optimum Moisture Content 16.0%

An examination of the test results on the natural soil showed that the material contained an excessive percentage of fines, or that it lacked the required granular material. Hence, to be used successfully as a base coarse material, a large percentage of a coarse granular material would be required.

An attempt was made to determine whether this material could be improved or up-graded when additives of dolomitic hydrated lime were used. Since the material contained only 13% clay, fly ash, as a second additive, was also used in conjunction with the lime to determine whether it increased the chemical activity between the clay and the lime.

The first experiment was the determination of the changes in the Atterberg limits when different percentages of lime alone, and lime with fly ash were used.

Mixture	Liquid Limit	Plastic Limit	Plasticity Index
Soil & No Additives	31.2	19.2	12.0
Soil & 1½% Lime	26.0	22.2	3.8
Soil & 3% Lime	25.2	24.1	1.1
Soil & 1½%Lime,3% Fly Ash	27.3	24.0	3.3
Soil & 3%Lime,6% Fly Ash	25.5	25.2	0.3

Table 2. The Variation in the Liquid Limit, the Plastic Limit and the Plasticity Index of Soil No.1 when Treated with Different Percentages of Lime and Fly Ash.

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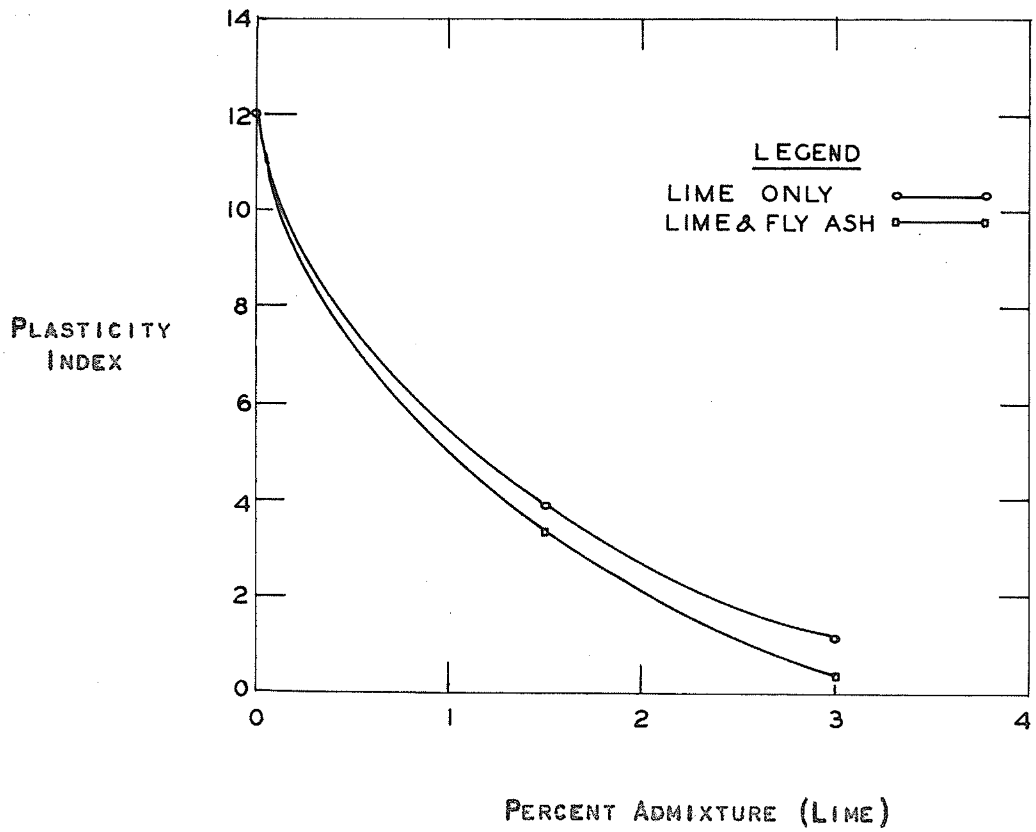

Figure 2. The Variation in the Plasticity Index of Soil No. 1 when Treated with Different Percentages of Lime and Fly Ash.

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The proportions of the additives were expressed as percentages of the total mixture, all materials being in an oven dry condition. In the case of the lime and fly ash admixtures, the optimum of 1:2 by weight [19] was used. According to numerous tests performed at Iowa State College, this proportion gave the highest strengths for most soils.

Since we were dealing with a fairly sandy material, a reduction of the plasticity index was not of too great significance, however, it is interesting to note the sharp reduction in plasticity when a percentage as low as $1\frac{1}{2}$ of lime alone was used. This tended to illustrate the theory of base exchange, as mentioned in Chapter IV. A marked change was noted when the tests were being run. When additives of lime were used, the soil-lime-water paste appeared less tacky and sticky at high moisture contents. The wet mix was observed to be more friable and workable and no longer had the plastic feel to it which is characteristic of an appreciable clay content.

<u>Test Results on Compressive Strength of Soil No. 1 with Lime Additives.</u>

The moisture density relationships were first determined for the natural soil. Although it was known that the maximum dry density and the optimum moisture content of the lime-soil mixtures would vary slightly when different percentages of additives were used in the compacted sample, all of the samples were compacted at the optimum moisture content of the natural soil and to Standard Proctor Density. From previous test results [22,23], where small percentages of lime were used, the optimum moisture content generally increased slightly for almost all types of soils, whereas the maximum dry density usually increased slightly for a fine grained soil but decreased for a coarse grained soil.

However, since only a limited amount of sample was available, and the changes in the moisture density relationships were not too substantial, all of the different mixtures were compacted to Standard Proctor Density and at the optimum moisture content obtained from the natural soil compaction test. For more precise work, the moisture-density relationships should be determined for each mix prior to compaction, to determine the correct optimum moisture content.

Mixing of Materials

The proper proportions of soil, lime and fly-ash were weighed out and then the constituents were dry mixed until the lime appeared to be thoroughly dispersed throughout the soil. With due allowances made for the original moisture content of the soil, the optimum amount of moisture, based on the Standard Proctor Compaction test with no additives, was added to the dry mix. Each mixture was then compacted to Standard Proctor Density in the conventional mold, which was 4.59 inches high and 4 inches in diameter, having a volume of 1/30 of a cubic foot. Since the Harvard Compaction Devide was not available at the time, compression test samples were obtained by pushing 4 thin walled tubes into the Proctor mold; this gave samples 1.375 inches in diameter and approximately 4 inches high. The ends of the samples were then trimmed to a height of 2.75 inches [20] and then capped with a sulphur compound to give the sample ends uniform bearing surfaces.

Some of the samples were tested immediately after molding, in the unconfined compression machine, while others were cured at room temperature (72°F), and near 100% relative humidity, for periods of 7, 14, and 28 days prior to being tested.

Mixture	Curing Time in Days											
	0			7			14			28		
	Compressive Strength (p.s.i.)	Dry Density (lbs/cu.ft.)	Moisture Content (%)	Compressive Strength (p.s.i.)	Dry Density (lbs./cu.ft.)	Moisture Content (%)	Compressive Strength (p.s.i.)	Dry Density (lbs./cu.ft.)	Moisture content (%)	Compressive Strength (p.s.i.)	Dry Density (p.s.i.)	Moisture Content (%)
Soil + No Additives	47.6	110.2	15.4	45.2	120.4	14.2	33.3	116.8	15.3	-	-	-
Soil + 1-1/2% Lime	68.1	112.2	15.9	146.8	116.0	14.0	151.0	118.0	14.4	-	-	-
Soil + 3% Lime	70.4	111.0	14.6	183.5	113.1	13.4	253.0	116.1	12.7	254.0	118.8	15.4
Soil + 1-1/2% Lime 3% Fly Ash	57.4	114.0	15.8	131.7	113.8	14.3	143.1	117.4	14.0	-	-	-
Soil + 3% Lime 6% Fly Ash	65.7	110.9	15.8	183.5	115.2	14.4	200.0	113.1	15.1	230.0	119.1	13.5

Table 3. Relationship Between Length of Curing and Compressive Strength of Soil No. 1 with Different Percentages of Lime and Fly Ash.

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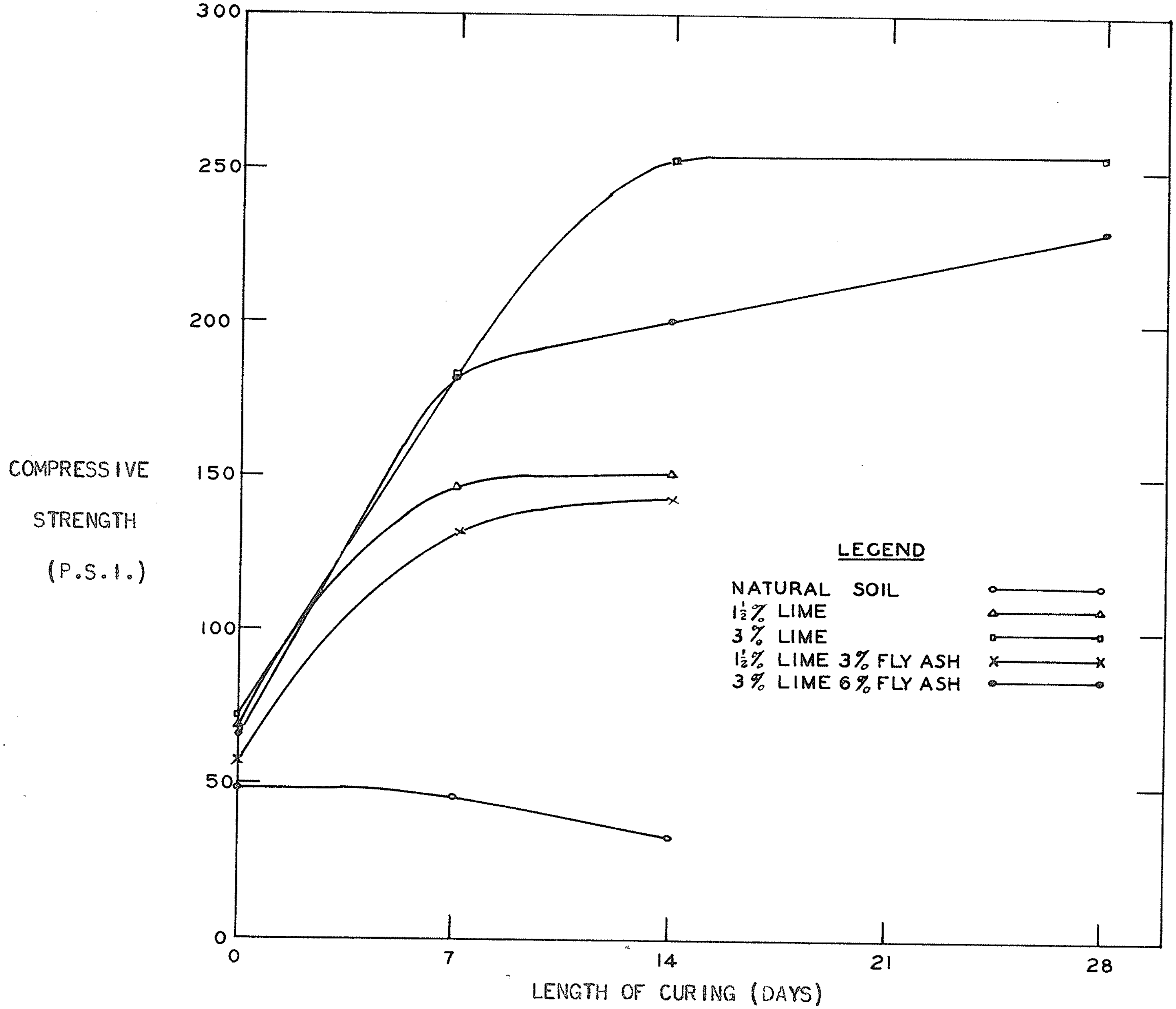

Figure 3. Relationship Between Length of Curing and Compressive Strength of Soil No. 1 with Different Percentages of Lime and Fly Ash.

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Discussion of Test Results

An examination of Table 3 and Figure 3 will indicate what effect the additives of lime and fly ash had on the compressive strengths of the soil.

In general, immediately after mixing and compaction, there was little increase in the strength of the treated soil. However, after 7 days curing, an addition of 1½% lime without fly ash gave a strength of almost 3 times the original. Using 3% lime, the strength was increased over 3½ times.

After 14 days curing, it appeared that the treated soils, using lime only, had attained almost their maximum strength. With 1½% lime only, the strength attained was approximately 150 p.s.i., over 3 times the original strength of the untreated soil. With a 3% admixture of lime only, the lime-soil sample had reached a strength of approximately 250 p.s.i., over 5 times the original strength of the natural soil.

During the 28 day curing period, the use of fly ash, which acted as an artificial pozzolan, showed lower strengths than that of the lime alone. However, judging by the relative slopes of the strength curves for lime only and lime-fly ash admixtures, it appeared that the use of this additional additive might have given higher strengths for curing periods longer than 28 days. The curves appeared to show that the fly ash tended to retard the initial strength gain, but that ultimately, after a longer period, the strength might have been greater than for the lime alone.

It was decided, however, not to evaluate the effects of a longer curing period, since in the case of this particular soil, relatively rapid strength gain was necessary for practical reasons. Rapid strength gain would generally be a governing condition to reduce traffic tie-ups on a newly constructed road.

Deflection Comparisons

Under the unconfined compression test, the natural soil experienced an average shortening of 0.27 inches at failure, for a sample 2.75 inches long and 1.375 inches in diameter. With 3% lime, the compressibility or the deflection, was greatly reduced. Even immediately after compaction, without any curing period, the deflection was as low as .09 inches for the same size of sample. With 28 days curing, the compression of the sample was reduced to .058 inches.

Types of Failures

The types of failures experienced on the unconfined compression test were also of particular interest. Due to the cementing characteristics of these additives, as previously disclosed, the treated samples experienced very little deflection. The treated samples did not show the plastic flow or bulging type of failure which was characteristic of the untreated samples. The failures at ultimate load were very sudden, and in almost all cases, very similar to the cone type of failure experienced in the testing of concrete cylinders. Shear failures occurred on planes inclined more than 60 degrees to the plane of maximum principal stress.

There appeared to be no crumbling of the treated samples, but a very definite shear plane type of failure, which was indicative of a high shear strength.

Sufficient material was not available to make up additional samples to carry these tests further, however, the results warranted further study of this soil type.

An examination of the calculated dry densities showed that the Proctor Compaction method of obtaining compression test specimens did not give uniform results. More consistency would probably have been obtained in the use of some device which would mold smaller samples which could be used directly in the unconfined compression test machine (e.g. Miniature Harvard Compaction Device).

Results of Detailed Soil Investigation

Having obtained fairly satisfactory results with the preliminary mixed sample (Soil No. 1), a detailed soil investigation of the existing road base and shoulders, for the access road into the town of Carberry, was made.

Soil samples, submitted by the Highways Branch, of the Province of Manitoba, were taken from the top 6 inches, both from the existing roadway and from the shoulders as well. The samples, numbering 18 in all, were taken at locations spaced approximately 0.2 miles apart. Two samples were taken at each location, one on the road, the other on the shoulder.

The samples were tested for Grain Size and the Atterberg Limits by standard accepted procedures [20]. The existing road material was classified according to the Public Roads Classification, 1945, Highway Research Board Modification, which groups the soils according to the particle size and plasticity index [24].

The results of this investigation are listed in Table 4, showing the mechanical analysis, the Atterberg Limits, the percentage passing the No. 10, 40, and 200 sieves, the class and group index, the color and texture.

Lime Proportioning

An examination of the test results showed that the soil varied somewhat along the two mile access road, which meant that this road had to be treated in sections.

In general, however, the soil from the existing road base was a fine-grained material, predominantly a silt and a sand.

The road material for the first mile from the Trans-Canada Highway was found to be mainly a silt; the second mile, mainly a sand. The shoulder material showed very little variation along the two mile stretch.

Distance From T.C.H. (Miles)	Mechanical Analysis (%) Gravel	Sand	Silt	Clay	Atterberg Limits Liquid Limit	Plastic Limit	Plasticity Index	Percent Passing No. 10	No. 40	No. 200	Class & Group Index	Color	Texture
Road Material													
0.2 R	11	15	48	26	42.6	18.9	23.7	89.3	86.2	81.7	A-7-6 (14)	Light Grey Brown	Clayey Silt, with Sand & Gravel
0.4 R	7	31	43	19	32.2	17.2	15.0	92.0	86.8	69.2	A-6 (8.8)	Dark Grey Brown	Sandy Silt, with Clay & Gravel
0.6 R	12	24	43	21	34.0	18.2	15.8	87.8	85.7	73.2	A-6 (9.9)	Grey Brown	Sandy Silt, with Clay & Gravel
0.8 R	2	16	52	30	44.8	19.2	25.6	97.7	95.4	86.7	A-7-6 (15.2)	Grey Brown	Clayey Silt, with Sand & Gravel
1.0 R	29	33	28	10	29.7	18.2	11.5	70.6	61.0	44.1	A-6 (2.2)	Grey Brown	Gravelly Sand, with Silt & Clay
1.2 R	24	39	27	10	28.7	18.2	10.5	75.2	63.3	41.7	A-6(1.4)	Dark Grey Brown	Silty Sand, with Gravel & Clay
1.5 R	22	57	16	5	24.8	17.2	7.6	78.0	52.4	29.3	A-2-4- (0)	Grey Brown	Gravelly Sand, with Silt & Clay.
1.7 R	6	40	38	16	31.2	16.5	14.7	93.9	88.9	67.6	A-6 (8.4)	Grey Brown	Silty Sand, with Clay & Gravel
2.0 R	14	61	20	5	21.7	17.0	4.7	86.2	81.9	37.3	A-4 (0.6)	Grey Brown	Silty Sand with Gravel & Clay.
Shoulder Material													
0.2 S	0	18	59	23	46.5	18.3	28.2	99.6	97.7	92.1	A-7-6-(16.6)	Grey Brown	Clayey Silt, with Sand
0.4 S	1	49	39	11	25.6	15.7	9.9	98.4	96.8	63.7	A-4 (5.7)	Grey Brown	Silty Sand with Clay & Gravel
0.6 S	0	31	49	20	38.0	18.2	19.8	99.4	98.1	83.5	A-6 (11.9)	Grey Brown	Sand Silt, with Clay
0.8 S	0	24	51	25	36.5	17.4	19.1	100.0	99.8	82.4	A-6 (11.6)	Light Grey Brown	Clayey Silt, with Sand
1.0 S	4	30	46	20	34.7	18.6	16.1	95.8	94.7	77.6	A-6 (10.4)	Dark Grey Brown	Sandy Silt, with Clay & Gravel
1.2 S	0	22	54	24	38.5	19.0	19.5	99.8	99.5	86.1	A-6 (11.8)	Dark Grey Brown	Clayey Silt, with Sand
1.5 S	0	42	40	18	26.7	16.9	9.8	100.0	99.0	69.0	A-4 (6.8)	Dark Grey Brown	Silty Sand, with Clay
1.7 S	0	37	42	21	20.0	16.3	19.7	99.6	98.8	73.1	A-6 (11.5)	Dark Grey Brown	Sand Silty, with Clay
2.0 S	0	64	23	13	23.8	17.2	6.6	99.9	98.8	45.3	A-4 (2.1)	Dark Grey Brown	Silty Sand, with Clay
Preliminary Mixed Sample													
No. 1	5	54	28	13	31.2	19.2	12.0	95.1	93.6	44.5	A-6 (2.5)	Dark Grey Brown	Silty Sand with Clay & Gravel

Table 4. Physical Characteristics, Class, Group Index, Color and Texture of the Existing, Top 6 inches, of the Carberry Access Road, from the Trans Canada Highway to the Town of Carberry, Manitoba.

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An attempt was then made to determine how the existing surface of this two mile test strip could be stabilized with lime. The design was made for the stabilization of the top 6 inches [3] of the existing base with dolomitic hydrated lime. The results of the laboratory tests on soil No. 1 illustrated that fairly successful results could be obtained with the use of lime alone, since it appeared that the fly ash additive would retard initial strength growth. (See figure 3). Also, a fairly large amount of fly ash would be required if the recommended ratio of 1:2 was used for the lime-fly ash design. Since very little additional benefits were achieved by this second additive its use was considered not warranted.

According to published literature [25], the recommended percentages of lime to employ when dealing with coarse grained soils, which contain up to 50% clays and silts, are 2, 3, and 5% by weight. For fine-grained soils, containing more than 50% of combined silt and clay, 5, 7 and 10% are recommended.

The proportioning of the lime additives was based upon the above rules for the road material, however, lower percentages were specified for the shoulder material. Since this was a tentative design for a test strip, it appeared advantageous to use a wide range of percentages of lime by the installation of a number of different sections.

Table 5 summarizes the soil types in each half mile section and lists the recommended percentages of lime additives. The percentages of hydrated lime ($Ca(OH)_2$) are expressed as the percentage of the total mix of soil and lime, both in the dry state.

Section (Miles) From T.C.H.	Average % of Silt and Clay	Average Plasticity Index	Average % Passing 200	Recommended % Lime
ROAD				
0 - ½	68	20	75	6%
½ - 1	61	18	68	5%
1 - 1½	32	10	40	3%
1½ - 2	34	9	45	3½%
SHOULDERS				
0 - ½	66	20^{+}	78	3½%
½ - 1	70	19	81	4%
1 - 1½	67	15	78	3%
1½ - 2	53	12	63	2½%

Table 5. Summary of the Average Percentage of Silt and Clay, the Average Plasticity Index, the Average Percentage Passing a No. 200 Sieve and the Recommended Percentage of Lime for Half Mile Sections of the Carberry Road.

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The preliminary sample tested was fairly representative, however, it contained slightly more sand and less gravel than most of the later samples. The clay content of the preliminary sample was quite low, when compared with the test results on the first mile.

Table 6 illustrates the effect of these recommended percentages of lime on the Atterberg Limits, and the Class and Group Index [24] of the Carberry soils as determined by tests.

According to the modified Public Road Classification [24], the inorganic soils are classified in 7 groups, corresponding to A-1 through A-7. The classification A-1 is given to the soil best suited for highway subgrades. This is a well-graded material, composed largely of sand and gravel, but containing a small amount of necessary clay binder. All other soils are grouped roughly in decreasing order of stability down to A-7. These, in turn, are divided into a total of 12 subgroups. In addition, any soil containing appreciable fine-grained material is more completely identified by the determination of the group index. The group index is obtained from a formula which takes into consideration the per-cent passing the No. 200 sieve, the liquid limit, and the plasticity index of a fine grained soil. The formula will give values ranging from a fraction of 1 to 20, and is so weighted that a maximum influence of each of the three variables mentioned above is in the ratio of 8 for the per cent passing the No. 200 sieve, 4 for the liquid limit, and 8 for the plasticity index. Hence, it follows that a group index of zero indicates a "good" subgrade material and a group index of 20 indicates a "very poor" subgrade material.

An examination of Table 6 will illustrate how the class and group index were altered when small percentages of lime were added to a fine-grained soil. For example, in the second half mile section from the Trans Canada Highway (sample No. 0.8 R), the initial class and group

Section (Miles) From T.C.H. (Road Material)	Recommended % Lime	Samples in Section	Atterberg Limits No Lime			Atterberg Limits Recommended % Lime			Class and Group Index		
			Liquid Limit	Plastic Limit	Plasticity Index	Liquid Limit	Plastic Limit	Plasticity Index	No Lime	Recommended % Lime	Trans-formation
0 - 1/2	6	0.2 R	42.6	18.9	23.7	41.1	31.0	10.1	A-7-6(14)	A-5(8.3)	Clay to Silt
		0.4 R	32.2	17.2	15.0	32.5	27.4	5.1	A-6(8.8)	A-4(6.8)	Clay to Silt
1/2 - 1	5	0.6 R	34.0	18.2	15.8	34.2	28.0	6.2	A-6(9.9)	A-4(7.6)	Clay to Silt
		0.8 R	44.8	19.2	25.6	40.3	30.7	9.6	A-7-6(15.2)	A-4(8.1)	Clay to Silt
		1.0 R	29.7	18.2	11.5	30.2	26.4	3.8	A-6(2.2)	A-4(1.8)	Clay to Silt
1 - 1-1/2	3	1.0 R	29.7	18.2	11.5	32.7	26.6	6.1	A-6(2.2)	A-4(1.8)	Clay to Silt
		1.2 R	28.7	18.2	10.5	28.0	25.6	2.4	A-6(1.4)	A-4(1.3)	Clay to Silt
		1.5 R	24.8	17.2	7.6	23.0	19.8	3.2	A-2-4(0)	A-2-4-(0)	No Change
1-1/2 - 2	3-1/2	1.5 R	24.8	17.2	7.6			0	A-2-4(0)	A-2-4(0)	No Change
		1.7 R	31.2	16.5	14.7	28.8	24.6	4.2	A-6(8.4)	A-4(6.5)	Clay to Silt
		2.0 R	21.7	17.0	4.7	-	-	0	A-4(0.6)	A-4(0.6)	No Change

Table 6. The Effect of Lime Additives on the Atterberg Limits, Class and Group Index and the Transformations that Occurred.

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index was changed from A-7-6(15.2) to A-4 (8.1). The significant constituent of an A-7 type is clay, whereas, for the A-4 type, the soil is mainly a silt with a low percentage of sand. Although this method of evaluating the effectiveness of lime additives on fine-grained soils has not been widely used in lime stabilized subgrade designs, it can prove to be a reliable method of proportioning lime admixtures to fine-grained soils.

Due to the design of the class and group index classification, it should be noted that no transformation was experienced by the use of lime on the coarse-grained material. Hence, it follows that other methods of evaluation, such as triaxial or unconfined compression investigations must be made to determine the merits achieved by the use of lime.

CHAPTER VIII

Laboratory Results with Soil No. 2

A 25 pound sample was obtained from the top 6 inches of a quarry road at the Steep Rock Iron Mine in Ontario. The material was a waste product of the crushing operation and was used as fill material for road construction in the vicinity of the quarries and the refinery.

The in-place material was relatively loose and resisted compaction due to the absence of a suitable binding agent. The sample taken had fairly good gradation, with approximately 50% retained on the No. 20 sieve and a maximum size of approximately one-half inch. The material, however, was very low in cohesion and would require some type of binding agent to produce a satisfactory base course. Samples of this material were therefore obtained and tested to determine whether additives of lime or lime with fly ash could be used to convert this waste product into a useful road building material. In this investigation the triaxial compression test was utilized. In this test, prepared specimens of the material with and without additives were tested in compression, under a lateral confinement, to determine the effects of the additives on its stability.

All of the test specimens were made from the material finer than the No. 40 sieve. The samples were each compacted in a small cylinder, $1\frac{3}{8}$ inches in diameter and 4 inches high, in three layers; each layer received 25 blows with a 0.8 pound hammer falling through a height of 6 inches. This method of compaction gave densities very close to that of Standard Proctor. The moisture content was selected from typical test results of an investigation [19] at Iowa State College, where studies were made on lime admixtures with coarse-grained soils.

Immediately after compaction the samples were placed in a humidifier and cured for a period of 12 days. The humidifier maintained a constant temperature of 80°F and a constant relative humidity of 100%. Table 7 lists the results of these tests after 12 days of curing.

Mixture	Natural Soil	3% Hydrated Lime	3% Hydrated Lime 6% Fly Ash
Cohesion (p.s.i.)	4.0	13.0	16.0
Angle of Internal Friction (o)	19°	14°	13.5°
Wet Density ($\#/ft^3$)	133.8) 133.0) 133.4	122.4) 119.8) 121.1	120.8) 123.8) 122.3
Dry Density ($\#/ft^3$)	117.6) 117.4) 117.5	108.2) 105.5) 106.8	106.3) 108.7) 107.5
Moisture Content (%)	13.8) 13.3) 13.6	13.1) 13.3) 13.2	13.6) 13.8) 13.7

Table 7. The Effect of Lime, and Lime with Fly Ash, on the Cohesion, the Angle of Internal Friction and the Densities of a Coarse-Grained Material (Soil No.2, Steep Rock Mine).

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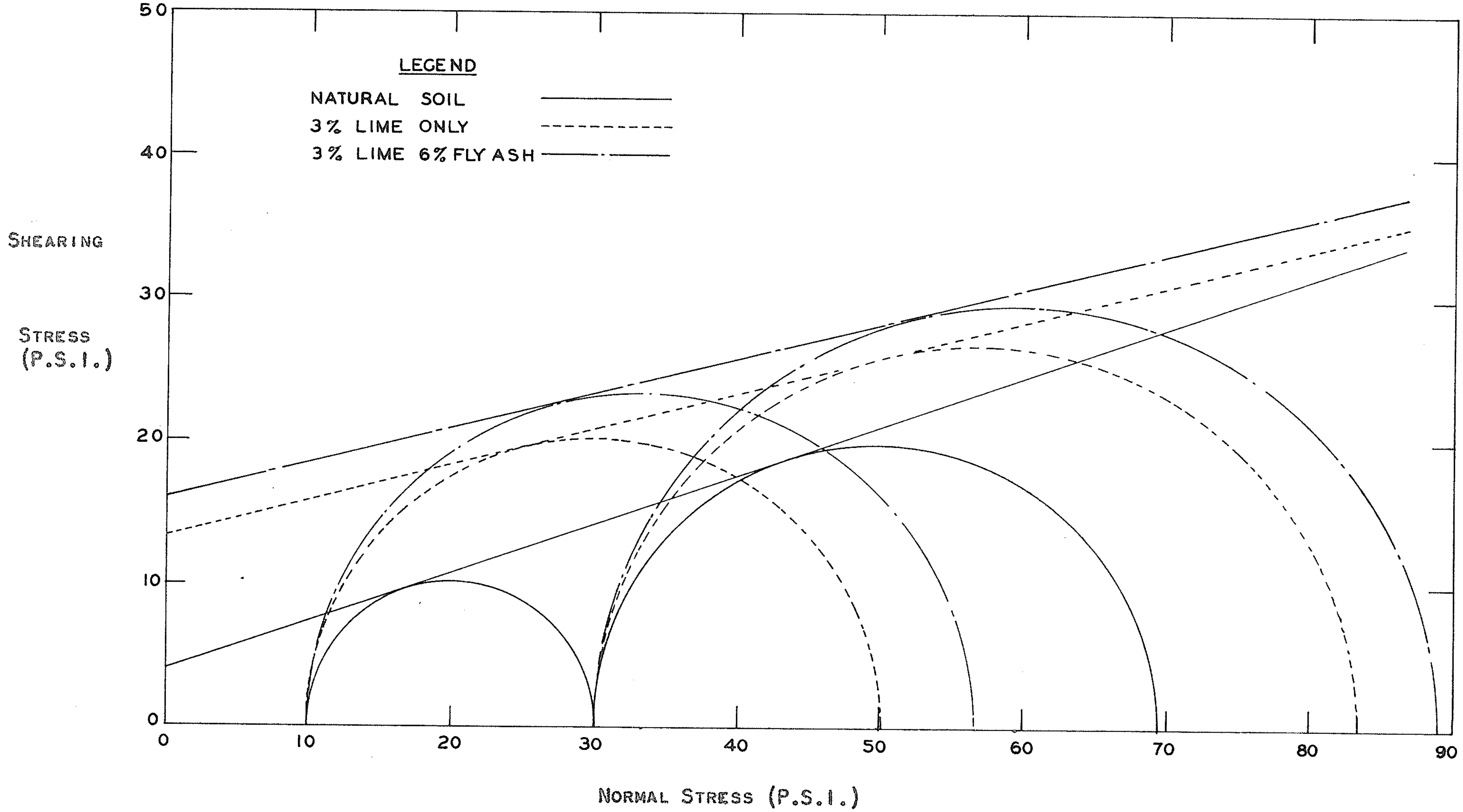

Figure 4. The Effect of Lime, and Lime with Fly Ash, on the Cohesion and the Angle of Internal Friction of a Coarse-Grained Material (Soil No. 2, Steep Rock Mines.)

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An examination of the data obtained (Table 7 and Figure 4), illustrated the effect of lime, and lime with fly ash, on the samples tested. With 3 per cent hydrated lime, there was a marked increase in cohesion from 4.0 p.s.i. to 13.0 p.s.i., illustrating the bonding effect of the admixture. It was also noted that the angle of internal friction of the soil dropped slightly. The use of the additional admixture, fly ash (1:2 by weight) [19], gave even a higher value of cohesion; however, the additional reduction of the angle of internal friction was only a half degree.

CHAPTER IX

Laboratory Results on Soil No. 3

A 10 pound sample was obtained from a tailings stockpile at Red Lake, Ontario. This material was a waste product derived from the gold extraction process in that region. The material, at present, is being disposed of into lakes and depressions.

This material could be described as a coarse angular sand (approximately 80% passed the No.20 sieve), and would have little stability due to a lack of cohesion between particles. An investigation was undertaken to determine whether additives of lime, and lime with fly ash, had any effect on improving this waste product so that it could be converted into a useful road material.

The same method of preparation of the test specimens was used as outlined in Chapter VIII. The samples were cured under the same conditions of temperature and humidity, as for Soil No. 2, for a period of 7 days. Table 8 outlines the results of this investigation.

The results obtained with Soil No. 3 were much the same as those for the previous test. As before there was an increase in the cohesion and a decrease in the angle of internal friction. With this soil, however, the action of the second additive, fly ash, was much more pronounced, possibly due to an absence of natural pozzalons in the soil, as compared to a clay gumbo which is high in pozzalonic material.

Mixture	Natural Soil	3% Hydrated Lime	3% Hydrated Lime 6% Fly Ash
Cohesion (p.s.i.)	5.0	9.0	16.0
Angle of Internal Friction (°)	33°	31°	23°
Wet Density (#/ft^3)	117.2) 119.0)117.9 117.5)	112.6) 114.0)113.3 113.2)	107.2) 107.8)107.5 107.5)
Dry Density (#/ft^3)	103.0) 104.5)103.5 103.1)	100.0) 100.5)100.1 99.9)	94.5) 95.0) 94.7 94.5)
Moisture Content (%)	14.0) 13.9) 14.0 14.1)	12.8) 13.3) 13.2 13.4)	13.5) 13.4) 13.6 13.9)

Table 8. The Effect of Lime, and Lime with Fly Ash, on the Cohesion, the Angle of Internal Friction and the Densities of a Coarse-Grained Material (Soil No.3, Red Lake Tailings).

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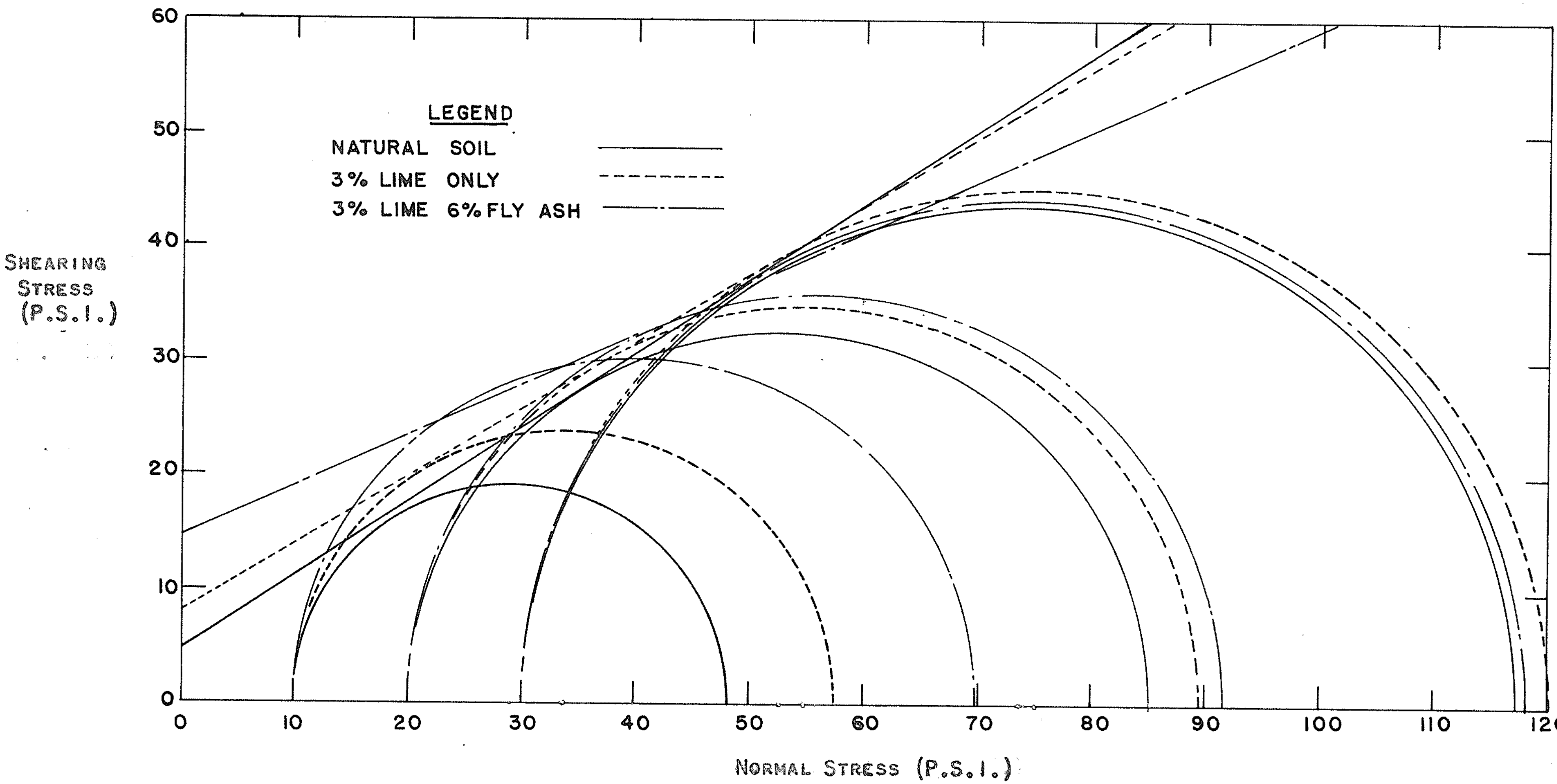

Figure 5. The Effect of Lime, and Lime with Fly Ash, on the Cohesion, and the Angle of Internal Friction of a Coarse-Grained Material. (Soil No. 3, Red Lake Tailings).

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The results obtained with Soil No. 3 were much the same as those for the previous test. As before there was an increase in the cohesion and a decrease in the angle of internal friction. With this soil, however, the action of the second additive, fly ash, was much more pronounced, possibly due to an absence of natural pozzalons in the soil, as compared to a clay gumbo which is high in pozzalonic material.

CHAPTER X

Laboratory Results on Soil No. 4

A representative 50 pound sample was obtained from the St. Paul's College excavation, on the University of Manitoba campus grounds, at a depth of 2 to 6 feet.

Physical Properties of Soil No. 4

(a) The soil was described as a fine-grained, heavy clay gumbo, very common in the Red River Valley, usually found at depths of approximately 2 to 6 feet.

(b) Mechanical Analysis - 99.7% passed a No. 200 sieve.

(c) Hydrometer Analysis -

	Sand	1%
	Silt	11%
	Clay	88%

(d) Field Moisture Content - 35.0%

(e) Atterbert Limits -

	Liquid Limit	100.5%
	Plastic Limit	35.7%
	Plasticity Index	64.8%

As a road builders material, clay gumbo is limited to subbase construction where it is not subjected to high bearing stresses. It is an objectionable material in the base course due to its high capillarity and plasticity properties. Clays have a tendency to hold large percentages of moisture and considerable difficulty is encountered when handling the material if it is used as a binder for granular material. Even at lower moisture contents pulverization is almost impossible, hence difficulties arise in dispersing a small amount of clay binder in a cohesionless granular material.

An investigation was made to determine how the plasticity characteristics were altered when additives of lime (hydrated) and lime with fly ash were mixed in with the soil. The optimum ratio [19] of lime to fly ash of 1:2 was used when the second additive was incorporated. Table 9 lists the results of this investigation.

Figure 6 illustrates how the plasticity index of the material was substantially reduced with 5 per cent of lime alone and lime with fly ash. In all the tests performed with lime alone, the reduction of the plasticity index was due to a combined decrease of the liquid limit and an increase of the plastic limit. At 5 per cent lime, the plasticity index had been reduced from 65 down to about 18.

The use of fly ash gave even better results in the reduction of the soils plasticity properties. An examination of the data showed that for 1 percent lime and 2 per cent fly ash, both the liquid limit and the plastic limit decreased, however, the net result was a reduction of the plasticity index. An examination of the slopes of the curves at various points showed that the effectiveness of lime was the greatest at small percentages, for example at 1 per cent. At 5 per cent, although, the plasticity index was still dropping , the rate of reduction had decreased. This illustrated that a simple test of this nature can provide a criterion for selecting a most economical or optimum percentage of lime.

Mixture	Liquid Limit	Plastic Limit	Plasticity Index
Soil + No Additives	100.5	35.7	64.8
Soil + 1% Lime	87,0	37.7	49.3
Soil + 3% Lime	71.0 65.5	38.4 39.5	32.6) 26.0) 29.3*
Soil + 5% Lime	63.0 63.5	44.3 46.8	18.7) 16.7) 17.7*
Soil + 1% Lime 2% Fly Ash	78.5	32.5	46.0
Soil + 3% Lime 6% Fly Ash	69.0 72.0	46.3 48.3	22.7) 23.7) 23.2*
Soil + 5% Lime 10% Fly Ash	58.5 59.0	44.9 46.0	13.6) 13.0) 13.3*

Table 9. The Variation in the Liquid Limit, the Plastic Limit, and the Plasticity Index, of Soil No. 4, when Treated with Different Percentages of Lime and Fly Ash.

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* Four of the tests were duplicated to determine the accuracy of the results.

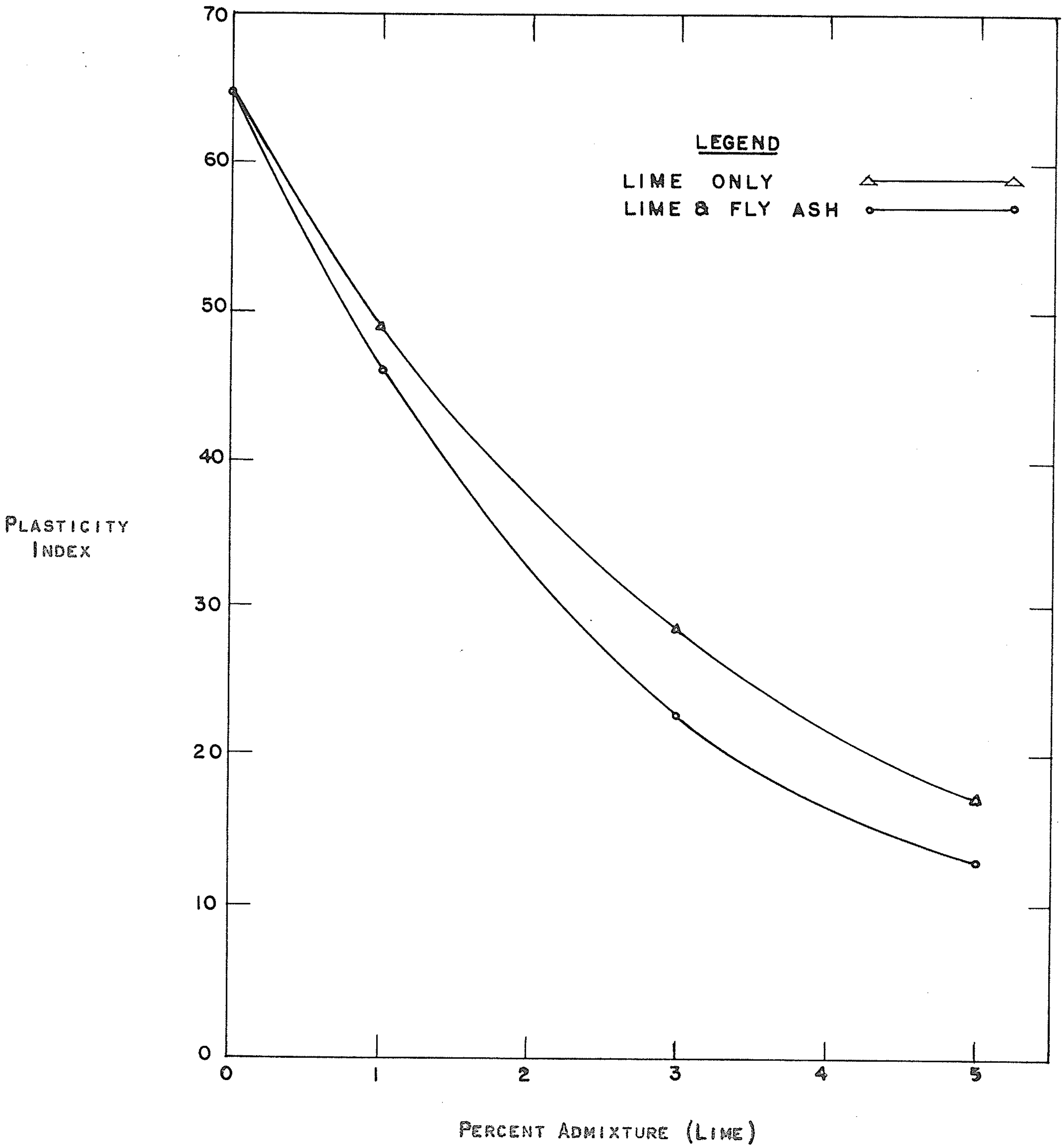

Figure 6. The Variation in the Plasticity Index of Soil No. 4, when Treated with Different Percentages of Lime, and Lime with Fly Ash.

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CHAPTER XI

Strength-Time Relationships of Three Manitoba Soils with Additives of Dolomitic Quicklime.

This chapter will illustrate a joint investigation [26] carried out at the University of Manitoba, Civil Engineering Department, 1958, in conjunction with six undergraduate students, who submitted theses on the topic "Lime Stabilization".

Laboratory Results on Soil No. 5

A representative 50 pound sample was taken from the top 6 inches of the existing road surface, a half of a mile south of the Trans Canada Highway, on the Carberry access road. The material was described as a silty sand, with traces of clay and gravel.

Before test specimens were prepared with the Miniature Harvard Compaction Device [21], this apparatus was calibrated with the Standard Proctor Method for each soil tested. The Harvard [21] compaction apparatus molds specimens 1 5/16 inches in diameter and 2.816 inches high. Densification of the soil in the mold is achieved by a tamper; the load transferred by the tamper is controlled by a compression spring. Hence, by fixing the number of layers of material in the mold, and the spring load of the tamper, it is possible to achieve any desired soil density in the range of Standard and Modified Proctor. This is done by determining the number of blows required per layer to give the desired density for the soil under consideration.

The Harvard Compaction Method of forming specimens was used since it had several advantages over the Proctor Method used earlier in this investigation:

(a) The time and effort in forming test specimens was greatly reduced.

(b) Lesser quantities of materials were required to form test specimens.

(c) The ratio of height to diameter was greater than 2:1, hence the compacted specimens were suitable for either unconfined or triaxial compression tests without further trimming.

(d) There was less disturbance of the specimens when they were removed from the mold.

(e) Calculations are greatly simplified since the wet weight of the soil specimen in grams gave the wet unit weight in pounds per cubic foot directly.

(f) Research [27] has shown that by the proper selection of spring load, the number of layers, and the number of blows per layer, it was possible to duplicate more closely field compaction with a laboratory kneading type of compactor than with either a dynamic or static type.

The following results were obtained using the Standard Proctor Method of compaction for Soil No. 5:

Maximum Dry Density - - - - - - - - - - 107.0 $\#/ft^3$
Optimum Moisture Content - - - - - - - 16.5%

Using the above optimum moisture content, 5 layers in the Harvard mold, and a 25 pound tamping force, the number of blows per layer was varied until the desired dry density of 107.0 lbs per cu. ft. was obtained. It was found, that for this type of soil, a silty sand, 33 blows per layer were required.

Specimens containing the natural soil without additives, and others with 5 per cent of dolomitic quick-lime, were then compacted using the Harvard method. All of the samples were then wrapped in foil and waxed. They were then placed in the humidifier and cured at room temperature (72°F) and near 100% relative humidity. Some of the samples were tested immediately after compaction, while others were left to cure for periods of 4, 7, 14, 21, and 28 days. After 10 days of sealed curing, the protective wax seal was removed from the soil-lime samples and they were exposed to the curing atmosphere for the remaining 18 days. This was done to determine the effect of sealed curing on the early strength gain characteristics of soil-lime specimens.

An examination of Tables 10 and 11, and Figure 7, illustrated the effect of 5 percent dolomitic quicklime on the strength of Soil No. 5. The natural soil without additives had an average strength of approximately 90 p.s.i. during the curing period of 28 days. For the sealed specimens, with 5 percent lime, the initial strength gain was fairly slow, and at 21 days the soil appeared to have reached its ultimate strength. The moist cured specimens, containing the same percentage of lime, were opened after 10 days of sealed curing and were then subjected to moist curing for the remaining 18 days. During the early stages of moist curing, the specimens showed a very rapid strength growth. At 18 days, the strength was 250 p.s.i. and judging by the slope of the strength-time curve at the end of 18 days of moist curing, the specimens had not as yet reached their ultimate strength. This investigation proved the recommendations of published literature on the correct methods of curing lime stabilized bases. Hence, it is imperative that any soil-lime compacted base receive adequate moisture during the early stages of curing.

Sample Number	Curing Time (Days)	Strength (p.s.i.)	Average (p.s.i.)
26	0	79.5	
2	0	88.3	86.5
3	0	91.6	
14	4	73.3	
12	4	81.6	78.3
22	4	80.1	
24	7	101.2	
8	7	99.2	101.2
25	7	106.2	
4	14	100.0	
20	14	98.5	105.2
11	14	117.1	
9	21	101.2	
19	21	90.4	97.3
23	21	100.3	
7	28	99.0	
18	28	104.1	100.1
17	28	97.2	

Table 10. Compressive Strengths in Pounds per Square Inch, of a Silty Sand (Soil No. 5, Carberry), without Additives after Various Curing Periods.

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Sample Number	Curing Time (Days)	Strength (p.s.i.)	Average (p.s.i.)
A	0	73.4	
B	0	67.7	67.0
D	0	59.8	
Q	3	84.2	
X	3	68.3	75.8
H	3	74.8	
L	7	101.0	
M	7	94.4	100.5
K	7	106.0	
J	14	125.4	125.4
R*	14	95.4	112.3
T*	14	129.2	
E	21	217.0	202.3
S	21	187.6	
F*	21	153.0	153.0
O*	21	152.6	
N*	28	258.0	252.0
U*	28	246.0	
W	28	201.0	201.0

Table 11. Compressive Strengths in Pounds per Square Inch, of a Silty Sand (Soil No.5, Carberry), when Treated with 5% of Dolomitic Quicklime after Various Curing Periods.

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* These samples were opened and cured in the humidifier after 10 days of sealed curing. The remainder of the samples were seal-cured for 28 days.

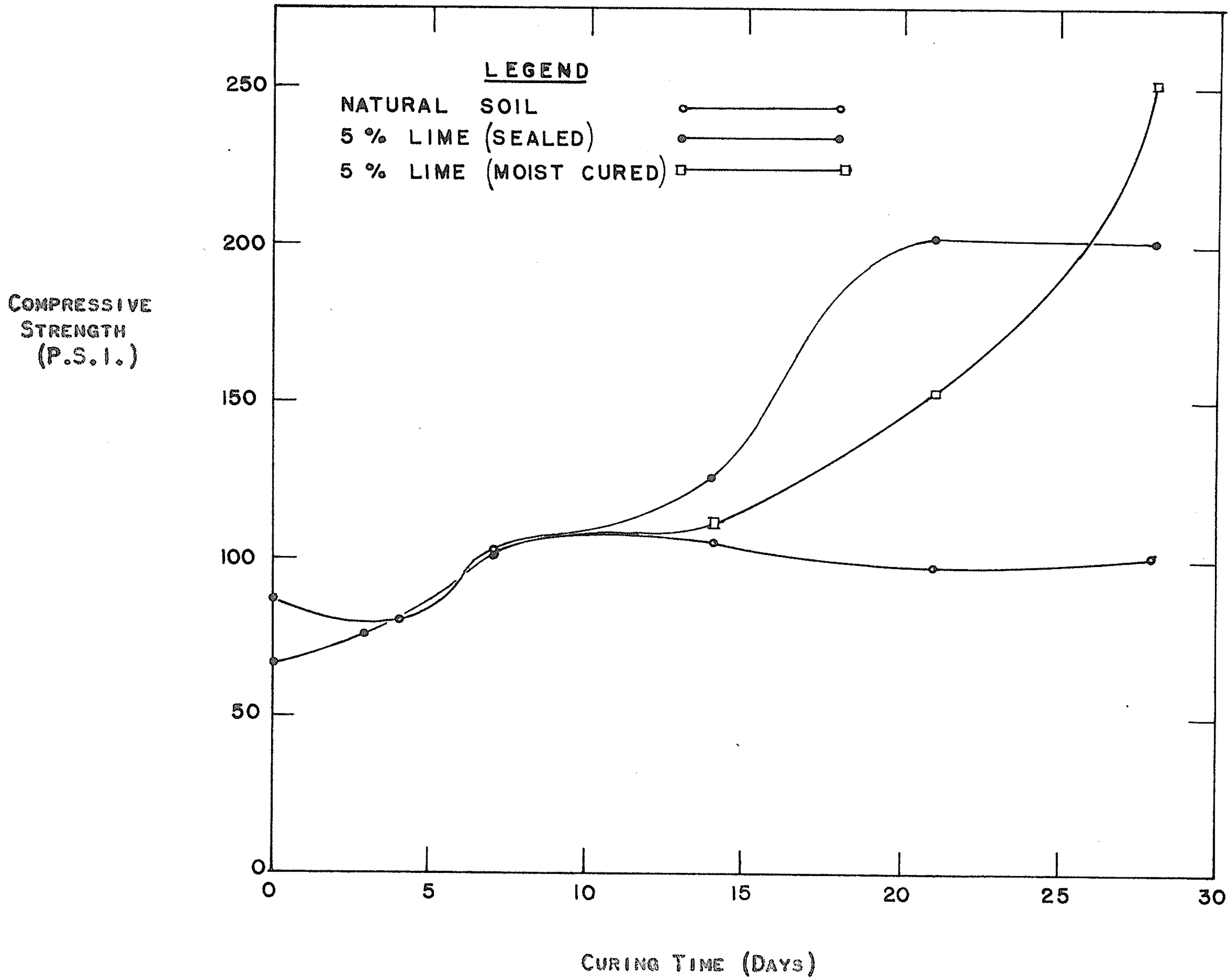

Figure 7. Strength-Time Relationship of a Silty Sand (Soil No.5 Carberry), when Treated with 5 Percent Dolomitic Quicklime.

C.E. Department U. of M. 1958.

Laboratory Results on Soil No. 6

A representative 50 pound sample was obtained from the St. Paul's College excavation, on the University of Manitoba campus grounds, at a depth of 2 to 6 feet. This soil would have physical properties similarly to those of soil No. 4. It was described as a fine-grained , heavy clay gumbo, very common in the Red River Valley, found at depths of approximately 2 to 6 feet.

An investigation was carried out to determine what effects additives of dolomitic quicklime had on the strength of this soil. The purpose of the investigation was to determine whether a clay gumbo could be stabilized with lime to produce a satisfactory subgrade material.

Before test specimens were prepared, the Miniature Harvard Compaction Device was first calibrated with the Standard Proctor Method for this soil, as described when tests were run with Soil No. 5. The following results were obtained using the Standard Proctor Method of compaction for Soil No. 6:

Maximum Dry Density - - - - - - - - - 85.5 $\#/ft^3$
Optimum Moisture Content - - - - - 27.5%

Using the above optimum moisture content, 3 layers in the Harvard mold, and a 25 pound tamping force, the number of blows per layer was varied until the desired dry density of 85.5 lbs. per cu. ft. was obtained. It was found, that for this type of soil, a heavy clay gumbo, 17 blows per layer were required.

Specimens containing the natural soil without additives, and others with 10 per cent of dolomitic quicklime were then compacted using the Harvard Method. The specimens (unwrapped) were then placed in the humidifier and cured at room temperature (72°F) and near 100% relative humidity. Some of the specimens were tested immediately while others were left to cure for periods of 2, 7, 14, 21, and 28 days.

Tables 12 and 13, and Figure 8, illustrate the effect of 10 percent dolomitic quicklime on the strength of Soil No. 6. In its natural state, the soil averaged a compressive strength of approximately 50 p.s.i. during the 28 day curing period. The addition of 10 percent lime to the clay gumbo showed a very marked increase in strength, particularly in the early stages of moist curing. At 28 days of curing, the specimens with lime additives, showed a strength of approximately 4½ times that of the natural soil. An examination of the slope of the strength-time curve revealed that this strength gain would continue considerably after 28 days. This investigation tended to prove the theory that the chemical reaction is very prominent with clays of the gumbo type, as outlined earlier in this thesis.

Sample Number	Curing Time (Days)	Strength (p.s.i.)	Average (p.s.i.)
S1	0	40.4	
S2	0	35.2	37.8
S3	0	37.8	
S18	2	35.6	
S19	2	44.3	40.1
S20	2	40.5	
S17	7	47.0	
S23	7	47.8	51.1
S24	7	58.5	
S16	14	45.1	
S22	14	44.0	45.2
S15	14	46.5	
S9	21	47.0	
S10	21	59.1	46.7
S11	21	36.0	
S12	21	44.8	
S13	28	60.1	
S8	28	41.0	63.2
S21	28	87.0	
S14	28	64.8	

Table 12. Compressive Strengths in Pounds per Square Inch, of a Clay Gumbo (Soil No. 6, Winnipeg), without Additives after Various Curing Periods.

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Sample Number	Curing Time (Days)	Strength (p.s.i.)	Average (p.s.i.)
L1	0	35.8	
L2	0	32.4	33.3
L3	0	31.6	
L23	2	51.0	
L14	2	68.0	68.8
L13	2	87.5	
L21	7	150.0	
L16	7	132.0	131.0
L15	7	106.0	
L24	7	136.0	
L20	14	199.0	
L18	14	180.0	162.0
L17	14	107.0	
L4	21	217.0	
L5	21	225.0	226.0
L6	21	237.0	
L9	28	276.0	
L8	28	249.0	273.0
L7	28	299.0	
L10	28	269.0	

Table 13. Compressive Strengths in Pounds per Square Inch, of a Clay Gumbo (Soil No. 6, Winnipeg), when Treated with 10% of Dolomitic Quicklime after Various Curing Periods.

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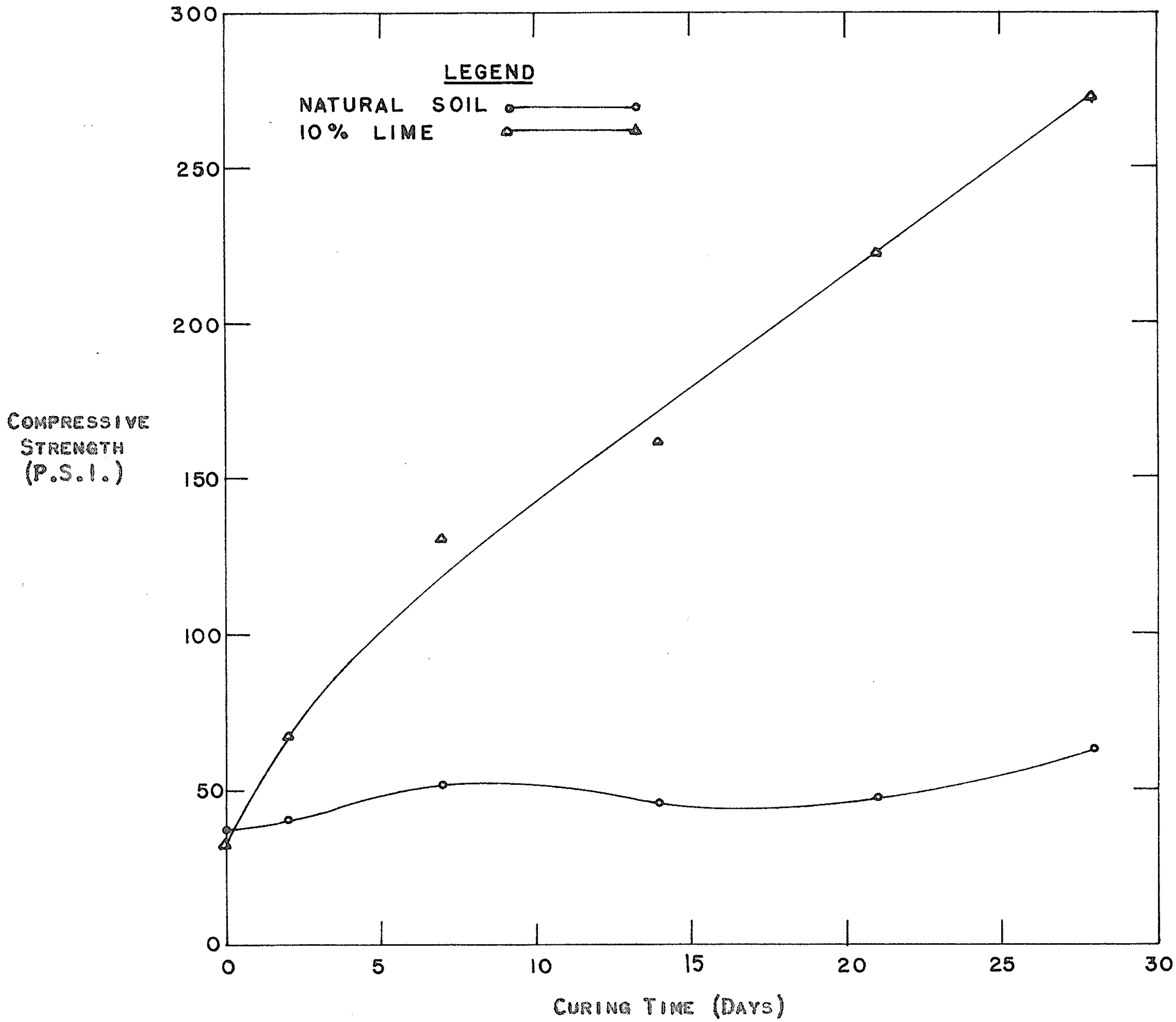

Figure 8. Strength-Time Relationship of a Heavy Clay Gumbo (Soil No. 6, Winnipeg), when Treated with 10 Percent Dolomitic Quicklime.

C.E. Department U. of M. 1958.

Laboratory Results on Soil No. 7

A representative 50 pound sample was obtained from a stockpile of clay binder near West Hawk Lake. The clay was being used as a binding agent to produce a clay-gravel stabilized base course on the Trans Canada Highway, east of West Hawk Lake to the Ontario boundary. Considerable difficulty was being experienced in mixing in the binding agent with the granular material at the central mixing plant. On days when the humidity was fairly high, the binding agent tended to clog screens and also segregate into clay balls. This situation gave rise to a great deal of difficulty in producing a stabilized base course material with a consistent amount of clay binder.

Previous tests on soils of this type (See Table 6) illustrated that the plasticity index would be reduced, or that the soil treated would be more friable, if additives of lime were mixed in with it. An investigation was therefore carried out to determine whether the lime would also give an additional benefit; namely that of increasing the bonding action between the clay binder and the granular material.

As in the two previous strength evaluations, the Miniature Harvard Compaction Device was first calibrated with the Standard Proctor Method for this particular soil. The following results were obtained using the Standard Proctor Method of compaction for Soil No. 7:

Maximum Dry Density - - - - - - - 109.5 $\#/ft^3$
Optimum Moisture Content - - - - 15.0%

Using the above optimum moisture content, 3 layers in the Harvard mold, and a 30 pound tamping force, the number of blows per layer was varied until the desired dry density of 109.5 lbs. per cu. ft. was obtained. It was found, that for this type of soil, a clayey silt, 20 blows per layer were required.

Specimens containing the natural soil without additives, and others with 3 percent of dolomitic quicklime were then compacted using the Harvard Method. The specimens (unwrapped) were then placed in the humidifier and cured for periods of 3, 7, 16, 21, and 30 days. The samples were tested as previously disclosed.

An examination of Tables 14 and 15, and Figure 9, illustrated the effect of 3 percent dolomitic quicklime on the strength of Soil No. 7. The natural soil averaged approximately 40 p.s.i. in compressive strength. The strength gain was fairly slow for this type of soil, a clayey silt, however, after 28 days of moist curing, the strength was more than doubled for 3 percent lime. Comparing these results with those of Soil No. 5 and Soil No. 6, slightly higher percentages of lime would have increased the strength. Also, since this material was fairly low in clay content, the introduction of an artificial pozzalon, fly ash, would probably have increased the strength. Hence, it appeared that clay-gravel stabilized base courses could greatly be improved in consistency and strength if a small percentage of lime is mixed in with the materials. This also brought out the fact that lime could be used to upgrade gravels containing excessive amounts of clay by reducing the amount of clay binder and also adding the benefit of additional bonding strength of the granular material.

Sample Number	Curing Time (Days)	Strength (p.s.i.)	Average (p.s.i.)
S-1	0	27.9	
S-2	0	34.3	32.2
S-3	0	34.4	
S-7	3	30.1	
S-5	3	33.2	32.8
S-6	3	35.3	
S-8	7	41.8	
S-9	7	41.8	41.6
S-10	7	41.1	
S-11	16	51.2	
S-12	16	45.5	46.6
S-13	16	43.2	
S-17	21	62.1	
S-15	21	58.5	59.0
S-16	21	56.4	

Table 14. Compressive Strengths in Pounds per Square Inch, of a Clayey Silt (Soil No. 7, West Hawk Lake), without Additives after Various Curing Periods.

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Sample Number	Curing Time (Days)	Strength (p.s.i.)	Average (p.s.i.)
L-1	0	32.5	
L-2	0	26.2	29.8
L-3	0	30.6	
L-4	3	37.8	
L-5	3	47.3	42.0
L-6	3	40.8	
L-7	7	51.7	
L-8	7	48.0	48.6
L-9	7	46.0	
L-12	16	65.5	
L-13	16	58.8	61.8
L-14	16	61.1	
L-15	21	76.4	
L-16	21	76.8	75.7
L-17	21	74.0	
L-18	30	94.1	
L-19	30	82.0	86.9
L-20	30	84.7	

Table 15. Compressive Strengths in Pounds per Square Inch, of a Clayey Silt (Soil No. 7, West Hawk Lake), when Treated with 3% of Dolomitic Quicklime after V..rious Curing Periods.

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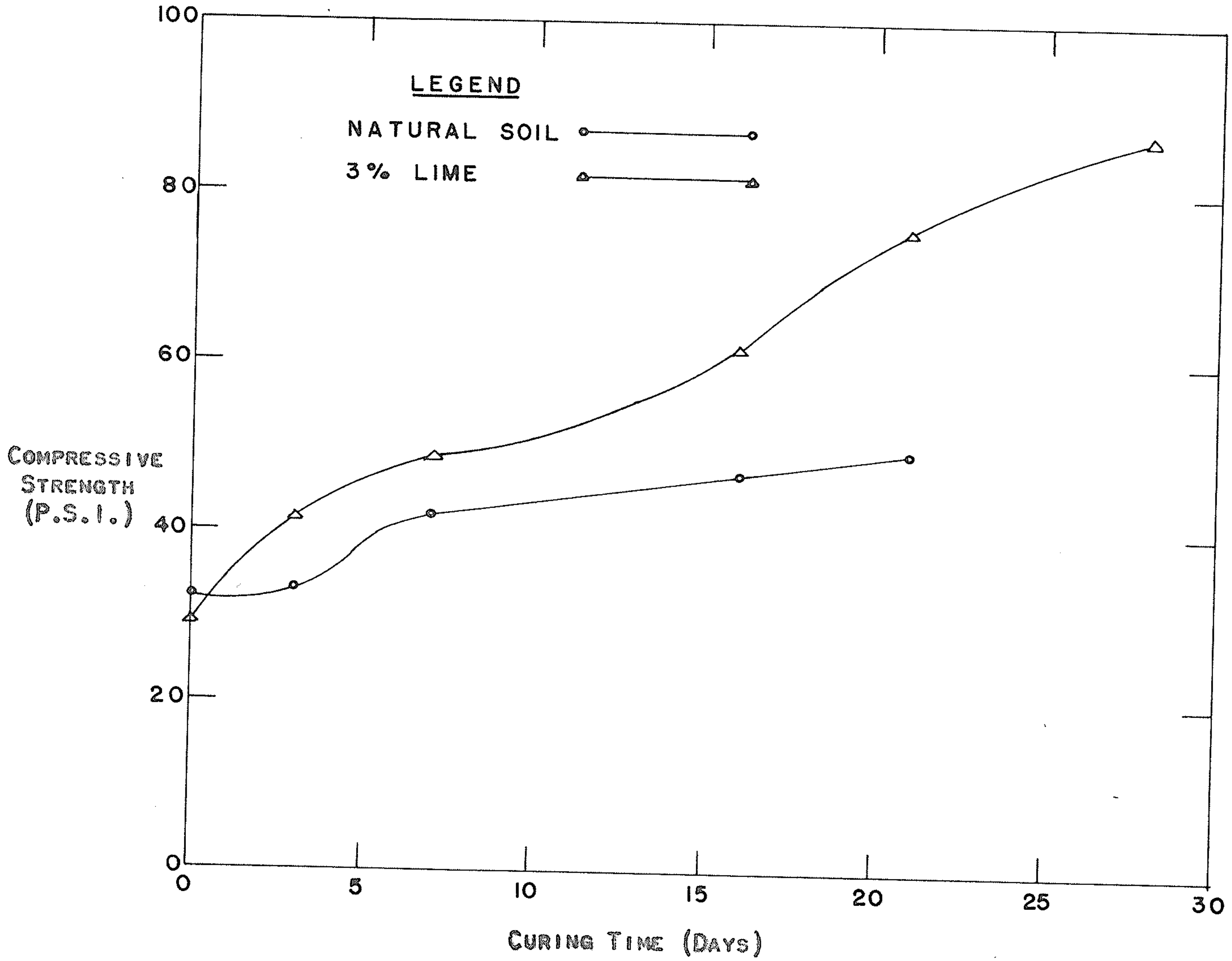

Figure 9. Strength-Time Relationship of a Clayey Silt (Soil No. 7, West Hawk Lake), when Treated with 3 Percent Dolomitic Quicklime.

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CHAPTER XII

Summary of the 1957-58 Lime-Soil Investigation at the University of Manitoba

In general, it can be concluded from this investigation that lime produced some marked effects on improving the stability of Manitoba soils. As a rule, the addition of lime and lime with fly ash, in different percentages and to various soils, gave results in conformity with those of published data. The following is a summary of how the physical properties of the various soils tested were altered when additives of lime and lime with fly ash were used:

(1) The use of these additives reduced the plasticity indices of all the fine-grained soils tested. This reduction was most substantial when highly plastic soils, like heavy clays were treated with lime. The use of the additional additive, fly ash, caused an additional decrease in the plasticity of the soil. For effective plasticity reduction it was found that percentages slightly higher than 5 would be required for clays of high plasticity. For soils containing more silt and sand, it was found that percentages of only 2, 3, and 5 were required.

(2) Laboratory experience, derived in this investigation, showed that the workability of fine grained soils was greatly increased when small admixtures of lime were used. Dehydration and pulverization of fine grained soils was greatly aided when these admixtures were incorporated.

(3) The use of these admixtures gave substantial improvements to the compressive strengths of the soils tested. In general:

(a) The strength gain was fairly slow, however, rapid enough for practical use in road building.

(b) Strength gain was most effective with soils high in clay content due to the natural pozzalons in these soils.

(c) Coarse grained soils, high in silt and sand content, required fly ash to promote greater action between the soil and the lime.

(d) In most cases, judging by the slope of the strength-time curves, the strength of the specimens would continue to gain strength after a period greater than one month.

(e) Low percentages of lime, added to soils with fairly high clay contents, improved the strength of these soils over 5 times after one month of moist curing.

(f) Both the use of quicklime or hydrated lime showed effective strength improvements.

(g) Moist curing gave higher initial strength gains than did sealed curing.

(h) A typical failure in the unconfined compression machine was that of shear with little lateral or vertical deformation.

ADDENDUM

The results of this soil-lime investigation has shown that the use of this additive has the possibility of a bright future in soil stabilization. It is unfortunate, however, that due to the lateness in the season, no test strip was installed to make this investigation more complete. The results showed, that for the soils tested, lime greatly improved the physical properties, however, it is recommended that a few simple laboratory tests be performed as outlined in this thesis, prior to actual field application.

Further research on this aspect of soil stabilization would be of great value to make Canadian soils engineers more familiar with this fairly new method of road-building. The following are a few recommendations on future research goals:

(1) Design of base course thicknesses utilizing the triaxial compression machine.

(2) Durability of soil-lime mixes compacted at densities above Standard Proctor.

(3) Effects of freeze-thaw and wet-dry conditions on local soils.

(4) A comparison of the relative merits of high calcium versus dolomitic lime for soil stabilization.

(5) The use of secondary additives in conjunction with lime to obtain the greatest effectiveness out of the lime-soil reaction.

American soil engineers have had very favourable results on the above mentioned topics, particularly in regards to the durability of different soils against freezing and thawing, and wetting and drying when compacted above Standard Proctor density. Research [28], at the Iowa Engineering Experimental Station, has shown that in the case of clays, freeze-thaw cycles actually benefited the strengths of the specimens.

There are, however, several differences of opinion as to the relative merits of high calcium lime versus dolomitic. However, both types of lime have been used and have proved to be very satisfactory soil stabilizing agents.

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