

"LIME STABILIZATION OF LACUSTRINE CLAYS IN THE CANADIAN PRAIRIES"

by

GANI VENKATARAMAN GANAPATHY

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ABSTRACT

The investigation reported in this thesis was conducted at the University of Manitoba between August 1978 and May 1980.

The objectives of the investigation were:

1.) To conduct routine tests on clays from two Canadian Prairie regions (specifically from Winnipeg and Regina) treated with various percentages of commercially available calcitic quicklime between 2% and 10% and observe the effect of lime on such engineering properties of the clays as plasticity, compaction and strength. This was primarily to confirm previous test results and to add to the existing data.

2.) To study the fabric of lime treated clays with the help of X-ray diffraction, DTA and electron microscopy with a view to gain a better understanding of the mechanism of the action of lime on these clays and to identify the reaction products. It was also the intention to ascertain the potential application of these methods in such studies.

In addition to the samples prepared in the laboratory, shelly samples were recovered from the test sections with lime treated clays established, by the Departments of Highways of the Provinces of Manitoba and Saskatchewan, about twenty years back. Tests on these samples helped to determine the permanency of lime treatment as well as providing an opportunity to study the fabric of old treated clays.

The thesis consists of two parts. Sections 1 through 3 review the literature on lime treated soils upto May 1980 and establish the need for a comprehensive investigation in Canada on this subject on a regional basis. The literature review was summarized as an "Annotated Bibliography on Lime Stabilization" and has been published by the Department of Geological Engineering of the University of Manitoba.

Sections 4 and 5 report on the tests conducted in this investigation and on the conclusions drawn from these tests. It was found that the soils from both regions confirm, generally, the results published in the literature, i.e. a change in grain size distribution to the coarser side, a reduction in plasticity, increase in the unconfined compression strength and the pH dependence of the reactions. However, subtle differences in the behaviour were also noted. These are attributed to the different morphology and bedrock formation in these regions. A more intensive research is needed to clarify these differences. Such differences with respect to their swelling behaviour has been reported in literature. (Hamilton, 1969).

X-ray diffraction and DTA analyses showed the formation of new minerals of the type tobermorite and ettringite, both complex aluminosilicates. These results again showed some differences in the behaviour of the two clays. Electron micrographs showed a massive agglomeration of the clay particles in sharp contrast to the flaky structure reported in the literature (eg. Baracos, 1977).

The samples from the test sections from the highways in the two provinces showed, even after 20 years in service, a coarser grain size distribution than natural clay, less plasticity and higher strength. X-ray patterns did identify some tobermorite-like silicates. Electron micrographs showed that the particles are still agglomerated. Thus lime treatment appears to be permanent at least in these test sections.

It is recommended that a more comprehensive research be undertaken on this subject. Such an investigation should include other aspects of lime stabilization such as compaction characteristics, curing properties, realistic cutoff dates for lime stabilized projects in this country, effects of different types of limes and durability characteristics of lime stabilized clays. In addition, a comprehensive investigation into lime-flyash stabilization is needed. With dwindling aggregate resources and lime and cement both being high energy products (high calcination temperatures) soil stabilization with waste products such as flyash would be very timely.

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LIME STABILIZATION OF LACUSTRINE CLAYS IN THE

CANADIAN PRAIRIES

1. Introduction.

Improvement of the physical and mechanical properties of marginal soils with a binder or pozzolonic material is an ancient concept. Lime as a pozzolan and binder has been known to man since the early days of history. The Romans are credited with the use of lime as a stabilizing agent in the construction of the extensive network of their roads and highways; with the use of lime mortar in the construction of their aqueducts and other structures. However, since the middle ages the use of lime for road building purposes was on the decline and was practically abandoned. This may be traced to the discovery and manufacture of other construction materials such as portland cement and bituminous cements, and also to newer construction techniques such as MacAdam type of roads. But towards the end of the last century and right through to the end of World War II the use of lime in highway construction has been revived. Increased urbanization, wartime efforts, non-availability of prime land for road building purposes, depletion of good aggregate sources and more recently, a concern for the ecology when borrow areas are left as gaping holes in the landscape, have all contributed to the renewed interest in this ancient material. Credit should be also given to the tremendous amount of research done on lime stabilization with and without such pozzolonic materials such as fly-ash and other chemicals, particularly in the post war period.

Use of lime in civil engineering practice can be classified in three categories:

1. As a construction expediency in wet, marginal and swampy conditions in silty clay to heavy clay soils (CL to CH type of soils).

2. As a means to counteract the swelling potential of certain highly plastic and swelling type of clays, such as the Lake Agassiz clays.

3. To improve the strength and bearing value of the soil.

Tens of thousands of kilometres of highways and road have been and are being built over clayey subgrades which have given rise to many problems resulting in huge sums of money in maintenance and upkeep. Swelling clays occur in many parts of the world, including the Canadian Prairies. Structures built on these have suffered severe damages. It is said that in the United States alone the litigation over damaged structures because of inadequate foundation design in swelling soils amounts to over two billion dollars (Wright 1973; Gromko, 1974). It is small wonder then, that research on improving these clays and stabilizing these as subgrades for roads has been extensive. These efforts have contributed to a better understanding of the behaviour of these clays and their stabilization with various chemicals, including lime. However, properties of soils are affected very much by their origin, geology, environment, and other regional factors. One cannot miss this point when one goes through the vast amount of research reports contained in the publications of the Highway Research Board. This led the

Board in 1976 to state "Soil-lime reactions are complex and not completely understood at this time. However, sufficient basic understanding and successful field experience are available to provide the basis for adequate technologyFuture research findings will further augment our technology.....concerning lime treatment of soils." Elsewhere the Board continues "At present only limited information is available concerning some of the properties of lime treated soils." (TRB Research Circular 180: 1976.) A need to study such soils on a regional basis was further demonstrated by Thompson (1966, 1970).

While research on lime stabilized soils has been very extensive and systematic in the United States it has been minimal and not quite so systematic in Canada. The research has been mainly confined to the work of the Departments of Highways of the Provinces of Manitoba and Saskatchewan and to two thesis work by Klym (Young, 1964; Christison, 1968; Culley, 1970; Klym 1958, 1965). Young and Culley were mainly concerned with the roughness of pavements on the expansive Prairie clays and the resulting transverse ridging. Klym's original work was a general review of lime stabilization while later, he investigated the durability of Canadian clays after lime treatment. He pointed to the necessity of further work in this direction. Therefore there exists a need to launch a systematic research on the lime stabilization of the expansive clays found in Canada.

2. OBJECTIVES OF THIS STUDY

The work reported herein is the initial phase of such a research programme. Specifically the investigation sets out to address the following questions:

1. Review of the literature in Canada and elsewhere.
2. Engineering properties of the Canadian Prairie clays with and without lime treatment.
3. Investigate the potential use of X-ray diffraction, differential thermal analysis (DTA), and scanning electron microscope (SEM), in the study of the mineralogy and fabric of clay-lime mixtures.
4. Verification of the hypotheses concerning lime stabilization and to suggest any additional factors peculiar to Canadian Prairie clays.
5. Discussion of future phases of this research.

3. REVIEW OF LITERATURE

3.1 Historical

Literature on lime stabilization is vast (see bibliography at the end of TRB Research Circular 180: 1976; Herrin and Mitchell, 1961; and Ganapathy, 1980. It is nearly impossible to read all the published reports, but as Herrin and Mitchell point out it is not necessary either. The reports fall into three categories:

Category I: Lime modification used as a construction expediency. The bulk of the literature prior to 1945 falls into this category. In many cases construction techniques are described but little, if any, geotechnical data on the soil except general classification was given. It is obvious that the importance of geotechnical aspects had not been fully understood at that time.

Category II: Lime modification or stabilization of marginal soils used specifically to reduce plasticity, swelling or to improve bearing values. These could be a construction job, a specially laid out test project or a pre-construction design. All pertinent geotechnical data are reported. A good number of papers

published during World War II and since then, fall into this category. Laboratory behaviour and field correlations are reported but still no fundamental postulations concerning the mechanism of stabilization were made.

Category III: Basic research work seeking to explain the mechanism of stabilization of clay by lime. This group of publications appeared mostly in the late fifties to early seventies. These investigations were spurred by a better appreciation of the importance of mineralogy as it affects the behaviour of clays and the development of good and reliable equipment for X-ray diffraction, DTA, and electron microscopy.

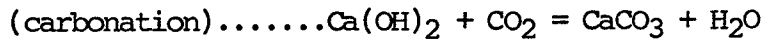
3.1.1 Literature prior to 1945

Most, though not all, published reports prior to 1945 (see Herrin and Mitchell, 1961) describe case histories where lime was used as a construction expediency to cope with a wet swampy site condition. The engineers involved knew either by intuition or from previous experience in similar conditions that lime could be used to obtain a "working platform". Not much light is shed on the geotechnical aspects of stabilization. Nor was there any post construction analysis. Some of the theories advanced during this

period for the success of lime in stabilizing the soil were:

1. Lime has an affinity for water and hence draws water from the wet soil. At the same time a mortar is formed which binds the soil together (Kuonen, 1963).

2. In the presence of water lime enters into a chemical reaction:



The calcium carbonate provides the cementing medium. These reports failed to appreciate that lime would not be successful in all wet and swampy conditions; not all clays would react the same way. In other words the influence of mineralogy was not appreciated. It was also not realized that different limes react differently. Some of the unsatisfactory results after using waste lime (a by-product in the calcination of limestone) could be traced directly to this lack of knowledge. Furthermore it was wrongly assumed that carbonation was essential in soil stabilization. In fact, early carbonation or "spoiled" lime could be detrimental to stabilization (Mateos, 1965). These early interpretations of lime modification attempted to draw parallels between portland cement concrete or lime masonry. While reviewing lime stabilization during this period, McDowell (1959) states that ".....prior to 1945 field experiments were conducted in a number of States without consideration of control of mixing, curing, and compaction.....and resulting performance was disappointing. One such job in Texas probably delayed the development of lime stabilization in that State for ten years." Elsewhere he continues ".....these same materials that failed on one job.....when used on another nearby have given a splendid performance under medium heavy traffic for 14 years."

It was, thus, clear that prior to 1945 successful lime stabilization was a matter of chance. There was no systematic investigation prior to construction and no analysis or monitoring after the construction. In some cases pavements having lime stabilized subgrade or subbase were inspected and the condition reported. But these do not shed much light on the problem due to lack of adequate data prior to construction during the design stage (Manz, 1965).

3.1.2 Literature Subsequent to 1945.

The post war period saw the rapid development of geotechnical engineering and a systematic approach to the design and construction of roads, highways, and airfields largely due to the efforts of the different State Highway Agencies and the U.S. Army Corps of Engineers. Soil stabilization was a subject of intense research at the MIT (Lambe, 1954; Ladd et al, 1960; Lambe and Whitman, 1969), at the University of Illinois (Eades, Grim, Thompson, and their associates), at the Iowa State University (Handy and his associates) and at the Bureau of Public Roads (Diamond et al). The National Lime Association began a systematic documentation of all lime stabilized projects and funded extensive research. This culminated in the publication of a manual for lime stabilization (1954). The literature in the post war period fell into two groups:

1. Reports on the effect of lime on soils with extensive laboratory research. Different aspects of stabilization were investigated. Often these were relevant to particular projects and thus field data were collected and correlated to laboratory data. The reports concerned themselves with testing procedure, apparatus,

construction techniques, etc. No attempt was made to explain the mechanism of stabilization. Statistical methods were used to correlate the various results and these were attempted to fit the different regional soil profiles (USAE, 1951, 1956, 1962; Thompson, 1966; Ozier and Moore, 1977).

2. Reports attempting to give a mechanistic picture of lime stabilization using data from X-ray diffraction analysis, differential thermal and thermogravimetric analysis, optical and electron microscopy and other chemical analyses. Principles of clay mineralogy and colloid chemistry were applied to the clay-lime-water-electrolyte system to explain the behaviour of these data (Eades, 1962; Eades and Grim, 1960; Eades et al, 1962; Ho and Handy, 1962, 1963; Diamond et al, 1965).

3.2 Review of Research on the Effects of Lime on Soil Properties

Addition of lime to a soil produces certain changes in the properties of the soil. These changes take place in two distinct stages.

- i) An immediate change in the colour, texture, and plasticity.
- ii) A long term improvement in the strength and deformation properties.

The nature and amount of change and the time required for the change to take place depend on a number of factors such as the type of soil, the mineralogy, type of lime, quantity of lime, period of aging and curing, the chemistry of the pore water, and regional factors. While a basic hypothesis for lime reaction has been advanced and is generally accepted, soils should be evaluated on a regional basis.

3.2.1 Immediate Amelioration

This is the term used, in the literature, to denote the changes that take place in the soil within the first few hours after adding lime to it. If the soil has been broken down sufficiently (maximum size 40 mm) adding lime and working in a mechanical mixer or kneading with hands soon breaks down the clods to small granules of the size of fine gravel within a few minutes. The temperature generally rises and may reach 60° C. Hydrated lime results in a somewhat lower temperature because of the lower heat generated in the slaking process. Adding lime to bigger blocks does not break down the block and the immediate amelioration effect is not observed. Figure 1 shows the grain size distribution of a plastic clay in the

natural state and after adding 4%, by dry weight, of high calcium quicklime. The agglomeration of the clay platelets into silt or sand sized particles appear to be permanent and resists disintegration even after stirring in a mechanical mixer used normally in the laboratories for fifteen minutes (cf also Lund and Ramsey, 1959; Wolfe and Allen, 1964). Figure 1 also shows the grain size analysis of a sample recovered from a highway test section treated with lime and which has been in service for over fifteen years. Thus it appears that the immediate amelioration effect observed and reported in the literature is permanent.

Not only the texture and grain size but the plasticity of the clayey soils is drastically reduced. In low plastic or non-plastic soils the reduction in plasticity is not appreciable or may not take place. But in plastic clays, particularly in montmorillonitic clays, the plasticity is greatly reduced (Eades, 1962). The amount of reduction in plasticity depends on the amount of lime added. Khatti and Bharve, (1965), reported that for lime contents less than 1% by dry weight there was an increase in plasticity for a gumbotil. Similar results were quoted by Glenn, (1974). For organic soils the lime content to initiate the amelioration process could be even higher (Arman and Munfakh, 1972; Glenn, 1974). Above 1% the plasticity is progressively reduced with increasing amount of lime up to a certain lime content, see Figure 2. Increasing lime content beyond this has very little effect on the plasticity. Hilt and Davidson, (1960), called this point the "lime fixation" point. Ho and Handy (1963, 1965) and Diamond and Kinter (1965) call this the "lime retention point". Mark and Haliburton (1970) call this the "modification optimum".

The reduction in plasticity may be achieved either by an increase in the plastic limit or a reduction in the liquid limit or both simultaneously. In most published reports the last mentioned possibility seems to be the case. While there is almost always an increase in the plastic limit, liquid limit seems to decrease in many cases and increase in others (Clare and Cruchley, 1951; Herrin and Mitchell, 1961; Zolkov, 1962; Manz, 1965; Thompson, 1966, 1970). However, the increase in plastic limit is always of such magnitude that the net result is a reduction in the plasticity index. Even highly plastic soils are reduced to friable, non-plastic soils. This reaction appears to be irreversible. Cores taken from a highway section some fifteen years after construction show that the modified soil is still friable and non-plastic.

Prior to 1960 one or more of the following processes was thought to contribute to the immediate amelioration effect:

1. Cation exchange.
2. Repression of double diffuse layer because of the increased concentration of Ca^{++} ions in the pore water.
3. Carbonation of $\text{Ca}(\text{OH})_2$ which provided the bonding between the particles and hence inhibited the flow properties.

Diamond and Kinter (1965) argued against these hypotheses and demonstrated that even calcium saturated clays show the reduction in plasticity. Hence cation exchange cannot be the primary mechanism responsible for this phenomenon. In early days lime stabilization was found to be most effective with Na-montmorillonitic clays such as the Wyoming bentonite. Since calcium is higher in the lyotropic series than sodium it was convenient to explain the reduction in

plasticity by cation exchange. However, dihydrated dolomitic lime, magnesium hydroxide or aluminium hydroxide do not cause a similar reduction in plasticity as lime (calcium hydroxide) (Wang et al, 1963). Yet, magnesium and aluminium are higher in the lytropic scale than calcium. Thus cation exchange cannot be the principal mechanism in this process.

Repression of double layer may be rejected as a plausible mechanism for much the same reason. While the thickness of the double layer is reduced due to the addition of lime, the presence of higher valence ions should cause greater amelioration than calcium. Evidence is contradictory to this.

Carbonation as a reason for the reduction in plasticity is also not acceptable. Waste lime or slaked lime which has been left to stand for some time before reacting with the soil does not reduce plasticity to the same extent as quicklime. Further, lime-soil mixtures which are left in sealed bags or containers in carbon dioxide-free atmosphere still show reduced plasticity. Therefore formation of calcium carbonate as bonding agents responsible for this phenomenon cannot be considered seriously.

The foregoing discussion should not be taken to mean that these reactions do not take place in lime-soil mixtures. They do, but they are not the principal mechanisms responsible for the amelioration effects.

Davidson and Handy (1959) explained that the addition of Ca(OH)_2 increases the Ca^{++} density in the pore water which are then attracted to the normally negatively charged clay surfaces. They termed this "crowding of Ca^{++} to the clay surfaces". The crowding effect increases the viscosity of the water held on the surfaces and the flow properties are affected. Moreover, because of

the increased thickness of the adsorbed water layer the clay particles come closer or there is an agglomeration of particles. The growth in the particle size proceeds to a certain point when additional lime cannot produce any further growth at which state the "fixation point" is reached.

Diamond and Kinter, however, point out that the number of Ca^{++} ions absorbed on the clay surfaces should come from the ionization of $\text{Ca}(\text{OH})_2$ in the clay water system. Therefore the ratio of $\text{Ca}^{++}:(\text{OH})^-$ should be stoichiometric. The data by Davidson and Handy does not seem to support this view. They suggest that $\text{Ca}(\text{OH})_2$ is physically absorbed on the surface of clay with Ca^{++} facing the negatively charged platelets. This absorbed layer can be of few molecular thickness thus reducing the viscosity and flow properties of the clay. They presented data where after leaching the clay the ratio of $\text{Ca}^{++}:(\text{OH})^-$ was approximately stoichiometric. They attribute the small differences to the fact that the Ca^{++} ions could take their place at the edges of the clay platelets where the silicon and probably even aluminium could get dislodged in the strong alkaline solution. That silica could go into solution in a strong alkaline environment has been known to ceramic chemists for a long time (Grimshaw, 1971). Eades (1962), Ormsby, and Kinter (1968), have presented evidence that lime does cause ragged edges on the kaolin platelets.

Diamond and Kinter have not, however, explained all the facts. For example, other calcium salts such as CaCl_2 do not produce the same effect as lime though these salts do have Ca^{++} and they do ionize. However CaCl_2 with NaOH does produce the amelioration effect because the reaction product is $\text{Ca}(\text{OH})_2$.

Also on leaching a lime treated soil Diamond and Kinter note that the ratio of $\text{Ca}^{++}:(\text{OH})^{-}$ reduces from 1.7 at the first washing to 1.0 after a few washings. This would point to the fact that $\text{Ca}(\text{OH})_2$ is not physically absorbed, at least not all the molecules; that there is a layer of Ca^{++} which is firmly held on to the surface and cannot be dislodged by the dilution of pore water during leaching; that this is why the process is irreversible. It would also point out the necessity of a high alkaline environment needed to change the plasticity of the soil. In organic soils which are weakly acidic due to the humus, part of the lime goes to neutralize the acidity and hence these soils do require more lime than less organic or inorganic soils. The importance of pH in the reactions, in the reduction of the interlayer viscosity and its relation to the lime retention point is demonstrated in Figure 3 (Ho and Handy, 1963a). Eades and Grim (1966) have even devised a test based on the pH to determine the requirements of lime to stabilize a soil. Literature shows that many agencies are using this quite successfully to treat problem soils with lime (Allen et al, 1977).

Since the paper by Diamond and Kinter in 1965 and the subsequent discussions by Glenn, Handy and others, no further explanations have been offered concerning the mechanism of amelioration of clays by lime. A combination of all the processes is thought to be feasible and is generally accepted by subsequent investigators. If structural changes to the clay platelets should occur then these should be observable under an electron microscope. Yet, published reports of electron micrographs are practically nonexistent. Figure 4 shows what is believed to be an agglomeration

of clay particles due to the addition of lime. The sample was from one of the test sections constructed by the Saskatchewan Highways in the mid-60's. It is not included here as a conclusive evidence, but merely to suggest that more work in this direction may be useful in understanding the mechanism of stabilization with lime.

3.2.2 Potential for Volume Change

One of the applications of lime stabilization is to reduce the swelling potential of the soil. Highly plastic soils are also generally highly swelling, Figure 5. Therefore a reduction in plasticity and an increase in the shrinkage limit can be expected to inhibit the swelling potential of a soil. Many authors have investigated the effect of lime on swelling clays (Spangler and Patel, 1949; Mehra and Chadda, 1955; Mitchell and Hooper, 1961; Holtz, 1973; and many others). Young (1964) and Culley (1970) have successfully used lime to minimize the roughness caused to the pavement surfaces on the swelling Lake Agassiz and Lake Regina clays.

It is now generally accepted that clays swell on the addition of water because of the increase in the electrical repulsive forces (Lambe, 1960; Yong and Warkentin, 1975). It was also verified that clays containing divalent cations on their surfaces would swell less than those containing monovalent ions. In general, the swelling potential decreases as the cations ascend on the lyotropic scale (Bailey, 1965). Therefore, as in the case of amelioration effects, the reduction of swelling potential was explained by cation exchange, repression of double layer. Unfortunately a review of literature shows that there has not been much effort to correlate existing theories on swelling to the behaviour of lime stabilized soils.

This may form the basis for some future research.

Since swelling is closely associated with the plasticity of the soil, the same phenomena responsible for the reduction in the plasticity should be responsible for the reduction in swelling potential in lime stabilized clays. However, factors other than those partaking in the reduction of plasticity also play an important role in the modification of the swelling behaviour of the clays. These are the compactive effort, aging and curing. Mitchell and Hooper (1961) showed the effect of these factors on the swelling potential of some plastic clays, Figure 6. It is obvious that unlike plasticity, there is no optimum amount of lime to reduce swelling. Whereas in an untreated clay compaction to a higher density would lead to more swelling and reduction in strength, in lime treated clays these seem to enhance these properties. Therefore it is suggested that some work is needed before one can satisfactorily explain the swell resistance of lime treated clays.

3.2.3 Moisture-Density Relationships

Lime treatment found a major application as a simple, economical and efficient way to deal with problem subgrades in the construction of highways and airfields. Hence the moisture-density relationship has been studied in great detail. Almost all the investigators had demonstrated that the addition of lime causes a reduction in the maximum dry density attainable under a given compactive effort, Figure 7. At the same time the optimum moisture content is increased from what would be obtained for untreated soil. Many investigators have shown that the greater the lime content the greater will be the reduction in the maximum dry density and the

greater will be the optimum moisture content. This, however, is not always the case. For small percentages of lime added the increase in OMC is significant. But for higher percentages the change in OMC does not appear to be significant. It is interesting to note that in these cases, the increase in the OMC is observed for lime contents in the neighbourhood of the lime retention point. Beyond the retention point additional lime does not appear to have much effect on the OMC. While a reduction in the maximum dry density for a given compactive effort and an increase in the OMC appear to be the general rule there are a few reports which do not show this trend. Test results on a reddish brown Virginia clay (PI = 35) treated with 5% and 10% high calcium hydrated lime show practically no difference in the compaction characteristics from the untreated clays (Ashraf Jan, and Walker, 1963). In their review of literature Herrin and Mitchell, (1961), quote similar results obtained by Johnson, (1948). In general, it appears that highly plastic clays show a density reduction between 2% and 4%. Organic clays (Arman and Mufakh, 1972), silty plastic clays or micaceous clays (Herrin and Mitchell, 1961), and tropical or subtropical clays (Harty and Thompson, 1973), show greater reduction in the dry density. The reduction could be as high as 10%. The effect of lime seems to be more pronounced on the dry density than on the OMC.

The reason most commonly found in the publications for this behaviour of lime treated clays is as follows: Addition of lime to the soil causes a flocculated-aggregated structure. (The terminology used here follows the one suggested by Mitchell (1976).

The usual terminology found in the research reports are "flocculation" and "agglomeration"). Because of this structure , it takes more compactive effort to bring the soil to the oriented state which is believed to be the case at the point of maximum dry density (Lambe, 1958; Seed, 1958; Seed and Chan, 1959). In other words, for a given compactive effort (eg., AASHTO T-99) the particles will be in a less dense packing. Also due to the cardhouse of double T structure that is supposed to result (Diamond and Kinter, 1965), the soil can hold more water in its intra aggregation pore space (nomenclature after Mitchell, 1976). Thus lime treated soil would have smaller dry density and greater OMC for a given compactive effort.

It is unfortunate that the research reports have so far not published any electron micrographs of treated and untreated soils to verify these hypotheses. It is suggested that such an effort would shed more light on the compaction characteristic of lime treated clays.

It has been observed that a delay in compaction after the soil is mixed with lime causes more reduction in the dry density than when the soil is reworked and compacted soon after the mixing is done. This has led to specifications and recommendations that treated soil be compacted as soon as possible after mixing (Culley, 1970). This is discussed in a little more detail under a subsequent section (3.2.4, Aging). However, an explanation for this behaviour will be attempted here. If one assumes that addition of lime causes a flocculated-aggregated structure immediately, and also that complex cementing agents are beginning to be formed (Glenn and Handy, 1963;

Diamond and Kinter, 1965) then it is easy to conceive that the edge to face contacts will grow stronger with time. Hence the more delay one permits between mixing and compacting the more rigid will become these bonds and for a given compactive effort the less will be the densification of the soil. It would again be interesting to observe under high magnifications whether this is indeed the case.

The strength and bearing values of lime treated clays are discussed under section 3.2.5. But a few comments regarding these properties as they pertain to compaction are relevant here. In civil engineering practice higher density is synonymous to higher strength. Since lime treated clays generally give lower densities for the usual compactive efforts specified on projects it is often feared that the bearing value of the treated soil will be inferior to normal construction. This is not true. Many soils do in fact show higher strength than untreated soils (Herrin and Mitchell, 1961), in spite of the lower densities. Except for prolonged aging, the reduction in strength is insignificant (Mitchell and Hooper, 1961; Mateos and Davidson, 1963; Arman and Saifan, 1967). Even if the mixture is allowed to mellow for an extended period it can be compacted with relative ease to a higher density and strength. Therefore, as Herrin and Mitchell have aptly put it "Lime soil mixtures are one material to which the general thought 'when density decreases strength decreases' does not always apply".

3.2.4 Aging

Aging is the term used for the period when lime soil mixtures are left without being reworked or compacted. In literature this has also been variously referred to as "rotting" (because of

the smell that accompanies the addition of lime to soil), "mellowing" (because of the ease with which the mixture can be worked later) or "loose curing" as against the curing and maturing process after the mixture has been compacted. The American Road Builder's Association (ARBA) recommends that lime soil mixtures should be allowed to mellow for 48 hours before being reworked or compacted (1954). McDowel (1959), also recommends 48 hours of mellowing time for moderately plastic soil and longer periods for heavy clays. The general construction practice appears to be to compact the mixture at the earliest practicable opportunity after thorough mixing (Culley, 1970; Young, 1964), which is generally 24 hours (Townsend and Klym, 1966). As discussed in the previous section aging causes a reduction in the dry density. Mitchell and Hooper (1961), have shown that a delay in compaction after mixing affects the density, unconfined compressive strength and swell resistance adversely in comparison with a mixture which is not allowed to age, Figure 8. The percentage reduction depends on the moisture content, compactive effort, and the aging time. However, two points are to be kept in mind. The adverse effects occur for the same compactive effort. If the aged mixture can be compacted to a higher density or if mixtures are compacted to the same densities then the aged mixture shows superior qualities to the unaged, Figures 9 and 10. Therefore they conclude that:

1. A delay between mixing and compaction is not detrimental provided compactive effort is increased.
2. The strength loss due to remolding treated samples "cured" (aged) for considerable periods at constant water content is not large.

3. For the same densities the swell resistance of the treated clay aged longer is much less than a mixture aged for a shorter period or the raw soil.

4. In practice the advantage of improved mix uniformity and handling characteristics that may result from a delay between mixing and reworking prior to compaction may well offset or may justify the expenditure of more compactive effort.

3.2.5 Strength and Bearing Values.

By far the most discussed topic in the literature about lime treated soils is their strength. The most common type of strength test is the unconfined compression test. But other types of tests such as unconsolidated undrained tests (Thompson, 1966; Fossberg, 1969), flexural strength tests (Swanson and Thompson, 1967; Thompson, 1970), have been used too. Split tensile tests have been performed by many (Thompson and Dempsey, 1969; Thompson, 1965; Kennedy and his associates, 1971-a and 1971-b). More recently the so-called double punch shear has been used by Fang (1976), and Cumberledge et al (1976). Apart from these, many state highway agencies have used the CBR values, the cohesiometer and the stabilometer values to evaluate the lime soil mixtures depending on the design practice of that particular agency. Kennedy and Moore (1971), and Moore et al (1971), have shown positive correlation between the indirect tensile test (split tensile) and all other types of tests. Ramiah et al (1970), have investigated the residual strength of clays treated with many chemicals including lime and found that the residual strength for treated clays is higher than untreated clays.

The general relationship between lime content and unconfined compression strength or the CBR is shown in Figure 11. The development of strength in a lime soil mixture depends on many factors: type of soil, type of lime, amount of lime, compactive effort, curing time, curing temperature, etc. In the following each of these factors is discussed as reported by the many investigators.

3.2.5.1 Type of Soil

Thompson (1966), defined lime reactivity as the increase in the unconfined compression strength of the lime soil mixtures over the untreated soil. This idea had been picked up by many who have evaluated particular soils for lime reactivity. From these reports it is clear that all soils are not reactive to lime from the strength point of view. Organic soils inhibit the formation of the strengthening bonds and hence do not show much gain in strength below 10% or 12% of lime. Some tropical soils do not show as much strength gains as others. Montmorillonitic clays are more reactive than illites or mixed layer soils. In a study extending Thompson's earlier study in 1966, Moore and Jones (1971), found that the drainage of the soil can alter the mineral composition and can affect the reactivity with lime. They correlated the reactivity to extractable oxides of aluminum, and iron, and to silica. The form in which the iron oxide exists (Fe^{++} or Fe^{+++}) appears to affect the reactivity. This could explain why some of the lateritic soils are not particularly reactive. Therefore one cannot agree more with Thompson that a stabilization program should be developed on a regional basis. It should be emphasized that "reactivity" as

defined by Thompson refers only to strength. There could be instances when strength is not of primary concern and his concept of reactivity may not be applicable.

3.2.5.2 Type of Lime.

Lime used in the stabilization can be any of the following types:

1. High-calcium quicklime, CaO
2. High-calcium hydrated lime, Ca(OH)_2
3. Hydraulic lime
4. Dolomitic quicklime, $\text{CaO} + \text{MgO}$
5. Monohydrated dolomitic lime, $\text{Ca(OH)}_2 +$
 MgO
6. Dihydrated dolomitic lime, $\text{Ca(OH)}_2 +$
 Mg(OH)_2

Of these, the most commonly used limes are the high calcium hydrated lime, high-calcium quicklime, and the monohydrated dolomitic lime. The choice of a particular lime in a certain region depends on the availability and the parent rock in that area. Most investigators have compared hydrated lime and quicklime (calcitic limes) and concluded that high-calcium quicklime yielded better strengths than hydrated limes. Other authors have reported the results of special studies comparing the effect of the different limes on the strength properties of lime soil mixtures (Laguros and Davidson, 1956; Wang et al, 1962, 1963; Alexander et al, 1972). On the basis of these results it may be concluded that the different limes can be ordered as shown below:

1. Monohydrated dolomitic lime
2. Dolomitic quick lime
3. High-calcium quick lime
4. High-calcium hydrated lime
5. Hydraulic lime
6. Dihydrated dolomitic lime

These evaluations were based on the unconfined compression strengths. Figures 12 - 14 show the results obtained by Wang et al. These results should be taken with some caution. Two of the soils tested are silty (friable loess type of soil) while the other was a gumbotil. Their earlier study (1962) included a plastic till. While all soils were plastic and were said to contain appreciable amounts of montmorillonites, the percentage of fines has an effect on the reactivity of the soil with lime. The study shows that from the plasticity point of view calcitic limes are better than dolomitic limes. For low lime contents, it appears that the difference in strength between calcitic and dolomitic limes are not significant. At high lime contents, it appears that calcitic lime would gain as much if not more strength than dolomitic limes in the long term. In their review of lime soil mixtures Herrin and Mitchell (1961) cite such results. Figures 15 - 17 show a conceptual scheme of how the reaction products responsible for strength are developed with different lime at different ages.

Wang et al report that dolomitic limes show a greater variation in their manufacture than calcitic limes. The type of kiln (rotary or shaft), the firing conditions, the crystallite size of MgO, the silicon and sesquioxides in the lime and carbonate and $Mg(OH)_2$ content all affect the properties of the dolomitic

limes. They found that for dolomitic limes the finer the MgO crystallites the higher the strength. However, Alexander et al (1972) found that for calcitic limes the coarser the lime the higher is the strength of the mix.

As mentioned earlier the choice will in many cases depend on the availability of one or the other type of lime. This must be then evaluated on a regional basis taking into consideration the soil type and other environmental factors.

3.2.5.2 Lime Content

The amount of lime added influences the strength gained by the mix. The higher the lime content, the greater the strength. The rate of gain in strength appears to be higher for lower lime contents than higher percentages. There does not seem to be an optimum lime content for strength. This is in contrast to other chemicals which show an optimum content of the additive for maximum strength (Mateos and Davidson, 1961). However, it will be noted that Alexander et al (1972) have reported results for a medium plastic clay which show a definite optimum lime content for maximum strength, Figure 18. The mineralogy of this soil is not given.

3.2.5.3 Aging

Aging has been discussed under section 3.2.4. For a given lime content and compactive effort, the longer the mix is allowed to age the lower is the strength obtained. However at the same densities, which could involve somewhat higher compactive efforts, the more aged the sample the higher will be its strength, Figure 9. From this figure it may be seen that samples which were not aged did not gain strength with longer curing times whereas

samples which were allowed to mellow before compaction did make significant strength gains provided the densities are maintained the same. Unlike the expansive raw soils which are generally compacted to wet of optimum moisture contents to minimize swelling and strength loss, lime soil mixtures could be compacted to modified proctor densities or higher to give a dense compacted mass.

3.2.5.4 Curing

In contrast to aging, curing is the term employed to denote the time the mix matures after compaction. Lime-soil mixtures cure all their service life. Like portland cement concrete or cement stabilized soils, lime soil mixtures continue to gain strength with time, Figure 11. Most of the gain in strength is achieved in the early period while the strength continues to rise during the latter life. However, the strength gain is much slower for lime modified soils than for portland cement concrete. Many investigators have pointed out that it may take months and years for a mix to gain strength which would be shown in two or three weeks in the laboratory.

The reason for this is that the strength of lime stabilized soils depends not only on the time but also the temperature of curing. Ozier and Moore (1977) analyzed statistically the different factors influencing the development of strength in lime modified clays. Among other things, they determined a positive correlation between the strength of the soil and the combined effect of time and temperature of curing. This explains why many investigators found discrepancies between laboratory results and field measurements. Also, in comparing the different field records

the combined effect of time and temperature should be kept in mind. For instance, in the study by Ozier and Moore they found that lime modified soil showed a strength of 250 kPa at 4.4° C for a certain curing period. The same soil, for the same curing period but at a temperature of 43.3° C, showed an unconfined compression strength of 1200 kPa. This demonstrates clearly the importance of length of curing and temperatures of curing of lime soil mixtures.

The above discussion has some important practical implications. For one thing a reasonable curing procedure should be developed in the laboratory which will reliably predict the long term strength of the stabilized mix in the field so that the amount of lime can be predicted and an economical design arrived at. Secondly, in lime stabilized projects the specifications should be able to specify realistic cutoff dates for mixing lime. Anday (1961, 1963), has investigated the former problem, that of developing a laboratory procedure to predict the long term strength of lime soil mixtures. He combined the effects of time and temperature into degree days and compared the strength of many field cured samples with different degree days of curing with the strength of laboratory cured samples at 120° F and at 140° F. On the basis of his study he concluded that laboratory samples cured at 120° F for two days would yield the same strength as field samples cured for 700 degree days (F) if 0° F is taken as the datum or for 3000 degree days if 50° F is taken as the datum. Since not much significant reaction between lime and soil would take place below 40° F and since most construction projects would not start at a temperature as low as 0° F, 50° F would be a more realistic datum. Many investigators and highway agencies

have used Anday's concept and used two day curing at 120° F to predict the strength of lime stabilized mixtures. The investigators at Illinois use 140° F as the curing temperature based on the work by Eades (1962).

MacMurdo and Barenberg (1973) and Rosen and Marks (1974), have extended this idea to field work and have developed criteria for cutoff dates for lime stabilized work. Using a theoretical heat flow model and data from weather records MacMurdo and Barenberg arrived at cutoff dates for the Chicago area. This work demonstrates again the need for a regional approach for successful and rational application of lime stabilized construction. Rosen and Marks analyzed only one sample and one lime content and hence did not advance any generalized comments or recommendations. But their work confirmed the validity of the general concept.

3.2.5.5 Stress-Deformation Characteristics

Stress-deformation characteristics have been investigated by Thompson (1966), and Neubauer and Thompson (1976). These investigations show that the modulus of deformation, measured as the initial tangent by Thompson and as a secant to 75% ultimate stress level by Neubauer and Thompson, showed substantial increases when the soil was treated with lime. The ultimate strain for the lime treated soil was lower than for untreated soil. The results obtained by many others confirm these findings. The significance of these findings is that the lime treated soil behaves like a brittle material. Because of the high modular ratio between the treated layer and the underlying untreated subgrade, lime treated soil will exhibit a slab action under wheel loads on pavements. Therefore,

flexural strength, fatigue and resilient modulus should be of great interest in the case of lime treated soils (Ahlberg and McVinnie, 1962; Laguros, 1965; Swanson and Thompson, 1967; Robnet and Thompson, 1976).

These authors showed that lime stabilized soils would exhibit high flexural strength. However, flexural strength under repeated loads would be an important consideration in the design of pavements. Investigation by the above authors show that the resilient modulus of treated soils would be higher than untreated soils. Though the major increase in the strength of the lime treated soil is due to the increase in cohesion, the material behaves like a $C-\phi$ material as far as resilient modulus is concerned. It is typical of such materials that their resilient modulus will depend both on the confining pressure and the principal stress ratio (Monismith et al, 1972). In undrained unconsolidated triaxial tests where the specimen was subjected to repeated applications of deviator stress Fossberg (1969), found a linear relationship between $\log(\sigma_1'/\sigma_3')$ and the resilient modulus.

While Fossberg used the triaxial test to investigate the behaviour of the soil under repeated load, Thompson and his associates and Laguros used the flexural tests under cyclic loading. Their investigations show a linear relationship between $\log N$ and the ratio σ_1'/M_R , where N is the number of cycles to failure.

The resilient modulus is shown to be dependent on the initial moisture content, degree of compaction and the time of curing.

3.2.6 Durability

Whereas stabilized soils might show high strength and high modulus of deformation, it is important that they would be also resistant to the damages caused by environment conditions. This is generally termed durability. The most common form of tests for durability of stabilized material are: 1) the wet-dry test, and; 2) the freeze-thaw tests. These tests were originally developed by the Portland Cement Association to test cement stabilized soils and has been generally adopted for other stabilized materials. The tests have since been standardized by the ASTM (ASTM D 559 and D 560). In a report prepared by PCA (1959), lime-soil and cement-soil mixtures were treated by these procedures and it was concluded that lime-soil mixtures would not withstand the wet-dry and freeze-thaw cycles and hence were inferior to soil cement bases. It should be noted that these comparisons were done on granular base materials treated with lime or cement.

Durability is of prime concern to engineers and has been investigated by many. Herrin and Mitchell (1961), have summarized the test results prior to 1960. Since 1960, durability of lime treated bases have been investigated by Townsend and Klym (1966), Walker et al (1967), Esmer et al (1969), and Andres et al (1976). While Andres and his associates had still used the ASTM test procedure others have used different tests to determine the durability. These are the strength loss after freeze-thaw cycles by unconfined compression test, the split tensile tests, and more recently the so called double punch shear test. Because of the hypotheses for the deterioration of a sample after the

freeze-thaw cycles is based on Powers pore pressure theory, Klym had also used the soil suction and degree of saturation to measure the durability (Powers, 1955; Klym, 1965). Based on these tests all the investigators conclude that lime stabilized soil has very good durability properties. This is substantiated by many field tests (Dawson and McDowell, 1961; Ramsey and Lund, 1969; Culley, 1970; McDonald, 1970; Uppal et al, 1970; Stewart et al, 1971; Donnelly et al, 1977). These laboratory and field investigations show that the ASTM procedure may not be a realistic test for lime treated soils, though Andres et al (1976), concluded after following the ASTM procedure that lime treated soils do have good durability properties. A need still exists to develop a rational standard test procedure to evaluate the durability of lime treated soils.

Review of literature shows that the same factors that affect the strength of stabilized soils affect durability too. These are, among others, the initial molding water content, density, curing, and lime content. Most of the strength loss in the treated soil subjected to the freeze-thaw cycle occurs in the first one or two cycles. Therefore, it is essential that lime treated soils have a sufficient curing period before being subjected to the freeze-thaw cycles in the field. But it was also found that autogenous healing takes place in lime treated soils (Thompson and Dempsey, 1969; Robnet and Thompson, 1976).

3.2.7 Other Engineering Properties

Permeability of lime treated soils is found to be less than that of untreated soils. This was offered as one of the explanations of the freeze-thaw durability by some authors.

Neubauer and Thompson (1976), had investigated the consolidation characteristics of lime stabilized soils. The e -log p curve is much flatter for the treated soils than for the untreated soils. Hence the treated soils would undergo much less consolidation than raw soils. Ramiah et al (1970) have measured the residual strength of lime treated clays and found an increase in the residual strength. This could have some practical implication in slope stability problems. Broms (1977), had indicated the use of lime columns in other foundation problems. Whether this idea could be applicable in slope problems is not clear and should be investigated.

3.3 Reaction Products of Lime and Soils

While a great number of researchers have investigated the effect of lime on the engineering properties of the soil, a handful of investigators have studied the reaction products which are responsible for the changes in the property of the soil. The present state of understanding about these products comes mainly from Handy and his associates at the Iowa State University, Eades, Grim, and their associates at the University of Illinois, and Diamond and his associates at the Bureau of Public roads. All of these have shown that complex silicate hydrates and alumino silicates are formed when lime is added to the soil. The methods they have used to identify different products are X-ray diffraction, differential thermal analysis (DTA), scanning electron microscopy (SEM), and chemical analyses.

3.3.1 X-ray Diffraction Analysis

Table I summarizes the results of X-ray diffraction analyses by the different investigators. The X-rays used were mostly Cu-K α with a nickel filter. The commonly used instruments were the North American Philips geiger counter type or the General Electric XRD-5. Glenn and Handy (1963) have also used the Debeye-Sherrer camera using the strip film. Both powder mounts and oriented slide specimens have been used. The results do not show any differences.

The products which are identified mostly are complex silicate hydrates and aluminosilicates similar to the products of hydration of portland cement paste in concrete or soil cement (Herzog and Mitchell, 1963; Noble, 1967). Since Ca(OH) $_2$ is one of the products formed during the hydration of cement, some of the products which are formed during the hydration of soil cement mixtures can be expected to form during the reactions between lime and soils. These products are tobermorite, ettringite and hydrogarnets. Table II shows a few of the products identified during the hydration of different types of cements (Kantro et al, 1960). Comparing Table I and Table II one can see that some products formed during the hydration of cement can, indeed, be formed during lime soil reactions. Though many stated that tobermorite is formed, electron micrographs fail to show the lath or fibre shaped crystals of tobermorite as published by Brunauer (1962).

There are several difficulties in identifying the lime soil reaction products from the X-ray diffraction patterns. First tobermorite, ettringite and hydrogarnets are poorly

crystallized gels at least in the early stages of their formation and thus are difficult to identify by diffraction. Also their intensities, base widths and basal spacings change depending on the moisture and humidity. Finally in a naturally occurring clay or mixed layer mineral there are many mineral groups which give well defined peaks at the same Bragg angle as these products. Thus these often mask the reaction products of soil and lime (cf. Figure 19). For example one of the products with a peak between 3.04 A and 3.07 A which is reported by everybody is not very different from the peak due to calcite at 3.05 A. Calcite is a very common mineral in many Canadian clays and in many other soils (Lambe, 1952; Mitchell, 1976). One way to overcome this difficulty would be to scan the sample at a much slower rate than is usually done, for instance at $2\theta = 1/8^\circ$ per minute. Thus in effect, expanding the diffractogram (Figure 20, Parrish, 1960).

Another possibility is to observe the sample under fairly high magnification and isolate a single crystal and then observe its diffraction pattern or thermogram (Hilt and Davidson, 1961; Glenn and Handy, 1963). The single crystal technique has so far been reported only for pure clay minerals. Whether this will be successful for natural soils and mixed layer minerals is not clear. Reaction products from actual road projects have been identified by Eades et al (1962).

3.3.2 Differential Thermal Analysis

Figures 21A - 21G summarize some of the published thermograms on lime soil mixtures. Thermograms of many soils all over the world and of many of the commonly occurring soil minerals have been published by Lambe (1952), and by Lambe and Martin (1953 to

1959). Differential thermal analysis is a useful tool in the study of the fabric and structure of clay minerals but only in combination with other techniques such as X-ray diffraction, optical and electron microscopy, thermogravimetry and chemical analysis. The results are dependent on many factors such as the apparatus, soil preparation, humidity, type of thermocouples, the position of thermocouples, rate of heating, etc. (McKenzie, 1965; Blazek, 1972).

However, from published reports it would appear that:

1. The predominant peak would be an endotherm between 150° C and 220° C. This is usually interpreted as the loss of adsorbed water from the clay surface.

2. The beginning of the reaction, the peak and the end of reaction will occur between the temperatures noted above, depending on the amount and thickness of the water layer.

3. Calcium saturated minerals generally show a shoulder on their return limb of the endotherm. This has been interpreted as an overlapping peak due to the rigidly held water when Ca^{++} crowded to the surface (Glenn and Handy, 1963).

4. The lime added to the soil does not give its characteristic endotherm at about 550° C.

5. In some cases the double peak of the dolomitic lime has been observed after treating the soil.

6. Addition of lime would alter the shape, area, and slope ratio of the endotherms so that these can be taken as a measure of the lime reactivity.

7. The reaction products which are found on the X-ray diffraction patterns do not show any peaks on the thermograms.

3.3.3 Electron Microscopy

The development of reliable and high magnification electron microscopes have spurred the investigation of soil fabric. Since the mid 60's many reports have appeared showing the crystal structure of many clay minerals. While cement chemists and ceramic chemists have used the SEM extensively to study the fabric of reaction products of cement or ceramic clays, few of the micrographs have appeared showing the fabric of stabilized clays (Sloan, 1964; Diamond et al, 1966; Wang, 1966). Most of these studies are done with pure clay minerals and mostly kaolin. Eades (1962), reported ragged edges of the kaolin platelets after adding lime. He hypothesized that the Ca^{++} dislodges the silica or alumina and breaks the structure of kaolin. Much work needs to be done in this direction. If clay minerals with and without lime can be observed under high magnification, many of the hypotheses regarding cation exchange, destruction of lattice structure, repression of double layer and formation of new minerals can be verified or modified.

3.4 Summary of Section 3. "Review of Literature"

In the foregoing pages the literature on lime stabilization of soil was reviewed. An annotated bibliography of the literature between 1960 and 1980 is given at the end of this report. Herrin and Mitchell (1961), had reviewed the literature between 1925 and 1960. These together with the extensive bibliography given at the end of the TRB Research Circular No. 180 covers most of the work done to date. From these reviews one may conclude that:

1. Lime stabilization is a practical and efficient method to stabilize marginal soils, particularly the highly plastic and swelling clays.

2. Addition of lime improves the plasticity properties, strength and swell resistance.

3. The final product will depend upon the construction techniques, particularly the compaction and curing.

4. There is a need to standarize some of the test procedures, especially for testing the durability of lime treated clays.

5. While basic mechanisms for the action of lime soil have been offered and generally accepted, there still remains a need for additional work to confirm these hypotheses and refine them.

6. One of the major factors in successful lime stabilization is the soil itself and the environmental conditions under which the treated soil will be put to service. This calls for a regional approach to the problem.

7. While a tremendous amount of work has been done in the United States on a regional basis, the research in Canada has been minimal and piecemeal. Therefore there exists a need for concerted research on this topic in Canada.

4. TEST PROGRAMME FOR THE PRESENT STUDY

4.1 General

The present study is the preliminary phase of a more comprehensive programme to be undertaken in the future with an ultimate objective of investigating the lime stabilization of the Canadian Prairie clays from the many aspects as has been done by Thompson (1966) for the Illinois soils. Such a study has not been done before in Canada and it is suggested that it should be done, particularly in the present time of dwindling aggregate resources.

Previous Canadian research has been minimal and is restricted to the work at the University of Manitoba (Klym, 1958), at the University of Alberta (Anderson and Shields, 1963), and by the Departments of Highways of the Provinces of Manitoba and Saskatchewan (Young, 1964 and Culley, 1970). Reports of actual stabilization projects have also been published (Joy, 1977 and Peto, 1977).

4.1.1 Tests and Objectives

As outlined in Section 2.0, "Objectives of This Study", the primary aim of this preliminary phase was to investigate the possibility of identifying the reaction products in lime soil mixtures using X-ray diffraction and DTA and to study the fabric of lime treated clays using a scanning electron microscope. The DTA apparatus was acquired rather late in the programme and still requires a few refinements (see Section 4.4.7). Lack of resources and time restricted the electron microscopic study to just three samples though over forty samples were prepared for observation.

However, the limited results presented herein prove that these techniques can be used in the study of lime stabilized samples.

Details of materials (soils and lime used), tests and test procedures, and the number of samples tested in each test are presented in the subsequent Sections 4.2 to 4.4. An overview of the test programme is presented herein.

Bulk samples of the clays were obtained from Winnipeg and Regina. These were treated with 2, 4, 6, 8, and 10% commercial grade quicklime and loose cured for 3, 7, and 28 days. Thus there were thirty-two samples from these bulk samples, including the untreated clays. In addition, thirteen Shelby tube samples were recovered from the test sections constructed by the two provincial Departments of Highways (Table III). The specimens were identified by a numbering system as follows: The origin of the soil is identified by the letters W or R (for Winnipeg or Regina). This is followed by a numeral to denote the percent of lime added. Finally, the number of days of loose curing is denoted by 3d, 7d, or 28d. Thus the sample W-4-28d identifies the Winnipeg clay treated with 4% lime and loose cured for 28 days. The Shelby samples were recovered by the Regional Laboratory crew of the Federal Ministry of Transport and bear their sequential laboratory identification numbers (see Table III).



4.2 Materials

4.2.1 Soils

The soils for this investigation were obtained from excavations in the City of Winnipeg and at the airport at Regina. In addition the Department of Highways of Manitoba and Saskatchewan provided a few Shelby tube samples from the test sections they built in the late 50's and early 60's. Further, data from some investigations done by the Federal Ministry of Transport have been included. The latter soils were obtained from St. Andrews in Manitoba and from Gravelbourg and Leader in Saskatchewan. Figure 22 shows the geographical locations of these sites. Figures 23 and 24 show the locations of the test sections by the two Departments of Highways. Table III gives the details of the samples, sample depth, etc.

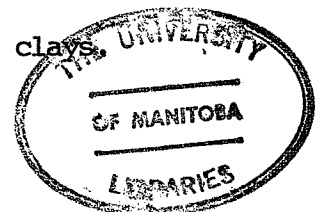
4.2.1.1 Geology and Origin of Canadian Prairie Clays

The geology of these sites is varied. This is believed to be the reason for the subtle differences observed in the behaviour of these clays and their reactivity to lime.

The plastic clays from Winnipeg, St. Andrews, and the subgrade material under the test sections by the Manitoba Highways Department are deposits of the large glacial Lake Agassiz (Figure 25). The genesis and geology of Lake Agassiz have been described by Upham (1896), Johnston (1916), Maynard (1925), and Elson (1965). According to these authors the deposits were formed in two different glacial lakes, Lake Agassiz I and Lake Agassiz II. Lake Agassiz I formed approximately ten to twelve thousand years ago and drained in

a relatively short period into Hudson Bay. However, subsequent advancing ice sheets blocked these drainage channels and hence a second lake was formed about eight thousand years back. This too drained in the next two thousand years leaving the Manitoba escarpment and lake remnants such as Lake Winnipegosis and Lake Manitoba. The deposits of Lakes Agassiz I and II consist of granular morainic material in the north formed during seasonal ice melts and glacial drifts. The deposits in the southern part are contributed to by the Assiniboine and Red Rivers. These rivers cut their channels in the Bearpaw shale which underlay much of north central USA and Western Canada. Therefore the lake deposits are material eroded from the shale.

The brown plastic clays from Regina and Gravelbourg are lacustrine deposits of a different glacial lake known as Lake Regina (Figure 26). The origin and geology of this lake has been discussed by Christiansen (1961). The deposits of Lake Regina are brought by the Qu'Appel and Arm Rivers as well as by the glacial meltwaters from the west. The coarser granular material was dropped in a smaller lake, Lake Moose Jaw, west of Lake Regina. The fines were transported eastwards and deposited in Lake Regina. The rivers and the drainage channels of the western ice sheets were eroding the same Bearpaw formation mentioned earlier. Thus much of the fine material deposited in Lake Regina was derived from the Bearpaw formation. Christiansen had identified seven different phases in the formation of Lake Regina. However, unlike Lake Agassiz, this lake was only one lake from the formation until it's subsequent drainage. Thus Lake Regina clays would be more homogeneous than the Lake Agassiz clays.



The clays of southwestern Saskatchewan were not lacustrine deposits. These are glacio-fluvial deposits of the South Saskatchewan River and its tributary, the Red Deer River. These made deep channels in the underlying Bearpaw formation but also carried the morainic material from the edges of the ice sheets which occupied much of southern Alberta at one time. The non-plastic granular materials are found in the Kindersly area and the Cypress Hills area. At Leader and surrounding areas, one encounters the plastic clays derived from the Bearpaw shale. The geology of this area has been studied by Williams and Dyer (1930).

4.2.1.2 Mineralogy

The mineralogy of these clays has been studied by many. Wicks (1965), Quigley (1968), Last (1974), and Baracos (1977) have reported on the Lake Agassiz clays. Hamilton (1968) and Fredlund (1975) have reported on the Lake Regina clay. Brady (1962) has analysed the clays from southwestern Saskatchewan. Their published results are summarized in Table IV.

It would be evident from the discussion under the previous sections that all these clays were derived from the same parent material, the Bearpaw shale. This shale is rich in montmorillonite and hence all the three clays are rich in this mineral. From the different reports it would appear that the montmorillonite content would vary between 55% and nearly 80%. The illite content has been variously reported between 15% and 20%. However, from the reports of Kodama and Brydon (1966) and from the results of this investigation, the illite does not occur as pure illite or mica but as mixed layer mineral mixed with montmorillonite. Kaolinite up to 10% has been identified by many. In fact, certain

layers of the clay from southwestern Saskatchewan near the Cypress Hill area contain such significant amounts of kaolinite that Brady and his associates identify these clays as suitable for pottery. Other clay minerals identified in traces are micas and chlorites. The predominant nonclay minerals are quartz, calcite, dolomite, gypsum, and feldspars. These are however, present only in minor quantities.

4.2.1.3 Geotechnical Properties

The geotechnical properties of these clays have been studied by many. Baracos (1957, 1960, 1969, and 1977) and Mishtak (1964) have reported, in detail, on Lake Agassiz clays while, Hamilton (1963 and 1968) and Fredlund (1975) have discussed the Lake Regina clay. There have been no systematic investigations on the clays from southwestern Saskatchewan known to this writer.

The soils contain between 50% and 80% clay fraction. Because of the predominance of the mineral montmorillonite these clays are highly plastic, the plasticity ranging between 35% and 55%. The liquid limit can exceed 90%. The plastic limit is typically around 25%. The upper 3.0 to 3.5 metres of the clays are usually dessicated resulting in highly fissured structure. In the dry state the strength of these soils is high, being in the order of 15 to 20 kPa. They show some sensitivity (between 3 and 5) as measured by field vane tests. In the saturated state the clays have low strengths in the order of 5 to 8 kPa. The clays are highly swelling (Figure 40) attributed to the expanding lattice mineral. Table IV summarizes the geotechnical properties of these clays.

There are subtle differences in the present day environment of these sites. The southwestern Saskatchewan area has been called dry and humid, the Winnipeg area sub-humid, and the Regina area as somewhere between sub-humid and semi-arid (Hamilton,

1963). Hamilton also showed that these environmental differences could lead to significant differences in their swelling characteristics.

4.2.2 Lime

A commercially available high calcium quick lime was the only lime used in the present study. The lime is manufactured and marketed by Winrock Supplies Ltd. of Winnipeg. The manufacturer's analysis of the lime is given as follows: Cao - 96.8%;

available Cao - 95.1%;

SiO₂ - 0.8%;

R₂O₃ - 0.7%;

MgO - 0.5%;

loss on ignition - 0.9%.

A future study should also consider the use of other limes such as dolomitic and hydrated limes and their effect on the stabilization of these particular clays. The lime was kept stored in the supplier's kraft paper bags. After every use the bag was closed and kept covered by polyethylene sheets. Samples were prepared in two batches separated by a few weeks. Almost identical pH values of 12.3 and 12.4 obtained for the two batches were interpreted to indicate that the lime had not been affected by atmospheric carbon dioxide during storage between batches.

4.3 Sample Preparation

The bulk samples as received from the field were initially dried in an oven for 24 hours at 50° C. The dried samples were then broken down to clods having a maximum size of 40 mm using an iron pestle and mortar. In some cases the soil was pulverized with a soil pulverizer which yielded soil passing the No. 100 sieve. 2000 grams of the clods or the pulverized soil were taken in a clean

mixing tray. The necessary lime to give the predetermined percentage was added to the dry soil and mixed with a trowel for one or two minutes. During this time it was also attempted to break the clods further with the trowel. 300 - 500 ml of water was then added to the dry mixture and mixed thoroughly for 2 - 3 more minutes. This operation completely broke down the clods to the size of coarse sand or fine gravel. However, in the case of pulverized soil passing the No. 100 sieve the soil granulated to this size. The colour turned lighter and the texture changed to that of a moist fine sand. Temperature of the mix was read, which was generally 55° C and 60° C. The mixture was then further mixed well by kneading between the fingers. Small amounts of samples were set aside for moisture content and pH determinations. The rest of the sample was transferred in double polyethylene bags with the opening of the first bag being covered by the bottom of the second. The bags were then sealed with masking tape, labelled, and kept in a carton at room temperature and humidity and away from direct heat, sunlight or any drafts until the time of testing. When the bags were opened on the day of the tests, small quantities of sample were immediately weighed for moisture content determinations. The results of all the moisture contents obtained are shown in Table V. Comparison of "mixing" and "prior to compaction" moisture contents show only small differences in moisture contents. These are probably due to the non-uniformity of the samples. The absence of any large changes would indicate that no appreciable evaporation or carbonation had taken place during the 3, 7, and 28 day storage period. A single exception may be sample W-2-28d which underwent a change from 29.98% to 23.22%, although the latter moisture content could be in error.

The Shelby tube samples obtained from the two Departments of Highways were kept in a humid room at 23° C and 100% humidity till ready for testing. The samples proved somewhat difficult to extrude from the tubes. Two of the samples (9323 and 9325) required almost 4500 kg force on the hydraulic piston to be extruded. These samples are, therefore, definitely disturbed and not much reliance can be placed on the strength values. Some of the samples were so brittle that they crumbled on being pushed out of the tubes. This is probably an indication that the original clays have been permanently changed in their plasticity characteristics. Figures 27 to 29 show the condition of some of these samples as extruded. Moisture contents were determined on these samples as soon as they were extracted. Preparation of samples for strength are described elsewhere.

4.4 Tests Conducted

The following tests were run on these samples:

1. ph values, 43 tests;
2. grain size distribution (only on the 28 day samples, 15 tests;
3. Atterberg's limits (only on the 28 day samples), 15 tests;
4. compaction, 60 tests;
5. unconfined compression strengths, 75 tests;
6. X-ray diffraction, 75 tests;
7. differential thermal analysis, 9 tests;
8. electron microscopy (only on three samples).

In addition to these, X-ray diffraction and DTA were done on samples of Peltonite (a bentonitic material manufactured by RocTest Ltd., Montreal), bentonite, lime and portland cement (as from bags and as hydrated).

4.4.1 Determination of pH Values

100 grams of the soil were mixed with 150 ml of distilled water in a glass beaker, stirred well, and allowed to stand for an hour. The pH was determined at the end of 1 hour, 24 hours, 3, days, 7 days, and 28 days. In the case of the core samples the age was many years. The pH meter used was a Coleman pH meter using Calomel fibre type and glass electrodes. The meter was calibrated with a buffer solution (pH = 7.0) certified by the American National Standards Bureau before each use. Table VI shows the results of the pH tests.

4.4.2 Grain Size Distribution

The test was done in accordance with ASTM D 422-63 (1972) using a 6% solution of calgon (sodium hexametaphosphate) as a dispersing agent.

4.4.3 Atterberg's Limits

These tests were done in accordance with ASTM D 423-66 (1972). No shrinkage limit test was done on these samples.

4.4.4 Compaction Tests

The compaction tests were done using the Miniature Harvard apparatus. The molds had an average diameter of 3.336 cm and an average height of 7.144 cm. The compactive energy was supplied by a 40 lb. (18.2 kg) spring force pressed for 1 inch (2.54 cm). Five layers were used to fill the mold and 10 tamps per layer were used. This compaction is supposed to be equivalent to the Standard Proctor compactive energy (USAE 1952). This test method was chosen primarily because of shortage of material, but also because it yielded cylindrical samples which could be readily used for the strength

tests after some minimum trimming of the ends. Since the same procedure was used the moisture density relationships shown here are valid. In future research this procedure can be adopted, but it would be advisable to compare the compaction energies with that of standard and modified Proctor as well as some field compaction results. The compaction data provided by the Ministry of Transport (St. Andrews, Gravelbourg, and Leader) were for the standard Proctor tests (ASTM D 698-70).

4.4.5 Unconfined Compression Tests

These tests were done on cylindrical samples extruded from the Miniature Harvard mold after the compaction tests were completed. The ends of the samples were trimmed square and parallel and height to diameter ratio of minimum 2 was maintained. In the case of Shelby tube samples, the specimens for the strength tests were obtained by pushing a thin walled tube through the extracted cores. The samples were pushed from the thin walled tubes with a small hydraulic samples extruder. The compression testing machine was a Soiltest apparatus using compressed air to control the movements of the platens. The rate of strain was 0.15 inch per minute. ASTM D 2166 requires that a strain between 0.5 and 2.0% per minute (0.5 and 2.0 mm/min) be used. It also requires that the test be completed within 10 minutes to prevent excessive drying of the samples. It was found that the latter criterion governed for samples from Shelby tubes, and a strain rate between 2.5 and 3.8 mm/min was used. This was adopted as the minimum rate for all tests. This rate allowed tests to be completed within 10 minutes, thus avoiding drying of the samples at the slower rate. However, it is felt that the higher rate of strain does not affect the results seriously.

4.4.6 X-ray Diffraction

The X-ray diffraction analyses were done on a Philips 1200 diffractometer (Figure 30). The details of the setup were as follows:

X-ray	Cu K- α
Filter	Nickel
Power	40 kV: 20 mA
Scan Rate	$2\theta = 1^\circ/\text{min.}$ or $2\theta = 1/8^\circ/\text{min.}$
Chart Speed	300 mm/hr.
Suppression	zero
Time Constant	4 for scan rate $2\theta = 1^\circ/\text{min.}$, 16 for scan rate $2\theta = 1/8^\circ/\text{min.}$
Z-factor	3
Frequency	400 cps, in some cases 1000 cps was used

Most of the samples used in this study were of the oriented slide type. Initially both powder mounts and oriented mounts were run on a few samples to determine whether this would make any significant difference. It was seen that no significant differences occurred. Hence in further analyses only oriented slides were used. The only exception to these were the X-ray patterns for Peltonite, bentonite, lime and portland cement (unhydrated). In the case of Peltonite and bentonite it was impossible to get good slides because of the enormous swelling and gelling action exhibited by these materials. In the case of lime and portland cement the intention was to obtain patterns in their unhydrated states.

All X-ray analyses were done on material passing No. 140 sieve. For powder mounts the samples were ground with a ceramic

pestle and mortar and sieved through a No. 140 sieve. The fraction passing No. 140 sieve was held between two sheets of cellophane tape in the slot of an aluminium sample holder. For the oriented mounts, the samples were wet ground in an agate mortar using an agate pestle and washed through the No. 140 sieve. The wet grinding and washing through the sieve were done repeatedly till a sufficient quantity of slurry was passing through the sieve. The slurry was stirred well and a few drops were placed on a clean glass slide and spread around by tilting the slide from one side to another. It was allowed to stand for a few minutes until the soil particles settled to the plate and excess water was standing on top. By slowly tilting the slide this excess water was decanted leaving a smooth thin film of soil on the slide. The slide was air dried for about an hour when most of the water evaporated. It was then placed in a vacuum chamber under 1 atmosphere vacuum (76 mm of Hg) for about an hour or two when it was ready for testing. This process generally yielded slides with a smooth coating of the minus 140 fraction without any cracks or shrinkage.

4.4.1.7 Differential Thermal Analysis

The differential thermal analysis was performed in an analyzer Model DTA-13-M manufactured by Robert L. Stone and Co. of Austin, Texas (Figure 31). The thermocouples were made of platinum and platinum + 10% rhodium. The reference sensor was also of the same material. The two differential thermocouples were arranged symmetrically in the horizontal plane and extending at about half depth in the sample holders. The sample holders were two cavities of inconel approximately 10 mm in diameter and 13 mm deep, built in a ceramic block. The cap was made of the same material as the

cavities. The apparatus had provision for testing in dynamic gas atmosphere (O_2 , N_2 , and CO_2).

The apparatus was acquired late in the programme and hence all the samples were aged between six and seven months. The samples were ground dry with a ceramic mortar and pestle and sieved through No. 140 sieve. The powder was placed in the cavity of the sample holder in two lifts. After each lift the soil was packed by tamping lightly with a glass rod about 9 mm in diameter and 15 cm long under the action of only the weight of the rod. The reference material used was $-Al_2O_3$ passing No. 100 sieve. Because of the small quantities involved, the samples were not weighed.

The samples were heated with a furnace voltage setting between 54 and 56 mV. The scanning range used in most cases was 500 μV . However, in the case of lime and bentonite which gave large endothermic peaks the setting was altered to 800 μV . Both data are presented (Figure 82).

It was observed that the apparatus did not have a constant rate of heating although Wicks (1965) had used the same DTA-13 M type analyzer and reported that the apparatus heated at a constant rate of $10^\circ C$ per minute. The $10^\circ C$ per minute rate appears to be the general rate of heating for soil samples as reported in the various research reports (Mackenzie, 1970; Kodama and Brydon, 1966; and Brady, 1957). Figures 32 and 33 show the regression equation fitted to the actual heating rate observed in the 70 odd runs during this investigation. The equation obeys a power law. On a log-log scale (Figure 33) the relationship appears as two straight lines. To obtain a constant rate of heating would require modification to the equipment, not justified considering the age (about twenty-five years) of same.

The samples were heated to 700° C only, though montmorillonites and kaolinites are supposed to show their dehydroxylation endotherms and an exotherm between 800° C and 1000°C. Wicks (1965) obtained such endotherms for the lake Agassiz clays (Figure 34). The reaction products CSH-I, CSH-II, and tobermorite like gels generally give a broad endotherm below 300° C and minor ones in the neighbourhood of 900° C. Despite the possibility of missing these peaks the samples were not heated beyond 700° C because it was feared that the platinum-rhodium thermocouples might be damaged if heated above 700° C (manufacturer's suggested limit in an inert atmosphere was 1200° C). In future research higher temperatures should be attempted probably using a different thermocouple whose limit is not too close to 1000° C. No dynamic gases were used in these investigations.

4.4.8 Scanning Electron Microscopy

Though about forty samples were prepared for observation under the microscope, only three were actually observed. One sample was the clay from Winnipeg with 2% lime added to it and the other two were from the Shelby samples from the Saskatchewan Highway test sections (sample no. 9325).

The instrument used was a Cambridge Stereo Scanner MK-II-A which belongs to the Department of Plant Science at the University of Manitoba. The photographs were taken at a power output of 50 kV by the Department's technician, Mr. Bert Luit.

Two types of samples were prepared:

1. Samples from the shear face of compacted and sheared specimens;
2. Samples as compacted.

Shear Samples: The soil was compacted in the Harvard Miniature apparatus and sheared in unconfined compression as described before. Two small nuggets about 5 to 6 mm cube, one parallel and the other perpendicular to the shear face, were then cut out and mounted on aluminum stubs with Electrodag 416, a silver based cementing compound manufactured by Kent-Cambridge Co. of Montreal, Quebec. The top surface was protected from dust and other extraneous matter by placing a piece of cellophane tape over it. The stubs were kept in a vacuum chamber under 1 atm vacuum for about a week. At this time they were taken out, the tape removed, and coated with 500A gold with a Balzer sputter coater. The samples were then returned to a vacuum chamber till ready for use. The micrographs were taken with a Panatomic X (ASA 32°) black and white film using a 36 mm camera. The scanning screen was 108 mm x 108 mm. The magnification factor is shown on each of the micrographs.

Compacted Samples: The soil as compacted was broken into two parts by slightly bending the cylindrical specimen. From the central parts two nuggets about 10 mm cube were then isolated and mounted as before. While the soil was still plastic a groove was cut all around the sample at mid-height. The sample was allowed to dry in the vacuum chamber. The dry sample was then broken off at the grooves thus exposing a fresh face. The face was protected with a cellophane tape till ready for sputtering with gold. The rest of the procedure is the same as above.

4.5 RESULTS AND DISCUSSIONS

4.5.1. pH Values

Figure 35 shows the variation of pH with lime content and with aging. Addition of lime to the soil raises the pH value to well above 11.0 and it stays at this value for many months. The "long term" value shown on Figure 35 was read on six to seven month old samples. It is also interesting to note that the long term pH value for low lime content (less than 4%) is lower than for higher lime contents. It is suggested that below 4%, the lime or more precisely the Ca^{++} ions are being used up rapidly in the amelioration effect and hence the pH drops to below 10.0. But above 4% the Ca^{++} requirements for amelioration effects have been largely satisfied and hence the pH remains high at least for months. Cores taken from the test sections by the two provincial Departments of Highways show pH values of 7.6 for the Lake Regina clays and 7.9 for the Lake Agassiz clays. pH values for the natural clays were 6.9 - 7.0 for the Regina clay and 7.3 - 7.5 for the Winnipeg clay. This would suggest that the excess Ca^{++} ions would eventually leach out or more likely form CaCO_3 by reacting with the atmospheric carbon dioxide. X-ray diffratograms definitely show calcite peaks, but from the intensities it is not possible to say whether the material from the test sections had any more calcite than natural soils. Moreover one of the reaction products, tobermorite, has a basal spacing of 3.05 A which could enhance the calcite peak present.

Glenn and Handy (1963) have used very elaborate viscosity flow measurement to measure the change in plasticity associated with lime. However, the plasticity index is a recognized standard of measure. It was therefore decided to use this plasticity index as the measuring criterion for changes in flow properties.

4.5.2 Grain Size Distribution

The grain size distribution of Lake Agassiz clays is shown in Figure 36 and that for the Lake Regina clays in Figure 36A. These are for the 28 day loose cured samples only. However, since no significant changes are likely to take place between a few hours after mixing and 28 days, it is suggested that nearly the same distribution would have been noted even during earlier stages of aging of the samples. Also shown are some of the samples from the test sections built by the two Highways Departments. As mentioned earlier some of these samples were still brittle and noncohesive. Hence it would appear that the agglomeration that was noticed immediately after mixing lime to a soil is permanent and irreversible. The three electron micrographs taken during this study support this view. These show massive agglomeration and cubical structures rather than the flaky ones reported in literature (for example, compare Baracos, 1977).

4.5.3 Plasticity

The effect of lime on the plasticity of the Lake Agassiz clays is shown on Figure 37 and that for the Lake Regina clays on Figure 38. Included with these results are tests on St. Andrews clays (Lake Agassiz) and from Gravelbourg (Lake Regina). Figure 39 shows similar results for the clay from Leader in southwestern Saskatchewan. This clay was deposited by glacio-fluvial process.

All the clays show a drastic reduction in the plasticity index on the addition of lime. The increase in the plasticity at 4% lime for the St. Andrews clays cannot be explained and probably represents an experimental error. For both Lake Agassiz and

Lake Regina clays the reduction in the liquid limit is much smaller than the increase in the plastic limit. In the case of the clay from Leader the decrease in the liquid limit is much more significant than for the other two clays. The increase in the plastic limit is correspondingly smaller than for the other two. It is suggested that this difference in behaviour between these clays is due to the different morphology of these clays. Though all these clays are derived from montmorillonite rich Bearpaw shale the different modes of their deposition and the present day environment account for their difference in behaviour. There has been no study reported in the literature linking the behaviour of lime modified clays to their geological origin except in one case of a loess soil where the results were linked to the aeolian nature of the deposits.

Thompson (1966) attempted to characterize the behaviour of soils from Illinois by determining regression equations for the various properties. Since his study many have done similar analyses. A similar exercise was attempted here too. In order to have a broader range of data, previously published results from Young, Culley, and Klym were also used. The results show:

For Lake Aggasiz clays: $I_p = 61.22 - 5.83 L\%$: $r^2 = 0.80$

For Lake Regina clays: $I_p = 44.34 - 4.63 L\%$: $r^2 = 0.87$

where I_p = plastic limit and $L\%$ is lime content.

When the results from all the sites were used in an attempt to find one common equation the correlation was very poor, suggesting the influence of morphology and environment. When one considered only lacustrine clays the correlation improved but still remained less than for individual clays. This demonstrates that lacustrine clays behave differently from clays of glacio-fluvial origin; between the

two lacustrine clays the differences are probably due to the origin and the present day environment.

It should be emphasized here that these results are based on limited test results and their validity should not be over-emphasized. A future research programme should undertake to evaluate these factors in a more systematic way.

4.5.4 Volume Change Properties

The effect of lime on the volume change properties of these clays was not investigated in this programme. However, a qualitative idea of this effect may be gained by the use of Williams' chart linking swell properties to plasticity and clay fraction (Figure 40).

This figure shows the volume change potential of the three clays in the natural state and when treated with lime. The results would indicate that addition of lime drastically reduces the swell potential of these clays. As the lime content increases the potential for swelling decreases from the "very high" range to "low" range. With 10% lime the soils exhibit virtually no swelling at all.

4.5.5 Moisture-Density Relationship

Figures 41 to 43 show the results of compaction tests for Lake Agassiz clays. Similar relationships for the Lake Regina clays are shown on Figures 44 to 46. Figure 47 shows the test results for the Leader clay. Whereas the Lake Agassiz and the Lake Regina clays were compacted in a Harvard Miniature mold, the clays from Leader were compacted in the Proctor mold using both standard and modified methods. From the relationships between the Harvard Miniature, standard and the modified Proctor methods, it appears that the assumption regarding the compactive energy of the Harvard apparatus

is valid. Using the 40 lb. spring, 10 tamps per layer, and five layers, is equivalent to the standard Proctor compactive energy.

The results (Fig.48) show that for a given compactive effort lime treated soils attain less maximum dry density than untreated soils. The reduction in density is of the order of 0.3 kg/cm^3 . The reduction in density is more pronounced for lower lime contents (less than 4%) than for higher lime contents. The optimum moisture content is greater for lime treated clays than for untreated soils (Fig.49). Again the effect is more noticeable at lower lime contents than at higher lime contents. Indeed, in some cases the optimum moisture content decreases.

While all the clays show this effect there are subtle differences between the different clays. For the Lake Agassiz clays the reduction in the density continues up to 6% of lime before levelling off while for the Lake Regina clays the greatest reduction in density occurs at about 3% lime content. For the Leader clay (the results are not shown plotted) the critical percent of lime appears again to be 3% (see Fig.48).

The variation of OMC with lime is somewhat erratic, particularly the 3-day curve (Fig.49). No reasonable explanation can be offered for this. It would appear that the variation of OMC for Lake Agassiz clays is somewhat steeper than for the Lake Regina clays. The significance of this conclusion is that the Lake Agassiz clays have a more flocculated structure than the Lake Regina clays. This could be due to their different origin, mode of deposition, and the present day environment. Unfortunately this could not be verified by electron microscopic studies during the present programme. A future research programme should attempt to observe the structure of these two clays under the high magnification possible with an electron microscope.

Finally, the OMC appears to be levelling off to a maximum between 35% and 40%. That is, beyond a certain lime content the OMC does not increase. It is of interest to note that the saturation water content in the Prairie clays is in this range too. This means that addition of lime does not change the saturation water content, or the long term soil moisture regime.

Aging does not seem to have any significance on the dry density or the OMC of these clays. The 3, 7, and 28 day curves are all in the same range. No 24 hour compaction results are available from these series. Thus the difference between compaction after 24 hours and after 3 days could not be assessed. Such an assessment would be important from the practical point of view because if the delay of two additional days does not contribute substantially to decreased density it may be well worth the time to let the soil cure for some additional time resulting in a product which is more uniformly reacted upon and easier to handle. It will be demonstrated in the following section that whatever loss of density the soil might suffer by delayed compaction it can be more than compensated for by using a higher compactive effort.

Figures 50 and 51 show the effect of higher compactive effort on the compaction characteristics of the two clays. The higher density and the lower OMC are easily recognized. No such comparative results are available for the Leader clay. But by comparing the standard Proctor curve for the natural clay and the modified Proctor curves for the treated clay, one may infer that the density from standard Proctor compaction for lime treated clays would be much less than for the modified Proctor samples.

A question often arises as to how to compact such clays. It is usual practice to compact untreated clays at 2 to 3% wet of optimum using standard Proctor compactive energy. But in the case of lime treated clays this might result in very low densities and possibly lower strengths. Therefore it is suggested that lime treated clays be compacted with modified Proctor energy at or near the optimum water content. Since clays even with moderate lime contents have only low to medium swell potential, it is unlikely that such soils would exhibit any detrimental swelling due to later intake of water.

An attempt to fit a regression curve to the compaction data yielded an equation of the type:

$$\gamma_d = C.(L\%)^{-n} \text{ where } C \text{ and } n \text{ are constants and } L\%$$

is the lime content in percent by dry weight. The values of the constants for the different clays are as follows:

	C	n	r ²
Lake Agassiz clay:	1.30	0.008	0.84
Lake Regina clay:	1.59	0.011	0.99
Leader clay:	data too meagre for analysis		

4.5.6 Strength and Bearing Values

The results of unconfined compression tests for the Lake Agassiz clays are shown on Figures 52 to 54 and that for the Lake Regina clays on Figures 55 to 57. The results show that all the clays have a strength between 1.0 and 3.0 kg/cm² (10.0 to 30.0 kPa) regardless of the age of the sample. Therefore loose curing does not contribute to any increased strength. Some samples with low lime contents show higher strengths at early stages, suggesting that

the earlier the compaction, the higher would be the strength for the same compactive effort.

The shape of these curves suggested a comparison with the moisture density curves. Such a comparison would lead to the following inferences:

1. At low moisture contents (less than 20%) the samples showed low strengths. They were dry, crumbly and had brittle failures.

2. As the moisture contents increase, the strength increases until a moisture content approximately 3% to 5% less than the corresponding optimum moisture content is reached. Beyond this point the strength progressively decreased.

3. For the type of testing carried out (loose cured and compacted) the clays with lower lime contents yielded higher strengths than those with higher lime contents. This is thought to be the effect of loose curing. Hence lime treated soils should not be left uncompacted for prolonged periods or alternatively they should be compacted with higher compactive energy than standard Proctor effort.

The last mentioned conclusion is clearly demonstrated by the results shown on Figures 58 and 59. These compare the strengths of natural soil compacted by standard Proctor method (mostly wet of optimum), lime treated soils compacted in accordance with standard and modified Proctor methods. First it is clear that regardless of the compactive effort and method of curing, lime treated soils show higher strengths than natural clays. But the treated soils which were compacted after 3 and 7 days of loose curing show no definite relationship and plot all over within a narrow band (shaded area in

Figure 58). In contrast the results of modified Proctor tests which were carried out within 24 hours of mixing show much higher strengths. They also show a definite pattern: the higher the lime content the higher is the strength. The strengths of the clays compacted with higher compactive efforts show high strengths even at water contents as high as 35% to 40%.

Hamilton (1963) has shown that for the clays in the Canadian Prairies the saturation or equilibrium moisture content is between 35% and 40% unless the land is heavily irrigated. Hence lime treated Canadian Prairie clays, particularly at high lime contents in the order of 10% or more, can show unconfined compression strengths of 80 kPa or more, even at saturation water contents. However, it is not clear whether lime treated soils would tend to achieve a higher equilibrium moisture content in which case there could be a reduction in strength. Further research is needed as to the suction potential and moisture migration in lime treated soils. They may be important not only from the strength point of view as discussed above, but also from the durability point of view as suggested by Townsend and Klym, (1966).

Figure 59 shows the results of Figure 58 from another perspective. This figure demonstrates that lime treated soils can be allowed to mellow for some periods (a practical time will be 3 days) and compacted with higher compactive effort to obtain considerably higher strengths than would be possible with standard Proctor efforts. While Figures 50, 51, and 59 demonstrated this conclusion for Lake Agassiz and the Lake Regina clays, no such data was obtained for the Leader clay in this test programme. But the same conclusion should also be valid for this clay too.

Figure 60 shows the results of unconfined compression strengths for the Leader clay treated with 3%, 5%, and 7% lime. The results of the 7% treated clay do not follow the general trend and are very erratic. No reasonable explanation can be offered for this, particularly when one compares these results with the CBR results shown on Figure 62.

Figures 61 and 62 show the results of the CBR tests on Lake Regina and Leader clays. These figures show that lime treatment increases the bearing values considerably. The soaked CBR values which are predictably less than the dry values are still considerably higher than that of natural soils, especially for lime contents greater than 6%. No CBR tests were done on the Lake Agassiz clays.

A regression analysis did not show any relationship between lime, moisture content, and compressive strength or CBR. As is known from a review of the literature, strength and CBR depend on a host of other factors such as curing time and temperature, compactive effort, etc. A future research programme should evaluate these factors systematically for these clays.

The strength tests showed that a majority of samples broke in brittle failure. The slope of the stress-strain curve was steep. These suggest that lime treated soils would behave like brittle material and would possess a high resilient modulus.

4.5.7 X-ray Diffraction Analysis

All the X-ray traces obtained in this investigation have been re-arranged in such a way that they can be readily compared with each other and the influence of lime on the clay minerals may be studied (Figures 63 - 74). Table VIII shows the peaks obtained with these samples.

No quantitative analysis was attempted here. With a complex material such as these clays, X-ray analysis can be at best only semi-quantitative. Even then, it is possible only when an internal calibration with an inert material has been carried out. When one considers lime treatment of these clays then the inert material should not react with lime. Quartz would probably meet these requirements. Wicks (1965) and Last (1974) have reported on such techniques on Lake Agassiz clays. But from their conclusions one can also infer that an approximate idea of the relative quantities of the minerals can be gained by comparing the intensities of the different peaks with one standard peak. Using these and other concepts reported in literature (e.g., Klug and Alexander, 1974), the following procedure was adopted in this investigation. In each diffractogram the 3.36 Å quartz peak was taken to be the reference and was assigned a value of 100% intensity. The heights of other peaks were then compared to the quartz intensity and expressed as a percentage. Assuming that the quartz would be unaffected by the lime treatment the other changes either in the intensity or in the spacing would be due to the reaction of lime added. Also, the intensity of the new peak or the enhancement of a peak would be a measure of that

reaction product.

Analysis of the natural clays from Lake Agassiz and Lake Regina basins showed the presence of the following minerals:

	<u>Lake Agassiz</u>	<u>Lake Regina</u>
Montmorillonite	14.72 A	12.8 A to 17 A
Illite	9.2 A and 1.55 A	1.55 A
Kaolinite	7.25 A, 4.50 A, 3.59 A	7.1 A, 4.52 A
Quartz	4.29 A, 3.36 A, 2.45 A, 2.29 A	3.37 A, 2.49 A, 2.30 A
Dolomite	2.9 A, 2.2 A	2.9 A
Calcite	3.05 A	3.18 A
Feldspars	3.71 A to 4.23 A	3.7 A to 4.23 A

In addition, some minor unidentified peaks were obtained. Also some illite could be occurring as mixed layer minerals at 5 A.

On treating the samples with ethylene glycol, the montmorillonite peaks were not usually picked up within the scanning range of $2\theta = 4^\circ$ to 60° . Presumably the layer expanded to beyond a spacing of 22 A. On heating the clays at 550° C the peaks collapsed.

Addition of lime to these clays had several effects:

1. Reduction in the relative intensities of the clay minerals;
2. A small increase in the basal spacing;
3. Enhancement of the 3.05 A calcite peak;
4. Appearance of new peaks which could be due to the formation of such minerals as tobermorite, CSH I and CSH II,

ettringite and hydrogarnet.

4.5.7.1 Reduction in the Relative Intensities of the Peaks of Clay Minerals

Table VII shows the basal spacings and the relative intensities of the peaks due to montmorillonite, illite, and kaolinite. The table shows that the intensities of the minerals (compared to 3.36 A quartz) are reduced, but montmorillonite is the most affected mineral, the reduction in the intensities being of the order of 50% or more. The disappearance of the 9 A - 10 A illites show that the illites occur as interstratified with montmorillonites. The results from Wicks (1965), Kodama and Brydon (1966), and Quigley (1968) confirm these findings. In the interstratified layer montmorillonites must be the dominating mineral because illite is not very reactive with lime (Eades, 1962). Therefore the fraction of pure illites cannot be very high in these clays. It is suggested that the reduction in the peak heights of montmorillonite and the disappearance of the interstratified illite-montmorillonite peak are due to the attack by lime on these minerals and the formation of the new minerals shown in Table VIII.

4.5.7.2 Increase in Basal Spacing

Addition of lime appears to shift the peaks towards lower angles or greater basal spacings. Results from other researchers show such results sometimes (e.g., cf. Laguros and Handy, 1965), though never discussed. The shift in basal spacing is surprising at first glance because it was demonstrated earlier that addition of lime to these clays causes agglomeration of particles. Some of the

explanations offered for the shift are:

1. The reflections due to montmorillonites depend, to some extent, on the humidity of the atmosphere. Hence the experimental conditions could be causing these shifts in the location of the peaks.

2. Some of the reaction products, notably tobermorite, can show basal spacing between 9 A and 17 A, depending on the humidity and degree of crystallinity. These can also be interstratified with the unreacted montmorillonite and thus cause shifts in the peaks.

3. The nature of the lime reaction with the minerals is discussed in some greater detail under the following section on DTA. Suffice it to say here that if one accepted the theory by Diamond and Kinter (1965) then the adsorption of the $\text{Ca}(\text{OH})_2$ on the clay surface would displace the clay platelets apart resulting in increased basal spacings.

It is suggested that some of the peaks could be in reality tobermorite and not montmorillonites. This question is not resolved in this investigation, but a future research may attempt measuring the cation exchange capacity and surface area measurements. Whereas montmorillonite will have a CEC of 80 meq/100 g, tobermorite and CSH like silicates will show CEC between 5 and 25 meq/100 g, depending on the degree of crystallization.

4.5.7.3 Enhancement of the Calcite Peaks

Calcite is a naturally occurring mineral in these clays and it shows up as a strong to very strong peak at 3.05 A.

Tobermorites and hydrogarnets also give peaks at 3.05 A. While there exists a possibility that the enhancement of the calcite peak could be due to the carbonation of the lime added, this possibility is discounted here because it was demonstrated earlier that the method of preparation and storage of samples could not have permitted any significant degree of carbonation. Therefore it is believed that the enhancement of the 3.05 A peak is largely due to the formation of tobermorite or similar silicates rather than calcite.

Figure 75 shows the relative intensities of the 3.05 A peak for the different samples. The figure shows a difference in the behaviour of the two clays. The enhancement of the calcite peaks is much more pronounced in the case of Lake Agassiz clays than for the Lake Regina clay. Based on the results presented here, the following inferences can be made:

1. In both clays there is evidence that tobermorite like reaction products are formed on adding lime to the soil.
2. The products are not detectable at low lime contents but can be definitely recognized at higher lime contents (8%). W-10-3 appears to be contradicting this trend.
3. The relative intensities seem to get higher with the age of curing in the early stages, but appear to decrease in the long term. On the other hand a sample of peltonite treated with 10% lime and cured for various periods of time showed a continuous increase in the intensity of the 3.05 A peak. The relative intensities were 26.1% and 53% (taking the quartz at 100%) for 3 days and 9 months respectively. The untreated peltonite did not show any

peak at 3.05 A. This demonstrates that lime reacts with the montmorillonite in the peltonite and yields a reaction product with a 3.05 A basal spacing; that this product continues to grow with age provided the hydration conditions are right. It may be argued that the intensities of the peaks in a diffractogram does not necessarily have any bearing on the amount of that mineral. But this is likely to give a relative idea of the quantities when the experimental conditions are kept the same during the different runs.

In order to ascertain whether tobermorite would continue to form and whether the 3.05 A peak will get stronger with time, X-ray patterns of normal Portland Cement were obtained for fresh powder, and hydrated pastes after 1-1/2 hours, 2, 3, 7, and 28 days. The traces are shown on Figure 76. The first three pastes (1-1/2 hours, 2 and 3 days) were air cured while the 7 and 28 day samples were moist cured by keeping the slides under water in a covered petri dish (100% saturation condition). The slides were air dried just prior to testing on the day of testing. The results throw some light on the behaviour noticed with the lime treated clays.

First the formation and growth of any mineral during the hydration of Portland Cement cannot be studied by using an internal standard because there was no such mineral in Portland Cement. All the minerals in Portland Cement can undergo hydration and hence change their intensities and basal spacing. However, it was noticed that a peak at 2.78 A (probably some form of alumino silicates) was relatively stable. Therefore the 3.05 A peak was compared with this

peak after hydration for 1-1/2 hours, 2 and 3 days. The relative intensities for the fresh cement, 1-1/2 hours, 2 and 3 day samples were 62%, 76%, 68%, and 97% respectively. It should be pointed out that the fresh cement showed a moderately strong peak at 3.05 A which could be calcite formed due to carbonation during the storage of cement. The increase in the I/I₀ was not significant during the first 48 hours and for air curing. However, when the samples were moist cured under 100% saturation condition, the 3.05 A peak grew rapidly and what is more, the 2.78 A reference peak disappeared. This showed that tobermorite would grow under moist or saturated conditions; that as curing time increased so would its intensity. This fact is, of course, well recognized by the cement technologists and that is why fresh concrete is cured under moist conditions for 28 days. Once this fact was recognized, it was possible to explain why the 3.05 A tobermorite in the lime treated clay did not show any rapid growth with time of curing. The method of preparation and storage of the lime treated samples precluded any saturated atmosphere to cure in. Thus in the initial stages of the formation of tobermorite there was sufficient excess water for the mineral to form. However, at later stages there was not enough moisture for this reaction to grow rapidly. Thus the 3.05 A did not show much spectacular growth.

The above reasoning demonstrates the importance of the curing condition in the development of strength in lime treated clays. This has been pointed out by many previous researchers (refer to Section 3 on Review of Literature). This also explains why

delayed curing would not result in higher strengths or even inhibit the development of strength. For, the water added during the mixing process is likely to be well below that required for the proper and complete development of the reaction products. While in the first day or two the mixture might show some gain in strength, it is likely to level off or even decline with prolonged loose curing because the conditions are not conducive to the development of these reaction products and the strengthening bonds.

4.5.7.4 Other Reaction Products

In addition to the 9 A and 14 A tobermorites and the 3.05 A tobermorites, other reaction products of the type ettringites and hydrogarnets and unspecified aluminosilicates were also identified. Table VIII lists these products for the two prepared soils as well as for the soils recovered from the highway test sections constructed by the two provincial Departments of Highways.

Since many of the reaction products have the same or very nearly the same basal spacings as some of the soil minerals it was difficult to distinguish some of the peaks. In an attempt to obtain a better resolution of the overlapping peaks, the samples were scanned at a lower speed of $2\theta = 1/8^\circ$ per minutes. These diffractograms are shown on Figures 77 and 79.

4.5.8 Differential Thermal Analysis

Results of the DTA for the prepared samples from the Lake Agassiz and Lake Regina basins are shown on Figures 80 and 81. Individual thermograms are included in Appendix IV. Figure 82 shows

the thermogram for the quicklime used. Figure 83 is the DTA curve for "Peltonite" while the results for bentonite are shown on Figure 84. As mentioned earlier the rate of heating was not constant. The effect of this was a drift in the thermogram. This combined with the possibility of a glassy product being formed (Wicks, 1965) made the interpretation of these curves uncertain and difficult, if not impossible. It was difficult to distinguish the legitimate peaks and shoulders from the inflexions and steps caused by the baseline drift.

The technique used in the analysis of these curves followed the reasoning proposed by Laguros and Handy (1965). All clay minerals had some hygroscopic moisture picked up from the atmosphere. In addition they have their interlayer and the adsorbed layers of water. The thickness of these layers and hence the thermal energy to drive these off depends on the clay mineral, cation substitution, and the pore water chemistry. In the DTA the various stages do not appear as well defined consecutive peaks but as a continuous curve with small shoulders and inflexions to denote the evaporation of a particular type of water from the soil. Therefore the DTA curves of most natural soils will be in reality a series of overlapping peaks. Laguros and Handy propose a certain method to calculate these areas. The necessity for this particular method is not clear. But their concept of overlapping peaks is accepted as valid. These are then separated into individual peaks. The shape of the peaks is somewhat arbitrary but is done in such a way so that the slope ratio (Bramao et al, 1952) is 1.0. The slope ratio of 1.0 is taken to be correct judging from the thermogram for peltonite and bentonite. Figure 85

illustrates this concept. Table IX shows the areas obtained by measuring with a planimeter.

Mackenzie (1970) described the following reactions on heating an interlayered illite-montmorillonite:

1. Dehydration peaks between 100° C and 300° C (Endothermic);
2. Dehydroxylation between 500° C and 600° C (Endothermic);
3. Structural breakdown between 800° C and 900° C (Exothermic and Endothermic).

Similar results have been reported by Kodama and Brydon (1966) for the prairie clays from Alberta, Saskatchewan, and Manitoba. Each of these reactions are discussed below using the thermograms obtained in this investigation.

4.5.8.1 Dehydration Peaks

The dehydration peaks between 0° and 300° C represent the various stages in which the water from the soil is driven off. It should be emphasized that these stages do not occur one after another but could be occurring simultaneously. However, one particular reaction will be dominating in one temperature range. With this in mind we can identify the different stages as:

1. Evaporation of hygroscopic moisture;
2. Removal of interlayer water;
3. Removal of adsorbed water.

Hygroscopic moisture is the water held by the soil primarily as a result of atmospheric humidity. It depends on the fineness of the

material rather than on the mineralogy. It begins to evaporate as soon as the temperature exceeds the room temperature and passes into the vapour phase completely at about 100° C. Therefore, for a given fineness of the sample the peak area of this stage should be more or less the same. Furthermore the shape of this peak would be similar for all samples. This is demonstrated by the thermograms in Appendix IV and the areas presented in Table IX. In this investigation all samples were passed through a No. 140 sieve and the test conditions (temperature and humidity) were more or less the same. The small variations can be explained by the actual cumulative fineness of the minus 140 fraction, weight of the sample taken, and some judgemental factors in drawing the peak.

Above 100° C the interlayer water begins to dissociate from the clay surfaces and goes into the vapour phase. The energy for this reaction depends on the force with which the water is held between the clay platlets and hence is a function of the mineralogy of the clay. Montmorillonitic clays generally yield a sharp narrow peak with the peak temperature between 160° C and 200° C. For example, one can compare the peaks obtained for the peltonite and for the bentonite (Figures 83 and 84). The peaks due to this stage of the reaction with the two clays tested in this programme are shown on the thermograms in Appendix IV. The areas are given in Table IX. These results suggest some interesting inferences. For the Lake Regina clay there is a significant reduction in the area when 2% lime is added to the clay. Above 2% there is reduction, but not very significant. This result can be directly related to the effect of

lime on the plasticity of the clay. For the Lake Agassiz clays reduction in the peak area is not significant till higher percentages of lime have been added to the soil. Also, the area under the peak for the natural clay is significantly smaller than for the Lake Regina clay. This could be the effect of the bedrock mineral in these two areas. The Lake Agassiz basin is underlain by dolomitic limestone whereas under Lake Regina the bedrock is reportedly sandstone. Thus the Lake Agassiz clays are likely to be dolomitic with less interlayer water. Referring to X-ray traces, one can see that the basal spacing of the montmorillonite in the Lake Regina clay is somewhat larger than for the Lake Agassiz clays. This may be verified in a future research programme.

Heating the sample above 200° C begins to affect the adsorbed water or water which is held more tightly to the clay surface. The thermograms show two or three shoulders or inflexions in this range. It is suggested that these represent successive layers of water. Referring to Table IX one notices the differences in the behaviour of these clays. The differences are in the magnitude of the areas and the amount of lime needed to effect significant reductions in the area. The similarities are that both the clays show reduction in the peak areas with increased lime contents. It may be noted here that the sample W-4 is somewhat at odds with the other samples. No reason can be given for this behaviour.

The progressive reduction in the areas with the addition of lime could mean that the lime replaces the water as Ca(OH)_2

molecules are physically adsorbed on the surface of the clays, as suggested by Diamond and Kinter (1965), or as Ca^{++} ions are adsorbed on the surface as postulated by Handy et al (1965). It would be interesting to speculate on these two theories. If one accepted the concepts by Diamond and Kinter, then it would mean that $\text{Ca}(\text{OH})_2$ simply displaces the water layer further away from the surface. In other words the lime simply pries clay surfaces apart. Thus, in effect, the basal spacings would increase which should be measureable by X-ray diffraction. It will be recalled that one of the effects observed on X-ray traces during this investigation was an increase in the basal spacing. However, this would also mean that there should be no reduction in the energy needed to drive away this water. In other words the area of the peak for lime treated clays should not be any different from that of natural clay. This appears to be partly true for the Lake Agassiz clay but not for the Lake Regina clay. On the other hand if one would accept the theory by Handy et al then $\text{Ca}(\text{OH})_2$ is not physically adsorbed on the clay surface but ionizes in solution, with the Ca^{++} ions crowding to the surface displacing some H^+ of the water and probably some other exchangeable cations as well. Some of the OH^- forms hydroxides with the alumina and the silica dislodged from the structure of the clay in the highly alkaline environment. In this case the amount of adsorbed water progressively decreases as more and more Ca^{++} ions crowd to the surface. Thus the area of these peaks on the thermogram would also decrease with increasing amount of lime. The increase in the basal spacing is probably also due to the

other factors discussed when discussing the X-ray diffraction results. The question as to which hypothesis would be valid is not resolved by this investigation. Probably both are valid and the mechanism would depend on the nature of the clay as appears to be the case with these two clays. Perhaps a future research could link the type of reaction to the type of soil.

4.5.8.2 Dehydroxylation Peaks

Dehydroxylation peaks are reactions occurring when the OH^- ions in the minerals are dissociated from their lattice positions. This is different from driving off the water molecules which exist as H-O-H dipoles. The dehydroxylation requires more energy than dehydration and hence occurs at temperatures higher than the low temperature dehydration endotherms.

In normal montmorillonites the dehydroxylation occurs at temperatures between 600°C and 700°C . However, in the interlayered minerals where montmorillonite is the predominant mineral, this peak occurs in the neighbourhood of 550°C . Whereas the peaks of the normal montmorillonites are sharp, narrow and well defined, those of the interlayered montmorillonites (also referred to in the literature as "abnormal montmorillonite") are shallow and broad. Mackenzie (1970), has shown some peaks for interstratified illites and montmorillonites in general. Kodama and Brydon (1966) have reported on the results of DTA on the interstratified illite-montmorillonite clays from the Canadian Prairies. Their report does not contain the actual thermograms but the Fourier transforms of these. But they have identified the endotherms at 530°C . Wicks (1966) has also

reported on this endotherm for the Lake Agassiz clays.

On the thermograms in Appendix III and on Figures 80 and 81, these broad shallow endotherms due to the interstratified illites and montmorillonites have been identified between 500° C and 540° C. It may be noticed that these peaks become progressively smaller with the addition of lime till for lime contents greater than 8%, they practically disappear. It is suggested that this represents the same phenomenon as observed on the X-ray traces for these clays where the 9 A to 10 A peaks disappeared on adding lime to the soil. The disappearance of these peaks is due to the attack by lime and the breakdown of the structure of the clay mineral to form other reaction compounds.

4.5.8.3 Structural Breakdown

Structural breakdown or spline formation occurs at high temperatures usually between 800° C and 900° C, and in some cases near 1200° C. The peaks are generally in the form of an S-curve having an endotherm-exotherm-endotherm in quick succession. Since the samples in this investigation were not heated above 700° C, this could not be observed.

4.5.8.4 Other Peaks

It is interesting to note that the endotherm for quicklime (Figure 82) does not appear on the thermograms for the lime treated soils. Peaks for calcite generally occur at about 1000° C and hence could not have been recognized during this investigation. The quartz inversion peak at 573° C was also not observed on these endotherms. It should be remembered that these peaks were identified

on the X-ray traces. It is not understood why these peaks do not appear on the thermograms. Probably one may accept Wick's thesis (1965) that the formation of silica glass compounds above 500° C obliterates all these peaks. It will also be noted, however, that Kodama and Brydon (1966) do not mention or discuss any such phenomenon for the Canadian Prairie clays. These latter authors also report on obtaining an exotherm at 330° C for these clays. The thermograms obtained during this investigation do suggest an exotherm in this temperature range. Kodama and Brydon do not explain what this peak is due to. Such peaks are not obtained for the pure clay minerals or for the Bearpaw shale (Lambe, 1952) in general, though one sample of Bearpaw shale tested by Lambe does show a faint suggestion of an exotherm at 400° C. This sample had 70% montmorillonite and 15% illite while in the other samples illite was the predominant mineral (70% illite, 20% montmorillonite). The clays tested in this programme had montmorillonite between 55% and 70% and illites between 15% and 20%. Thus it appears that the exotherm could be a characteristic of interstratified montmorillonite-illite where montmorillonite is the predominant mineral. However, unlike the endotherm at 530° C this exotherm does not seem to be affected by the addition of lime. More work is needed to clarify this question. There is also some difficulty in assessing the effect of lime on this exotherm because the baseline drift due to the variable rate of heating starts just in this temperature range. Hence it is hard to say whether a certain inflexion is a legitimate peak or a shoulder or just the step wise drift of the baseline. Future research should try

to overcome this problem with the apparatus.

Mackenzie (1970) had published formalized peaks for tobermorites CSH I, CSH II, and other similar silicates (Figure 86). It may be noted that some of the low temperature endotherms discussed earlier could very well be due to these reaction compounds. Further chemical and thermogravimetric analysis would be needed to confirm whether the formation of the reaction products contribute to some of the low temperature endotherms. On Figure 86 one can also notice small, shallow and broad endotherms and exotherms in the temperature range 250° C to 320° C. The thermograms show a series of inflexions in this range. But as discussed above it is difficult to determine whether these are legitimate peaks or baseline drifts.

4.5.9 Scanning Electron Microscopy

The work done with the SEM during this investigation is minimal. Figures 87 to 89 show some of the micrographs obtained. Figure 87 is the photograph of the horizontal face of a compacted specimen of the clay from Winnipeg treated with 2% lime. One can still recognize the flaky platelets of clay (cf. Baracos, 1977), but these are not individual flakes anymore but aggregated to form larger particles. Also one can notice some edge-to-face flocculation of these aggregations. This micrograph suggests that lime does produce an aggregated flocculated structure but the individual platy configuration of the clay particles is still not completely obliterated by 2% lime.

Figures 88 and 89 show samples prepared from the cores obtained from the Saskatchewan Highways Department (Sample #9325).

Figure 88 is the shear plane of a sample prepared from the core and sheared in unconfined compression. One notices the massive agglomeration of the particles. There is no suggestion of the flaky structure recorded in the literature. Small granules of the soil can be discerned on the sheet like structure. There are also cubical nugget-like particles which Mitchell denotes "connectors" (Mitchell, 1976).

Figure 89 shows also a sample from the same Shelby core #9325. But this sample is from the supposedly untreated part of the subgrade below the lime treated upper part. There is some evidence of the typical flaky and curly montmorillonite particles but also considerable agglomeration possibly due to the migration of lime over the years.

The work is too limited to warrant any definitive conclusions. But it demonstrates that observations can be made with the SEM to understand the effect of lime on these clays. It is interesting that these micrographs do not suggest the formation of tobermorite-like products which have a lath-like structure. The works of Diamond, Kinter, and Ormsby (1974, 1975) in which such photographs were published were with pure clay minerals and at lime contents in the range of 20%, or with clay minerals immersed in lime slurry.

4.6 CONCLUSIONS

Some aspects of lime stabilization of the lacustrine clays found in the Canadian Prairies were investigated. The clays were obtained from Winnipeg and Regina and from the test sections built by the two provincial Highways Departments some fifteen years ago. Based on the tests reported here, the following conclusions are drawn.

1. A review of the literature on lime stabilization showed that tremendous 'amount' of systematic research 'has' been done in other countries, notably in the U.S.A. However, research on the subject in Canada has been minimal and not quite so systematic. Hence there exists a need for well planned research on the clays encountered in the Canadian Prairies.
2. Routine geotechnical tests confirmed, generally, results reported by researchers elsewhere.
3. Addition of lime resulted in the agglomeration of the clay particles. The clay fraction which was over 50% in these clays in their natural state was reduced to between 15% and 20% on the addition of lime. Cores taken from the highway test sections still showed particle agglomeration suggesting that this process is permanent. Some evidence of this was also obtained by direct observation of the treated soils with a scanning electron microscope.
4. Plasticity indices of the clays were reduced from above 40% for the natural clay to less than 10% for clays treated with 10% quicklime. Attempts were made to find a deterministic equation for plasticity as a function of lime added. The data from

this investigation is still meagre and hence needs to be refined by future research.

5. Using Williams' chart linking plasticity, clay fraction, and swell potential it was demonstrated that the very high swelling potential of these clays can be reduced drastically by the addition of lime. For high lime contents (greater than 8%) the clays appear to be practically nonswelling.

6. Compaction characteristics were modified by the addition of lime. The maximum dry density of the compacted clay was 0.2 to 0.3 kg/cm³ less than for the natural clay. The optimum moisture content increased by 10% to 15%. Both maximum dry density and the OMC tended to level off after certain lime contents, though not at the same lime contents. Regression equations are presented for maximum dry densities as a function of lime content. But the results are only tentative and need to be modified, based on more data from further research.

7. Type of curing and compactive effort played an important role in the densities achieved. Loose curing affects the dry density adversely. However, by using a higher compactive effort, higher densities can be achieved. Hence in practice a compromise should be reached between lower costs due to ease of handling, compaction and uniformity of material resulting from loose curing on the one hand and the higher cost of compaction necessitated by loose curing on the other.

8. Unconfined compression tests on compacted samples loose cured for 3, 7, and 28 days showed that lime treated samples have higher strengths than natural soils even at higher moisture

contents. Samples which were not allowed to cure in the loose state or which were compacted with greater compactive effort showed significantly higher strengths. If the samples are loose cured, age of curing does not contribute any increase in strength. Therefore lime treated samples should not be loose cured for prolonged periods unless higher compactive efforts are used in the final placing which would result in a densely compacted high strength material.

9. Most of the samples failed by brittle failure. The stress-deformation curves were very steep. This may suggest that lime treated clays could have significant flexural strength. However, durability and the effect of freeze-thaw cycles on the resilient modulus of these clays should be investigated further before evaluation of this property can be made.

10. CBR tests showed the same general trend as the strength tests. Lime treated clays have significantly higher soaked CBR values than natural clays especially at lime contents of 8% or more.

11. X-ray diffraction analysis showed that the montmorillonites (14 A to 17 A) and interstratified illite-montmorillonites (9 A to 10 A) were affected by the addition of lime to the soil. The peaks either diminished in intensities or totally disappeared.

Other reaction products could be identified, notably the 3.05 A tobermorite. However these could be only indirectly identified because of the overlapping of these peaks with the normal minerals found in these clays. Additional chemical analyses would

be needed to confirm these findings.

12. Differential thermal analysis showed that the area of the peak corresponding to the interlayer water between 160° C and 200° C decreased with the addition of lime for the Lake Regina clay, but no such relationship could be found for the Lake Agassiz clays. The "double layer" peaks showed progressive decreases in area for the Regina clay but only at higher percentages of lime for the Lake Agassiz clay. It is suggested that these differences could be due to the type of bedrocks in the two basins and the resulting cations in the clay structure.

On the other hand the low temperature endotherms could be also due to the calcium silicate hydrates formed during the reaction of lime with the clay minerals. Further work is needed to clarify this question.

The endotherm at 530° C due to the interstratified illite-montmorillonite disappeared on adding lime to the soil. This is in conformity with the behaviour exhibited by the X-ray traces where the 9 A to 10 A peaks disappeared.

The exotherm at 330° C reported by Kodama and Brydon (1966) for interstratified illite-montmorillonites in the Canadian Prairie clays was recognized. But it could not be ascertained whether addition of lime had any effect on this peak because of the baseline drift caused by the non-uniform rate of heating in the DTA apparatus. Future research should attempt to obtain a uniform rate of heating.

Likewise there were inflexions and shoulders

observed on the thermograms. These could very well be the endotherms and exotherms characterizing the tobermorite-like reaction products between lime and the clays. But these could not be confirmed because of the baseline drift mentioned above.

13. Electron micrographs were obtained for only three samples. These showed a massive aggregated flocculated structure very different from the "corn flake" like structure reported in the literature. This is taken to be a demonstration of the effect of lime on the clays. However, the lath-like tobermorites or silicate hydrates published in the literature could not be observed. This is probably due to the fact that these reports dealt with soils immersed in a lime slurry or with lime additions of the order of 20%.

14. An attempt to verify the two theories regarding the mechanism of lime reaction did not lead to any definite conclusions. Further work is needed.

15. Though the Prairie clays were derived from the same parent material, Bearpaw shale, their geology, morphology, and possibly environments make them different, which makes them react differently to lime treatment. A more systematic study is needed to establish this fact than is attempted during this investigation.

5. RECOMMENDATION FOR FUTURE RESEARCH

Throughout this study, the limitation of the present investigation was pointed out and the need for further study was indicated. In this section it is intended to collect all these pointers to future research.

1. Any amount of data from routine tests on engineering properties would provide an additional result which would lead to predictive equations relating these properties to the lime content.

2. The effect of curing temperature and time should be investigated so that realistic specifications and cutoff dates can be developed for Canadian conditions.

3. The differences in the behaviour of these clays should be investigated and related to the geology, morphology, and environment.

4. The durability of the lime treated clays should be investigated.

5. Moisture migration, suction potential, etc., of lime treated clays could be important from the point of view of swell resistance, durability, and frost heave.

6. Fabric and mineralogical studies of lime treated clays should be continued using X-ray diffraction, DTA, and SEM supplemented by other techniques such as chemical analyses, thermogravimetry, etc.

7. While addition of lime is proved to have beneficial effects on these clays, it is still a product with high energy input

involving calcination and grinding. Numerous reports have been published where many of the benefits can be enhanced by using lime with a pozzolonic material such as fly ash. Abundant fly ash is available in Canada and particularly in the Prairie provinces. Research should be undertaken to investigate whether lime-fly ash soil mixtures would be an effective alternative to depleting granular resources and such high energy alternatives as cement or lime-cement stabilized materials.

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A P P E N D I X I.

FIGURES.

STANDARD SIEVE SIZES (ASTM- E 11).

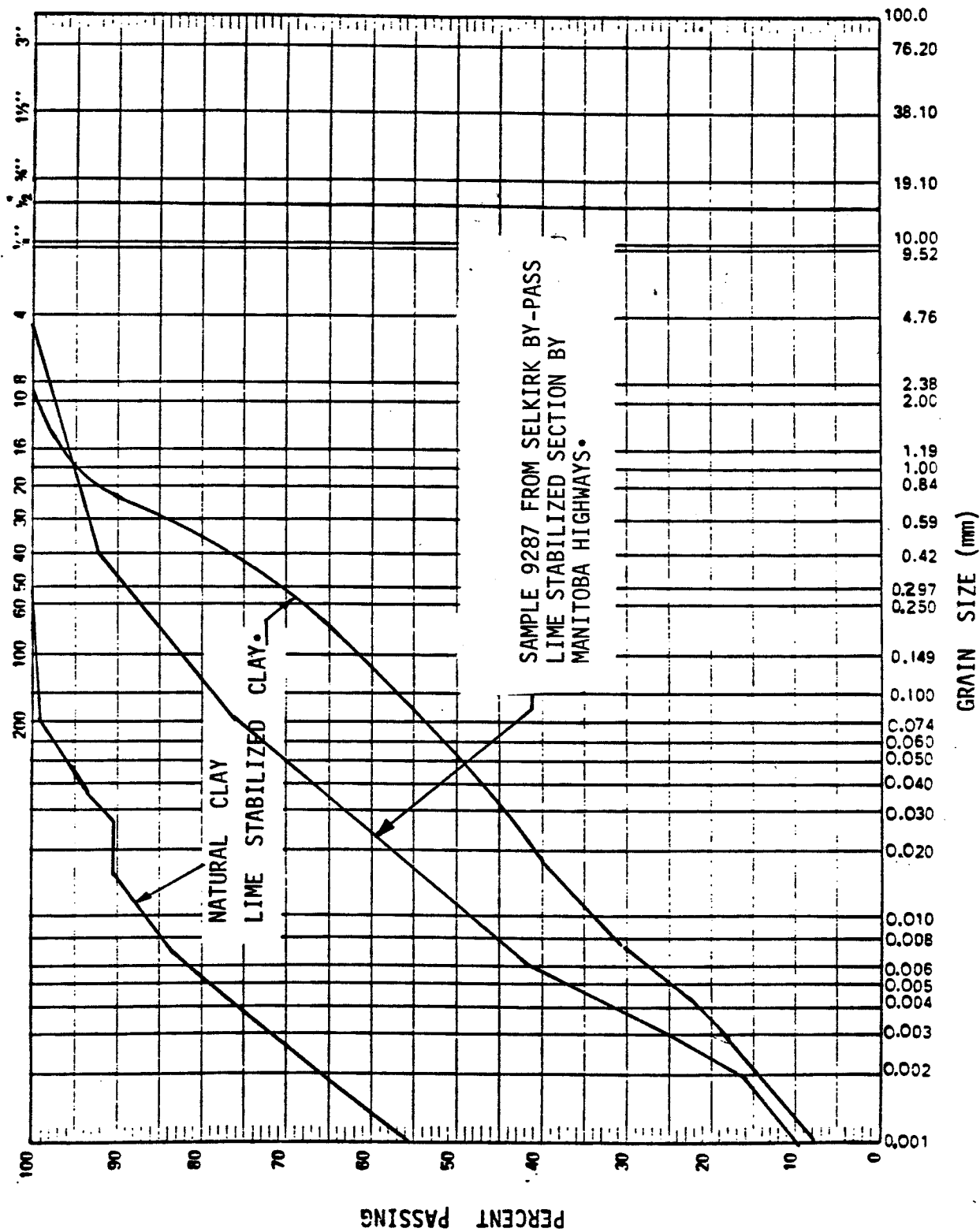


FIG. 1 EFFECT OF LIME ON THE GRAIN SIZE DISTRIBUTION OF CLAYS.

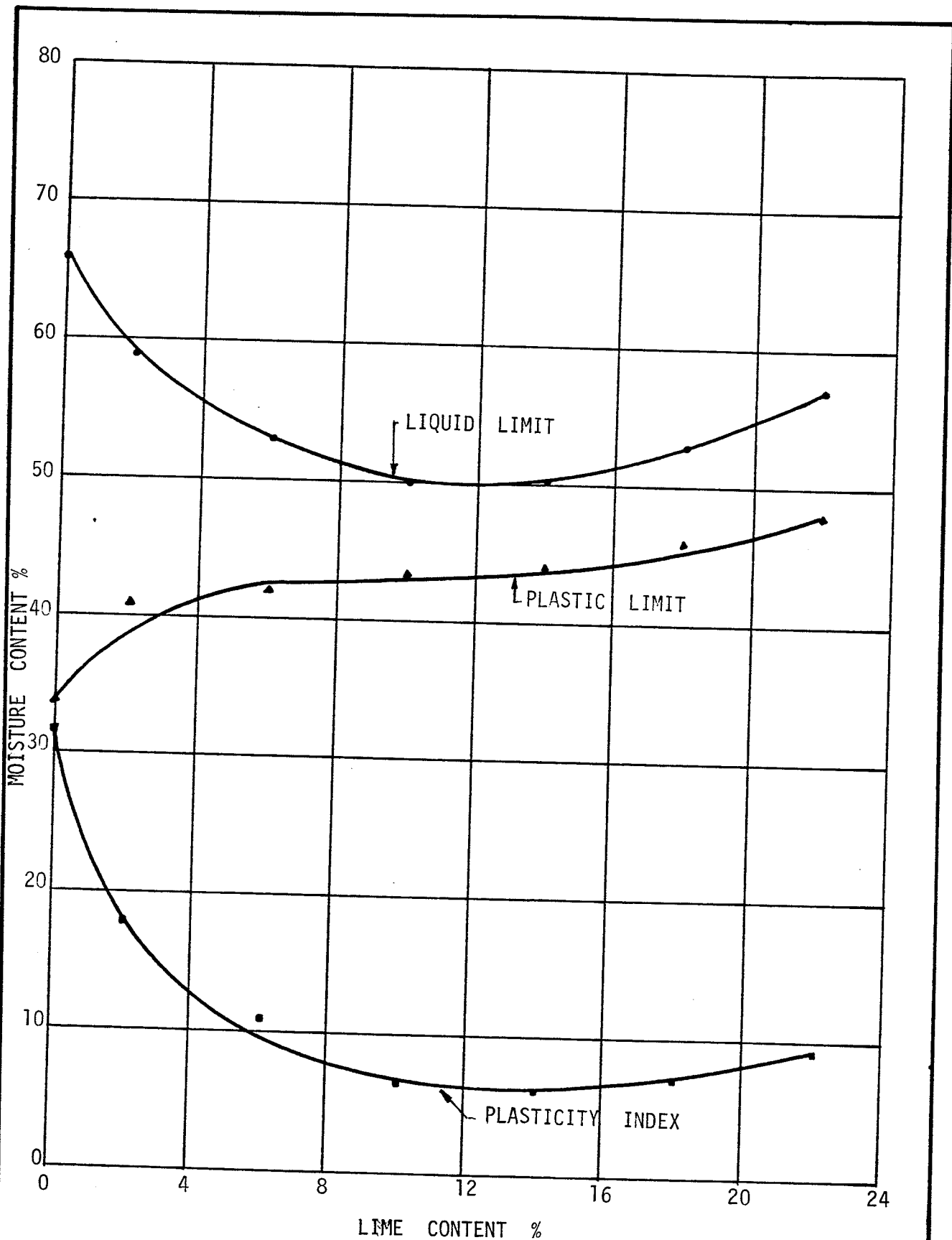


FIG. 2 EFFECT OF LIME ON THE PLASTICITY CHARACTERISTICS OF CLAYS.
(After Klym, 1958)

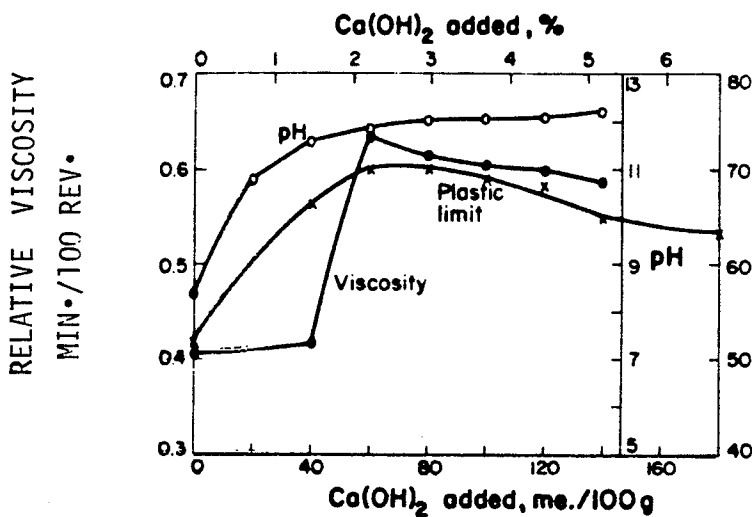
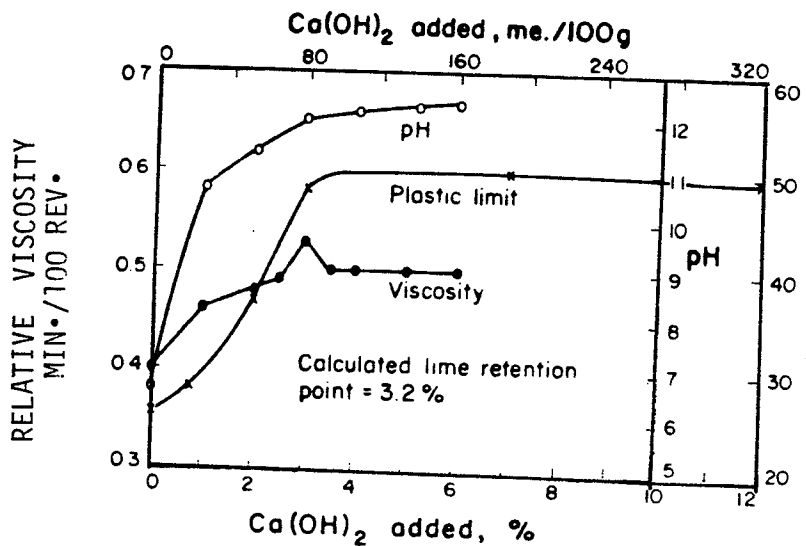


FIG. 3 EFFECT OF Ca(OH)_2 ADDED ON THE pH, RELATIVE VISCOSITY AND PLASTIC LIMITS OF IOWA GUMBOTIL (TOP) AND 30% SUSPENSION Ca-BENTONITE. (BOTTOM)

(After Ho and Handy, 1963)

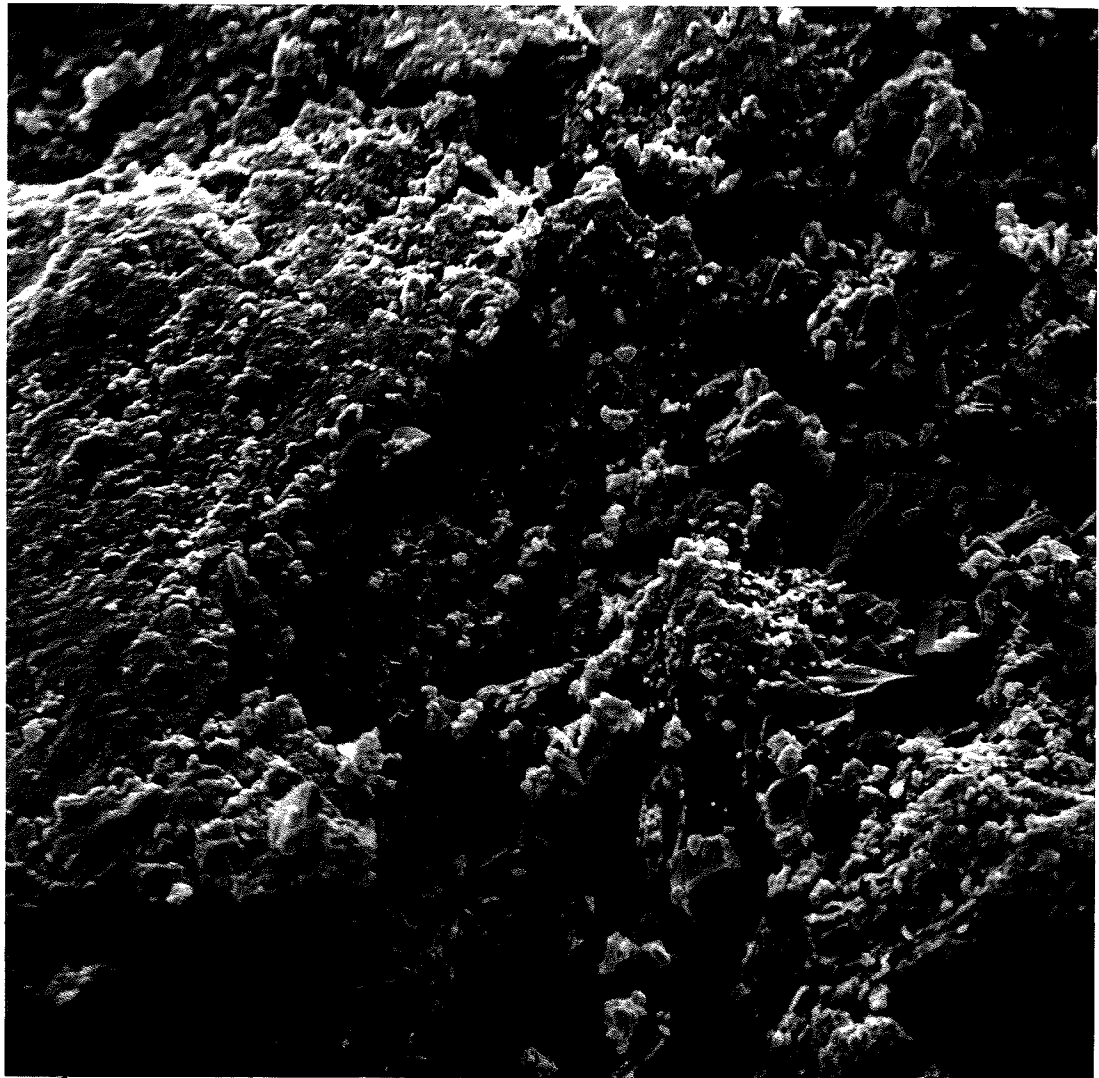


FIG. 4. ELECTRON MICROGRAPH OF A SAMPLE OF COMPACTED CLAY
TREATED WITH 4% QUICKLIME (SAMPLE OBTAINED FROM TEST
SECTION BY SASKATCHEWAN HIGHWAYS)
MAGNIFICATION: 3000 X.

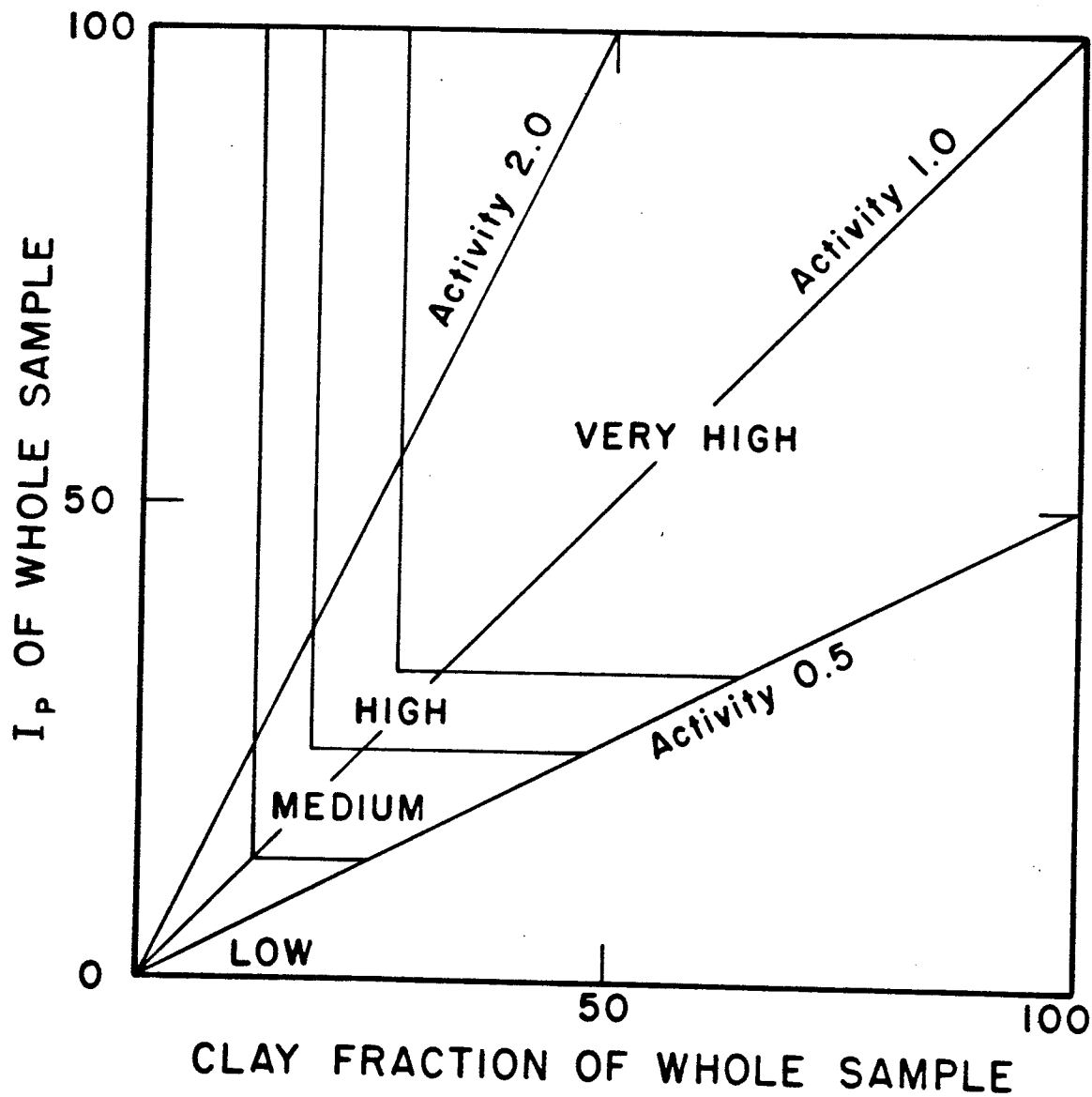


FIG. 5 VOLUME CHANGE POTENTIAL CLASSIFICATION FOR CLAY SOILS
(After Williams, 1958)

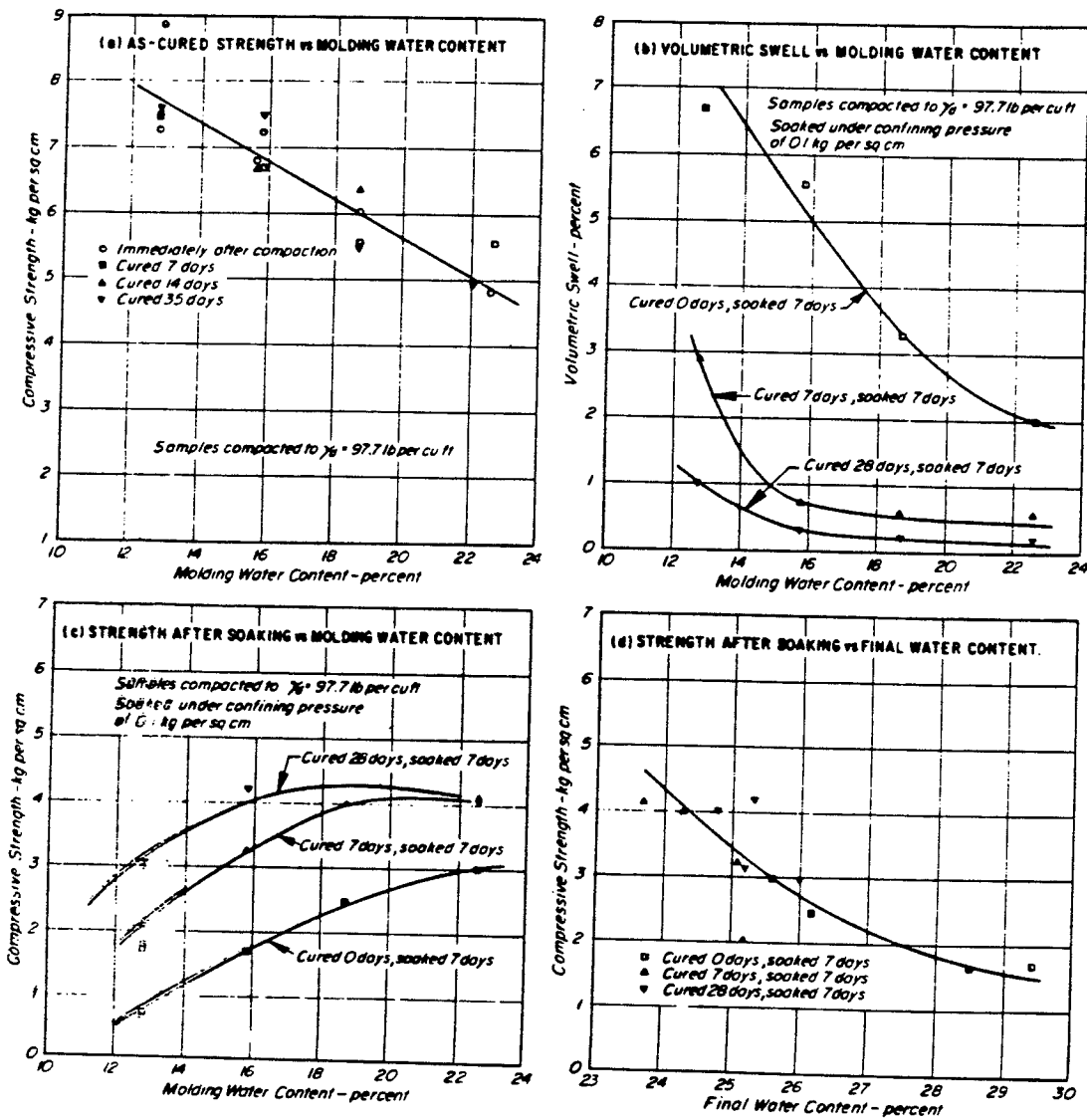


FIG. 6 STRENGTH, WATER CONTENT AND SWELL RELATIONSHIPS FOR SAMPLES OF LIME-TREATED EXPANSIVE CLAY COMPACTED 40 MIN. AFTER MIXING. (After Mitchell and Hooper, 1961).

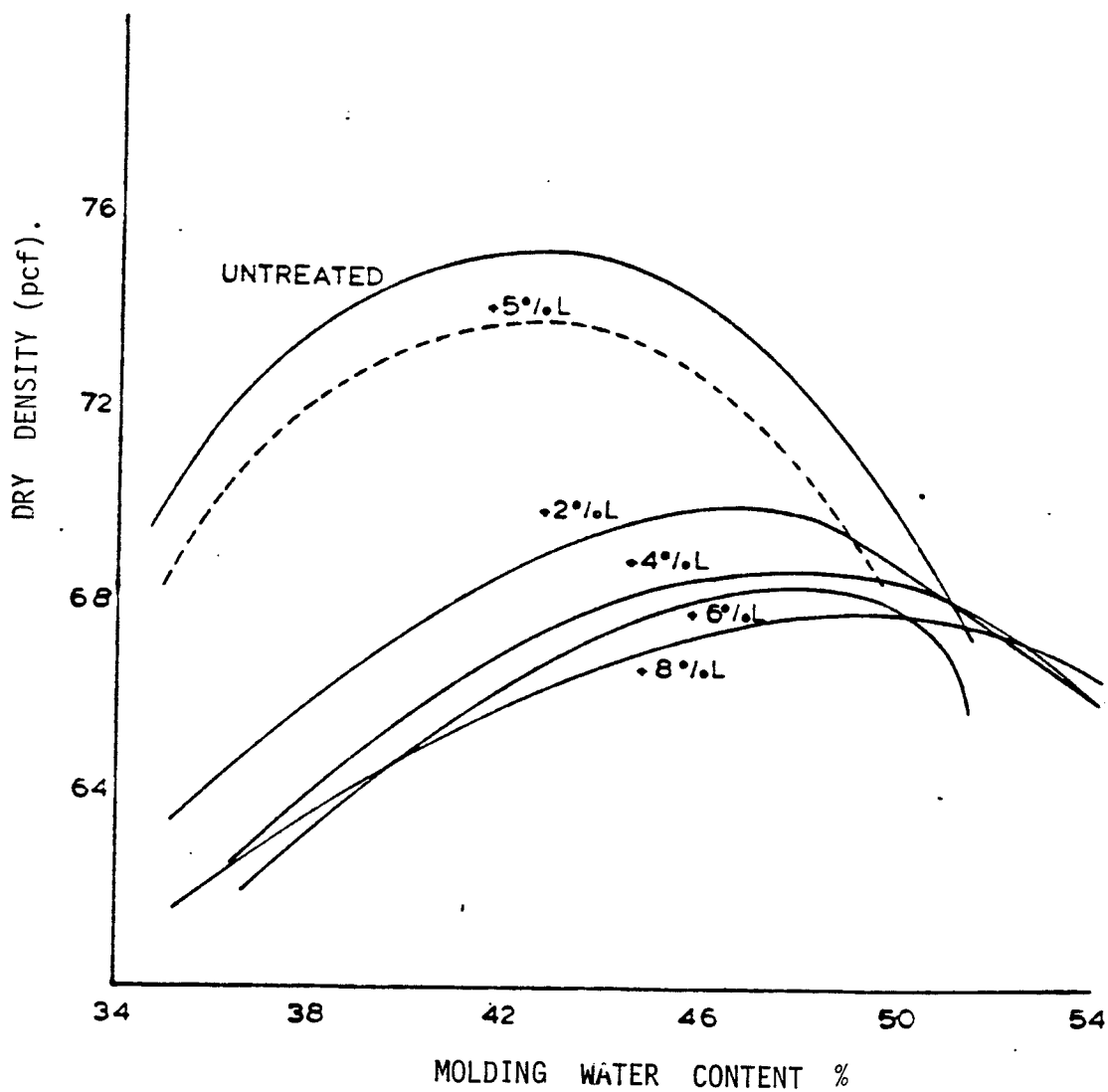


FIG. 7 EFFECT OF LIME ON MOISTURE - DENSITY RELATIONSHIP.
(After Eades, 1962).

(The results for soil treated with 5% lime do not seem to fit the general pattern. No explanation is offered by Eades. This could be due to experimental errors. -- GVG).

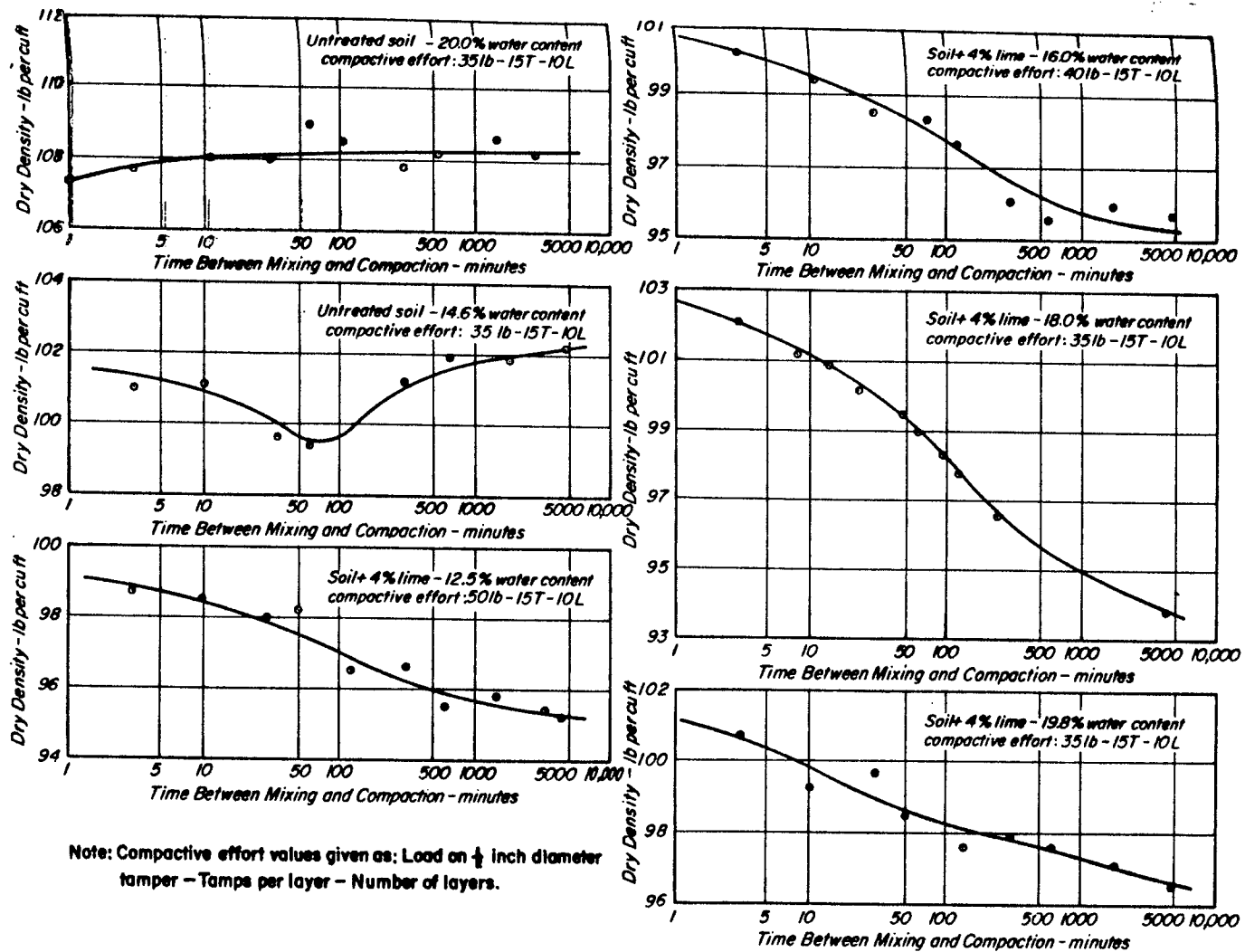


Fig. 8 Effect of delay between mixing and compaction on the dry density of lime-treated soils at the same compactive effort (After Mitchell and Hooper, 1961).

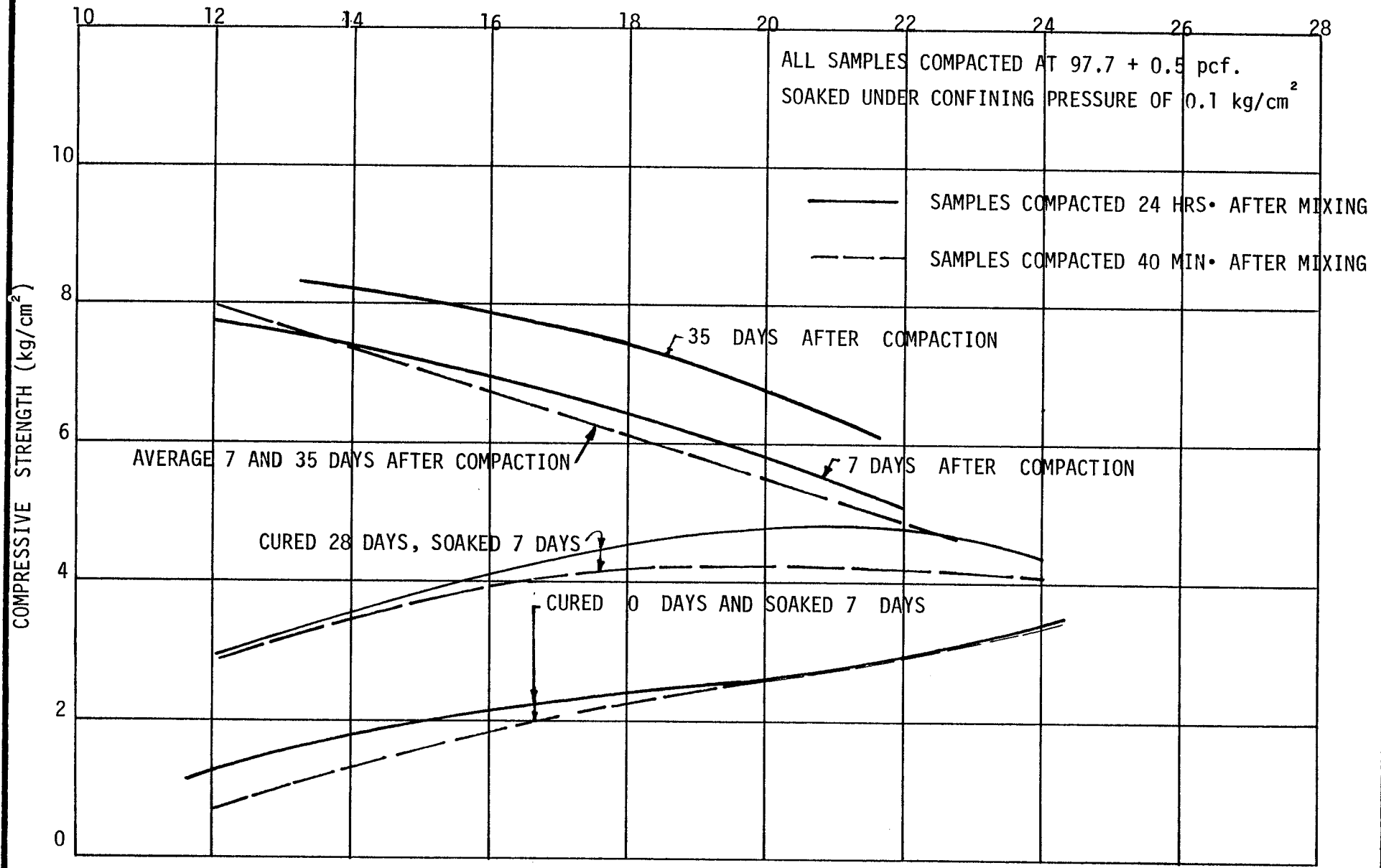


FIG. 9 STRENGTH OF LIME TREATED SOILS AT THE SAME DRY DENSITIES BUT COMPACTED AT DIFFERENT TIMES AFTER MIXING. (After Mitchell and Hooper, 1961)

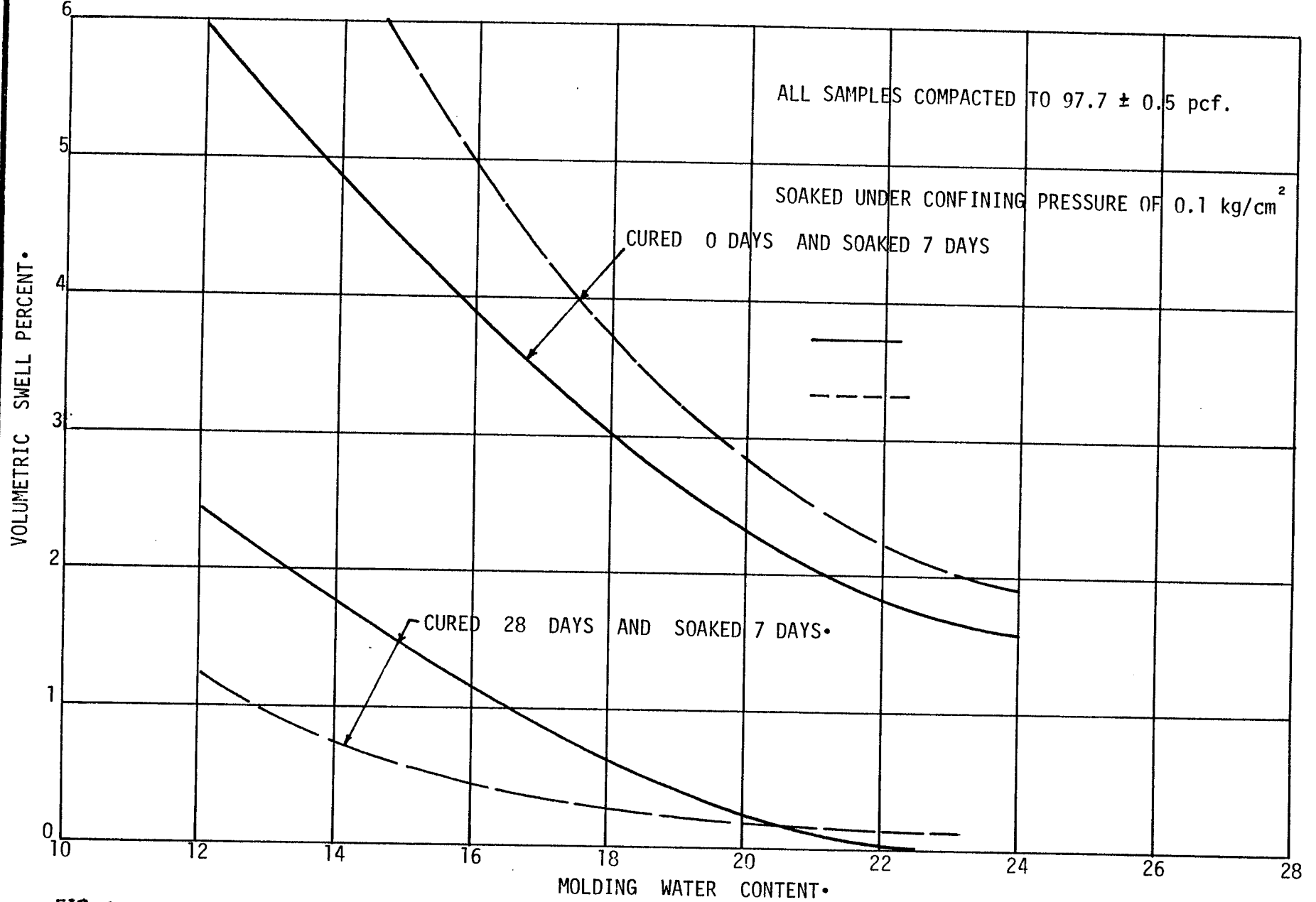


FIG. 10 VOLUME CHANGE OF LIME TREATED SOILS COMPARED AT EQUAL DENSITIES BUT COMPACTED AT DIFFERENT TIMES AFTER MIXING (After Mitchell and Hooper, 1961).

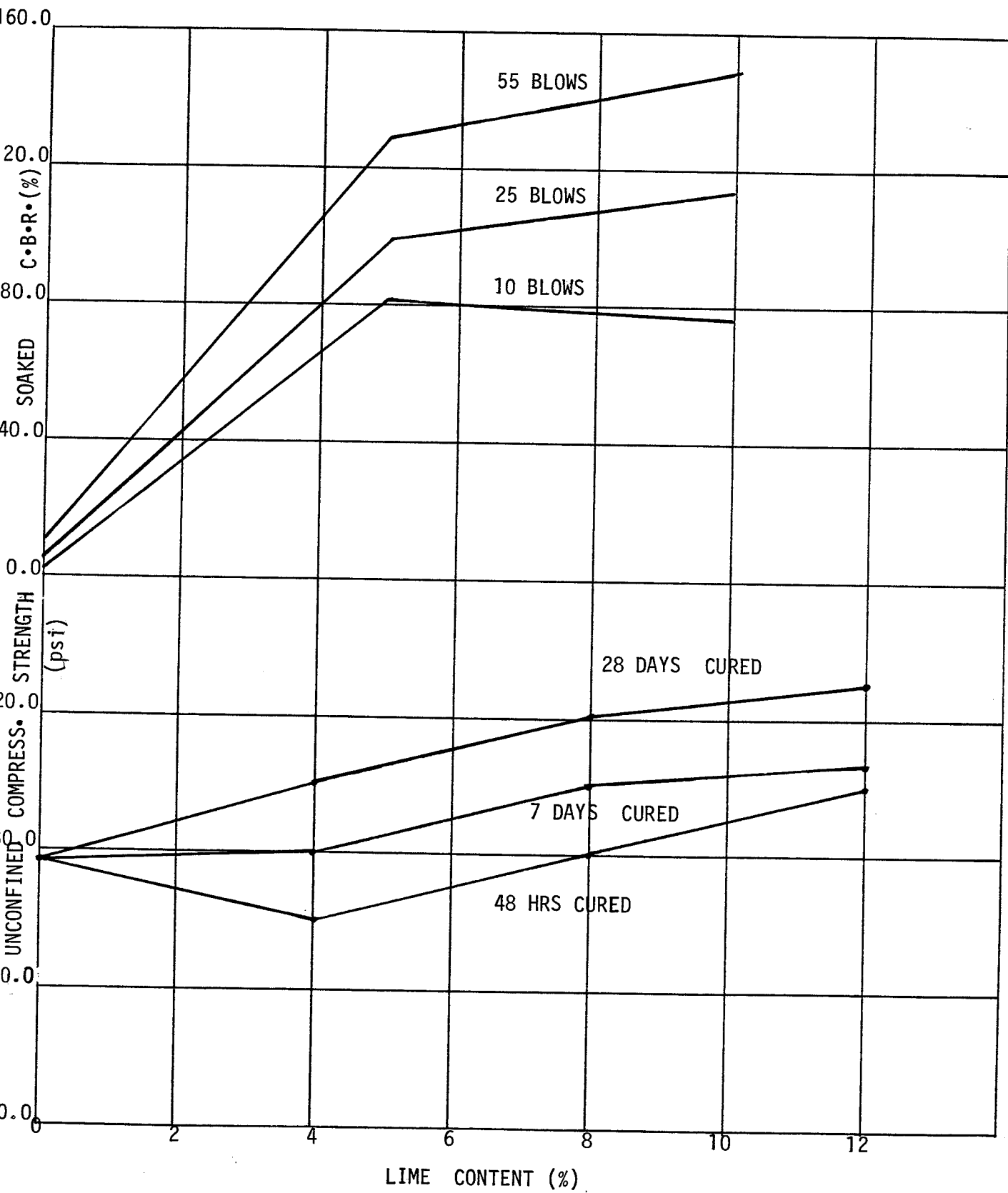


FIG. 11. EFFECT OF LIME ON THE C.B.R (TOP) AND UNCONFINED COMPRESSION STRENGTH OF CLAYS.

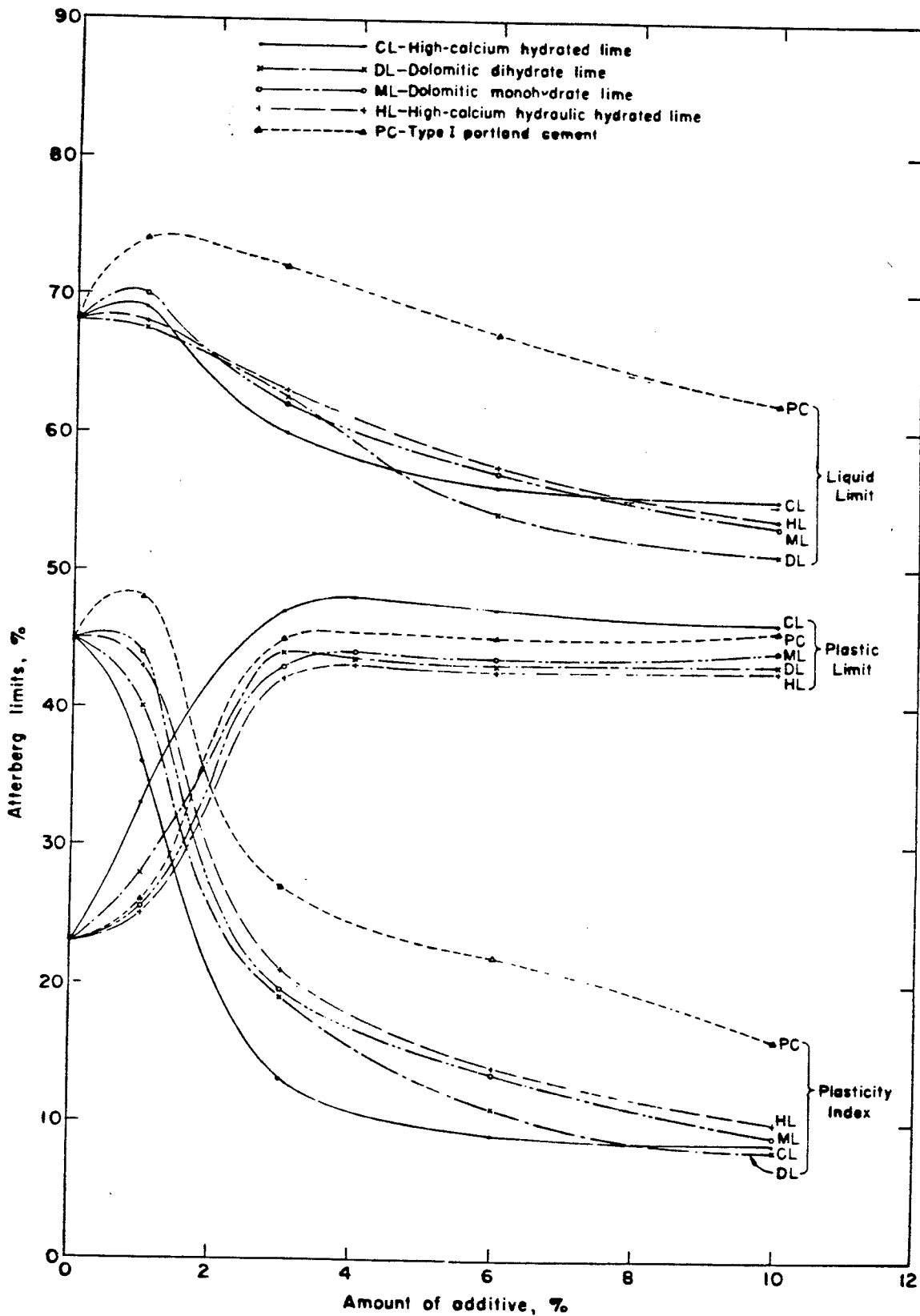


FIG.12 EFFECT OF DIFFERENT TYPES OF LIMES ON THE PLASTICITY OF SOILS.
(After Wang et al 1963)

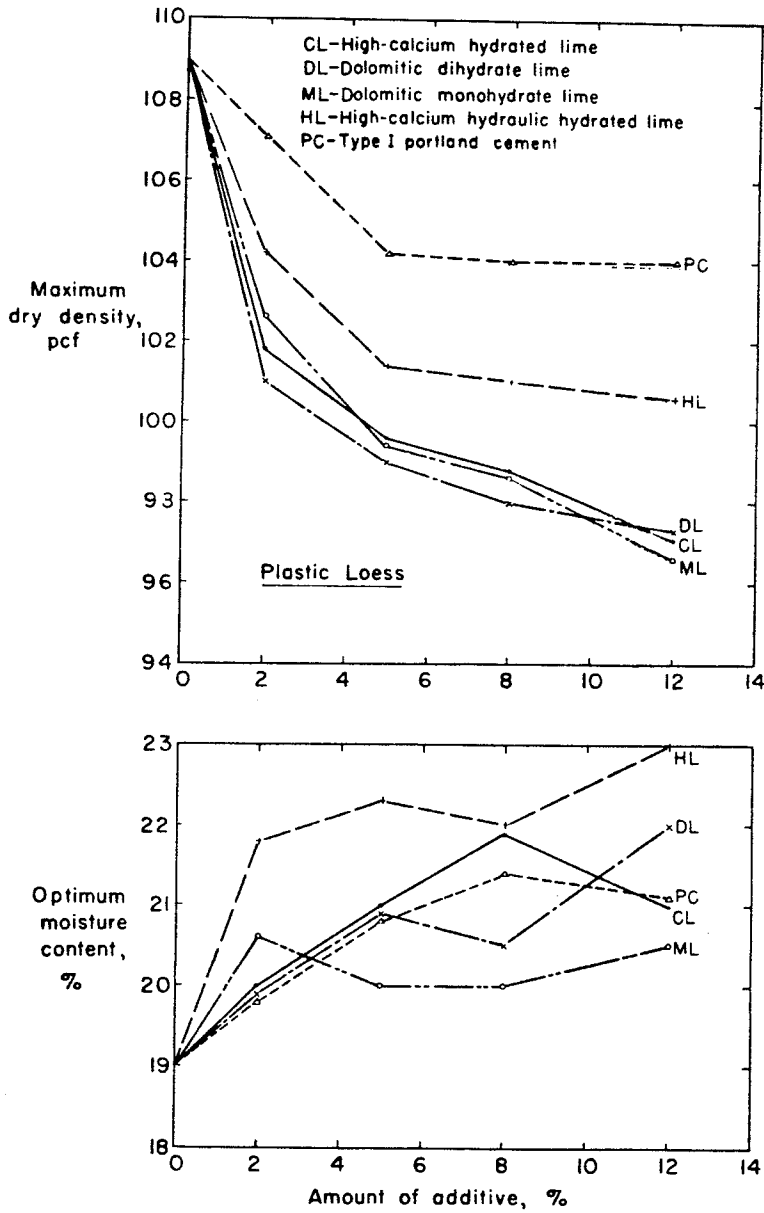


FIG. 13 EFFECT OF DIFFERENT TYPES OF LIMES ON THE DRY DENSITY AND OMC OF COMPACTED SAMPLES.

(After Wang et al 1963)

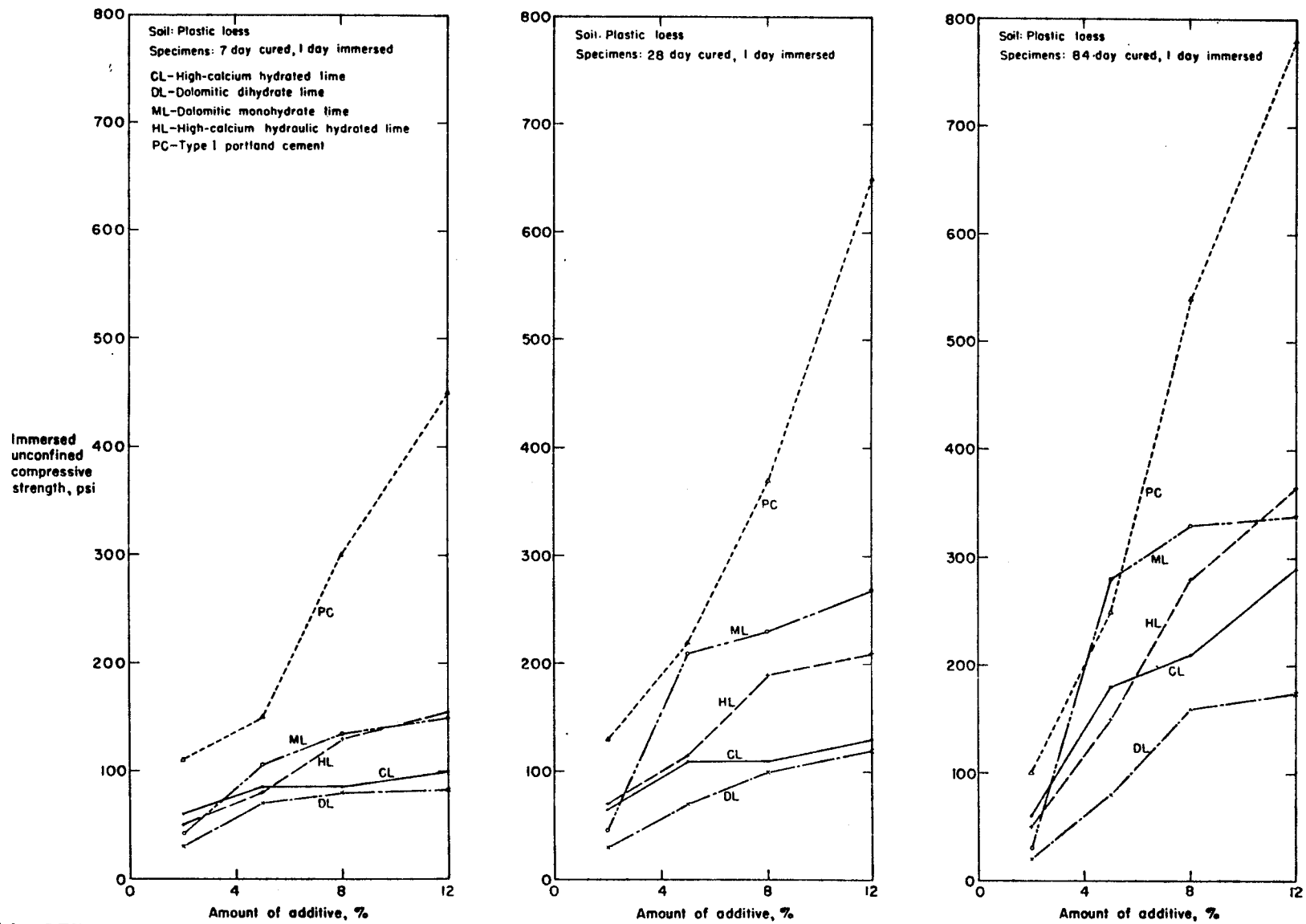


FIG. 14. EFFECT OF DIFFERENT TYPES OF LIMES AND PORTLAND CEMENT ON THE UNCONFINED COMPRESSION STRENGTH OF COMPACTED CLAYS. (After Wang et al, 1963).

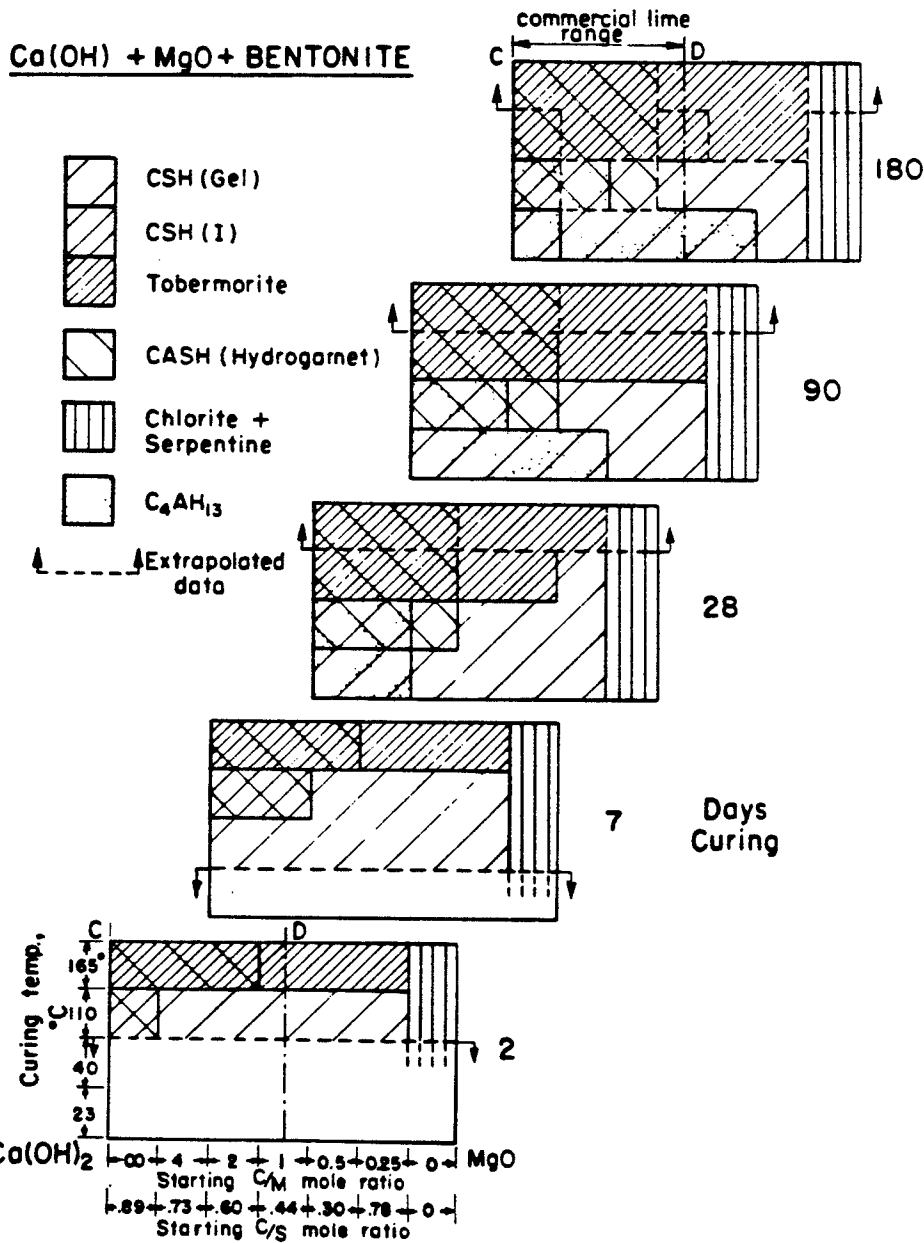


FIG. 15 DEVELOPMENT OF REACTION PRODUCTS IN A BENTONITE LIME SLURRY USING DOLOMITIC QUICKLIME AT DIFFERENT STAGES OF CURING. (After Wang and Handy, 1966).

DOLOMITIC DIHYDRATE LIME
+ BENTONITE

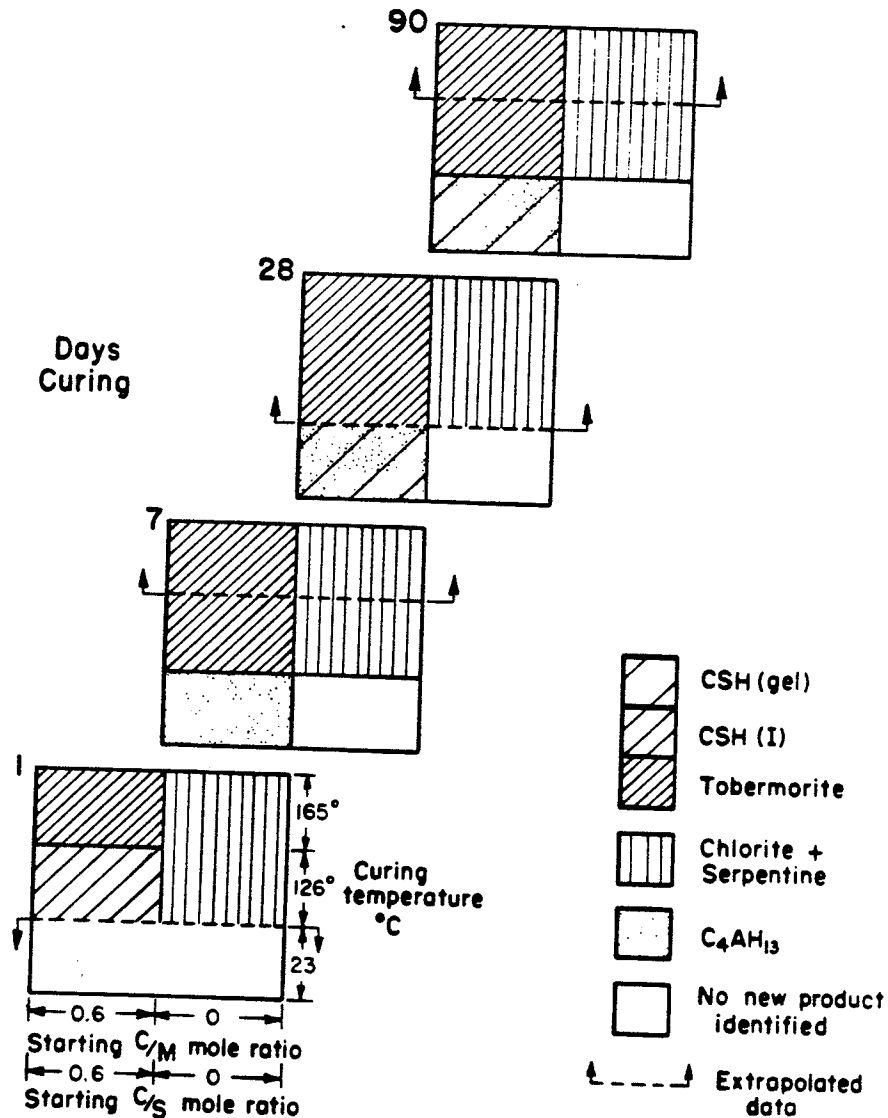


FIG. 16 DEVELOPMENT OF REACTION PRODUCTS IN A LIME BENTONITE SLURRY USING DOLOMITIC DIHYDRATE LIME AT DIFFERENT STAGES OF CURING.
(After Wang and Handy, 1966).

**DOLOMITIC MONOHYDRATE LIME
+ BENTONITE**

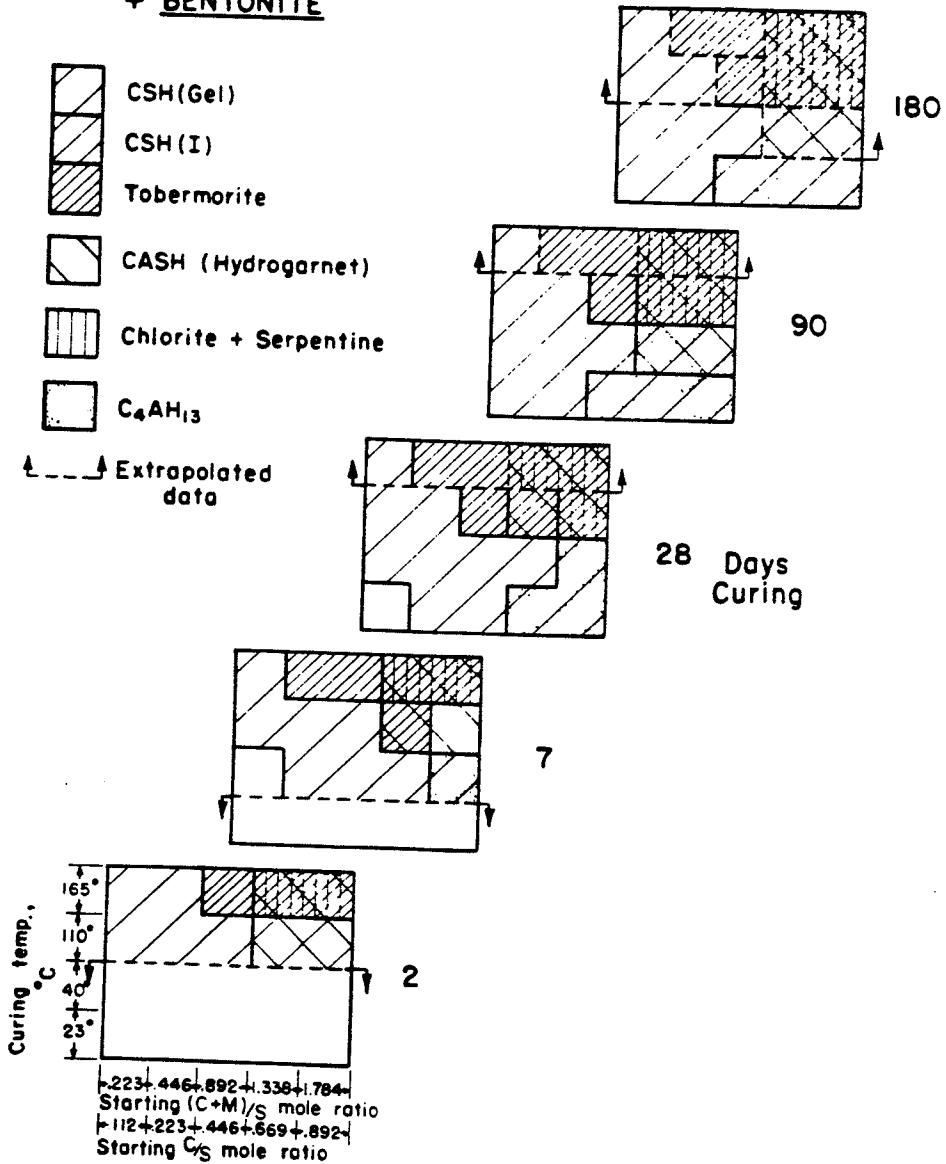


FIG. 17 DEVELOPMENT OF REACTION PRODUCTS IN A BENTONITE LIME SLURRY USING MONOHYDRATE DOLOMITIC LIME AT DIFFERENT STAGES OF CURING. (After Wang and Handy, 1966).

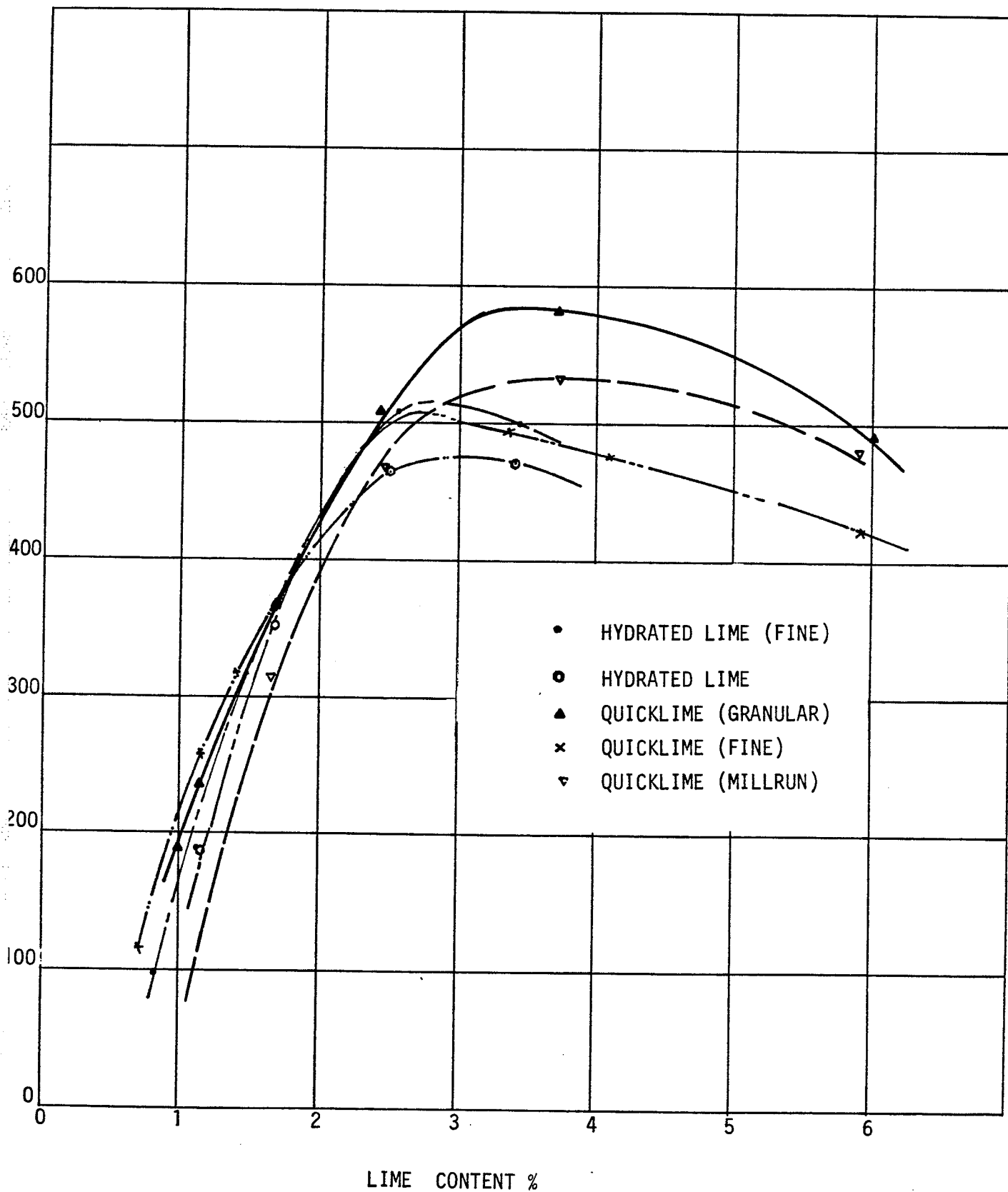


FIG. 18. RELATIONSHIP BETWEEN LIME CONTENT AND UNCONFINED COMPRESSION STRENGTH.

(After Alexander et al, 1972)

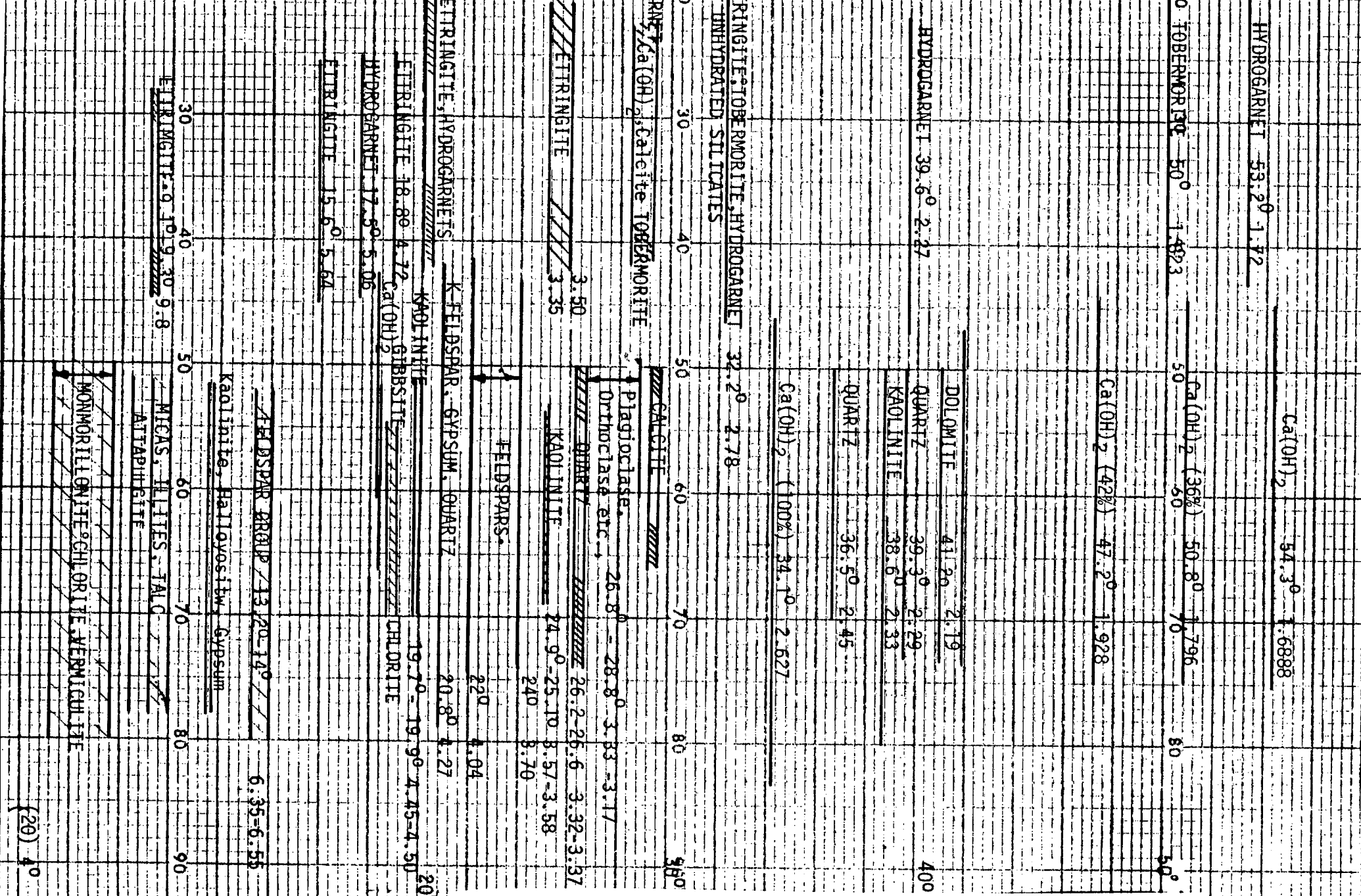
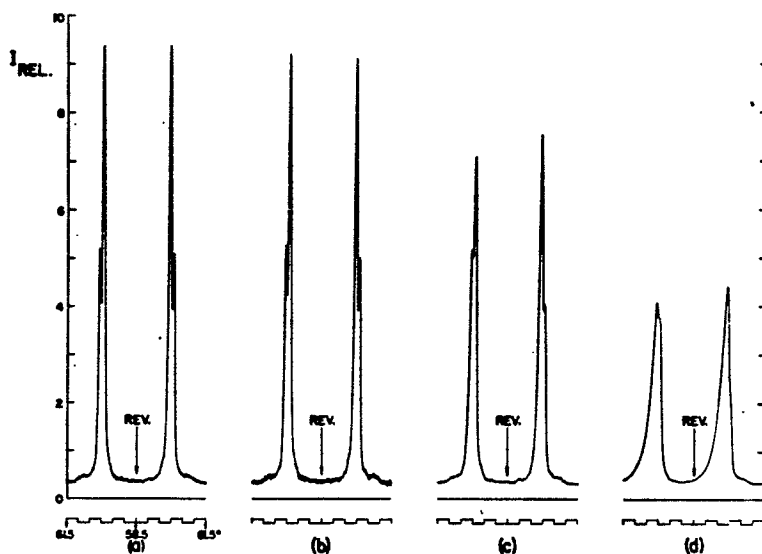
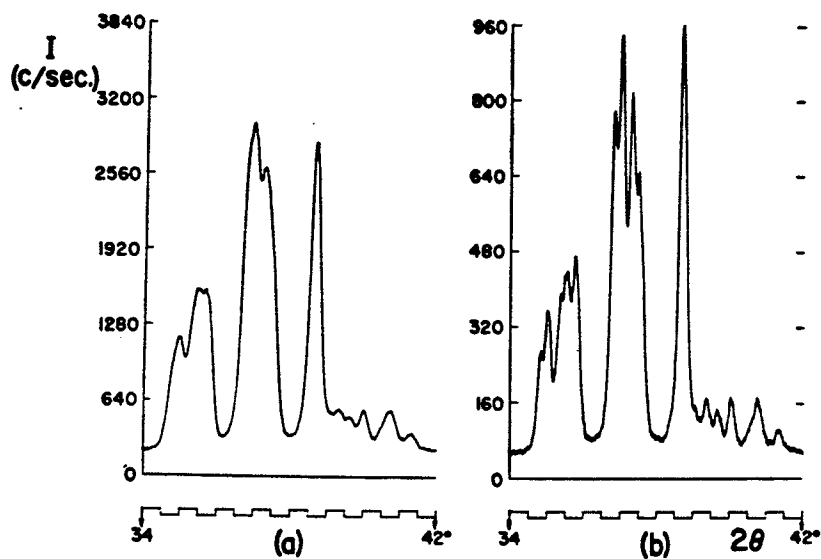


FIG. 19. PEAKS OF SOME COMMON CLAY MINERALS AND THOSE OF SOME REACTION PRODUCTS WITH LIME.



	(a)	(b)	(c)	(d)
Scan Speed ($^{\circ}/\text{min}$)	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	1
Time Constant (sec)	4	2	8	16
Product (sec \times $^{\circ}/\text{min}$)	1	1	4	16
Chart Speed (in./hr)	$7\frac{1}{2}$	$7\frac{1}{2}$	15	30



	(a)	(b)
Scan Speed ($^{\circ}/\text{min}$)	1	$\frac{1}{2}$
Time Constant (sec)	4	4
Product (sec \times $^{\circ}/\text{min}$)	4	1
Chart Speed (in./hr)	30	$7\frac{1}{2}$
Receiving Slip ($^{\circ}2\theta$)	0.20	0.05

FIG. 20 EFFECT OF DIFFERENT SCANNING SPEEDS AND TIME CONSTANTS ON THE X-RAY DIFFRACTOGRAMS. (TOP WYOMING BENTONITE; BOTTOM - DICKITE) (After Parrish 1958).

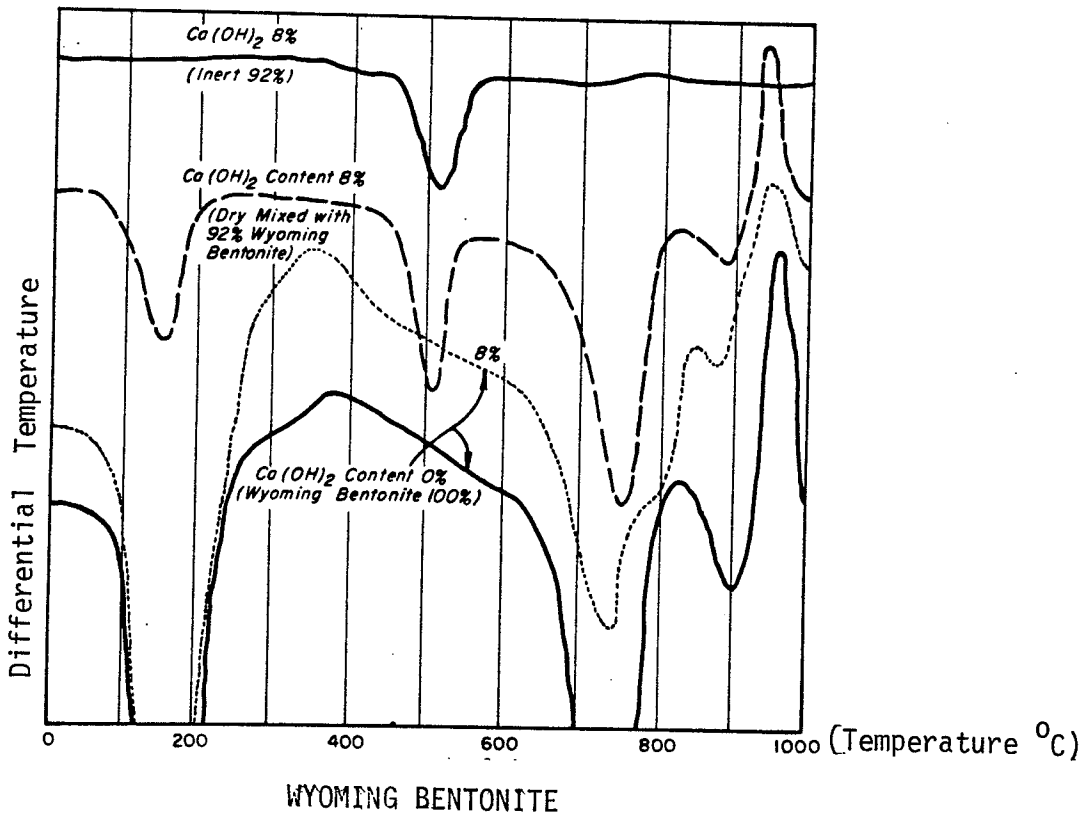
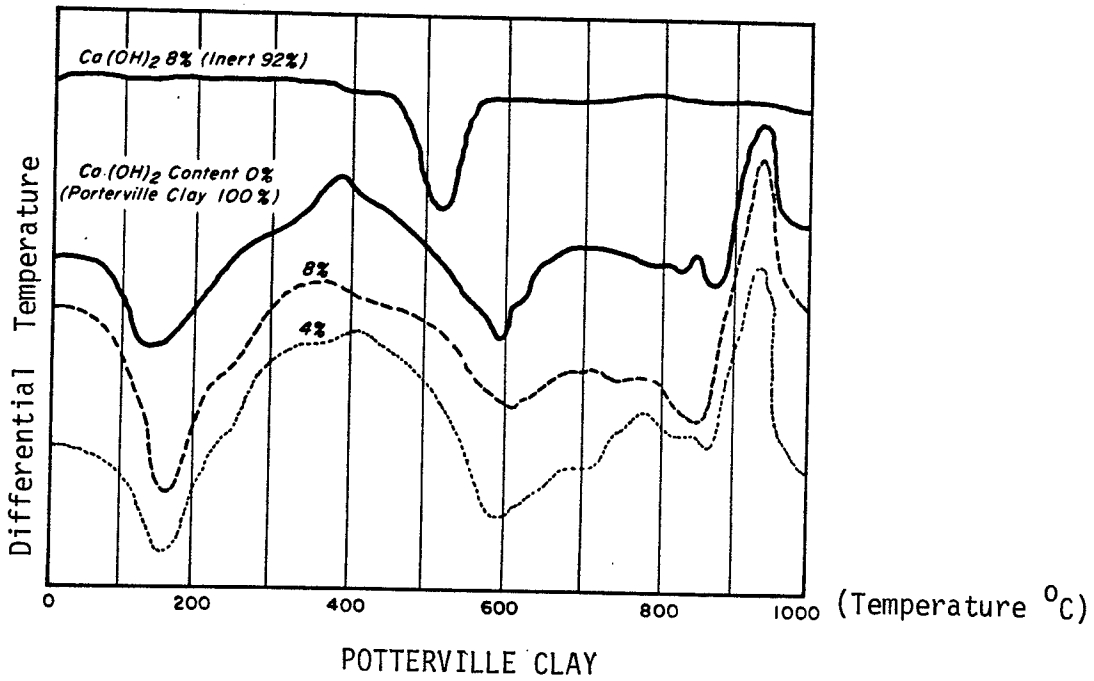
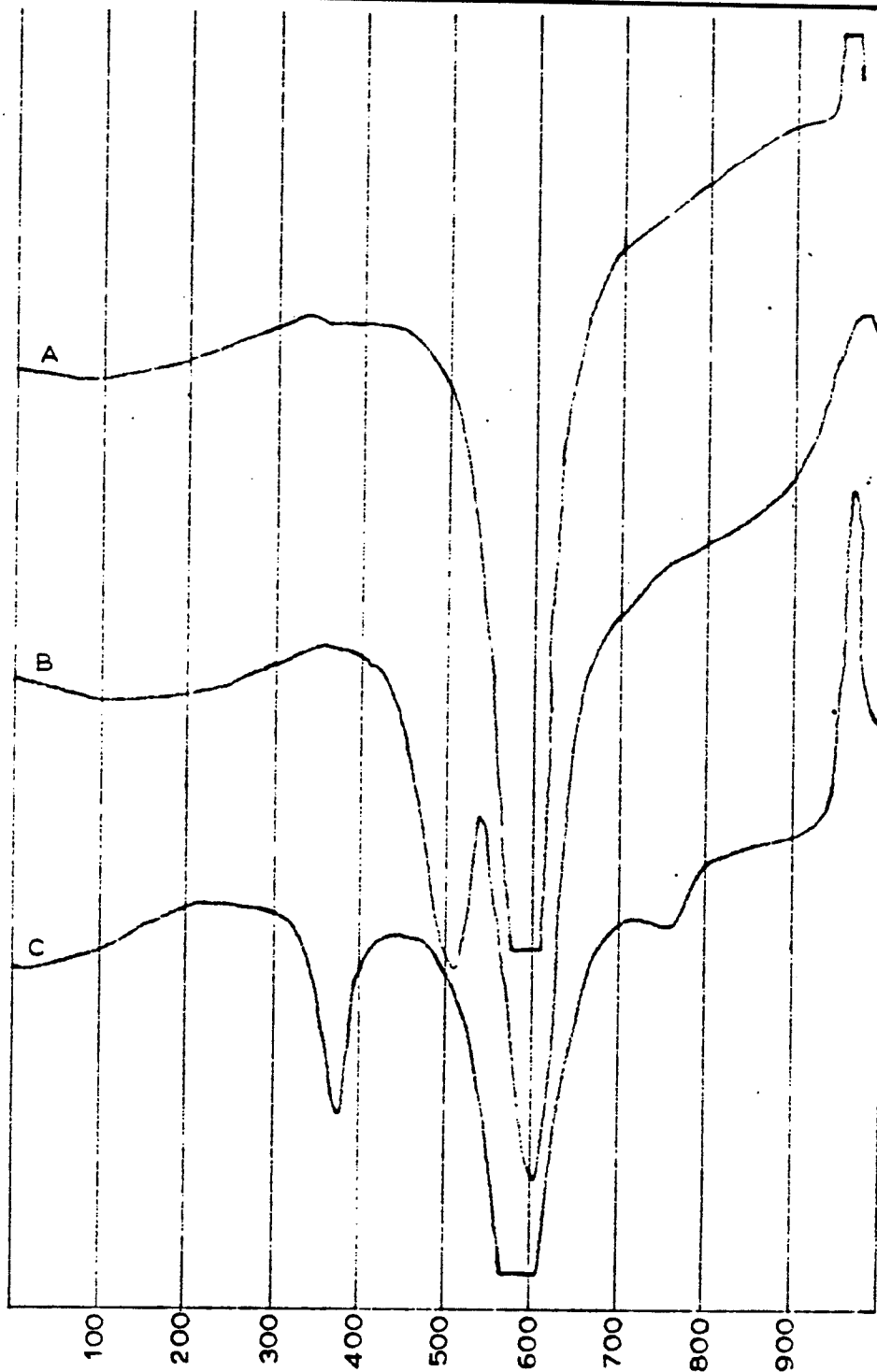
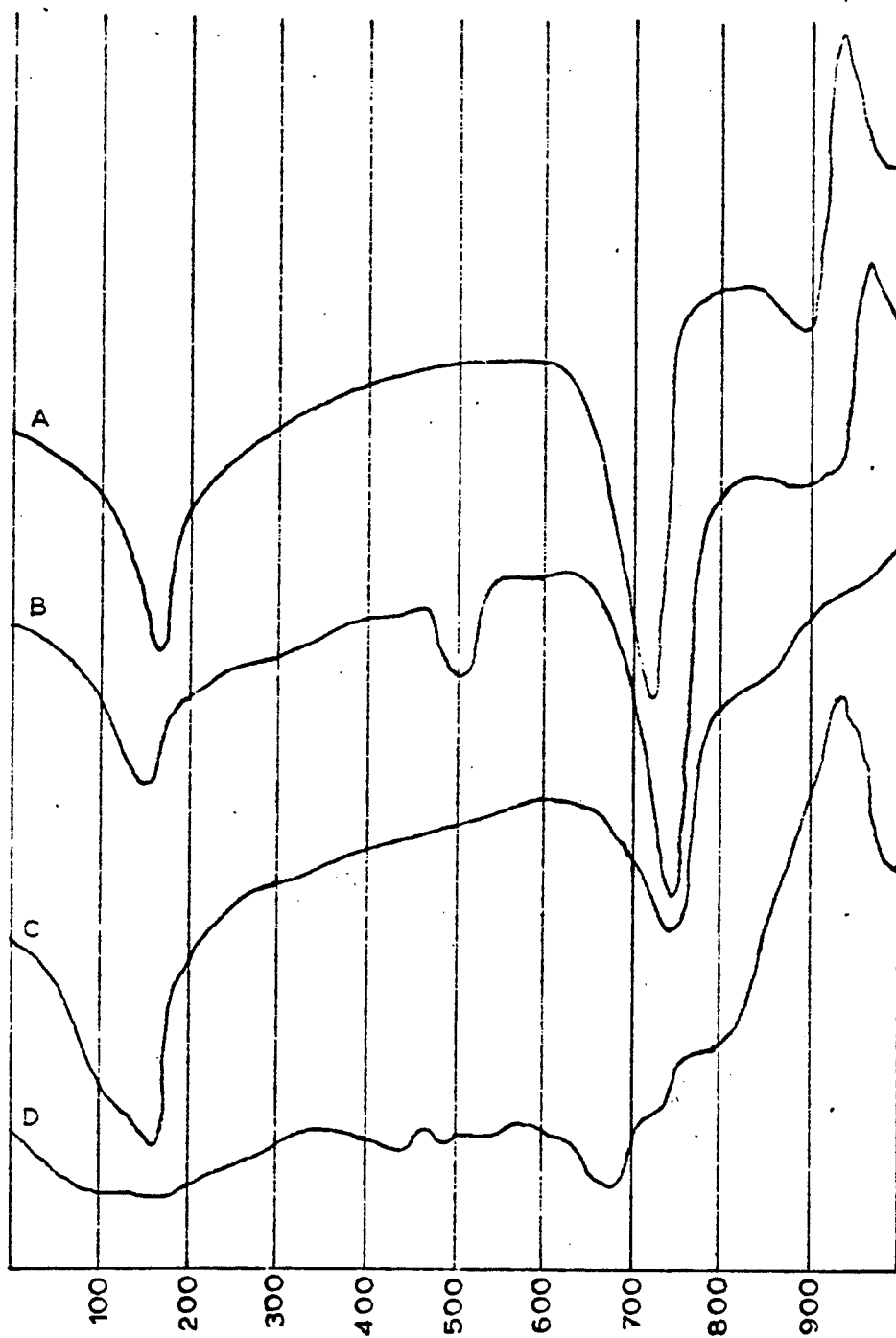


FIG. 21A DTA PATTERN FOR POTTERVILLE CLAY AND WYOMING BENTONITE WITH AND WITHOUT LIME TREATMENT (After Goldberg and Klein, 1957).



A-GEORGIA KAOLIN UNTREATED
 B-GEORGIA KAOLIN +20% HYDRATED LIME UNTREATED
 C-GEORGIA KAOLIN +20% HYDRATED LIME CURED 72 HRS. AT 140°F

FIG. 21B D.T.A. CURVES OF LIME-TREATED GEORGIA KAOLIN
 (After Eades, 1962)



- A - WYOMING MONTMORILLONITE UNTREATED
- B - WYOMING MONT. + 4% HYDRATED LIME UNTREATED
- C - WYOMING MONT. + 5% HYDRATED LIME CURED 72 HRS AT 140°F
- D - WYOMING MONT. + 20% HYDRATED LIME CURED 72 HRS AT 140°F

FIG. 21C D.T.A. CURVES OF LIME-TREATED WYOMING MONTMORILLONITE
 (After Eades, 1962)

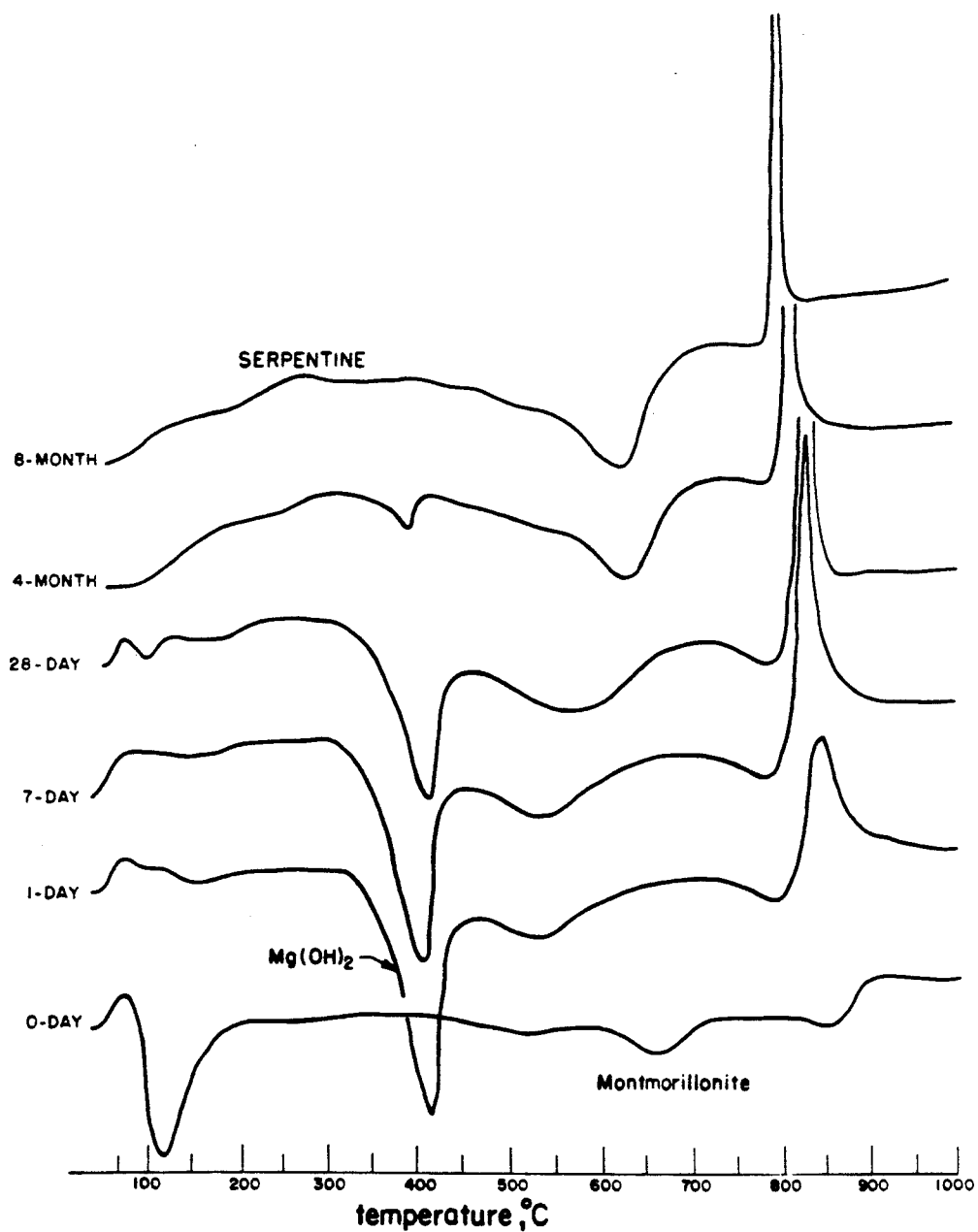


FIG. 21D DTA CURVES FOR MONTMORILLONITE-LIME MIXTURES
 AT DIFFERENT CURING PERIODS. (After Handy, 1962)

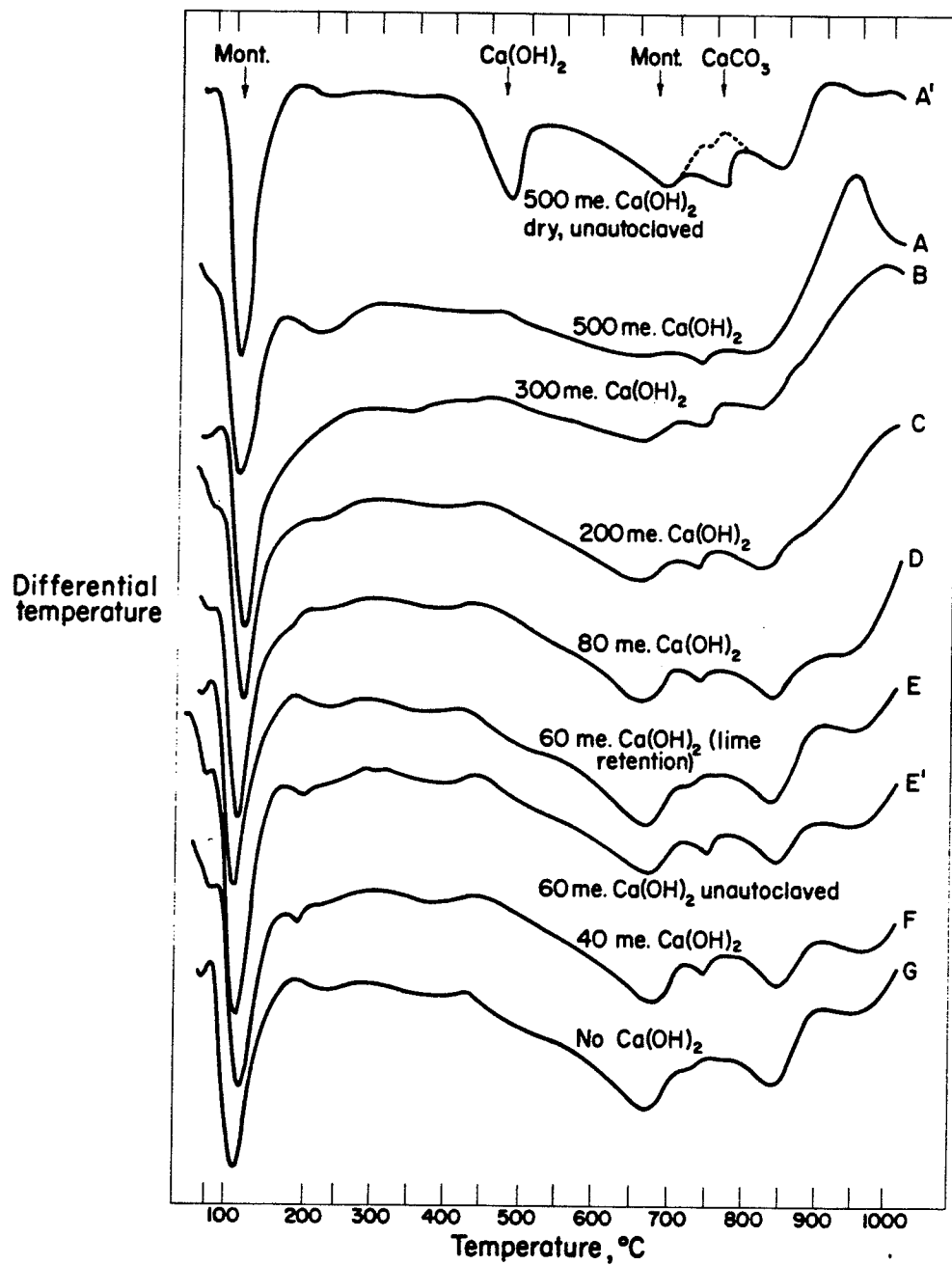
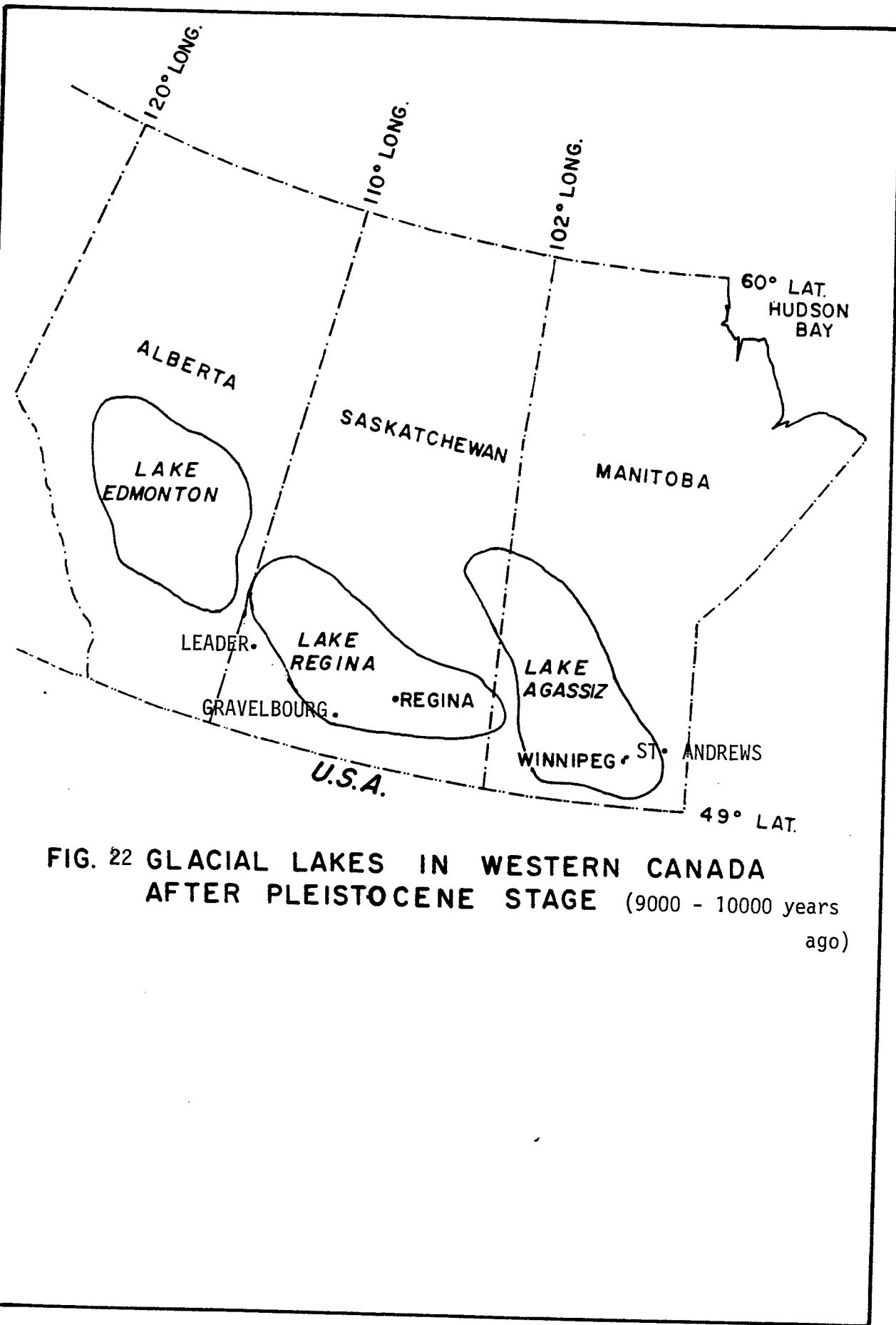


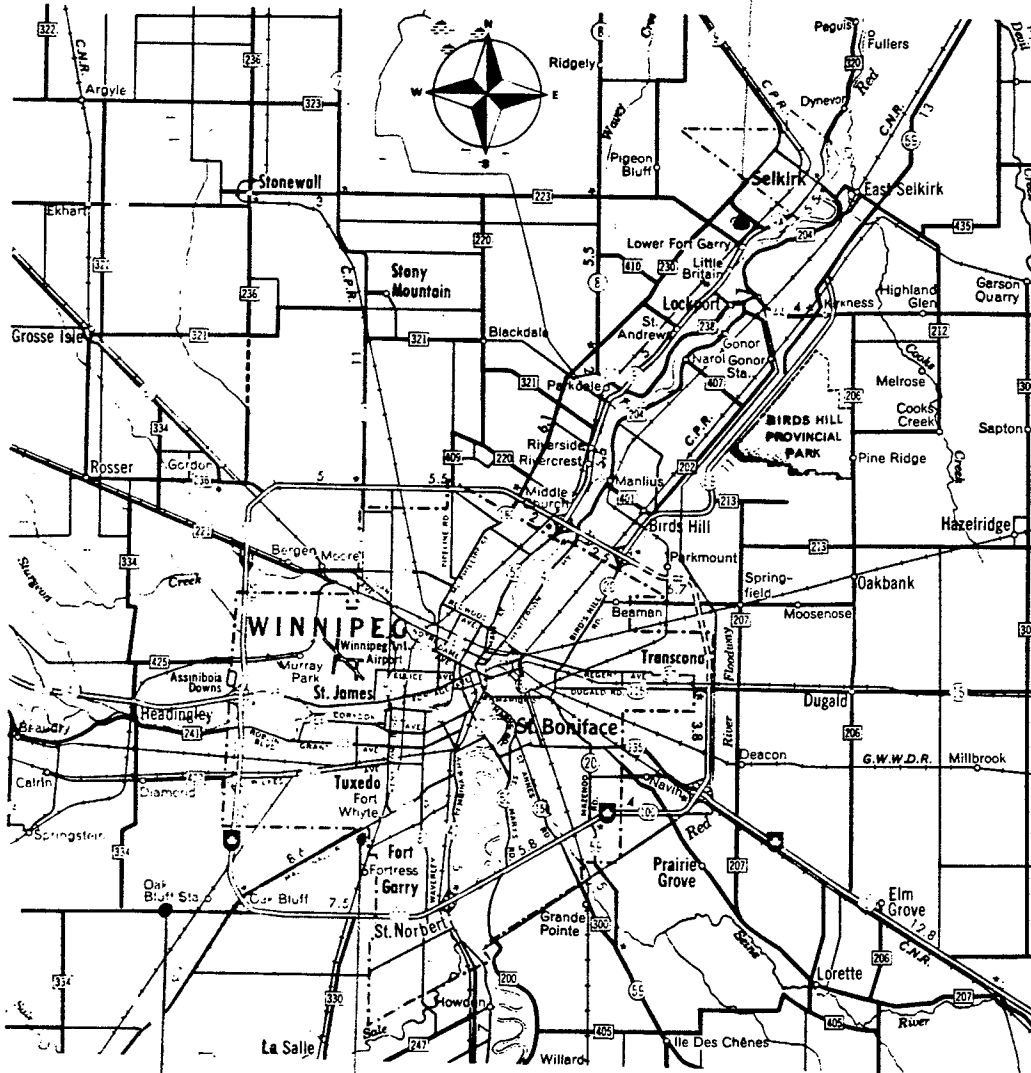
FIG.21E DTA CURVES FOR A Ca-BENTONITE TREATED WITH VARIOUS AMOUNTS OF LIME (MILLIEQUIVALENTS PER 100 g) AND AUTOCLAVED (126° C AND 22PSI PRESSURE FOR 8 DAYS).

(After Ho and Handy, 1963).



**FIG. 22 GLACIAL LAKES IN WESTERN CANADA
AFTER PLEISTOCENE STAGE (9000 - 10000 years
ago)**

SELKIRK BY-PASS
 TEST SECTION
 SAMPLES 928.7 - 9290



SAMPLES FROM WAVERLY HEIGHTS SUBDIVISION

— OAK BLUFF TEST SECTION (SAMPLES 9284 - 9286)

FIG. 23. LOCATION OF THE TEST SECTIONS BY THE MANITOBA HIGHWAYS.
 (ALSO SHOWN IS THE LOCATION OF SAMPLES OF W-SERIES)

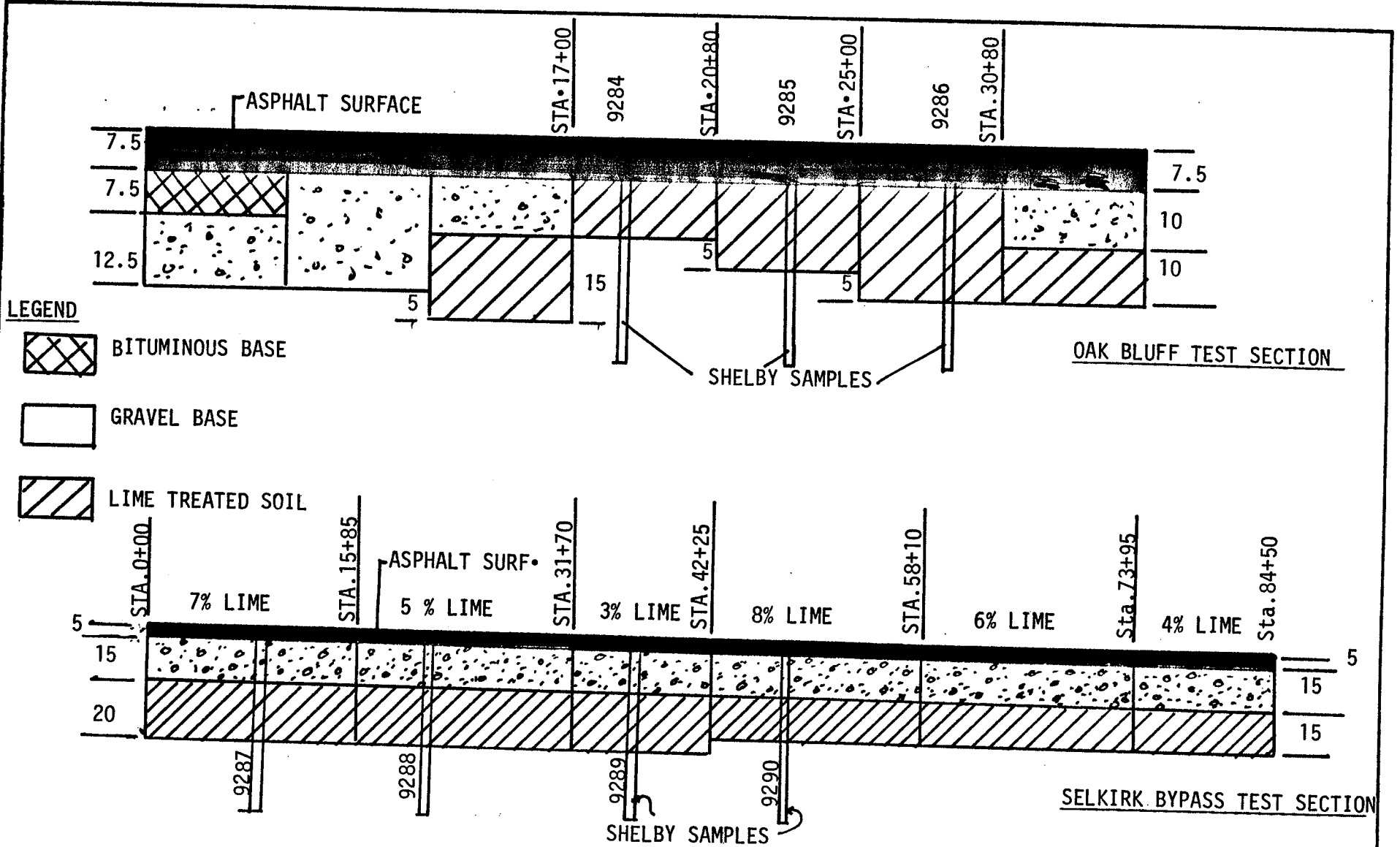
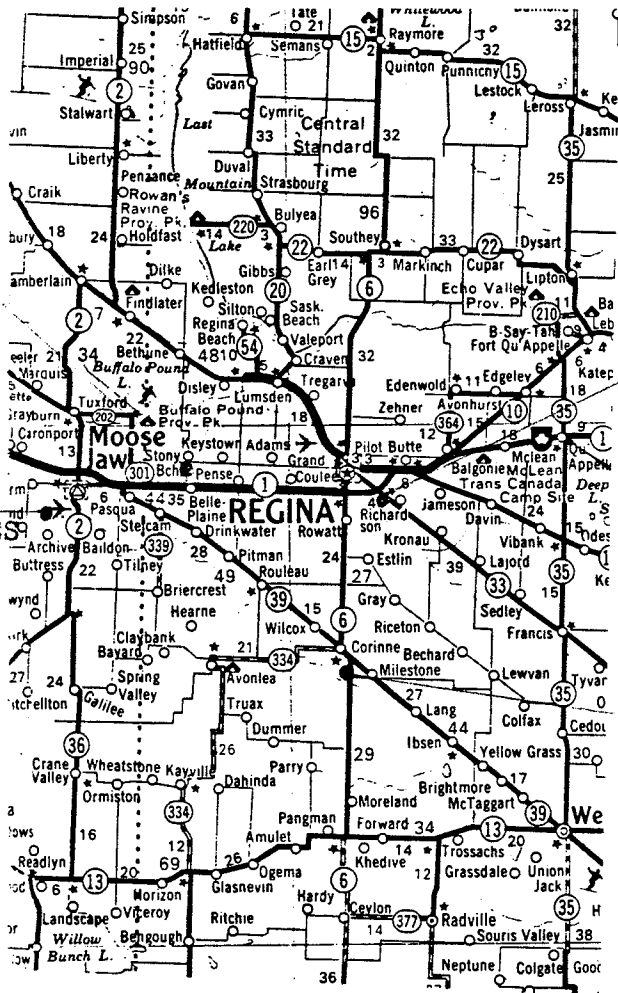


FIG. 23A DETAILS OF TEST SECTIONS BY MANITOBA DEPARTMENT OF HIGHWAYS (AFTER YOUNG 1964)
(ALL DEPTHS IN cm)

KALIUM RD
(9320 AND
'9324)

AIRPORT
SAMPLES R-SERIES



HWY 33/
HWY 1
BY-PASS
(SAMPLES 9322 &
9325)

HWY 6
SAMPLES 9321
AND 9323

FIG. 24. LOCATION TEST SECTIONS BY THE SASKATCHEWAN DEPARTMENT OF HIGHWAYS.

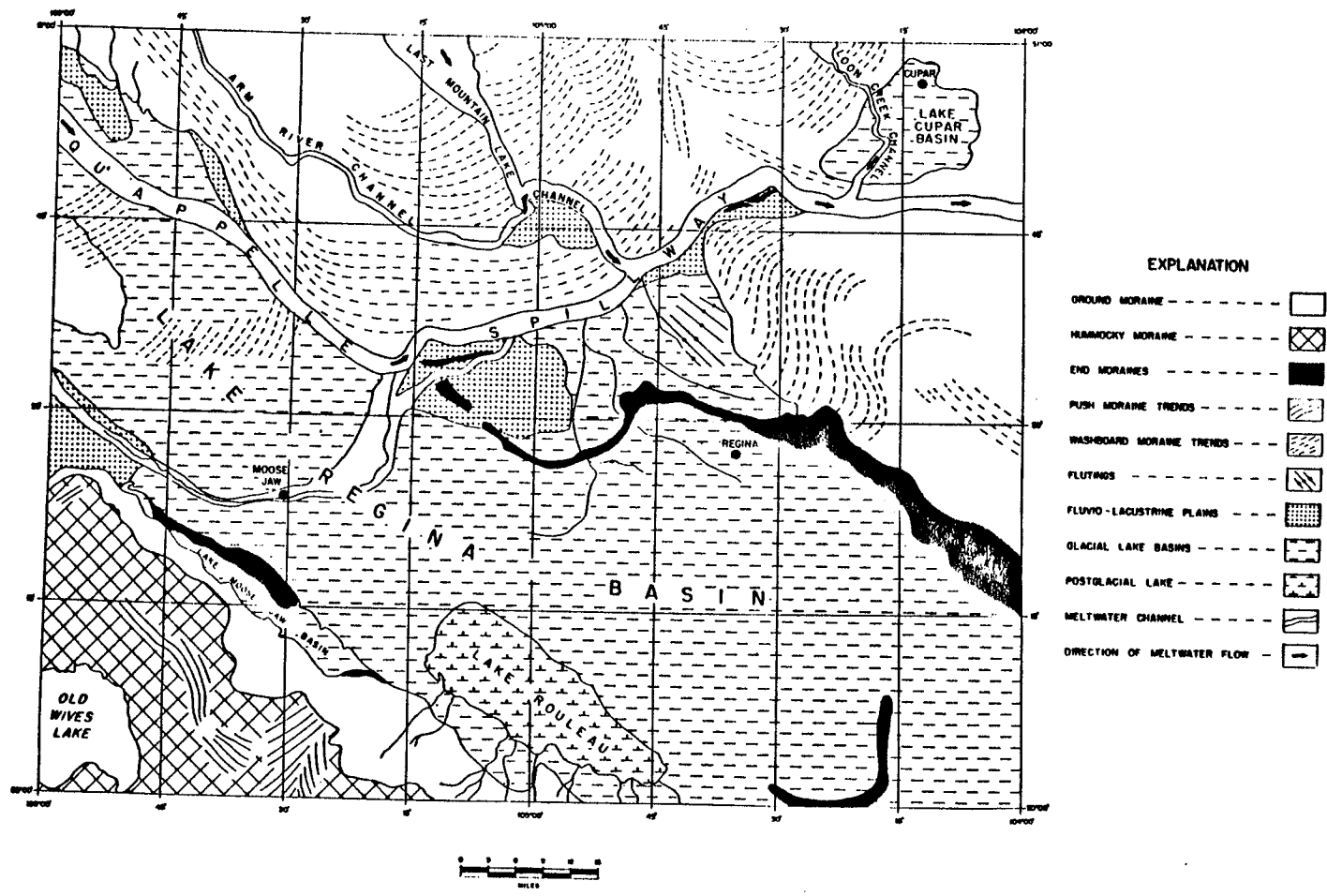


FIG. 26 GEOLOGY OF LAKE REGINA. (After Christiansen 1961).

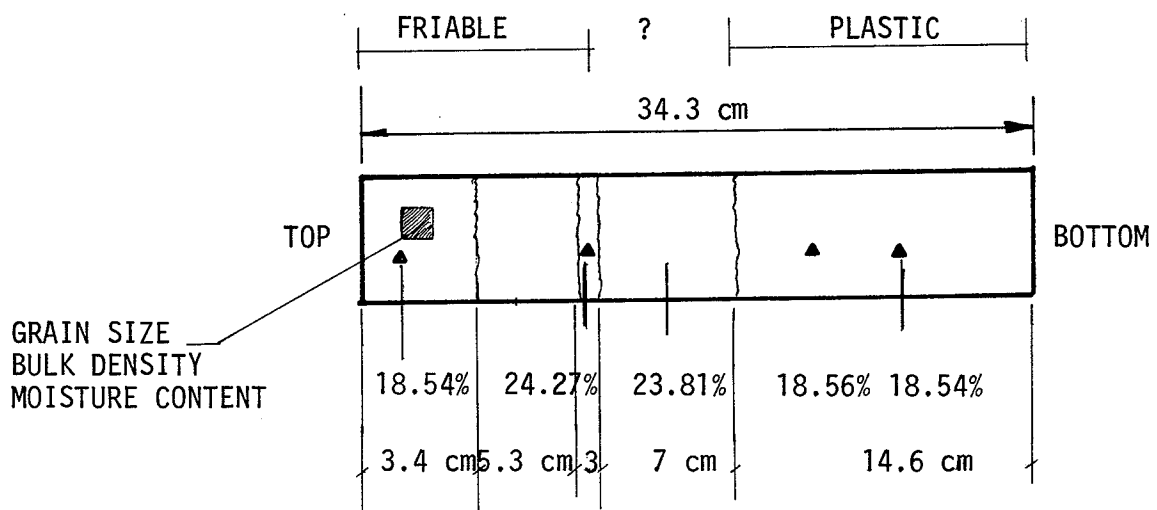


FIG. 27 SAMPLE 9284 FROM OAK BLUFF TEST SECTION, MANITOBA HIGHWAYS.

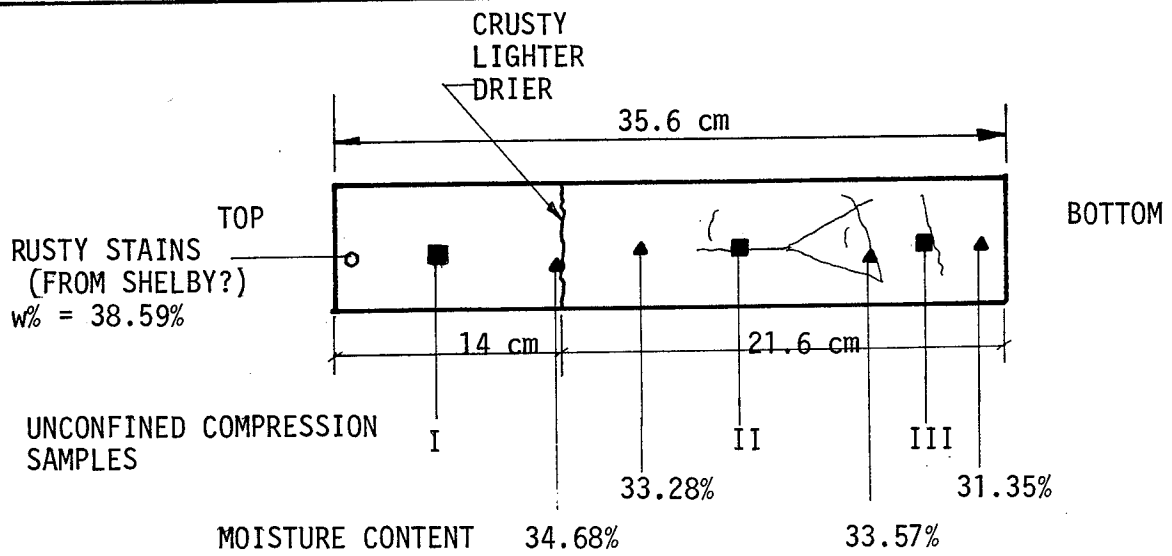


FIG. 28 SAMPLE 9285 FROM OAK BLUFF TEST SECTION, MANITOBA HIGHWAYS.

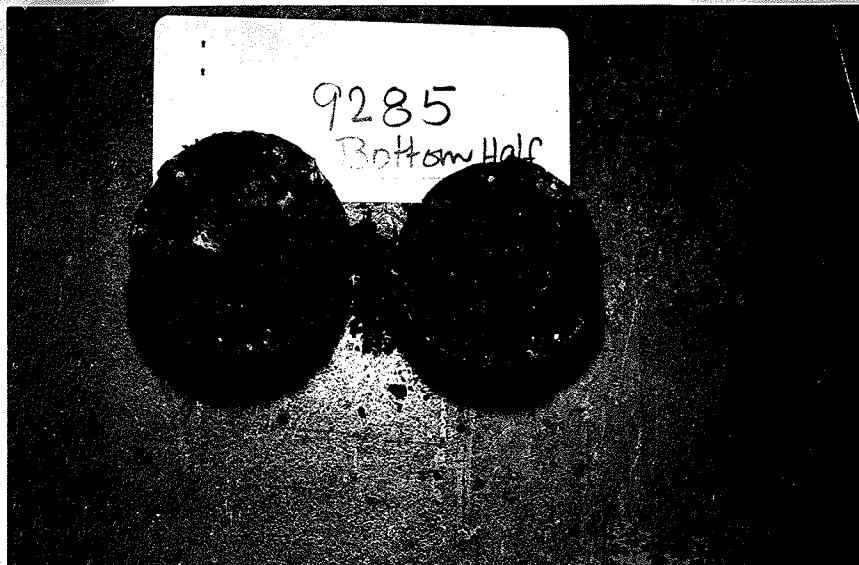
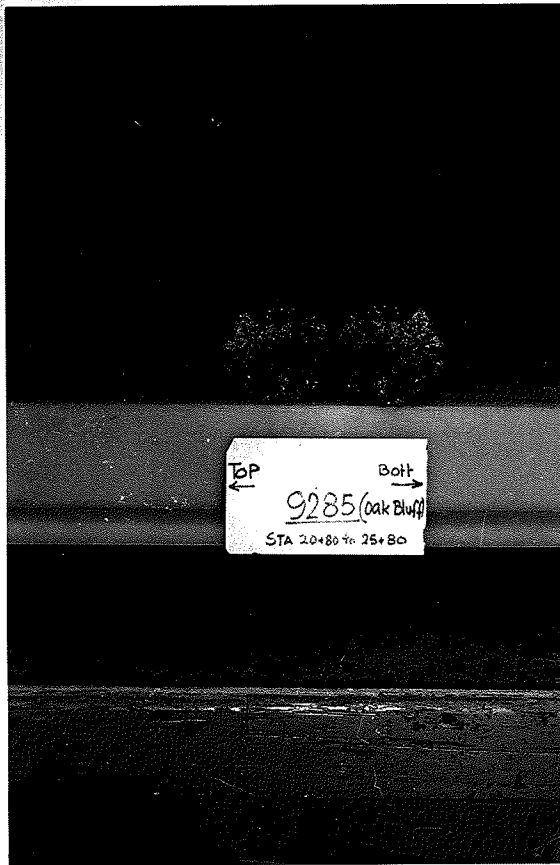


FIG. 28 A. SAMPLE 9285 SHOWING PRESENCE OF LIME.

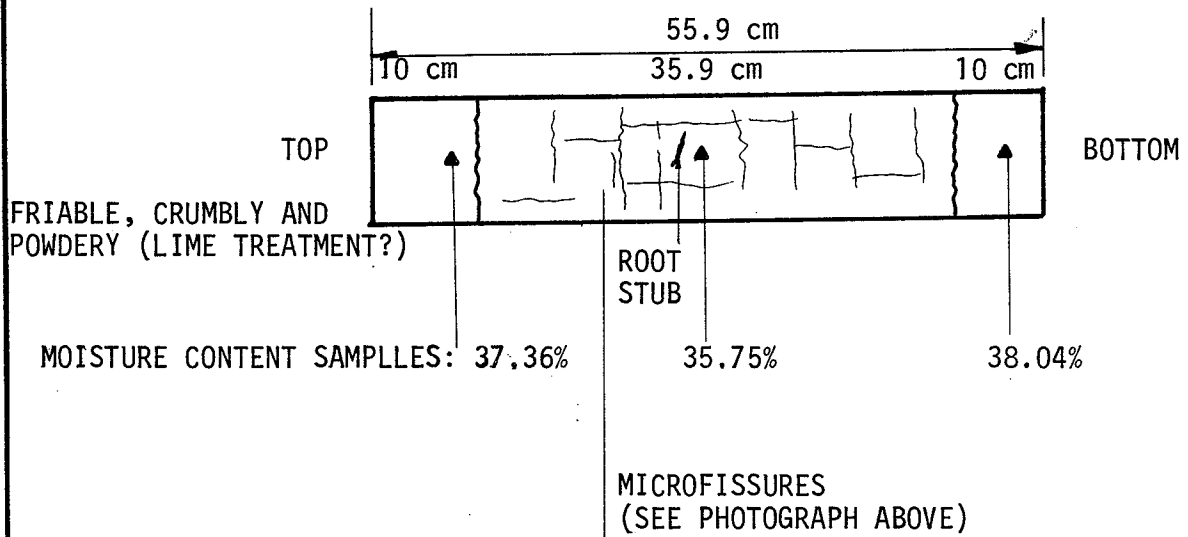
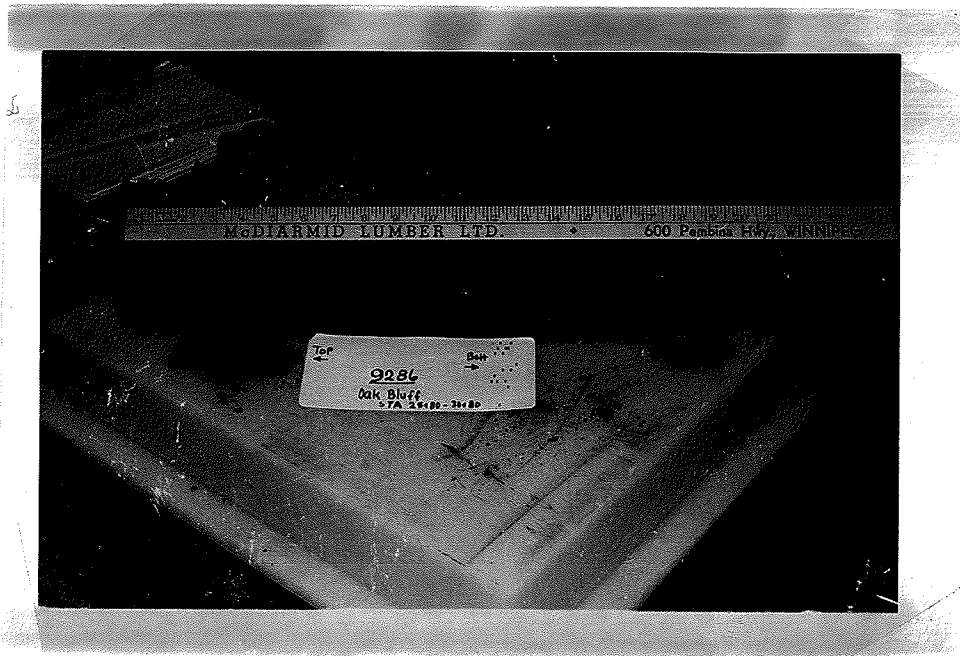


FIG.28 B SAMPLE 9286 FROM OAKBLUFF TEST SECTION, MANITOBA HIGHWAYS.

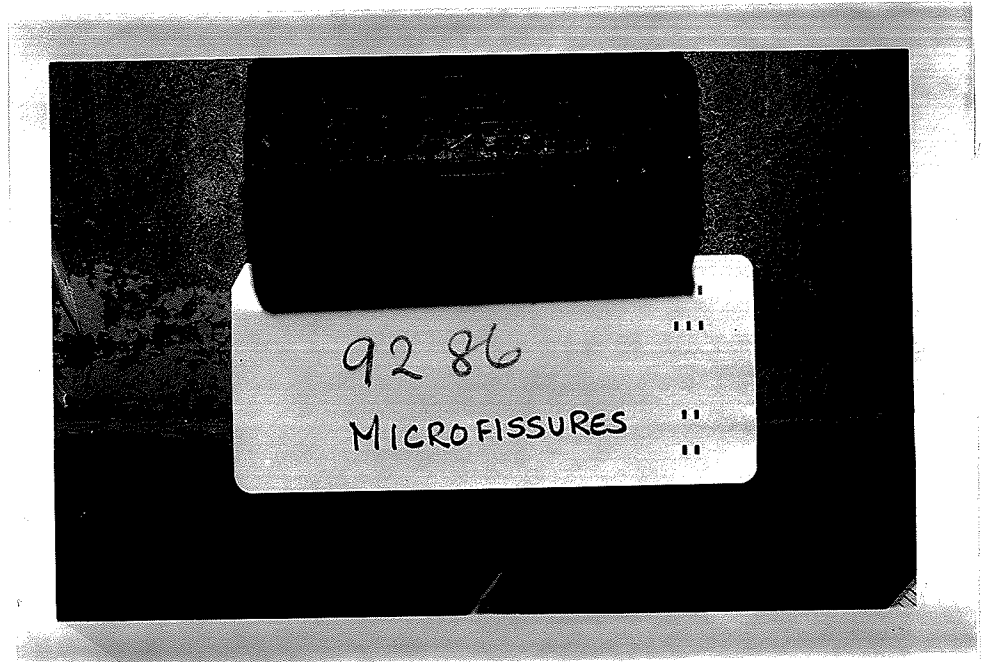


FIG. 28 C. SAMPLE 9286 FROM OAKBLUFF SHOWING MICROFISSURES.

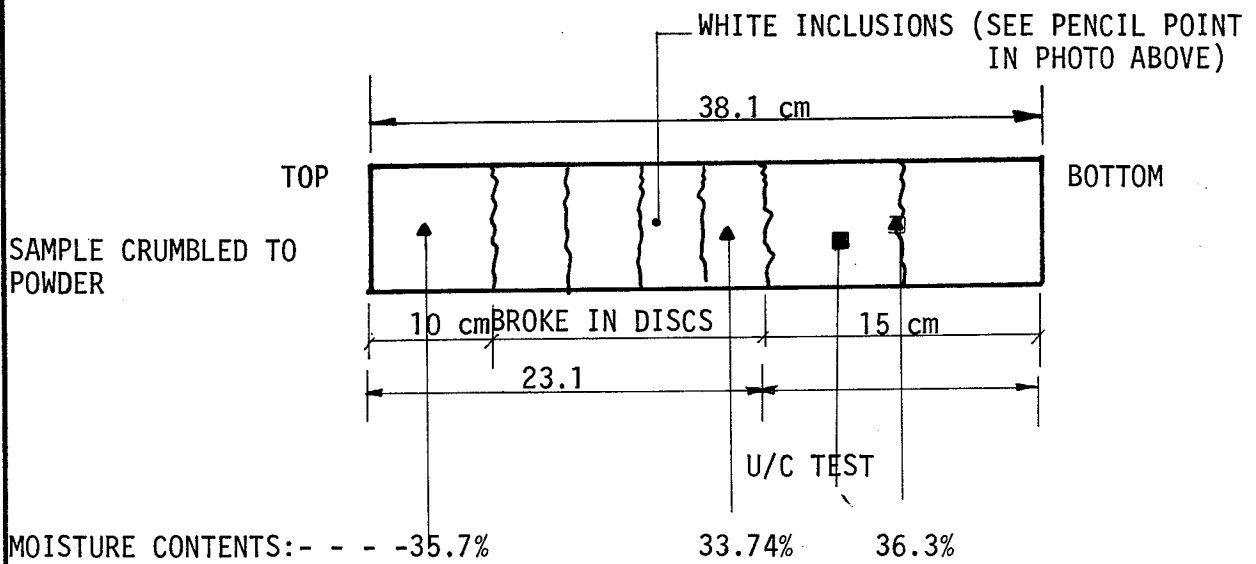
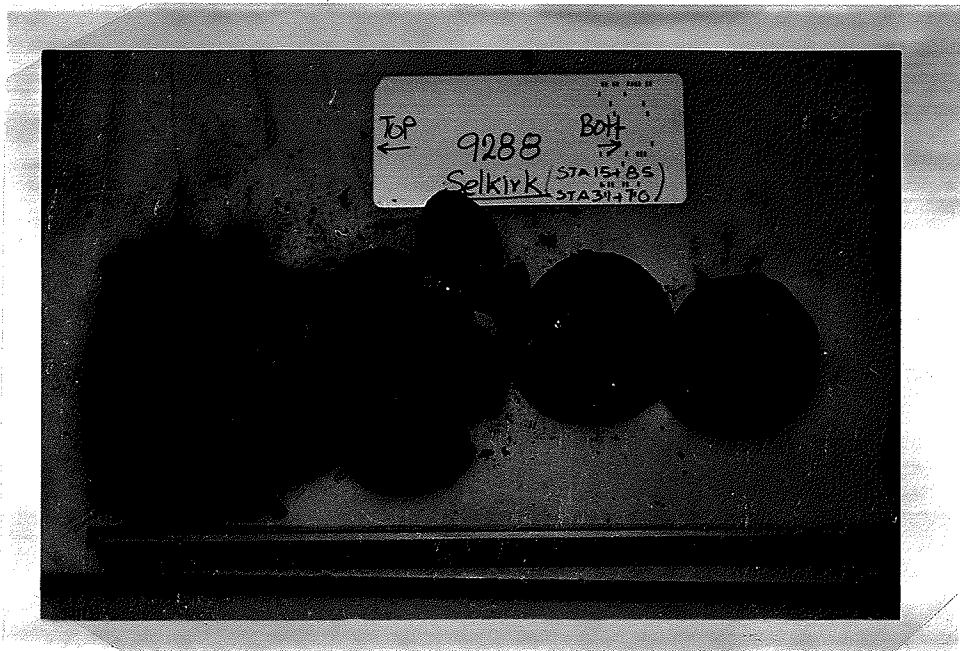


FIG 28D SAMPLE 9288 FROM SELKIRK BY-PASS TEST SECTION, MANITOBA HIGHWAYS.

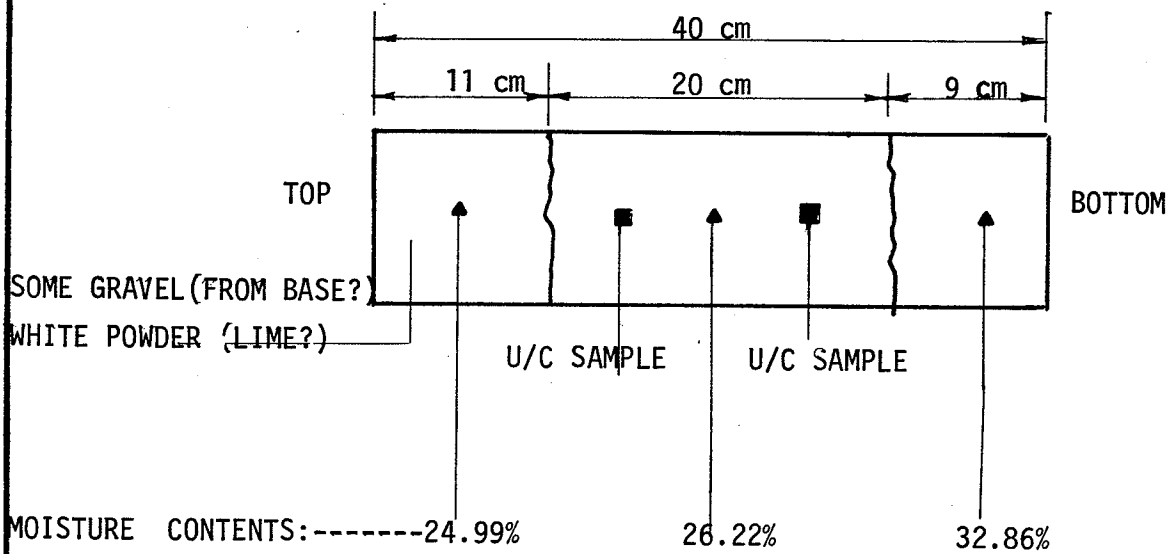


FIG. 29 A SAMPLE 9325 LIME TREATED SECTION HWY. 33/PTH 1 BY-PASS SASKATCHEWAN HIGHWAYS.

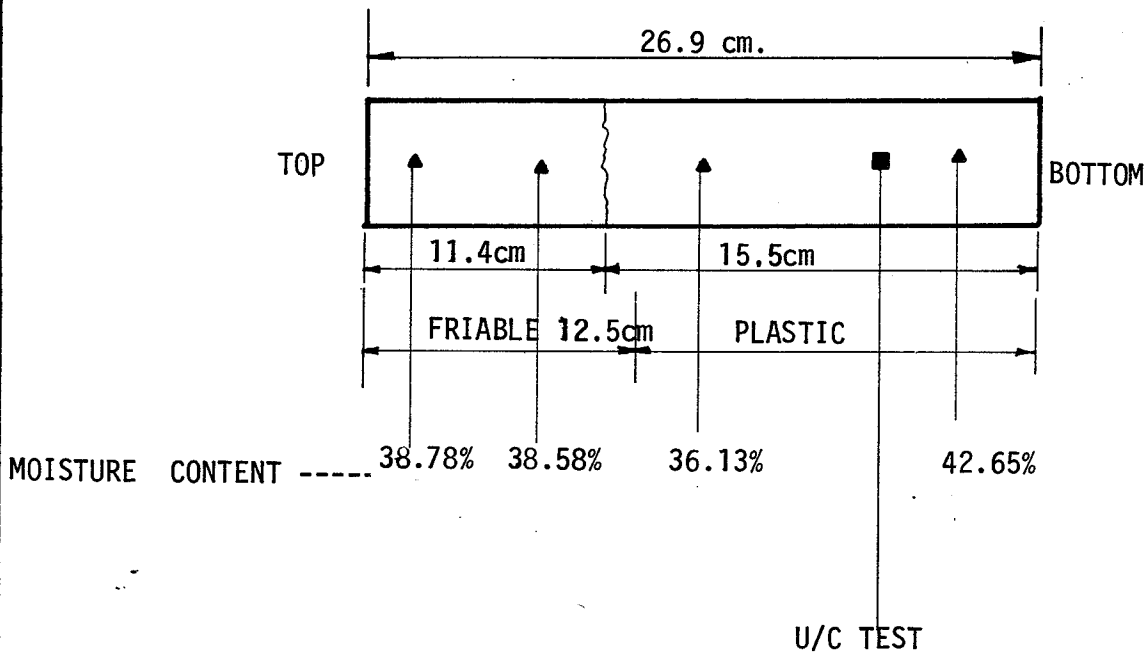


FIG 29B SAMPLE 9324 FROM LIME TREATED SECTION OF KALIUM ROAD, SASKATCHEWAN HIGHWAYS.

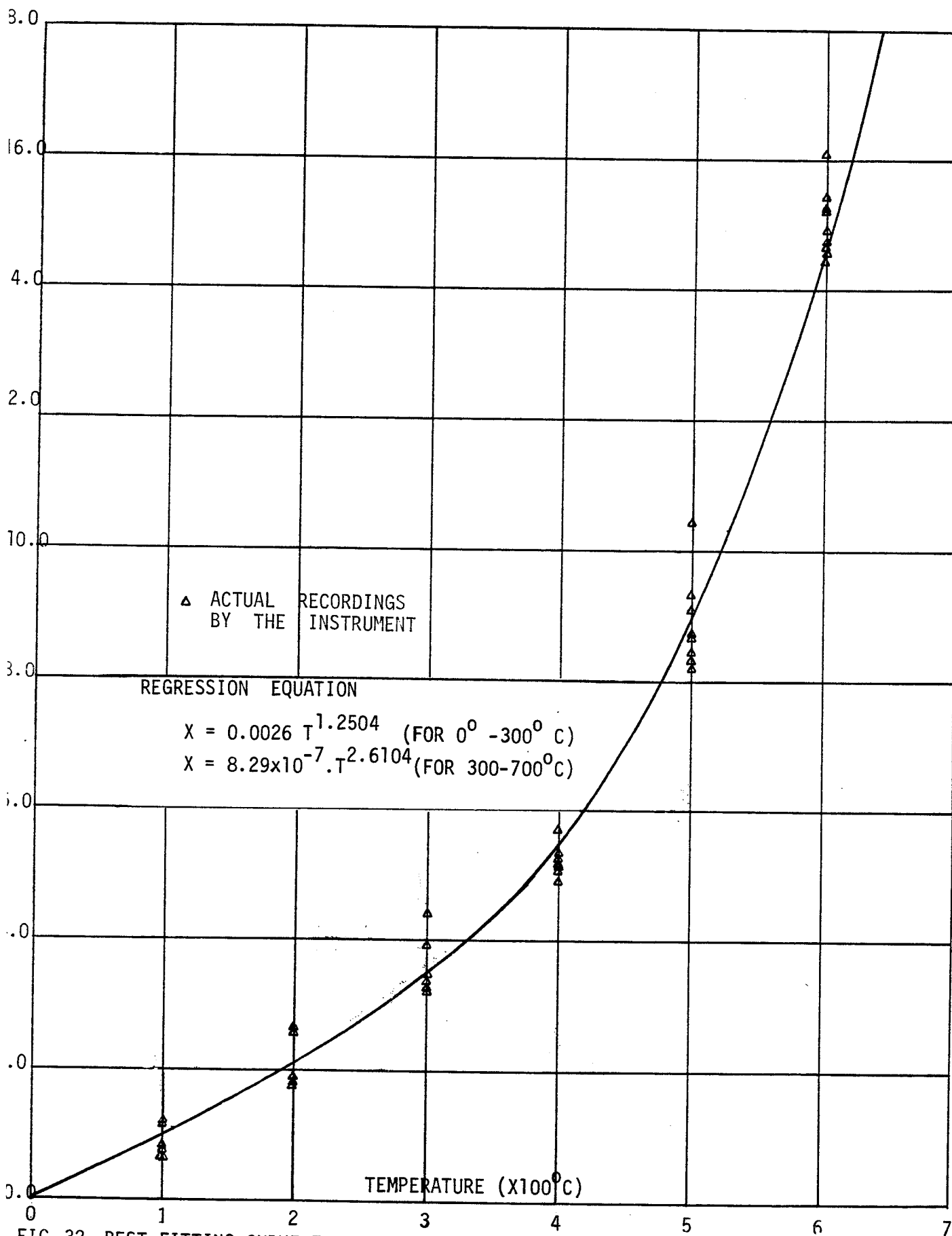


FIG. 32 BEST FITTING CURVE FOR RATE OF HEATING IN THE DTA APPARATUS. (NATURAL SCALE)

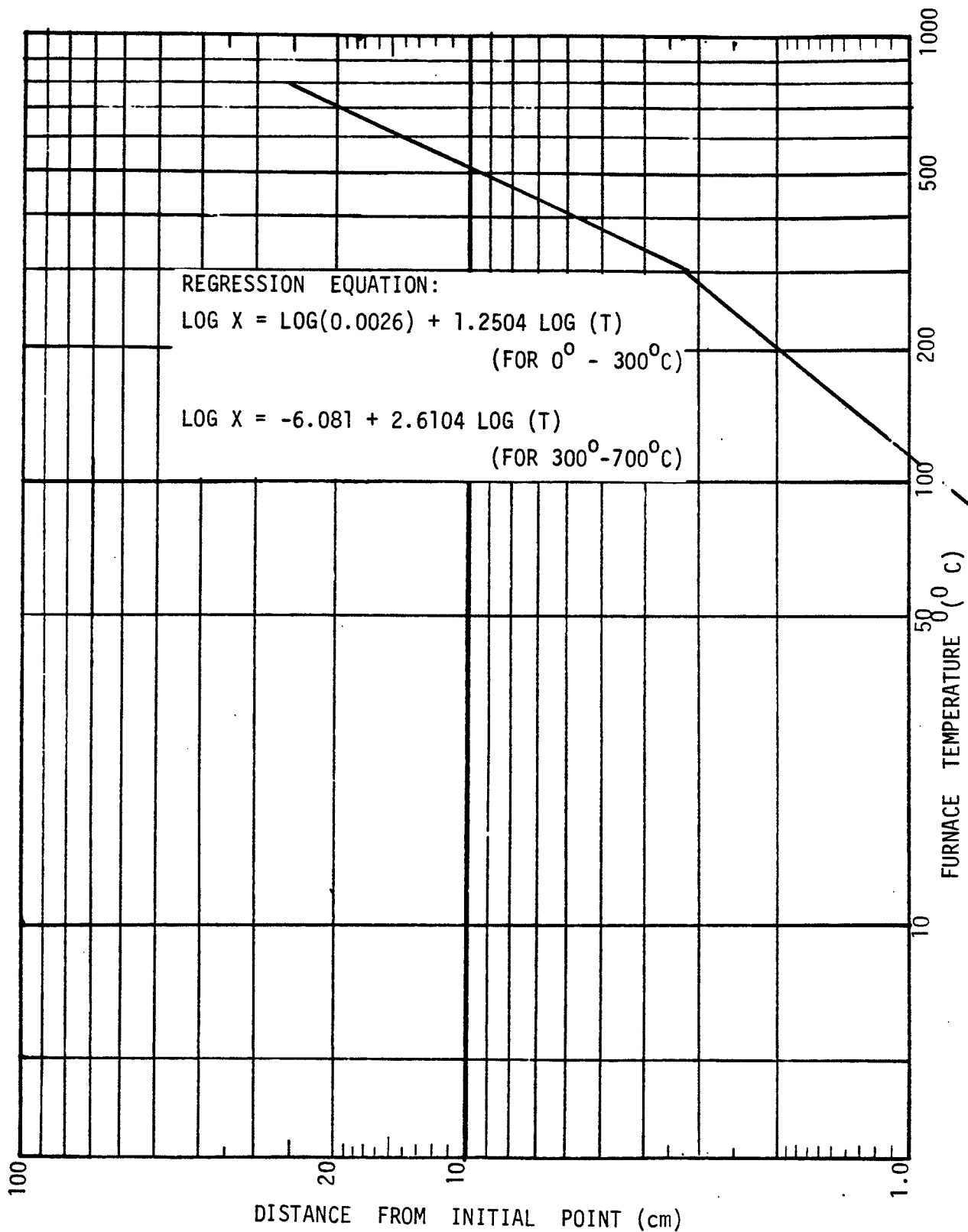


FIG.33 BEST FITTING CURVE FOR THE RATE OF HEATING IN THE DTA APPARATUS USED (LOG-LOG SCALE)

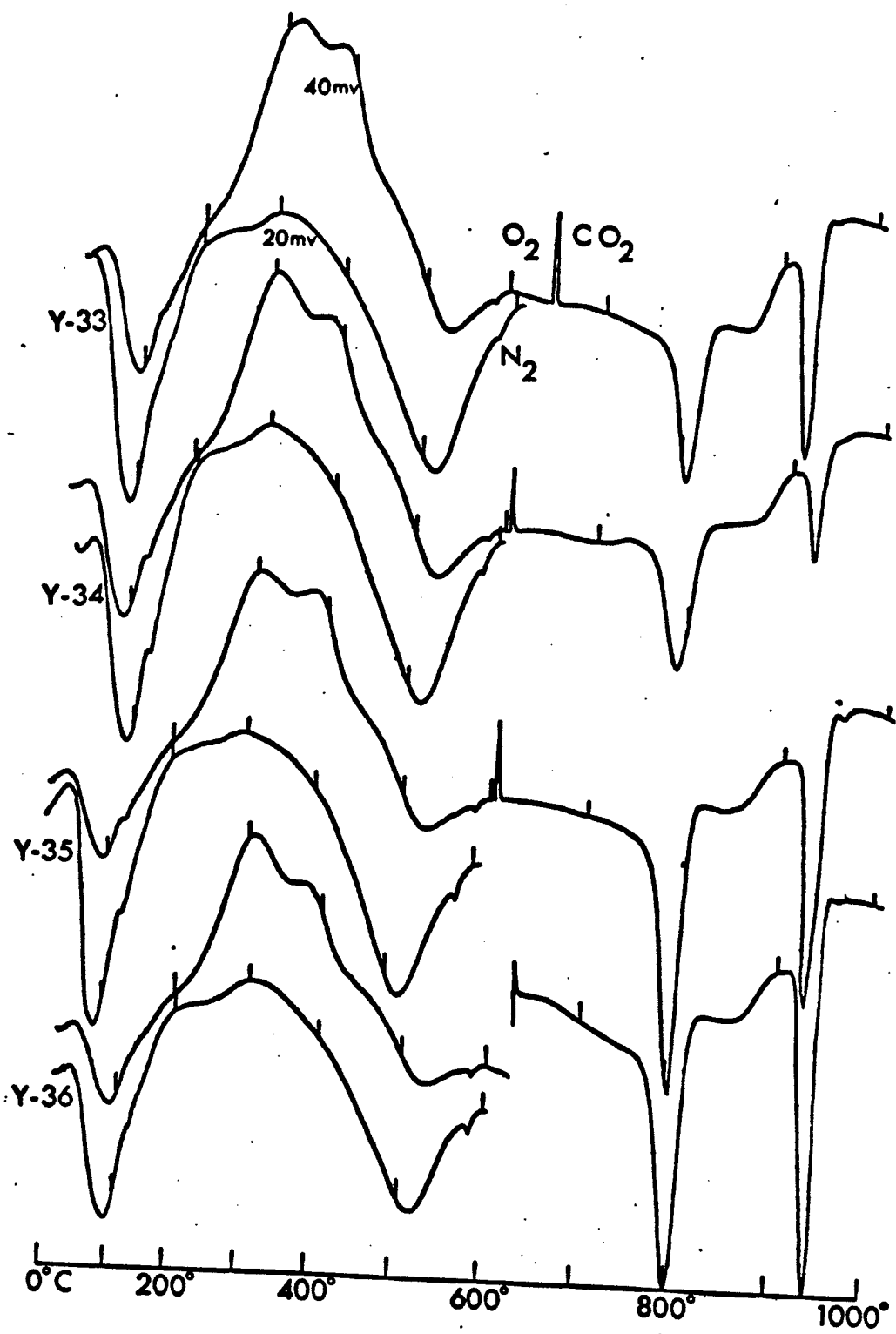


FIG. 34 DTA CURVES FOR THE LAKE AGASSIZ CLAY USING DYNAMIC GASES CO_2 , O_2 , AND N_2 (AFTER WICKS, 1965) (THE SAMPLES ARE FROM YALE AVE. IN WINNIPEG)

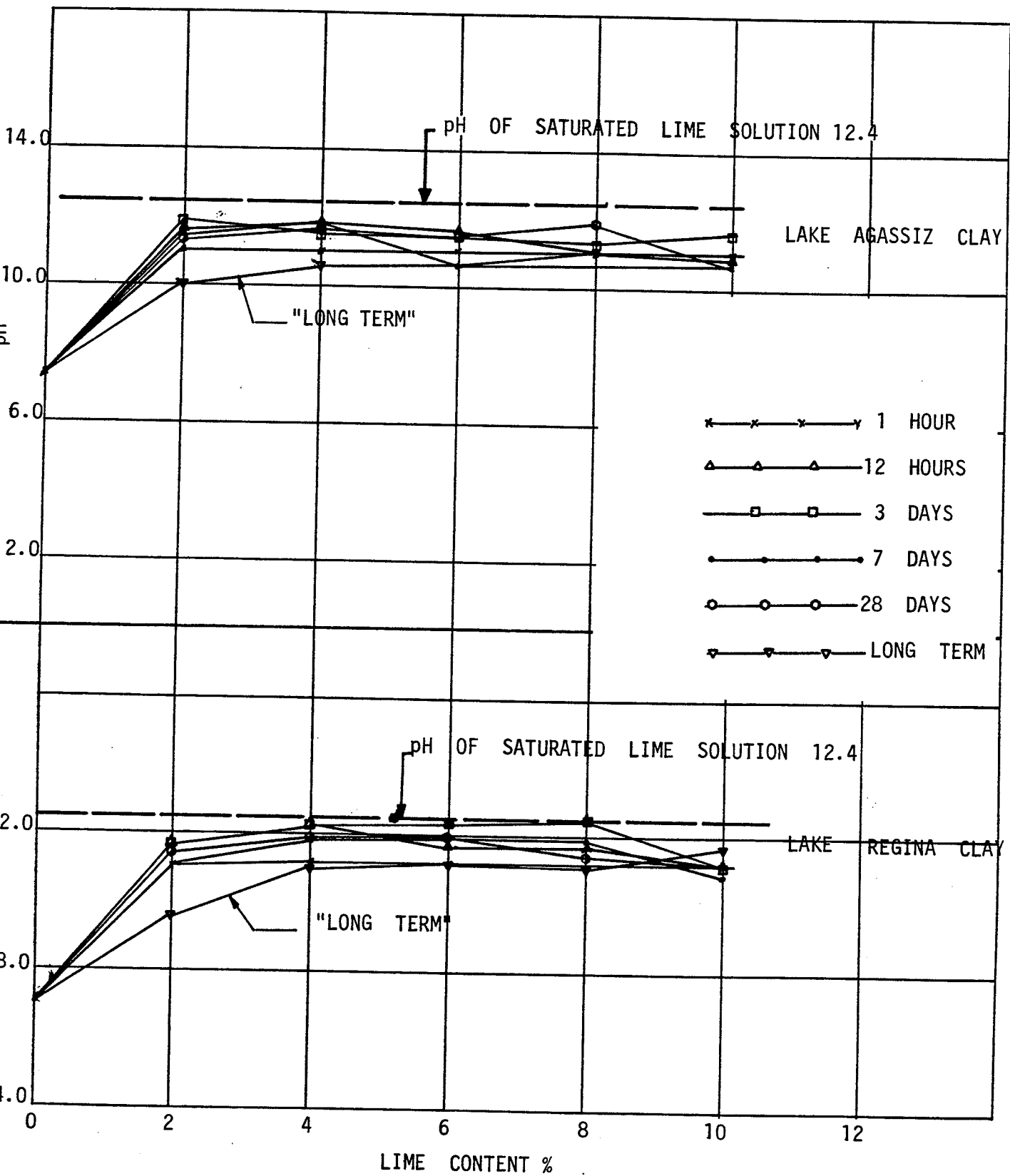


FIG. 35 pH OF LIME SOIL MIXTURES OF LAKE AGASSIZ AND LAKE REGINA CLAYS AT DIFFERENT LIME CONTENT AND AT DIFFERENT AGES (LOOSE CURING)

STANDARD SIEVE SIZES (ASTM- E 11).

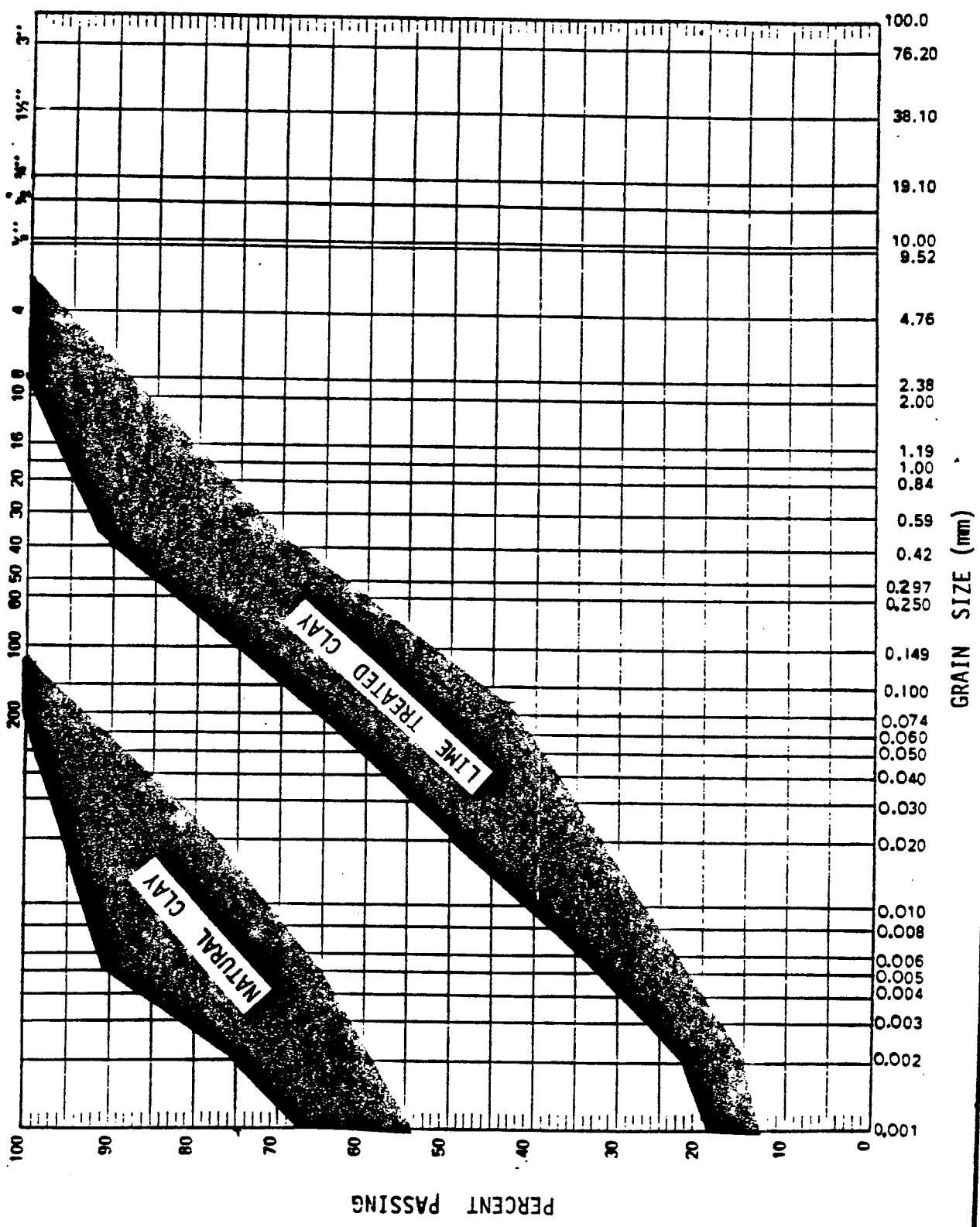


FIG-36 EFFECT OF LIME ON THE GRAIN SIZE DISTRIBUTION LAKE AGASSIZ CLAYS

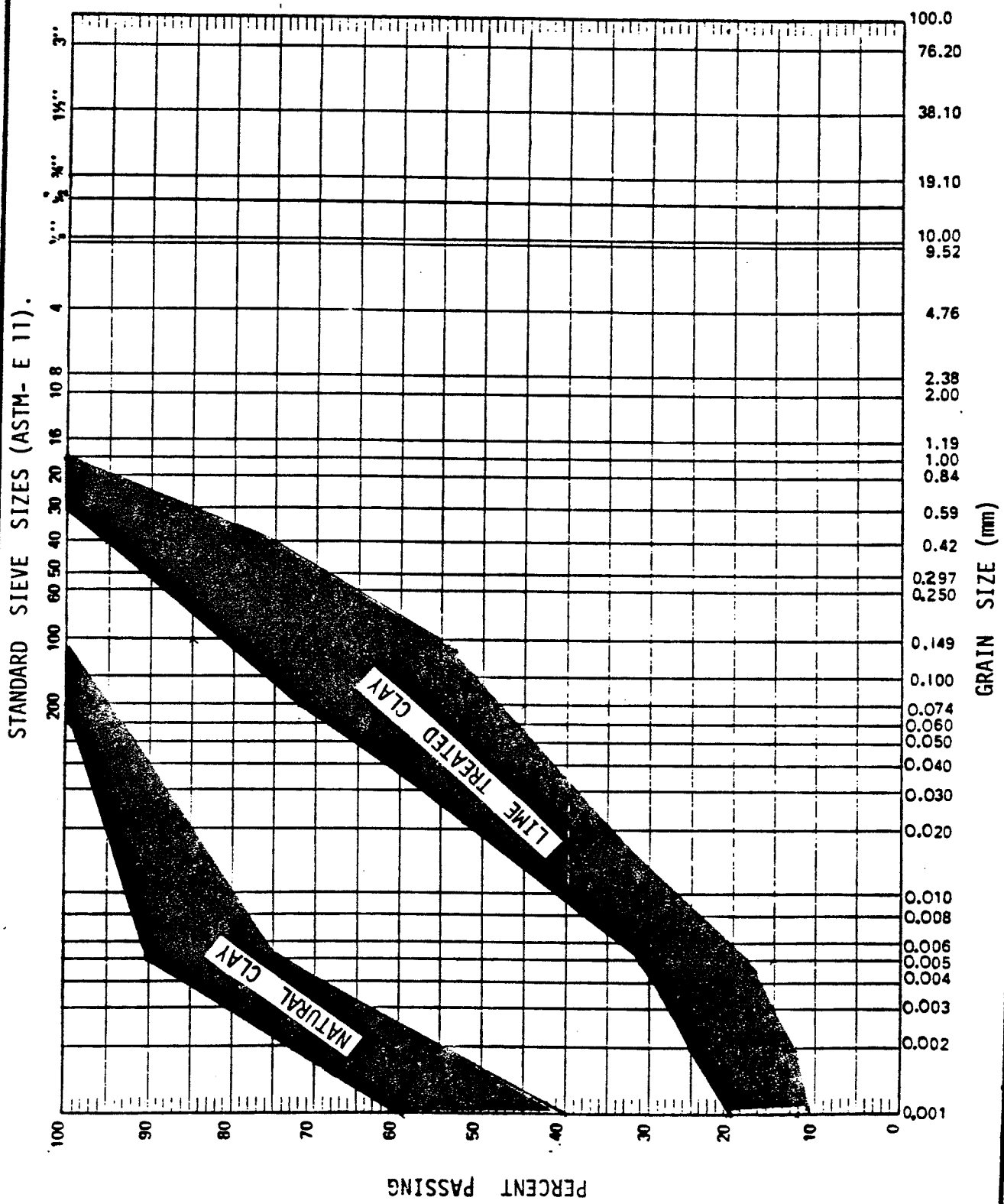


FIG-36A EFFECT OF LIME ON THE GRAIN SIZE DISTRIBUTION LAKE REGINA CLAYS

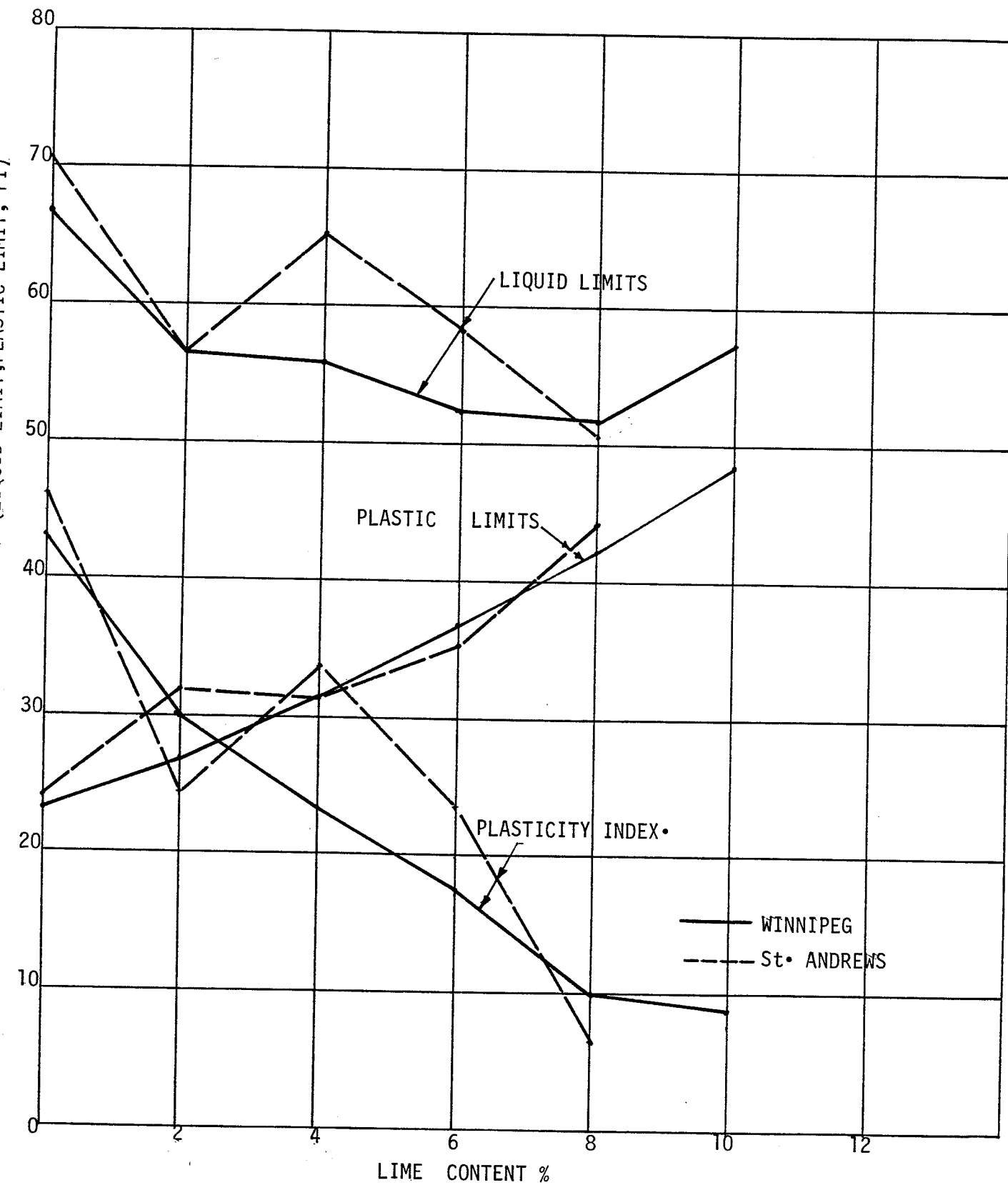


FIG. 37 EFFECT OF LIME ON THE PLASTICITY OF LAKE AGASSIZ CLAYS.

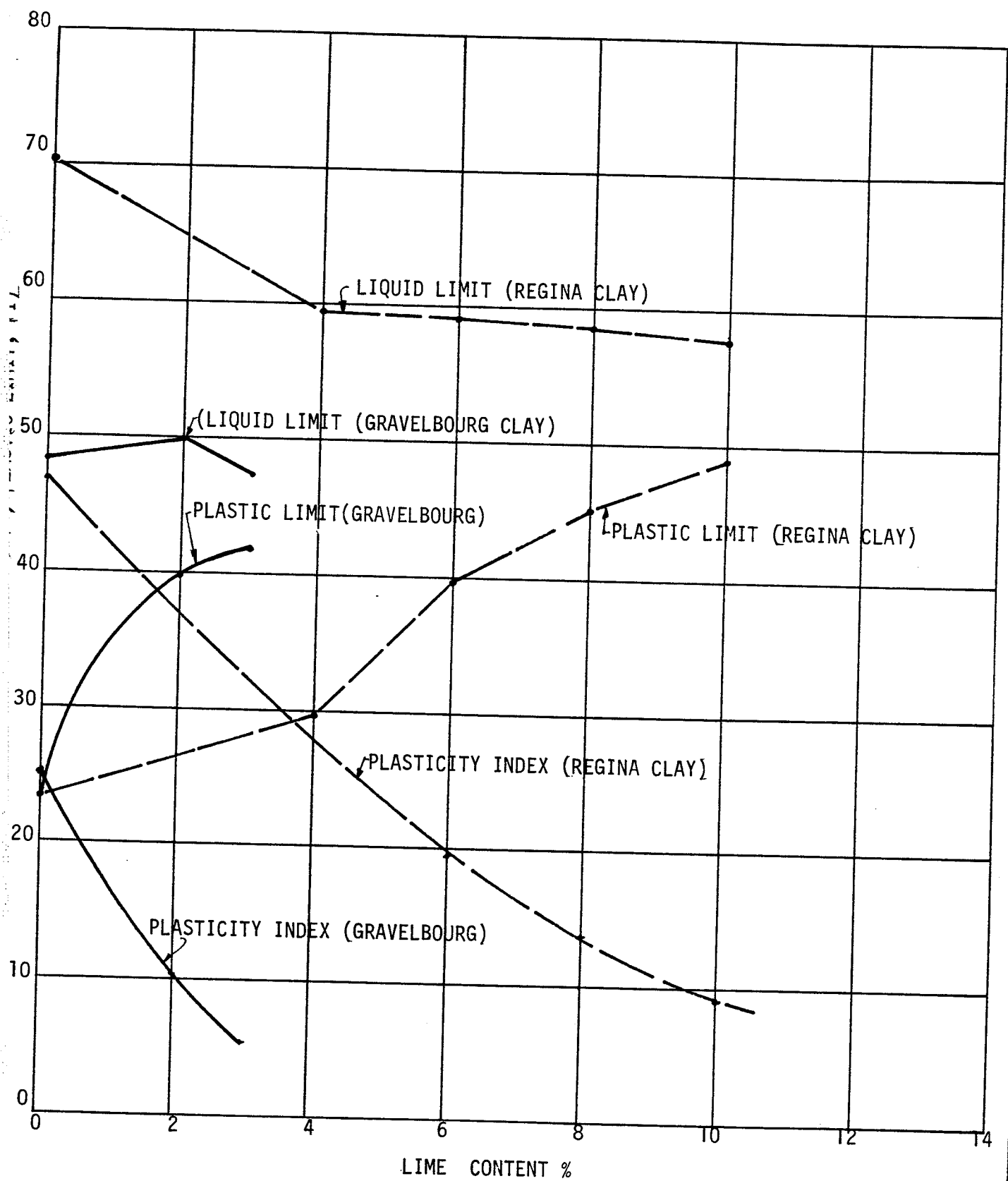


FIG. 38 EFFECT OF LIME ON THE PLASTICITY OF LAKE REGINA CLAYS.

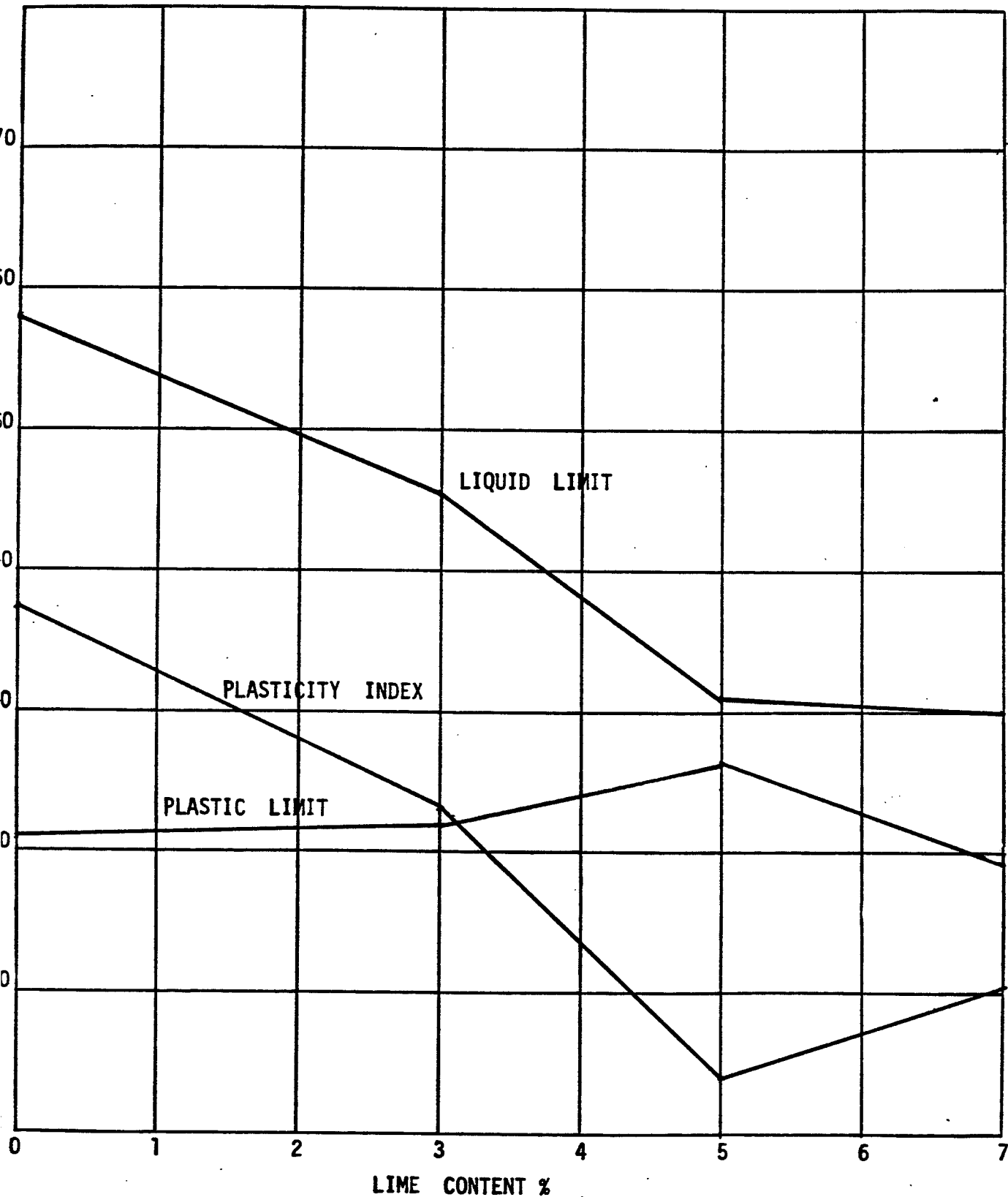


FIG.39 EFFECT OF LIME ON THE PLASTICITY CHARACTERISTICS OF CLAYS FROM SOUTH WESTERN SASKATCHEWAN (LEADER)

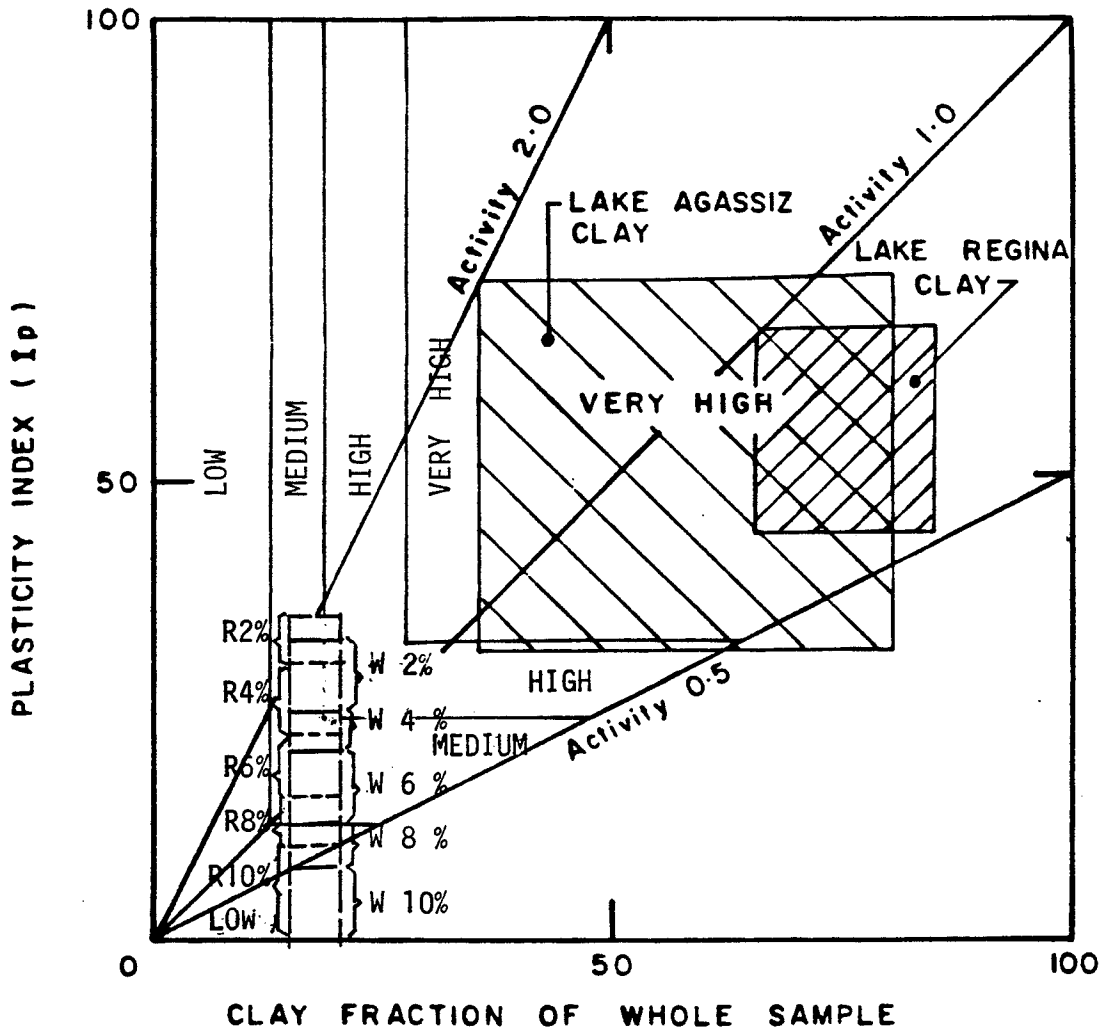


FIG. 40 VOLUME CHANGE POTENTIAL FOR WINNIPEG AND REGINA CLAYS.

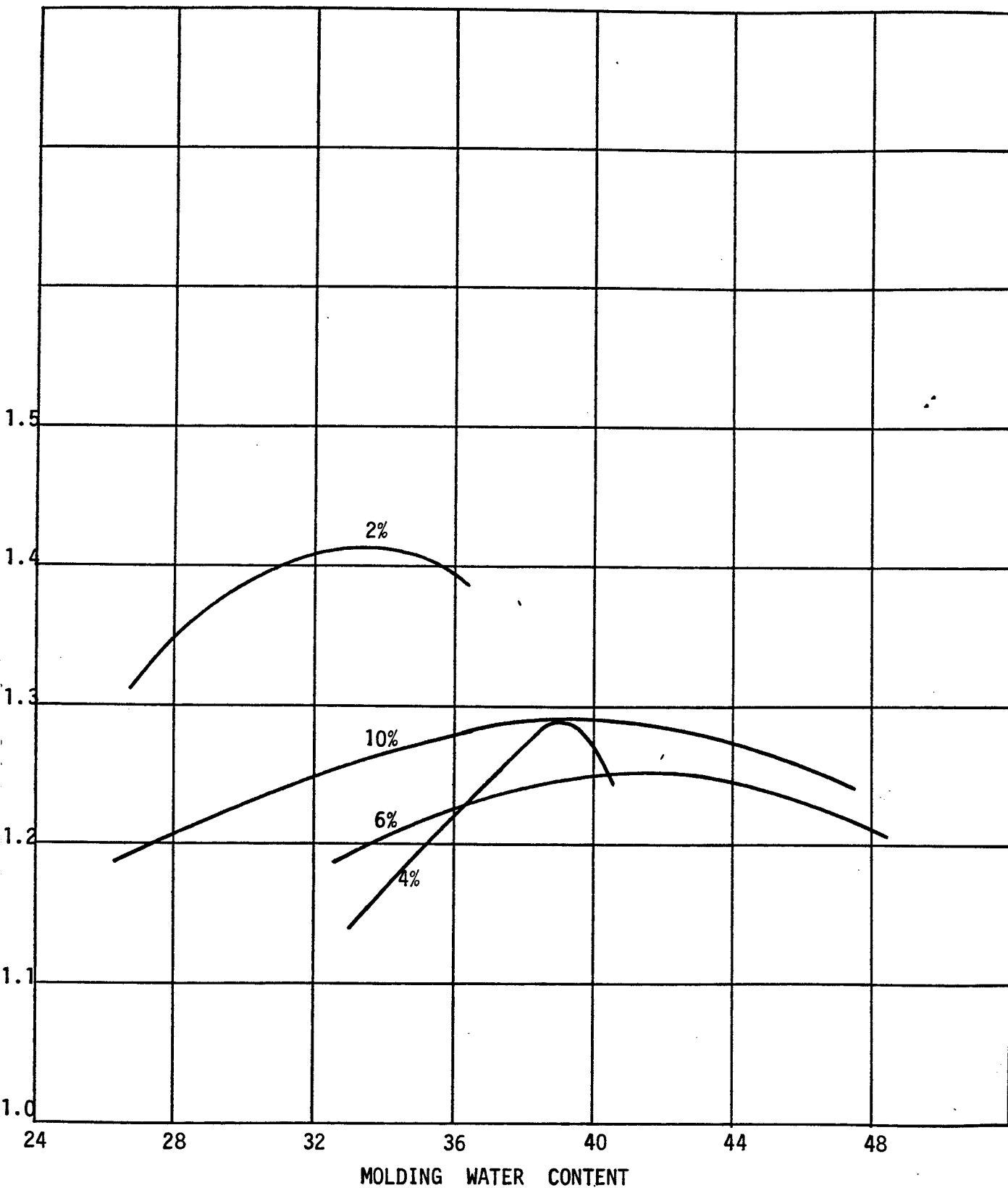


FIG. 41 MOISTURE DENSITY RELATIONSHIP FOR LAKE AGASSIZ CLAYS AT VARIOUS LIME CONTENTS AND AT 3 DAYS OF LOOSE CURING.

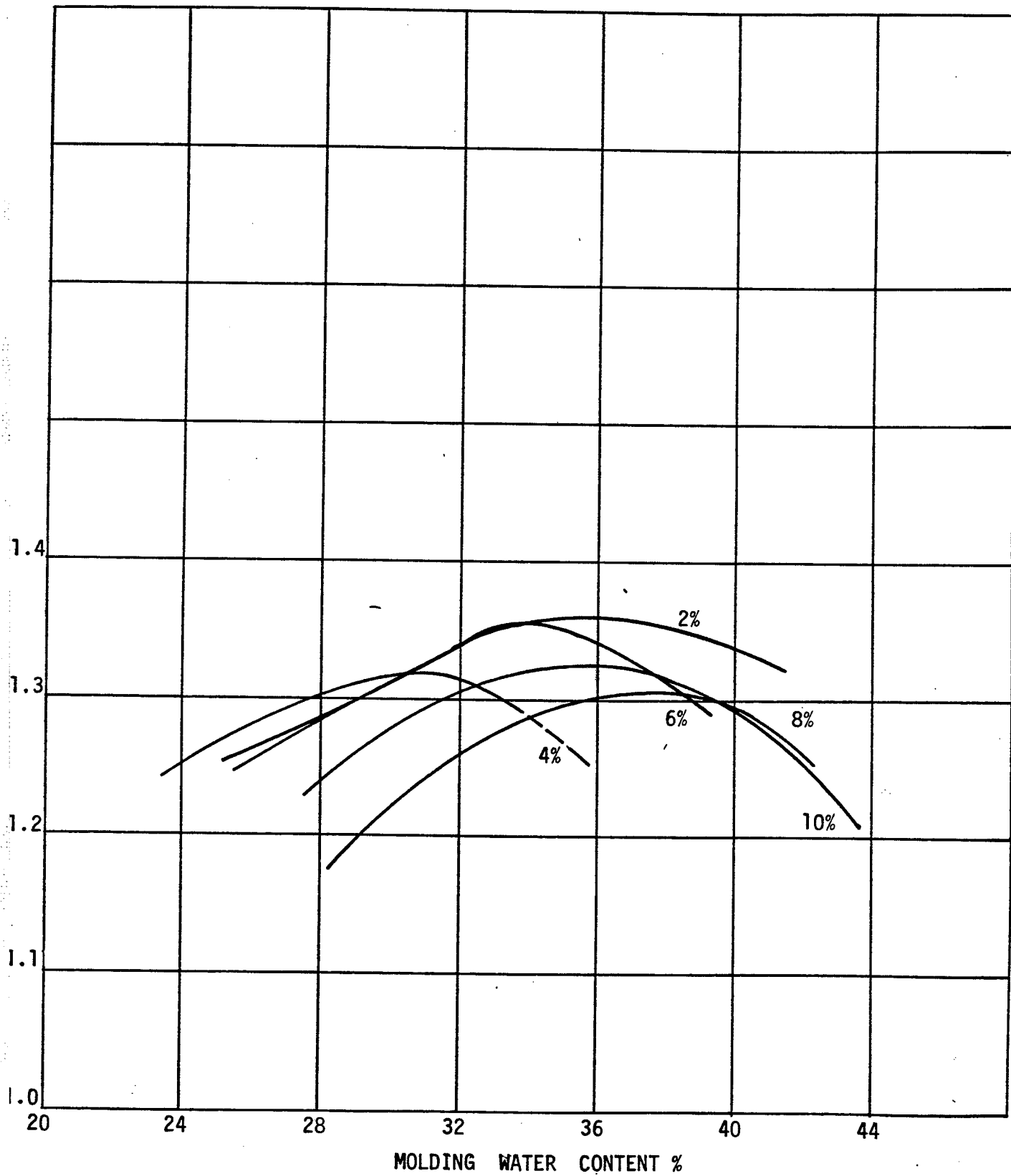


FIG • 42 MOISTURE DENSITY RELATIONSHIP FOR LAKE AGASSIZ CLAYS AT DIFFERENT LIME CONTENTS AND AT 7 DAYS OF LOOSE CURING

DRY DENSITY (kg/cm³)

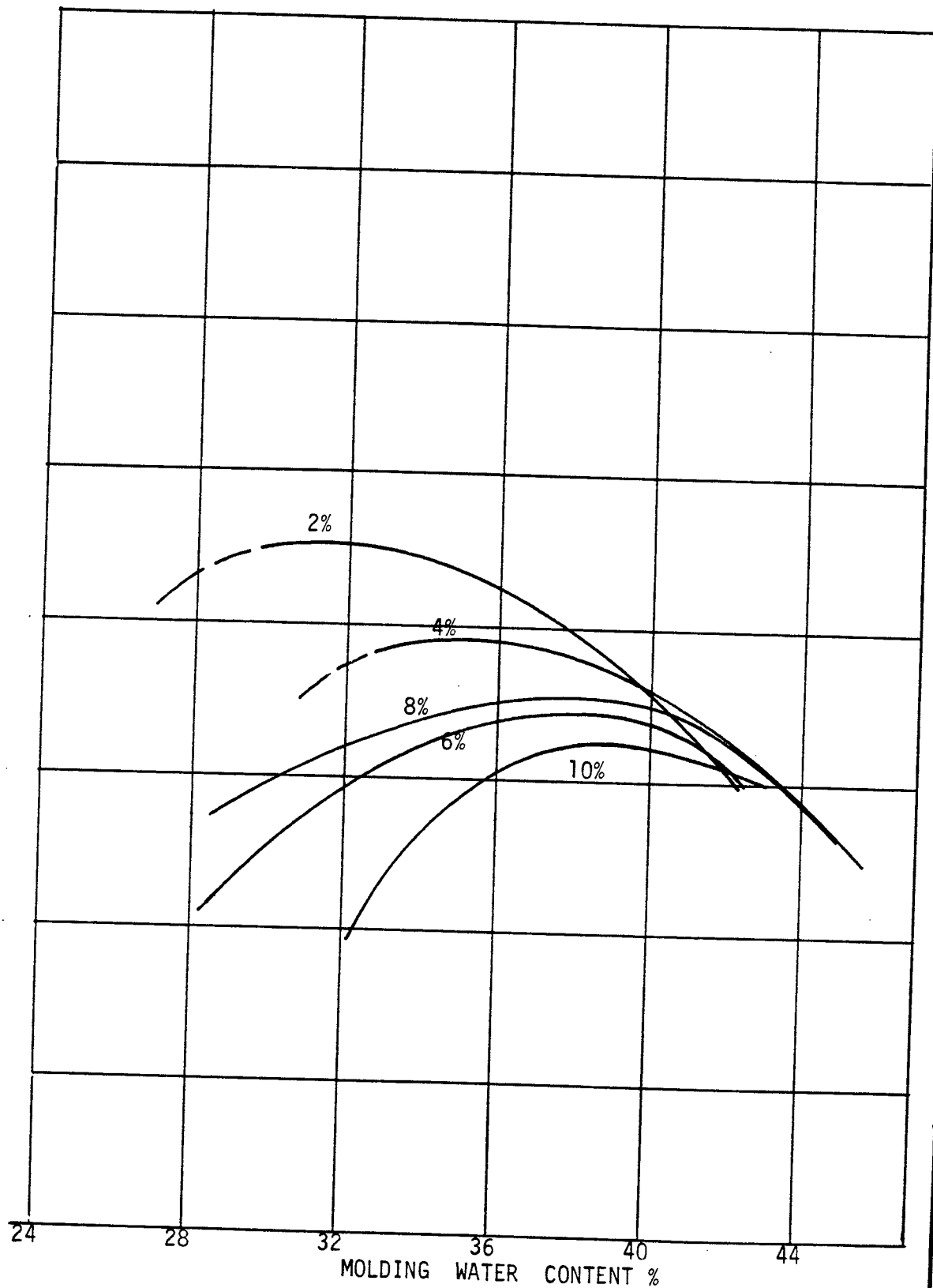


FIG. 43 MOISTURE DENSITY RELATIONSHIP FOR LAKE AGASSIZ CLAYS AT VARIOUS LIME CONTENTS AT 28 DAYS OF LOOSE CURING

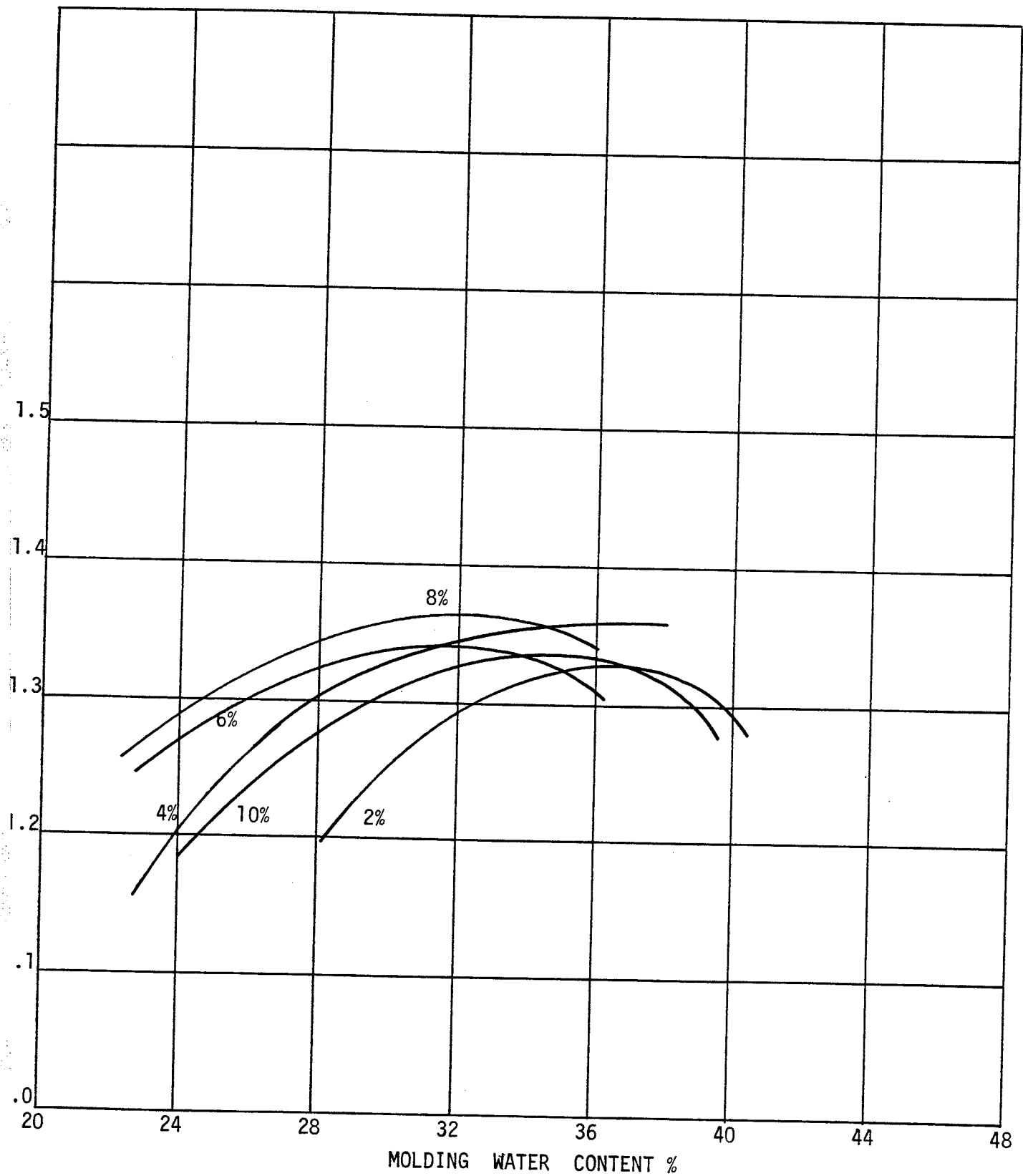


FIG.44 MOISTURE DENSITY RELATIONSHIP FOR LAKE REGINA CLAY AT VARIOUS LIME CONTENTS AND AT 3 DAYS LOOSE CURING.

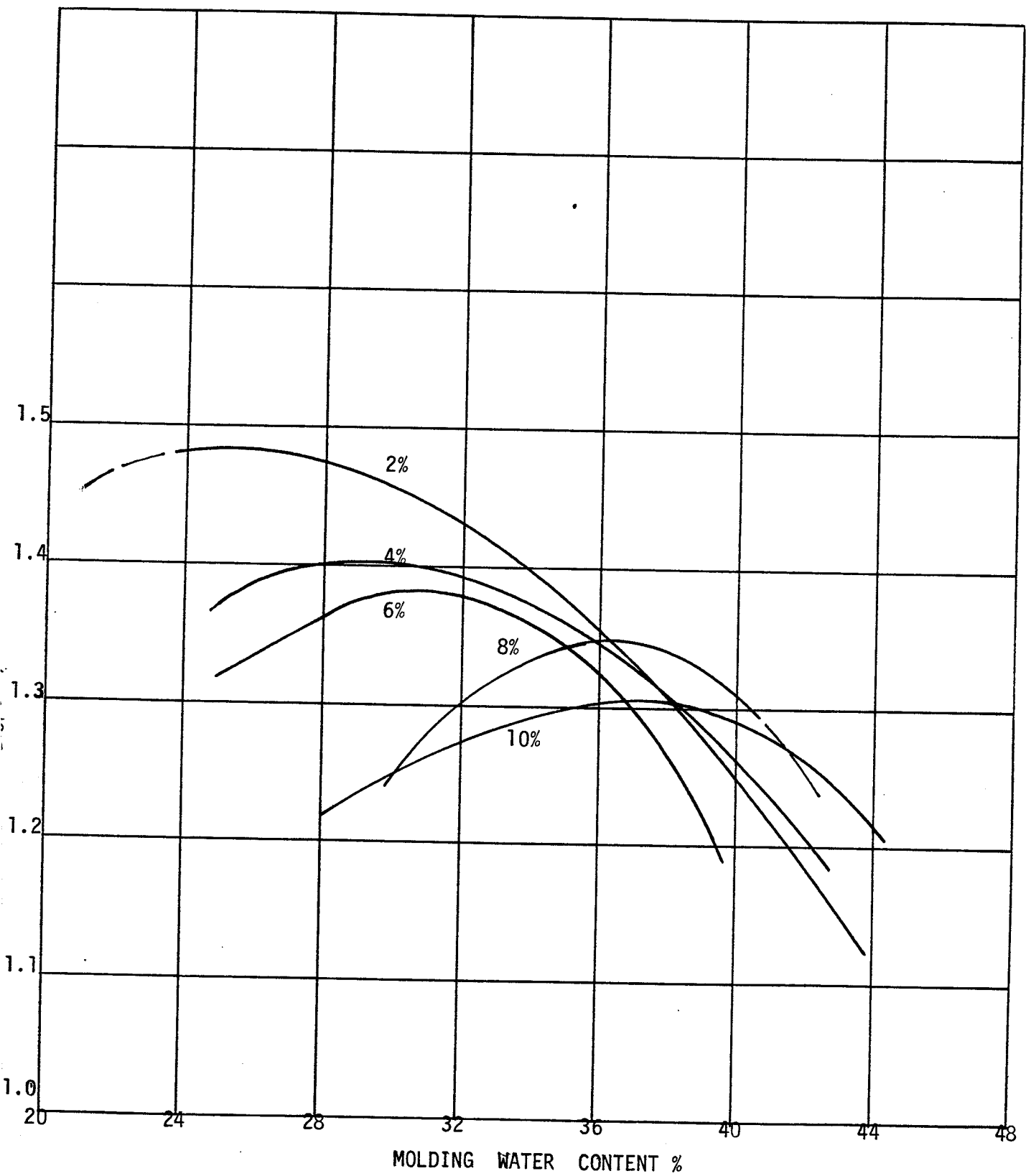


FIG. 45 MOISTURE DENSITY RELATIONSHIP FOR LAKE REGINA CLAY AT DIFFERENT LIME CONTENTS AND AT LOOSE CURING FOR 7 DAYS.

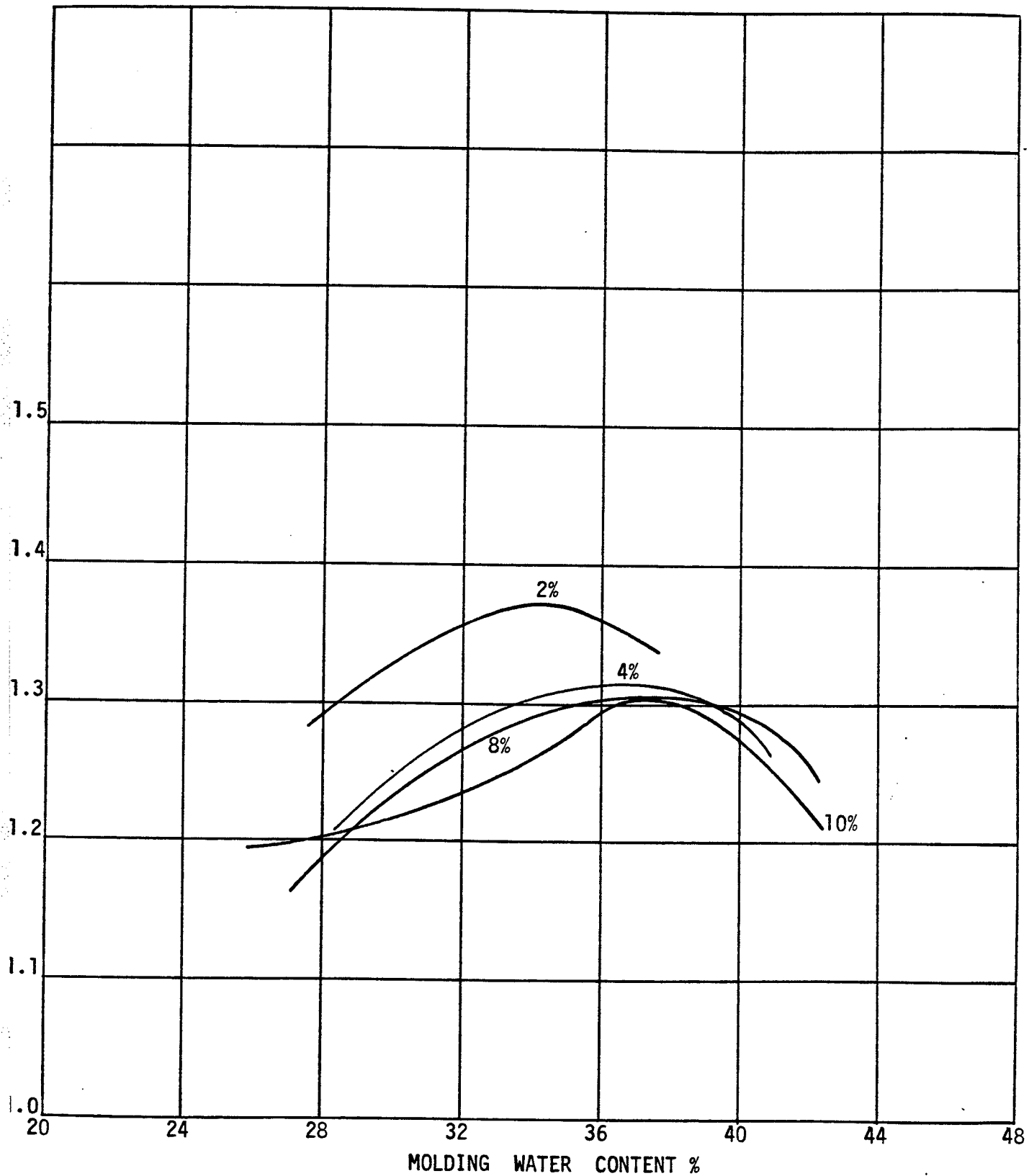


FIG. 46 MOISTURE DENSITY RELATIONSHIP FOR THE LAKE REGINA CLAY AT DIFFERENT LIME CONTENTS AND AT 28 DAYS OF LOOSE CURING.

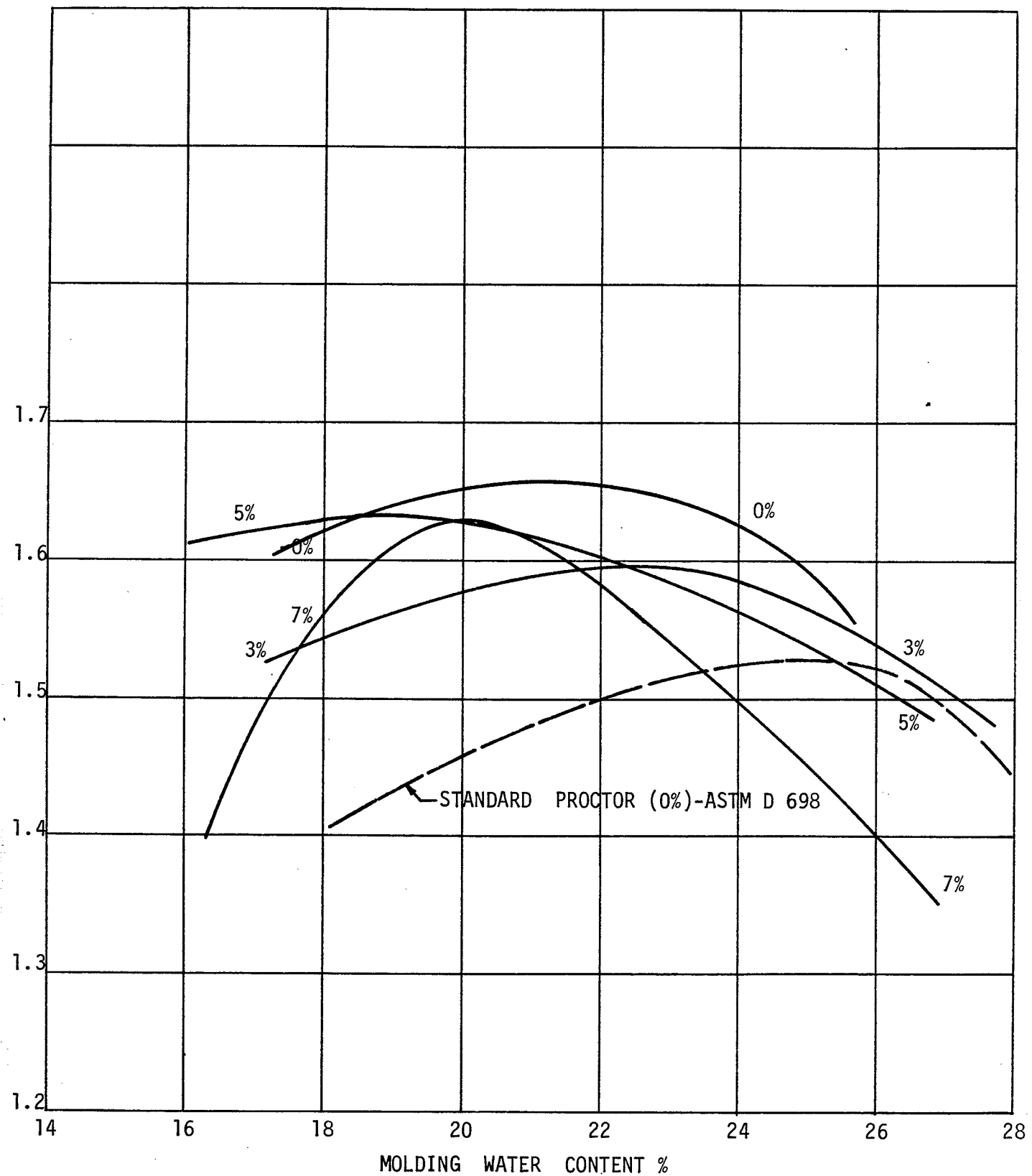


FIG. 47 MOISTURE DENSITY RELATIONSHIP FOR THE CLAYS FROM SOUTH WESTERN SASKATCHEWAN (LEADER) AT VARIOUS LIME CONTENTS. COMPACTED WITHIN 24 HRS. AFTER MIXING AND IN ACCORDANCE WITH ASTM D 1557-70.

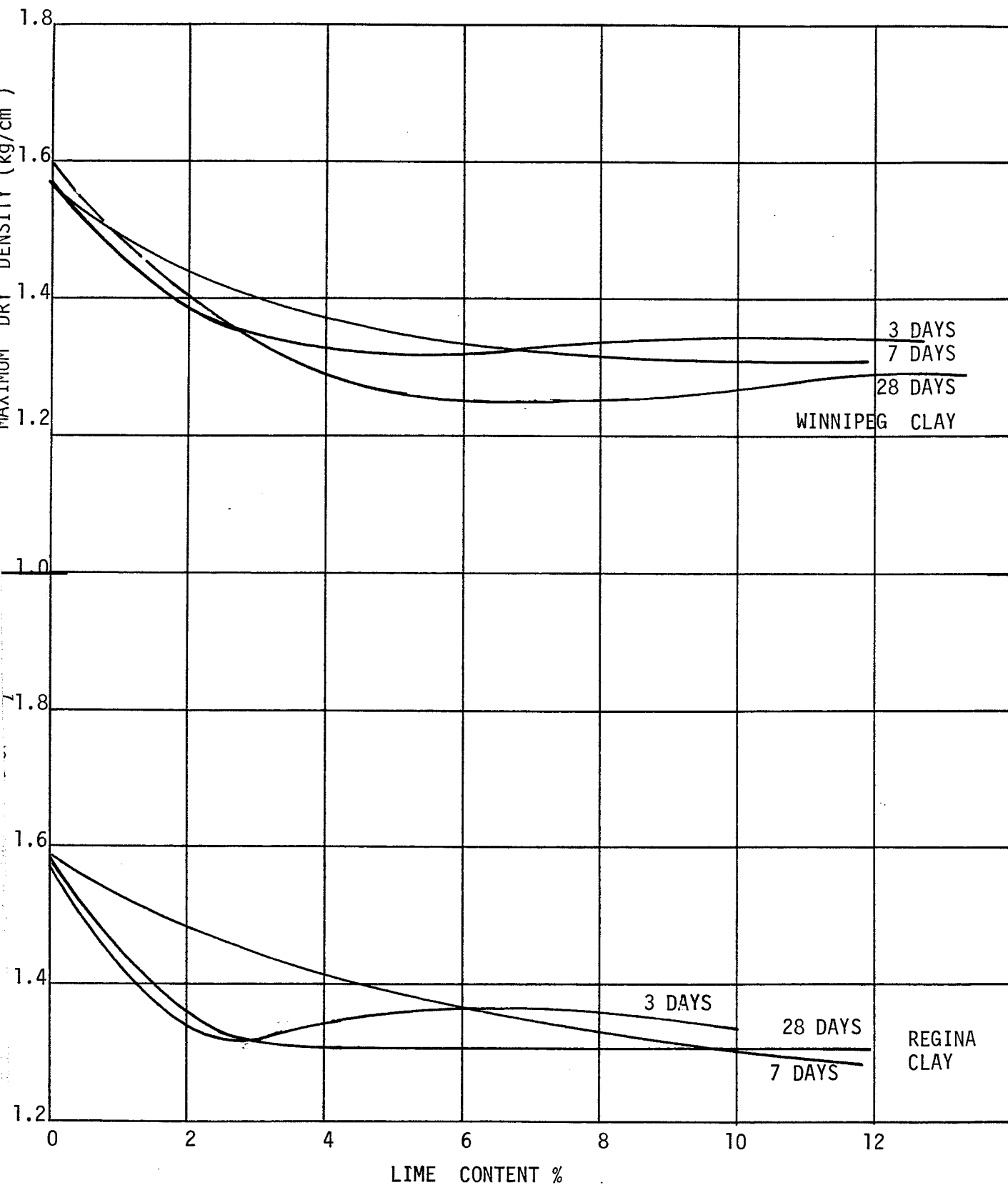


FIG.48 EFFECT OF LIME ON THE MAXIMUM DRY DENSITY OF WINNIPEG CLAY (TOP) AND REGINA CLAY(BOTTOM) AT THE SAME COMPACTIVE EFFORT. (COMPACTION BY HARVARD MINIATURE APPARATUS)

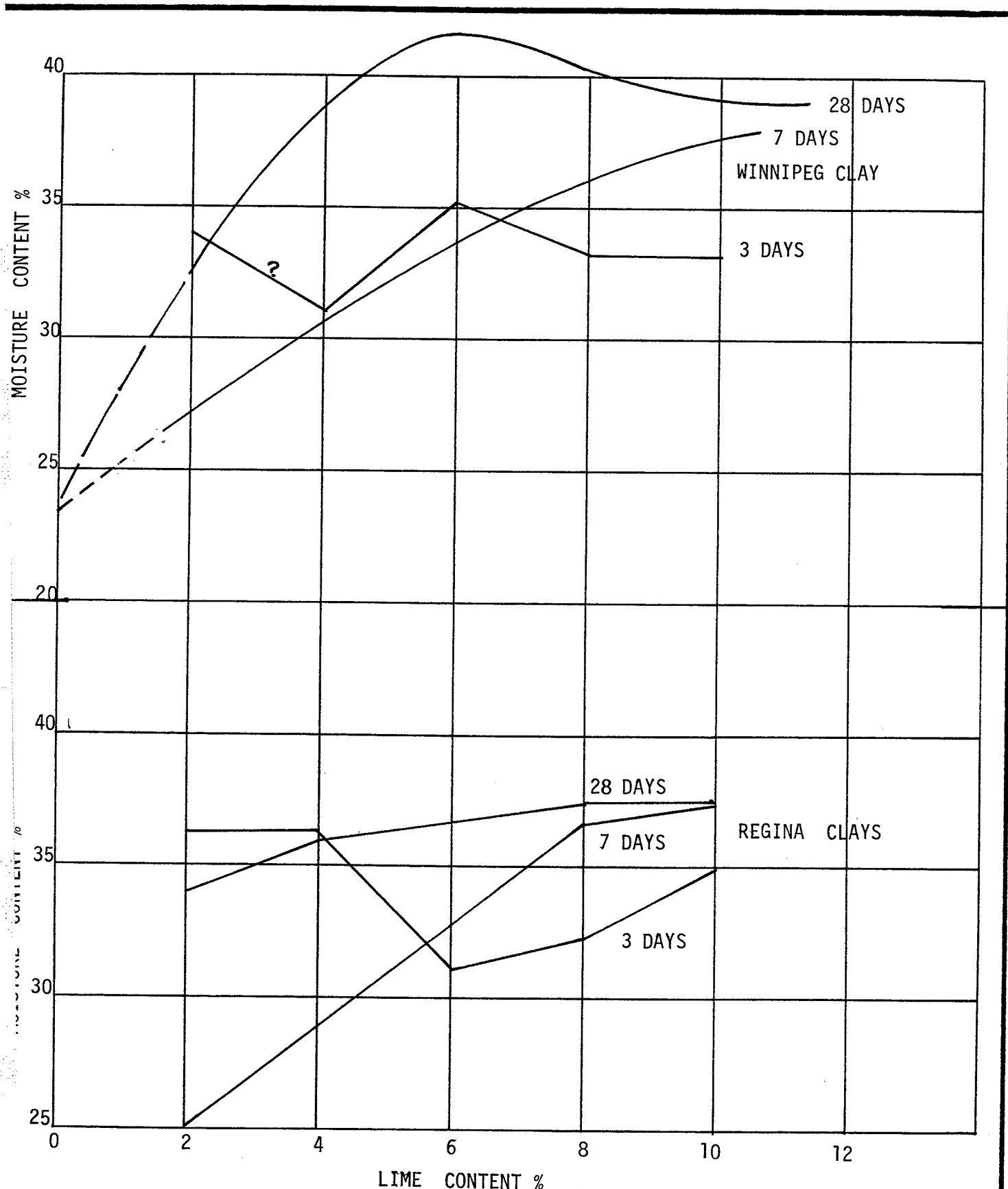


FIG.49 EFFECT OF LIME ON THE OPTIMUM MOISTURE CONTENT OF WINNIPEG AND REGINA CLAYS AT THE SAME COMPACTIVE EFFORT (COMPACTION WITH MINIATURE HARVARD APPARATUS)

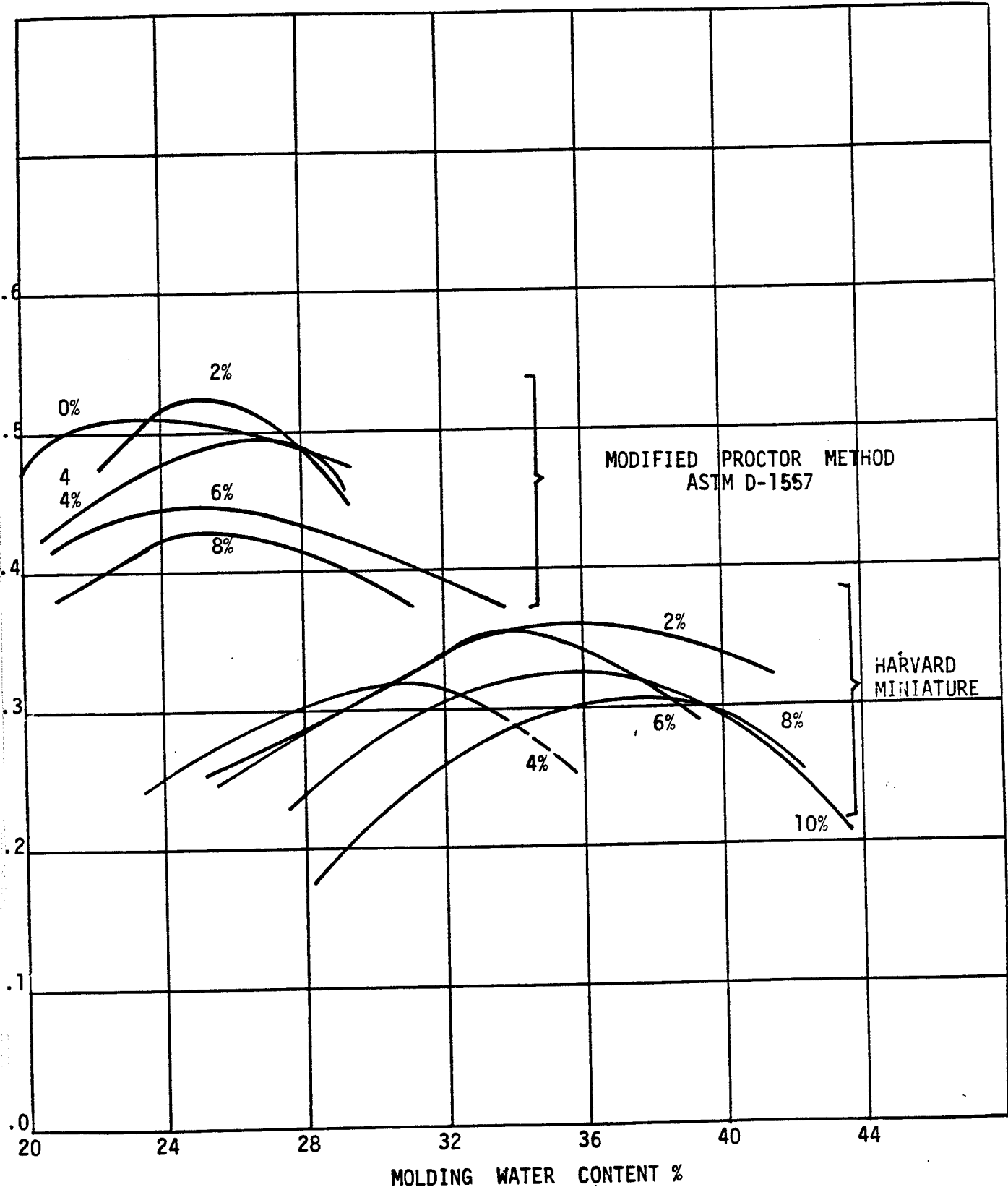


FIG • 50 MOISTURE DENSITY RELATIONSHIP FOR LAKE AGASSIZ CLAYS AT DIFFERENT LIME CONTENTS AND AT DIFFERENT COMPACTIVE EFFORTS.

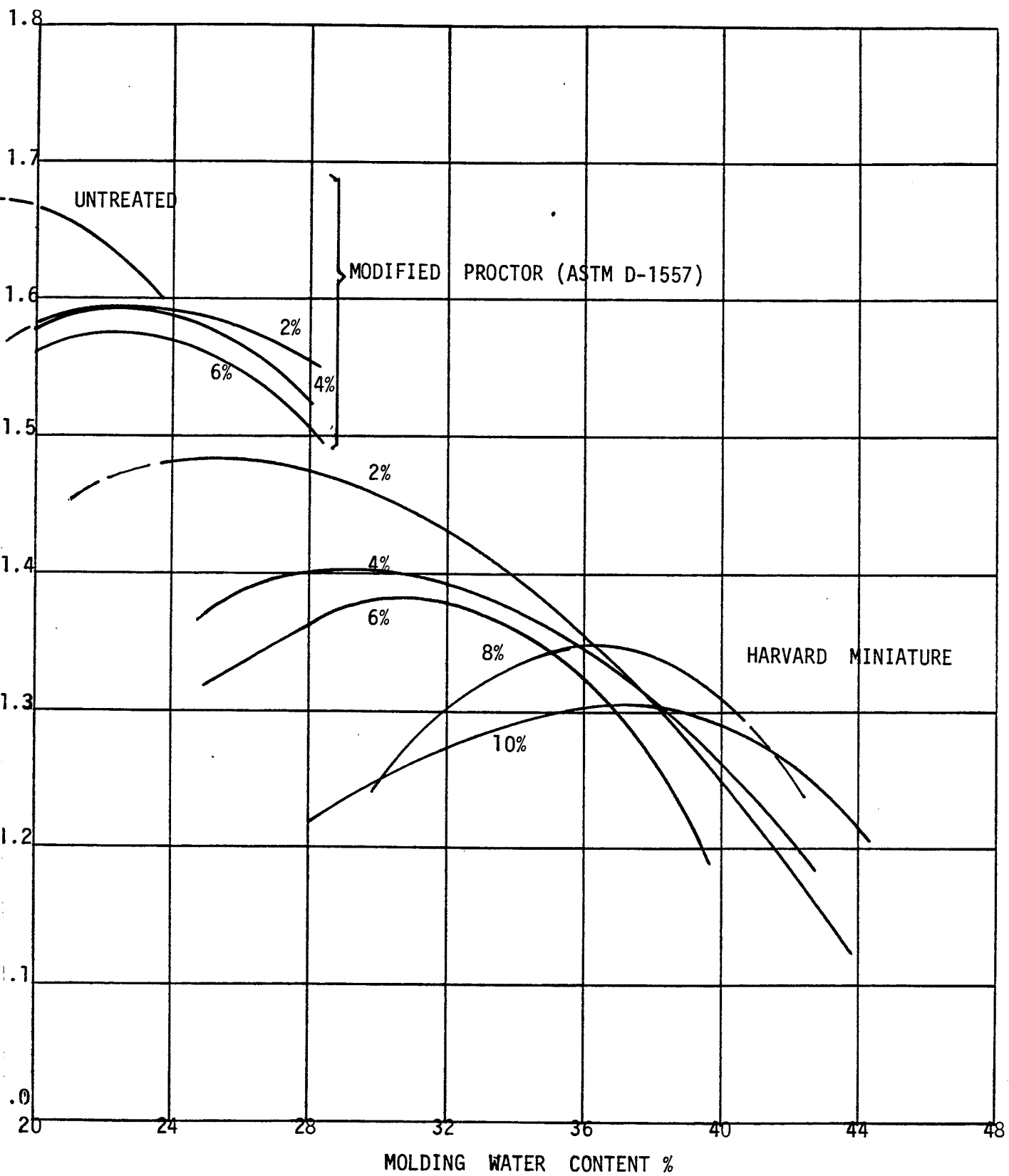


FIG. 51. MOISTURE DENSITY RELATIONSHIP FOR LAKE REGINA CLAYS AT DIFFERENT LIME CONTENTS AND AT DIFFERENT COMPACTIVE EFFORTS.

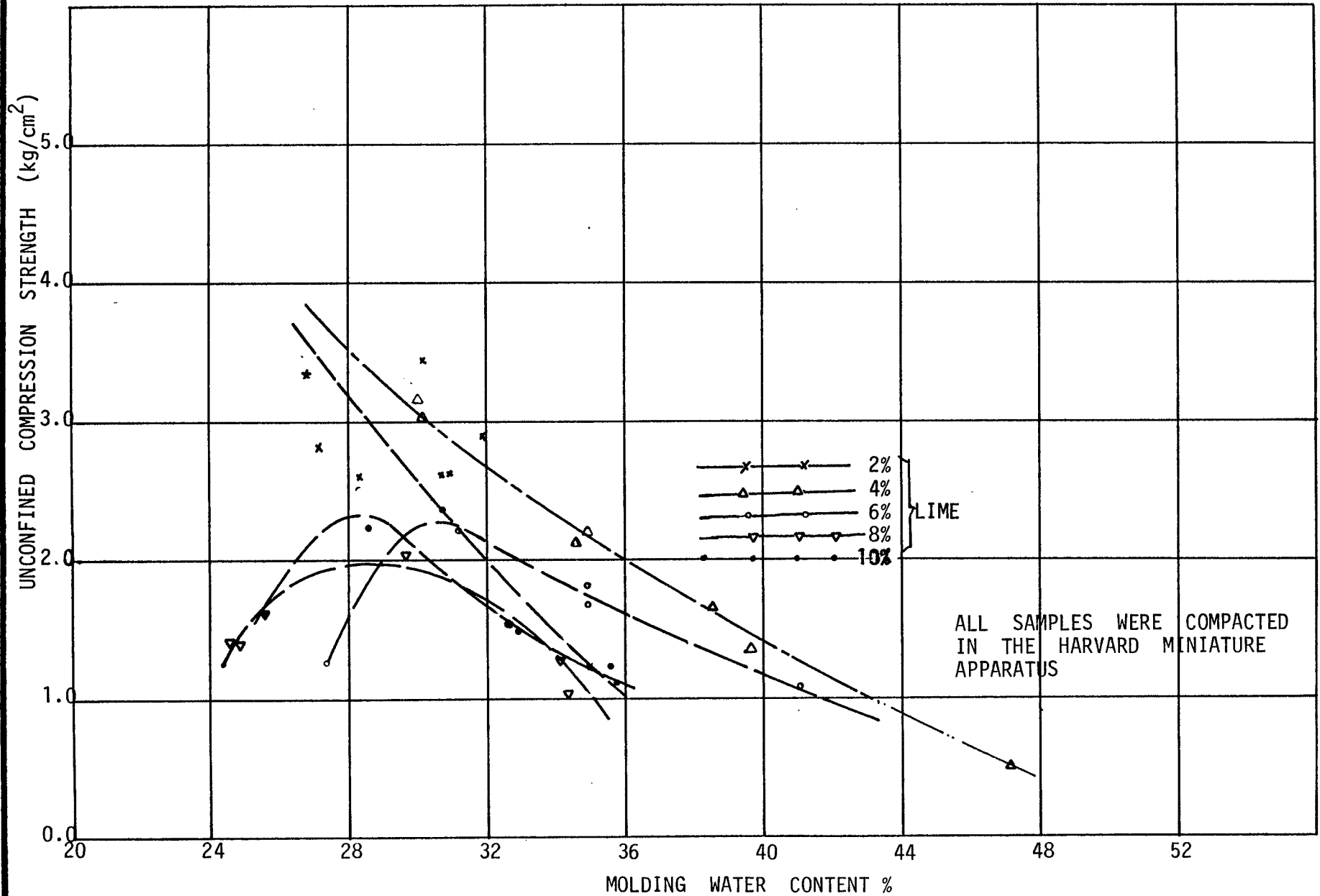


FIG. 52 UNCONFINED COMPRESSION STRENGTH OF LAKE AGASSIZ CLAY (FROM WINNIPEG) AT DIFFERENT MOLDING WATER CONTENT FOR VARIOUS LIME CONTENTS AND AT 3 DAYS LOOSE CURING.

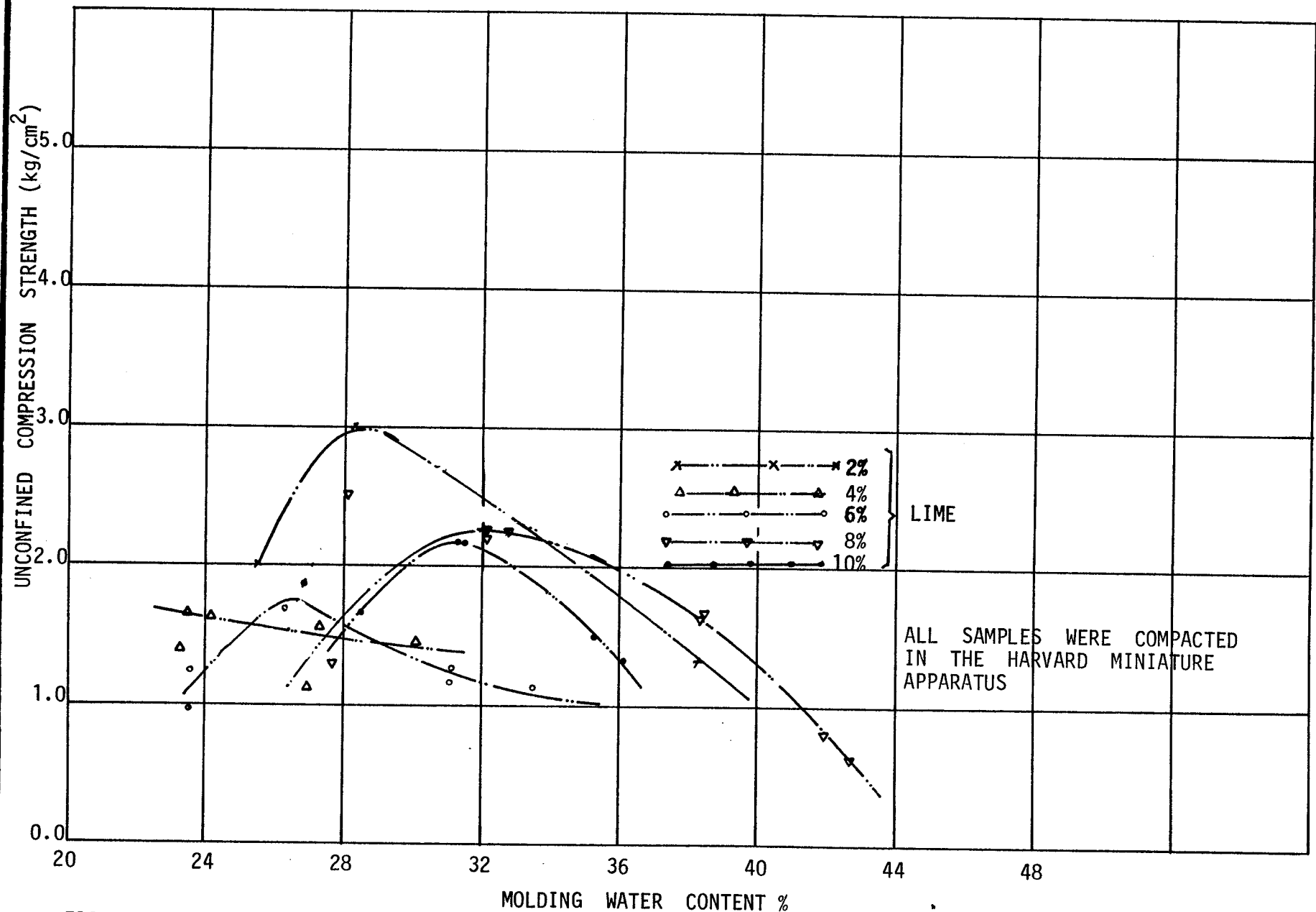


FIG. 53 UNCONFINED COMPRESSION STRENGTH OF LAKE AGASSIZ CLAY (FROM WINNIPEG) AT DIFFERENT MOLDING WATER CONTENTS FOR VARIOUS LIME ADDITIONS AND AT 7 DAYS LOOSE CURING.

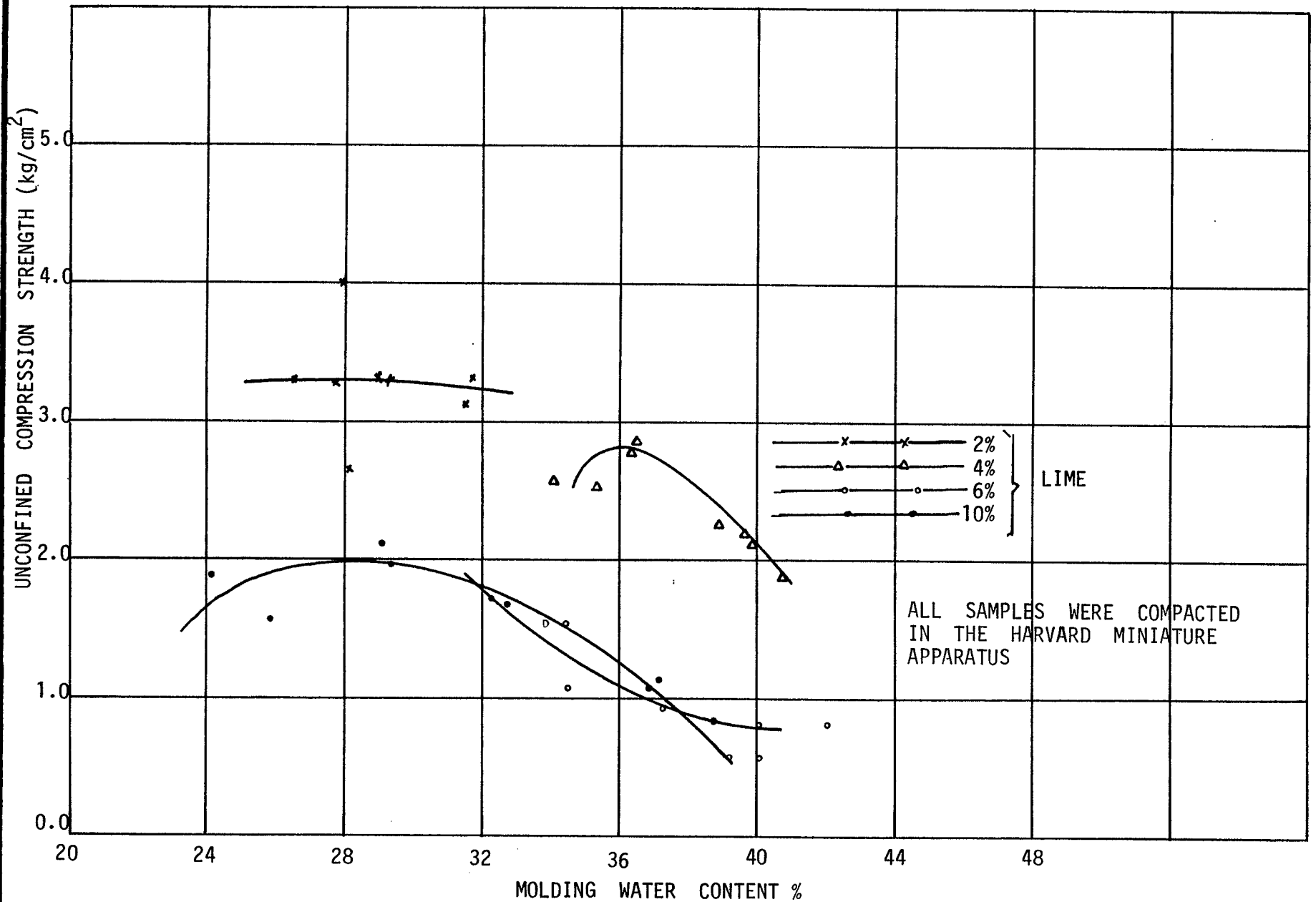


FIG. 54 UNCONFINED COMPRESSION STRENGTH OF LAKE AGASSIZ CLAY (FROM WINNIPEG) AT DIFFERENT MOLDING WATER CONTENT FOR VARIOUS LIME ADDITIONS AT 28 DAYS LOOSE CURING.

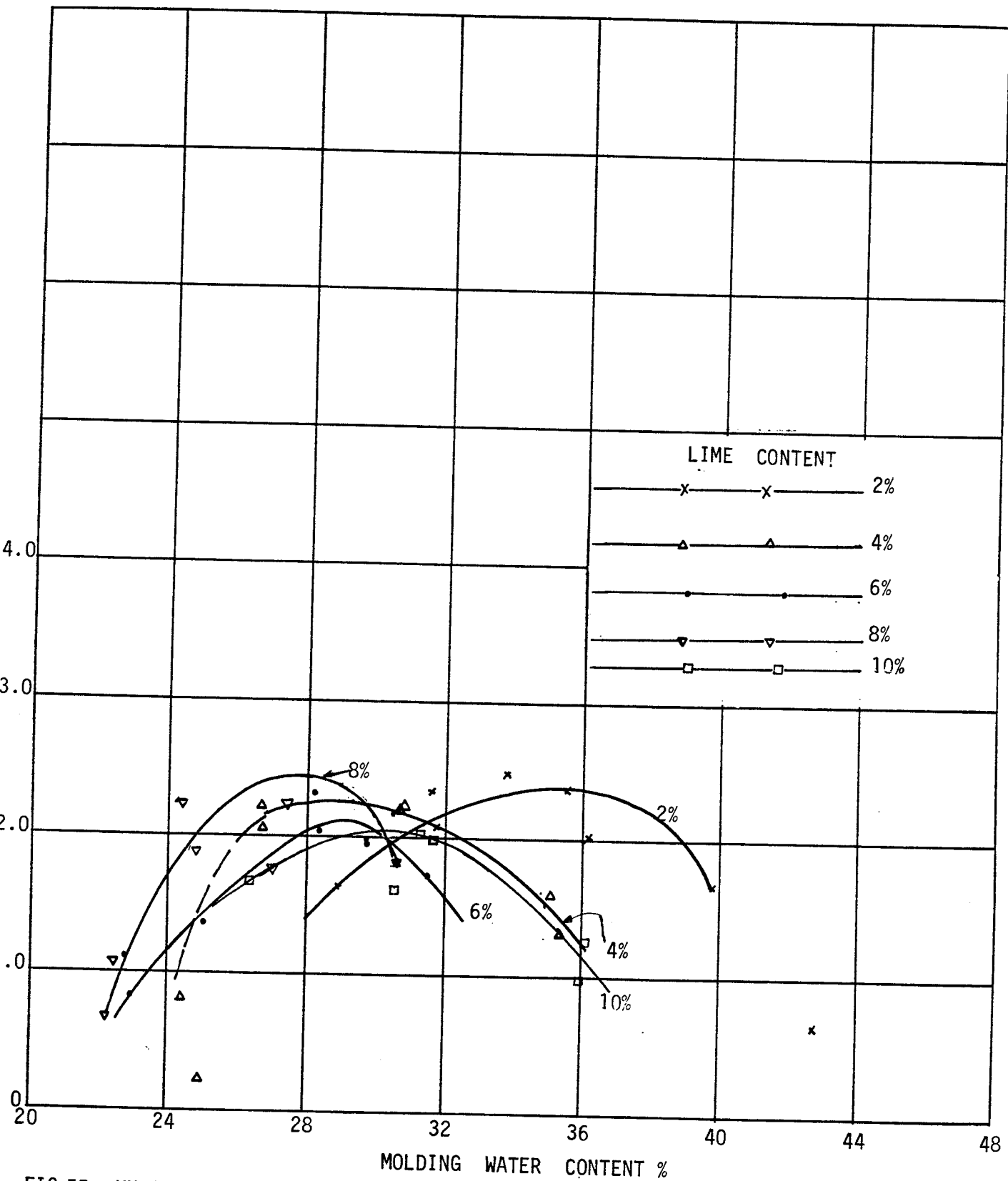


FIG.55 UNCONFINED COMPRESSION STRENGTH OF LAKE REGINA CLAYS AT DIFFERENT LIME CONTENTS AFTER 3 DAYS LOOSE CURING (COMPACTION BY HARVARD MINIATURE)

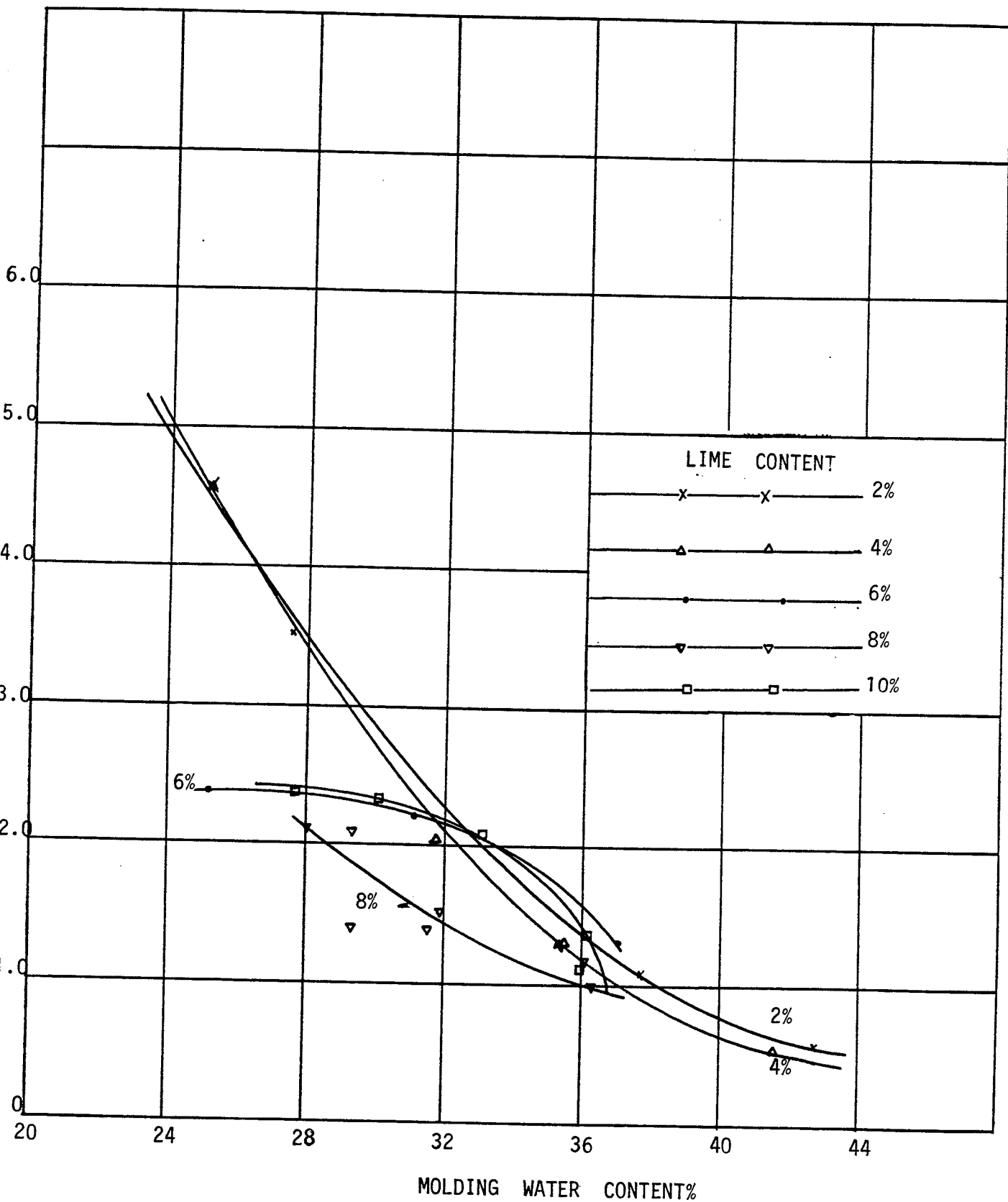


FIG. 56 UNCONFINED COMPRESSION STRENGTHS OF LAKE REGINA CLAYS AT DIFFERENT LIME CONTENTS AFTER 7 DAYS LOOSE CURING (COMPACTION BY HARVARD MINIATURE)

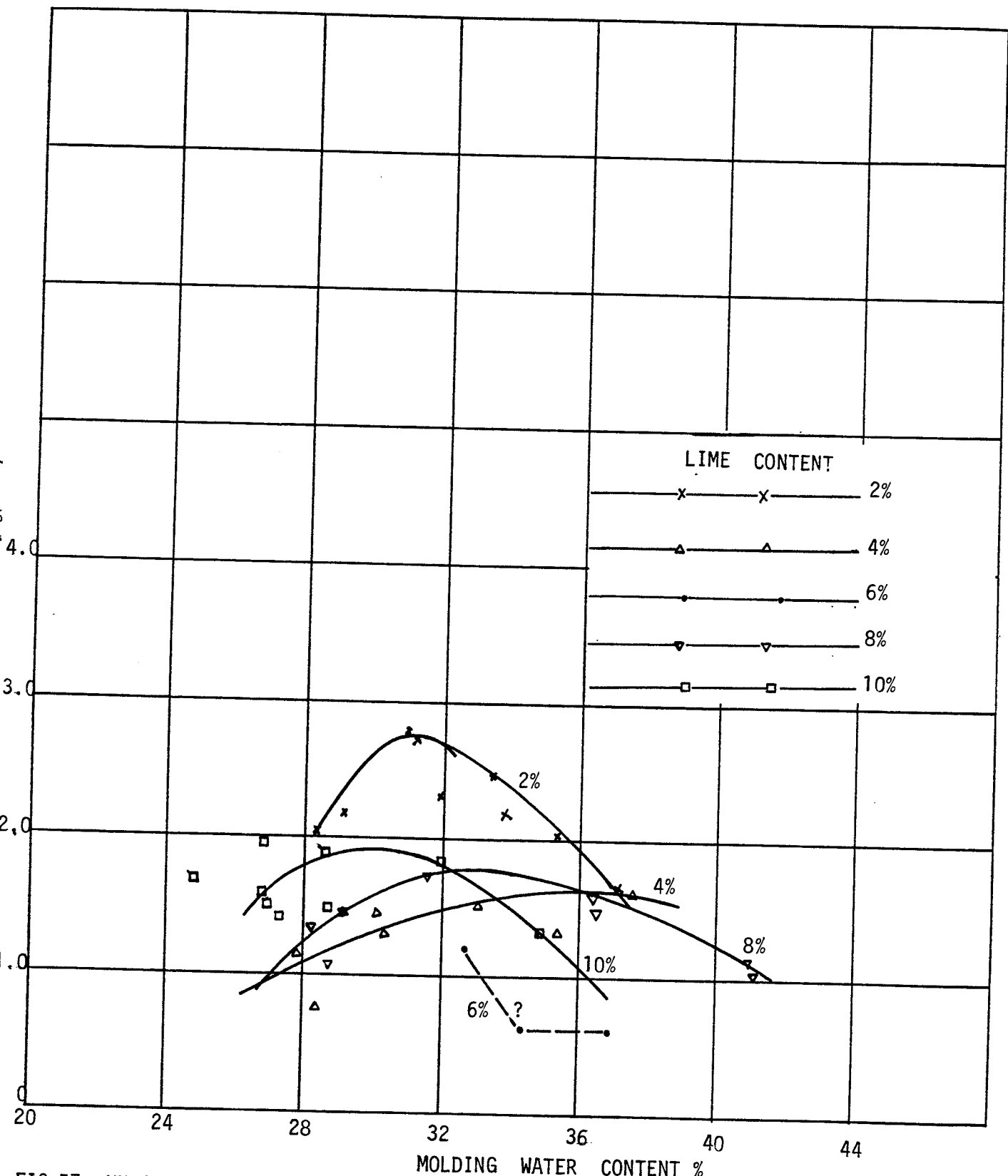
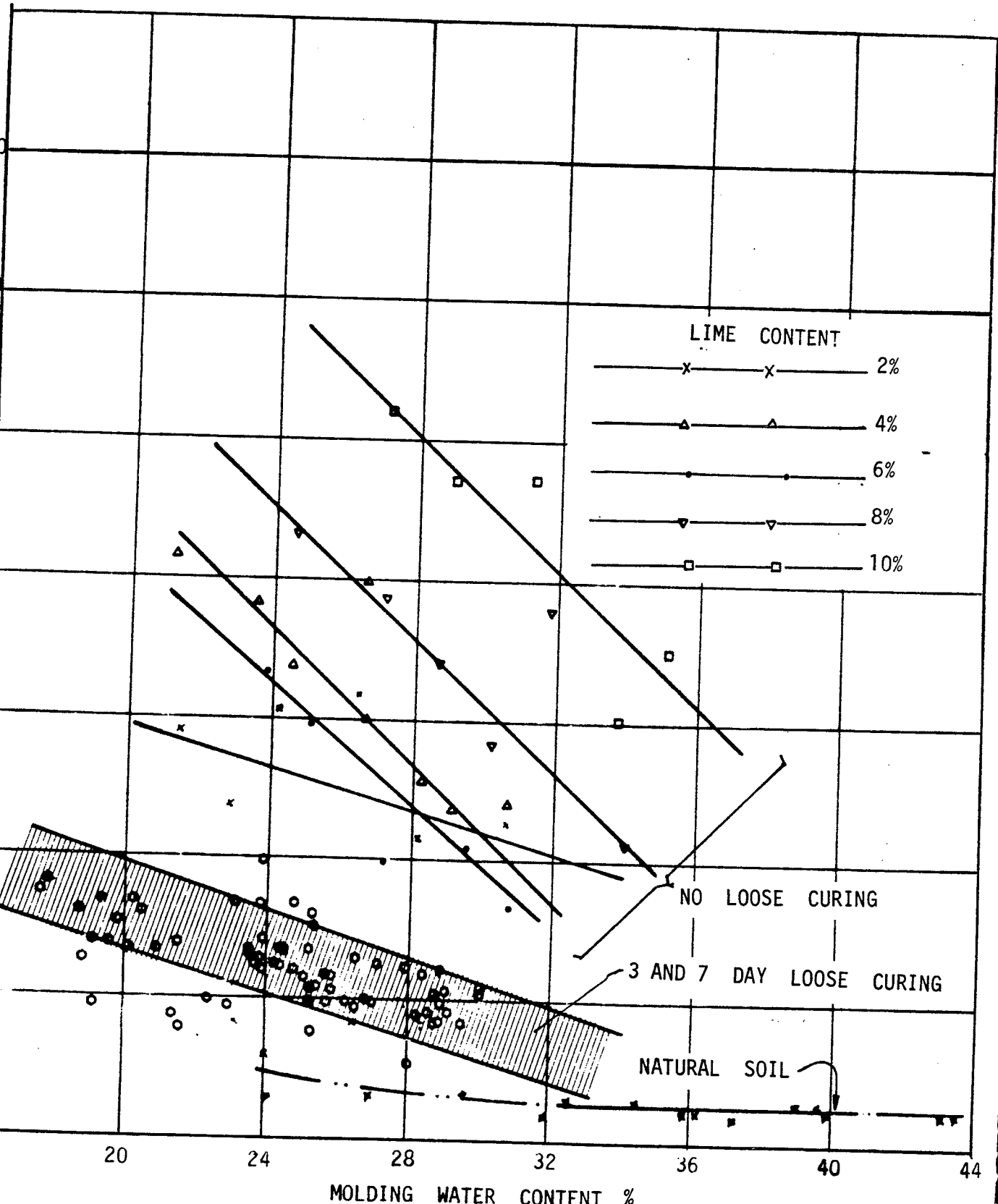


FIG.57 UNCONFINED COMPRESSION STRENGTH OF LAKE REGINA CLAYS AT DIFFERENT LIME CONTENTS AFTER 28 DAYS OF LOOSE CURING (COMPACTION BY HARVARD MINIATURE)



g. 58 UNCONFINED COMPRESSION STRENGTH OF LAKE AGASSIZ CLAYS COMPACTED IN ACCORDANCE WITH ASTM D 1557-70 (MODIFIED PROCTOR) BUT WITH NO LOOSE CURING AND WITH 3 AND 7 DAYS LOOSE CURING. (ALSO SHOWN IS THE STRENGTH OF UNTREATED SOIL)

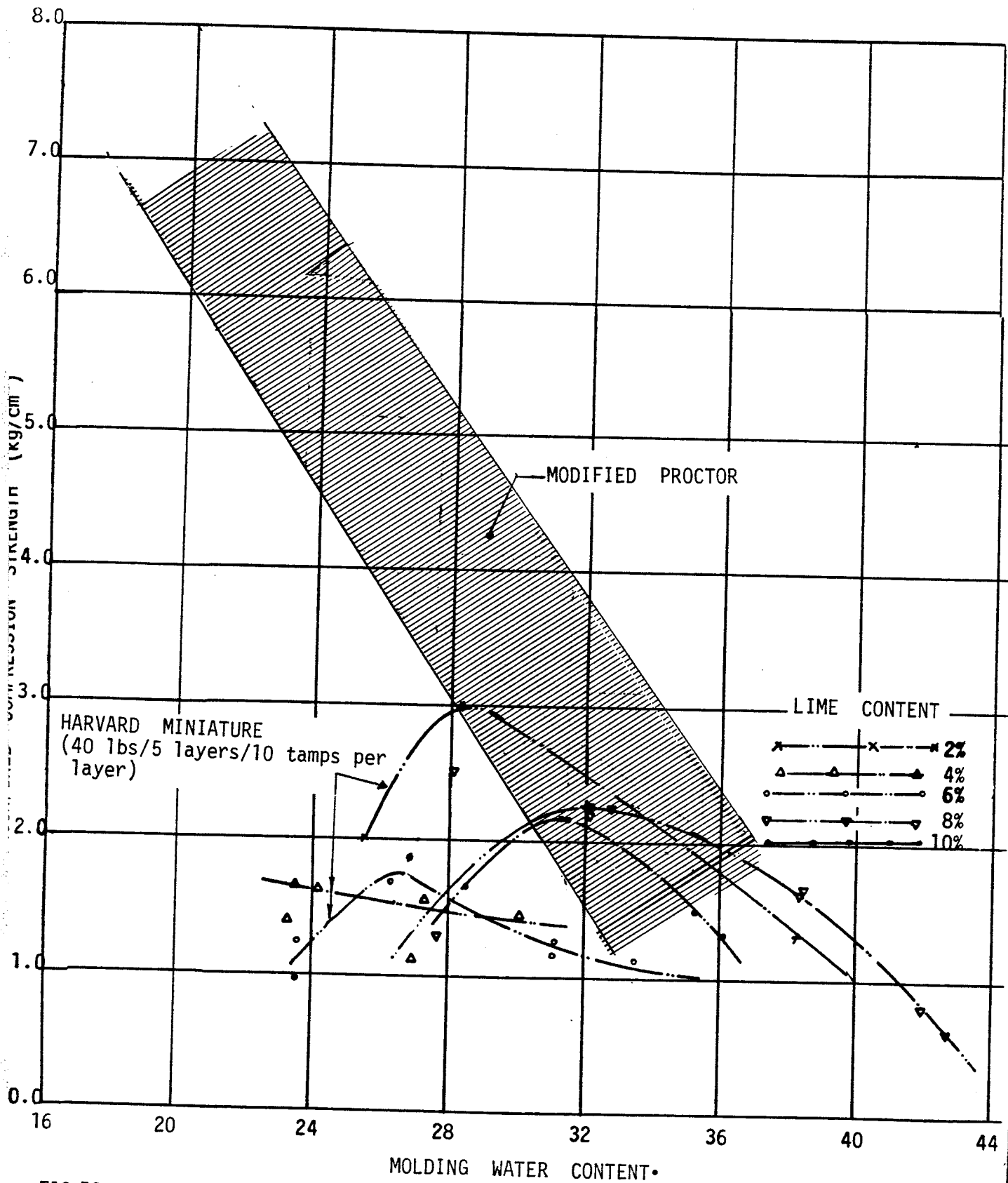


FIG.59 UNCONFINED COMPRESSION STRENGTH OF LAKE AGASSIZ CLAY WITH DIFFERENT LIME CONTENTS WITH 7 DAYS LOOSE CURING BUT COMPACTED AT TWO DIFFERENT COMPACTIVE EFFORTS (MODIFIED PROCTOR AND HARVARD MINIATURE)

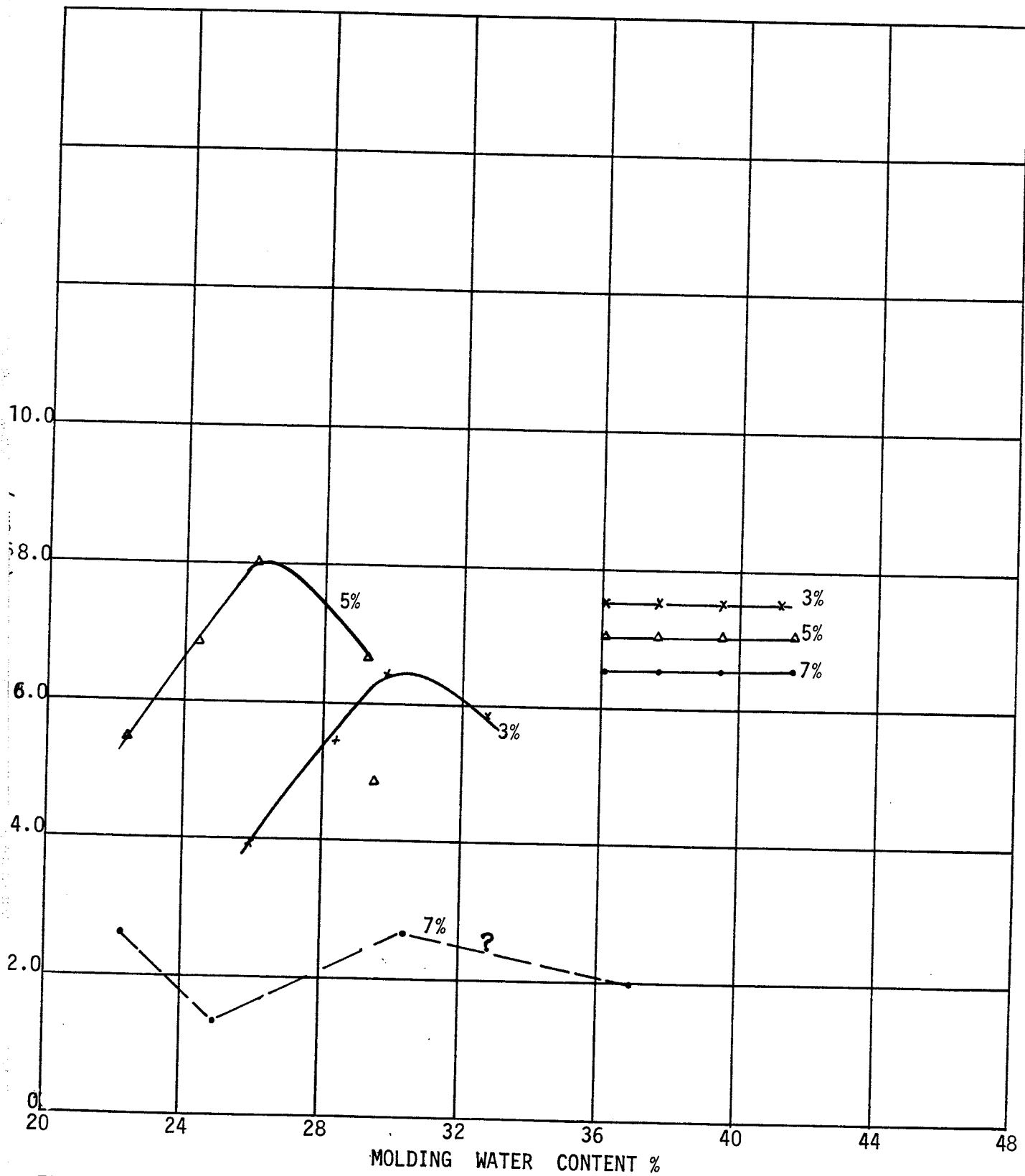


FIG. 60 UNCONFINED COMPRESSION STRENGTH OF THE CLAYS FROM SOUTHWESTERN SASKATCHEWAN (LEADER) AT 3,5 AND 7% LIME CONTENTS (COMPACTION IN ACCORDANCE WITH ASTM D 1557-70)

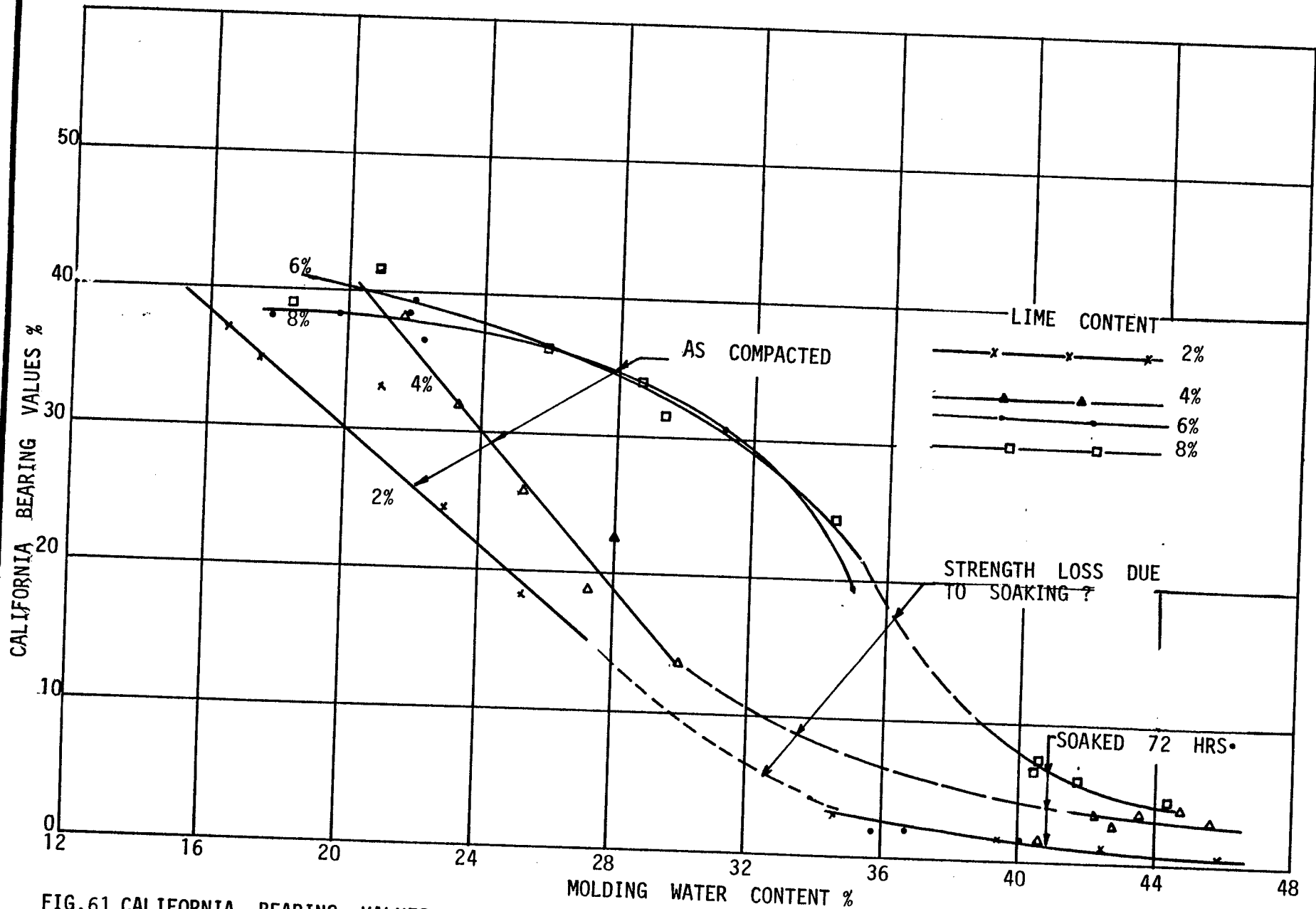


FIG.61 CALIFORNIA BEARING VALUES OF LAKE REGINA CLAYS AT DIFFERENT LIME CONTENTS. (COMPACTION AFTER ASTM D-1557-70 - 6inch mold; SOAKING PERIOD 72 HRS.)

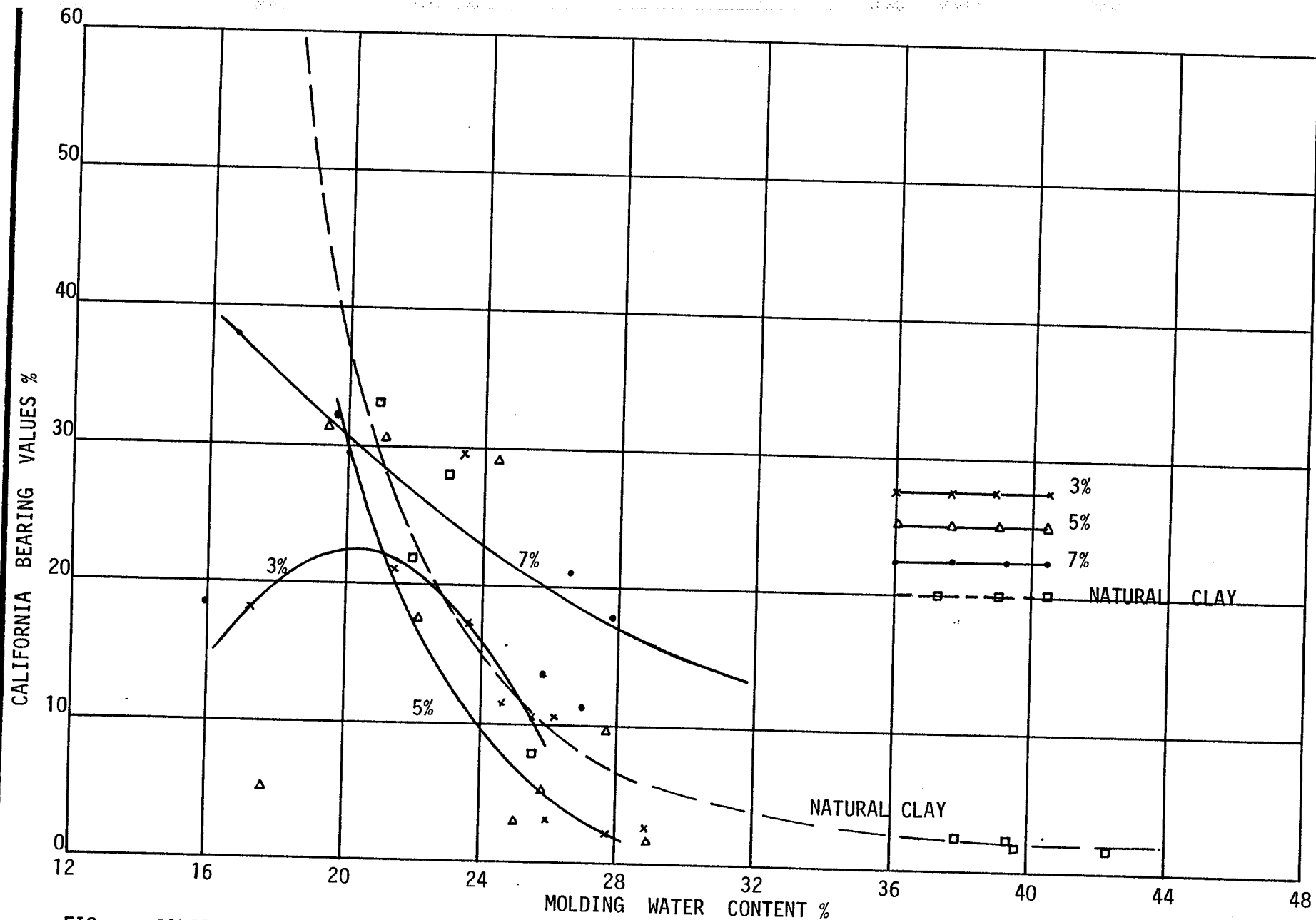


FIG. 62 CALIFORNIA BEARING VALUES FOR THE CLAY FROM SOUTHWESTERN SASKATCHEWAN (LEADER) AT VARIOUS LIME CONTENTS (COMPACTION IN ACCORDANCE WITH ASTM D1557-70: 6 INCH MOLD)

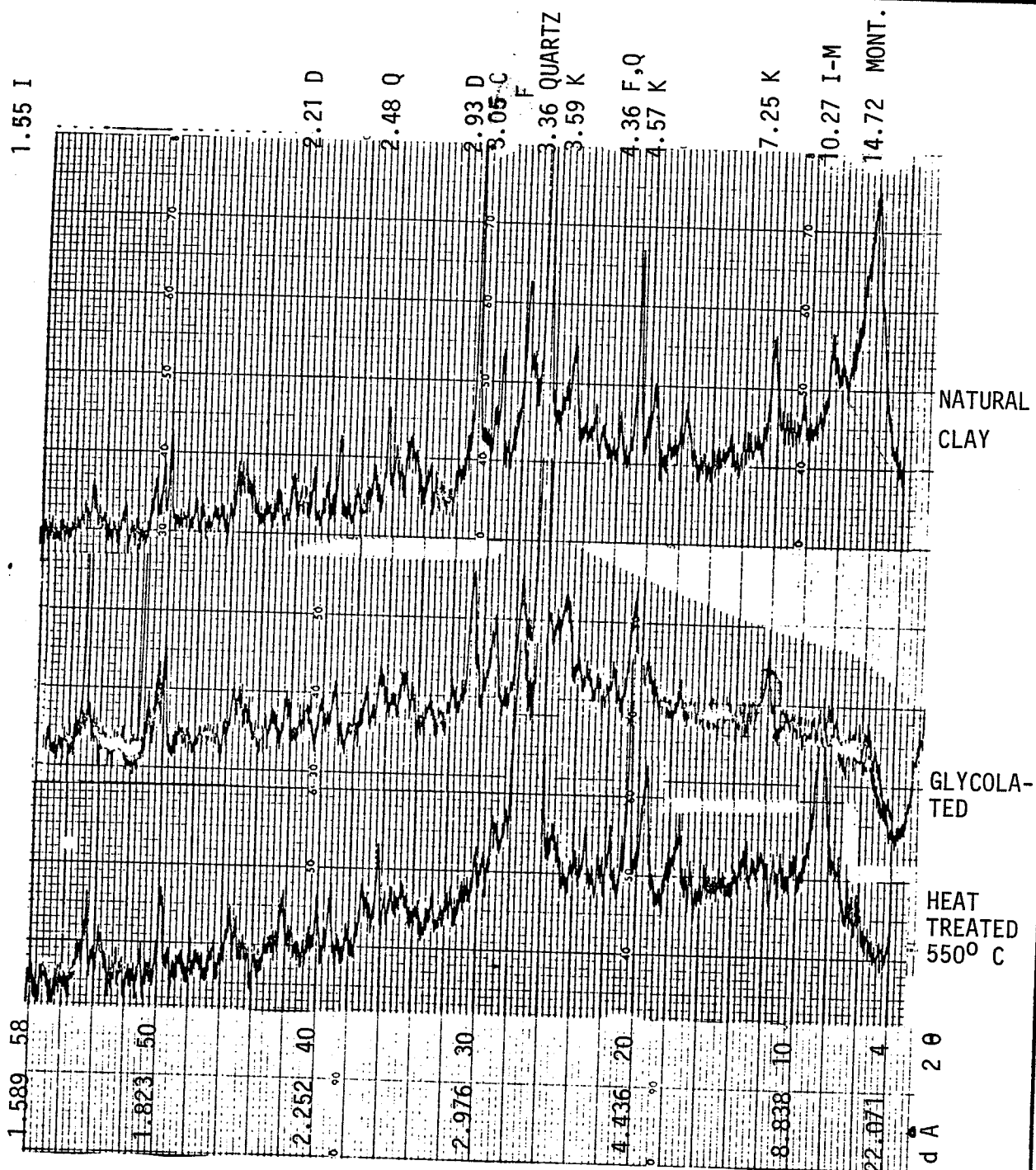


FIG. 63. X-RAY DIFFRACTION PATTERNS FOR UNTREATED WINNIPEG CLAY.

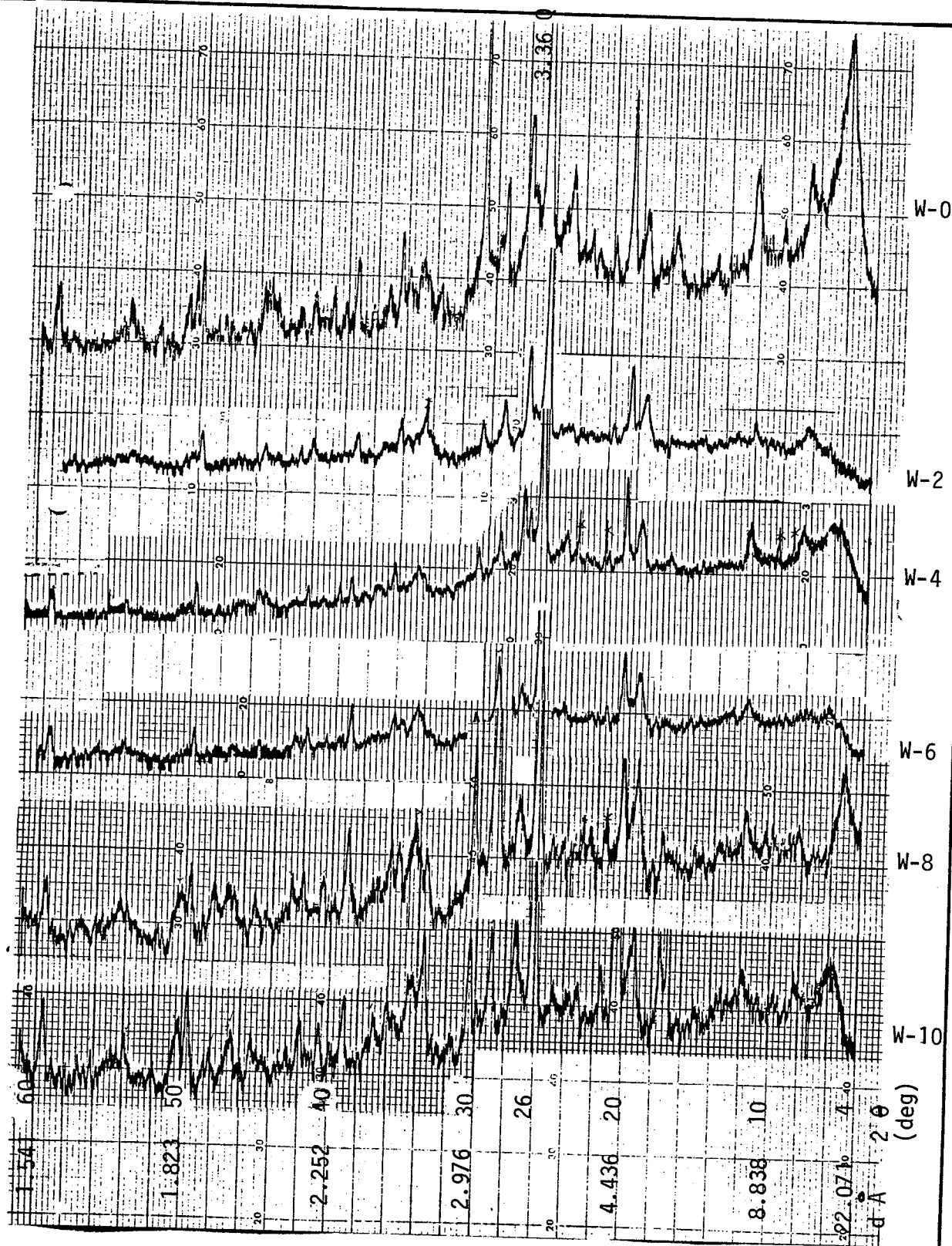


FIG. 64. X-RAY DIFFRACTION PATTERN OF LAKE AGASSIZ CLAY TREATED WITH LIME AND LOOSE CURED FOR 3 DAYS.

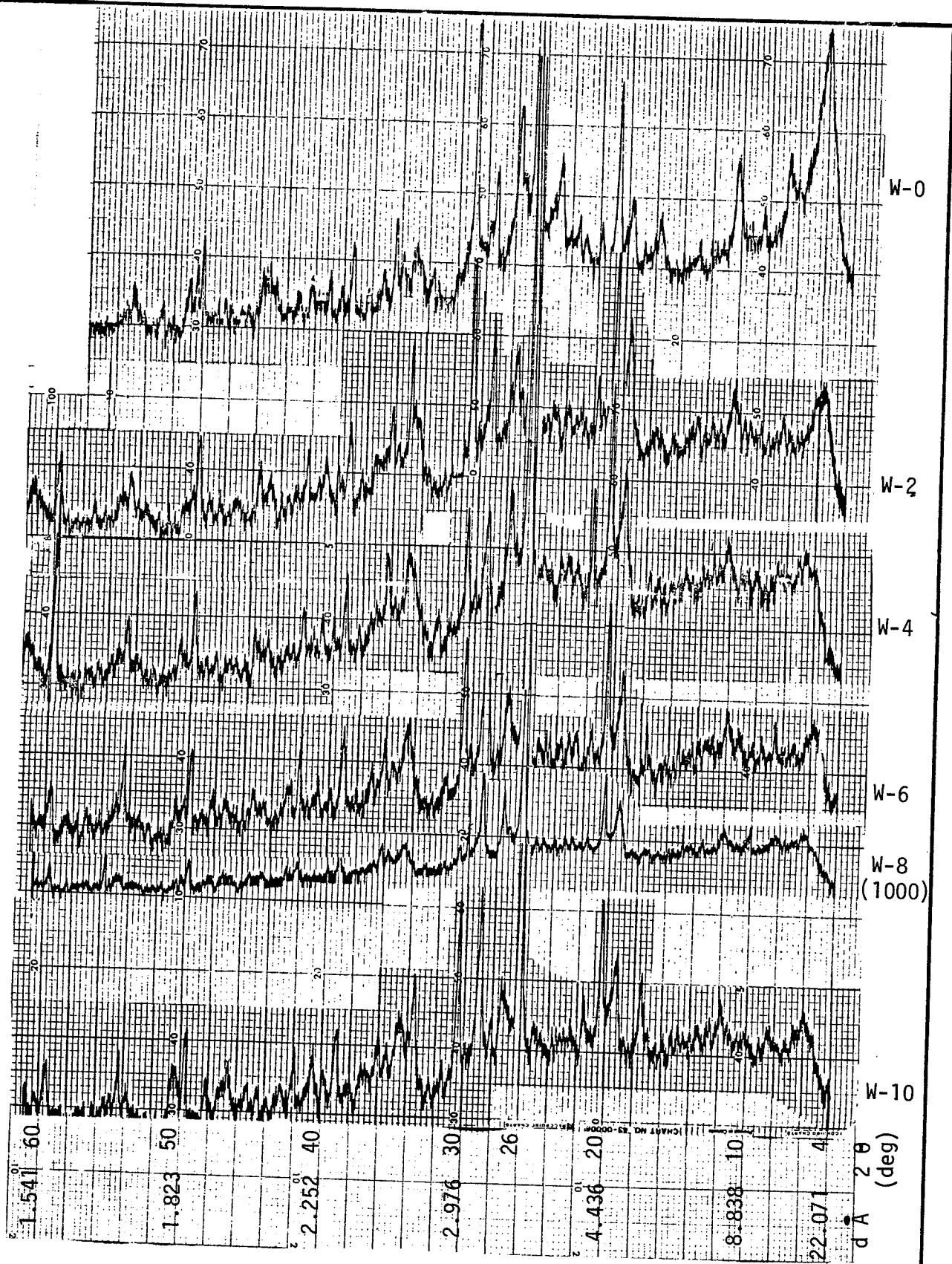


FIG. 65. X-RAY DIFFRACTION PATTERNS OF LAKE AGASSIZ CLAY TREATED WITH LIME AND LOOSE CURED FOR 7 DAYS.

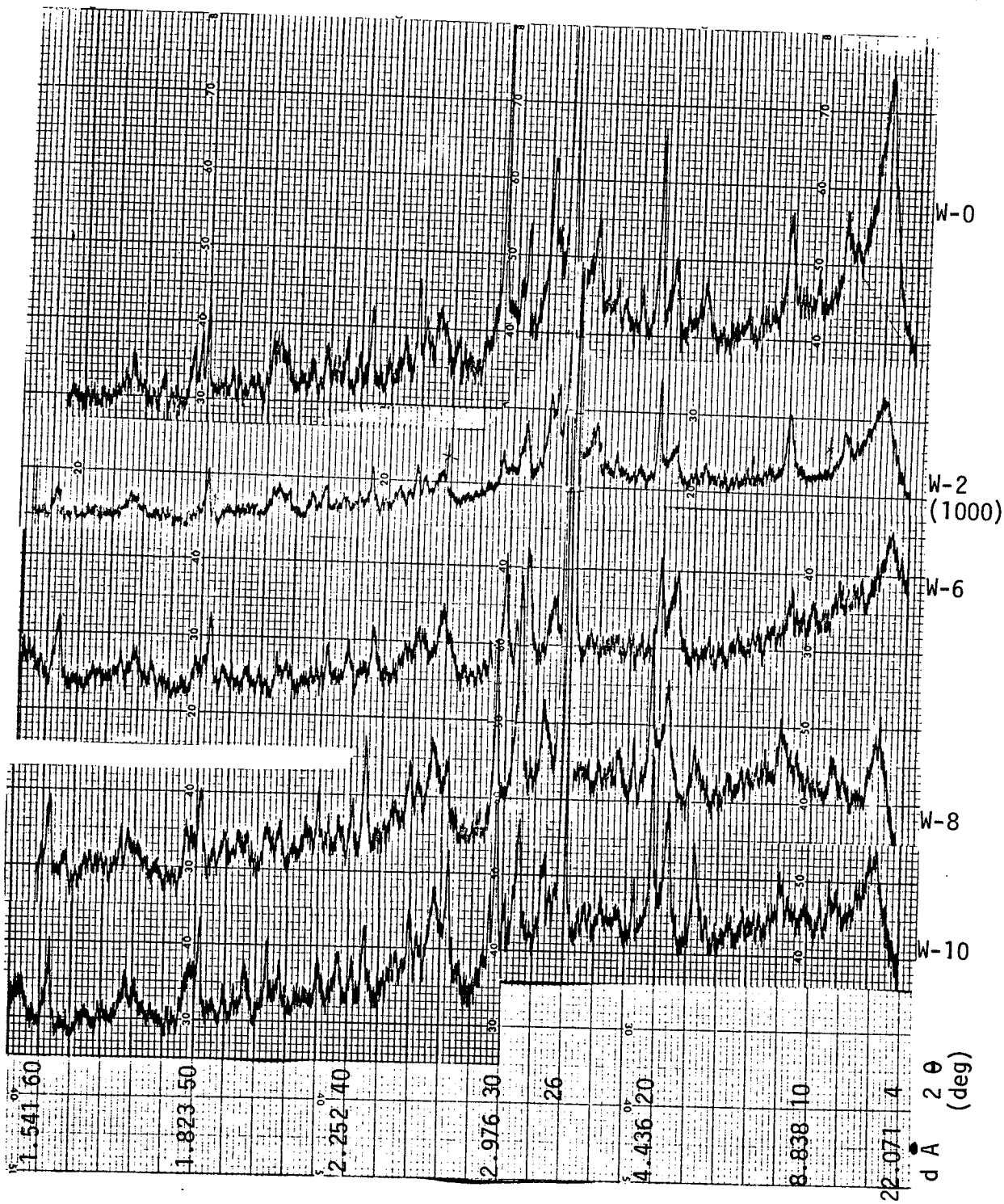


FIG. 66. X-RAY DIFFRACTION PATTERNS OF LAKE AGASSIZ CLAY
TREATED WITH LIME AND LOOSE CURED FOR 28 DAYS.

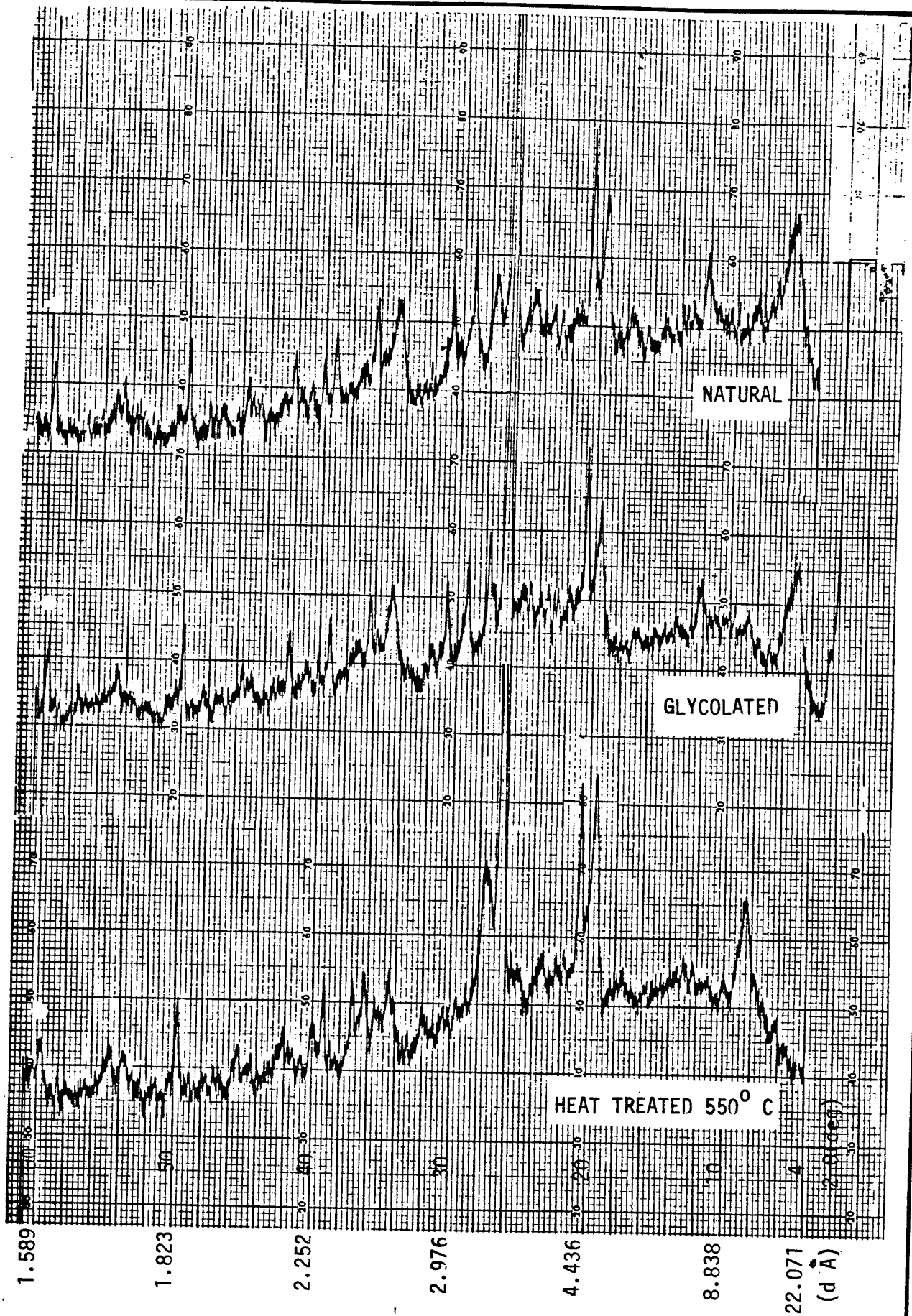


FIG. 67 X-RAY DIFFRACTION PATTERNS FOR UNTREATED LAKE REGINA CLAY.

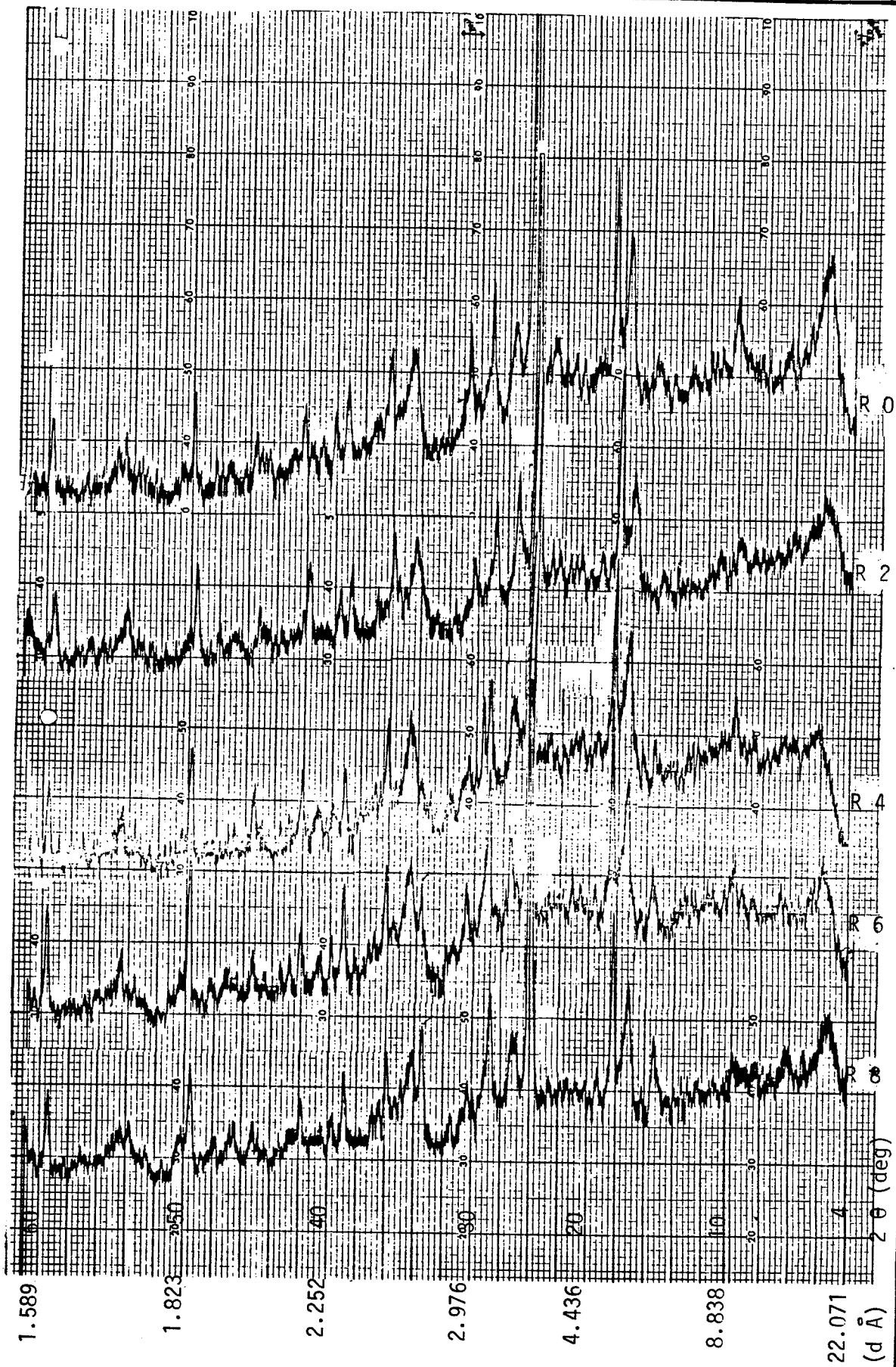


FIG. 68. X-RAY DIFFRACTION PATTERNS FOR LAKE REGINA CLAY
TREATED WITH LIME AND LOOSE CURED FOR 3 DAYS

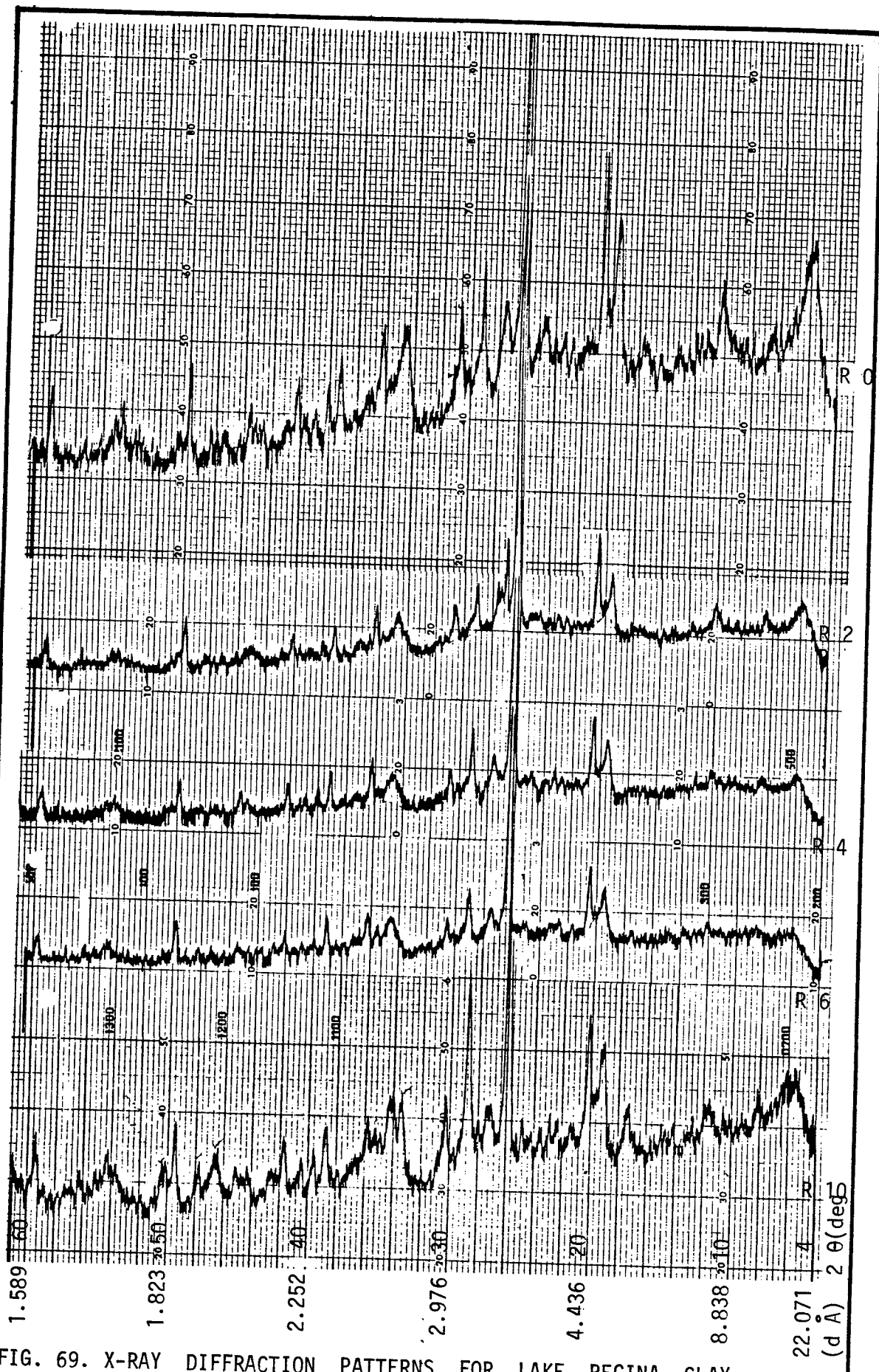


FIG. 69. X-RAY DIFFRACTION PATTERNS FOR LAKE REGINA CLAY TREATED WITH LIME AND LOOSE CURED FOR 7 DAYS.

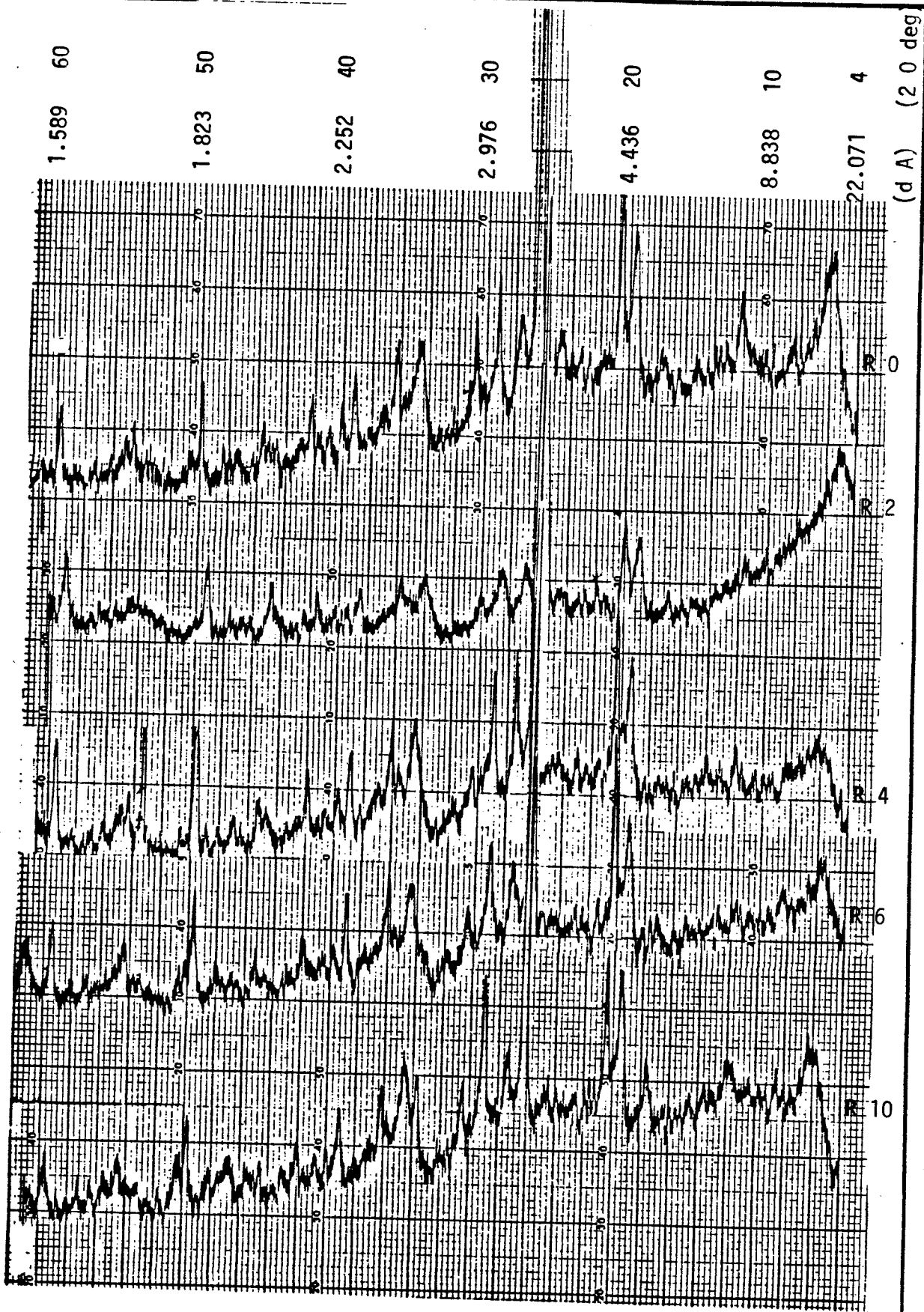


FIG. 70. X-RAY DIFFRACTION PATTERNS FOR LAKE REGINA CLAY TREATED WITH LIME AND LOOSE CURED FOR 28 DAYS.

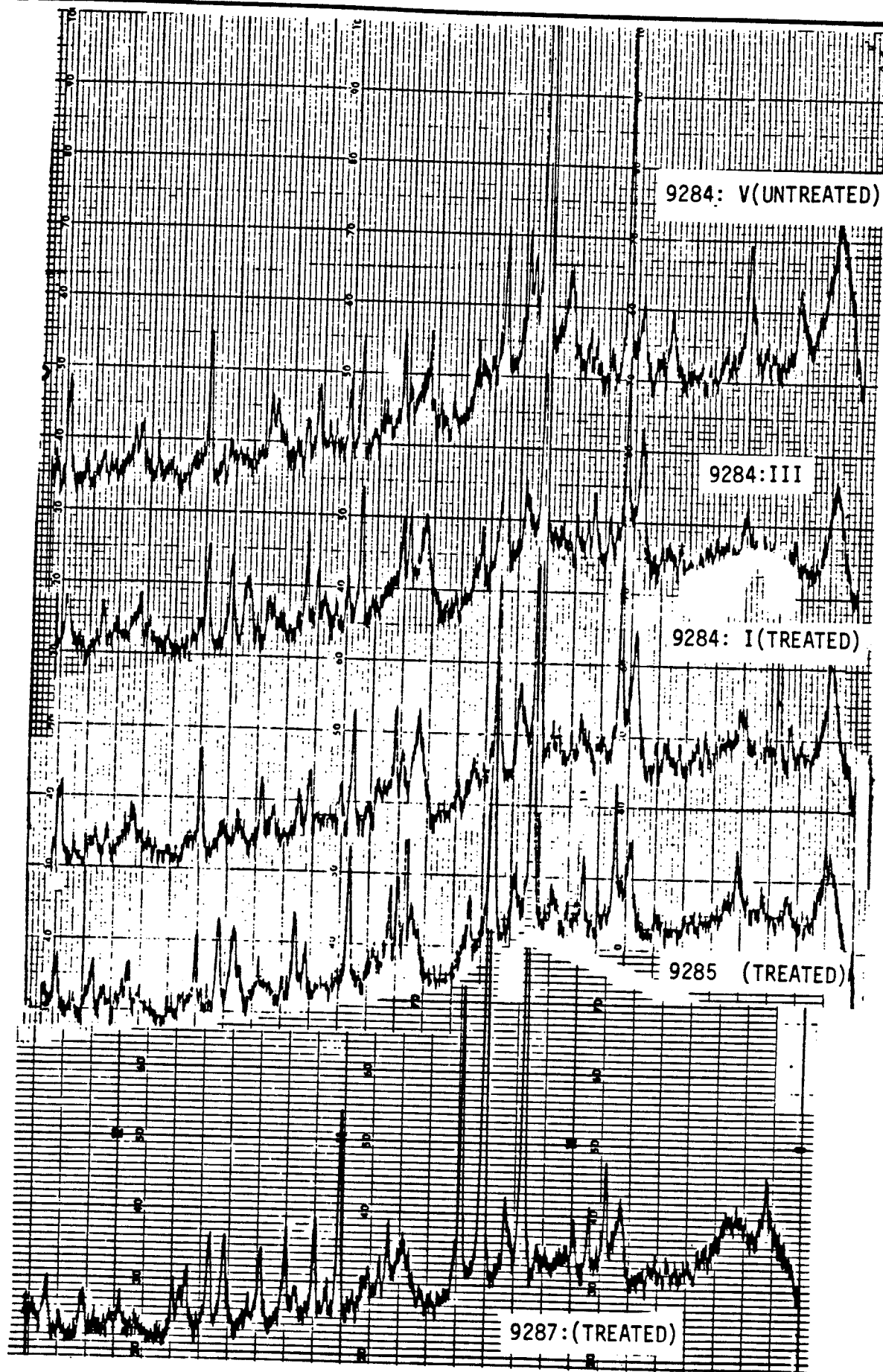
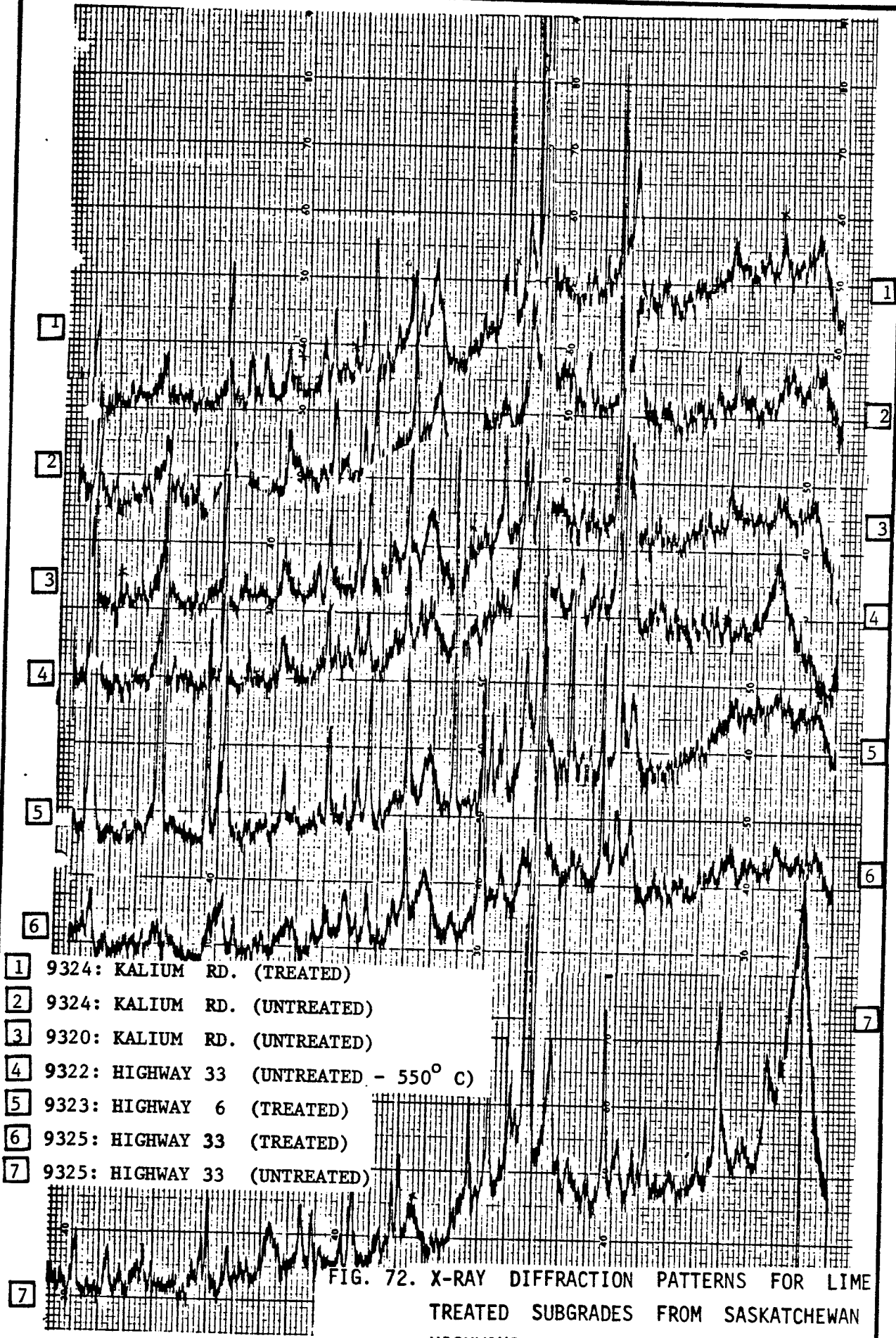


FIG. 71. X- RAY DIFFRACTION PATTERNS FOR LIME TREATED SUBGRADES RECOVERED FROM MANITOBA HIGHWAYS TEST SECTIONS.



- 1 9324: KALIUM RD. (TREATED)
- 2 9324: KALIUM RD. (UNTREATED)
- 3 9320: KALIUM RD. (UNTREATED)
- 4 9322: HIGHWAY 33 (UNTREATED - 550° C)
- 5 9323: HIGHWAY 6 (TREATED)
- 6 9325: HIGHWAY 33 (TREATED)
- 7 9325: HIGHWAY 33 (UNTREATED)

FIG. 72. X-RAY DIFFRACTION PATTERNS FOR LIME TREATED SUBGRADES FROM SASKATCHEWAN HIGHWAYS.

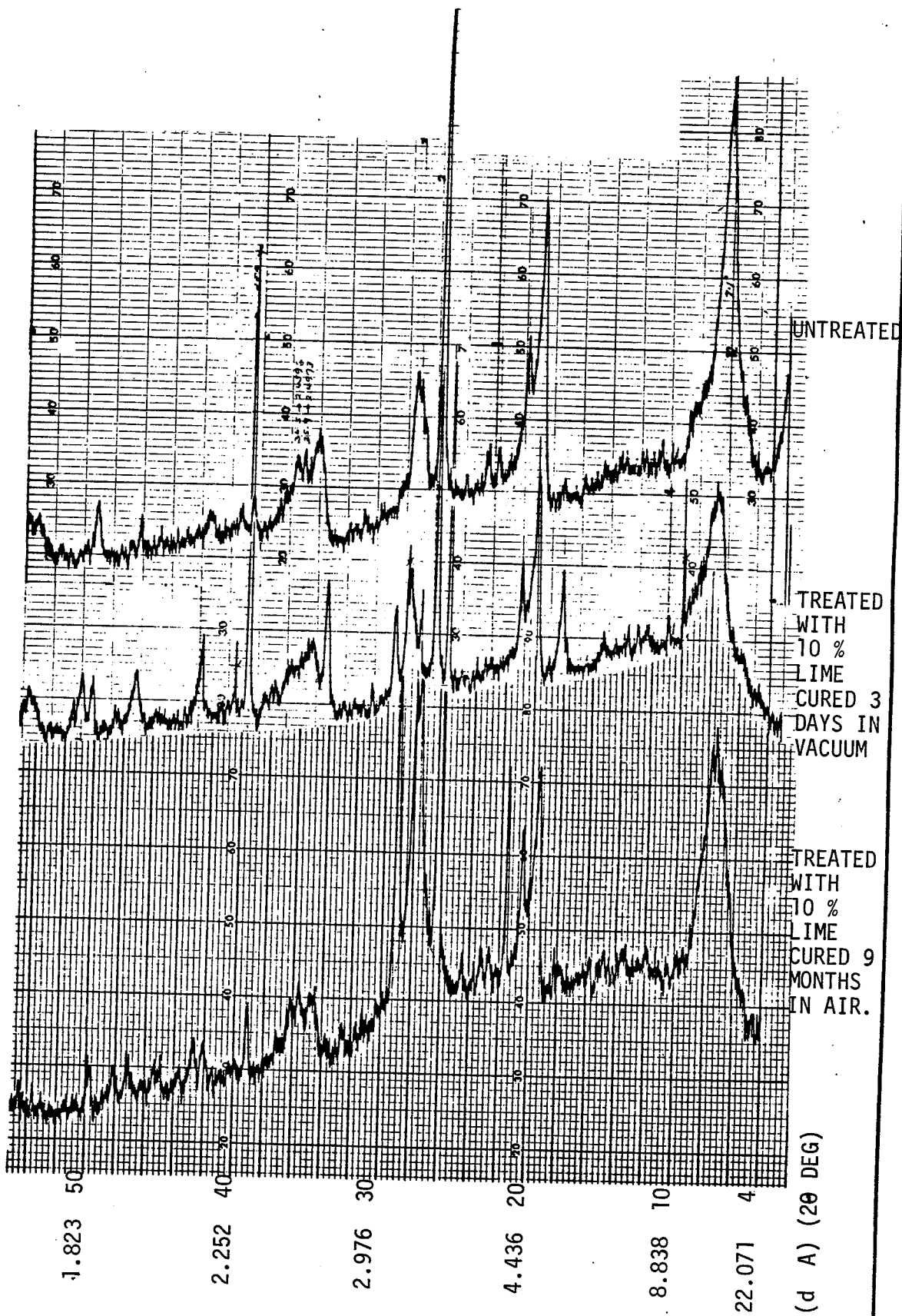


FIG. 73. X-RAY DIFFRACTION PATTERNS FOR THE SAMPLE OF "PELTONITE"

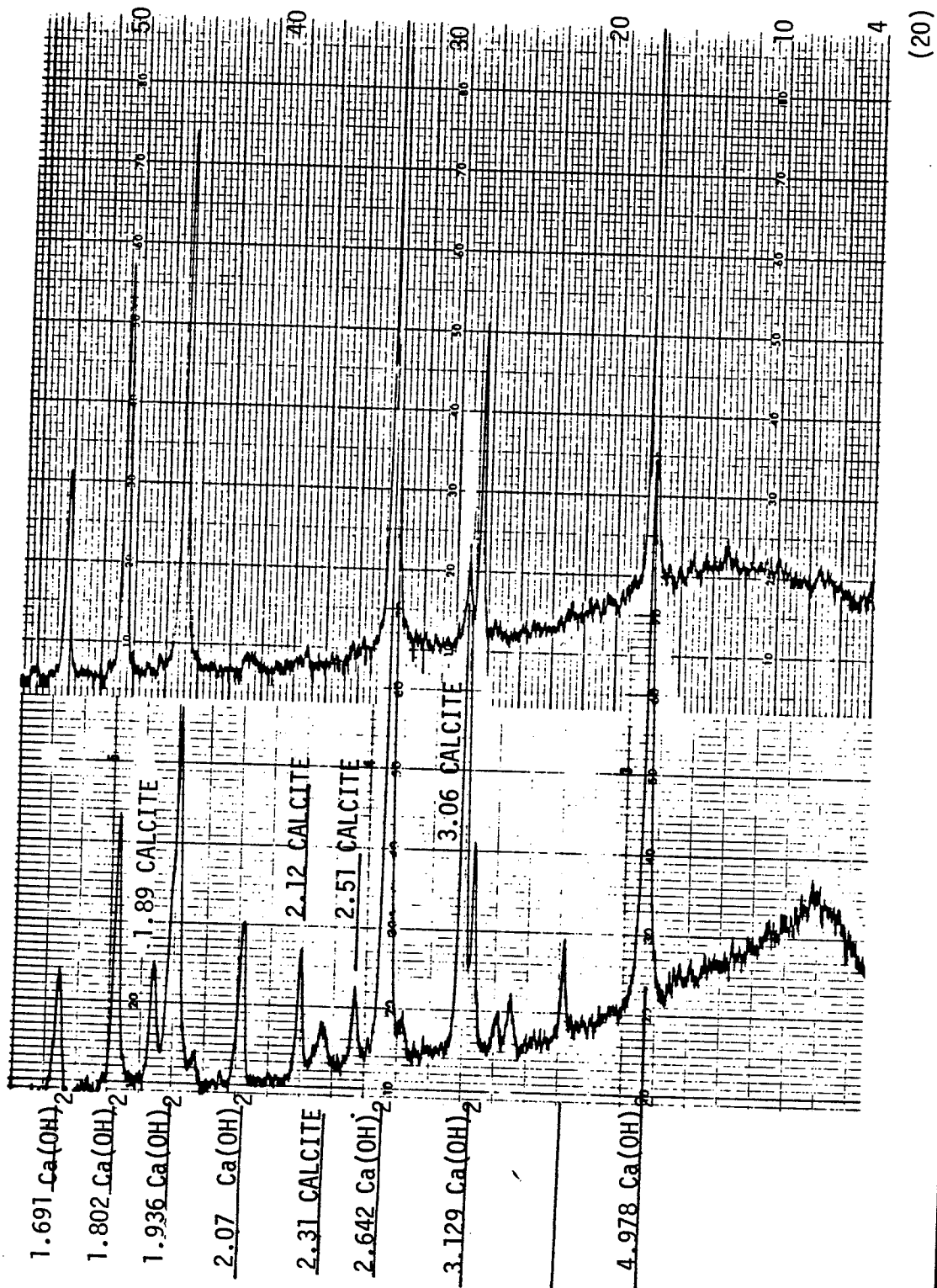


FIG. 74. X-RAY DIFFRACTION PATTERN FOR THE QUICKLIME USED IN THIS SERIES AND FOR A HYDRATED QUICKLIME.

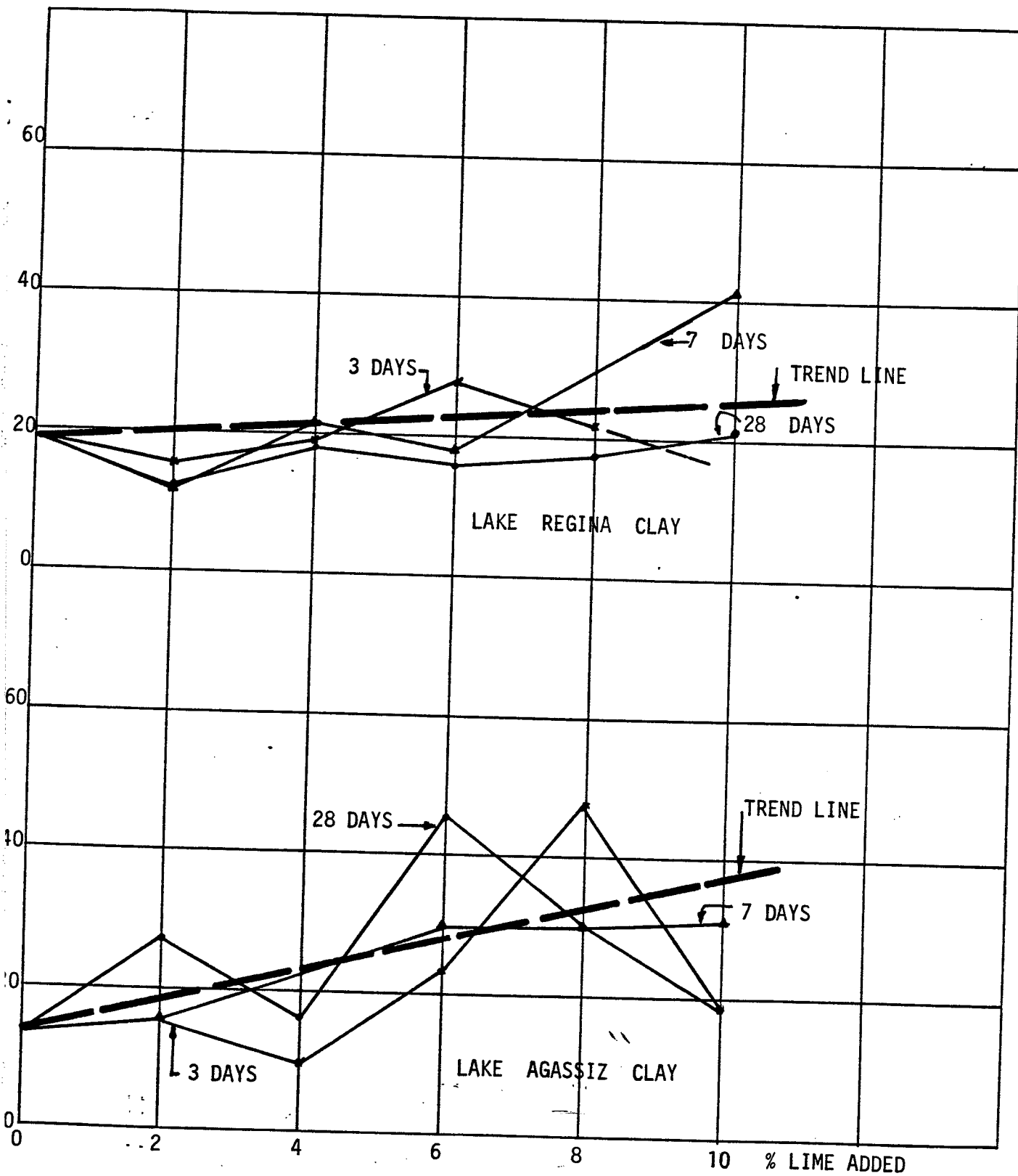


FIG. 75. RELATIVE INTENSITIES (I/I_0) OF THE 3.05 Å CALCITE PEAKS ON ADDING LIME TO LAKE AGASSIZ AND LAKE REGINA CLAYS. (3.34 Å QUARTZ PEAK WAS REFERENCE)

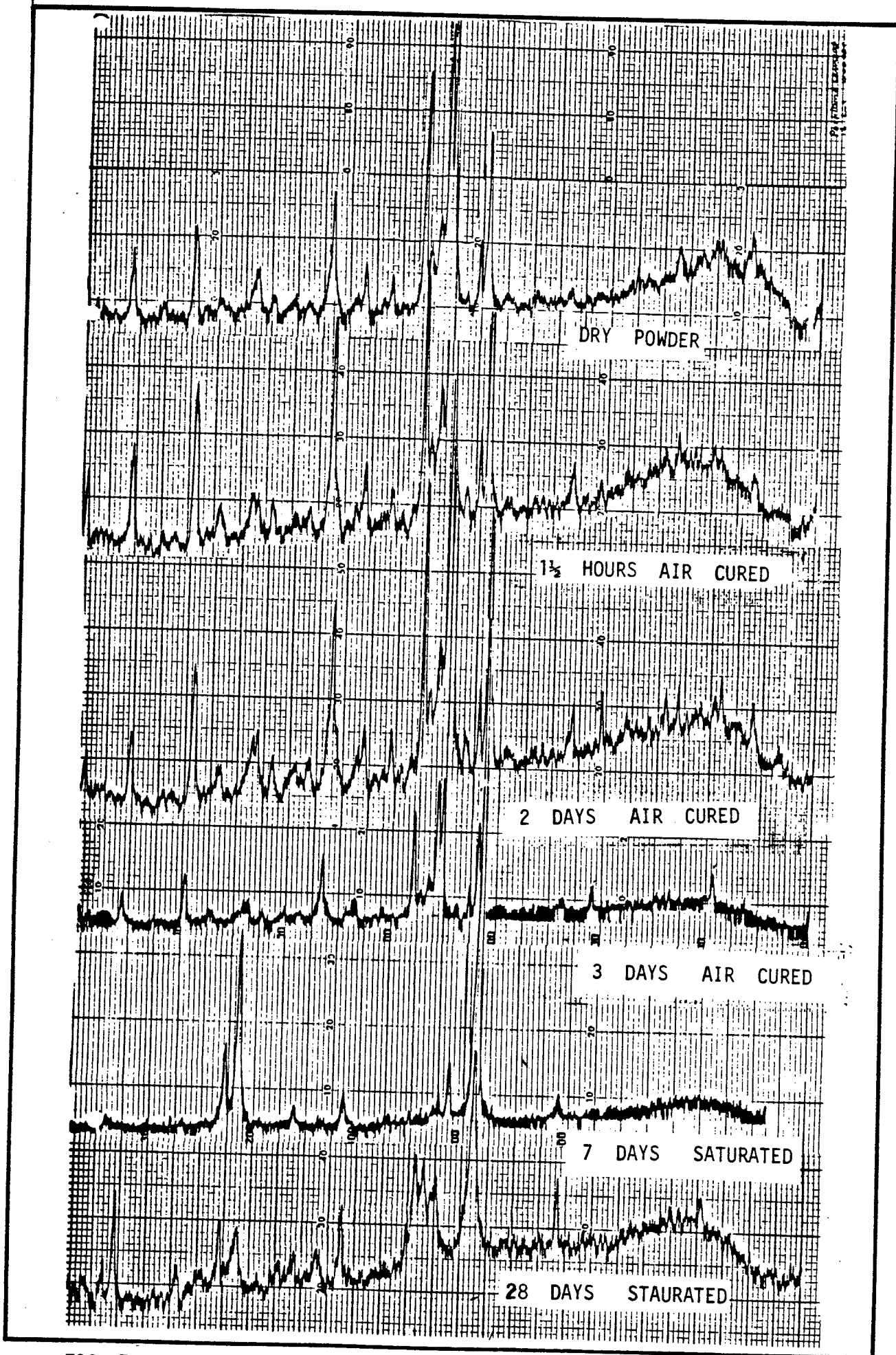
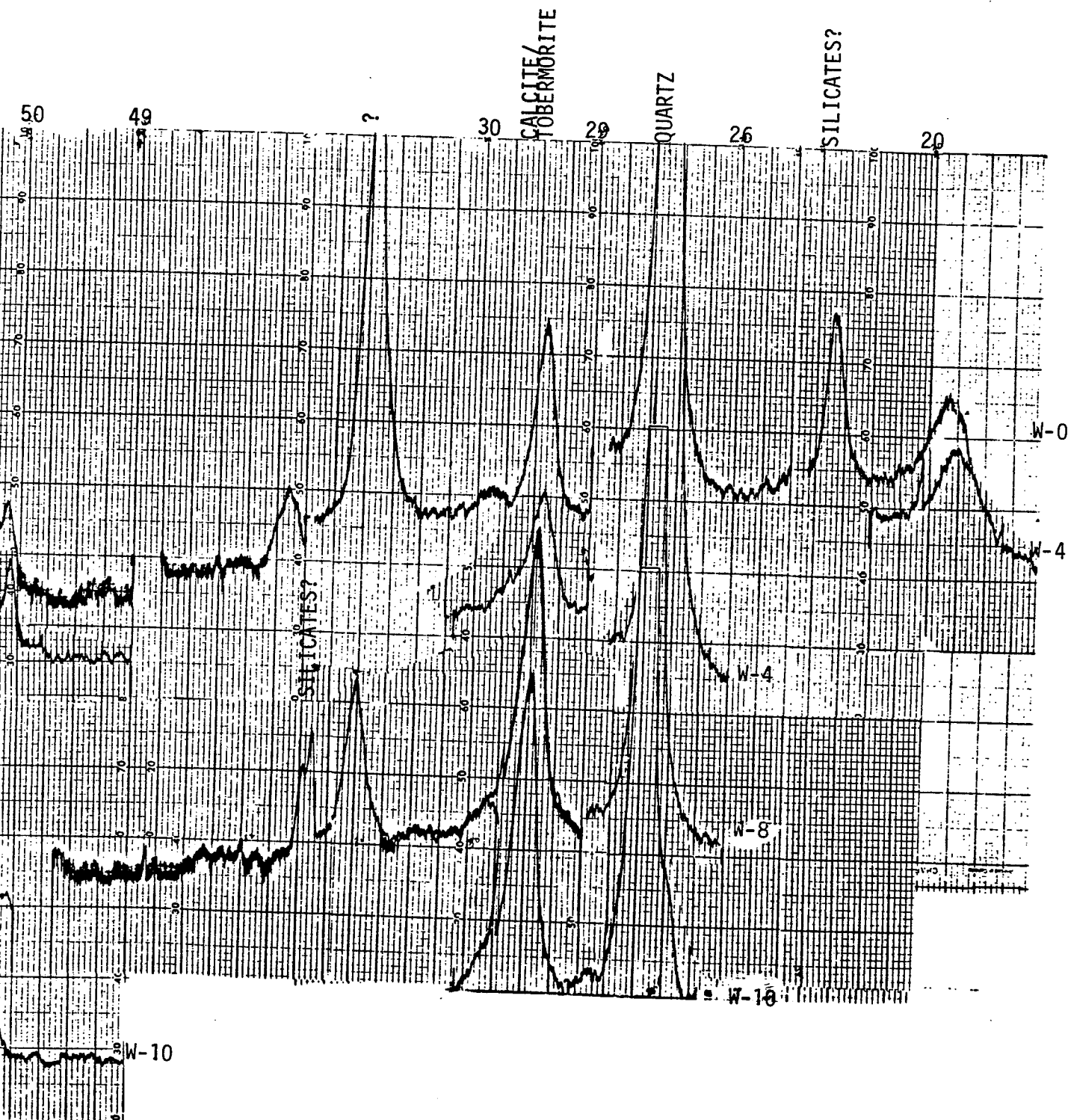


FIG. 76. X-RAY DIFFRACTION PATTERNS FOR PORTLAND CEMENT PASTES.



G- 77. X-RAY DIFFRACTION PATTERNS FOR LIME TREATED LAKE AGASSIZ CLAYS AT A SCANNING SPEED OF $1/8^\circ$ PER. MINUTE.

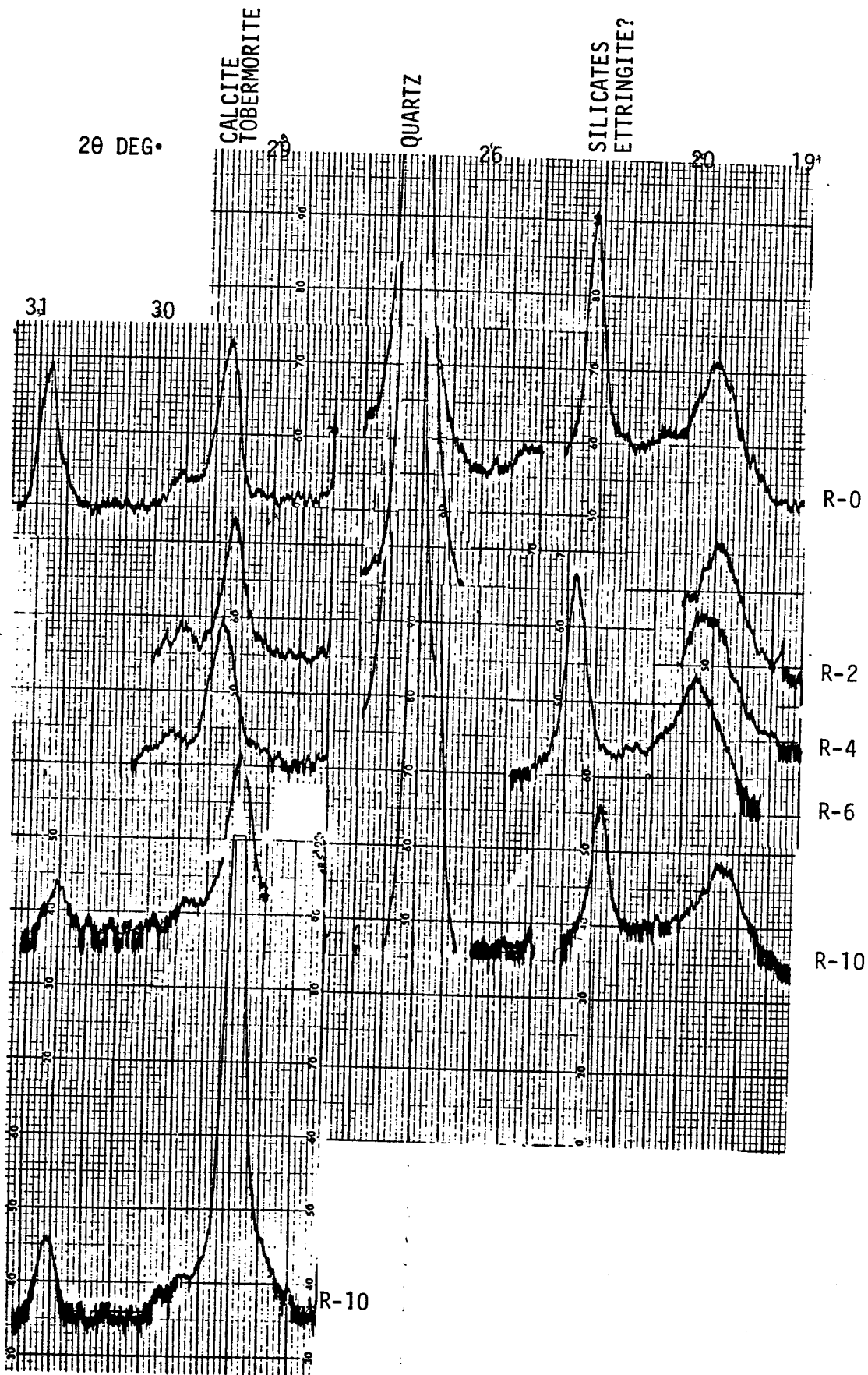


FIG. 78. X-RAY DIFFRACTION PATTERNS FOR LIME TREATED LAKE REGINA CLAYS AT A SCANNING SPEED OF $1/8^{\circ}$ PER MINUTE.

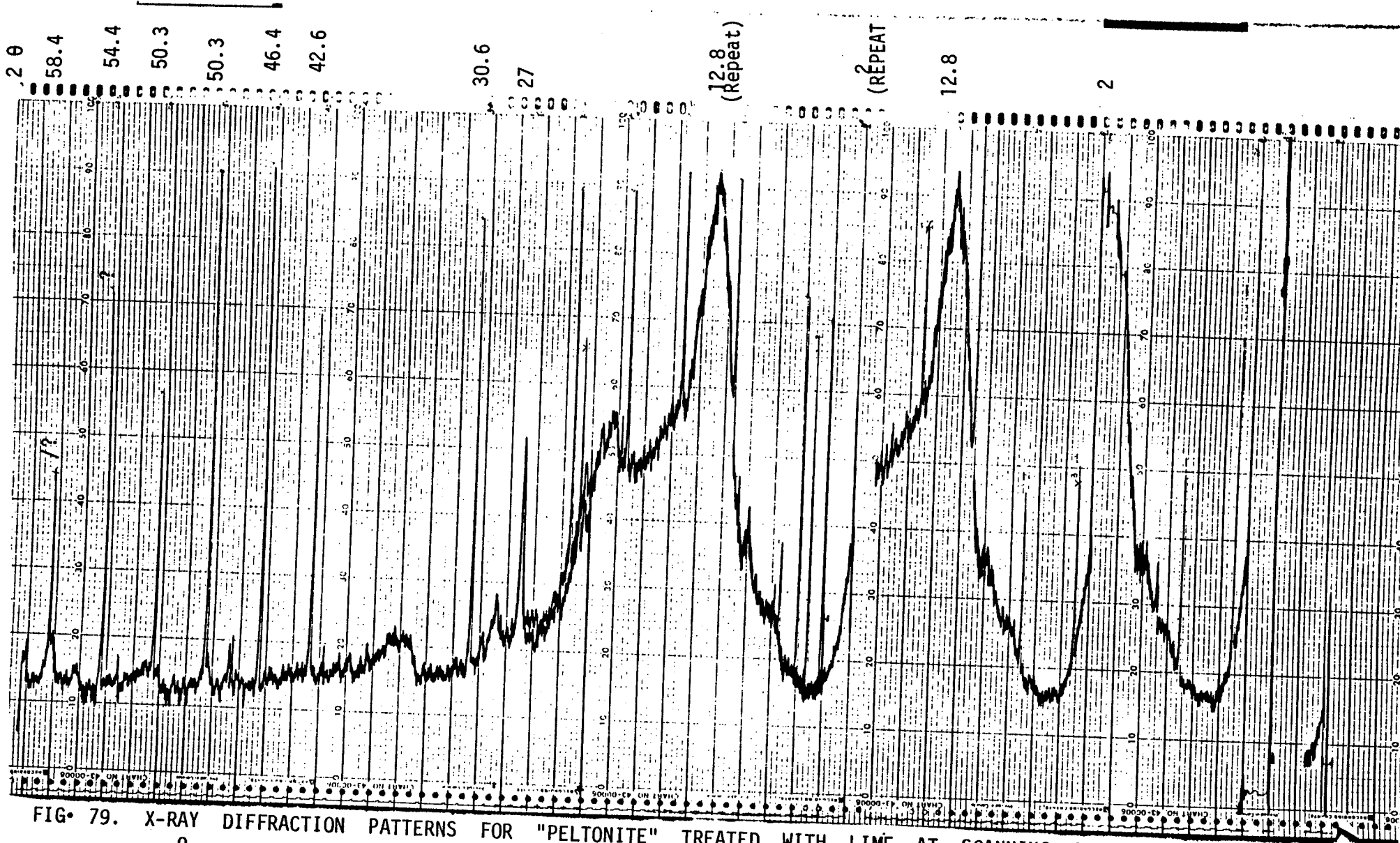
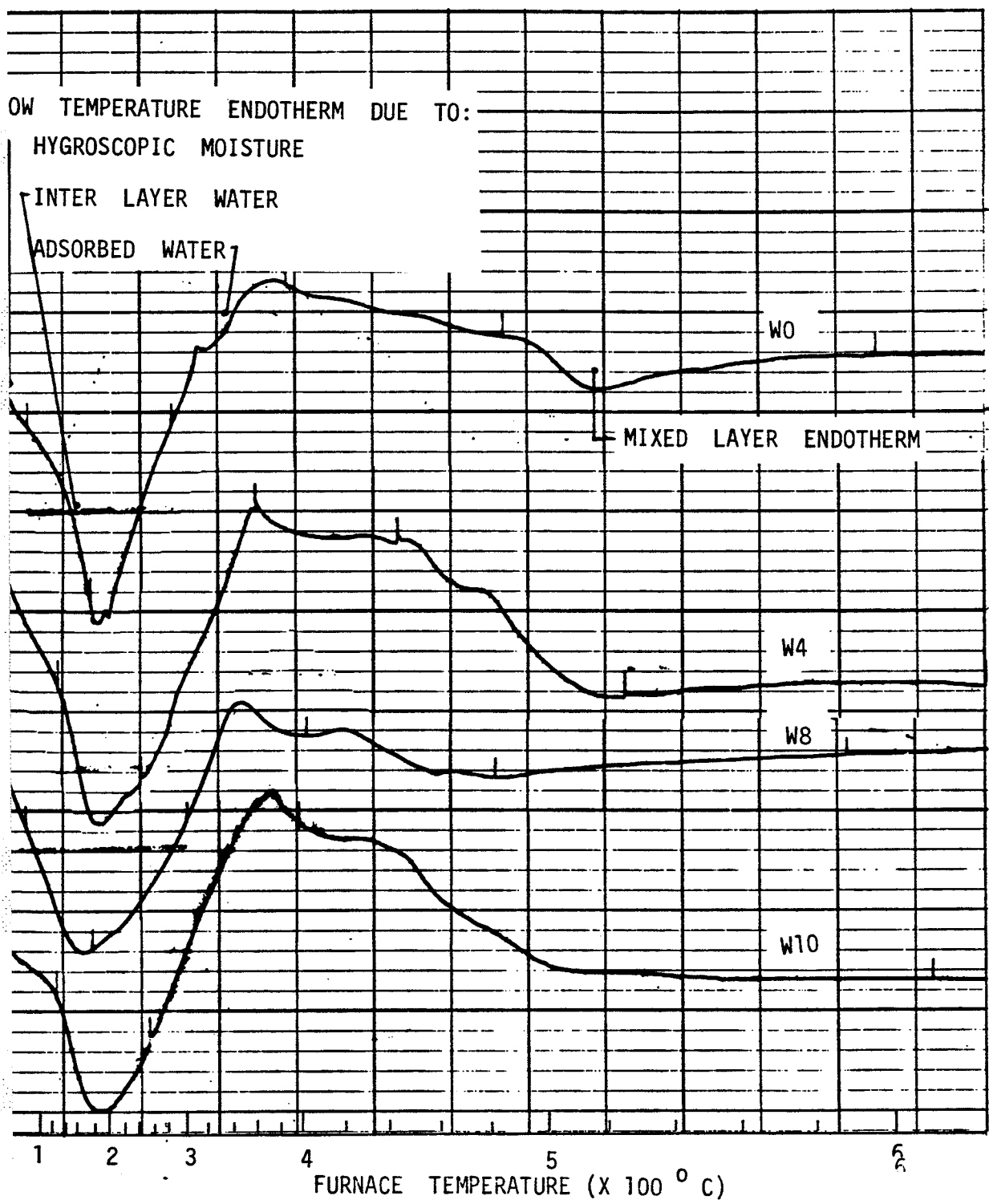


FIG. 79. X-RAY DIFFRACTION PATTERNS FOR "PELTONITE" TREATED WITH LIME AT SCANNING SPEEDS OF $1/8^\circ$ PER MINUTE.



80. DIFFERENTIAL THERMOGRAMS FOR LAKE AGASSIZ CLAYS TREATED WITH VARIOUS AMOUNTS OF QUICKLIME.

DIFFERENTIAL TEMPERATURE.

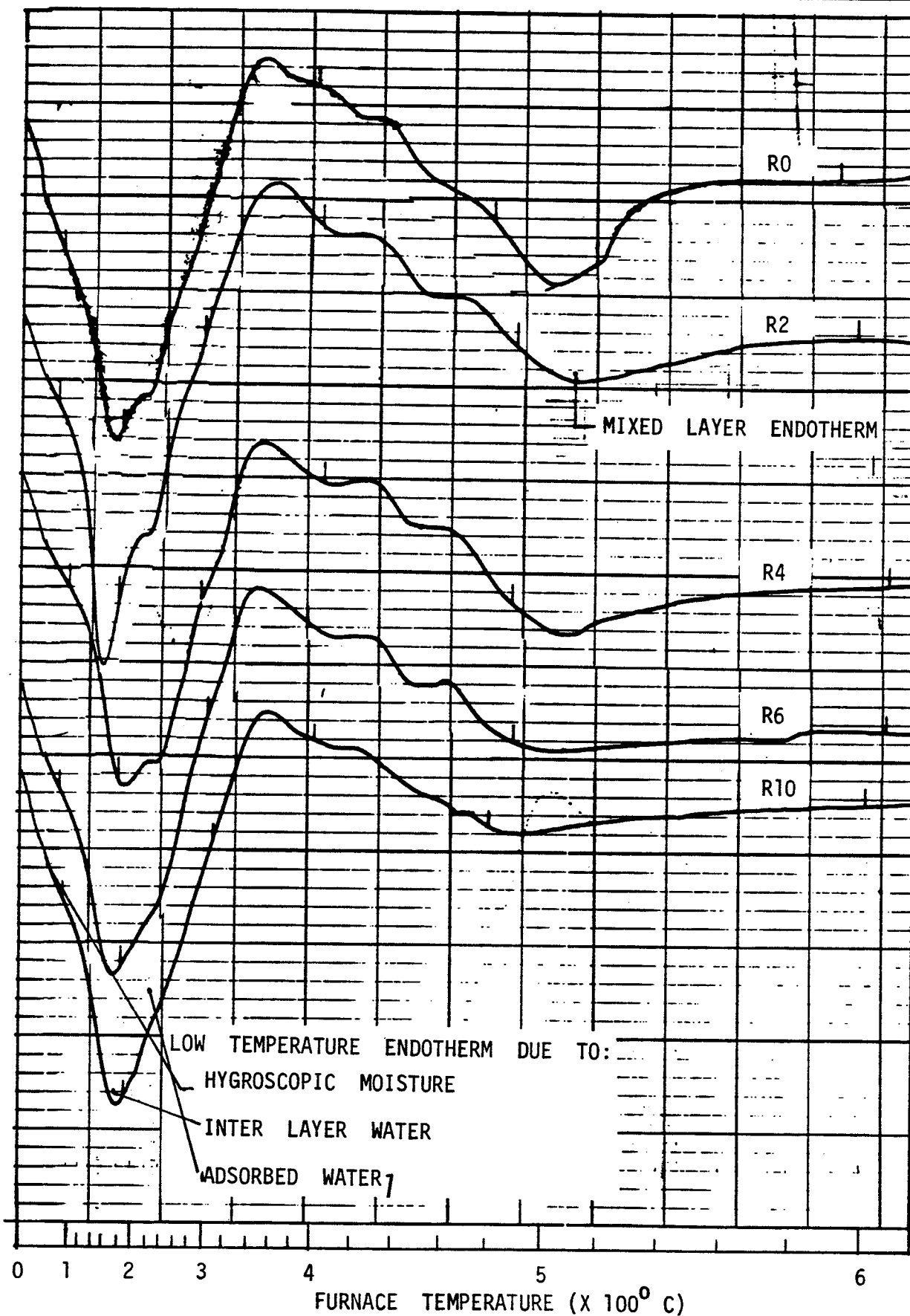


FIG. 81. DIFFERENTIAL THERMOGRAMS FOR LAKE REGINA CLAYS TREATED WITH VARIOUS AMOUNTS OF QUICKLIME.

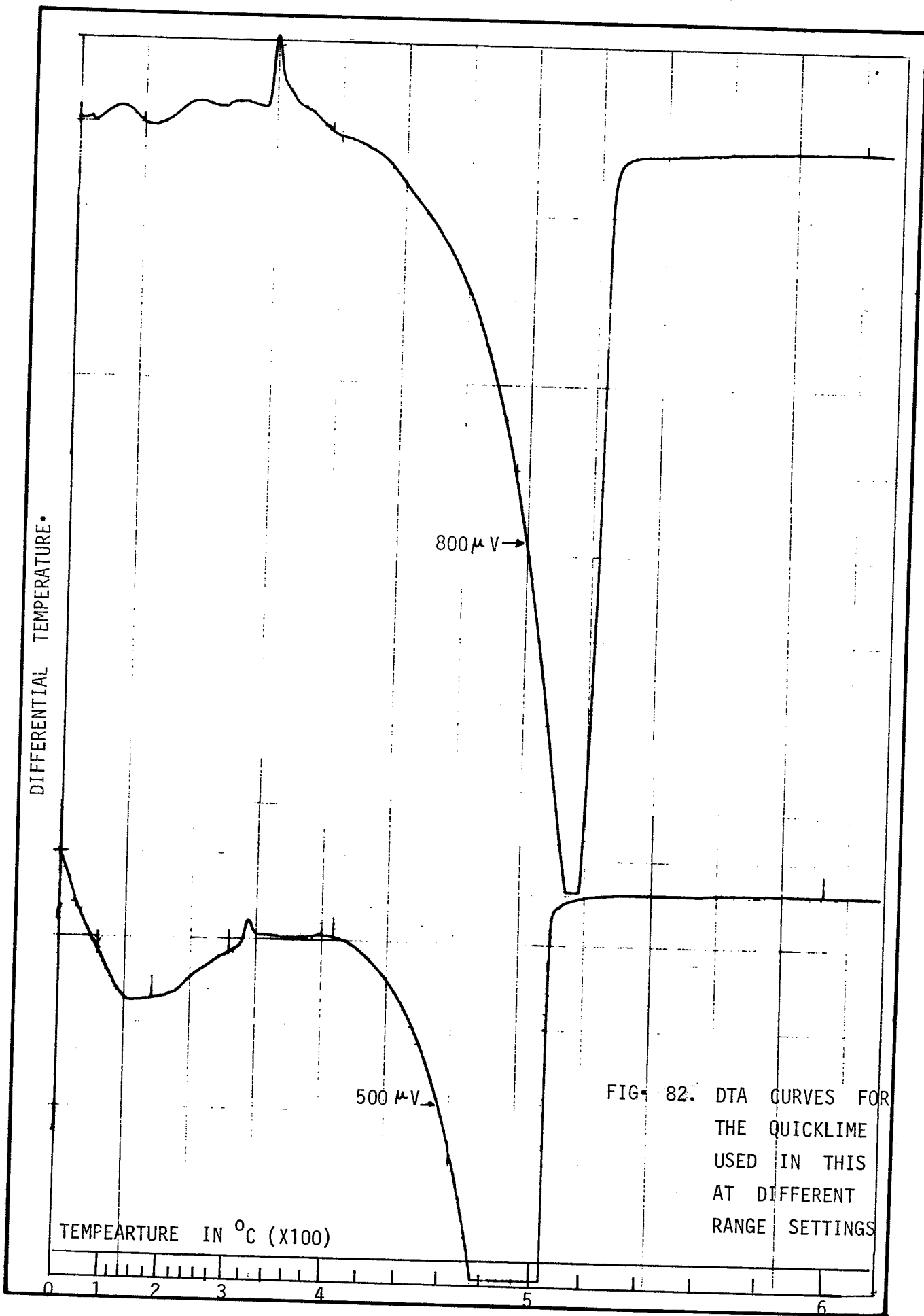
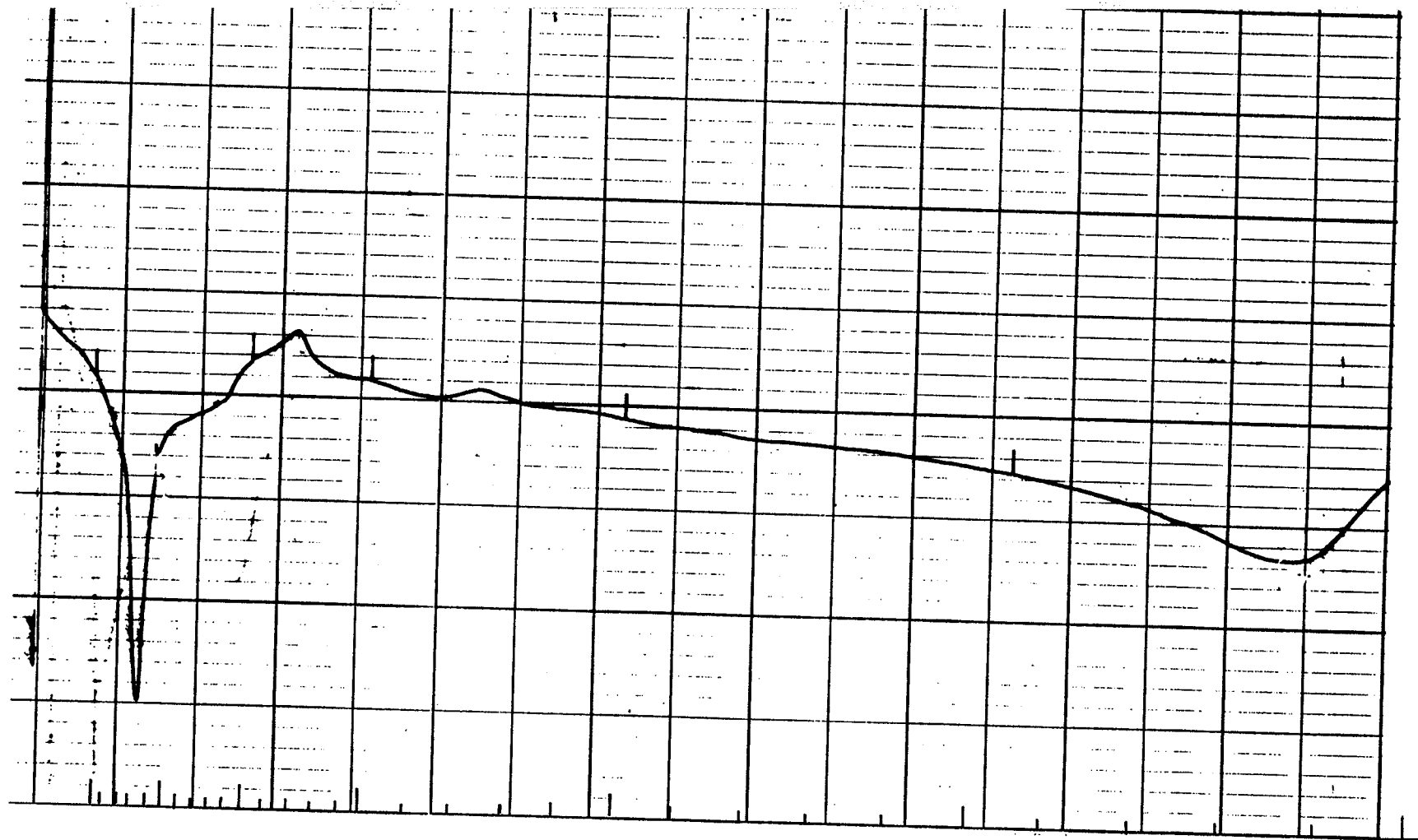
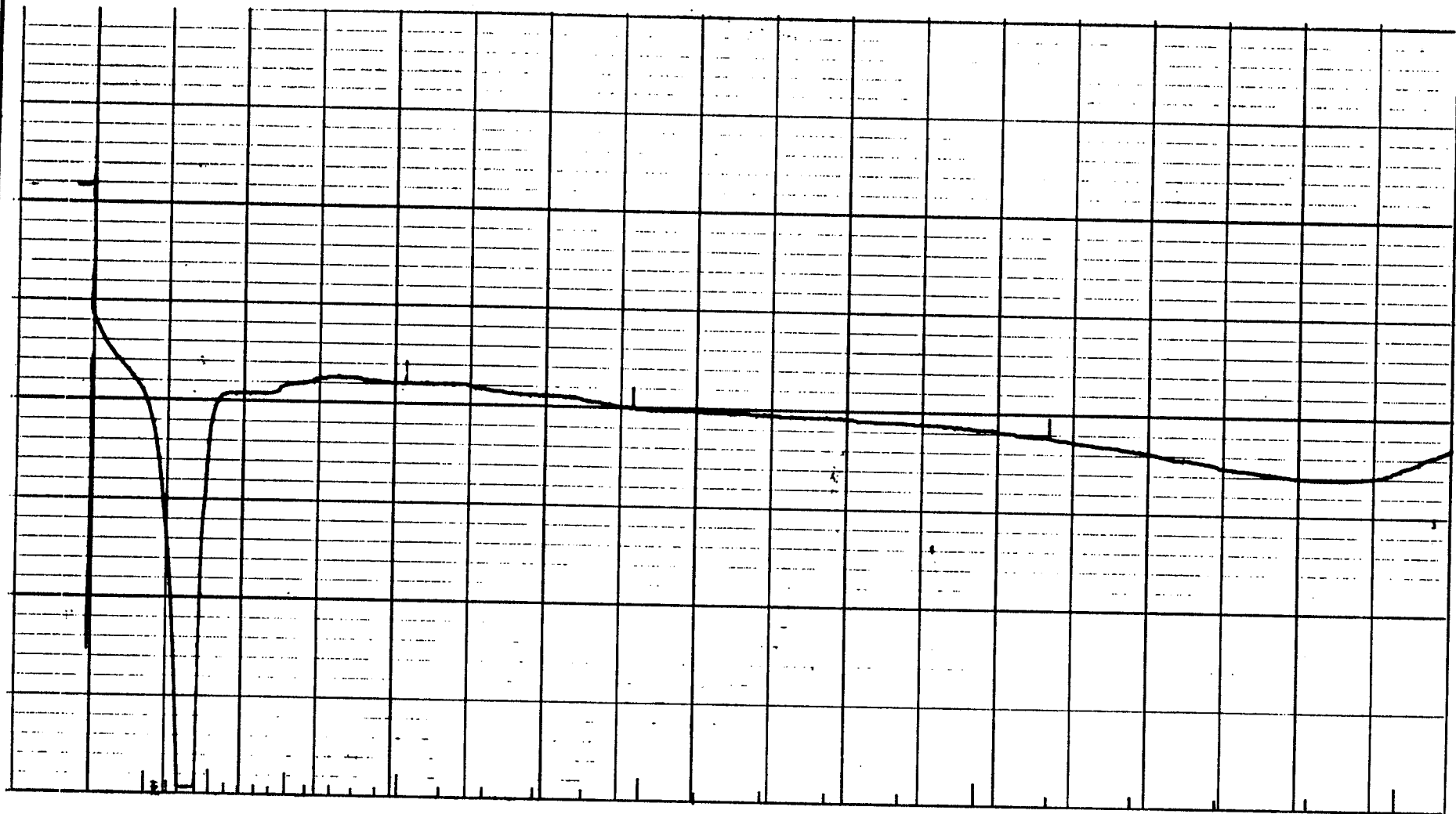


FIG. 82. DTA CURVES FOR THE QUICKLIME USED IN THIS AT DIFFERENT RANGE SETTINGS



FURNACE TEMPERATURE ($\times 100^\circ\text{C}$)

FIG. 83. DIFFERENTIAL THERMOGRAM FOR A SAMPLE OF "PELTONOITE"



FURNACE TEMPERATURE (1100° C)

FIG. 84. DIFFERENTIAL THERMOGRAM FOR A SAMPLE OF BENTONITE FROM MANITOBA.

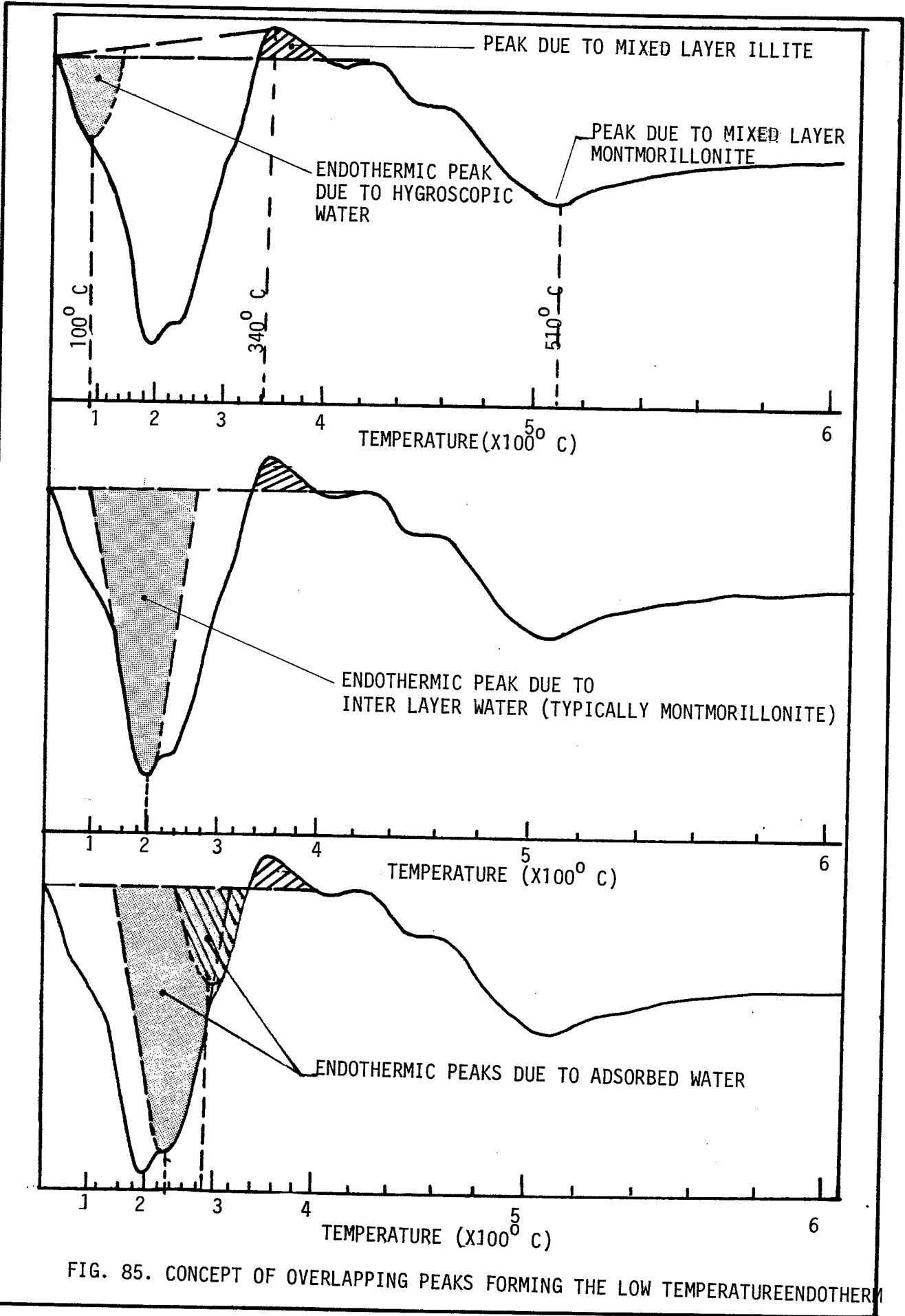


FIG. 85. CONCEPT OF OVERLAPPING PEAKS FORMING THE LOW TEMPERATURE ENDOTHERM

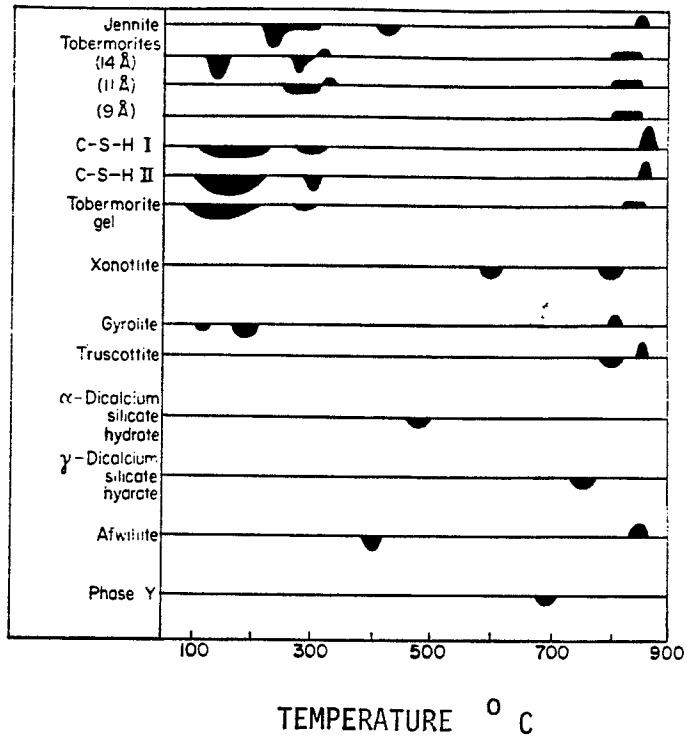


FIG. 86. FORMALIZED DTA PATTERNS FOR DIFFERENT CALCIUM SILICATE HYDRATES. (AFTER MACKENZIE, 1970).

AGGLOMORATION

EDGE TO FACE
CONTACTS.

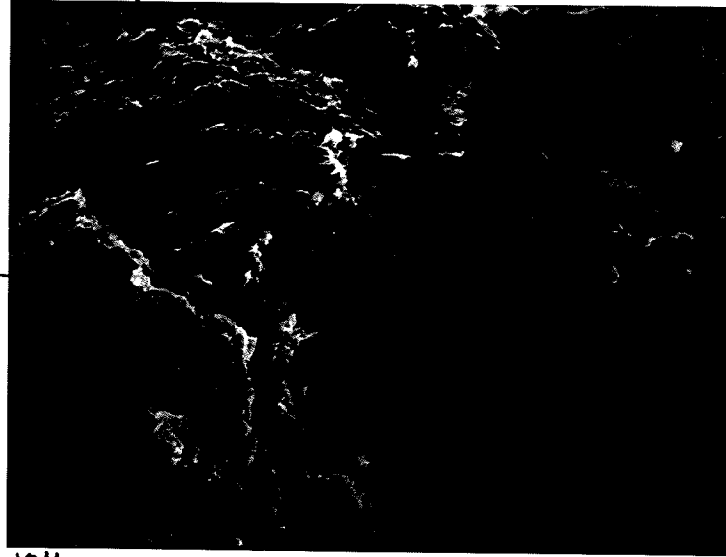
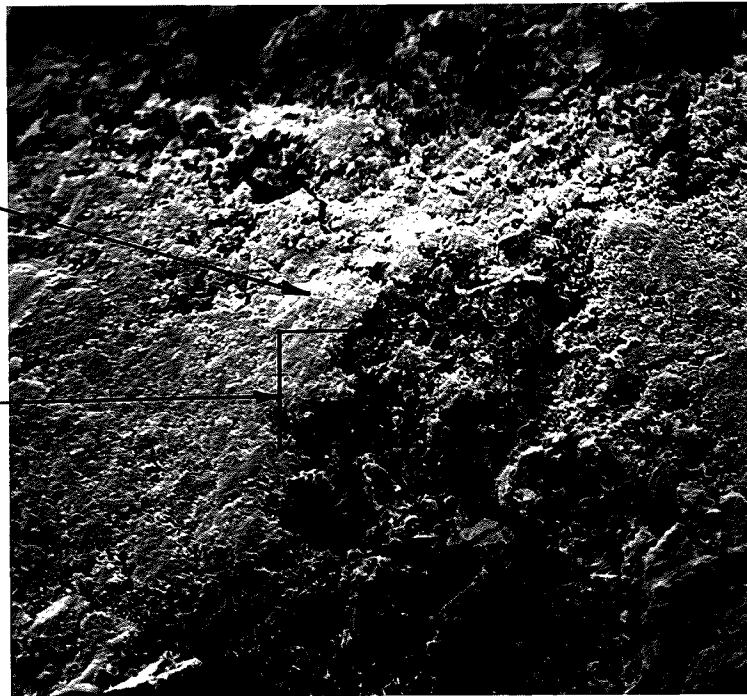


FIG. 87. ELECTRON MICROGRAPH OF SAMPLE OF CLAY FROM WINNIPEG
TREATED WITH 2% QUICKLIME. (1000 X)

MASSIVE
AGGLOMORATION

SEE FIG. 88A



10 μ

FIG. 88. ELECTRON MICROGRAPH OF A SHEAR SAMPLE OF LIME TREATED
SUBGRADE, SASKATCHEWAN DEPARTMENT OF HIGHWAYS (SAMPLE # 9325)
(1000 X)

THE AREA UNDER THE RECTANGLE IS SHOWN MAGNIFIED ON FIG. 88A

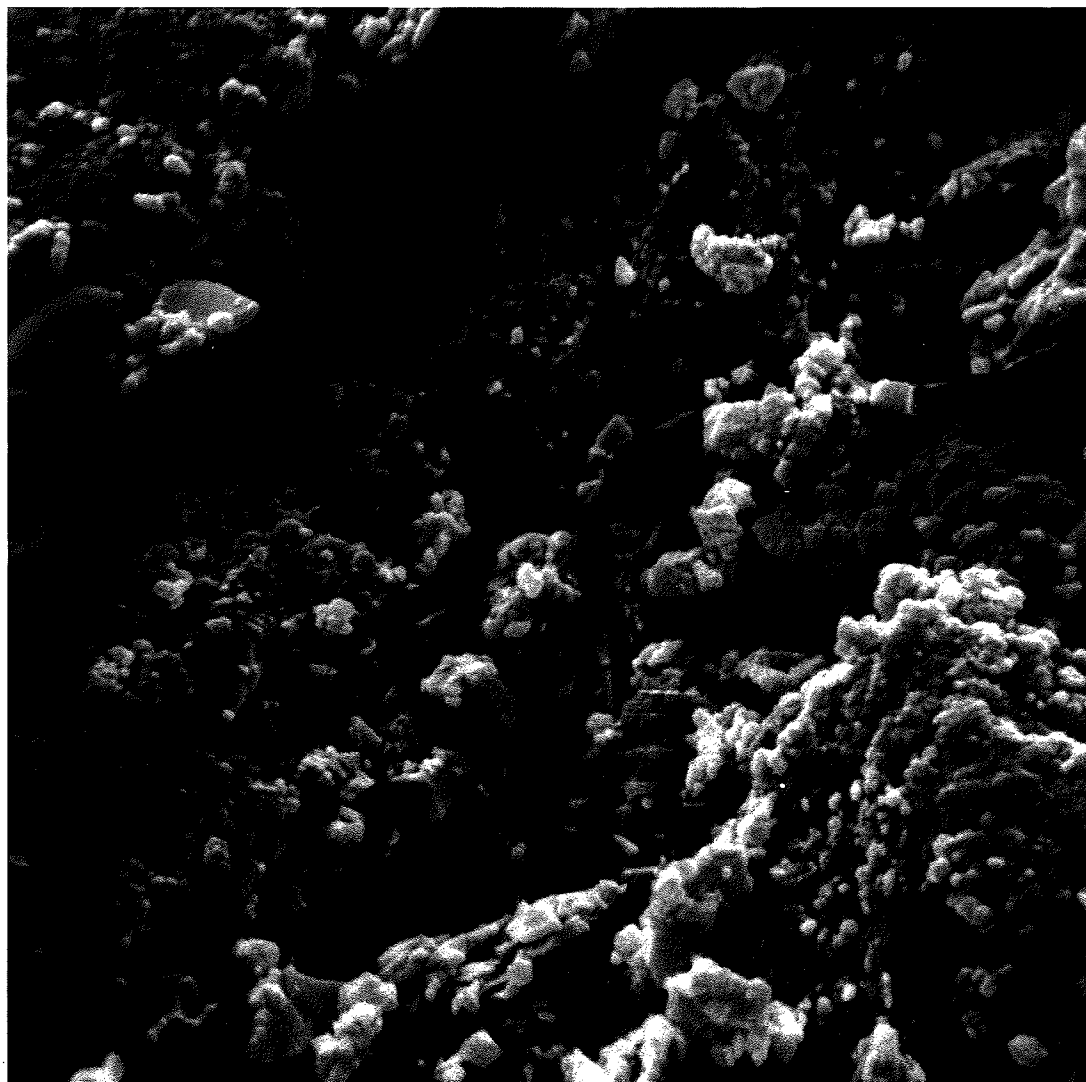


FIG. 88A. ELECTRON MICROGRAPH OF A SAMPLE OF LIME TREATED SUBGRADE FROM SASKATCHEWAN HIGHWAYS (# 9325). THE AREA SHOWN ENCLOSED BY THE RECTANGLE IN FIG. 88. (6500X).

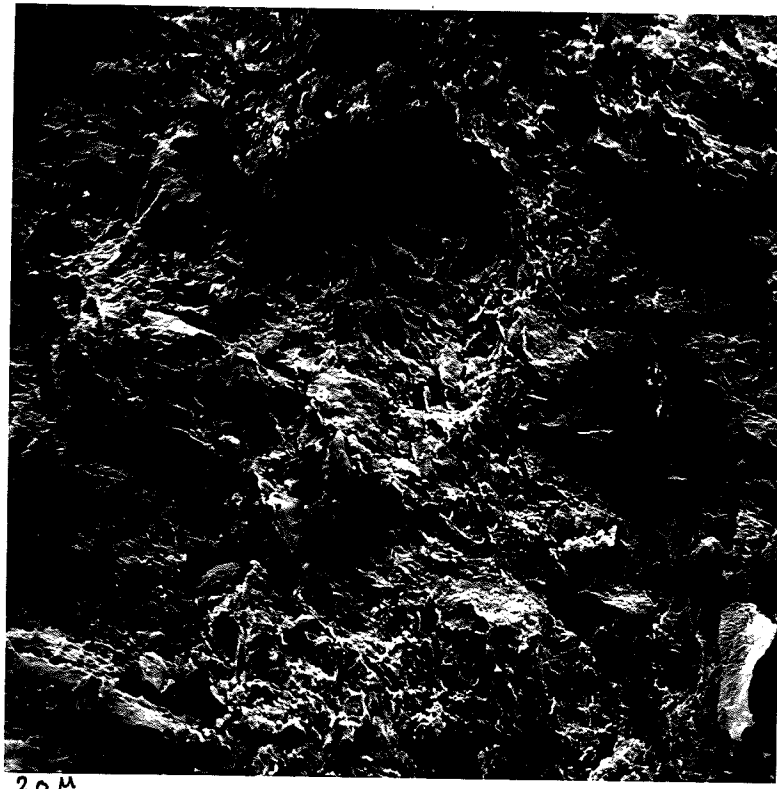


FIG. 89. ELECTRON MICROGRAPH OF A COMPACTED CLAY SUBGRADE
(UNTREATED) FROM SASKATCHEWAN HIGHWAYS (# 9325
BOTTOM PART) MAGNIFICATION 750 X.

A P P E N D I X II.

T A B L E S

TABLE I. RESULTS OF X-RAY DIFFRACTION ANALYSIS REPORTED BY DIFFERENT INVESTIGATORS.

No.	INVESTIGATOR(S)	MINERAL + TYPE OF LIME	BASAL SPACING (d Å).	REMARKS.
1.	GOLDBERG AND KLEIN (1952)	WYOMING BENTONITE GEORGIA KAOLIN ILLITE ALL REACTED WITH QUICK- LIME.	NO REACTION PRODUCT FORMED LIME APPEARS TO HAVE CARBONATED FORMING CALCITE	ASTM:STP 142(1952) pp.53-67.
2.	EADES AND GRIM (1960) EADES (1962)	GEORGIA KAOLINITE ILLITE (FITHIAN AND GRUNDITE) MONTMORILLONITE (UPTON; MISSISSIPPI) ALL REACTED WITH DRY HYDRATED LIME.	5.09: 3.04: 2.8: 1.8 AND OTHER CALCIUM SILICATE HYDRATES	HRB BULLETIN 266 (1960): pp 51 Ph.D. THESIS AT UNIVERSITY OF ILLINOIS (1962)
3.	EADES, NICHOLS AND GRIM (1962)	NATURAL CLAYS AND HYDRATED LIME CLAYS PREDOMINANTLY MONTMORILLONITIC	6.3-6.8 (2θ = 13-14°) 4.9-5.1 (2θ = 17-18°) 3.05 2.7 (2θ = 33°): 2.4 (2θ = 37°) 2.3 (2θ = 39°): 1.8 (2θ = 50°)	HRB BULLETIN 335 (1962): pp. 31.
4.	HILT AND DAVIDSON (1961)	ILLINOIS PLASTIC LOESS GUMBOTIL HYDRATED QUICKLIME SOILS PREDOMINANTLY MONTMORILLONITIC	8.11: 7.59: 4.90: 4.48: 4.26: 4.05: 3.78: 3.34: 3.04	HRB BULLETIN 304 (1961): pp.51
5.	DIAMOND, WHITE AND DOLCH (1963)	KAOLINITE, BATH, S.C. MONTMORILL. OTAY, CALIF. ILLITE, OKLAHAMA. MICA, "DELAMICA" HIGH CALCIUM LIME	5.12: 3.11: 3.04: 2.78: 2.28 3.06: 2.74 NO NEW PRODUCTS IDENTIFIABLE	12 th NATIONAL CONFERENCE ON CLAYS AND CLAY MINERALS:(1963) pp.359.

TABLE I (CONTD.). RESULTS OF X-RAY DIFFRACTION ANALYSIS REPORTED BY DIFFERENT INVESTIGATORS.

No	INVESTIGATOR(S)	MINERAL + TYPE OF LIME	BASAL SPACINGS (d Å)	REMARKS
6.	GLENN AND HANDY(1963)	BENTONITE, OTAY, CALIF. +HIGH CALCIUM HYDRATED LIME BENTONITE+Ca(OH) ₂ + MgO BENTONITE + DIHYDRATE KAOLIN, LEWISTON, MONT. +Ca(OH) ₂ KAOLIN + Ca(OH) ₂ + MgO KAOLIN + DIHYDRATE VERMICULITE, LINCOLN CO MONT. + Ca(OH) ₂ VERMICULITE+Ca(OH) ₂ + MgO VERMICULITE+DIHYDRATE MUSCOVITE	10.0: 7.6: 3.79: 2.54: 10.0: 8.8: 10.0 7.6: 3.05: 12.6: 7.6: 3.05: 12.6: 7.6: 3.05 8.1: 4.59 3.03(?) NONE 7.5 WITH Ca(OH) ₂ BUT NONE WITH OTHER LIMES.	10.0 A IS TOBERMORITE HRR 29:(1963):pp.70 HRR 92:(1965): pp pp. 51-58 HRB: SPECIAL REPORT No. 90:pp.475.
7.	LAGUROS,HANDY AND REIGN (1965)	BENTONITE Na-CLAY Ca(OH) ₂ Ca-CLAY		
8.	WANG AND HANDY (1960)	PANTHER CREEK BENTONITE WHITE SPRINGS, MISS. + Ca(OH) ₂ BENTONITE +Ca(OH) ₂ +MgO	3.05: 2.79: 1.82:1.635: 5.34: 2.667: 1.77: 7.56 15.2: 7.0:	

TABLE II.—X-RAY d -SPACINGS—WET SLICE AND REWETTED SAMPLE DATA FOR PASTES OF CEMENTS LOW IN TRICALCIUM ALUMINATE AND OF FINE CEMENTS. (Spacings in Å (uncorrected).)

Possible Contributing Product ^a	C33-1	C33-1 ^b	C65-F-22	C31-1	C60-C-16	C68-D-17	C32-1	C67-C-22
A.....	9.8	9.8	9.8	9.7	9.8	9.8	9.8	9.8
B.....	...	8.8 ^c	9.1	9.0	8.9	8.9 ^c
C.....	8.3	8.1
D.....	7.8	7.8	7.8	7.8	7.7	7.8	7.8	7.8
E.....	7.4	7.4	7.3	7.3
A.....	5.64	5.66	5.65	5.64	5.64	5.64	5.64	5.63
F.....	5.06 ^d	...	5.09 ^d	5.06 ^d	5.10	5.12
A.....	5.01	...	4.99
G.....	4.93	4.94	4.92	4.94	4.93	4.92	4.92	4.93
A.....	4.72	4.72	4.72	4.72	4.70	4.71	4.72	4.72
B.....	4.50
A, F.....	4.39	4.40	4.42	4.41	4.41	4.41	...	4.39
K.....	4.26	4.26	4.29	...
A, B.....	4.04	...	4.00	4.02	4.02	...
A, C, H.....	3.88	3.88	3.88	3.88	3.88	3.88	3.88	3.88
D.....	3.81
A, C.....	3.67	3.67	3.66	3.69	...
A.....	3.59	3.59	3.59	3.62	...	3.62
A.....	3.48	3.48	3.49	3.49	...	3.50	3.49	3.48
K.....	3.35	3.38	3.39	3.35	3.36	...
F.....	...	3.32	3.29	3.31	3.32	3.34
A.....	3.26	...
F, G.....	3.12	3.12	3.11	3.12	3.11	3.11	3.12	3.11
H, I.....	3.05	3.05	3.04	3.05	3.04	3.04	3.05	3.04
K.....	2.99	3.00
B, D.....	2.89	2.88
K.....	2.84	2.83
A, F, I, J.....	2.78	2.78	2.79	2.77	2.78	2.79	2.78	2.78
J.....	2.76
E.....	2.67	2.69	2.68	2.70	2.68
G.....	2.63	2.63	2.63	2.63	2.63	2.63	2.63	2.63
A.....	2.57	2.58	2.57	2.56	2.58	2.57	2.56	2.58
D, F.....	2.54	2.53	2.54	2.53	...	2.53	...	2.54
H.....	2.51	...	2.50	2.50	2.50	2.51	2.51	2.51
B, C, G.....	2.46	2.45	...	2.45	2.45	2.45
B, D, F.....	2.42	2.44	2.44	2.43	2.44	2.41	2.41	2.42
A, D.....	...	2.35	2.36	2.36	2.35	2.37
H.....	2.29	2.29	2.29	2.29	2.30	2.29	2.29	2.31
F.....	2.27	2.27	2.27	2.26	2.27	2.27	...	2.27
A, B, J.....	2.21	2.21	2.21	2.21	2.22	2.21	2.21	2.21
A, C.....	2.15	2.16	2.15	2.14	...	2.16	2.16	2.15
D, H.....	2.09	2.10	2.10	2.10	2.10	2.10	2.10	2.11
B.....	2.06	2.06	2.06	2.06	...	2.06	2.06	2.06
D, F.....	2.01	2.01	2.01	2.01	2.02	2.03	2.01	2.02
A, D, F.....	1.96	1.96	1.99	1.97	...
G, H.....	1.930	1.931	1.930	1.930	1.930	1.928	1.932	1.932
H.....	1.912	...	1.912	1.912	1.910	1.916	1.912	...
H.....	1.879	...	1.883	1.875	1.882	1.880	1.879	...
B, I.....	1.823	1.824	1.825	1.823	1.826	1.825	1.826	1.823
G.....	1.797	1.798	1.799	1.797	1.799	1.799	1.801	1.799
F.....	1.722	1.720	1.726	1.717	1.720	1.729	...	1.725
D, G.....	1.689	1.689	1.690	1.688	1.689	1.689	1.689	1.690
{B, C, D, F}.....	{1.657}	1.663	{1.660}	1.667	{1.660}	{1.664}	{1.665}	1.668
B, G.....	1.636	1.656	...	1.654	1.662
		1.638	...	1.638	1.641	1.637	1.634	1.640

^a The letters in this column represent the following substances: A, ettringite; B, calcium monosulfoaluminate; C, tetracalcium aluminate hydrate; D, calcium monocarboaluminate; E, unhydrated ferrite phase; F, hydrogarnet-like phase; G, calcium hydroxide; H, calcite; I, tobermorite; J, unhydrated silicate phase; K, not known.

^b Dried sample, rewetted in a saturated atmosphere.

^c Badly obscured by strong peak. Spacing cannot be estimated.

^d Shoulder on strong peak. Exact d spacing not certain.

^e Line is very weak. Its presence is questionable.

(AFTER KANTRO ET AL, 1960)

TABLE III DETAILS OF SAMPLING (75 mm DIA SHELBY'S) FROM THE TEST SECTIONS BY THE DEPARTMENTS OF HIGHWAYS OF THE PROVINCES OF MANITOBA AND SASKATCHEWAN.

SAMPLE No.	SITE	STATION	DEPTH (cm)	REMARKS
9284	OAK BLUFF, MANITOBA	17+00 to 20+80	7.5 TO 70	5% LIME
9285	OAK BLUFF, MANITOBA	20+80 TO 25+80	7.5 TO 70	5% LIME
9286	OAK BLUFF, MANITOBA	25+80 TO 30+80	7.5 TO 70	5% LIME
9287	SELKIRK BY-PASS, MAN	0+00 TO 15+85	20 TO 80	7% LIME
9288	SELKIRK BY-PASS, MAN	15+85 TO 31+70	20 TO 80	5% LIME
9289	SELKIRK BY-PASS, MAN	31+70 TO 42+25	20 TO 80	8% LIME
9290	SELKIRK BY-PASS, MAN	42+25 TO 58+10	20 TO 80	6% LIME
9320	KALIUM RD. SASK.		75 TO 135	UNTREATED
9321	HWY. 6 SOUTH OF CORINNE, SASK.		80 TO 140	UNTREATED
9322	HWY 33/PTH 1 BY-PASS REGINA, SASK.		80 TO 140	UNTREATED
9323	HWY. 6 SOUTH OF CORINNE, SASK.		20 TO 80	LIME TREATED
9324	KALIUM RD. SASK.		15 TO 75	LIME TREATED
9325	HWY. 33/PTH 1 BY-PASS REGINA, SASK.		20 TO 80	LIME TREATED

TABLE IV. SUMMARY OF GEOTECHNICAL PROPERTIES AND MINERALOGY OF
THE CLAYS FROM THE CANADIAN PRAIRIES.
(DATA COLLECTED FROM DIFFERENT REPORTS)

PROPERTIES	LAKE AGASSIZ	LAKE REGINA	SOUTHWESTERN SASKATCHEWAN.
<u>GRAIN SIZE</u>			
MINUS 4.75 mm	100	100	100
MINUS 0.075 mm	90 - 100	90 - 100	90 - 100
MINUS 0.002 mm	45 - 80	50 - 85	40 - 95
<u>ATTERBRG'S LIMITS.</u>			
LIQUID LIMIT %	55 - 90	55-90	35 - 85
PLASTIC LIMIT %	23 - 35	25-35	16 - 27
PLASTICITY INDEX %	35 - 55	30-60	20 - 55
<u>SHEAR STRENGTH</u>			
IN-SITU TORVANE kPa	5 - 20	8-20	10 - 25
REMOLED KPa	5 - 10	5-10	5 - 10
UNCONFINED COMPRESSION (kPa)	5 - 25	5-25	15 - 30
<u>BEARING VALUES (C•B•R)</u>			
AS MOLDED %	15 - 30	15-35	20 - 60
SOAKED 72 HRS. %	< 5	< 3	< 3
<u>CONSOLIDATION</u>			
COMPRESSION INDEX	0.4 - 0.6	0.15 - 0.20	NO DATA
SWELLING PRESSURE kPa	8 - 10	20 - 40	NO DATA
PRECONSOLIDATION PRESS. (kPa)	140 - 220	50 - 130	NO DATA
<u>MINERALOGY</u>			
KAOLINITE %	10 - 15	10 - 15	10 - 30
ILLITE %	15 - 45	15 - 45	10 - 15
MONTMORILLONITE %	55 - 80	55 - 80	40 - 60
QUARTZ %	5 - 15		
CALCITE %	< 5	< 5	
DOLOMITE %	10 - 15	< 10	
FELDSPARS %	< 5	< 10	
OTHERS %	< 10	< 10	

TABLE V WATER CONTENT OF LIME TREATED SAMPLES AT DIFFERENT STAGES OF TESTING.

SAMPLE	WATER CONTENT (%) AFTER				REMARKS
	MIXING	PRIOR TO COMPACTION	AFTER COMPACTION	AFTER SHEARING	
W-2-3d	30.17	-	30.99	29.70	
W-2-7d	29.31	27.62	27.33	25.52	
W-2-28d	29.98	23.22	27.59	26.43	
W-4-3d	32.19	-	31.07	30.14	
W-4-7d	25.36	22.26	23.88	23.24	
W-4-28d	33.40	-	34.65	34.04	
W-6-3d	30.13	28.30	28.23	27.29	
W-6-7d	24.06	22.36	23.61	23.51	
W-6-28d	(?)36.13	33.77	33.09	32.28	
W-8-3d	21.25	23.58	24.94 *	24.78	* ∴ Water Added
W-8-7d	30.99	27.98	28.20	27.67	
W-8-28d	-	-	-	-	
W-10-3d	24.18	23.56	24.83	25.54	
W-10-7d	30.40	28.64	29.20	28.85	
W-10-28d	24.18	-	26.17	25.96	
R-2-3d	29.67	29.86	29.20	28.81	
R-2-7d	29.67	-	27.53	27.53	
R-2-28d	26.99	25.60	27.89*	28.30	
R-4-3d	28.80	24.78	24.79	24.31	
R-4-7d	29.40	-	25.06	24.86	

TABLE VI. pH VALUES OF NATURAL AND LIME TREATED CLAYS.

SAMPLE	pH VALUE AFTER CURING PERIOD OF					
	1 HR.	12 HRS.	3 DAYS	7 DAYS	28 DAYS	LONG TERM
QUICKLIME	12.5	12.2				
W-0	7.4		7.4			
W-2	11.0	11.6	11.4	11.5	11.4	10.0
W-4	11.0	11.8	11.4	11.8	11.6	11.2
W-6	11.0	11.6	11.5	10.6	11.5	10.8
W-8	11.0	11.0	11.2	10.7	11.9?	11.2
W-10	11.0	10.8 ?	11.6	10.7	10.8	-
R-0	7.0	7.0	6.9			
R-2	11.0	11.7	11.7	11.0	11.4	9.7
R-4	11.1	12.1	12.1	11.7	11.8	10.9
R-6	11.1	11.5	12.2	11.7	11.8	10.8
R-8	11.1	11.5	12.4	11.7	11.2	11.0
R-10	11.1	11.0	11.0	10.8	11.0	11.7 ?
9323						7.6 -7.9
9324						7.9
9325						8.1
9287						7.8
9288						7.6
9289						8.4
9290						8.1

TABLE VII. BASAL SPACINGS OF KAOLINITE, ILLITE AND MONTMORILLONITE
IN NATURAL AND TREATED CLAYS FROM WINNIPEG AND REGINA.

SAMPLE	MINERAL	CURING PERIOD					
		3 DAYS		7 DAYS		28 DAYS	
		d Å	I/I ₀ ¹⁾	d Å	I/I ₀	d Å	I/I ₀
W-0	M	14.72	34				
	I	9.21	9.6				
	K	7.25	14.9				
W-2	M	NONE*		15.5-15.77	13.9	14.72-15.57	18.6
	I	10.77	7.1	NONE**		NONE**	
	K	7.37	9.5	7.31-7.37	12.5	7.37	13.6
W-4	M	14.72	7.1	12.6-14.7*	vw	NONE	
	I	9.71	10.0	NONE		NONE	
	K	7.37	10.0	7.25	8.0	NONE	
W-6	M	NONE		14.48-16.98*	9.6	17.66	15.4
	I	NONE**		10.27	7.2	NONE	
	K	7.43	6.1	NONE		7.62	13.5
W-8	M	16.98	17.1	NONE		15.76	12.4
	I	NONE**		8.75	10.0	10.01-10.27	6.2
	K	7.43	9.3	7.37	7.5	7.37	8.2
W-10	M	14.72	10.0	14.72	8.5	17.67	10.8
	I	NONE**		NONE**		10.39	6.0
	K	7.43	17.5	7.5	7.5	7.4	8.4

*could be tobermorite or other calcium silicate hydrate

**could signify the destruction of interstratified illite-montmorillonite

¹⁾ 3.34 Å Quartz peak was reference.

TABLE VII. BASAL SPACINGS OF KAOLINITE, ILLITE AND MONTMORILLONITE
(CONTD.) IN NATURAL AND TREATED CLAYS FROM WINNIPEG AND REGINA.

SAMPLE	MINERAL	CURING PERIOD					
		3 DAYS		7 DAYS		28 DAYS	
		d Å	I/I ₀ ¹⁾	d Å	I/I ₀	d Å	I/I ₀
R-0	M	13.68-16.98	25.9				
	I	MIXED LAYER: BROAD BAND PEAKS					
	K	7.13	14.1				
R-2	M	14.24-18.39	9.2	14.72	9.5	14.72-15.77	13.6
	I	?		10.39*	6.3	NONE	
	K	7.49		7.37	7.9	NONE	
R-4	M	11.94-17.66	10.0	NONE		14.72	6.1
	I	BROAD BAND PEAKS		5.0	NONE	NONE	
	K	7.37		NONE		7.43	8.2
R-6	M	15.49	13.2	NONE		15.25-16.35	10.2
	I	NONE		NONE		NONE	
	K	7.43	9.4	NONE		NONE	
R-8	M	16.35	13.0				
	I	10.77*	8.7				
	K	NO SIGNIFICANT PEAKS		NO TEST		NO TEST	
R-10	M			14.24-16.98	12.2	13.8*	9.5
	I	NO TEST		NONE		10.39*	6.7
	K			7.62	6.1	7.37	4.8

* possible tobermorite

¹⁾ 3.34 Å Quartz was reference

TABLE VIII. SOME REACTION PRODUCTS IDENTIFIED FROM X-RAY DIFFRACTION TRACES OF LIME TREATED CLAYS FROM LAKE AGASSIZ AND LAKE REGINA BASINS. (TABULAR VALUES ARE BASAL SPACINGS IN d Å).
 LEGEND: T=TOBERMORITE: E=ETTRINGITE: H=HYDROGARNETS: L=LIME: Q=QUARTZ

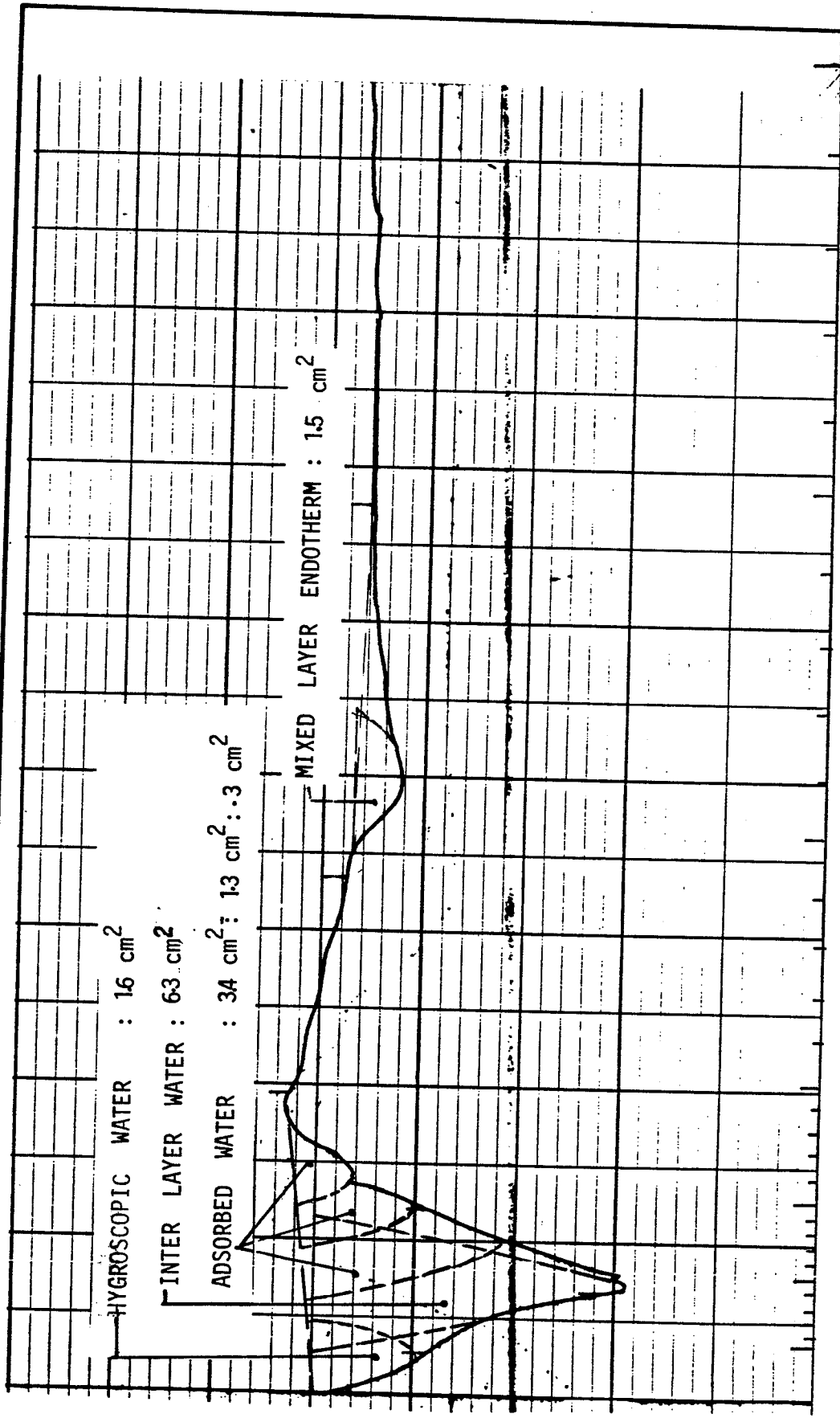
SAMPLE	AFTER 3 DAYS OF CURING	AFTER 7 DAYS OF CURING	AFTER 28 DAYS OF CURING
W-2	3.05(T): 2.92(T,H): 2.14(?): 2.11? 2.00(H)	4.52(E,H):4.33(E,H):3.06(T):2.21(?) 2.14(?): 1.99(H):	3.07(T):1.83(T):1.68(H,L)
W-4	3.05(T): 2.14(?):	4.53(E,H):3.06(T):2.74(E,H,?):2.25 (H):2.14(?):1.99(H)	NOT ANALYSED
W-6	3.08(T): 2.61(Q?):2.15(?)	5.82(E):5.03(H):3.07(T):2.59(?): 1.89(T?):1.697(H,L)	4.60(E,H):3.08(T):3.03(?):2.63(H,L) 2.24(H):1.84(T):1.68(H,L)
W-8	5.79(E): 5.06(H):3.10(T):3.0(T?): 2.66(H?):2.15(?):2.12(?):2.00(H)	4.57(E):4.35(E):3.13(T?):1.65(?)	5.90(E?):4.92(E,H):3.07(T):2.66(H) 2.26(H):1.83(T):1.68(H,L):1.67(?)
W-10	3.08(T):2.66(H?):2.22(H?):2.00(H): 1.697(H,L):	4.57(E):4.39(E):3.07(T):2.96(H,T): 2.24(H?):2.15(?):1.71(H):1.697(H,L)	5.06(H,E):3.08(T):2.99(T):2.66(H): 2.27(H,Q?):2.15(?):1.83(T):1.697(H,L)
R-2	4.43(E,H):3.10(T):2.16(?):1.69(H,L)	4.33(E,H):3.07(T):2.19(?):1.83(T)	4.35(E,H):3.07(T):2.60(H,L):2.01(H) 1.84(T)
R-4	5.09(H):4.35(E,H):3.08(T):2.15(?): 2.00(H):1.83(T):1.71(H,L?):1.69(H?)	3.07(T):2.14(?):1.83(T)	4.37(E,H):3.08(T):2.60(H,L):2.15(?) 1.83(T):1.69(H,L)
R-6	5.21(H):4.35(H):3.08(T):2.66(H,Q?) 2.22(?):2.15(?):1.83(T):1.71(H):1.69	3.07(T):2.14(?):1.83(T):	4.39(E,H):3.08(T):2.60(H?):2.26(?) 2.15(?):1.84(T):1.69(H,L)
R-8	5.06(H):4.39(E,H):3.10(T):2.68(H, Q?):2.27(?):2.15(?):1.84(T):1.71(H)	NOT ANALYSED	4.39(E,H):3.10(T):2.16(?):1.84(T)
R-10	NOT ANALYSED	5.12(H): 4.41(E):3.10(T):2.68(H,Q?) 2.27(?):2.23(H?):1.84(T):1.69(H,L)	4.35(E,H):3.08(T):2.65(H,Q?):2.15 (?):1.83(T):1.69(H,L)

TABLE IX. PEAK AREAS OF THE LOW TEMPERATURE ENDOTHERMS.

SAMPLE	PEAKS (cm ²) DUE TO				REMARKS
	HYGROSCOPIC WATER	INTER LAYER	ADSORBED WATER	MIXED LAYER	
W-0	1.6	6.3	3.4: 1.3: 0.3	1.5	
W-4	2.3	7.4	6.3: 4.3: 1.6	0	
W-8	1.2	6.0	6.5: 0 : 0	0	
W-10	1.3	6.0	5.3: 0 : 0	0	
R-0	1.9	10.4	6.9: 2.8:	4.2	
R-2	1.8	7.4	5.1: 0.7	3.3	
R-4	1.3	8.5	6.7: 1.6	1.7	
R-6	1.8	7.3	5.5: 0.8	0.3	
R-10	2.2	8.6	6.0: 4.6(?)	0.4	
PELTONITE	0.3	5.5	1.0		
BENTONITE	1.0	5.2	none.		

A P P E N D I X III.

DIFFERENTIAL THERMOGRAMS - ANALYSIS.



FURNACE TEMPERATURE (100° C)
 FIG. IV-1. DIFFERENTIAL THERMOGRAM FOR LAKE AGASSIZ CLAY (NATURAL)

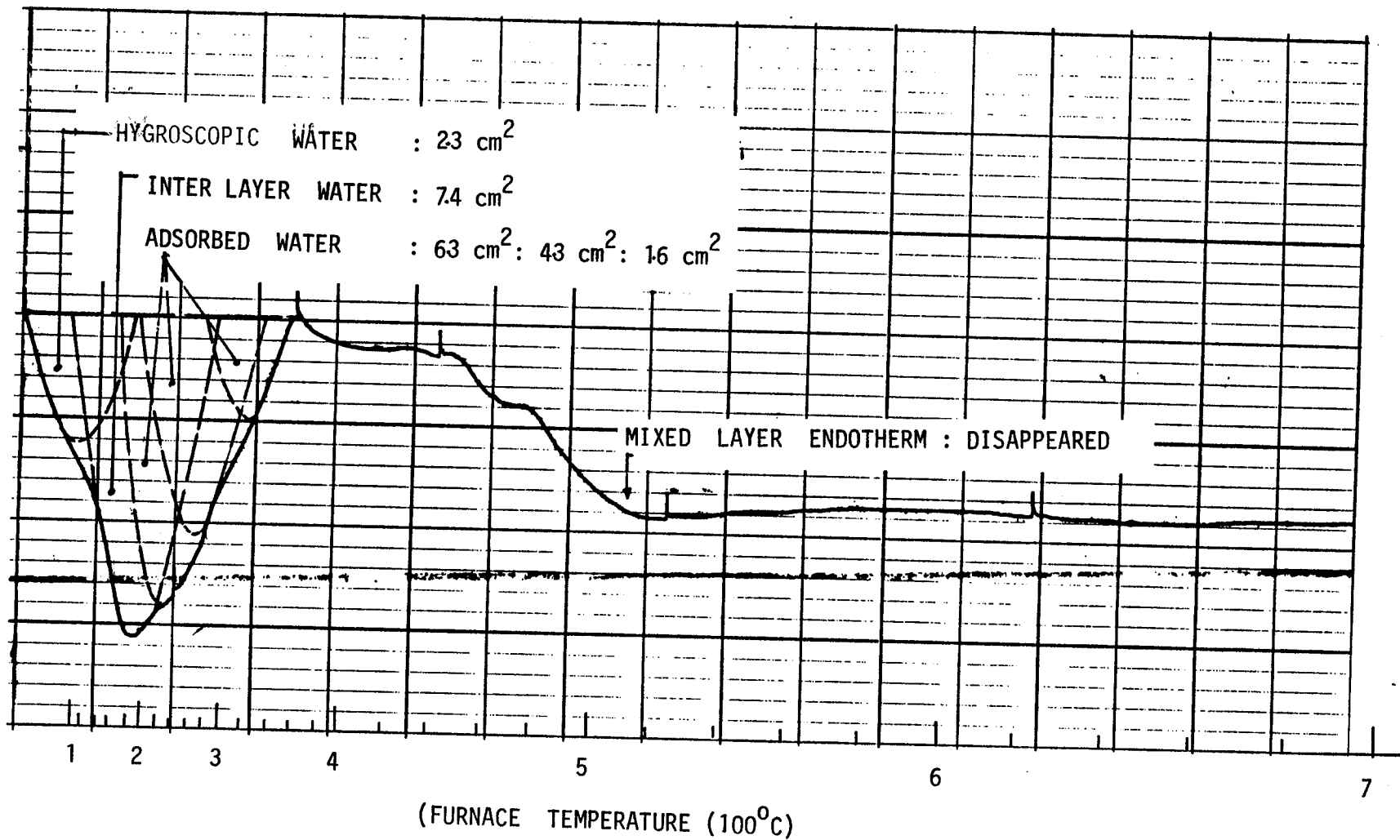


FIG. IV-2. DIFFERENTIAL THERMOGRAM FOR LAKE AGASSIZ CLAY TREATED WITH 4 % QUICKLIME.

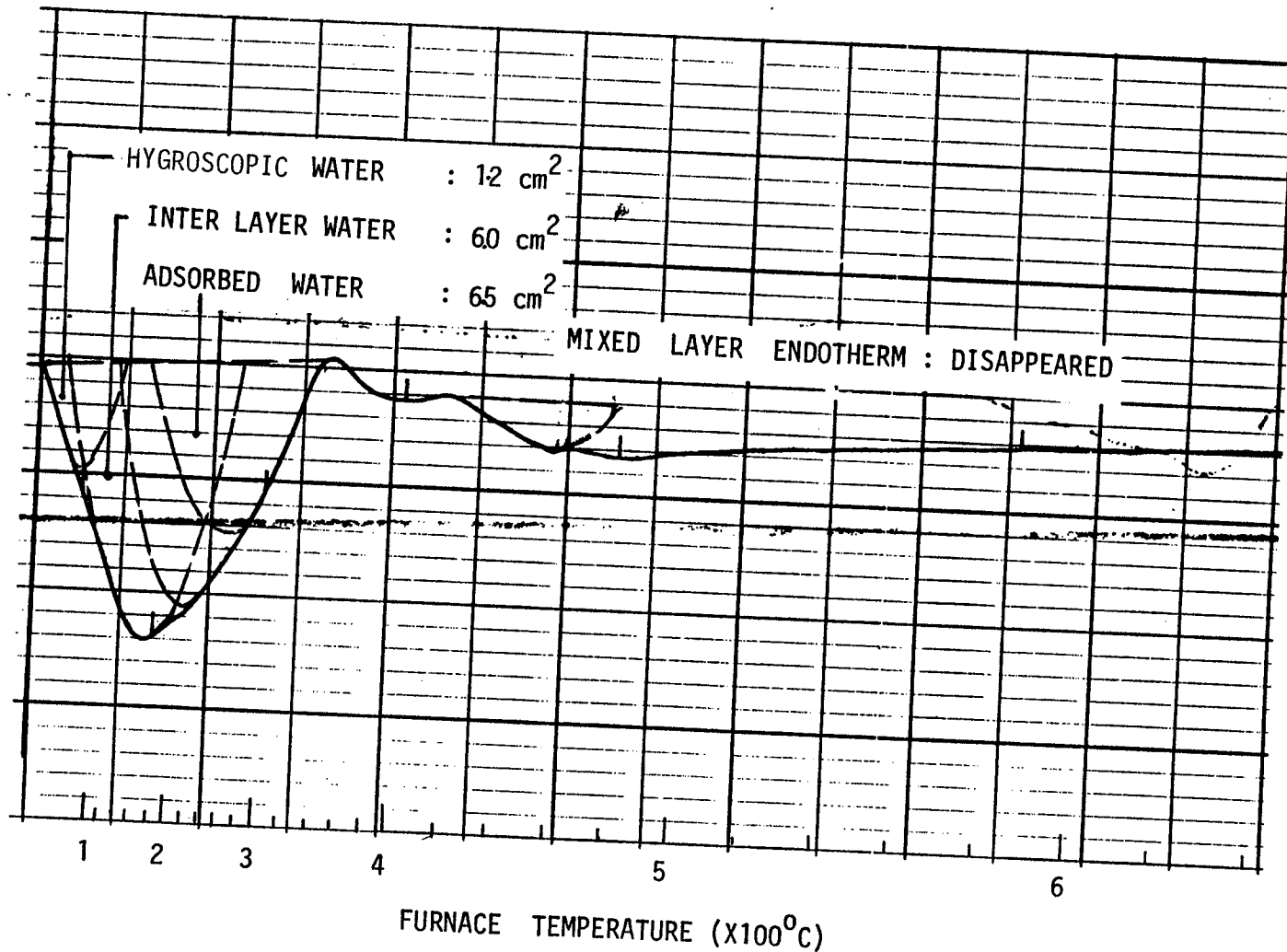


FIG. IV-3. DIFFERENTIAL THERMOGRAM FOR LAKE AGASSIZ CLAY TREATED WITH 8 % QUICKLIME.

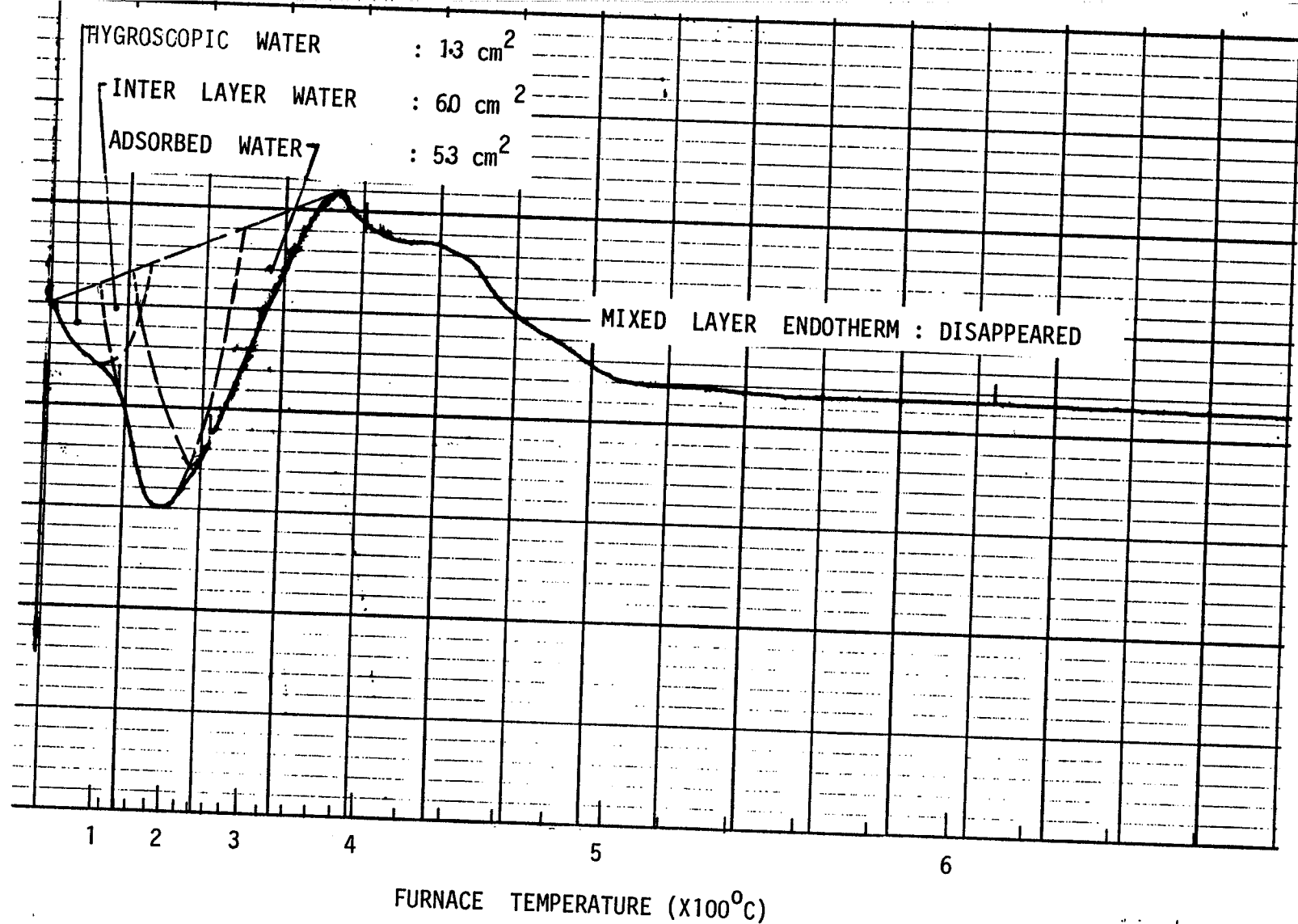


FIG. IV-4. DIFFERENTIAL THERMOGRAM FOR LAKE AGASSIZ CLAY TREATED WITH 10 % QUICKLIME.

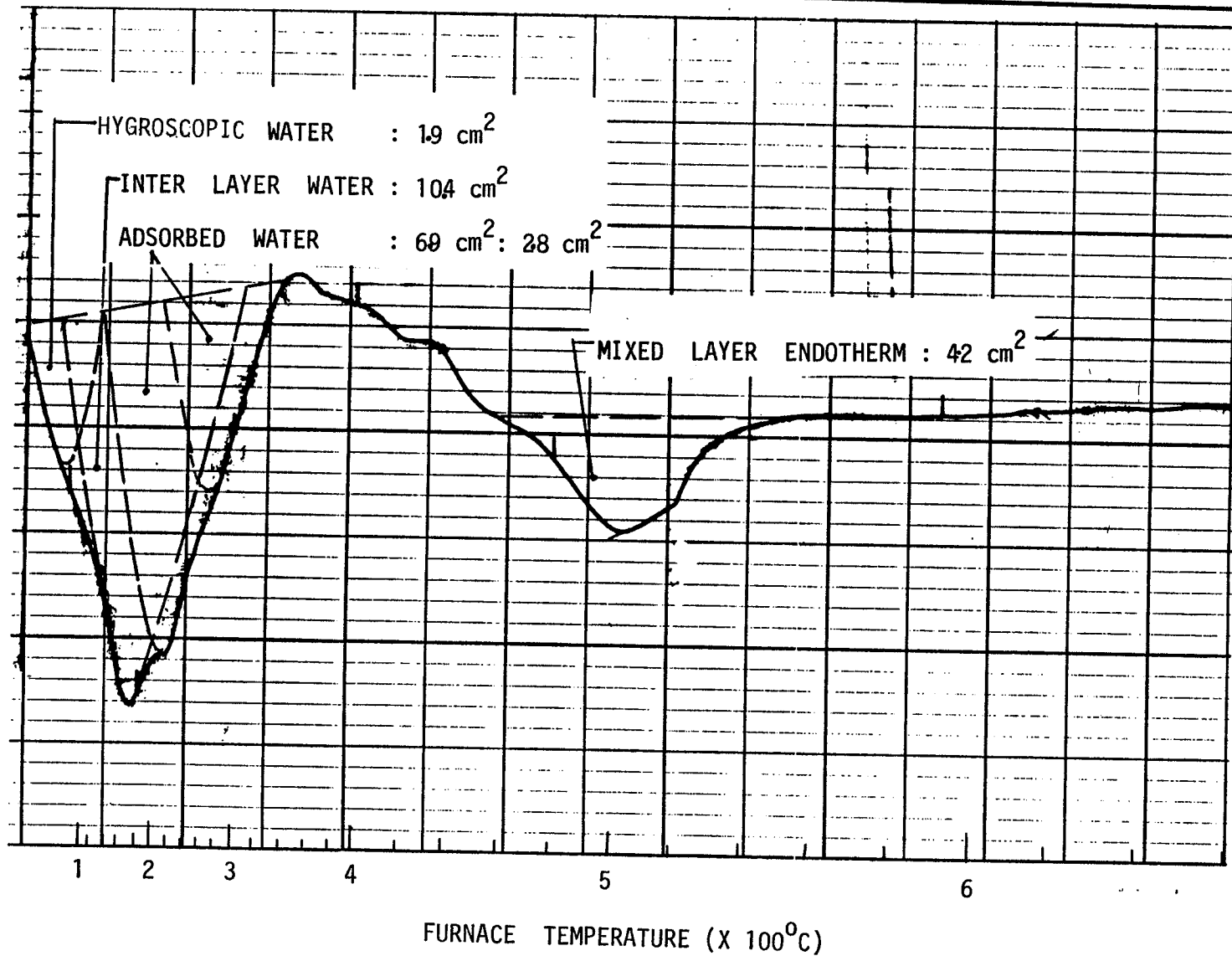


FIG. IV-5. DIFFERENTIAL THERMOGRAMS FOR LAKE REGINA CLAY (NATURAL)

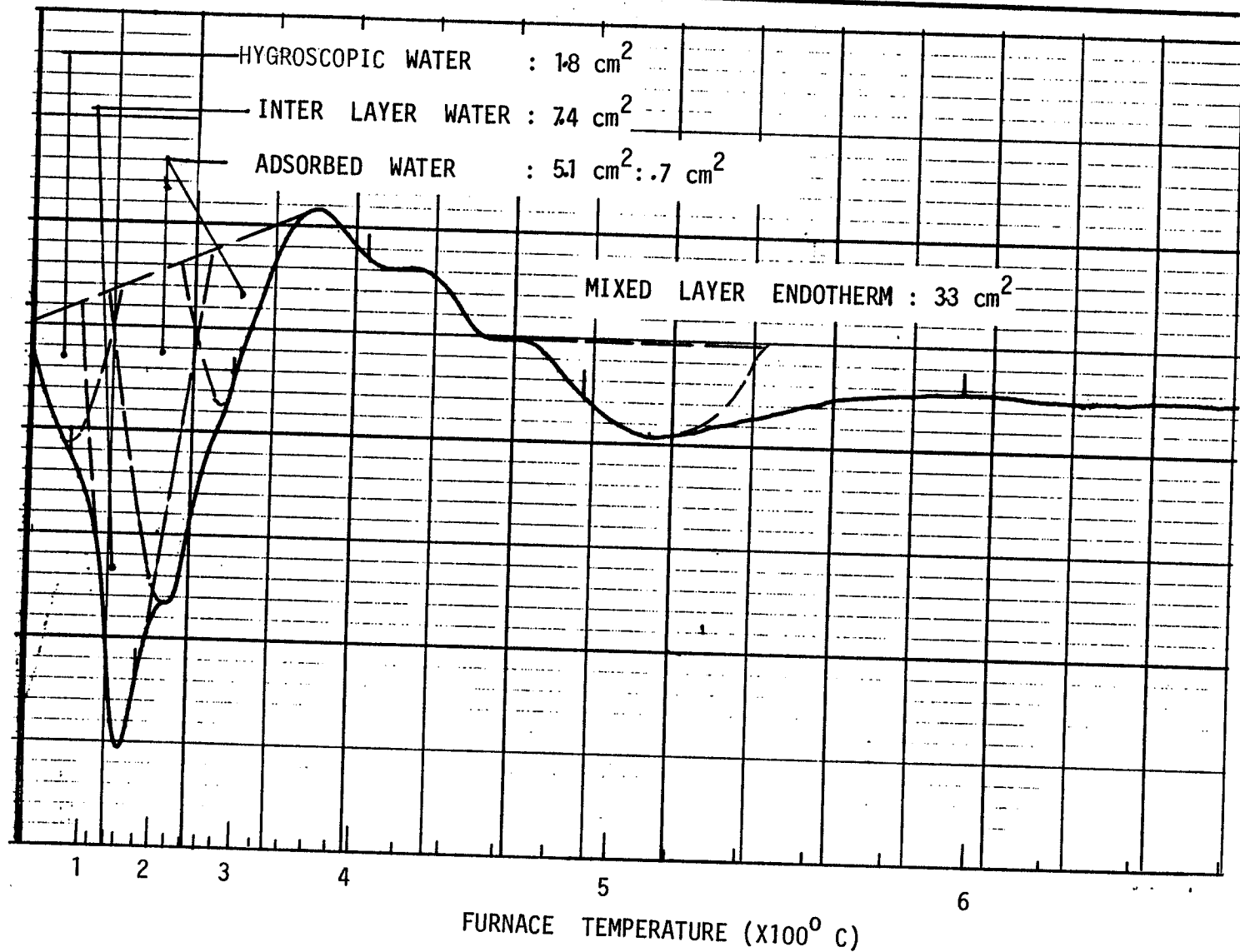


FIG. IV-6. DIFFERENTIAL THERMOGRAMS FOR LAKE REGINA CLAY TREATED WITH 2 % QUICKLIME.

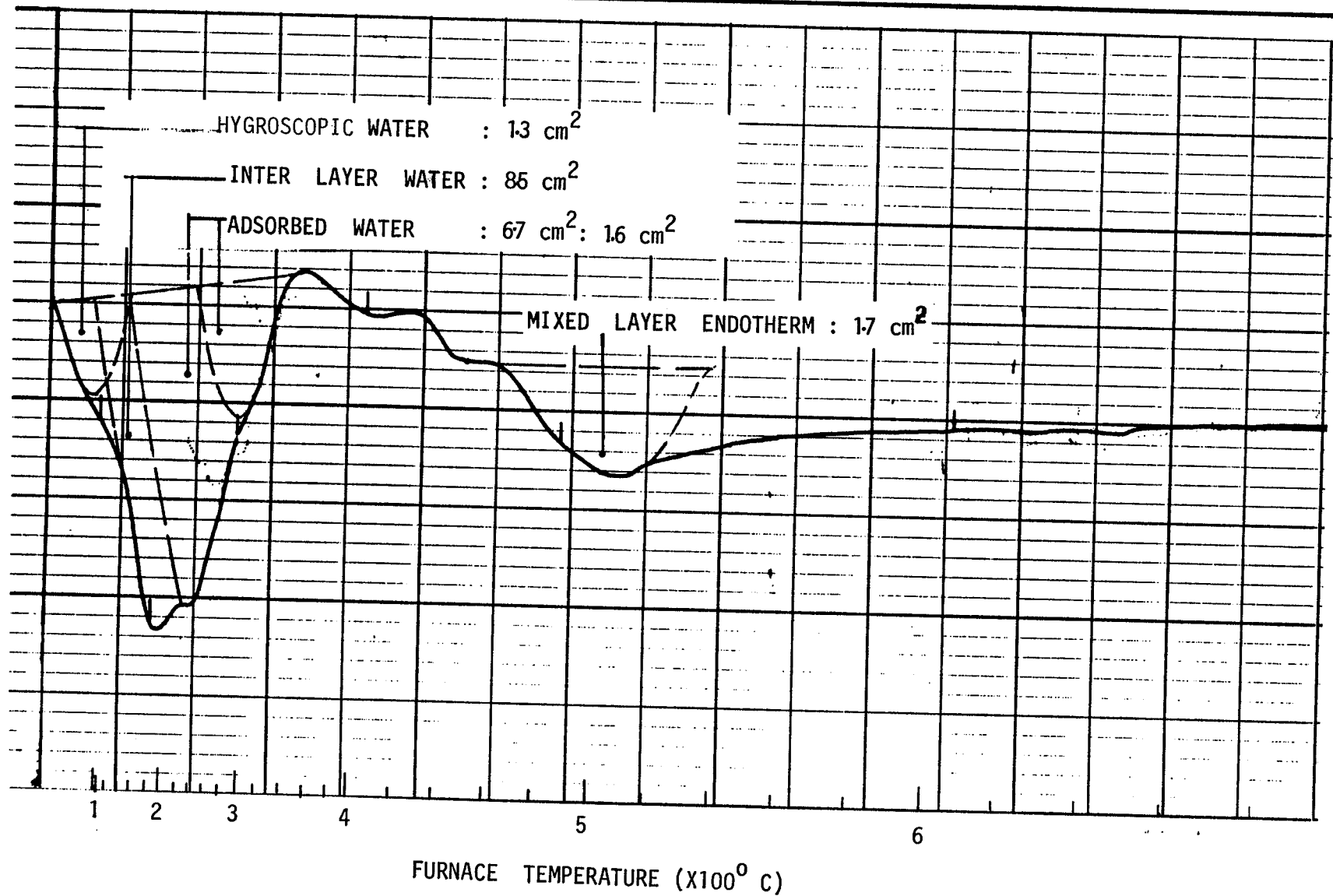


FIG. IV-7. DIFFERENTIAL THERMOGRAM FOR LAKE REGINA CLAY TREATED WITH 4 % QUICKLIME.

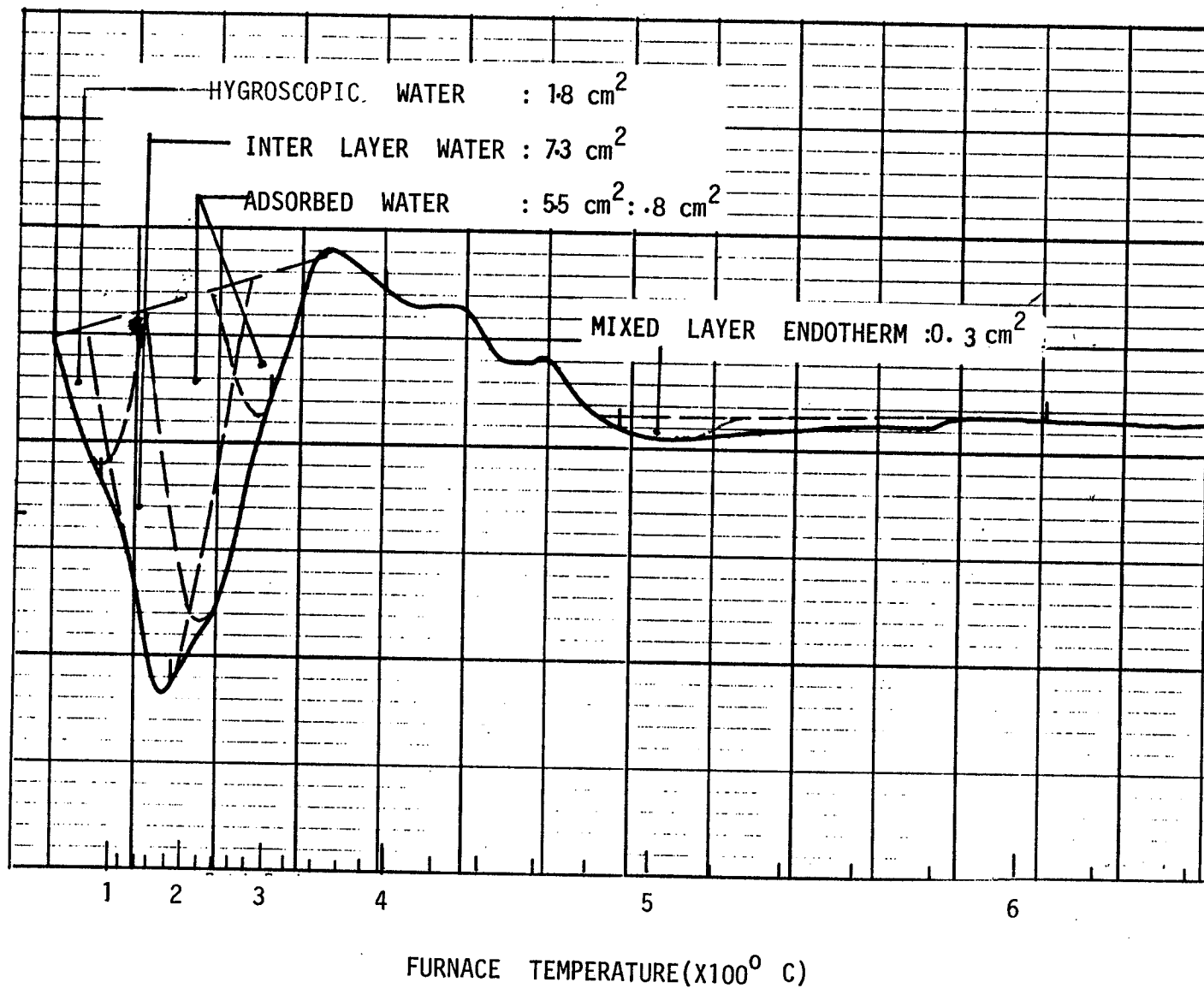


FIG. IV-8. DIFFERENTIAL THERMOGRAM FOR LAKE REGINA CLAY TREATED WITH 6 % QUICKLIME.

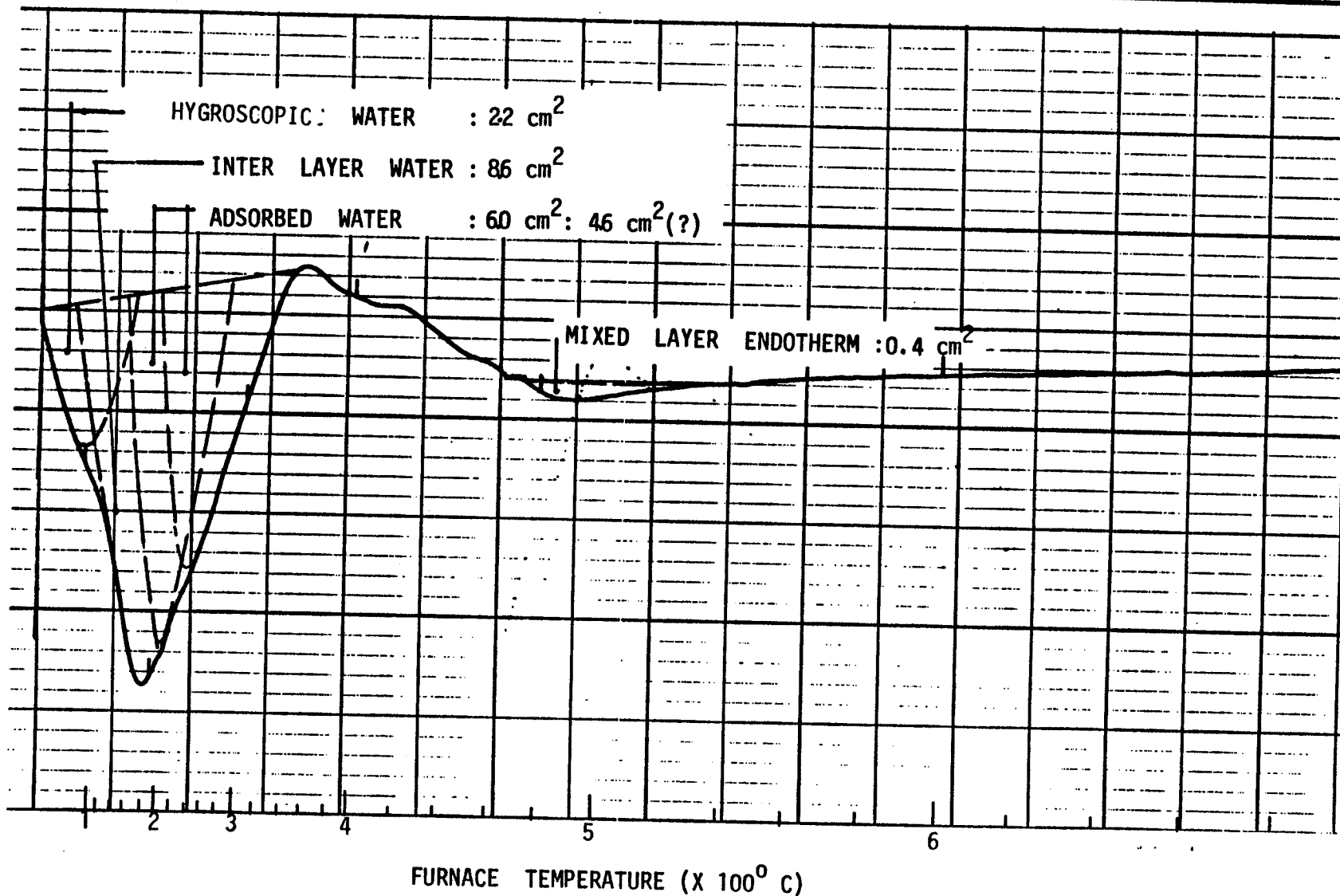


FIG. IV-9. DIFFERENTIAL THERMOGRAM FOR LAKE REGINA CLAY TREATED WITH 10 % QUICKLIME.

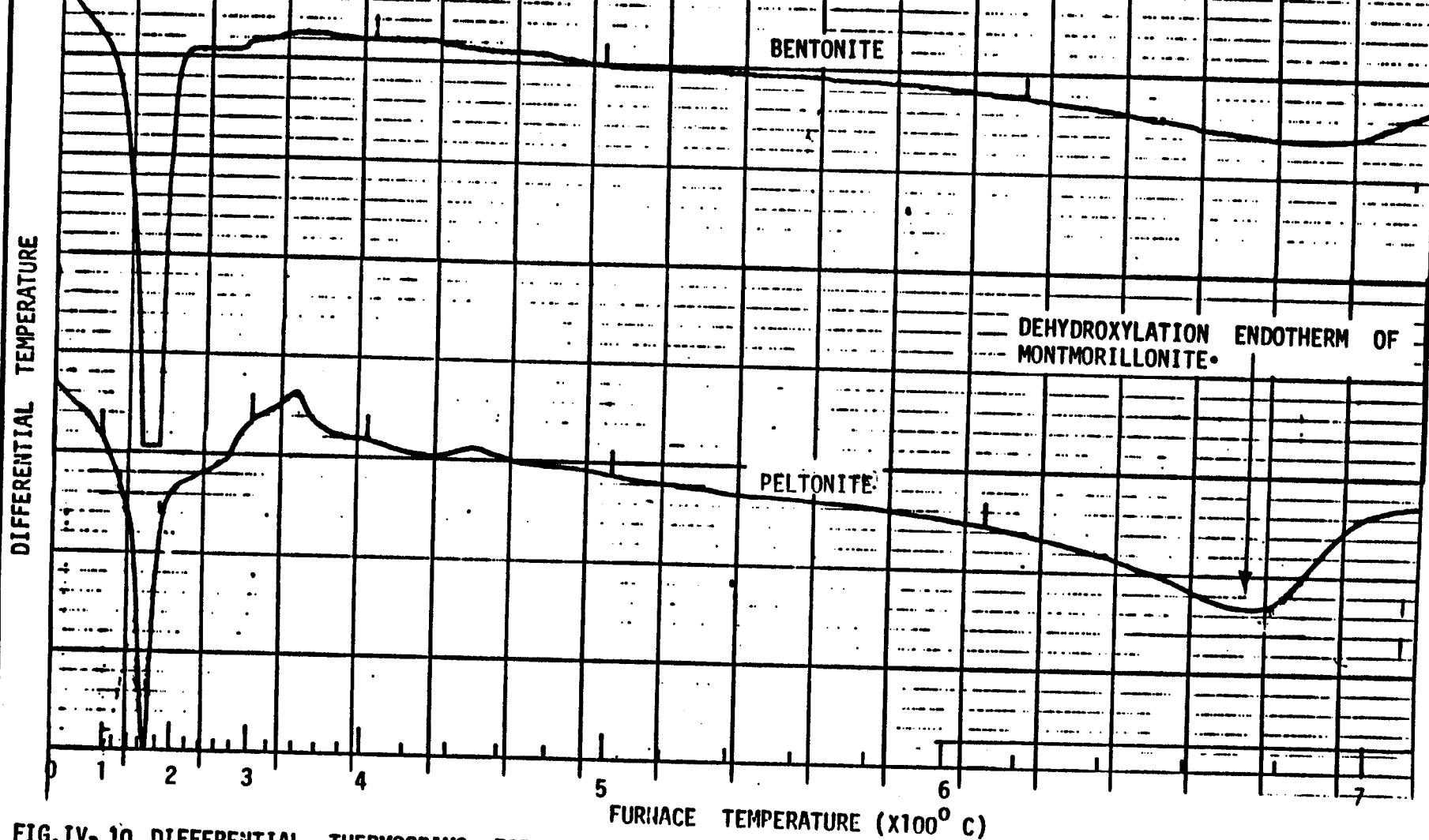


FIG. IV-10 DIFFERENTIAL THERMOGRAMS FOR A SAMPLE OF BENTONITE (TOP) AND A SAMPLE OF PELTONITE (BOTTOM)