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PAGE

COMPACTION OF CLAYS
AND
AN INVESTIGATION ON THE EFFECT OF SIZE AND AMOUNT OF COARSE AGGREGATE ON THE COMPACTION OF A BRICKEARTH

by

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PART I

COMPACTION OF CLAYS IN GENERAL

INTRODUCTION

The practice of transporting and placing earth materials to form fills or embankments for highway construction is older than the term " highway " itself. In fact the very name was adopted in ancient times to describe the more ambitious roads that had been built up above the surrounding terrain and hence were called "high ways to distinguish them from the casual paths or byways. For centuriesembankments were constructed by the most simple and dir methods, using hand barrows or horce drawn scrapers operating from side barrow pits. With the development of motorized equipment, longitudinal haul became more prevalent, moving material from the cuts and dumping into the appropriate low areas that need to be brought up to grade. The construction of fills by end dumping methods continued upto comperatively modern times, and in certain cases is still the only feasable method. No special efforts, however, were made to compact highway embankments because the roa surfaces were flexible enough to remain unharmed by the settleme of the fill. Until very recently railroad fills were also built up by loose dumping and allowed to settle under their own weight for several years before placement of high quality ballast. The settlement of uncompacted fills did not result in any serious inconveniences until the beginning of 20 th century, when the rapid development at the automobile created an increasingly high demand for hard surfaced roads. It soon became apparent that concrete roads on uncompacted fill were likely to break up, and that the surface of other types of high grade pavements had a tendency to become very uneven. The necessity for avoiding such undesirable conditions fostered the development of methods of soil compaction that would satisfy the requirements of both economy and efficiency.

On the other hand, by excavating soil masses and redepositin them without special care, the average porosity, permeability, an compressibility of the soil is increased, and the capasity to resist internal scour by water veins is greatly reduced. Therefo even in ancient times it was customary to compact fills to be use as dams or levees. A simultaneous increse in activity in 20th century in the field of earth dam construction provided an additi incentive for the development of compaction methods.

Attempts were made in California and elsewhere about 1925 to meet this problem by overloading the deeper fills, that is by building the fills temporarily above profile grade in an amount proportional to the depth of the fill. These "hump-backed" or "camel backed" fills presented rather a novel appearance in an ot wise conventional grade line but, with the well known perversity of inanimate things, most of the fills refused to settle where t greatest surcharge had been applied and all too often greatest subsidence occured at the ends of the fill near the point of jun tion with the existing ground. This effect accentuated the lum in the centre, so this expedient was soon discarded.

The California Division of Highways (U.S.A) Standard specif cations for 1927 included the requirement that all embarkments b constructed in layers and much much argument and controversy developed because the specification also required contractors to distribute haul equipment over the entire surface. About 1929, the division adopted the practice of requiring that the layers be thoroughly rolled in order to forestall settlements. This requirement immediately raised the question of control and demanded a means for checking the contractors' operations. The first work along this line was done by the California Division of Highways in 1929 when an extensive series of tests was conducted from which was developed field equipment and methods of compacting soil samples to determine optimum moisture requirements before construction and subsequently the relative compaction of the embankment. This procedure and equipment was adopted as standard in August 1929 and has been in use without substantial change to the present date. About 1933 the engineers of the Bureau of Water Works and Supply of the city of Los Angeles conducted a similar study, the results of which were described in a series of articles by R.R. Proctor,[1] field engineer of the bureau, published in several issues of the Engineering News Record. The compaction procedures were similar but the field apparatus was different. The Proctor method of compaction control became widely known and led to the widespread adoption of similar control test procedures such as the Standard AASHO method. With the tremendous expansion of military construction, particularly of airfields during war years, the Corps of Engineers stepped up the compaction requirements by adopting a compaction procedure known as the Modified AASHO which sets a higher standard of density, concluding that if embankments were to withstand the increasingly heavy loads and propeller vibrations of military planes a higher standard of construction compaction would have to be established. Thus some 30 years ago engineers began to talk about maximum density and optimum moisture content of soils and today many seems to believe that these terms express fundemental basic constants like the gravity constant or the boiling point of water. Table 1 lists the essential details of certain compaction test procedures used by various agencies under the designation shown.

The extensive studies, thus because of primary importance and the engineering practice revealed the fact that soils of different kind behaved differently in this respect. Thus the tendency to divide soils into groups based on various criterion became stronger. A most widely accepted classification is the M.I.T. Classification.

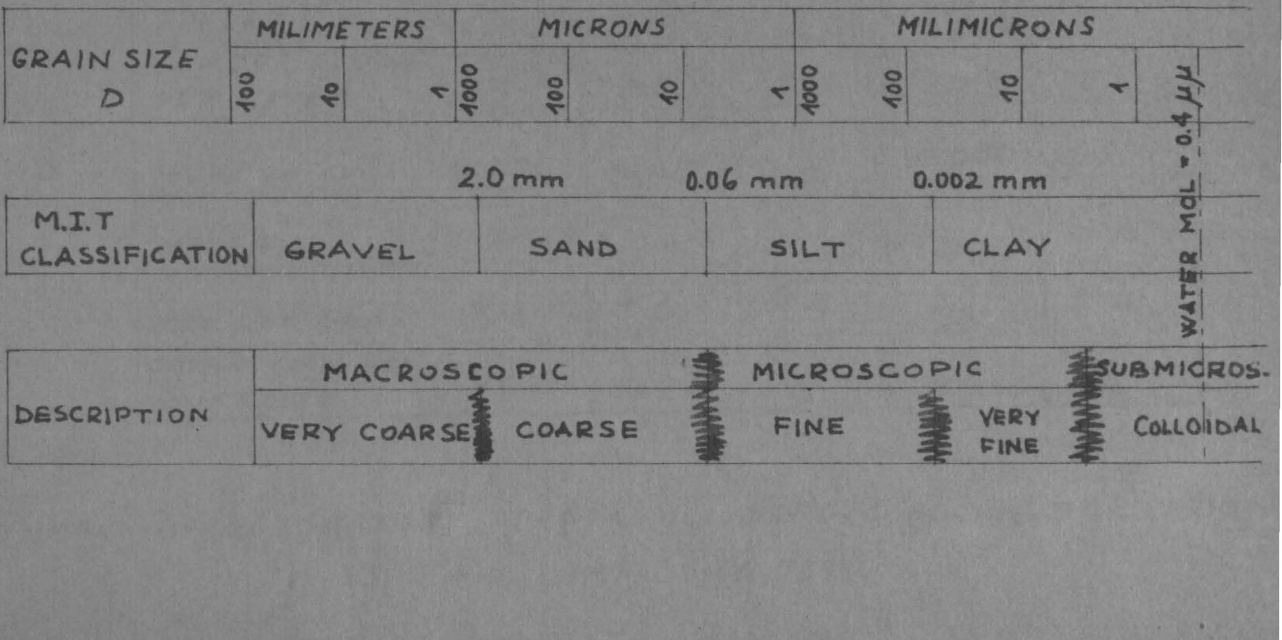


FIG-1

TEST IDEN.	STD. AASHO	BUREU REC	STD. PROCTOR	CALIF. IMPACT	MOD. AASHO
MOLD					
DIAMETER, in	4	4	4	3	4
HEIGHT, in	4 5/8	6	4 5/8	10_12	4 5/8
VOLUME, cu ft	1/30	1/20	1/30	Var.	1/30
TAMPER					
WEIGHT, lb	5.5	5.5	5.5	10.0	10.0
FREE DROP, in	12	18	12	18	18
FACE DIAM. in	2	2	2	2	2
FACE AREA sq in	3.1	3.1	3.1	3.1	3.1
LAYERS					
NUMBER, TOTAL	3	3	3	5	5
SURFACE AREA, EACH, sq in	12.6	12.6	12.6	7.1	12.6
COMPACTED THICKNESS EA. in	1 5/8	2 1/8	1 5/8	2 1/4	1
EFFORT					
TAMPER BLOWS PER LAYER	25	25	25	20	25
Ft_lb per cu ft	12 375	12 375		33 000	56 250
MATERIAL					
MAX. SIZE PASS.	#4	#4	#4	3/4"	#4
CORRECTION FOR OVERSIZE	NO	YES	NO	YES	NO

SUMMARY OF LABORATORY APPARATUS AND PROCEDURE FOR VARIOUS STANDARD COMPACTION TESTS

TABLE I

However any system of classification based on grain size alone is likely to be misleading, because the properties of the finest soil fractions depend on many factors other than the grain size. Hence on the basis of its grain size composition a natural soil can be designated by the names of its principal components such as "silty clay" or "sandy silt". Such designations are facilitated by the use of diagrams such as that adopted by the Public Roads Administration. (See fig. 2) In this diagram each of the three co-ordinate axes pertains to one of the grain size fractions, designated as sand, silt and clay. The chart is divided into regions to whic

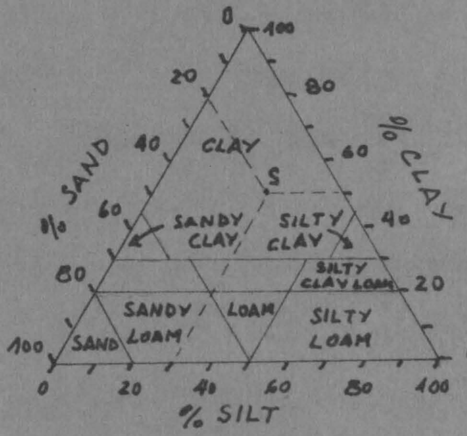


FIG-2

the names of soil types are assigned. The three co-ordinate of a point represent the persentages of the three fractions present in a given soil and determine the type to which the soil belongs. F example a mixed grained soil composed of 20% sand, 30% silt and 50% clay is classified as a clay. Therefore the major portion of clays consist of particles smaller than 0.002 m.m. It is obvious than, that the properties of this "very fine" fraction have a strong influence on the properties of the soil itself. Among the properties is the fact that the surface of each particle is negatively charged, which is responsible for the adhesion of water molecules and other cations that may be present to the particle surface. It was found[2] out that clays having different exchange cations showed different properties such as compressibility, coefficient of consolidation etc. A second property is that these soils consolidate and swell. Thirdly if a soil containing a large percentage of very fine particles is consolidated under a pressur of not more than 10 kg/cm2 the soil is likely to be plastic. And lastly they possess cohesion, or the capacity to resist shearing stress. It seems most likely that the cohesion is due not to direct molecular interaction between soil particles at the points of contact, but to the shearing strength of the adsorbed layers that seperate the grains at these points. This hypothesis is corroborated by the fact that, the cohesion of a sample of a give very fine soil fraction at a given water content depends to a large extent on the nature of the absorption complex. If the wat content of a very fine grained saturated soil is reduced by consolidation or surface evaporation, the volume of voids occupied by liquid water decreases, whereas the volume occupied by the adsorb substances remains unchanged. Therefore cohesion increases with decreasing water content.

Now let us take a look at the structure of clays. It was found out by the aid of the Electron Photomicrographs[3] that clay particles laminated crystalline structure whether they are flake-shaped like the kaolinite crystals or needle like crystals of halloysite. Thus clay particles can be spoken of as having an orientation. The attention of the reader is drawn to this propert which, as will be shown later, has a profound influence on some properties of clays. These particles are combined in a flocculent structure, which is a very complex arrangement of the particles

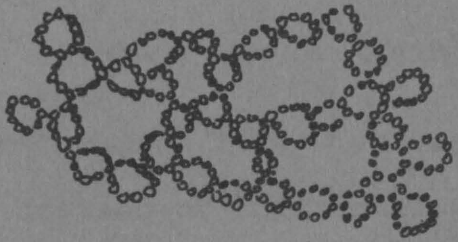


Fig. (3) Flocculent Structure

which have been water deposited. In this type of structure, particles of ultra-fine colloidal material are combined into groups or aggregations, called floc, by the attractive force of the electric charges of opposite sign on the surfaces of those particles. These aggregations were formed while the colloidal material was in suspention prior to deposition. When a sufficient number of individual particles adhered to one another, the resultant floc settled to the bottom under the influence of gravity, instead of remaining in suspention as is characteristic of individual clay particles. After settling, the aggregations joined wit one another to form minute arch-like boundary units over and over and around relatively large void spaces, arrangement being simila to that which is typical of honeycomb structure. Thus, flocculen structure may be described as honeycomb structure in which the

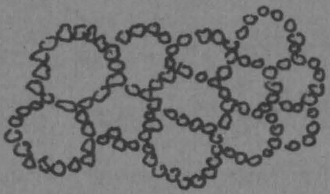


FIG- 4 HONEYCOMB STRUCTURE

units that form the arch like enclosures are aggregations of ultra-fine particles instead of individual particles. The remaining volume occupied by the soil bulk consists of the air voids and the soil water.

Among these two, water deserves special attention because all problems associated with the use, manipulation and treatment of soils arise from the effect upon soil properties of the presence of water. In essence at least, the major objective of the science of soil stabilization is to develop methods of treating soils so that they retain desired properties irrespective of the wetness o dryness of their environment. Alan S. Michaels[4] considers soil water in four types. See fig.(5)

1. Pore Water- This water has all the physical and chemical characteristics of normal liquid water. Such water is displacabl by normal hydrodynamic means provided it is not present in scaled-off pores, or in pores so small that caillary forces holding the water are greater than available hydrodynamic forces.

2. Solvation Water- This water is present in relatively thin layers around individual solid particles, and is held by polar,

electrostratic or ionic-hydration forces near the particle surfaces The water in these layers (probably not over 200 molecules thick) may be considerably denser and more viscious than ordinary liquid water, but is nevertheless mobile.

3. Adsorbed Water- This water is present in extremely thin (1-10 molecules) layers on both the exterior and interior surfaces. The forces holding these water molecules are extremely large, so this larger layer is inmovable by hydrodynamic mechanisms.

4. Structural Water- This is essentially not water at all, bu represents hydroxyl groups, which are integral parts of the solid crystal lattice. This "water" cannot be removed except by high temperature breakdown of the crystal structure.

The four types of water listed above are all very important in determining soil structure and properties, and it is difficult

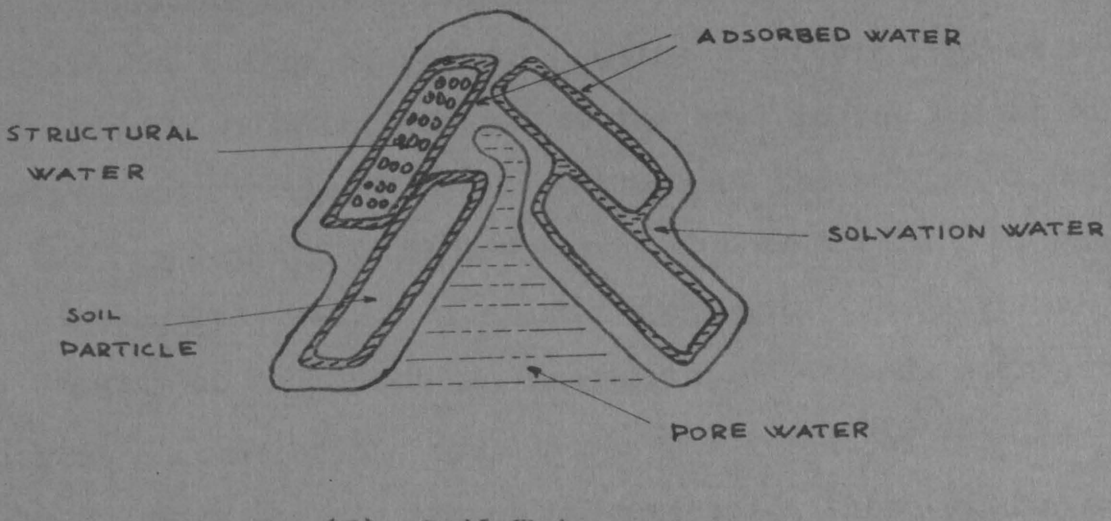
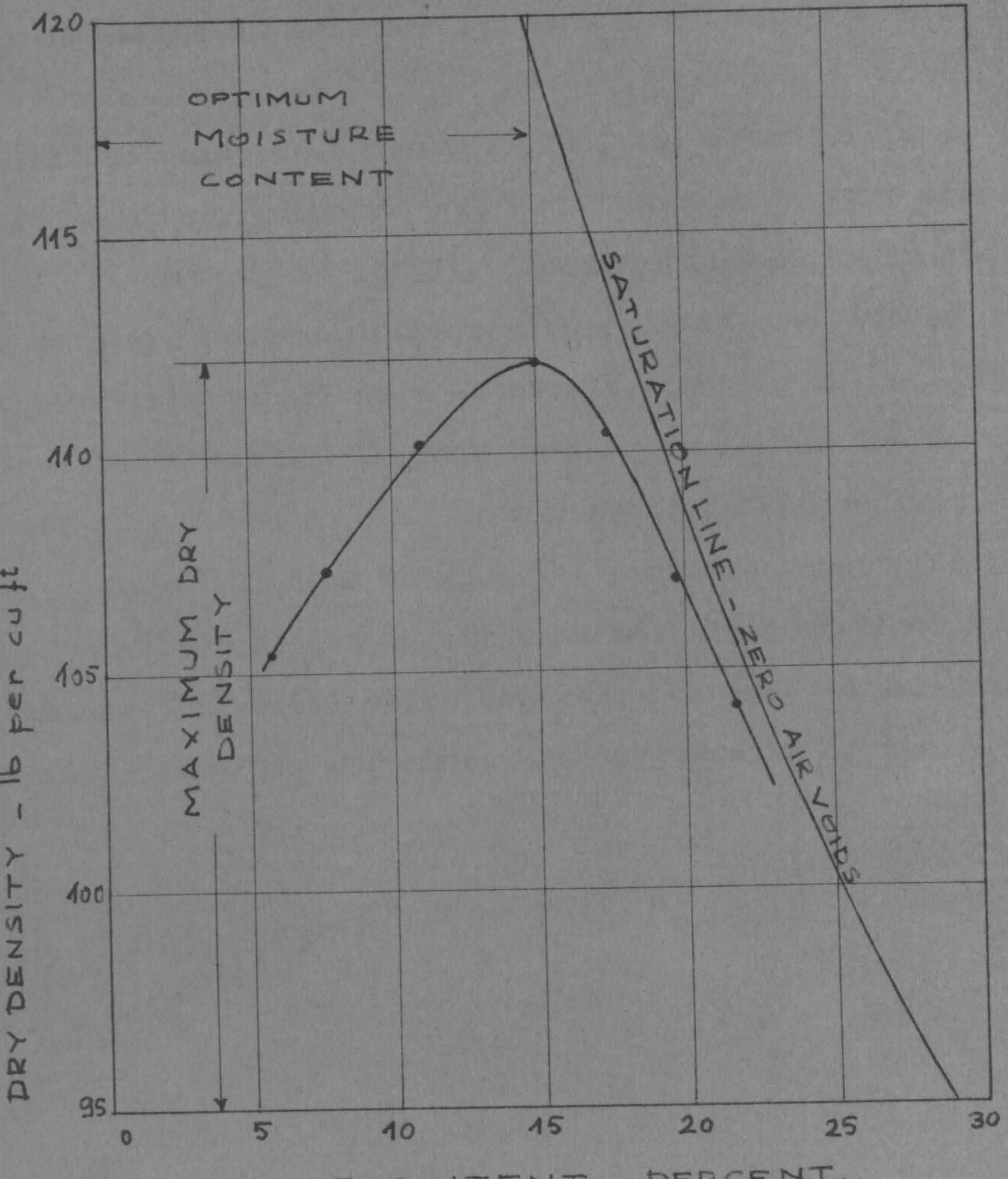


Fig.(5) Soil Water

to ascribe greater significance to the part played by any one. It is likely, however, that changes in physical properties of soils brought about by changes in water content are due to changes in pure or solvation waters. This assumption appears justifiable in light of the fact that adsorbed water and structural water are so tightly held by the soil solids that, little change in the amounts of these types present would be expected to occur during normal variations in temperature and humidity at the earths surface. Therefore in compaction process, which is the constrainising of soil particles to pack more closely together through a reduction in the air voids only, generally by mechanical means, our concern will be with the pore and solvation water. Let us point out now, a common source of confusion. Namely, that between compaction and consolidation. Consolidation is the packing closer of cohesive soils by the extraction of soil moisture under constant pressure.

Compaction is measured quantitatively in terms of the soil, which is the weight of the soil solids per cubic foot of the soil in bulk. The moisture content of the soil is the weight of moisture present expressed as a percentage of the weight of dry soil, and the dry density is thus determined from the bulk of the soil by deducting the weight of moisture present. The increase in the dry density of soil produced by compaction depends mainly on the moisture content of the soil and on the amount of compaction applied. With a given amount of compaction there exists for each soil a moisture content termed the "optimum moisture content" at which the maximum dry density is obtained (Fig. 6.). The behavio



RELATIONSHIP BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR CONSTANT AMOUNT OF COMPACTION

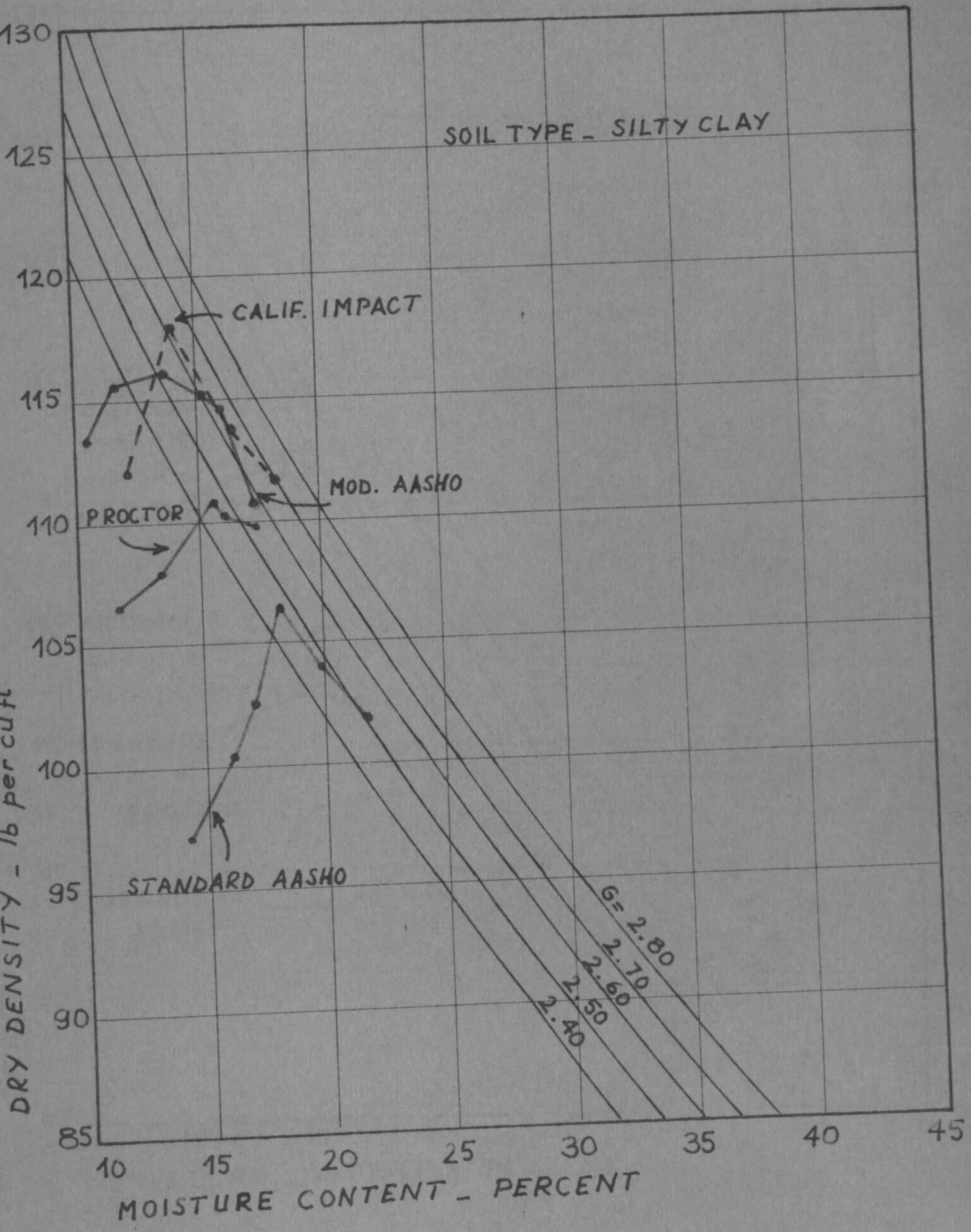
FIG - 6

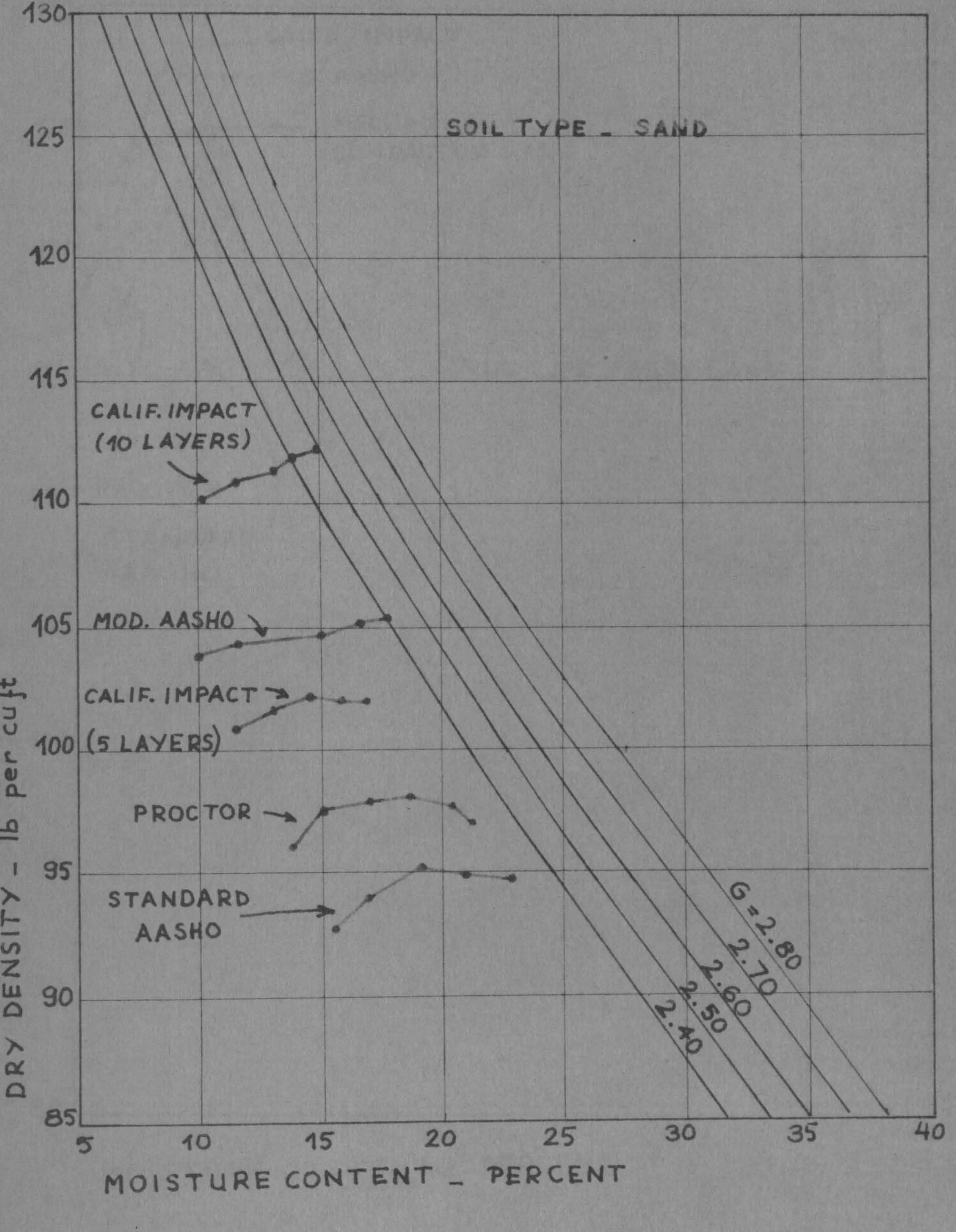
of soil at different moisture contents has been explained by Hogentogler[5] as follows:

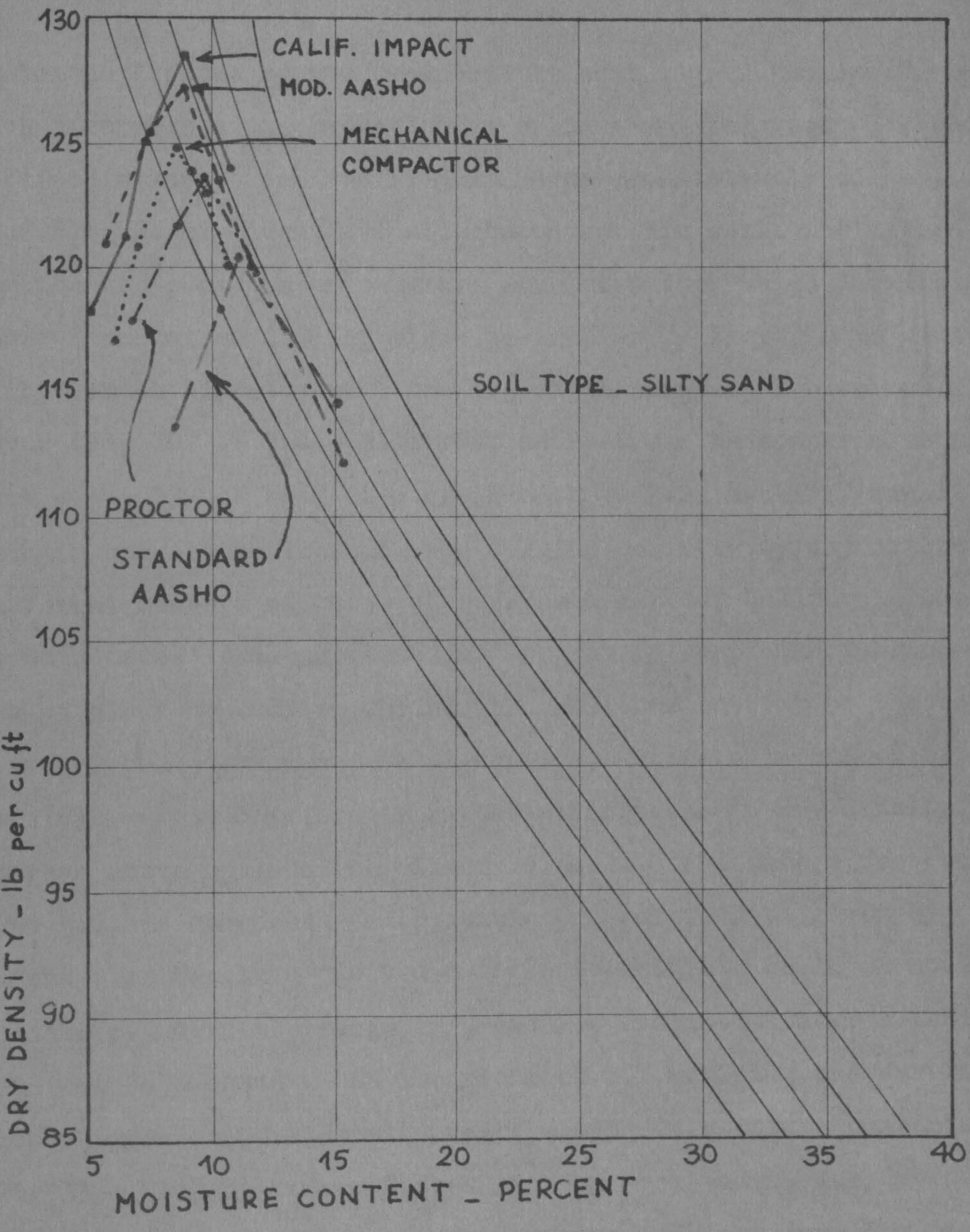
When the moisture content is low, the water films around the particles are very adhesive and thus the soil is very difficult to compress, giving way to low dry densities and high air contents. As the moisture content increases these adhesive forces decrease and water starts to act as a lubricant, causing the soil to soften and become more workable. This results in higher dry densities and lower air contents. As the air content becomes less, the wate and air combination tend to keep the particles apart and prevent any appreciable decrease in air content. This is the limit of lubrication. The total voids, however, continue to increase with the moisture content, and hence the dry density of the soil falls.

COMPACTION IN LABORATORY

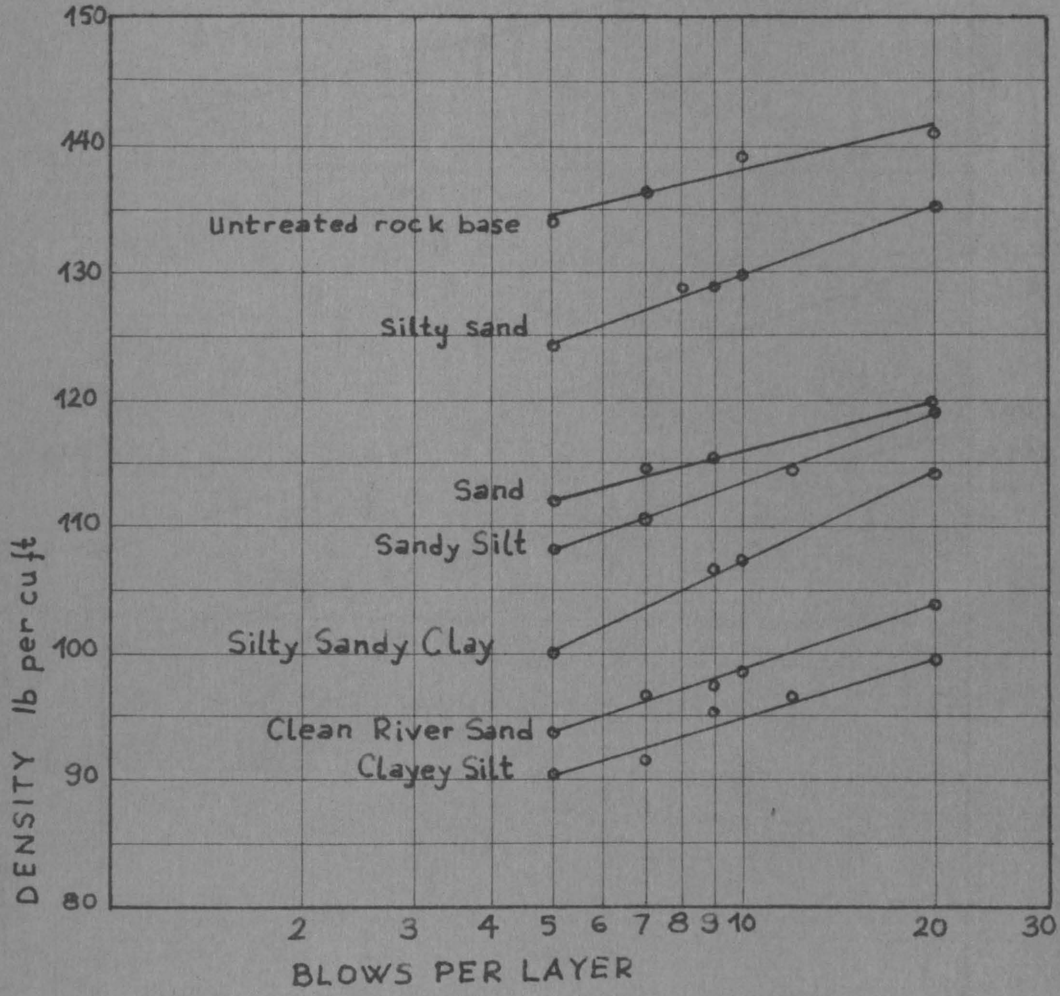
Moisture content of a soil having so profound an effect on the compaction, relation between the moisture content and the dry density has been sought, and procedures have been standardized by various agencies. These were given in Table 1. Looking at figures 7, 8, and 9 it is clearly evident that there are marked differences in the maximum dry weight per cubic foot obtained by these different "standard" laboratory procedures. It is also evident that the devices giving the higher density generally indicate a lower percentage of moisture as optimum. These figures then demonstrate a fact which is well known to many engineers, namely, that as the compactive effort is increased the moisture content needed to produce maximum density is generally reduced. The opinion, howeve expressed by many engineers as to the direct proportionality between the amount of compaction and the energy or force used in the process is erroneous. The same amount of energy may produce different degrees of compaction. For example the Modified AASHO develops about 56250 ft-lb per cu-ft of soil and California Impact develops 33000 yet the latter produces a greater density on many soils. By referring to figures 7 to 9 it will be noted that ther is a fairly consistent order in the maximum density values produc in a soil by the several compaction methods under consideration. First it is evident that in all cases Standard AASHO produces th lowest dry weight per cubic foot and the optimum moisture content is higher than it is for the other methods. In the same relative


SOIL TYPE - SILTY CLAY
CALIF. IMPACT
MOD. AASHO
PROCTOR
STANDARD AASHO
G= 2.80
2.70
2.60
2.50
2.40
DRY DENSITY - lb per cu ft
130
125
120
115
110
105
100
95
90
85
10
15
20
25
30
35
40
45
MOISTURE CONTENT - PERCENT

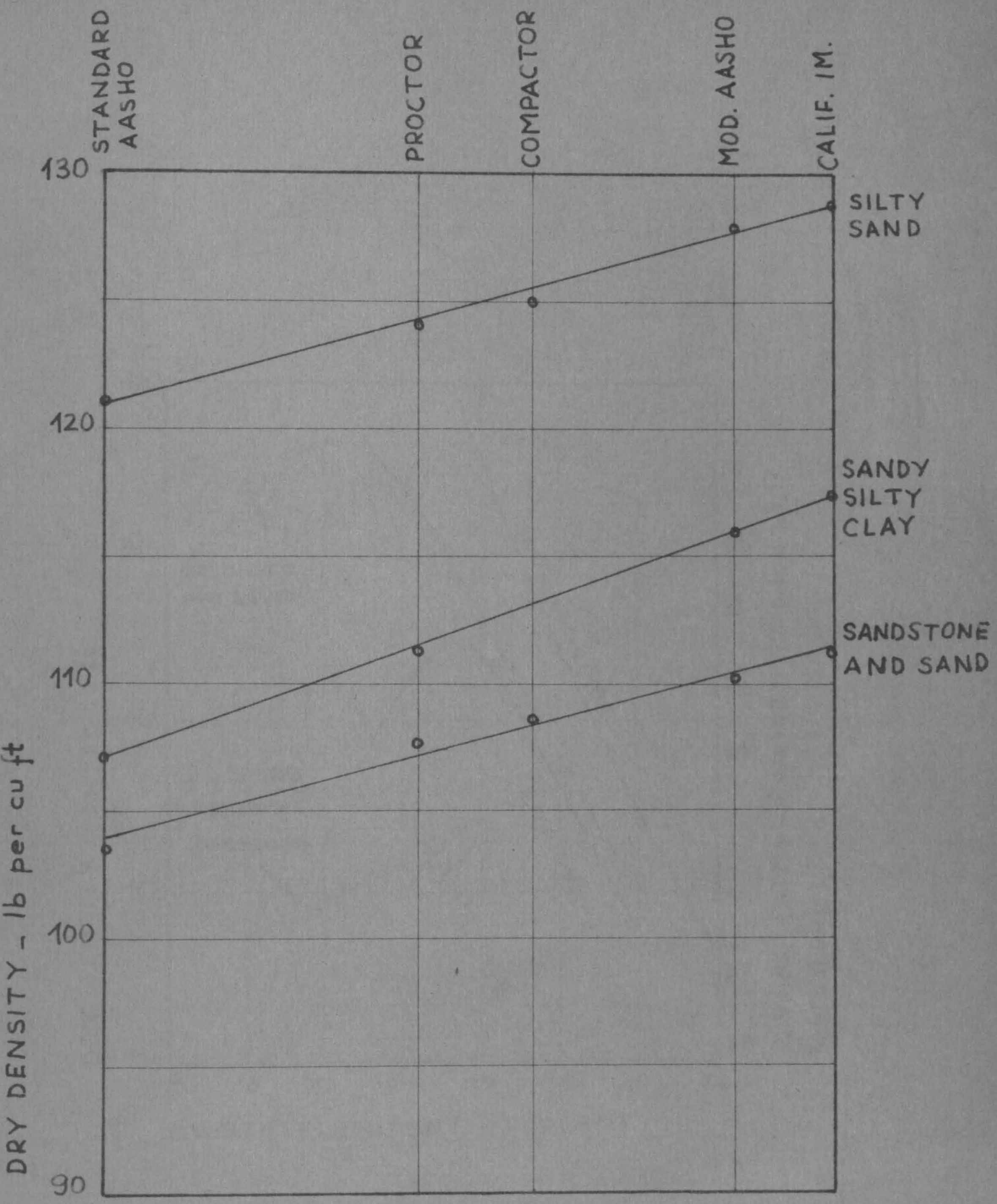

SOIL TYPE _ SAND
CALIF. IMPACT (10 LAYERS)
MOD. AASHO
CALIF. IMPACT (5 LAYERS)
PROCTOR
STANDARD AASHO
G = 2.80
2.70
2.60
2.50
2.40
130
125
120
115
110
105
100
95
90
85
5
10
15
20
25
30
35
40
DRY DENSITY - lb per cu ft
MOISTURE CONTENT _ PERCENT


CALIF. IMPACT
MOD. AASHO
MECHANICAL COMPACTOR
SOIL TYPE _ SILTY SAND
PROCTOR
STANDARD AASHO
DRY DENSITY _ lb per cu ft
130
125
120
115
110
105
100
95
90
85
5
10
15
20
25
30
35
40
MOISTURE CONTENT _ PERCENT

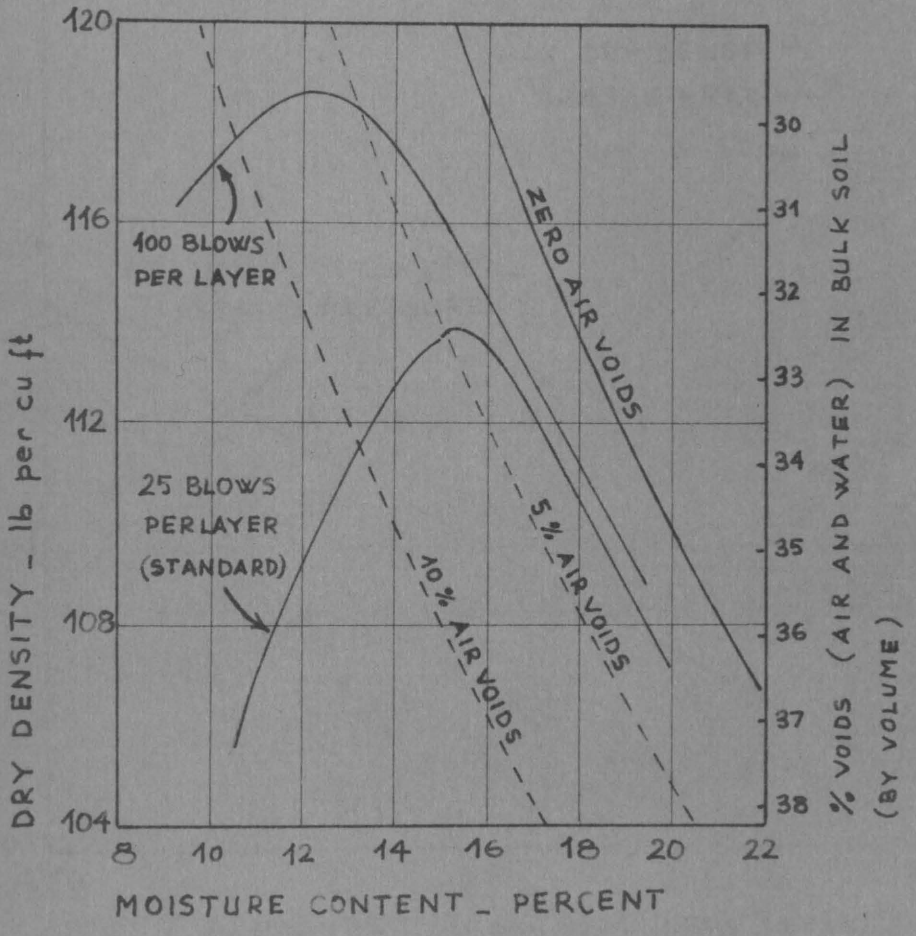
scale the Proctor method produces the next higher "maximum" density with a corresponding reduction in optimum moisture content, but California Impact and Modified AASHO are consistently higher, producing nearly identical weights on certain soils and alternating for the top position on others. A related fact which parallels the above observation is that there is an orderly increase in density which varies directly with the log of the number of blows per layer (Fig 10) . These different methods can be compared on the same datum by the help of a graph such as F.N. Hveem's[6] (Fig 11). This is done by duplicating the density achieved by any method by different numbers of blows of reference method, California Impact in this case. This increase in dry density due to increasing number of blows depends on whether the moisture content of the soil is above or below optimum. Below optimum, increase in number of blows per layer makes way for a substantial increase in dry density (Fig-12) whereas above optimum the effect is small. The same effect occurs when soil is compacted under static pressure. Data given by Hogentogler has been plotted using a logarithmic scale of moulding pressures. Over the range of pressures examined linear relations were obtained between the logarithm of the moulding pressure and both the maximum dry density and the optimum moisture content. (Fig 13) Had the range of pressures been extended, the dry density must have approached some limiting value. The maximum dry density that can be obtained either in laboratory tests or in the field with any soil depends on its type and varies from about 140 lb. per



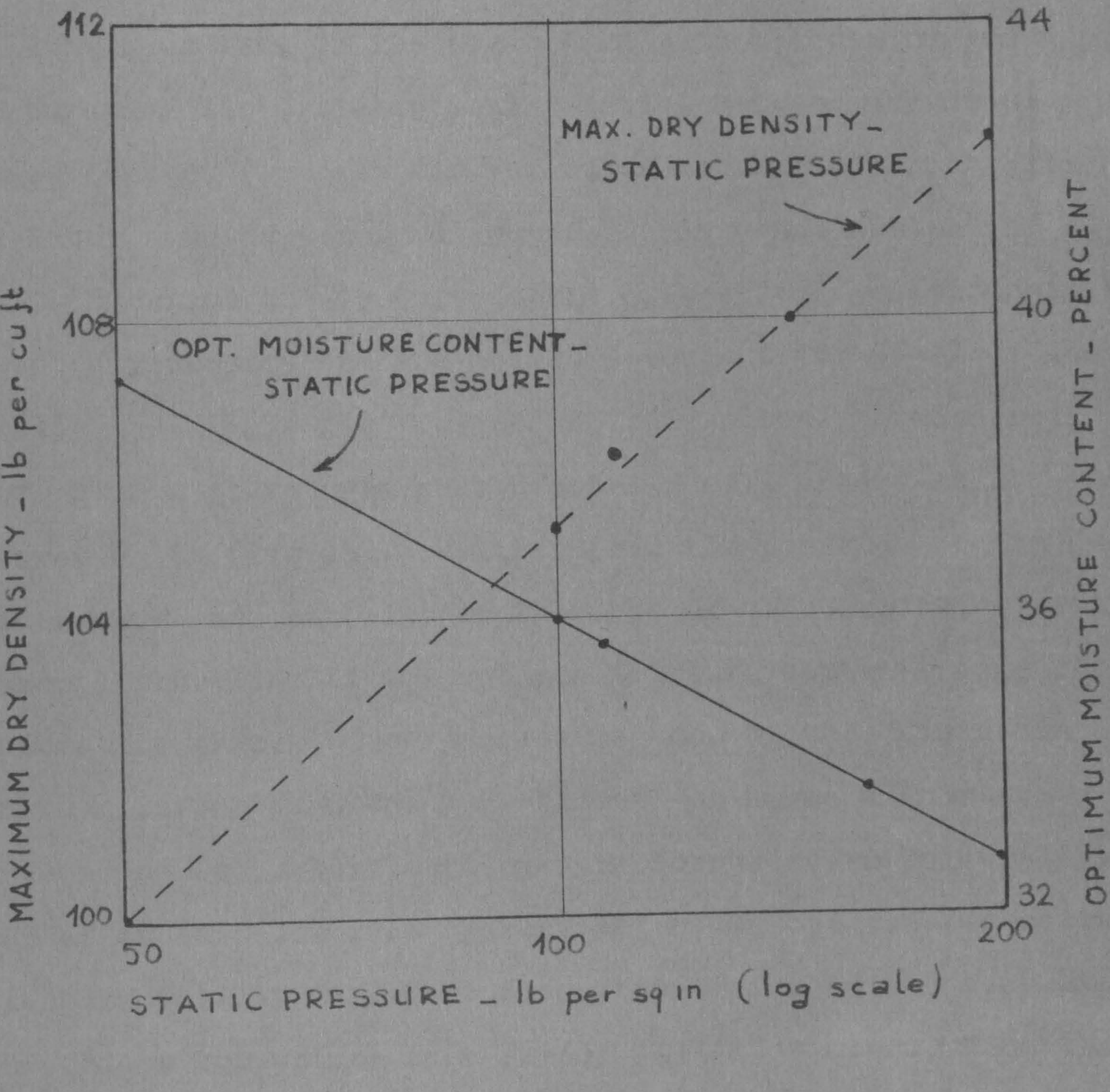
RELATION BETWEEN DENSITY AND NO. OF BLOWS ON EACH OF 5 LAYERS IN CALIFORNIA IMPACT METHOD.



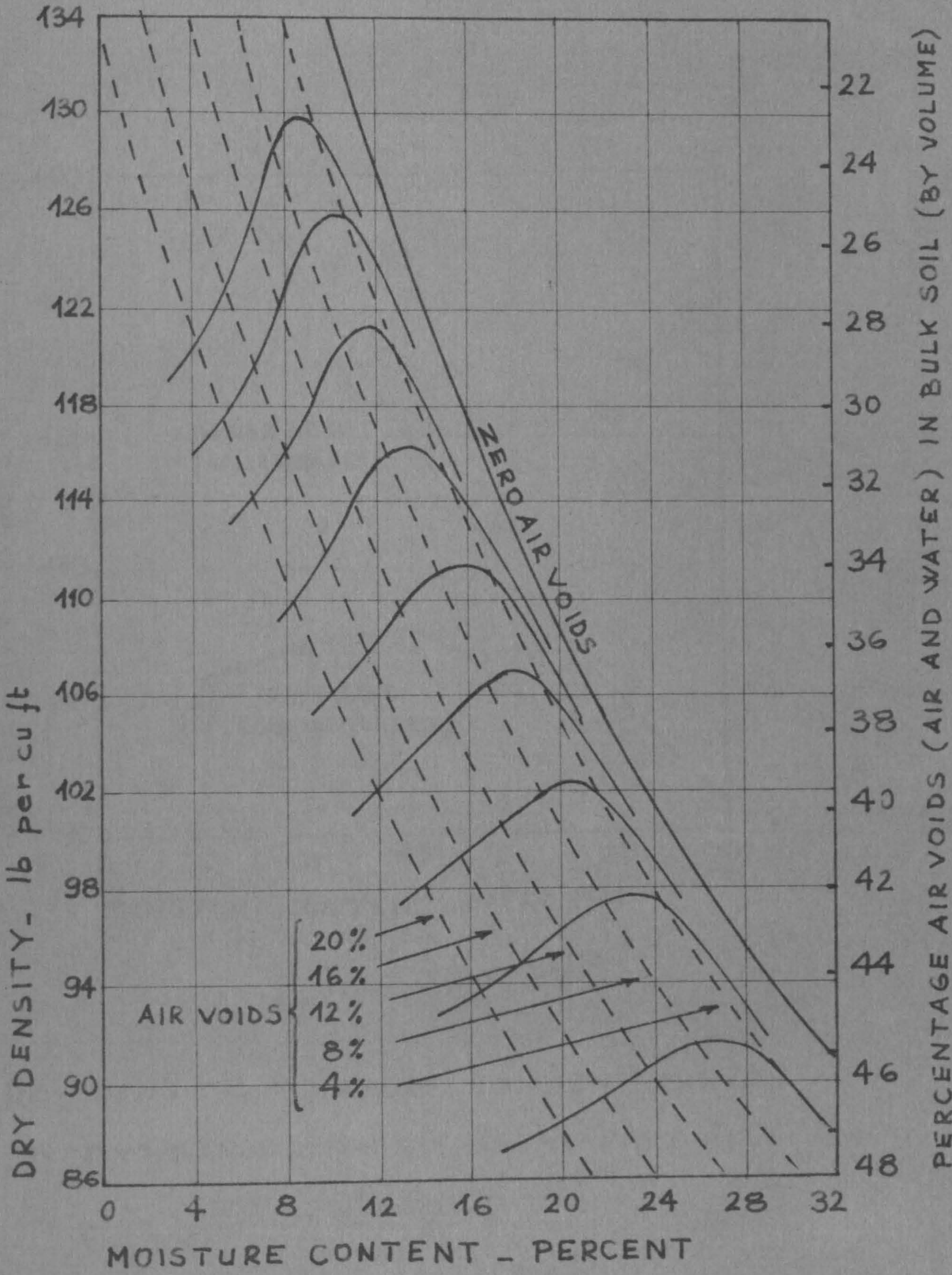
COMPARISON BETWEEN DIFFERENT LABORATORY METHODS OF COMPACTION



EFFECT OF DIFFERENT AMOUNTS OF COMPACTION ON DRY DENSITY OF SANDY CLAY SOIL

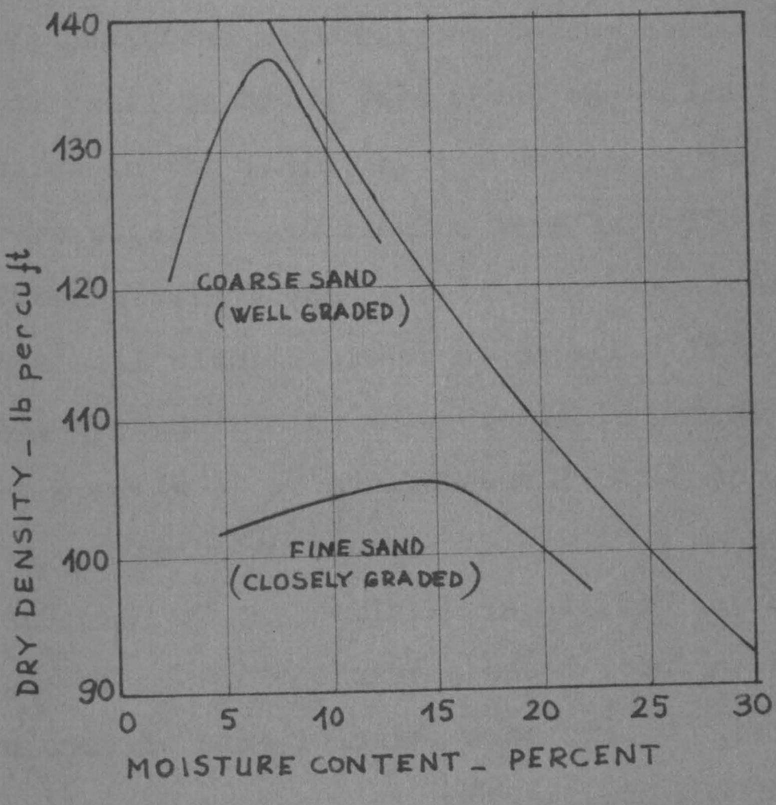


EFFECT OF STATIC PRESSURE ON MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT (BY VOLUME) OF SOIL cubic foot for a well graded gravel to about 90 lb per cubic foot for a heavy clay. Typical dry density vs moisture content curves obtained with a laboratory compaction test for soils having maximu dry densities differing by 5 lb. per cubic feet is shown in (Fig 1 It will be seen that on the right hand side all the curves approac the saturation line, theoretical relation between dry density and moisture content for zero air voids, an account of which will be given later, andthe peaks all occur at air voids content of appro imately 5%, which in its turn is the theoretical relationship between the dry density and the moisture content for 5% air voids. Generally speaking a flat curve denotes a closely graded soil and a curve with a pronounced peak denotes a well graded soil. This illustrated in (Fig 15). Furthermore, the position of the densi moisture curve depends also on the type of the compactive effort. For example the line of the optimum moisture contents come closer to saturation where the compaction is done by the "Miniature Compaction Device"of Stanley D. Wilson of Harward University and the "Kneading Compactor" developed in Northwestern University[7]. T latter of the two deserves special attention because its charact tistics are totally different from that of the impact compactors, which achieve compaction by a freely falling tamper. Thus these compactors are far from representing field conditions where the usual compaction plant is the smooth wheeled rollers, pneumatic tyred rollers or sheepsfoot rollers, and as will be shown later results obtained from such compactors do not compare with field



AVERAGE RELATIONSHIP BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR SOILS HAVING MAXIMUM DRY DENSITIES DIFFERING BY 5 lb per cu ft.

FIG - 14



DRY DENSITY_ MOISTURE CONTENT CURVES
FOR TWO SANDS WITH DIFFERENT PARTICLE
SIZE DISTRIBUTIONS

FIG. 15

ta. The Kneading Compaction was developed to answer this need, amely that the field compaction curves should entirely be duplica- ed by the laboratory results. In particular, an improved labora- ory compaction method should produce curves which have the same elative positions with respect to the zero air voids curve as the ield curves. To bring this about the characteristic of the roller ompaction in the field, by either sheepsfoot or rubber tyred rol- ers were observed and results were as follows:

The sheepsfoot of the roller or the surface of the tyre comes nto the soil with little or no impact. It pushes on the soil with definite pressure for an appreciable period of time, since the rollers operate at speeds between 2 and 6 mph, then the pressure is removed and comes onto the soil again at an adjacent location in the next pass of the roller. In addition, the rotation of the roller drum or the tyre causes a small rotating "kneading" or shoving action and, in some rollers, a rocking of the contact surface as it adjusts itself to the soil surface. In all cases the intensity of pressure exerted by the roller contact surfaces on the soil must be sufficient to cause relatively large shear deformations in the soil,since rapid compaction can be achieved only by shear deformations in which the relative positions of soil grains are changed with respect to eachother. In other words, the stresses on the contact surfaces must exceed the bearing capacity of the uncompacted soil and, after several passes, should be less than the ultimate bearing capacity of the partially compacted soil but large enough to cause effective shear deformations.

On the basis of this analysis of the characteristics of field compaction, P.C.Rutledge and J.O.Osterberg[7] at Northwestern University in 1946 set up the following desirable characteristics for a laboratory soil compaction device:

1. The compacting foot should not apply impact to the soil.

2. The compacting foot should apply a controlled pressure to the soil for a controlled period of time, and variation of both th contact pressure and the contact time over reasonable ranges corresponding to those anticipated in the field should be possible.

3. The compacting foot should cover a moderately small portion of the surface area of the soil sample being compacted so tha shear deformations involving lateral flow of the soil could take place.

4. The operation of the device should be as nearly automatic as possible.

The compaction device to meet these characteristics was designed by J.O. Osterberg and constructed by the Northwestern University Soil Mechanics Laboratory in 1946-1947. A photo-copy of this device is shown in (Fig 16). The results have indeed been satisfactory, such that:

1. The position of the optimum line can be shifted by changing the time increment for the compacting foot. Increasing the time increment causes the optimum line to move closer to the zero air-voids curve.

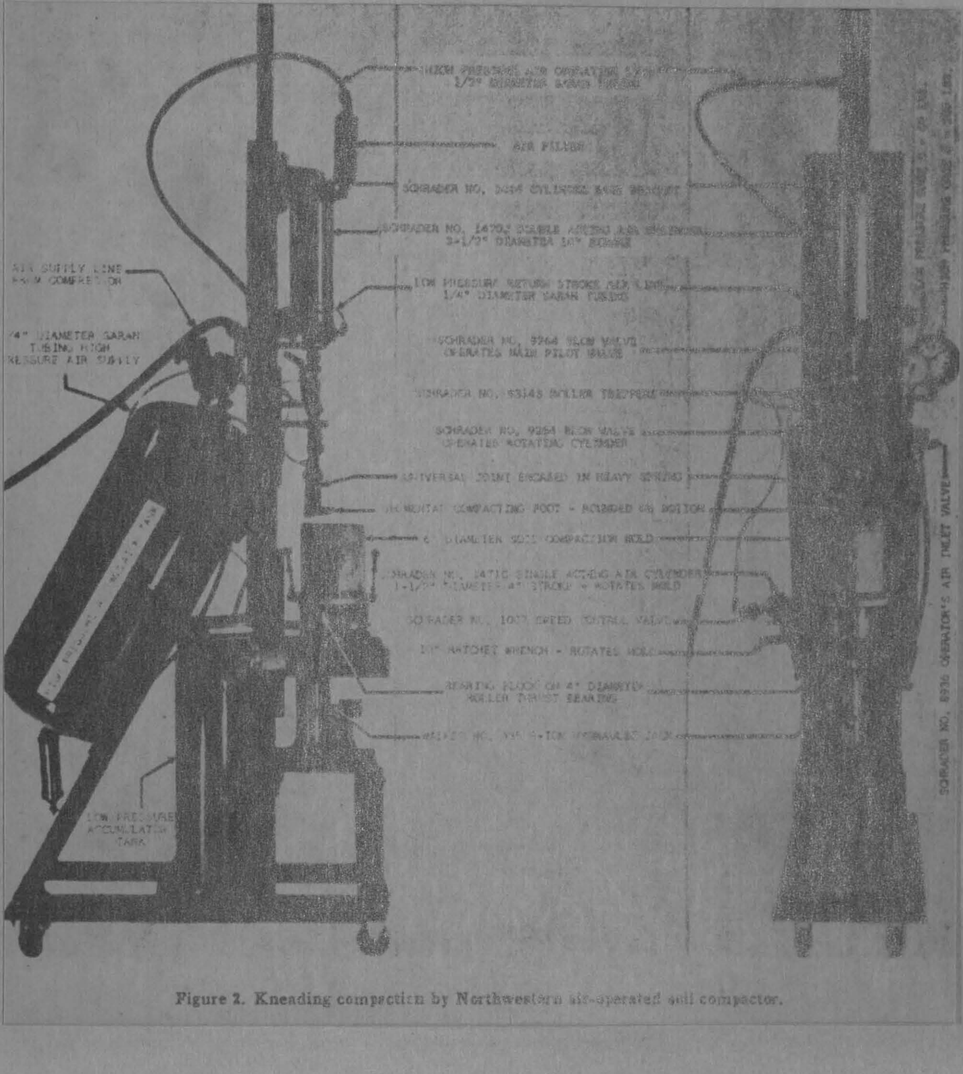


Figure 2. Kneading compaction by Northwestern air-operated soil compactor.

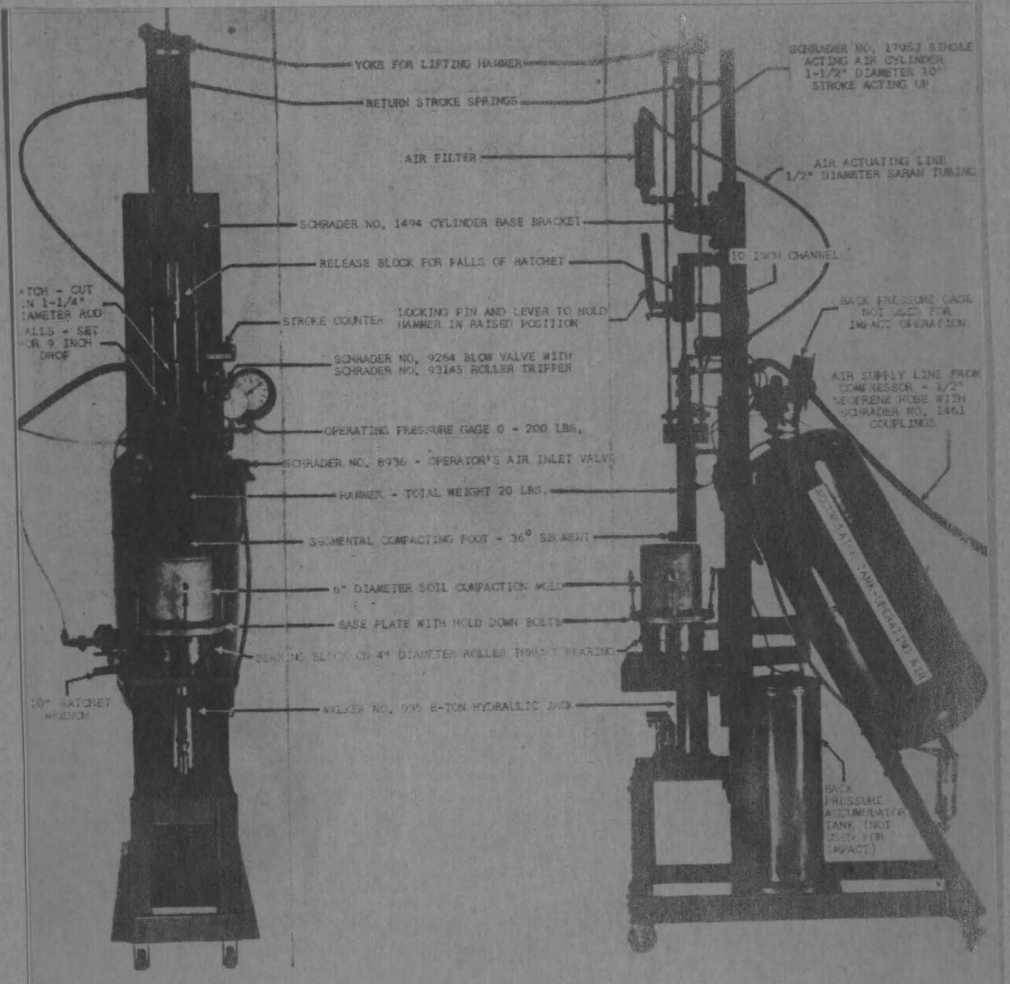


Figure 3. Impact compaction by Northwestern air-operated soil compactor.

FIG. 16 b

2. Compaction curves closely approximating those obtained by ield rolling compaction obtained for a particular kind of oil.

3. Compaction of 6 in diameter specimens closely approximate field compaction.

FIELD COMPACTION

In the field soil is compacted by applying energy in one of three ways, which in order of duration of the stress which they apply are:

1. Kneading
2. Pressure (Rolling)
3. Impact (Ramming)
4. Vibration

The type of compaction plant that are available can be listed under these headings as follows:

1+2. Rollers- Smooth wheeled, pneumatic tyred, sheepsfoot, lorries and pneumatic tyred construction plant, and track laying vehicles.

3. Rammers- Dropping weight(including piling equipment) internal combustion type and pneumatic type.

4. Vibrators- Out of balance weight type and pulsating hydraulic type.

The factors affecting the field compaction of soil are similar to those influencing the compaction of soil in the laboratory tests The more important factors are the moisture content of the soil and the amount of compaction applied with the particular plant. The performance of compaction plant is dependent on the soil type, its particle size and its moisture content. In general, smooth wheel rollers are most suited to crushed rock, hard core, mechanically stable gravels and sands, pneumatic tyred rollers to closely graded sands and fine-grained cohesive soils at moisture contents approaching their plastic limit, and sheepsfoot rollers to fine grained cohesive soils at moisture contents of from 7 to 12% below their plastic limits.

Before we take up the compaction of cohesive soils let us say a few words about the non-cohesive. The best method for these is vibration. The vibration frequency is the most important factor governing the efficiency of the method. If the vibration frequenc is considerably smaller than the natural frequency of the soil, a relatively small settlement is obtained. At the resonant frequency the settlement obtained is 20-40 times the settlement that would be obtained by the static load. This critical frequenc is different for different soils. Smooth wheeled rollers also giv good results with sands and gravels.

As the cohesion of the soil increases the effect of vibration rapidly decreases, because the smallest attachment between the so particles prevents them to move into a more compact form. Therefore for fine grained soils especially for closely graded sands w use pneumatic tyred rollers, and for more cohesive clays, sheepsfoot rollers. Pneumatic rollers can also be successfully used fo clays. A common type of pneumatic roller consists of a box or platform mounted between two axles, the rear of which has one mor wheel than the front, the wheels mounted on the front axle being arranged to track in between those mounted on the rear axle. Th number of wheels vary according to the make and the gross weight o the roller. Another type known as the wobble-wheel roller has wheels mounted at a slight angle with respect to the axle. This provides a kneading action which is claimed to give improved results.

Let us now go into a detailed survey of pneumatic tyred and sheepsfoot rollers, which are in our field of interest because the are used almost invariably in the compaction of cohesive soils. The main factors that can be related to the dry density, apart fro the moisture content are numbers of coverages, inflation pressure, wheel load, sheepsfoot type, load per unit area of the foot according to the type of roller used. Therefore the operation of these rollers will be considered in relation to the factors.

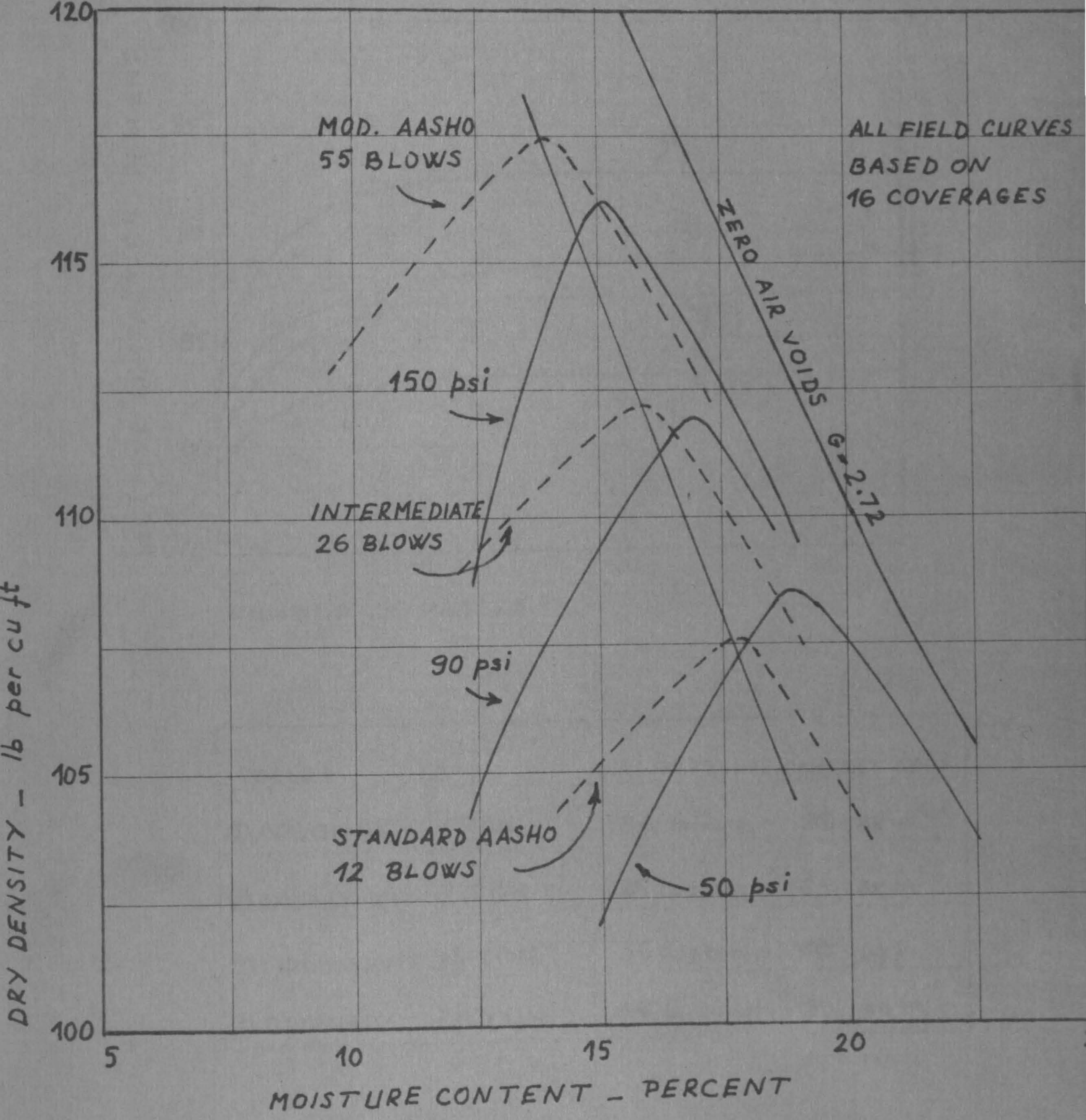
a) Pneumatic tyred rollers.

1. Inflation pressure.

Generally increased density resulted from increased inflation pressure so long as a reasonable wheel load is maintained. This i illustrated by (Fig 17) where the dry density-moisture content relation is shown for a lean clay of LL=38 and PI=18 is compacted for three inflation pressures, and by (Fig 18) where a brickearth of LL=31 and PI=12 is considered.

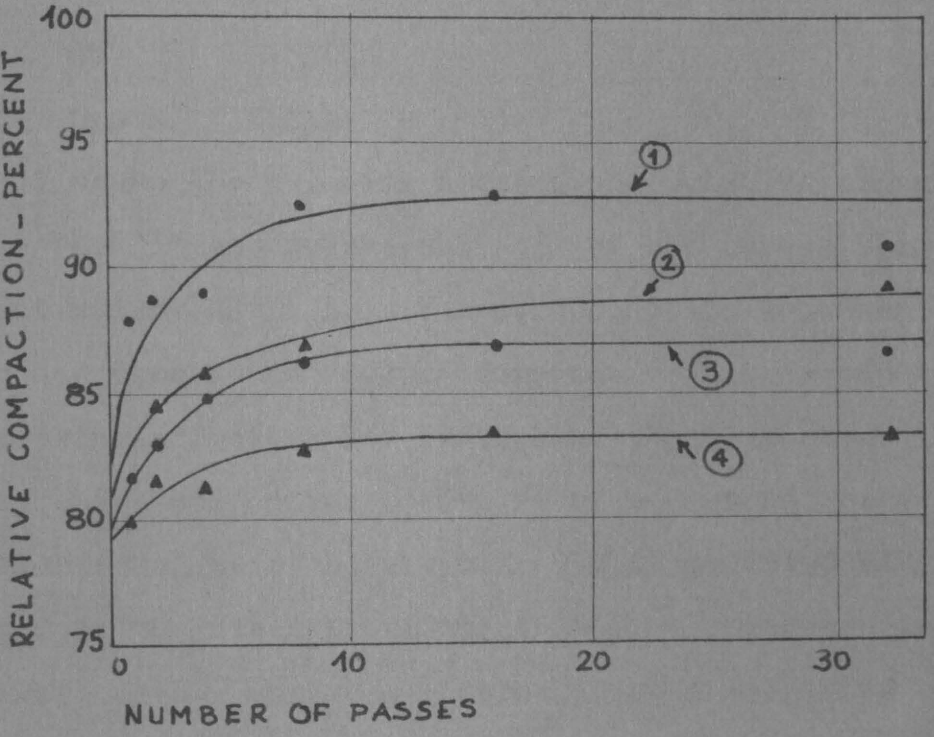
2. Number of coverages.

Adequate densities are usually achieved with four coverages, a small increase in dry density results from additional coverages, this increase is larger at moisture contents drier than the optim



COMPARISON OF LABORATORY COMPACTION WITH FIELD COMPACTION BY RUBBER TYRED ROLLERS

FIG. 17



ROLLER	LOAD	INF. PRESSURE	CONTACT AREA
① ORDINARY	7 TON	36 lb per sq in	39 sq in
② WOBBLY WH.	7 TON	25 lb per sq in	65 sq in
③ ORDINARY	3½ TON	36 lb per sq in	18 sq in
④ ORDINARY	3½ TON	14 lb per sq in	39 sq in

COMPACTION OF BRICKEARTH BY PNEUMATIC - TYRED ROLLERS

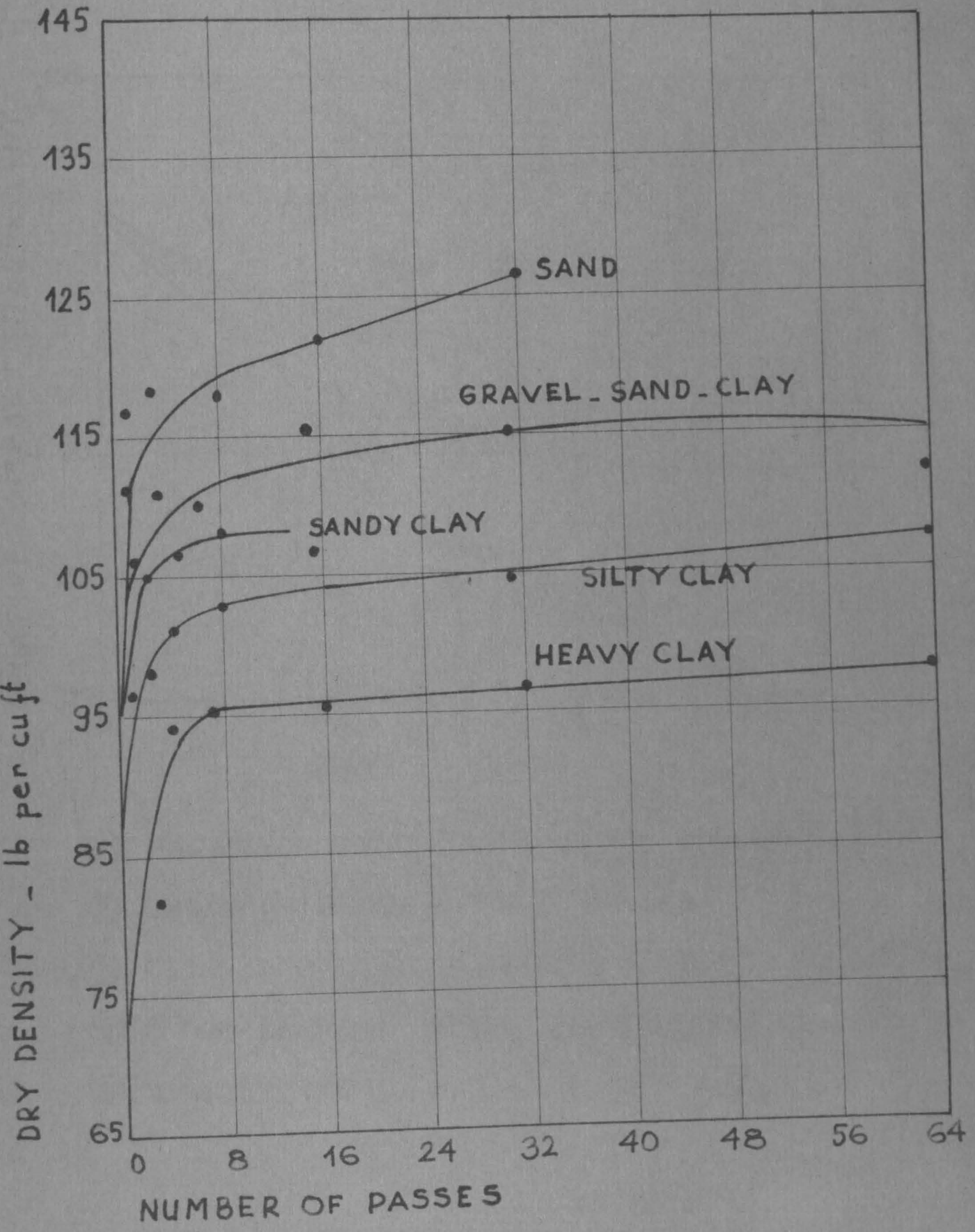
In other words the smallest number of passes is required when the soil is at or a little above the optimum moisture content. (Fig 18 and (Fig 19)

3. Depth of fill.

(Fig 20) shows the relation between the relative compaction and th fill depth. It is obvious that all of the curves tend to reach a constant value. With this token, compaction to a depth of 24 inches has been found satisfactory for ordinary compactors, wobble-wheels being effective for lifts less than 3 inches thick. In certain cases compaction to depths of 6 feet has been achieved. Densities equal to or better than 95% of modified AASHO are readil obtained in the field providing that tyre pressures in excess of 60 psi are used. Supercompactors having wheel loads up to 100000l and tyre pressures up to 150 psi have been used effectively on several jobs including the New Jersey Turnpike, Baltimore Friendship Airport and some airports in Morocco. However this is usuall considered a special purpose tool for effective deep compaction, detecting weak zones in the subgrade and final compaction of flexible pavement systems.

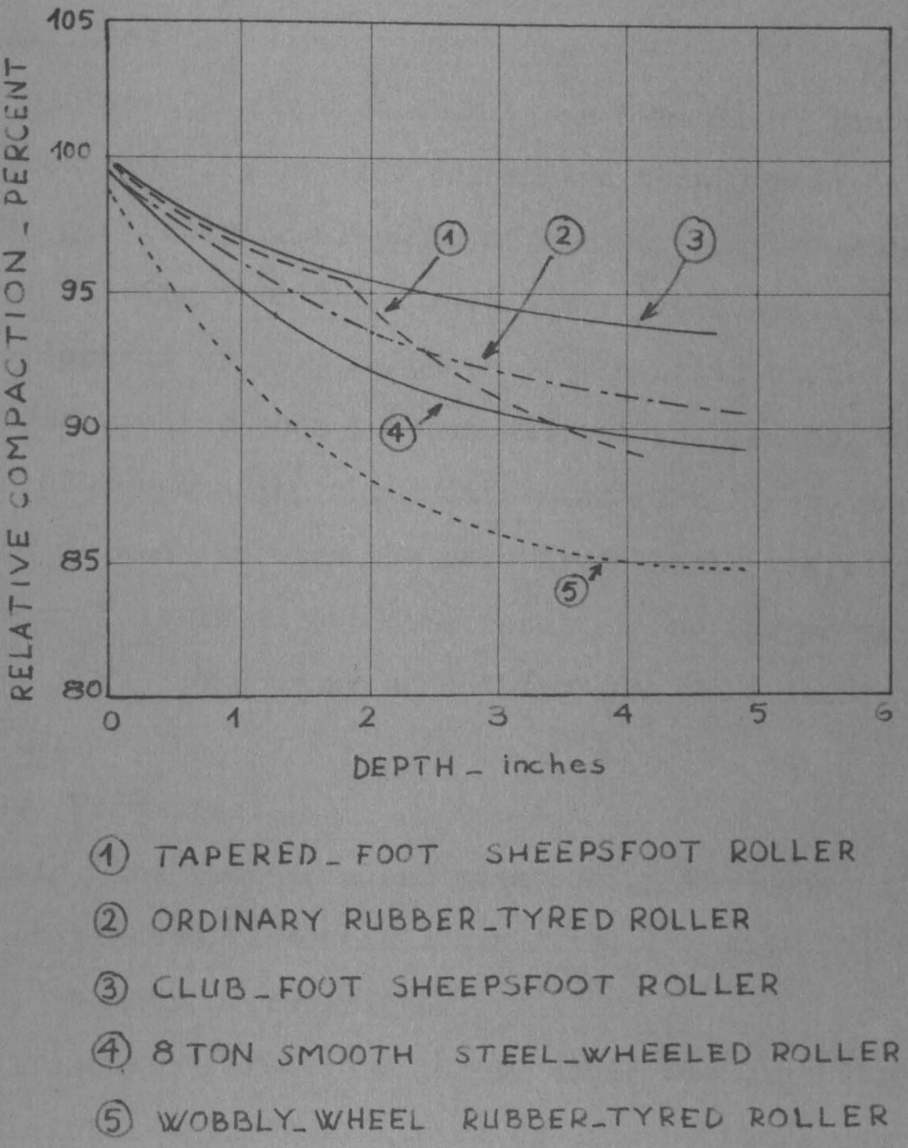
b) Sheepsfoot rollers

These are considered to be the best plants for the compaction of cohesive soils. They consist of a hollow cylindrical steel drum on which projecting feet are mounted. The drums can be ballested either with water or with wet sand and they are mounted either singly or in pairs on a welded steel frame. There are two types



DRY DENSITY _ NUMBER OF PASSES RELATIONSHIPS OF THE PNEUMATIC TYRED ROLLER WHEN COMPACTED AT OR JUST ABOVE THEIR OPTIMUM MOISTURE CONTENT FOR ROLLER COMPACTION

FIG _ 19



VARIATION IN RELATIVE COMPACTION WITH DEPTH
GIVEN BY VARIOUS ROLLERS

FIG_ 20

The Club foot type and the taper foot type.

1. Foot pressure

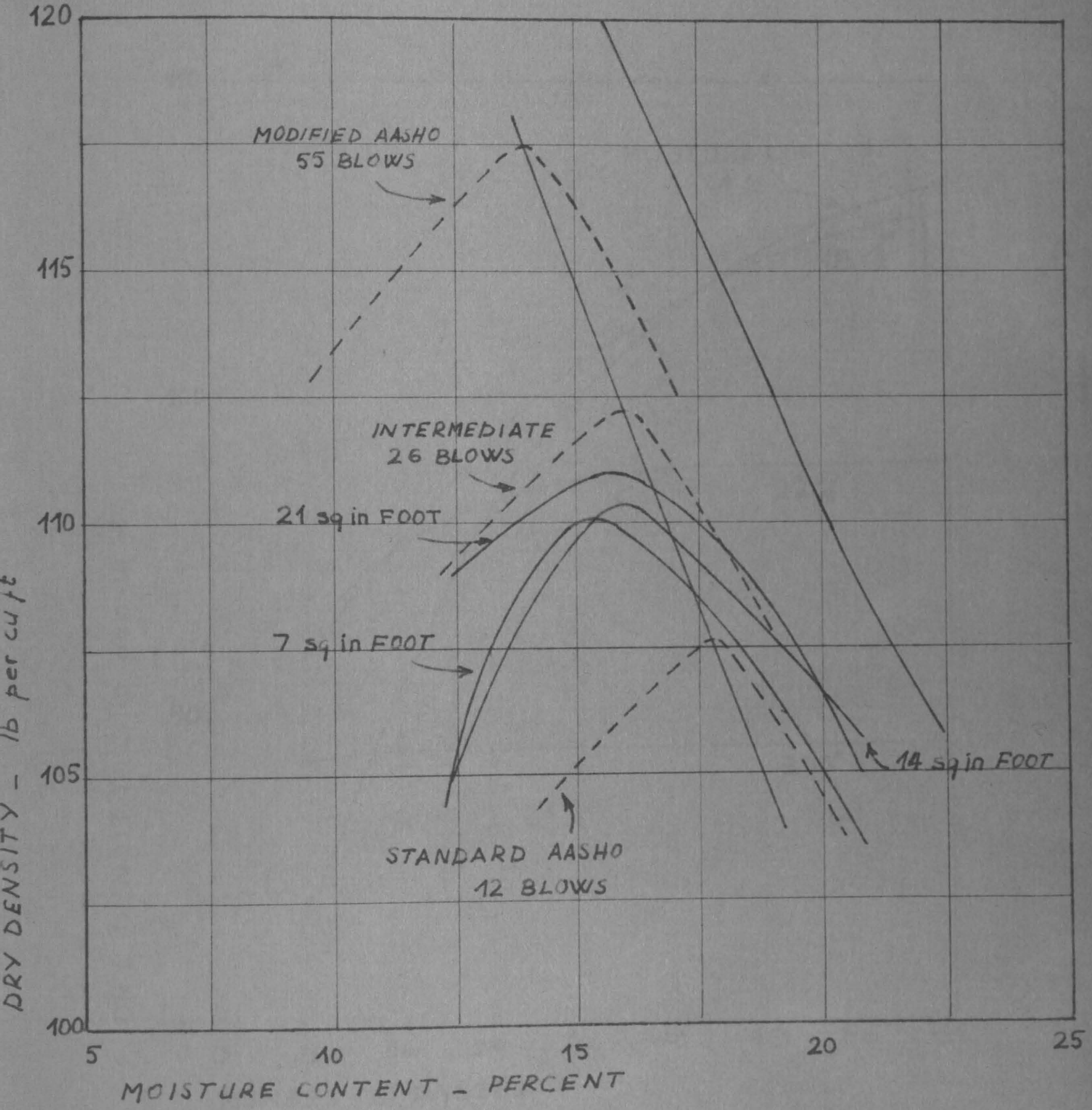
Tests within the range of foot areas 6 to 21 sq inches and of unit foot pressures 125 to 1110 psi showed that pressures in excess of 400 psi generally resulted in no greater compacted densities than those resulting from the use of lower pressures. This is attributed to lack of supporting power of the soils tested at unit pressures greater than 400 psi and rollers did not walk out. At a series of tests using three different foot sizes, none of the rollers walked out when the moisture content was wetter than the optimum. This is an expected result in so far as that it is quite certain that the bearing capacity of the soil in suchcases has been exceeded.

2. Foot area.

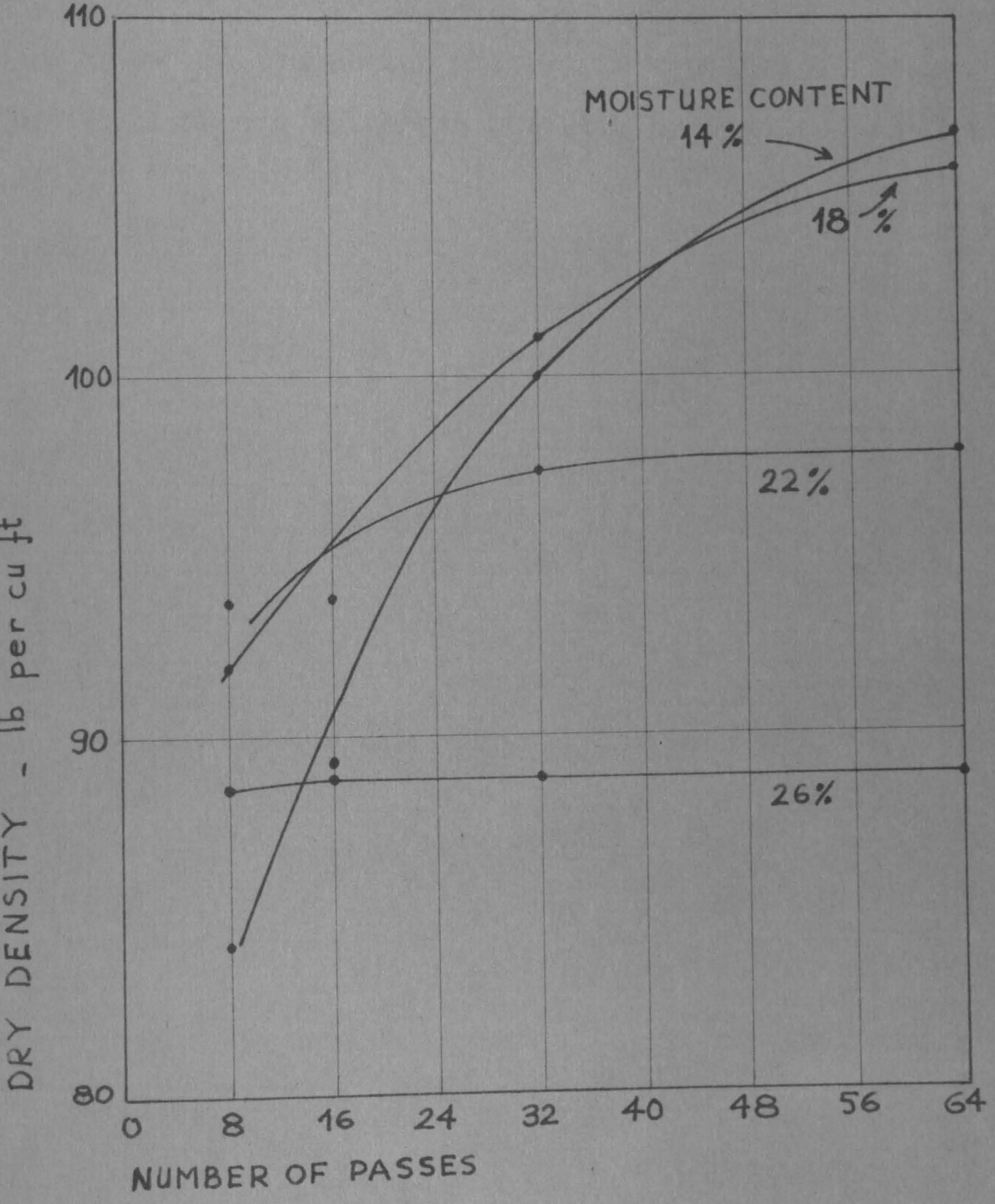
Increasing the foot area and maintaining the same unit pressure does not produce discernible benefits. (Fig 21)

3. Number of coverages.

A small increase of 4 to 6 lb per cubic foot in field density associated with a small decrease in optimum koisture content (3 to 1%) resulted as the number of passes increased from 6 to 24. At moisture contents wetter than the optimum, smaller number of coverages are requiered, to attain the density attainable for that particular moisture content, in comparison with moisture contents drier than the optimum. (Fig 22) Road research laboratory (England), owever, recommends at least 24 passes and that the lift should not



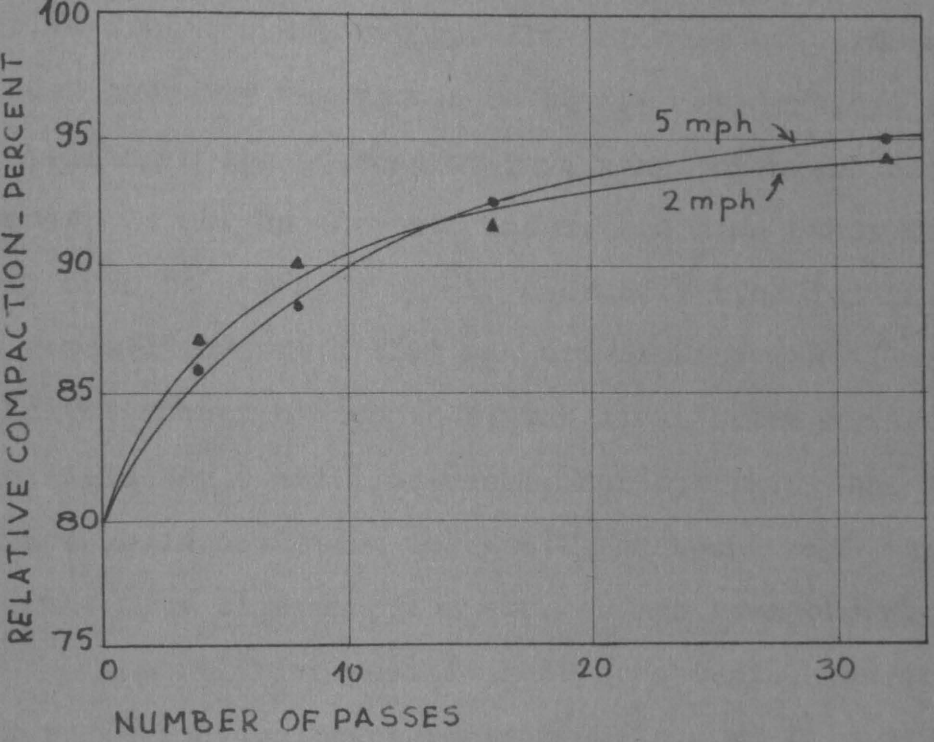
COMPARISON OF LABORATORY COMPACTION WITH FIELD COMPACTION BY SHEEPSFOOT ROLLERS - FOOT PRESSURE 25 psi AND 24 PASSES



DRY DENSITY _ NUMBER OF PASSES RELATIONSHIPS OF THE 4½ TON TAPER _ FOOT SHEEPSFOOT ROLLER FOR A HEAVY CLAY COMPACTED IN 9 in. LOOSE LAYERS AT DIFFERENT MOISTURE CONTENTS be 2 inches larger than the foot depth. It was seen that sheepsfoot rollers did not compact sand at all.

4. Speed of rollers.

Effect of speed in compacting brickearth was insignificant. (Fig 23) Therefore high speeds which can actually be realized are gaining popularity.



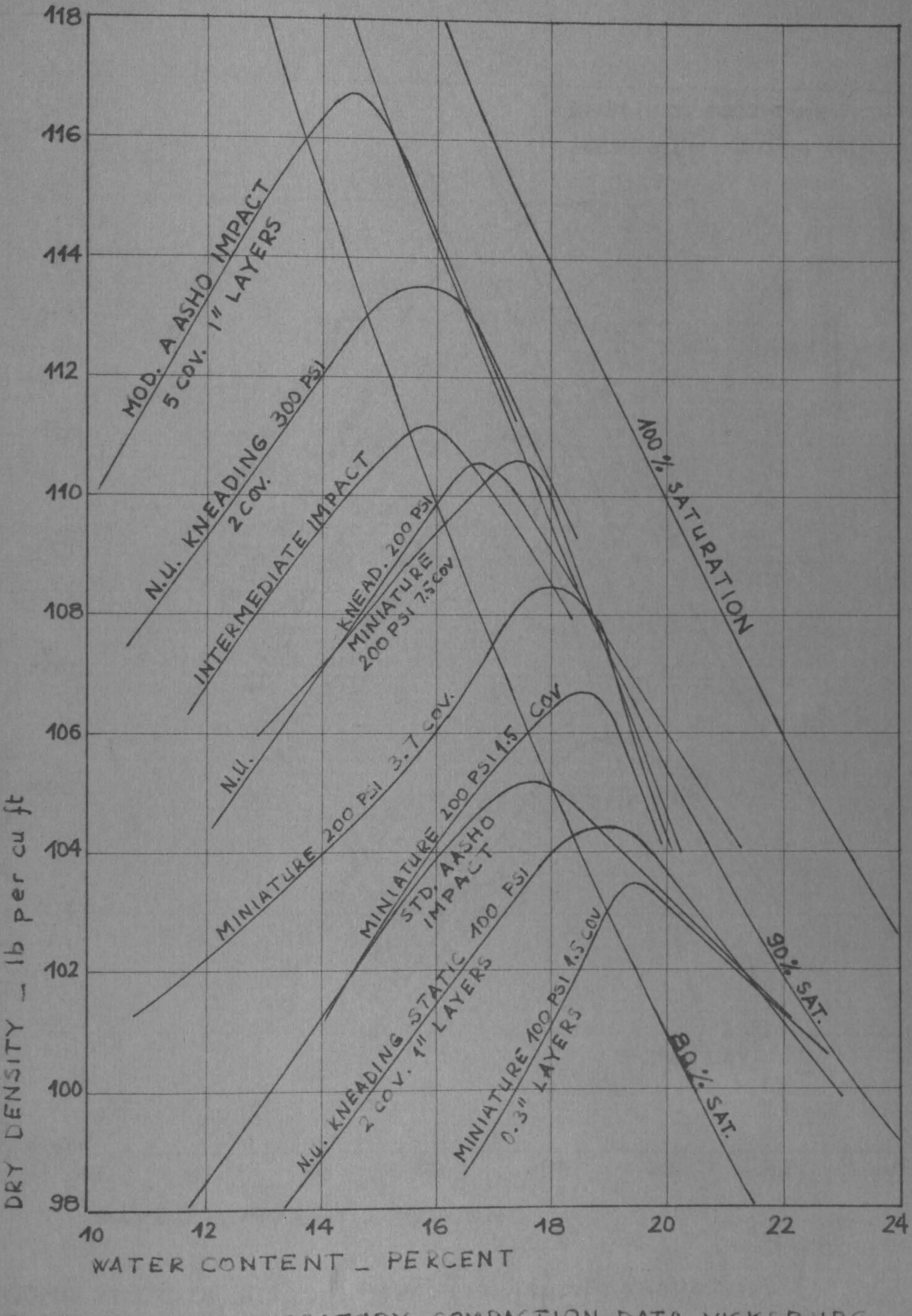
EFFECT OF SPEED IN COMPACTING BRICKEARTH WITH TAPERED - FOOT SHEEPSFOOT ROLLER

FIG 23

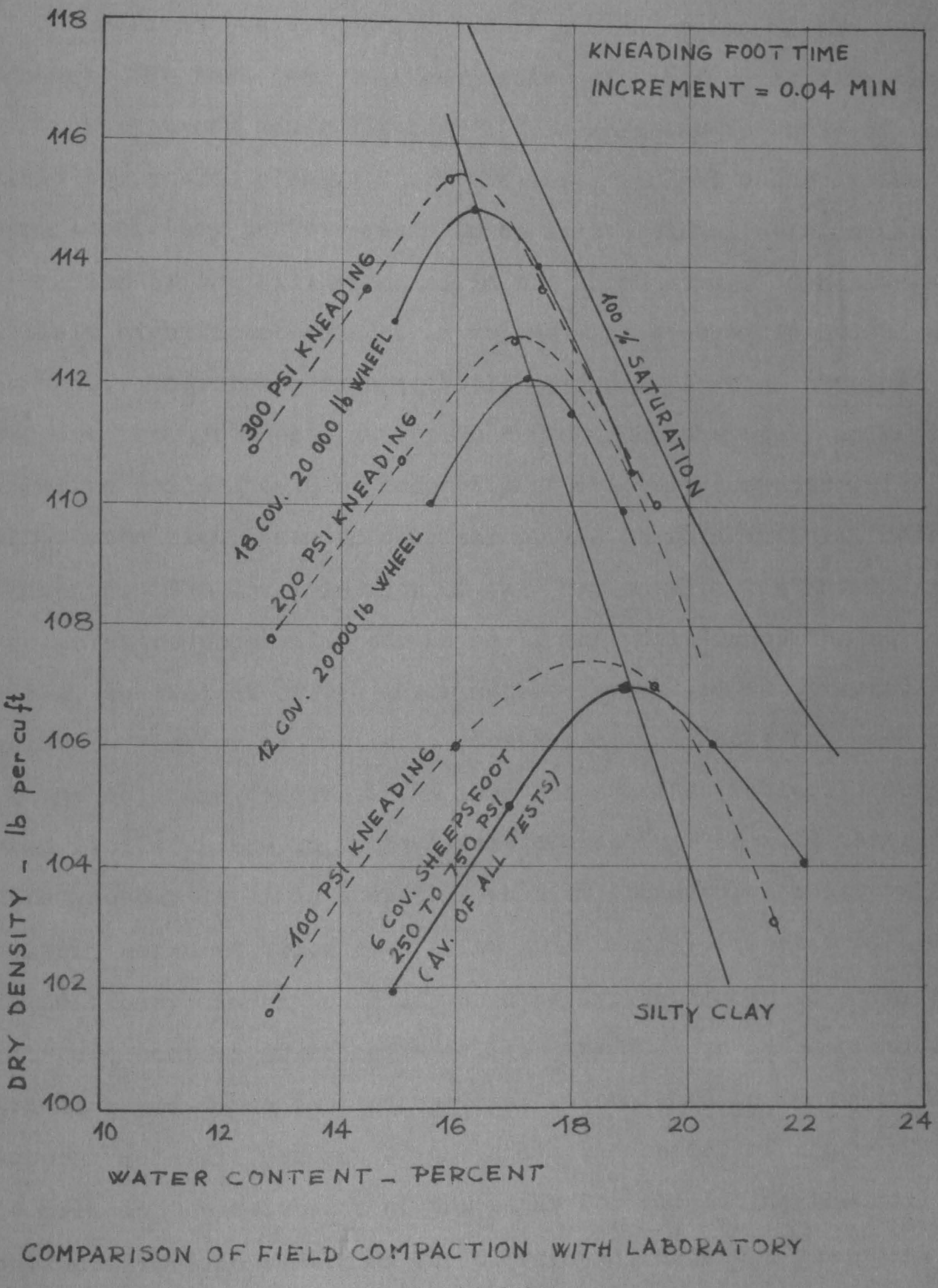
APPLICATION AND CONTROL

The very obvious reason for conducting laboratory tests is simply the fact that we can neither spare time nor money to see how a number of prototypes behave and therefore try to forecast the behavior of the structure by the help of an index property which we can derive from the laboratory. In the case of earthen structures this index property was chosen to be the dry density, even this choice raising a lot of controversy; many engineers claim that the dry density is not an adequate index, and some other more essential property such as strength should be substituted for it. Even if it was universally accepted that the dry density was illustrative in a sufficient number of respects, the links between the laboratory and the field would still be weak. For one thing the laboratory compaction is inconsistent in itself, changing with the applied energy, the type of compaction etc. as has been shown. (Fig 24) also shows the relative positions of some tests. Furthermore no laboratory method duplicate field results e.g. (Fig 21) with the possible exception of the N.U. Kneading Tests which has been reported[7] to produce compaction curves in close agreement to field results and the position of the optimum line of which can be moved by changing the time increment for the compaction foot; this second property in itself brings adjustibility to the method (Fig 25).

If we think of things like non-uniformity of compaction, non-uniformity of the soil it becomes clear that the application of compaction to the actual problems needs a broader outlook.



COMPARISON OF LABORATORY COMPACTION DATA_VICKSBURG SILTY CLAY



COMPARISON OF FIELD COMPACTION WITH LABORATORY
KNEADING COMPACTION

The first question arises as to the selection of the moisture content. We have seen that compactive efforts provided in the laboratory tests generally bear little relation to those of the field compaction plant. Therefore the principal value of the standard laboratory tests appears to be in the classification and selaction of the fill material in the earth works. Cohesive soils exhibit significant changes in volume with changes in moisture content. Therefore, as some field evidence has been obtained to support the hypothesis at the Road Research Laboratory in England, cohesive soils should be compacted at moisture contents which will not change significantly afterwards, i.e at their natural moisture contents. The determination of this may require the knowledge of the moisture properties of the soil, the position of the water table, the weight of the superimposed layers and the previous moisture history of the soil. However a study that has been made of the moisture conditions in road embarkments (Table II) suggests that little change in the moisture content of the fill material is likely occur if it is compacted when at its natural moisture content measured below 3-4 ft. of soil usually affected by weather conditions. In any case it would be impracticable to alter the moisture content of cohesive soils. Any full scale compaction trials carried out should therefore, be confined to tests at the natural moisture content of the soil; the object of the work being to help in the selection of the plant for compacting the soil at this moisture content. On the other hand, from the viewpoint of

SITE	KINGSTON BY-PASS	GODALMING BY-PASS	HAILSHAM BY-PASS	HAILSHAM BY-PASS	CRAWLEY BY-PASS	FARNHAM BY-PASS	LANCING BROOK
YEAR OF CONSTRUCTION	1926	1930	1934	1937	1937	1940	1947
TRAFFIC CONDITIONS	VERY HEAVY	HEAVY	HEAVY	HEAVY	HEAVY	HEAVY	MED.
DEPTH OF FILL (ft)	20	25	25	10	20	10	15
SOIL TYPE	CI	CL	CH	CH	CH	SU	CH
AVERAGE LIQUID LIMIT	46	27	64	52	59	NON PLASTIC	54
AVERAGE PLASTIC LIMIT	19	16	25	21	25		21
AVERAGE SPECIFIC GRAVITY	2.72	2.72	2.72	2.70	2.75	2.73	2.73
AVERAGE MOISTURE CONTENT - (%)	27	17	28	22	25	8	24
AVERAGE DRY DENSITY (lb per cu ft)	98	110	95	95	94	126	94
AVERAGE AIR VOIDS CONTENT (%)	0	6	1	10	8	10	9
APPROXIMATE AMOUNT OF SETTLEMENT OF ROAD SINCE CONSTRUCTION (in)	3	0*	6	0*	1	0*	0

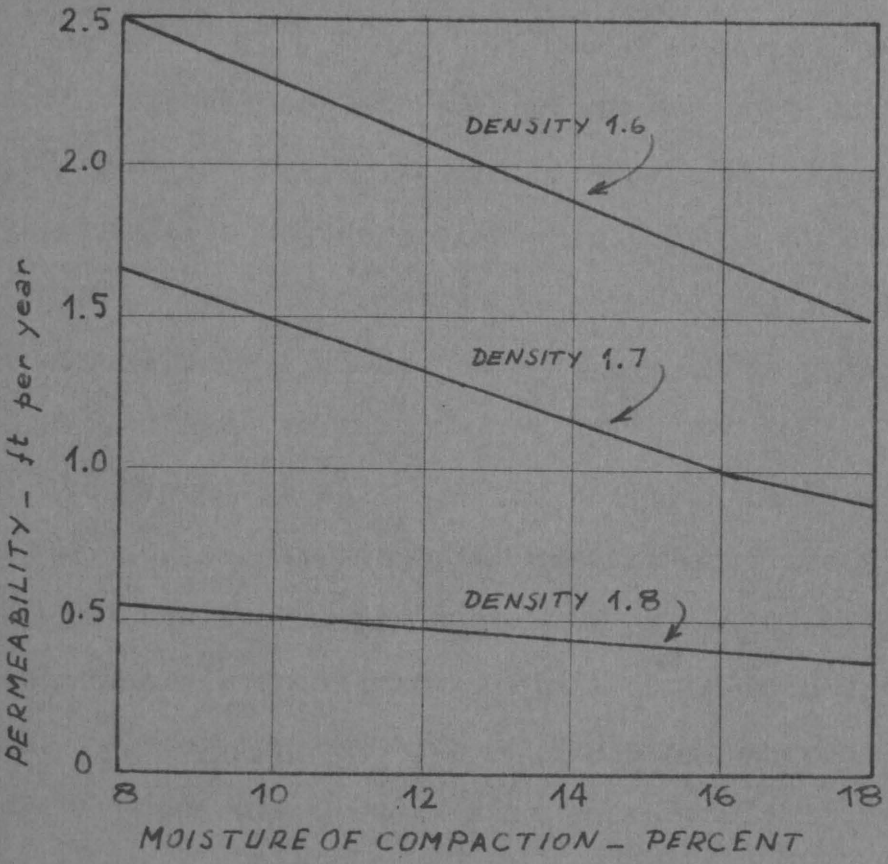
*) NO MEASUREMENTS ARE AVAILABLE BUT VISUAL INSPECTION OF THE ROAD INDICATES THAT LITTLE SETTLEMENT HAS OCCURED

the desired properties after the compaction the following criteria may be taken as rough guides:

1. Highway embankments and similar structures consisting of cohesive soils would best be compacted somewhat on the dry side of optimum in order to achieve high strength and resistance to deformation and low volume compressibility.

2. Impervious cores for earth dams should possess not only low permeability but be absolutely safe against cracking due to differential settlements or other causes. Therefore it is desirable to sacrifice strength and compact the materials for such cores on the wet side of the optimum in order to achieve low permeability, low resistance to deformation and high failure strains. For much works there is reported[8] to be a "stable density". This stable density is brought about by the wetting and drying of the soil. Decompaction starts even at the first cycle and a stable density between 95 and 103 lb per cubic ft. is achieved irrespective of the type of the soil, original density or the moisture of compaction. It was also found out that permeability decreased with increasing moisture of compaction (Fig 26).

3. Structural sections for earth dams, when built of cohesive soils, should be compacted on the dry side of the optimum in order to achieve maximum strength and maximum safety against developing pore pressures even though this involves the developement of cracks. If the structural section is to be dependent upon for safety against piping as in homogenous dams, then it is important


2.5
2.0
1.5
1.0
0.5
0
PERMEABILITY - ft per year
DENSITY 1.6
DENSITY 1.7
DENSITY 1.8
8
10
12
14
16
18
MOISTURE OF COMPACTION - PERCENT

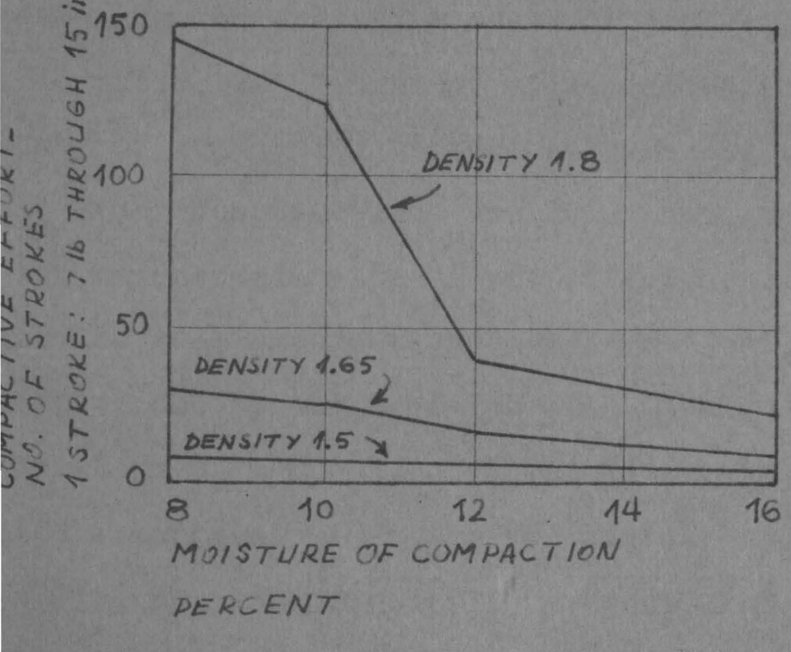

COMPACTIVE EFFORT-
NO. OF STROKES
1 STROKE: 7 lb THROUGH 15 in
150
100
50
0
DENSITY 1.8
DENSITY 1.65
DENSITY 1.5
8
10
12
14
16
MOISTURE OF COMPACTION
PERCENT

FIG_26_27

to subdivide the structural section and compact "transition sections" on the wet side in order to achieve the requirement of a core. There is another point which should be considered in the choice of the moisture content, and that is the fact that compactive effort decreases as the moisture of compaction increases (Fig 27).

The second question is the state of compaction to be attained. This depends on the amount of ensuing settlement which can be tolerated and the requirements for the stability of the embankment. The permissable settlement is influenced to a considerable extent by the type of construction of the superimposed road pavement as concrete roads are usually more susceptible to damage from settlement than bituminous surfaced roads. Except in the case of relatively high road embankments, problems of stability rarely arise and in general the question of settlement is regarded as the factor governing the necessary state of compaction to be produced. Little information is at present available on the relationship between the state of compaction of road embankments and the settlements occurring after construction, and it is not possible therefore, to say with any certainty the state of compaction that is required. To provide some information on this subject, investigations have been carried out by the Road Research Laboratory (England) to determine the state of compaction of the fill material of a number of old road embankments in which the settlement has now ceased. The results obtained (Table II) suggest that the settlement of such embakments is probably a function of the strength and compressibility properties of the fill material as well as the state of compaction obtained during construction. At sites 1 and 3 where the soils had high moisture contents in relation to their plastic limits and hence would be axpected to have low strength and high compressibility, a considerable amount of settlement occured. At sites 4 and 5, however, the soils were much drier , and little settlement occured despite the fact that the states of compaction of the fill were not particularly good. With clay soils at moisture contents close to their plastic limits the state of compaction required to reduce settlements to a negligible amount appears to be no higher than that corresponding to an air voids content of about 10%. Wetter clay soils, however, probably require a higher state of compaction. In structures like dams and levees, the governing factor is naturally the permeability. Soils compacted at very high densities need comperatively greater amount of over-load for keeping them in the stable position against the effect of swelling. Soils compacted for water reservoirs at dry densities lower than 95 lbs per cubic feet are standing quite stably.

The third consideration is the selection of plant. The results of the investigations at the Road Research Laboratory on some soils in the British Isles are given in Table III.as an example. The guiding principle in the selection of the typeof plant for compaction work is that the equipment should be capable of producing efficiently the desired state of compaction when the soil is at the

SOIL TYPE	NUMBER OF PASSES AND CORRESPONDING DRY DENSITY PRODUCED - lb per cu ft												
	PNEUMATIC - TYRED ROLLER	HEAVY SMOOTH WHEEL ROLLER	LIGHT SMOOTH WEEL ROLLER	CLUB - FOOT SHEEPSFOOT ROLLER	TAPER-FOOT SHEEPSFOOT ROLLER	12 CWT FROG RAMMER	2 CWT POWER RAMMER	4 CWT VIBRATING SM. WH. ROLLER	2.5 TON VIBRATING SM. WH. ROLLER	2 TON VIBRATING PLATE COMP.	1.5 TON VIBRATING PLATE COMP.	MEDIUM TRACK LAYING TRACTOR	HEAVY TRACK LAYING TRACTOR
HEAVY CLAY	4 94	4 91	8 89	*	8 90	2^+ 92	$2^\times$ 98	—	*	—	—	5 94	8 95
SILTY CLAY	4 102	4 103	8 98	*	20 100	2^+ 95	—	—	—	—	—	—	—
SANDY CLAY	4 103	4 106	8 102	*	*	2^+ 107	—	—	—	2 105	—	—	—
SAND	4 117	4 125	4 125	—	—	2^+ 122	$3^\times$ 122	6 112	6 126	2 125	2 121	3 127	—
GRAVEL SAND CLAY	4 120	4 128	8 128	32 123	20 120	2^+ 131	$3^\times$ 123	6 120	—	—	—	—	8 118
—	PLANT NOT TESTED		*	SOIL TOO WET FOR SATISFACTORY OPERATION			+	COVERAGES 6-8 BLOWS per COV.			×	COVERAGES 2-3 BLOWS per COV.	

TABLE - III

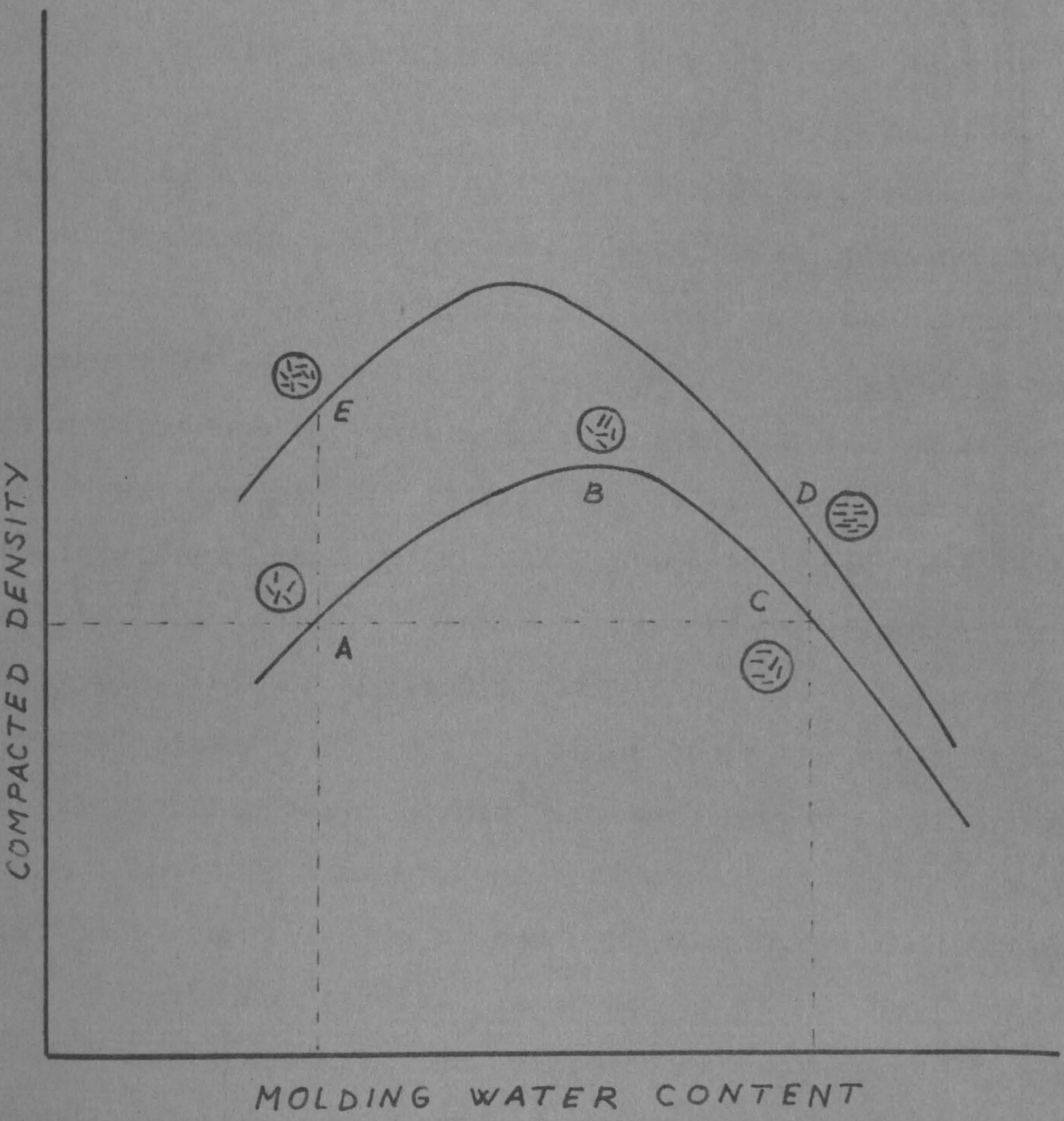
selected moisture content for compaction. Apart from the ability of the compaction plant to satisfy the requirements the availability and cost of operation of the plant are important considerations. In small works the question of availability will probably be the deciding factor in the choice of the compaction equipment. On large projects, however, the economics of the plant and methods of compaction will probably be governing factors. On large earthworks a small compaction trial should be carried out prior to the construction work to determine the type of plant, the optimum thickness of layer and the number of passes which will give the required state ofcompaction with the lowest cost per cubic yard of fill material.

The fourth question comes as to the control. The common practice is specifying a compaction as a percentage of the dry density obtained by a standard laboratory test, the drawbacks of which were already discussed. What is more, when it comes to the practice if the moisture content selected for compaction is found to be much lower than the optimum obtained by the laboratory test, a poor compaction measured in terms of the air voids of the soil would result in the field even if a high "relative compaction" wer actually relized. On the other hand, if the moisture content selected for field compaction is much higher than the laboratory optimum, the specified relative compaction might well be quite unattainable. A more logical approach would be to specify the state of compaction to be obtained in the field in terms of a maximum air voids content for the compacted soil. The problem of control of soil compaction would be simplified considerably by this procedure as it would eliminate the difficulty which often arises in distinguishing between variations in the dry density due to changes in the state of compaction and changes in the soil type. The determination of the air voids content of compacted soil is no more complicated than the calculation of the relative compaction as it requires only a knowledge of the specific gravity of the soil particles in addition to the dry density and the moisture content of the compacted soil. It was found out that,[9] twenty independent measurements of field density of an area of ground were required to obtain a mean relative compaction within limits of ± 1% with a probability of 9 changes in 10.

CHANGES IN PROPERTIES DUE TO COMPACTION

Having considered the process of compaction and the factors influencing it, one last question will be considered: Why do we require compaction of soils? First, as stressed in the introduction, it is necessary if embankments are to maintain a planned grade line, in other words, to avoid settlements in the embankment material itself. Secondly, many materials have improved bearing values when thoroughly compacted, although the amount of liquid present is usually more significant. Compaction also tends to reduce the size of the void spaces and decrease permeability.

Before going into a discussion of the above, a short explanation of the arrangement of the clay particles in a compacted clay will be given. This picture will help to visualize the property changes and will be referred to in the discussion. The arrangemen of the clay particles in a compacted soil is illustrated in Fig 28 At point A the small amount of water present results in a high concentration of electrolyte which prevents the diffuse double layer of ions surrounding each clay particle from developing fully. The double layer depression leads to low inter particle repulsion, resulting in a tendency towards flocculation of the colloids and a consequent low degree of clay particle orientation in the compacted soil. This type of structure has been termed a "flocculated" arrangement of soil particles. If the water conten is decreased to B, the electrolyte concentration is reduced, resulting in an expansion of the double layer, increased repulsio

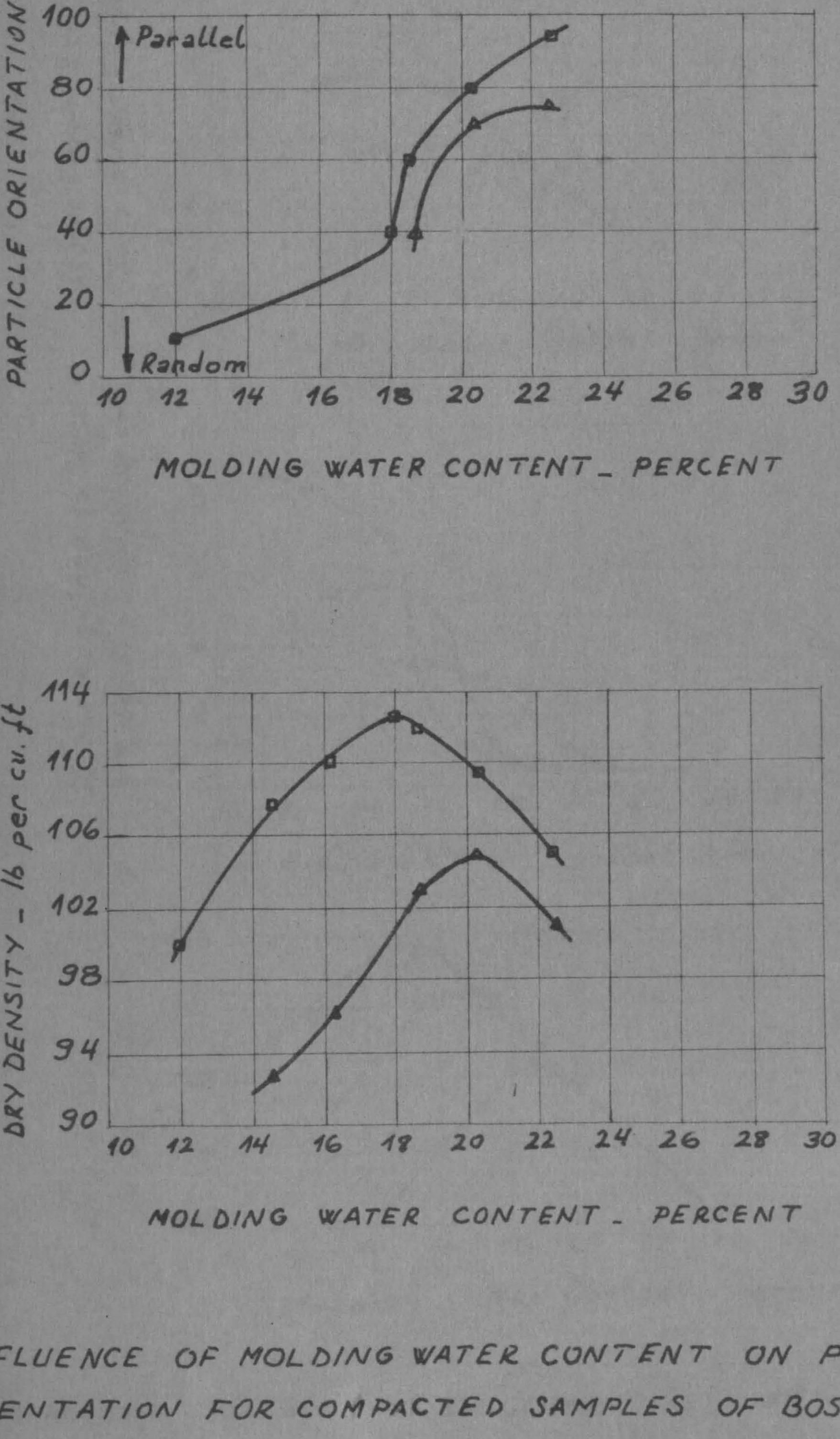


EFFECT OF COMPACTION ON SOIL STRUCTURE

FIG_ 28

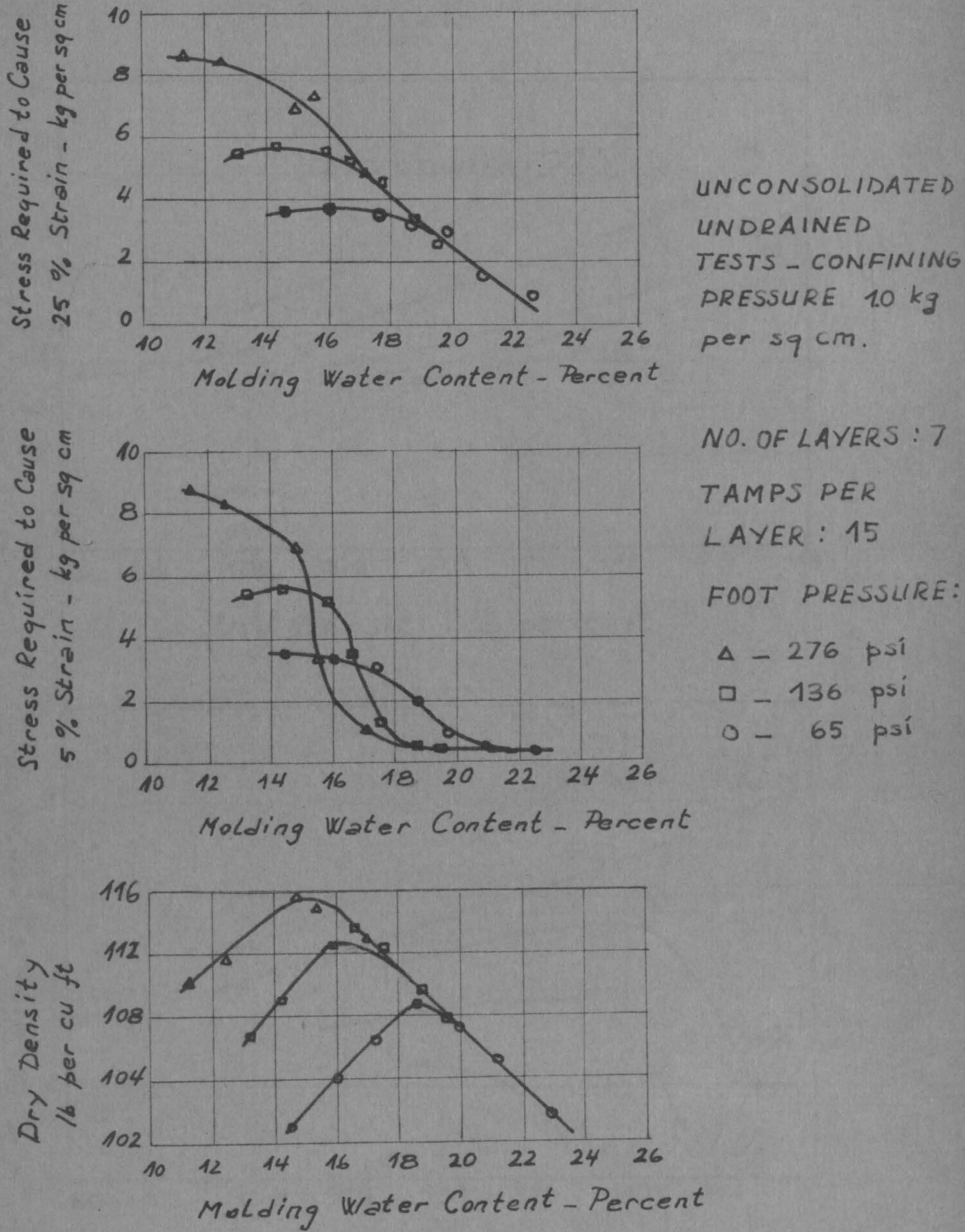
between particles and a low degree of flocculation, that is an increased degree of particle orientation. A system of parallel particles, which is approached at point C has been termed a dispersed system. Thus in general it may be said that compaction of clay dry of optimum tends to produce a flocculated arrangement of particles, while the compaction of the same soil "wet of optimum" tends to produce a disperced arrangement of particles.

Now let us turn to the important consideration of compacting soils to produce maximum strength. Conflicting opinions are prevalent at this point. Laboratory and field data have been presented to show that an increase in dry density at a given water content may produce a decrease in strength. On the other hand many engineers insist that this is not so and that increased density must inevitably lead to increased strength. Let us consider, for example, the density-strength-water content relationship for the silty clay (Liquid Limit 37, Plastic Limit 23, % finer than 0.002 mm=24) shown in Fig 29. In these tests the entire stress-strain relationship was recorded[10] for each sample of different dry density and moisture content, and the "strength" was determined both as the stress required to cause 5% strain and the stress required to cause 20% strain. It should be noted that these (Fig 29.b) "strengths" are not simply the points on the stress-strain curve corresponding to 5% and 20% strains but will often be maximum deviator stress which the sample can sustain if this stress has to be exceeded before the necessary strain can be developed. From

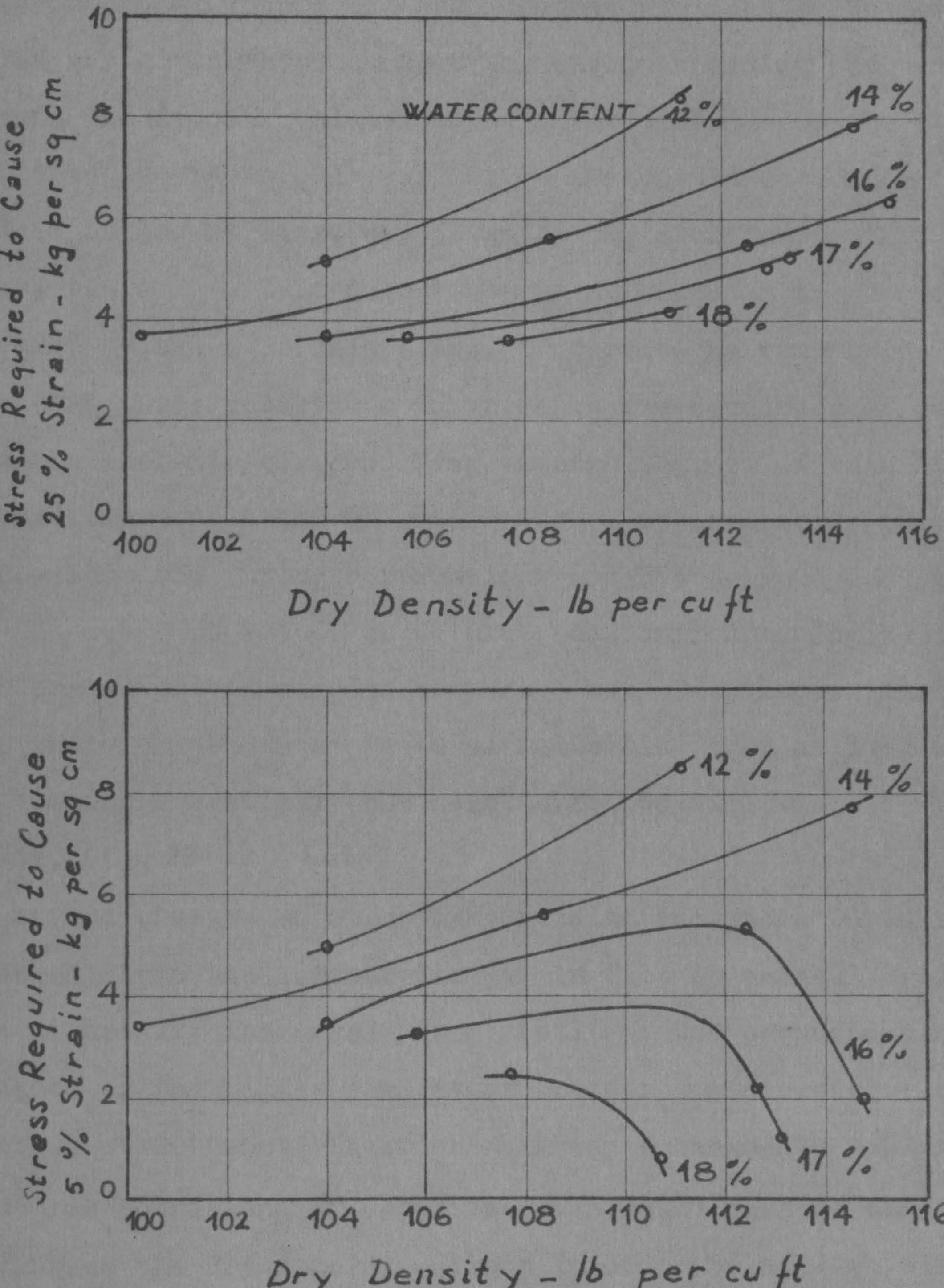


INFLUENCE OF MOLDING WATER CONTENT ON PARTICLE ORIENTATION FOR COMPACTED SAMPLES OF BOSTON BLUE CLAY

FIG. 29.a



RELATIONSHIP BETWEEN WATER CONTENT AND STRENGTH AS COMPACTED FOR SAMPLES OF SILTY CLAY - KNEADING COMPACTION



RELATIONSHIP BETWEEN DRY DENSITY AND STRENGTH

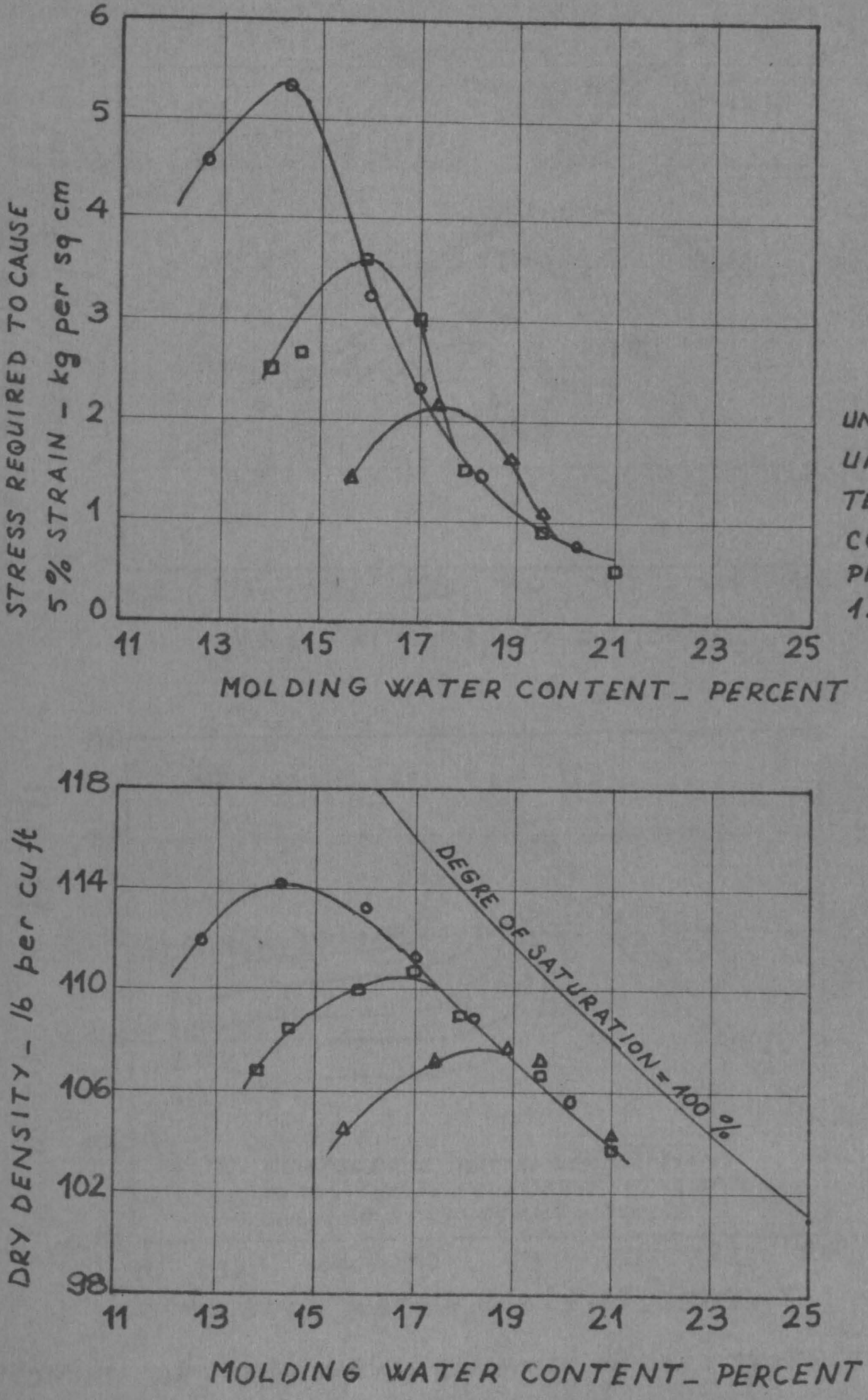
AS COMPACTED FOR SAMPLES OF SILTY CLAY - KNEADING COMPACTION

FIG. 29 C

this data the relationship between strength and dry density at various constant values of water content can readily be determined. Thus at any given water content three corresponding pairs of strength and density values can be interpolated from the curves and plotted as shown. It will be seen when the "strength" is determined at low strains, say 5% as is the practice in the pavement design, there is a marked decrease in strength as the density is increased beyond a certain stage. Reference to compaction curves shows that these reductions in strength are developed at densities and water contents corresponding to high degrees of saturation or to compacted conditions which are wet of optimum on a compaction curve. Thus the strength reductions would appear to be associated with the more dispersed structures (and correspondingly high pore-water pressures) of samples compacted wet of optimum. These data might therefore be interpreted as indicating that as long as structure remains essentially the same, strength will increase with density, (Fig 29.c), but when marked changes in structure are also incorporated in the data, the strength in the low strains may in fact be markedly reduced, in spite of density increases, as a result of the predominating influence of the higher pore water pressures associated with the dispersed structures. Therefore highway embankments and similar structures consisting of cohesive soils would best be compacted somewhat on the dry side of optimum in order to achieve high strength and resistance to deformation and low volume compressibility. However, if strength is determined at high strains, say 20%, as is the practice for foundation studies or earth dam design, samples of soil having the same composition exibit approximately equal strengths whether the structure is flocculated or dispersed. Consequently, the soil behaves from strength point of view as if the structures all the samples were the same, and the relationship between dry density and strength for a given molding water content shows no decrease in strength with increase in density as may be seen in Fig 29. Thus the relationship between strength, density and water content may vary greatly depending greatly on the manner in which the strength is determined, and this in turn will depend on the purpose for which the relationship is being used.

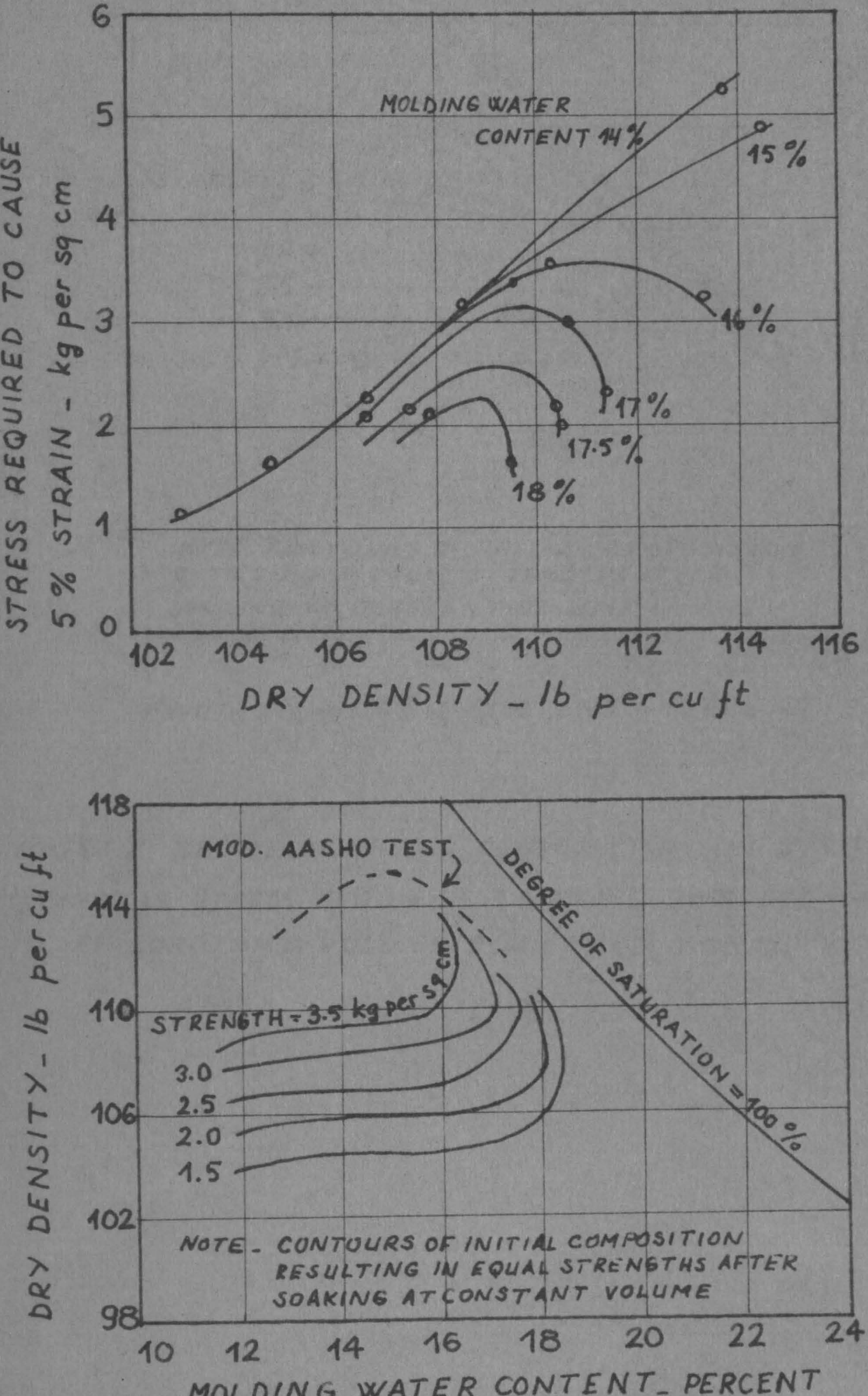
It should be noted, however, that the above considerations do not apply in their entirety to all compacted clays. For some soils the interparticle forces may be so strong that the changes resulting from compaction at various are insufficient to influence appreciably the tendency of the particles to flocculate or disperse Thus samples compacted wet and dry of optimum will have essentially similar structures. Again it is possible that in some soils the structures of the clay fraction of samples compacted wet of optimum are considerably more dispersed than those of samples compacted dry of optimum, but the influence of the difference in structure is mashed by other factors (such as high proportion of granular particles) which produce a steep stress-strain relationship in spite the dispersed particle arrangement in the clay fraction, in other words although the soil has a dispersed particle arrangement it may behave as if it has a flocculated one.

The above are considerations for the "as compacted" state of the soil. However, in practice, compacted soils are often soaked after compaction to a condition approaching saturation, for exampl under pavements and in earth dams, and the relationship between initial composition and strength after soaking for such soils are more important from practical point of view. Fig 30, Fig 31 and Fig 32 show such a study.[10] The material is a silty clay(Liquid Limit 37, Plastic Limit 23, Percent finer than 0.002 mm=24) prepared kneading compaction and soaked at constant volume. The samples were mixed to different water contents and then compacted using these three different compactive efforts. The water content and dry density of each sample were determined and then the samples were soaked under whatever surcharge was required to maintain constant volume. After a period of soaking, during which the samples became essentially saturated, the strength of each sample was determined in an uncolidated-undrained triaxial compression test using a confining pressure of 1 kg. per sq.cm. "Strengths" were determined as the stress required to cause 5% strain and stresses required to cause 20% strain. Fig 30 shows the results of such a test. It is obvious from this figure that water contents a littl below optimum lead to good strength values as determined at small strains. Fig 31 is derived from Fig 30 and Fig 32 from a similar



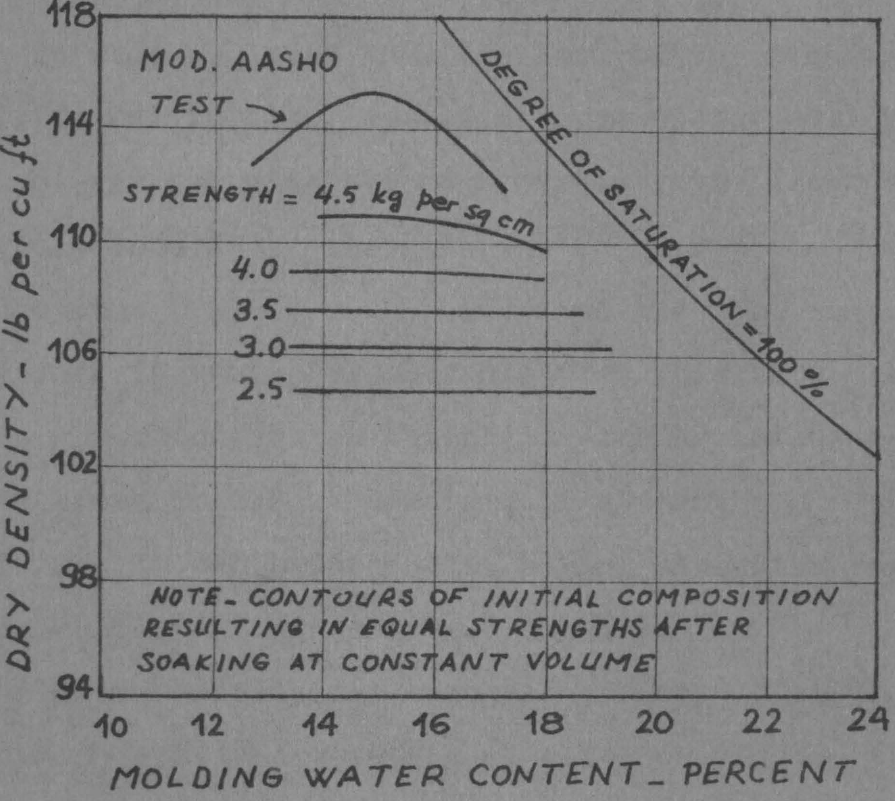
UNCONSOLIDATED
UNDRAINED
TESTS -
CONFINING
PRESSURE :
1.0 kg per sq.cm

RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SOAKING AT CONSTANT VOLUME FOR SAMPLES OF SILTY CLAY_ KN. COM



RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SOAKING AT CONSTANT VOLUME FOR SAMPLES OF SILTY CLAY_ KNEADING COM

FIG- 31

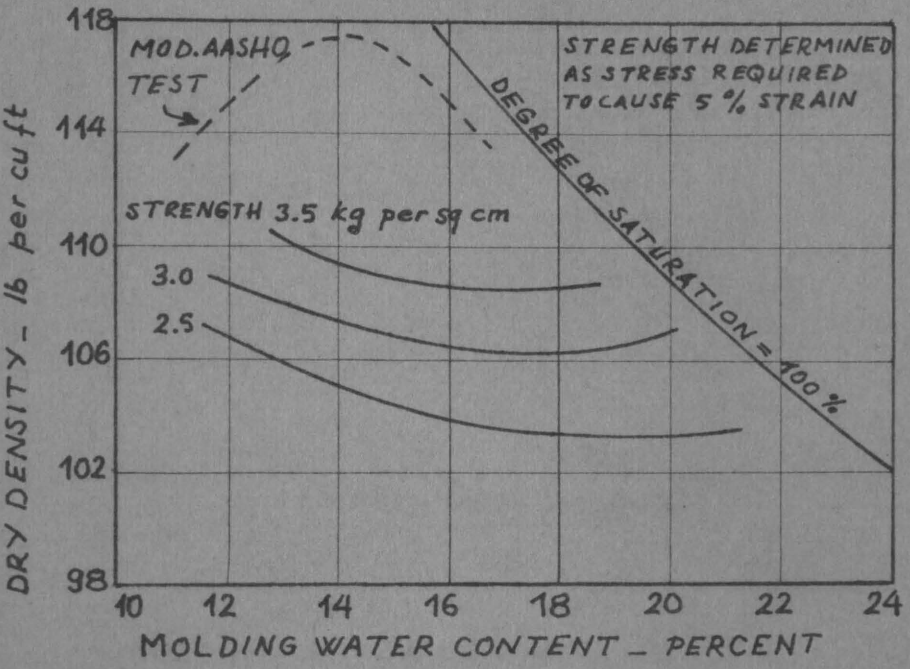


RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH, DETERMINED AS STRESS REQUIRED TO CAUSE 20% STRAIN, AFTER SOAKING AT CONSTANT VOLUME FOR SAMPLES OF SILTY CLAY - KN. COM.

FIG - 32

curve. It will be noted that the form of the resulting curves are quite different for the two definitions of the soil strength. If strengths are determined at high strains, the lines are essentially horizontal, indicating that samples compacted to a given density, no matter what the water at compaction have the same strength. In other words, it would be concluded from this data that for this soil the water content at compaction has no influence on the soil strength after soaking. However, if strengths are determined at low strains, it can be seen that higher strengths result with lower moisture contents for a constant density.

The above conclusions change according to whether the soil is expansive or not. More expansive clays obey these considerations better. In non-expansive soils higher moisture contents are necessary to obtain higher "strengths" (both for 5% strain and for 20% strain strengths) for a constant density achieved. In other words, for a constant moulding water content increase in density results in increase in strength for non-expansive soils, tested after soaking. Fig 33 and Fig 34 . Fig 35 shows the effect of moulding water content on permeability. It is clear that higher the molding water content, smaller the permeability. The reduction in permeability attained by compaction is due to the reduction in porosity Fig 35. A word of caution, however, should be added at this point, because a well graded sand containing 30% voids may be quite permeable but a clay may be quite resistant to the passage of water with porosity approaching 50%.

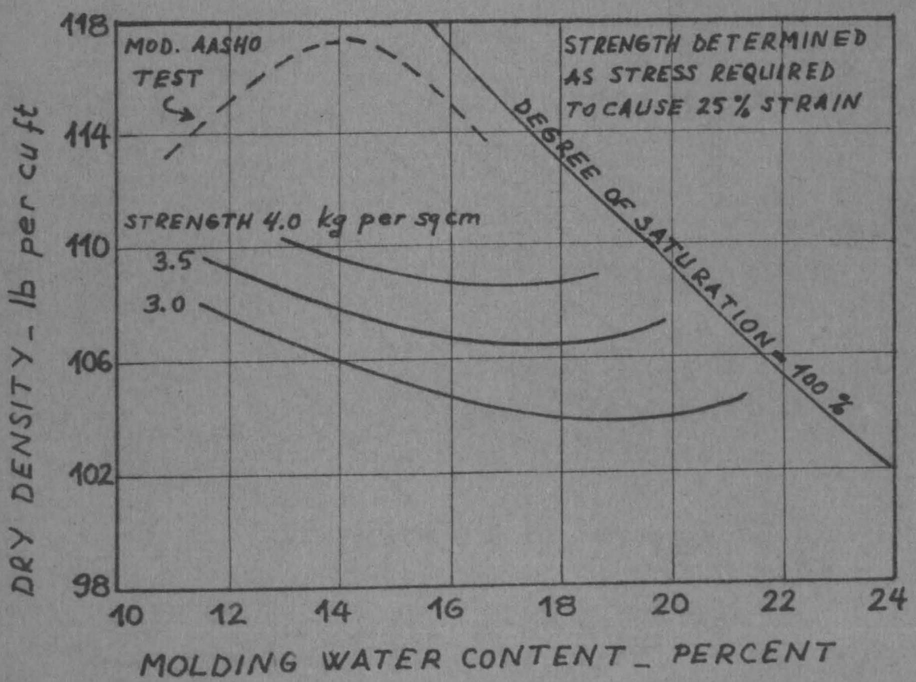


UNCONSOLIDATE

UNDRAINED TESTS -

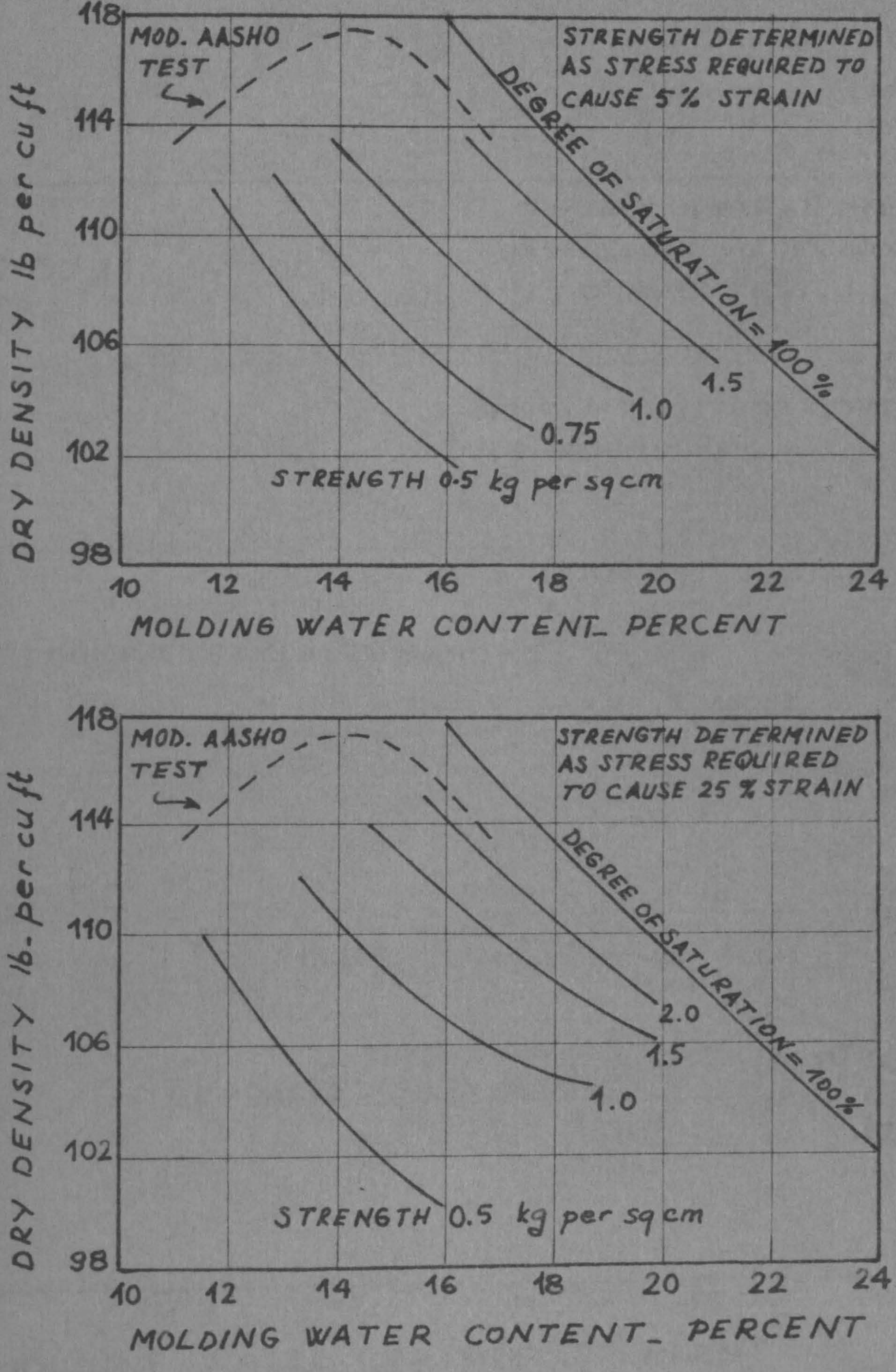
CONFINING PRESSURE :

1.0 kg per sq cm



NOTE _ LINES SHOW INITIAL COMPOSITIONS RESULTING IN EQUAL STRENGTH AFTER SWELLIN

RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SOAKING UNDER SURCHARGE PRESSURE OF 35 PSI FOR SAMPLES OF SANDY CLAY - KNEADING COMPACTION

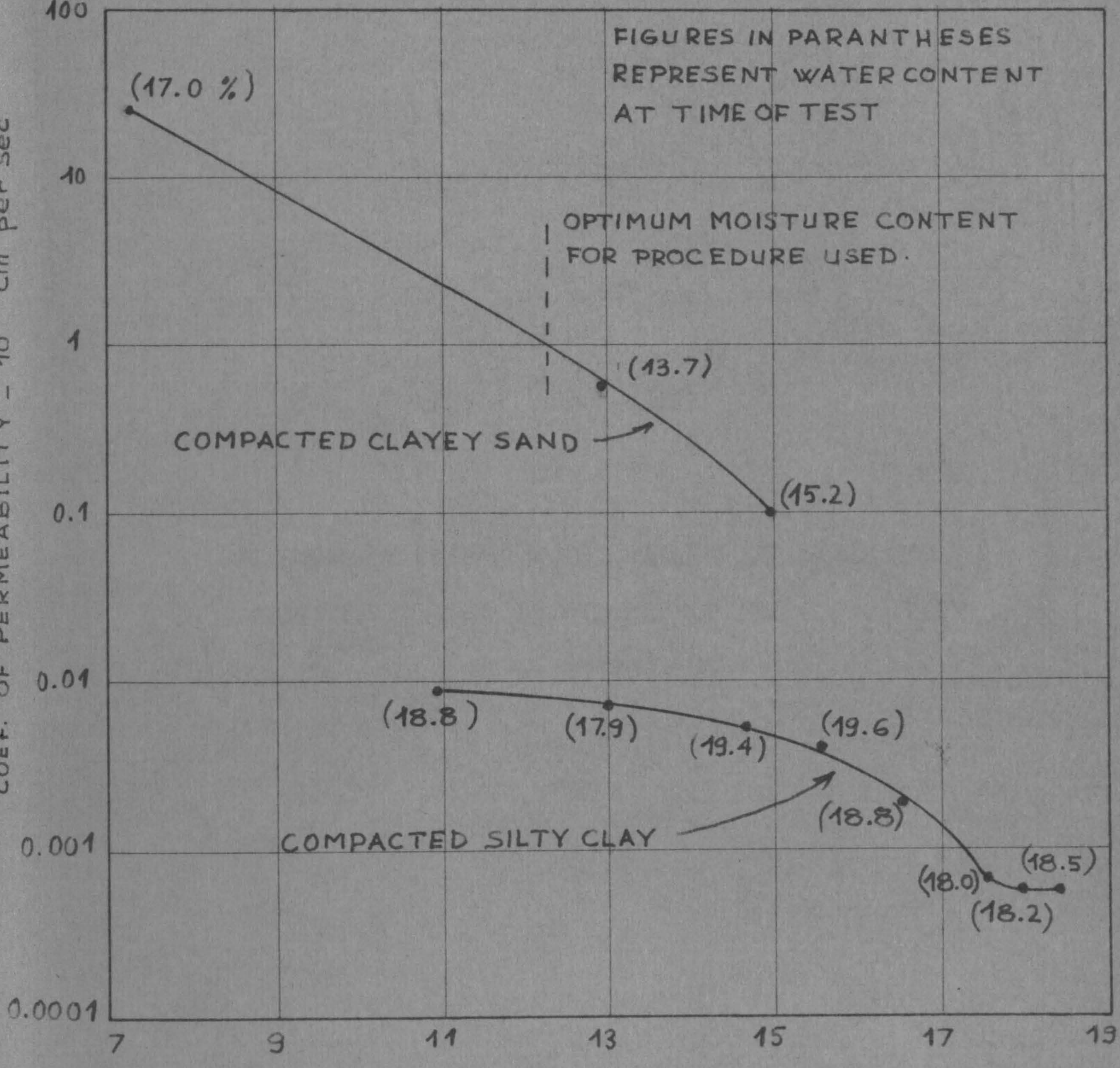


UNCONSOLIDATED UNDRAINED TESTS -

CONFINING PRESSURE : 1.0 kg per sq cm

NOTE - LINES SHOW INITIAL COMPOSITIONS RESULTING IN EQUAL STRENGTHS AFTER SWELLING

RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SWELLING UNDER SURCHARGE PRESSURE OF 1 PSI FOR SAMPLES OF SANDY CLAY - KNEADING COMPACTION



EFFECT OF MOLDING WATER CONTENT ON PERMEABILITY

FIG – 35

PART II

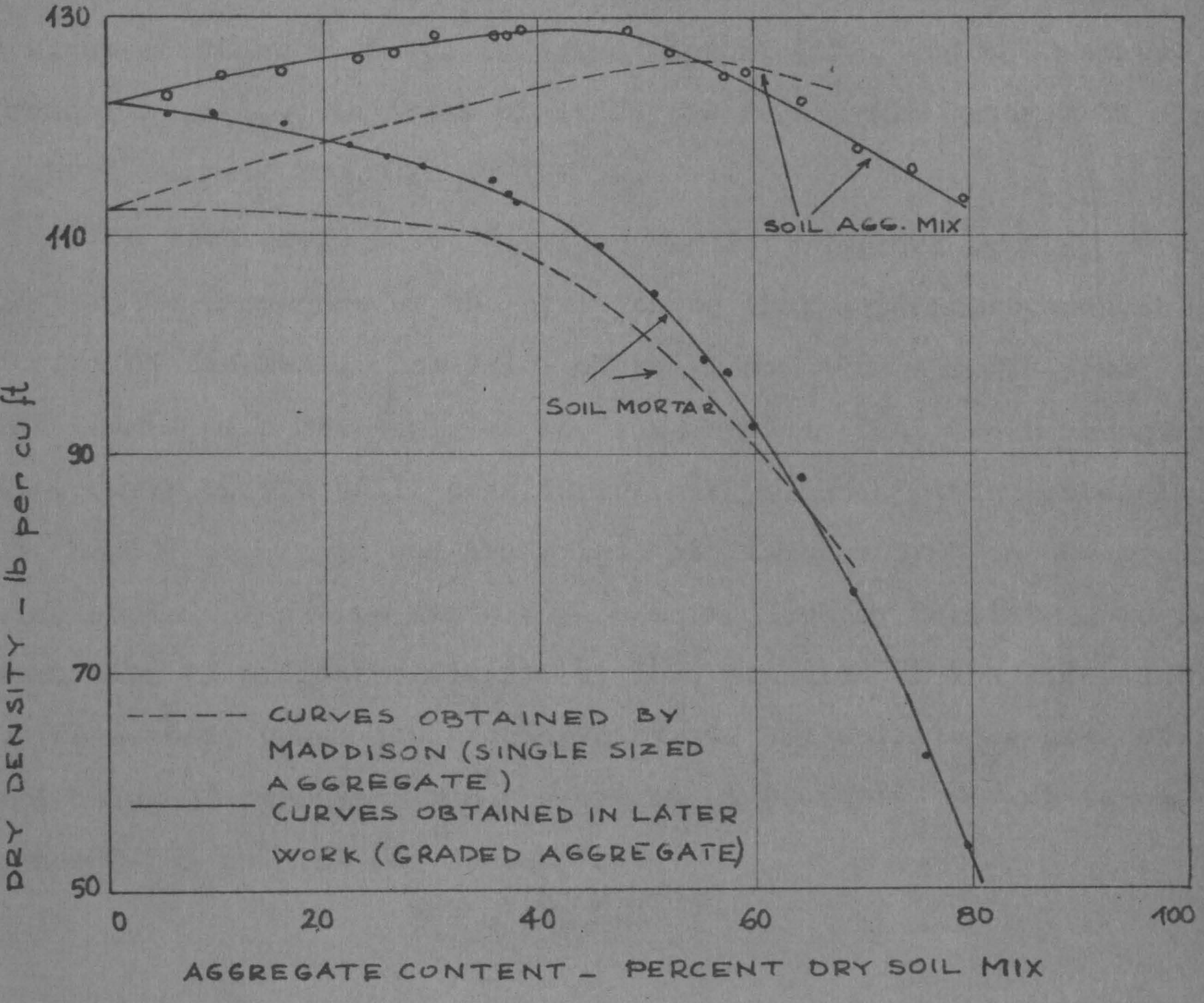
AN INVESTIGATION ON THE EFFECT OF SIZE AND AMOUNT OF COARSE AGGREGATE ON THE COMPACTION OF A BRICKEARTH

STATEMENT OF THE PROBLEM AND PREVIOUS WORK

The specific problem to be investigated is the effect of the amount of coarse particles added to samples of cohesive soil.

This problem was first investigated at the Road Research Laboratory by Maddison[15] and other workers, although in a somewhat different form; namely the added coarse material was single sized. It was found out that the admixture of single-sized aggregate upto about 25 percent of any one size (1 in-3/4 in, 3/4 in-1/2 in, 1/2 in-3/8 in) had very little effect on the compaction of the soil mortar, the stone acting merely as a displacer. At higher stone contents the dry density of the soil mortar decreased fairly rapidly and at a stone content of 65 percent had fallen to a value of 75 percent of the dry density of the soil when compacted alone. The dry density of the soil-stone mixture as a whole incresed with increasing amounts of stone up to a stone content of 40 percent, at stone contents higher than about 70 percent the stones were in contact with one another and so prevented any compaction of the soil mortar. The optimum moisture content of the soil mortar increased as the dry density decreased with addition of stone.

In later work an aggregate graded between 3/4 in and No. 7 B.S. siéve was used. The results obtained were similar to Maddison's, but the dry density of the soil mortar decreased on the addition of even small proportions of aggregate, although this decrease was not considerable until more than 45 percent of aggregate had been added. Also the dry density of the soil-aggregate



COMPACTION OF SOIL MORTAR AT OPTIMUM MOISTURE CONTENT WITH DIFFERENT PERCENTAGES OF AGGREGATE

FIG - 36

mixture incresed only slightly with increasing percentage of aggregate up to 50 percent and for higher percentages, decreased fig(36).

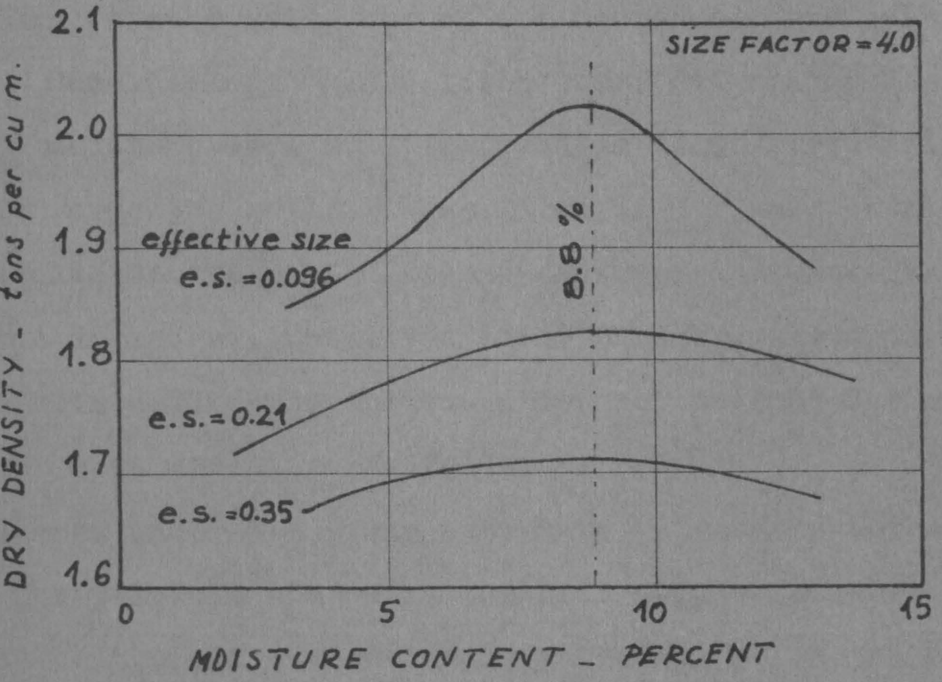
Later M.S. Youssef[16] investigated the relation between the optimum moisture content, the size factor, (SF), and the mean grain size, (d_m), in soils of different mean grain sizes with size factors ranging from 0.0 to 7.375.

The mean grain size, d_m, of any soil is defined by D.M. Burmister[17] as a measure of the position of that soil as regards the degree of fineness. The value of the mean grain size is equal to the length of a rectangle of the same area as that under the grain size curve of the soil, considering the height of this rectangle as the 100% ordinate and its origin at diameter 3/8" on the grain size scale. The size factor, also introduced by Burmister, is the summation of ordinates divided by 100, measured at the mid-point of elementary under the grading curve of the soil, each area of width log 2, starting from the origin at diameter 3/8" up to diameter 0.001 mm. It was proved that

$$\Sigma d_m = \frac{9.42}{2^{SF}}$$

where SF= y/100

The usual compaction tests were made for all these mixtures, and the typical curve giving the relation between the moisture content and the dry compacted density were drawn, fig(37), of soils of 3 different effective size, which is defined as the diameter at 10% finer, and of the same size factor. From these curves, it was



PROCTOR CURVES FOR DIFFERENT MATERIALS WITH THE SAME SIZE FACTOR found that the optimum moisture content was almost the same for a given size factor whatever the grading may be. This was interpreted as that the optimum moisture content does not depend on the grading of the soil, but on the degree of fineness of that soil.

Considering Hogentogler's theory of compaction which says "The moisture in a soil is found as films surrounding the particles that forms the soil. These films in low water content are very adhesive to the particles and as water increases this adhesive force decreases, until the limit of lubrication is reached. This limit is called the optimum moisture content" the results of these tests were explained as follows:

The area under the grain size curve gives the total surface area of the particle composing the soil sample and when we equate this area to that of the rectangle whose height is the 100% ordinate and the length equal to mean grain size d_m, we consider a soil sample composed only of grains equal to d_m.

As the adhesive force between the films of water and the soil particles decreases as water increases, i.e. as the thicknesses of these films increase, until we reach the lubrication limit, according to the same theory, it was concluded that this thickness is limited for any given soil grain at the lubrication limit, and that the thickness of the film depends on the diameter of the particles. Thus for a given mean grain size d_m whatever the grading may be we should get the same thickness of film of water at the lubrication limit, which thus means that for a given d_m we have a given opti-

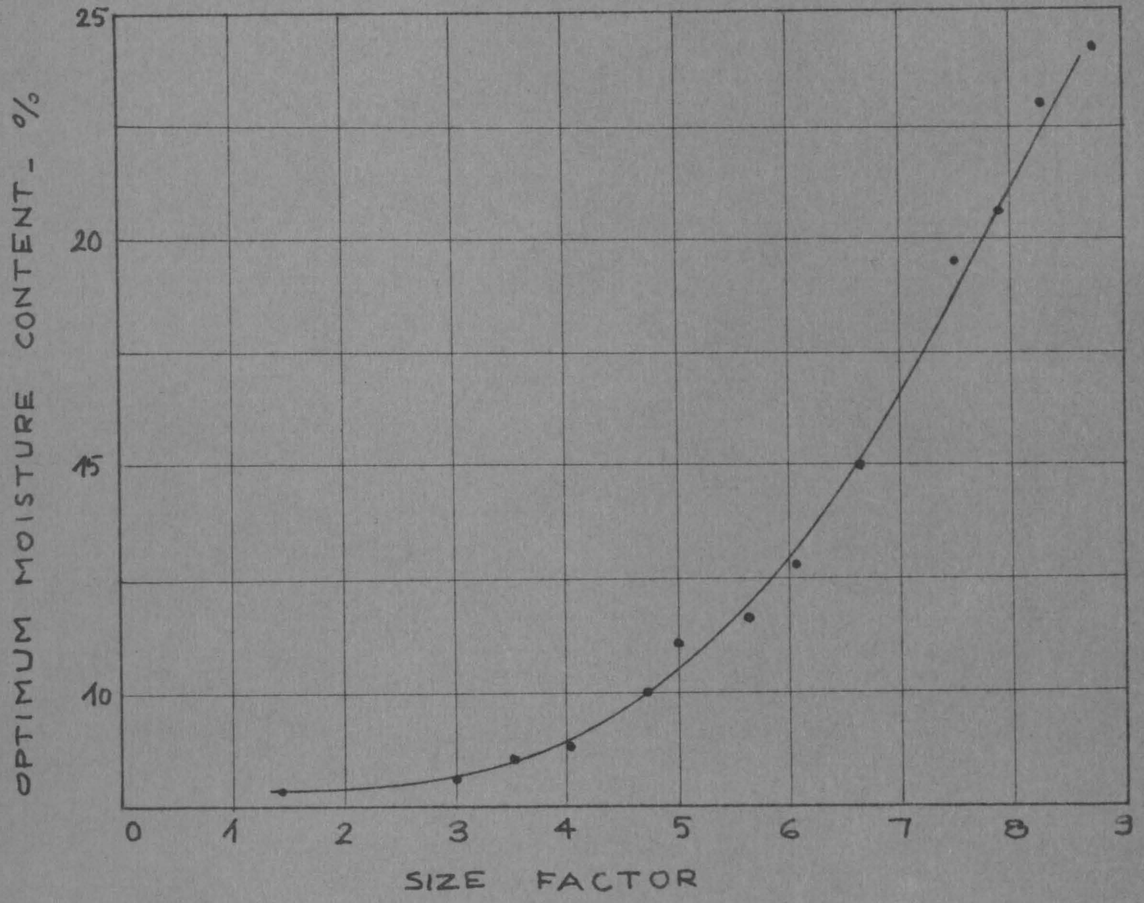

25
20
15
10
OPTIMUM MOISTURE CONTENT - %
0
1
2
3
4
5
6
7
8
9
SIZE FACTOR

FIG - 38

mum moisture content fig (38). Obviously increasing moisture content with increasing SF means it decreases with increasing d_m, since $d_m = \frac{9.42}{2^{SF}}$.

DESCRIPTION OF THE LABORATORY INVESTIGATION

Unlike the work done by Maddison in the Road Research Laboratory the aggregates added to the samples of cohesive soil were not single-sized. Three groups of aggregates were used:

1. Passing No.4 sieve- Retained on No.8 sieve
2. Passing No.10 sieve- Retained on No.20 sieve
3. Passing No.60 sieve- Retained on No.200 sieve

Various mixtures containing a percentage, between 5 and 25, of one of the above groups were prepared. For example sample c contained 9.66% by dry weight, aggregate passing No.4 sieve and retained on No.8 sieve. These samples and the natural soil, which is a brick-earth from the Topser Brick and Tile Factory site, andthe properties of which are summarized in figures(39), (40) and (41), were compacted by Standard Proctor Method. The results of these tests are shown in tables (IV) - (XVIII) and plotted in figures (42), (43) and (44).

The grain size distribution of the natural sample was also determined by sieve and hydrometer analysis, the data and the results of which are summarized in tables (XIX) and (XX), plotted in fig. (45). The grain size distributions of the other samples were derived from this key distribution fig (46) - fig (59). For example grain size distribution for sample H was derived as follows: Since the aggregate added was between diameters corresponding to No.10 and No.20 sieves, that portion of the curve to the left of No.10 sieve is not affected, i.e. it is identical with

SPECIFIC GRAVITY DETERMINATION

DETERMINATION NO.	1	2	3	4
BOTTLE NO	20	20	20	
WT. BOTTLE + WATER + SOIL, W_1, g	89.90	89.88	89.83	
TEMPERATURE, T, °C	24.6	25.1	29.4	
WT. BOTTLE + WATER, W_2, g	82.15	82.14	82.07	
EVOPORATING DISH NO.	7	7	7	
WT. DISH + DRY SOIL g	22.86	22.86	22.86	
WT. DISH g	10.86	10.86	10.86	
WT. DRY SOIL g	12.00	12.00	12.00	
SPECIFIC GRAVITY, WATER, G_T	0.9972	0.9971	0.9959	
SPECIFIC GRAVITY, SOIL, G_s	2.82	2.81	2.82	

$$G_s = \frac{G_T W_s}{W_s - W_1 + W_2}$$

$G_s = 2.82$

ATTERBERG LIMITS DETERMINATION

PLASTIC LIMIT

DETERMINATION NO.	1
CONTAINER NO.	5
WT. CONTAINER + WET SOIL IN g	16.50
WT. CONTAINER + DRY SOIL IN g	15.35
WT. WATER, W_w, IN g	1.15
WT. CONTAINER IN g	10.96
WT. DRY SOIL, W_s, IN g	4.39
WATER CONTENT, ω, IN %	26.2

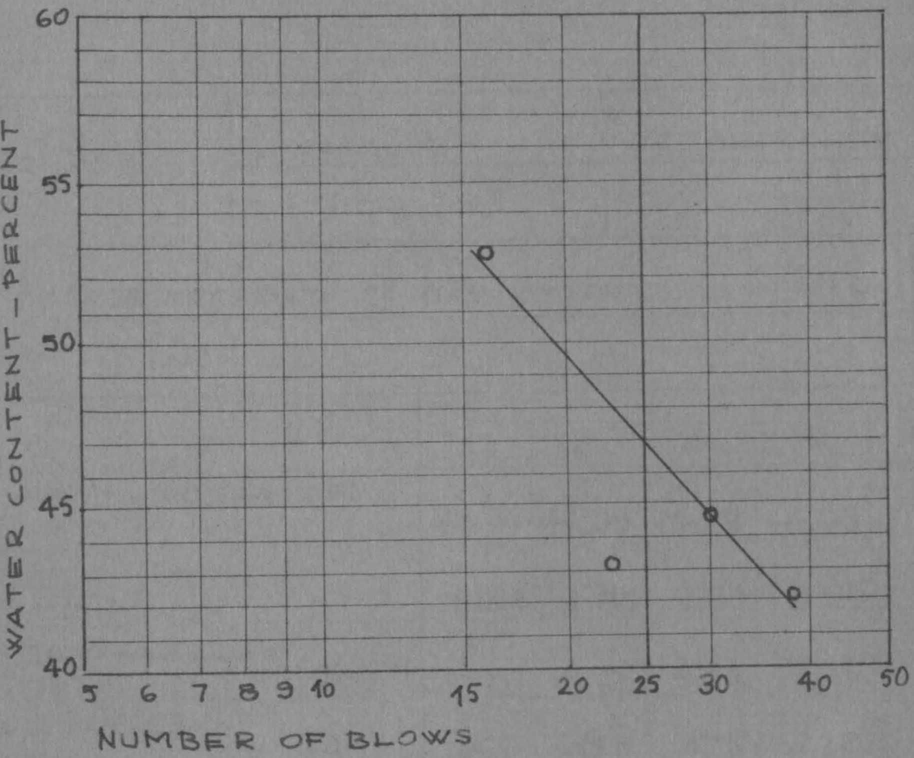
NATURAL WATER CONTENT

1	2
S-10	
88.5	
84.3	
4.2	
13.5	
70.8	
6.0	

LIQUID LIMIT

DETERMINATION NO.	1	2	3	4
NO. OF BLOWS	16	23	30	38
CONTAINER NO.	1	2	3	4
WT. CONTAINER + WET SOIL IN g	24.50	25.15	23.50	23.62
WT. CONTAINER + DRY SOIL IN g	19.75	20.72	19.47	19.65
WT. WATER, W_w, IN g	4.75	4.43	4.03	4.03
WT. CONTAINER IN g	10.71	10.50	10.43	10.32
WT. DRY SOIL, W_s, IN g	9.04	10.22	9.04	9.33
WATER CONTENT, ω, IN %	52.6	43.3	44.6	42.2

FLOW CURVE



SHRINKAGE LIMIT

DETERMINATION NO	1
UNDISTURBED OR REMOLDED SOIL PAT	REMOLDED
WT DRY SOIL PAT, W_s, g	21.7
WT. CONTAINER + Hg, g	188.5
WT. CONTAINER, g	24.5
WT. Hg, g	164.0
VOL SOIL PAT, cc	12.1
SHRINKAGE LIMIT, %	20.3

RESULT SUMMARY	
PLASTIC LIMIT	26.2
NATURAL ω	6.0
LIQUID LIMIT	47.0
SHRINKAGE LIMIT	20.3
PLASTICITY INDEX	20.8

SAMPLE: A

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	—	—
AMOUNT OF COARSE PRESENT, % DRY WT. OF CLAY		0

	NO.	1	2	3	4	5
DATA FOR CURVES	WT. OF MOLD + COMPACTED SOIL – lb	13.71	14.09	14.29	14.09	
	WT. OF MOLD – lb	9.84	9.84	9.84	9.84	
	WT OF COMPACTED SOIL – lb W	3.87	4.25	4.45	4.25	
	AVERAGE WATER CONTENT, ω	.066	.153	.175	.228	
	DRY DENSITY = $\frac{30\,W}{1+\omega}$ $\frac{lb}{cu\,ft}$	109.0	110.3	113.5	103.9	

WATER CONTENTS	TARE + WET SOIL – gm	95.1	113.3	107.4	120.0	
	TARE + DRY SOIL – gm	90.0	101.4	93.5	100.2	
	WT OF WATER – gm	5.1	11.9	13.9	19.8	
	TARE – gm	12.5	13.6	14.3	13.6	
	WT OF DRY SOIL – gm	77.5	77.8	79.2	86.6	
	WATER CONTENT – %	6.6	15.3	17.5	22.8	

TABLE – IV

SAMPLE : B

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO. 4 SIEVE	NO. 8 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		5.06

DATA FOR CURVES	NO.		1	2	3	4	5
	WT OF MOLD + COMPACTED SOIL – lb		13.40	13.60	13.90	13.78	13.70
	WT OF MOLD – lb		9.84	9.84	9.84	9.84	9.84
	WT OF COMPACTED SOIL – lb	W	3.56	3.76	4.06	3.94	3.86
	AVERAGE WATER CONTENT	ω	0.031	0.068	0.23	0.27	0.292
	DRY DENSITY = $\frac{30W}{1+\omega}$	$\frac{lb}{cu\ ft}$	103.5	105.8	99.1	93.0	89.5

WATER CONTENTS	TARE + WET SOIL	– gm					
	TARE + DRY SOIL	– gm					
	WT OF WATER	– gm	3.0	6.4	18.7	21.3	22.6
	TARE	– gm					
	WT OF DRY SOIL	– gm	97.0	93.6	81.3	78.7	77.4
	WATER CONTENT	– %	3.1	6.8	23.0	27.0	29.2

TABLE – V

SAMPLE : C

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO. 4 SIEVE	NO. 8 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		9.66

	NO.		1	2	3	4	5
DATA FOR CURVES	WT OF MOLD + COMPACTED SOIL – lb		13.7	14.2	14.3	14.1	14.0
	WT OF MOLD – lb		9.9	9.9	9.9	9.9	9.9
	WT OF COMPACTED SOIL – lb	W	3.8	4.3	4.4	4.2	4.1
	AVERAGE WATER CONTENT	ω	0.052	0.137	0.167	0.224	0.272
	DRY DENSITY = $\frac{30W}{1+\omega}$	$\frac{lb}{cu\ ft}$	107.0	114.0	112.0	103.0	95.0

WATER CONTENTS	TARE + WET SOIL	– gm	108.5	128.0	128.0	124.0	119.0
	TARE + DRY SOIL	– gm	105.0	117.0	115.0	108.0	101.5
	WT OF WATER	– gm	3.5	11.0	13.0	16.0	17.5
	TARE	– gm	37.0	37.0	37.0	36.5	37.0
	WT OF DRY SOIL	– gm	68.0	80.0	78.0	71.5	64.5
	WATER CONTENT	– %	5.2	13.7	16.7	22.4	27.2

TABLE – VI

SAMPLE: D

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 4 SIEVE	NO. 8 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		13.88

	NO.		1	2	3	4	5
DATA FOR CURVES	WT OF MOLD + COMPACTED SOIL – lb		12.50	13.08	13.15	12.90	
	WT OF MOLD – lb		8.60	8.60	8.60	8.60	
	WT OF COMPACTED SOIL – lb	W	3.90	4.48	4.55	4.30	
	AVERAGE WATER CONTENT	ω	0.078	0.155	0.190	0.238	
	DRY DENSITY = $\frac{30W}{1+\omega}$	$\frac{lb}{cu\,ft}$	109.0	116.4	114.2	104.0	

WATER CONTENTS	TARE + WET SOIL	– gm	55.3	55.3	76.0	84.2	
	TARE + DRY SOIL	– gm	52.2	49.7	66.2	70.6	
	WT OF WATER	– gm	3.1	5.6	9.8	13.6	
	TARE	– gm	12.5	13.6	14.3	13.6	
	WT OF DRY SOIL	– gm	39.7	36.1	51.7	57.0	
	WATER CONTENT	– %	7.8	15.5	19.0	23.8	

TABLE – VII

SAMPLE : E

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 4 SIEVE	NO 8 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		17.71

	NO.		1	2	3	4	5
DATA FOR CURVES	WT OF MOLD + COMPACTED SOIL - lb		13.68	13.90	14.23	13.88	
	WT OF MOLD - lb		9.84	9.84	9.84	9.84	
	WT OF COMPACTED SOIL - lb	W	3.84	4.06	4.39	4.04	
	AVERAGE WATER CONTENT	ω	0.060	0.095	0.163	0.215	
	DRY DENSITY = $\frac{30W}{1+\omega}$	$\frac{lb}{cu\,ft}$	108.4	111.2	113.8	105.0	

WATER CONTENTS	TARE + WET SOIL - gm	76.5	98.2	91.6	104.5	
	TARE + DRY SOIL - gm	72.9	91.4	80.9	88.4	
	WT OF WATER - gm	3.6	7.4	10.7	16.1	
	TARE - gm	12.5	13.6	14.3	13.6	
	WT OF DRY SOIL - gm	60.4	77.8	66.6	74.8	
	WATER CONTENT - %	6.0	9.5	16.3	21.5	

TABLE - VIII

SAMPLE : F

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 4 SIEVE	NO 8 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		21.25

			1	2	3	4	5
DATA FOR CURVES	NO.		1	2	3	4	5
	WT OF MOLD + COMPACTED SOIL - lb		13.73	14.04	14.33	14.18	
	WT OF MOLD - lb		9.84	9.84	9.84	9.84	
	WT OF COMPACTED SOIL - lb	W	3.89	4.20	4.49	4.24	
	AVERAGE WATER CONTENT	ω	0.064	0.112	0.159	0.199	
	DRY DENSITY - $\frac{30W}{1+\omega}$ $\frac{lb}{cu\ ft}$		110.0	112.6	116.8	111.3	

WATER CONTENTS	TARE + WET SOIL	- gm	90.0	115.9	120.1	119.7	
	TARE + DRY SOIL	- gm	85.5	105.5	105.6	102.1	
	WT OF WATER	- gm	4.5	10.4	14.5	17.6	
	WT OF DRY SOIL	- gm	72.1	92.6	91.2	88.6	
	TARE	- gm	13.4	12.9	14.4	13.5	
	WATER CONTENT	- %	6.4	11.2	15.9	19.9	

TABLE - IX

SAMPLE : G

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 10 SIEVE	NO 20 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		5.06

DATA FOR CURVES	NO.		1	2	3	4	5
	WT OF MOLD + COMPACTED SOIL – lb		12.60	12.80	13.30	13.00	12.90
	WT OF MOLD – lb		9.00	9.00	9.00	9.00	9.00
	WT OF COMPACTED SOIL – lb	W	3.60	3.80	4.30	4.00	3.90
	AVERAGE WATER CONTENT	ω	0.067	0.100	0.200	0.250	0.290
	DRY DENSITY= $\frac{30W}{1+\omega}$	$\frac{lb}{cu\,ft}$	103.0	105.0	108.0	96.7	91.0

WATER CONTENTS	TARE + WET SOIL	– gm	230.0	225.6	193.8	337.2	316.5
	TARE + DRY SOIL	– gm	217.6	208.3	167.7	277.5	253.5
	WT OF WATER	– gm	12.4	17.3	26.1	59.7	63.0
	TARE	– gm	37.0	37.0	37.0	37.0	37.0
	WT OF DRY SOIL	– gm	180.6	171.3	130.7	240.3	216.5
	WATER CONTENT	– %	6.7	10.0	20.0	25.0	29.0

TABLE – X

SAMPLE : H

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 10 SIEVE	NO 20 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		9.66

DATA FOR CURVES	NO.		1	2	3	4	5
	WT OF MOLD + COMPACTED SOIL – lb		13.40	13.70	14.18	14.06	13.91
	WT OF MOLD – lb		9.84	9.84	9.84	9.84	9.84
	WT OF COMPACTED SOIL – lb	W	3.56	3.86	4.34	4.22	4.07
	AVERAGE WATER CONTENT	ω	0.023	0.112	0.178	0.217	0.260
	DRY DENSITY = $\frac{30\,W}{1+\omega}$	$\frac{lb}{cu\,ft}$	104.0	104.5	110.0	103.5	96.7

WATER CONTENTS	TARE + WET SOIL	– gm	131.4	138.1	132.5	131.5	111.0
	TARE + DRY SOIL	– gm	129.0	126.8	116.5	112.8	93.6
	WT OF WATER	– gm	2.4	11.3	16.0	18.7	17.4
	TARE	– gm					
	WT OF DRY SOIL	– gm	102.3	100.1	89.8	86.1	66.9
	WATER CONTENT	– %	2.3	11.2	17.8	21.7	26.0

TABLE - XI

SAMPLE: 1

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 10 SIEVE	NO 20 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		13.88

DATA FOR CURVES							
	NO.		1	2	3	4	5
	WT OF MOLD + COMPACTED SOIL – lb		13.60	14.12	14.17	14.00	
	WT OF MOLD – lb		9.84	9.84	9.84	9.84	
	WT OF COMPACTED SOIL – lb	W	3.76	4.28	4.33	4.16	
	AVERAGE WATER CONTENT	ω	0.023	0.115	0.207	0.250	
	DRY DENSITY = $\frac{30W}{1+\omega}$	$\frac{lb}{cu\,ft}$	110.0	115.5	108.0	100.0	

WATER CONTENTS						
	TARE + WET SOIL – gm	102.3	147.8	130.4	141.6	
	TARE + DRY SOIL – gm	101.1	132.5	112.8	120.3	
	WT OF WATER – gm	1.2	15.3	17.6	21.3	
	TARE – gm	37.0	37.2	37.1	37.0	
	WT OF DRY SOIL – gm	64.1	95.3	75.7	83.3	
	WATER CONTENT – %	2.3	11.5	20.7	25.2	

TABLE – XII

SAMPLE: J

SIZE OF COARSE PRESENT	PASSING NO 10 SIEVE	RETAINED ON NO 20 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		17.71

DATA FOR CURVES	NO.		1	2	3	4	5
	WT OF COMPACTED SOIL + MOLD - lb		12.70	13.18	13.15	13.00	12.93
	WT OF MOLD - lb		8.60	8.60	8.60	8.60	8.60
	WT OF COMPACTED SOIL - lb	W	4.10	4.58	4.55	4.40	4.33
	AVERAGE WATER CONTENT	ω	0.034	0.161	0.195	0.206	0.237
	DRY DENSITY= $\frac{30W}{1+\omega}$	$\frac{lb}{cu\ ft}$	118.3	117.3	113.1	107.8	99.1

WATER CONTENTS						
	TARE + WET SOIL - gm	99.3	126.6	126.6	126.2	126.2
	TARE + DRY SOIL - gm	96.9	112.7	110.3	109.1	107.0
	WT OF WATER - gm	2.4	13.9	16.3	17.1	19.2
	TARE - gm	26.0	26.6	26.6	26.2	26.2
	WT OF DRY SOIL - gm	70.9	86.1	83.7	82.9	80.8
	WATER CONTENT - %	3.4	16.1	19.5	20.6	23.7

TABLE - XIII

SAMPLE : K

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 10 SIEVE	NO 20 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		21.25

DATA FOR CURVES	NO.		1	2	3	4	5
	WT OF MOLD + COMPACTED SOIL – lb		12.70	13.11	13.61	13.51	
	WT OF MOLD – lb		9.84	9.84	9.84	9.84	
	WT OF COMPACTED SOIL – lb	W	2.86	3.27	3.77	3.67	
	AVERAGE WATER CONTENT	ω	0.053	0.097	0.167	0.180	
	DRY DENSITY = $\frac{30W}{1+\omega}$	lb / cu ft	81.3	93.0	97.0	93.2	

WATER CONTENTS	TARE + WET SOIL	– gm	150.7	149.8	141.4	132.5	
	TARE + DRY SOIL	– gm	143.1	137.2	126.5	117.2	
	WT OF WATER	– gm	7.6	12.6	14.9	15.3	
	TARE	– gm	32.4	37.4	37.1	33.6	
	WT OF DRY SOIL	– gm	110.7	99.8	89.4	83.6	
	WATER CONTENT	– %	5.3	9.7	16.7	18.0	

TABLE – XIV

SAMPLE : L

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 60 SIEVE	NO 200 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		5.06

DATA FOR CURVES						
	NO.	1	2	3	4	5
	WT. OF MOLD + COMPACTED SOIL – lb	13.81	14.24	14.19	14.07	
	WT. OF MOLD – lb	9.84	9.84	9.84	9.84	
	WT. OF COMPACTED SOIL – lb W	3.97	4.40	4.35	4.23	
	AVERAGE WATER CONTENT ω	0.083	0.152	0.195	0.230	
	DRY DENSITY = $\frac{30W}{1+\omega}$ $\frac{lb}{cu\,ft}$	109.2	114.4	107.8	102.1	

WATER CONTENTS						
	TARE + WET SOIL – gm	143.5	119.2	106.8	100.0	
	TARE + DRY SOIL – gm	133.5	105.3	91.7	83.8	
	WT OF WATER – gm	10.0	13.9	15.1	16.2	
	TARE – gm	12.5	13.6	14.3	13.6	
	WT OF DRY SOIL – gm	121.0	91.7	77.4	70.2	
	WATER CONTENT – %	8.3	15.2	19.5	23.0	

TABLE – XV

SAMPLE : M

SIZE OF COARSE PRESENT	PASSING NO 60 SIEVE	RETAINED ON NO 200 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		9.66

		1	2	3	4	5
DATA FOR CURVES	NO.	1	2	3	4	5
	WT OF MOLD + COMPACTED SOIL – lb	13.71	14.10	14.27	14.10	
	WT OF MOLD – lb	9.84	9.84	9.84	9.84	
	WT OF COMPACTED SOIL – lb W	3.87	4.26	4.43	4.26	
	AVERAGE WATER CONTENT ω	0.067	0.127	0.178	0.215	
	DRY DENSITY = $\frac{30W}{1+\omega}$ $\frac{lb}{cu\ ft}$	109.0	113.1	112.1	104.1	

WATER CONTENTS	TARE + WET SOIL – gm	101.3	90.0	105.7	89.8	
	TARE + DRY SOIL – gm	95.8	81.3	91.9	76.3	
	WT OF WATER – gm	5.5	8.7	13.8	13.5	
	TARE – gm	13.4	12.9	14.4	13.5	
	WT OF DRY SOIL – gm	82.4	68.4	77.5	62.8	
	WATER CONTENT – %	6.7	12.7	17.8	21.5	

TABLE – XVI

SAMPLE : N

SIZE OF COARSE PRESENT	PASSING NO 60 SIEVE	RETAINED ON NO 200 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		13.88

	NO.		1	2	3	4	5
DATA FOR CURVES	WT OF MOLD + COMPACTED SOIL - lb		13.70	14.22	14.30	14.10	
	WT OF MOLD - lb		9.84	9.84	9.84	9.84	
	WT OF COMPACTED SOIL - lb	W	3.86	4.38	4.46	4.26	
	AVERAGE WATER CONTENT	ω	0.059	0.132	0.175	0.222	
	DRY DENSITY = $\frac{30\,W}{1+\omega}$	$\frac{lb}{cu\ ft}$	108.2	115.8	114.0	104.5	

WATER CONTENTS	TARE + WET SOIL - gm	82.8	133.9	80.9	97.6	
	TARE + DRY SOIL - gm	78.9	119.8	70.9	82.5	
	WT OF WATER - gm	3.9	14.1	10.0	15.1	
	TARE - gm	12.5	13.6	13.6	14.3	
	WT OF DRY SOIL - gm	66.4	106.2	57.3	68.2	
	WATER CONTENT - %	5.9	13.2	17.5	22.2	

TABLE – XVII

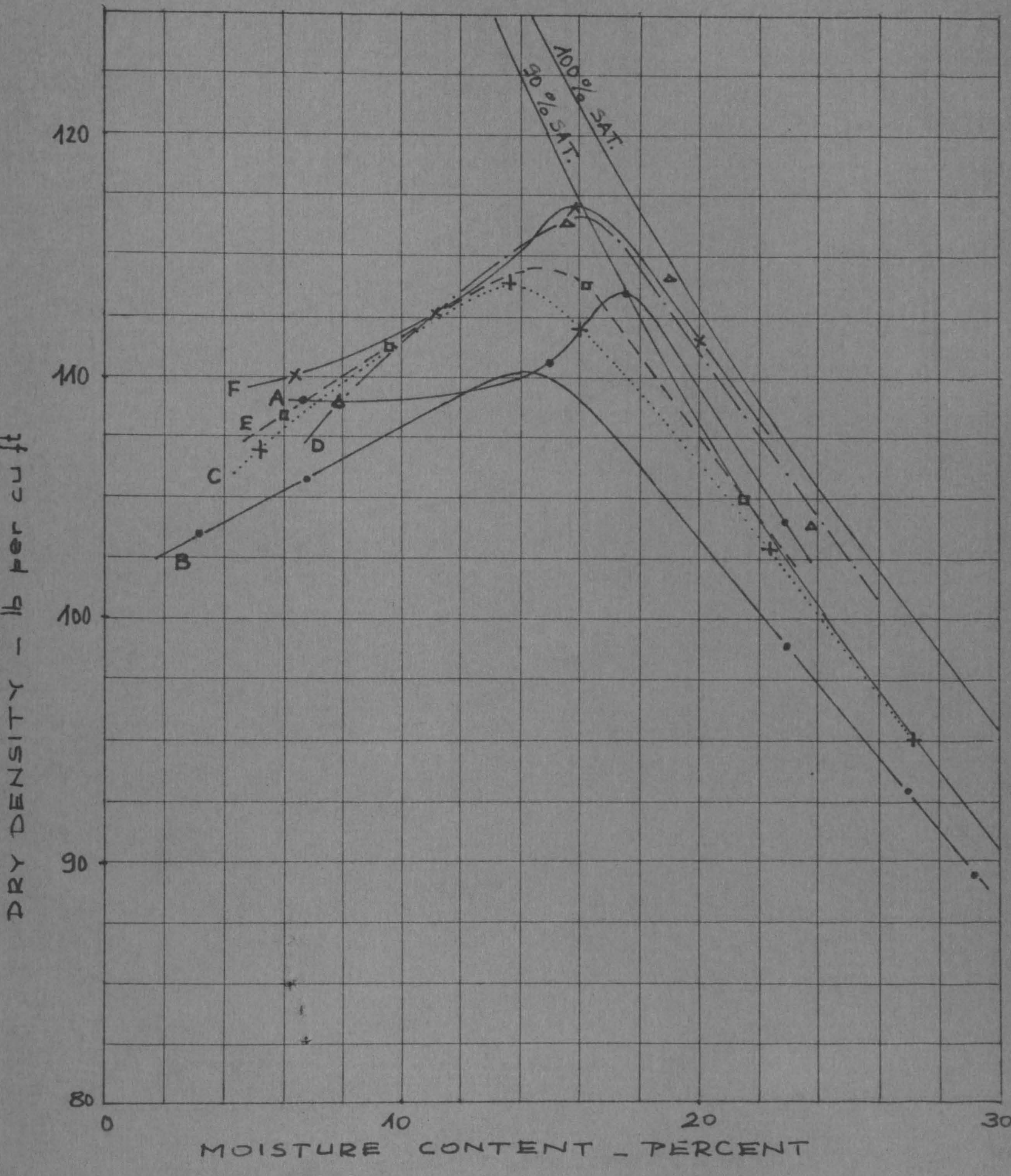
SAMPLE : O

SIZE OF COARSE PRESENT	PASSING	RETAINED ON
	NO 60 SIEVE	NO 200 SIEVE
AMOUNT OF COARSE PRESENT, % DRY WT OF CLAY		17.71

DATA FOR CURVES	NO.		1	2	3	4	5
	WT OF MOLD + COMPACTED SOIL – lb		13.77	14.28	14.39	14.18	
	WT OF MOLD – lb		9.84	9.84	9.84	9.84	
	WT OF COMPACTED SOIL – lb	W	3.93	4.44	4.55	4.34	
	AVERAGE WATER CONTENT	ω	0.068	0.133	0.156	0.202	
	DRY DENSITY = $\frac{30W}{1+\omega}$	$\frac{lb}{cu\ ft}$	110.0	117.0	115.0	107.0	

WATER CONTENTS						
	TARE + WET SOIL – gm	96.6	139.8	94.3	105.4	
	TARE + DRY SOIL – gm	91.3	124.9	83.5	90.0	
	WT OF WATER – gm	5.3	14.9	10.8	15.4	
	TARE – gm	13.4	12.9	14.4	13.5	
	WT OF DRY SOIL – gm	77.9	112.0	69.1	76.5	
	WATER CONTENT – %	6.8	13.3	15.6	20.2	

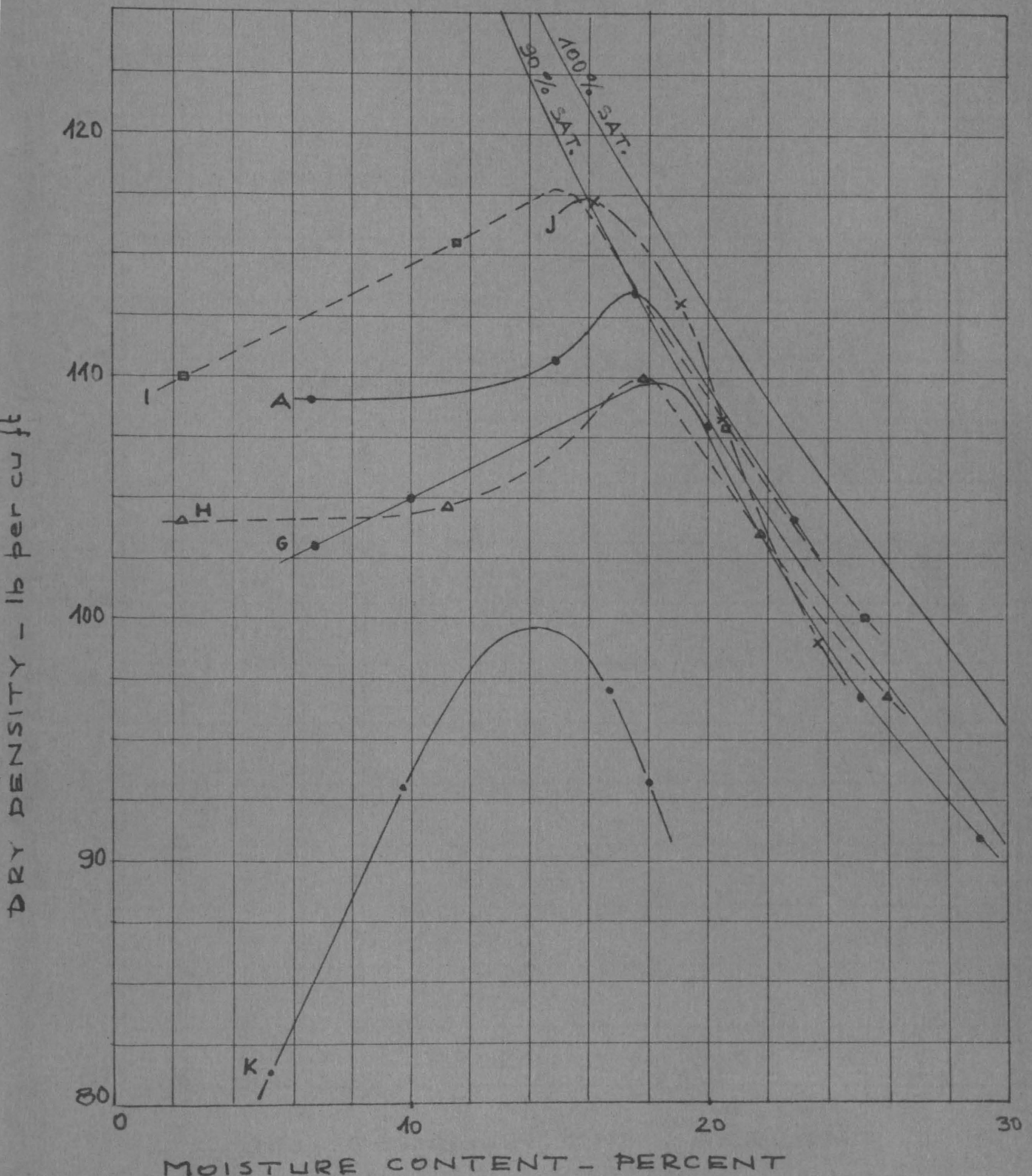
TABLE - XVIII



A _ CLAY ONLY

B _ F - SAMPLES CONTAINING VARIOUS PERCENTAGES OF MATERIAL PASSING NO. 4 SIEVE AND RETAINED ON NO. 8 SIEVE

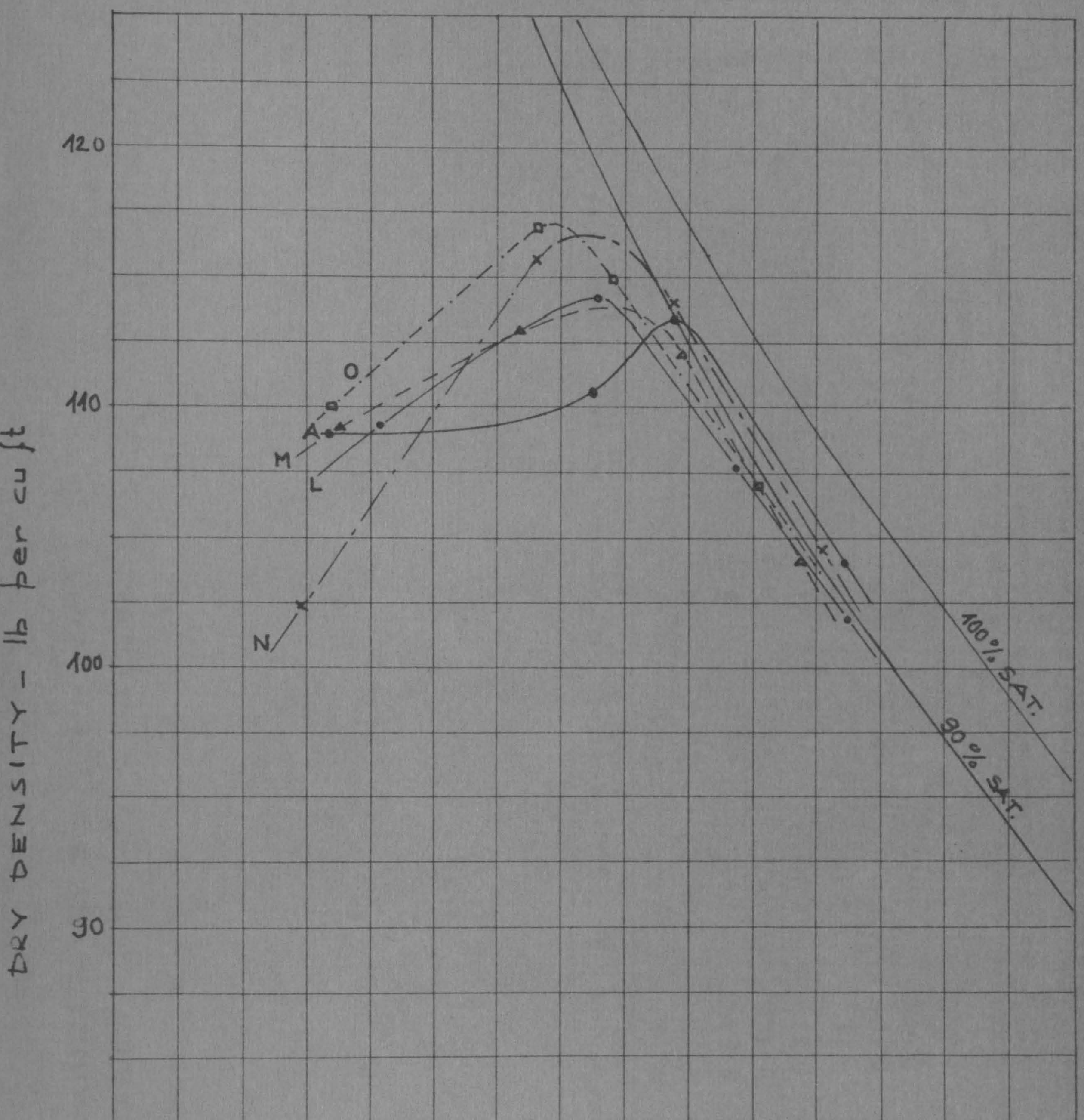
FIG. 42



A - CLAY ONLY

G - K - SAMPLES CONTAINING VARIOUS PERCENTAGES OF MATERIAL PASSING NO 10 SIEVE RETAINED ON NO 20 SIEVE.

FIG. 43



A _ CLAY ONLY

L _ O - SAMPLES CONTAINING VARIOUS PERCENTAGES OF MATERIAL PASSING NO 60 SIEVE RETAINED ON NO 200 SIEVE.

FIG_ 44

SIEVE ANALYSIS

SAMPLE – A

WT. CONTAINER + DRY SOIL, g = 2814

WT CONTAINER, g = 2502

WT DRY SOIL, W_s, g = 312

SIEVE NO.	SIEVE OPEN. mm	WT SIEVE g	WT SIEVE + SOIL g	WT RETAINED SOIL	PERCENT RETAINED	CUMULATIVE % RETAINED	% FINER
4	4.76	64.1	65.0	0.9	0	0	100.0
10	2.00	64.1	68.4	4.3	1.38	1.38	98.6
20	0.84	64.1	69.8	5.8	1.83	3.21	96.8
40	0.42	64.1	72.1	8.0	2.65	5.77	94.2
60	0.25	64.1	75.5	11.4	3.66	9.43	90.6
100	0.15	64.1	87.6	23.5	7.53	16.96	83.0
140	0.104	64.1	80.3	16.2	5.20	22.16	77.8
200	0.074	64.1	79.0	14.9	4.77	26.93	73.0

REMARK: 246 g OF SOIL WAS WASHED THROUGH NO. 200 SIEVE, OF WHICH 55 g WAS USED IN THE HYDROMETER ANALYSIS

TABLE XIX

HYDROMETER ANALYSIS

SAMPLE - A

SPECIFIC GRAVITY = 2.82

WT CONTAINER + DRY SOIL = 520 g

WT CONTAINER = 465 g

WT DRY SOIL, W_s = 55 g

HYDROMETER NO. 23786

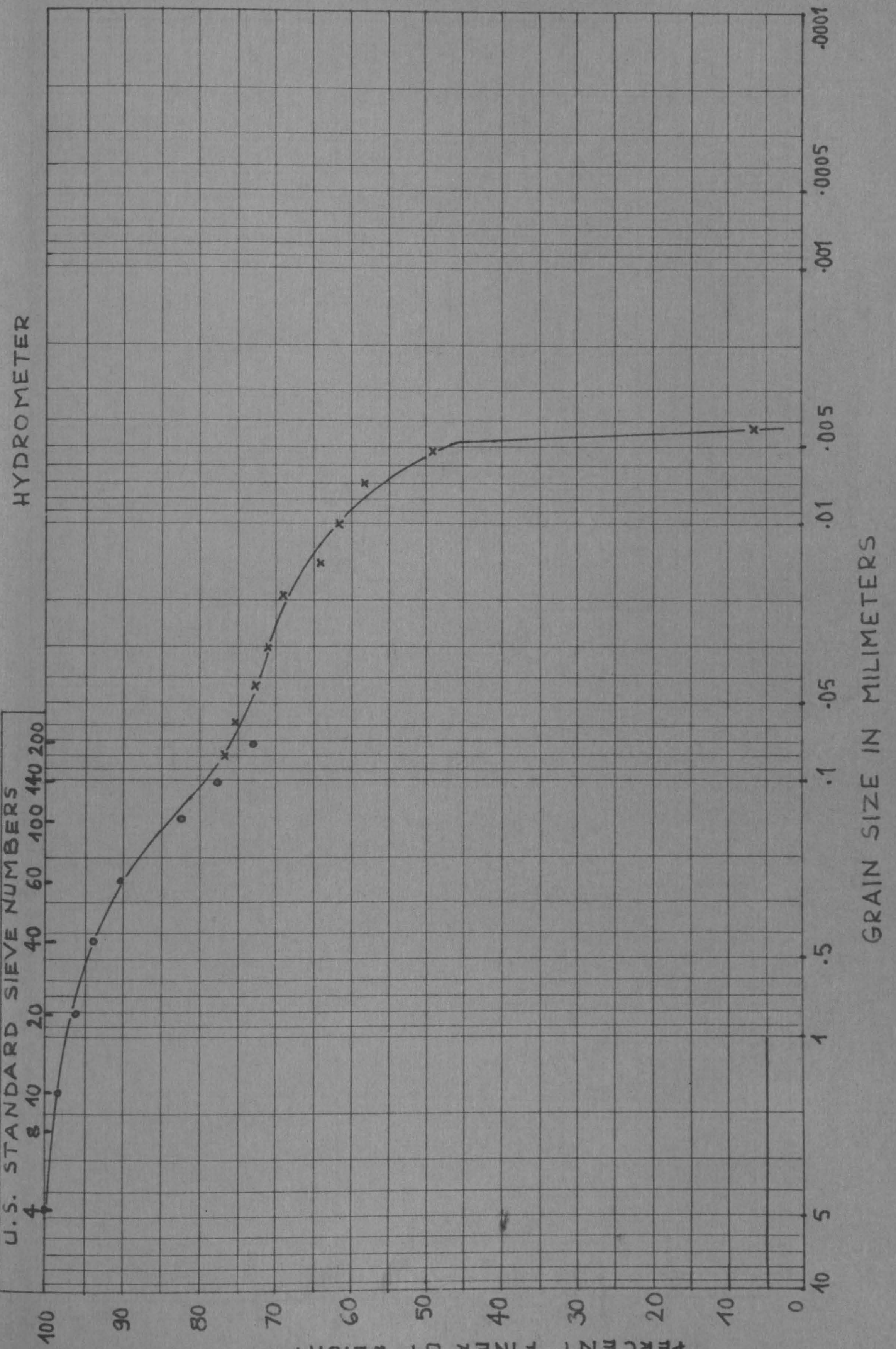
$$N = \frac{G}{G-1} \gamma_c (r - r_w) \times 100 = 2.82 (R - R_w)$$

$$N' = \% \text{ FINER NO. } 200 \times N = 2.22 (R - R_w)$$

$$D. \text{ IN mm} = K\sqrt{\frac{Z_r}{t}} = 0.013\sqrt{\frac{Z_r \text{ in cm}}{t \text{ in min}}}$$

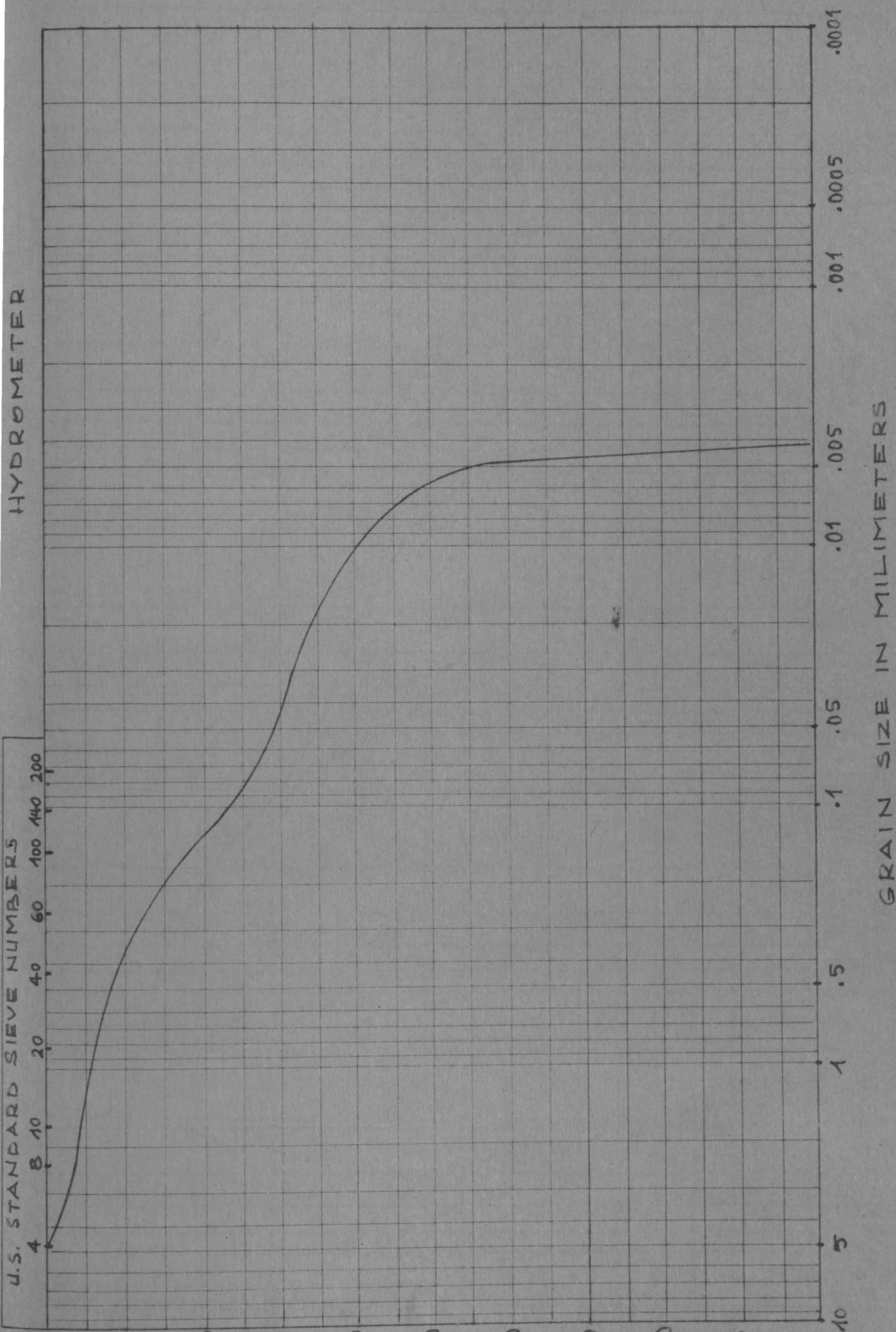
DATE	TIME min	R	R_w	TEMP. °C	$R-R_w$	Z_r cm	$\sqrt{\frac{Z_r}{t}}$	D mm	N	N′
2/1/65	1/4	34.5	0	20.0	34.5	9.8	6.2	0.080		76.5
	1/2	34			34.	10.0	4.5	0.059		75.5
	1	33			33	10.3	3.2	0.042		73.0
	2	32			32	10.5	2.3	0.030		71.0
	2	32			32	10.5	2.3	0.030		71.0
	5	31			31	10.7	1.46	0.019		69.0
	10	29	0	20.5	29	11.1	1.05	0.014		64.0
	20	28			28	11.3	0.75	.0098		62.0
	40	26			26	11.7	0.54	.0070		58.0
	80	22			22	12.5	0.40	.0052		49.0
	160	3	0	20.0	3	16.3	0.32	.0042		6.7

TABLE - XX



SAMPLE - A SF = 5.57 d_m = 0.200 mm

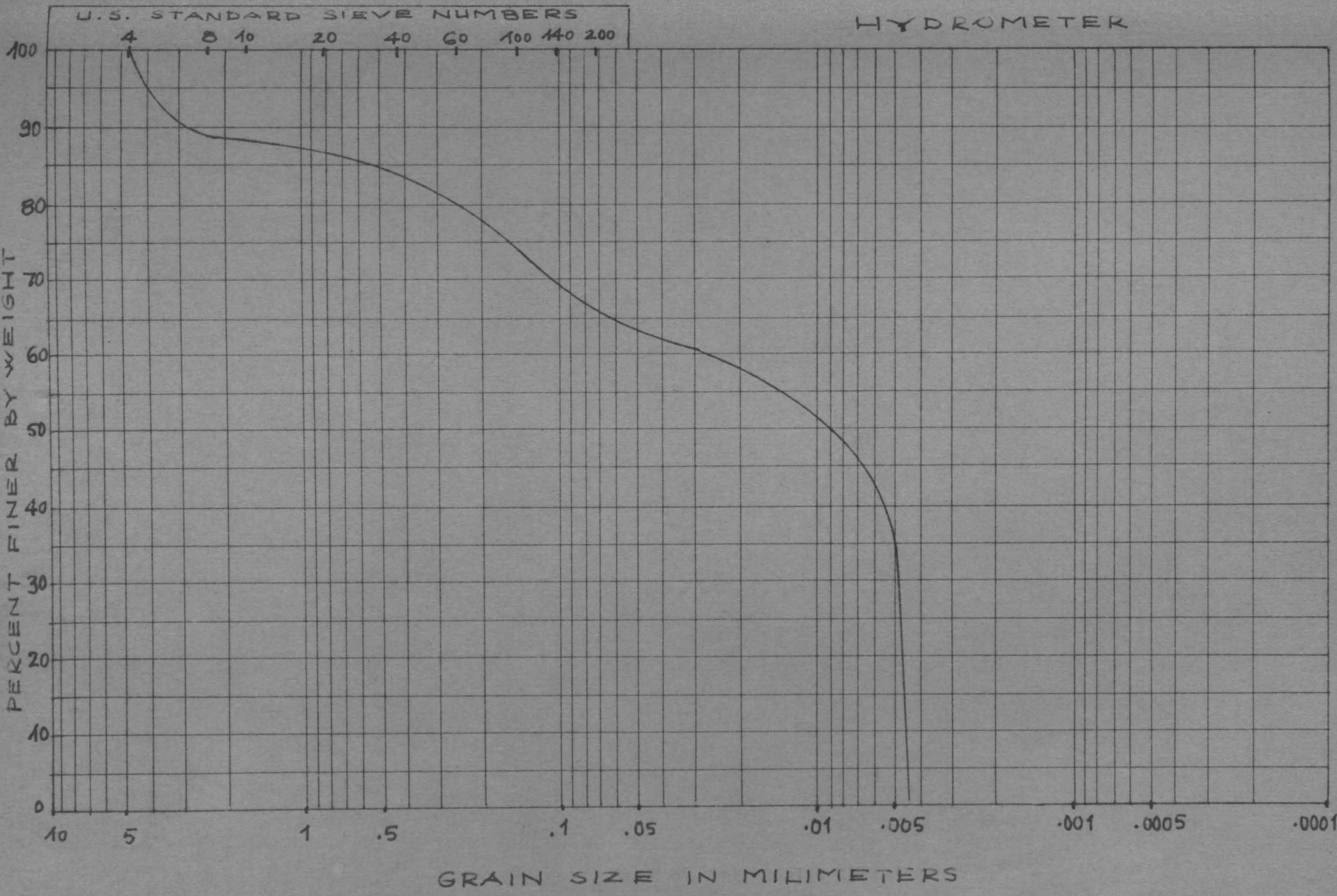
FIG - 45



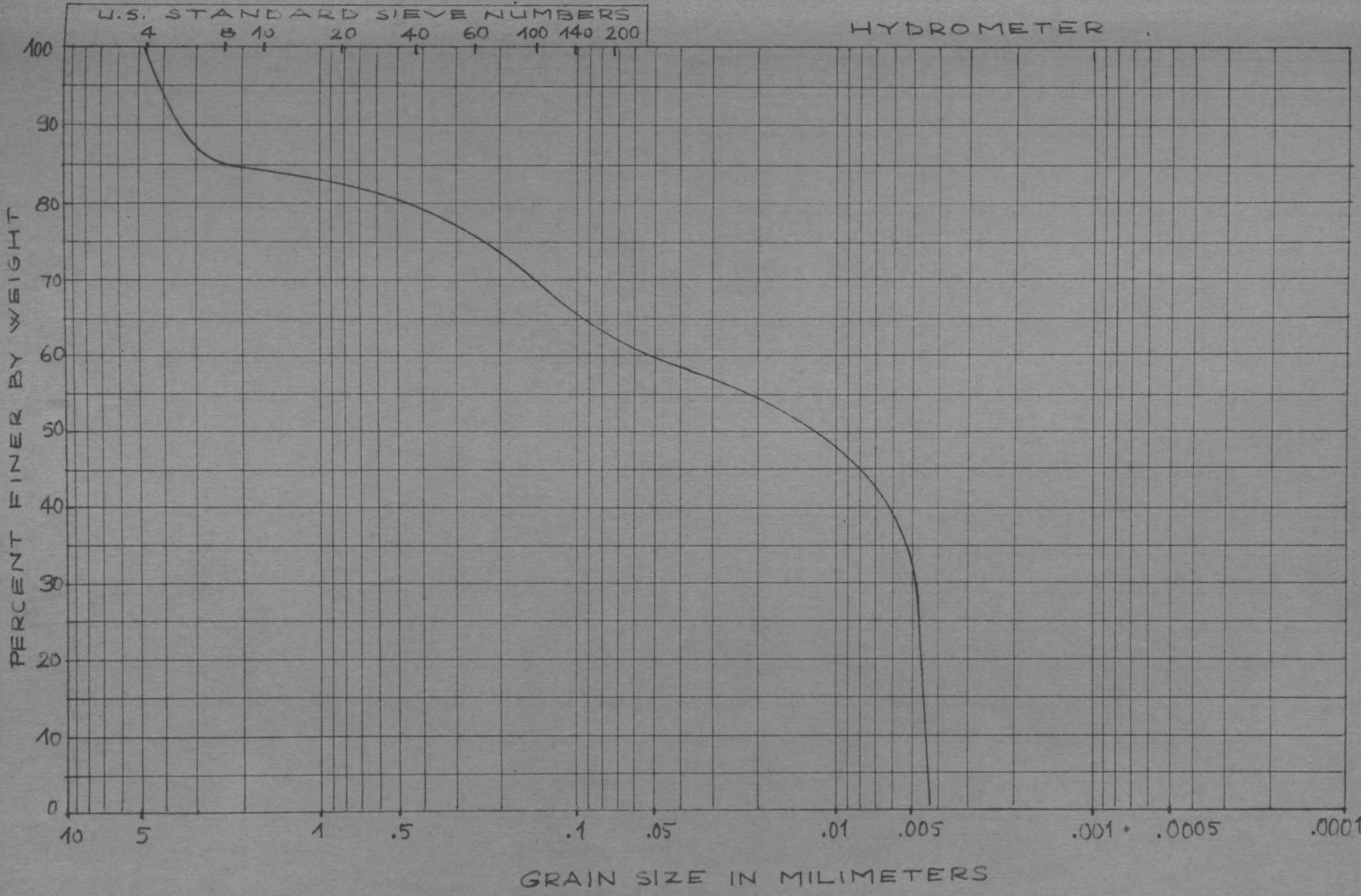
SF = 5.35 d_m = 0.255 mm

SAMPLE - B

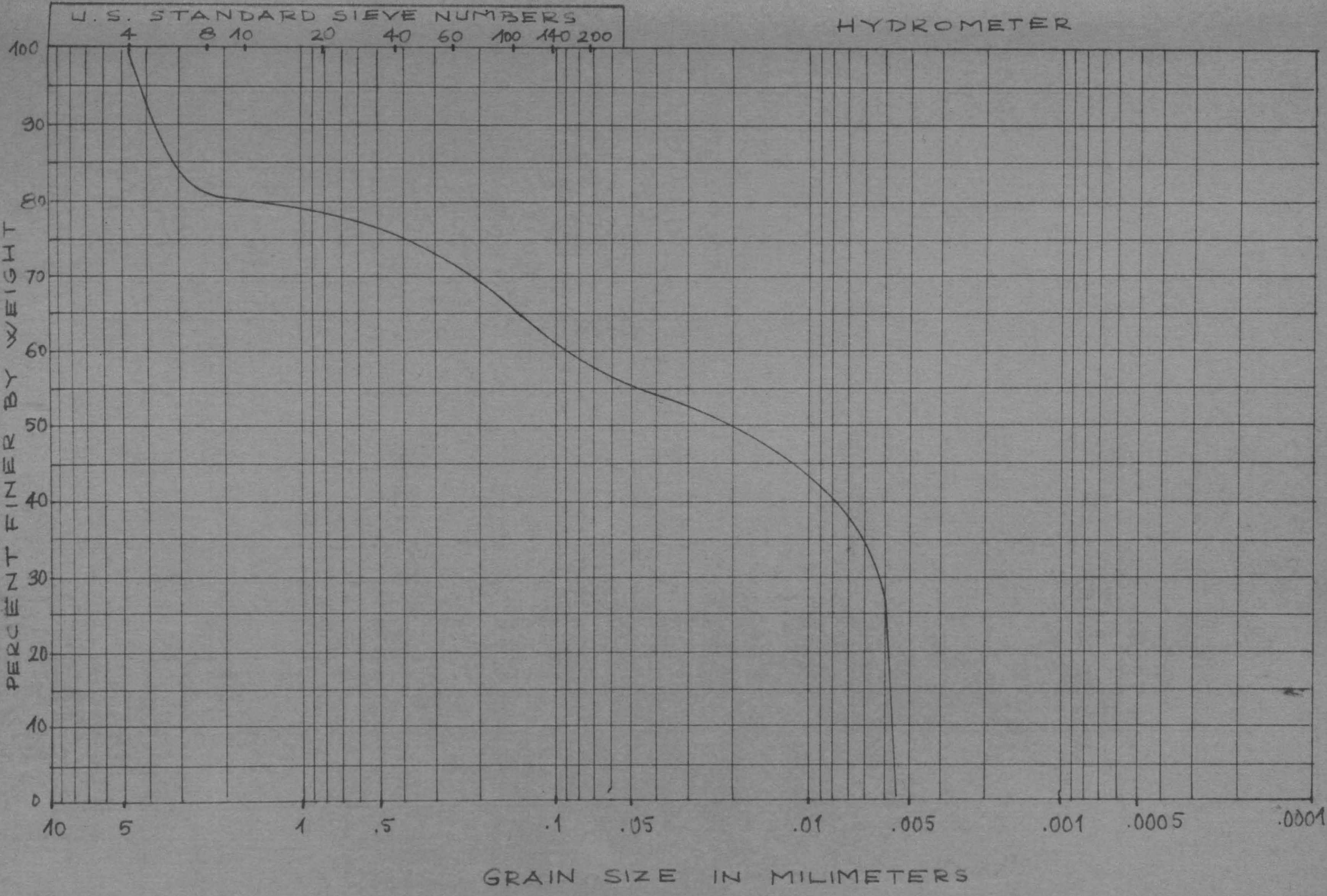
FIG - 46



SAMPLE - C SF = 4.84 d_m = 0.325 mm



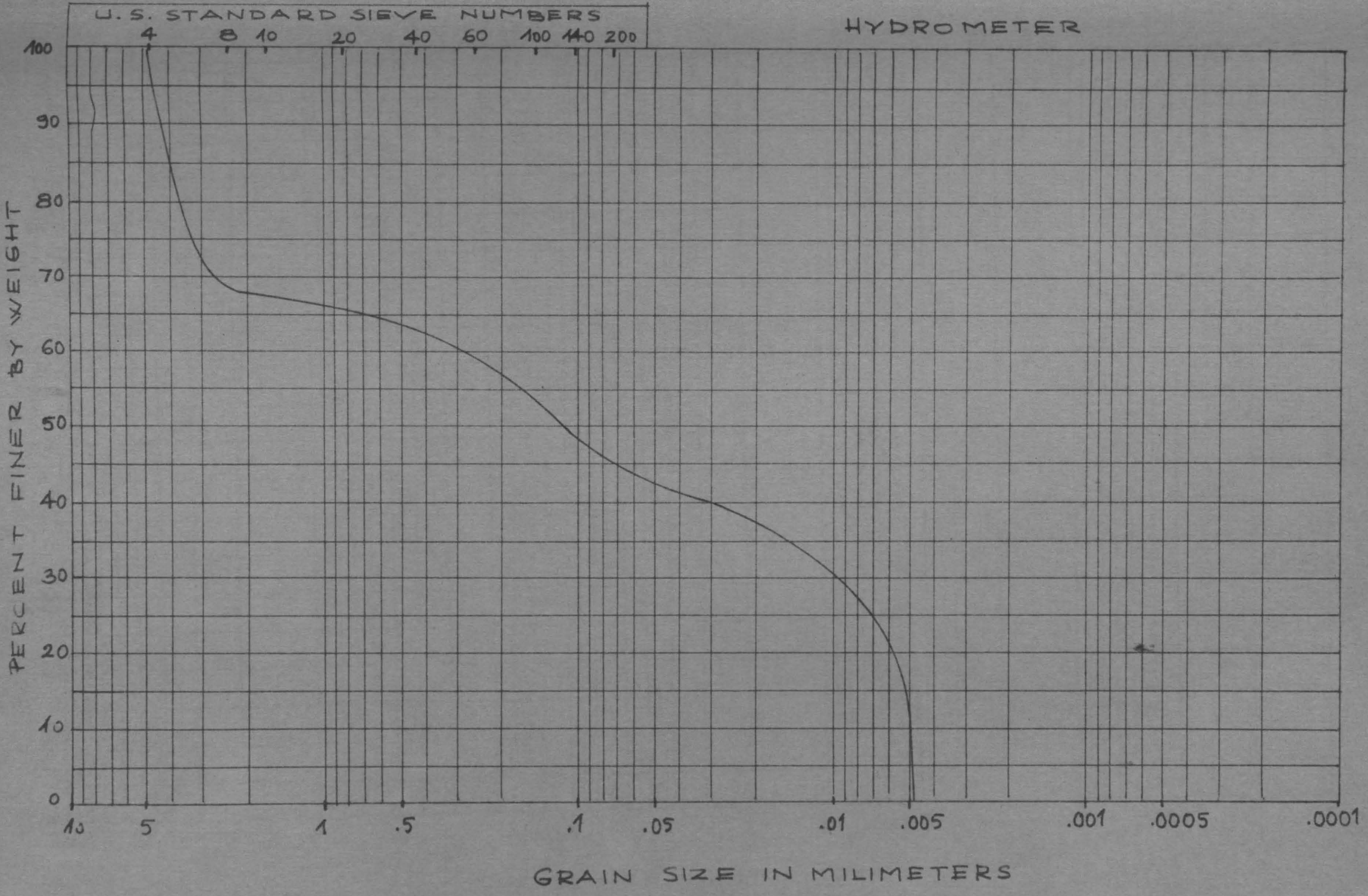
SAMPLE - D SF = 4.57 $d_m = 0.394$ mm



SAMPLE - E

SF = 4.32 d_m = .471 mm

FIG- 4.9

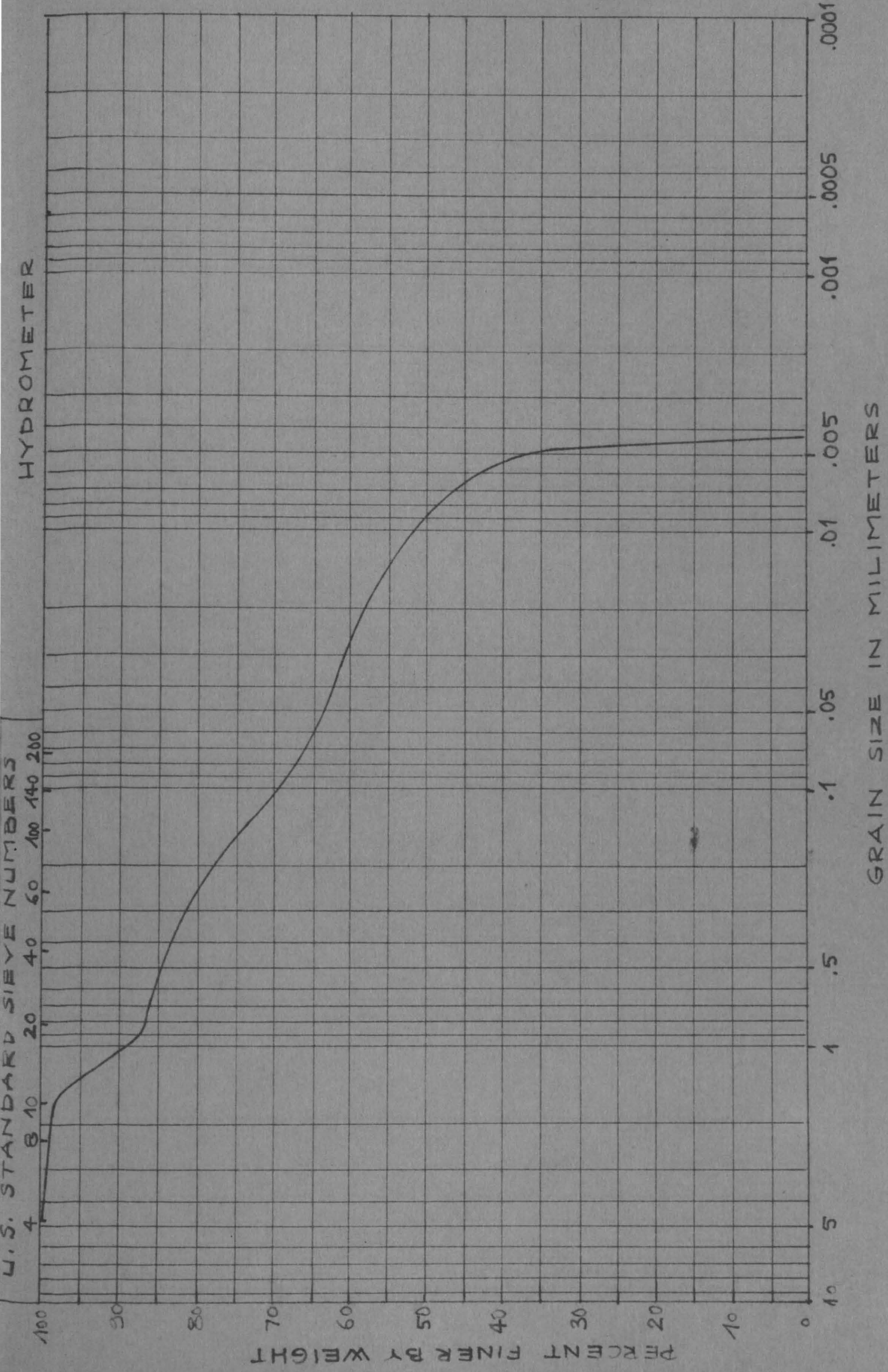


SAMPLE - F

SF = 4.09 d_m = 0.561 mm

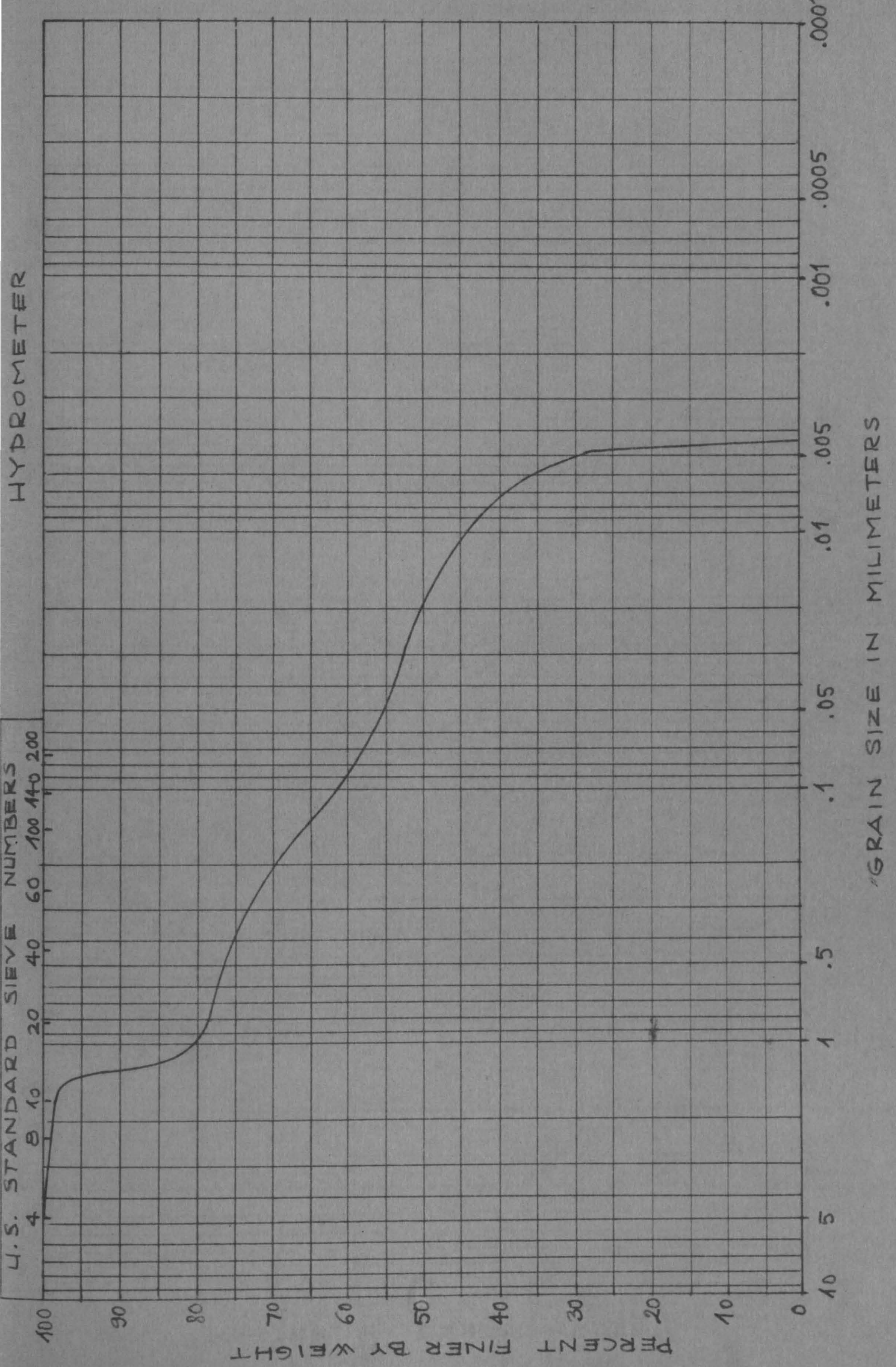
SAMPLE - 6 SF = 5.30 $d_m = 0.236$ mm

FIG_51



SAMPLE - H SF= 4.98 d_m = 0.306 mm

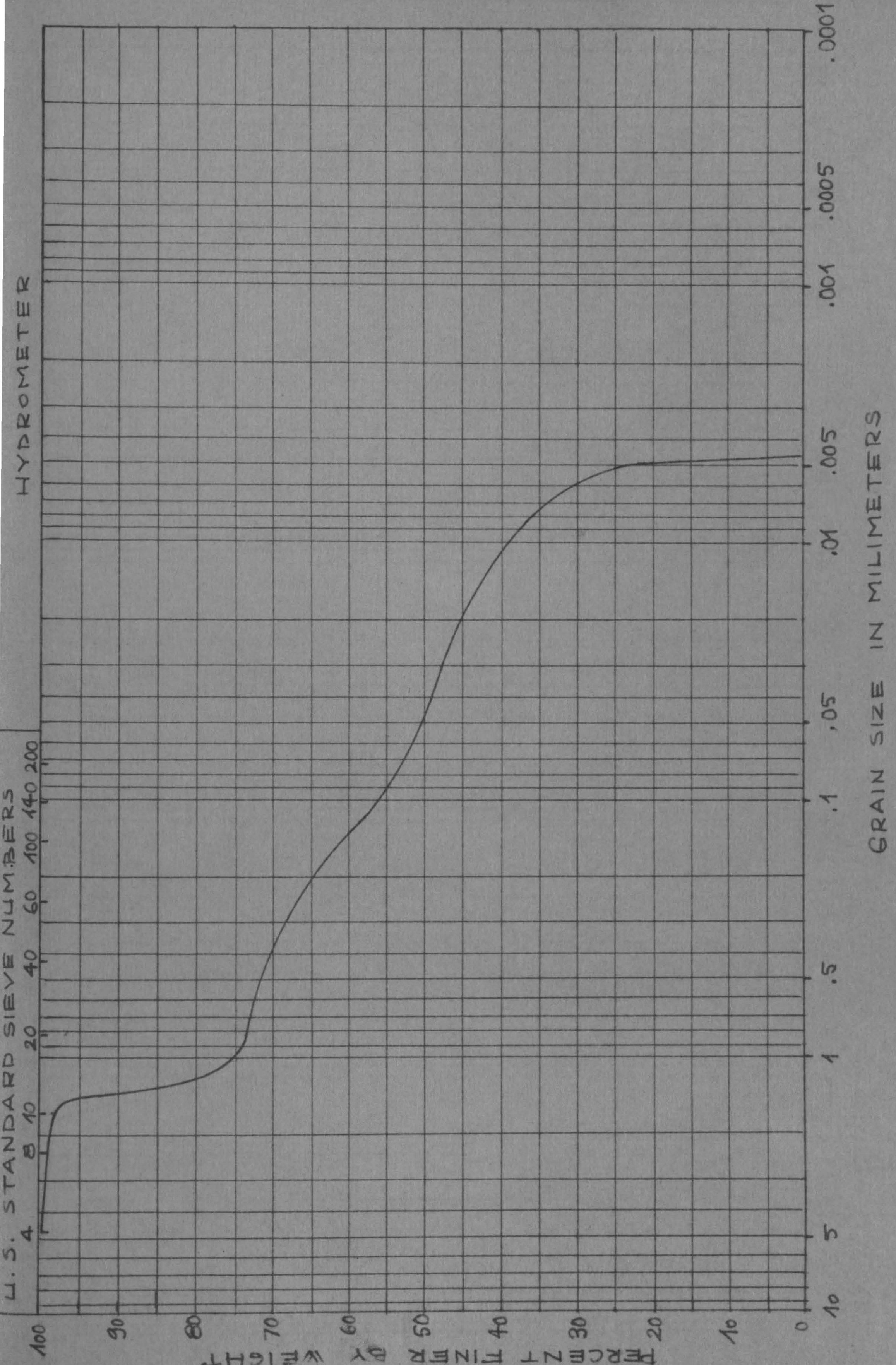
FIG-52



SAMPLE - 1

SF = 4.50 $d_m = 0.417$ mm

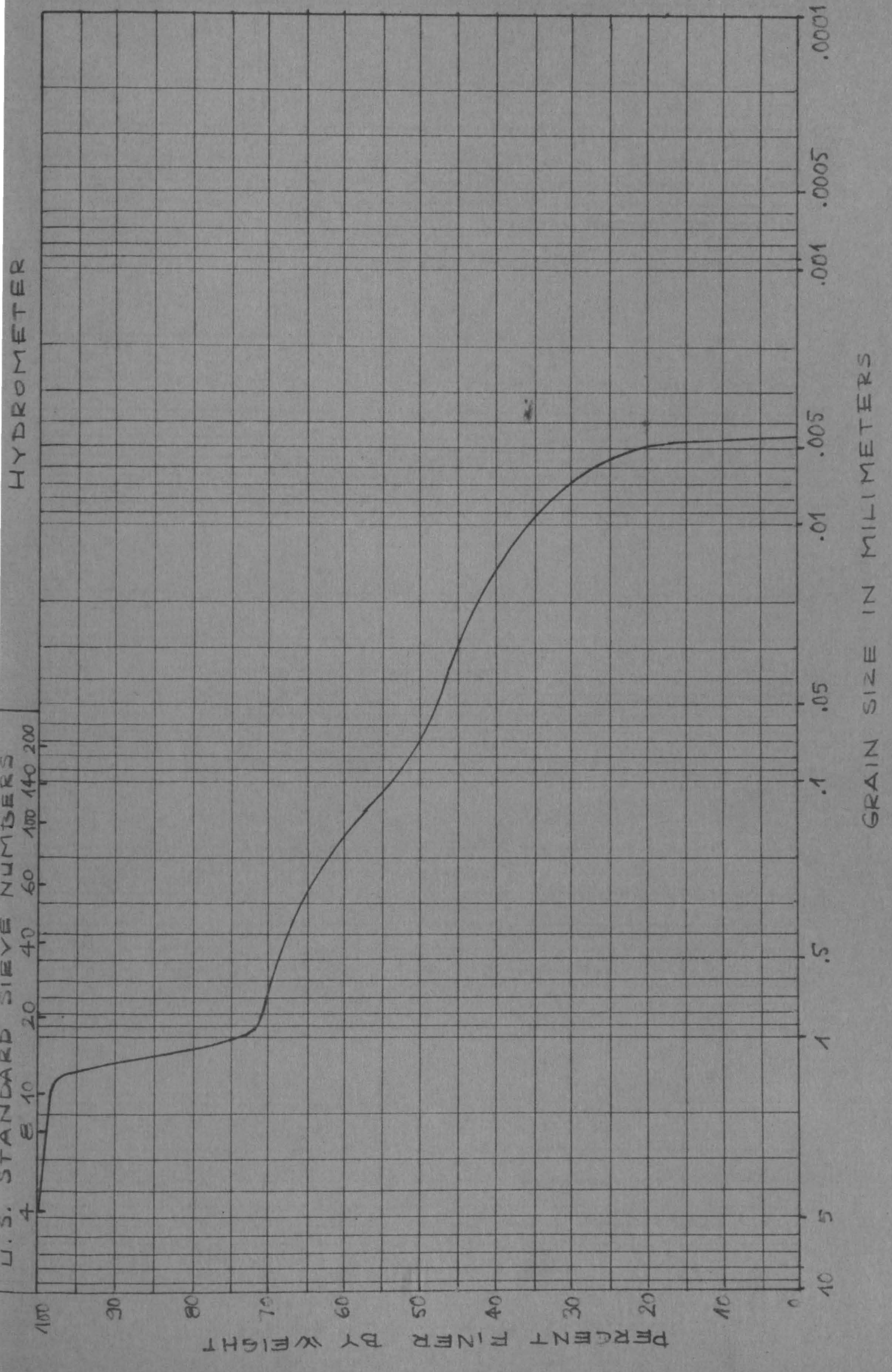
FIG-53



SAMPLE - J

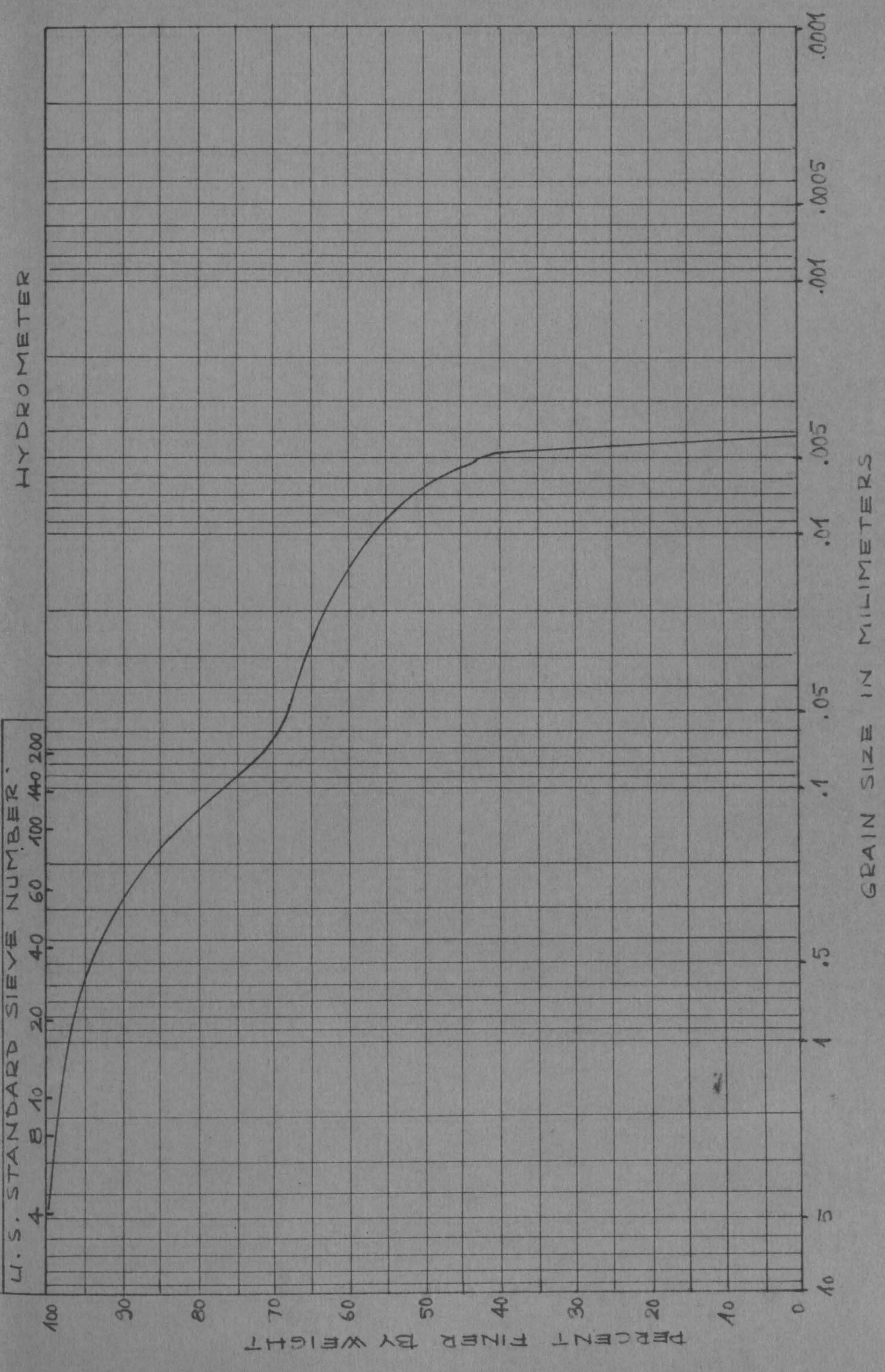
SF = 4.18 d_m = 0.520 mm

FIG. 54



SAMPLE - K   SF = 4.03 d_m = 0.570 mm

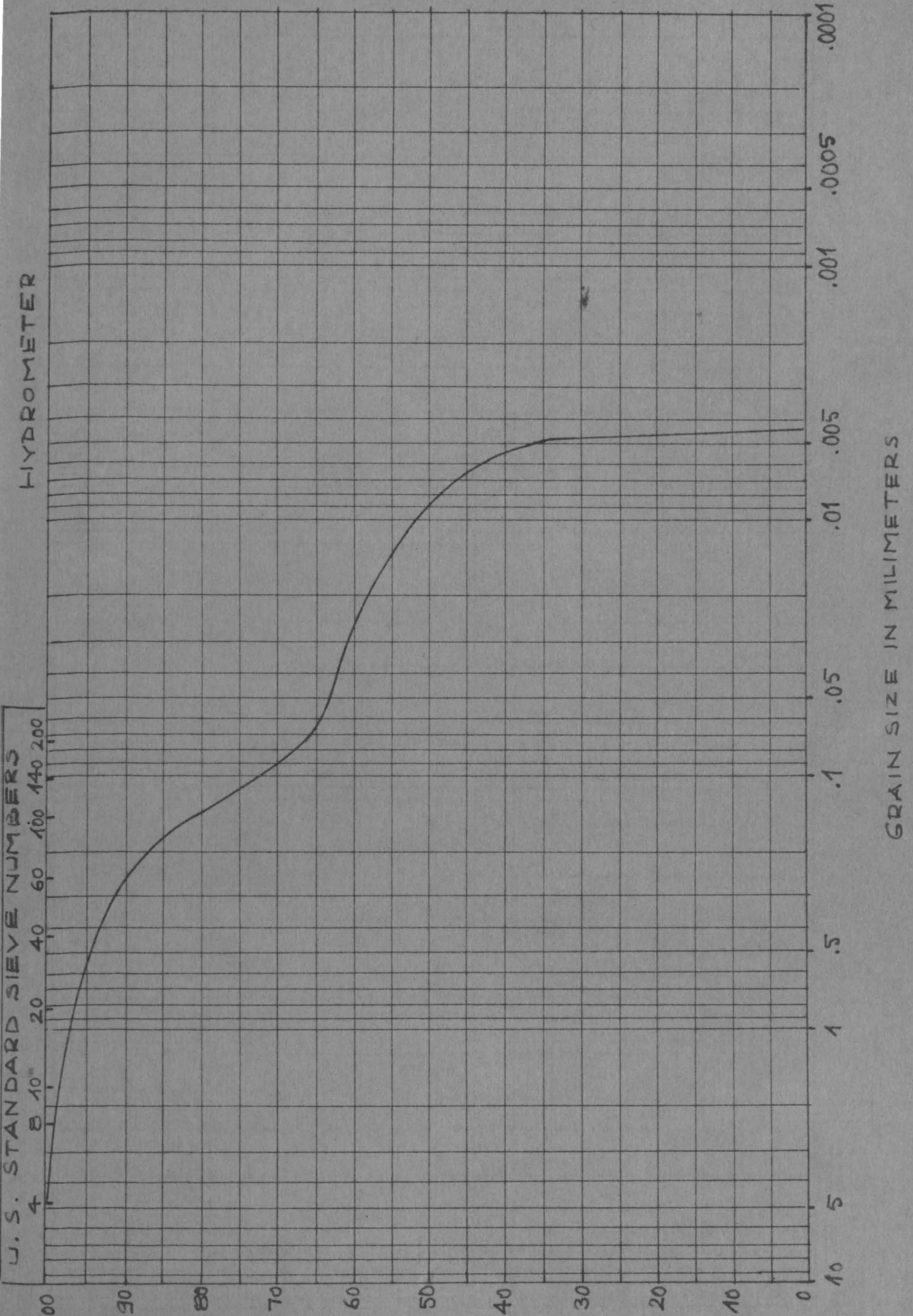
FIG-55



SF = 5.35 d_m = 0.236 mm

SAMPLE- L

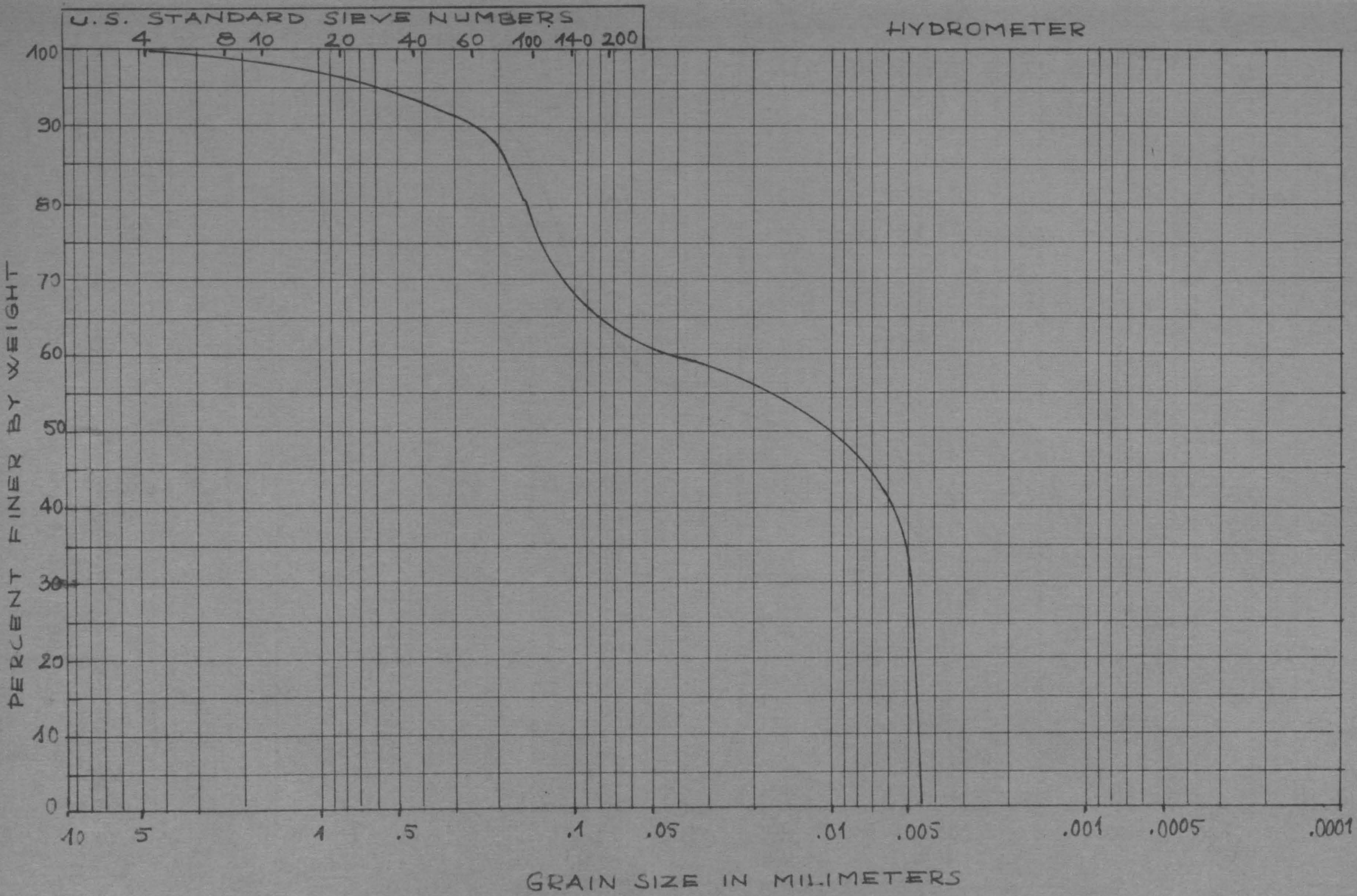
FIG_56



SAMPLE - M

SF = 5.18 d_m = 0.256 mm

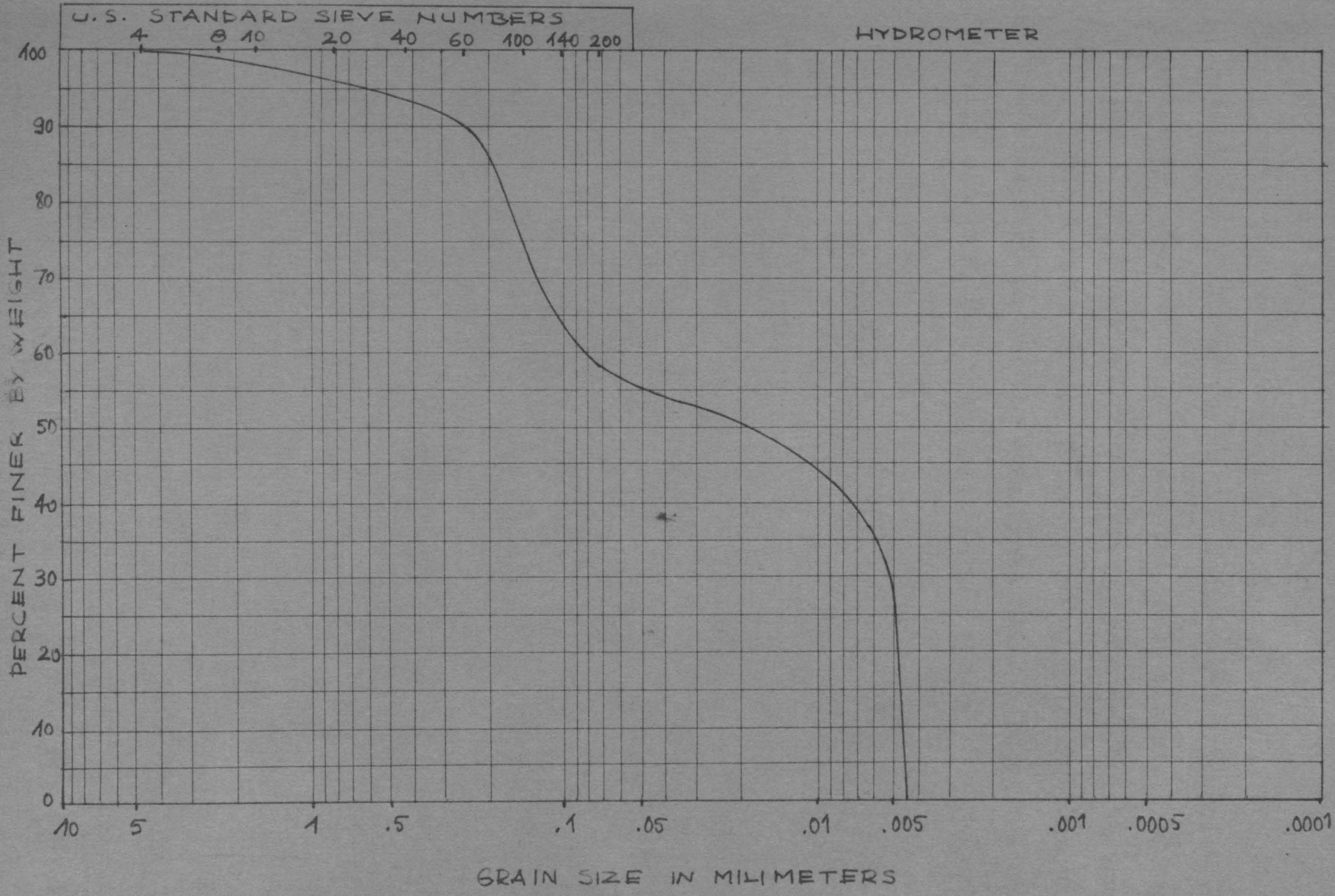
FIG - 57



SAMPLE - N

SF = 5.04 d_m = 0.286 mm

FIG_58



SAMPLE _ 0

SF = 4.83 d_m = 0.325 mm

FIG - 59

the curve of sample A. We have added a material of 9.66% which is retained on No.20 sieve, so the ordinate at this point must decrease by 9.66. Thirdly as all the added material is retained on No.20 sieve, the rest of the curve should follow the same contour as sample A but starting from the reduced ordinate mentioned above. The size factors and the mean grain sizes were calculated for every sample by the methods explained above.

RESULTS AND DISCUSSION

The results are collected in Table (XXI) and plotted in figures (60) to (64).

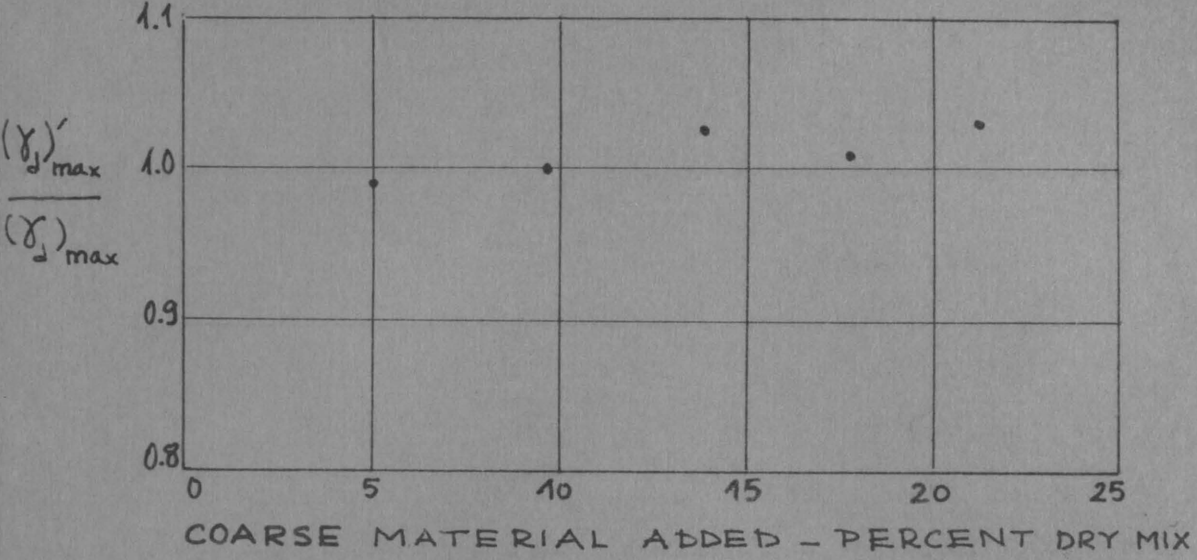
In plotting the curves dimensionless parameters are formed, for example by taking the ratio of the maximum dry density, obtained by compacting a sample containing a certain percentage of coarse material of certain gradation, to the dry density of the natural soil. Other dimensionless parameters were formed in the same manner for the optimum moisture content, the size factor and the mean grain size.

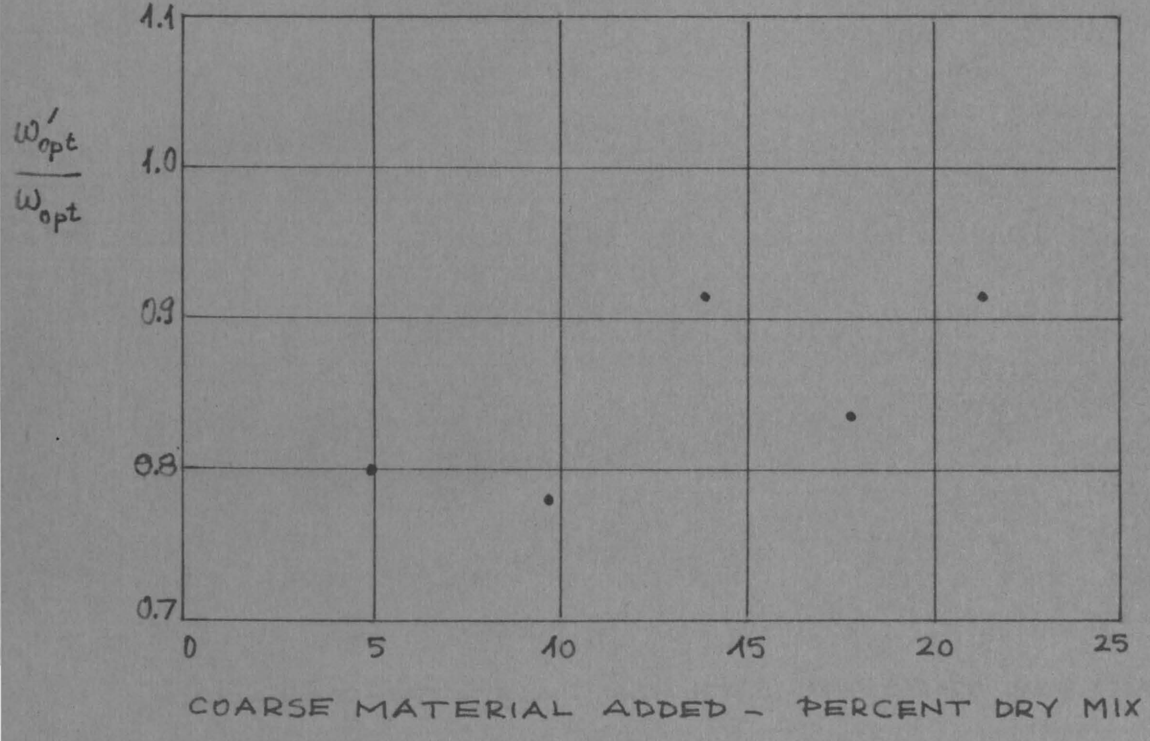
The upper plots in figures (60), (61) and (62) shows the ratio of the maximum dry density of aggregate added samples to that of the natural sample against the percentage of the coarse material present. A seperate plot is made for each group of aggregate. Almost the same tendency is observed in all three cases, namely a close scattering around 1.0 line. This means that for all the three groups of materials no change in the maximum dry density occurs by the addition of coarse material within 25%. This result denotes that for small percentages the coarse material merely acts as a displacer, being in accordance with the finding of Maddison but contradicting that of workers after him.

The lower plots in figures (60), (61) and (62) is shown the ratio of the optimum moisture content of the aggregate added samples to the optimum moisture content of the natural sample against the percentage of coarse material. All these plots are

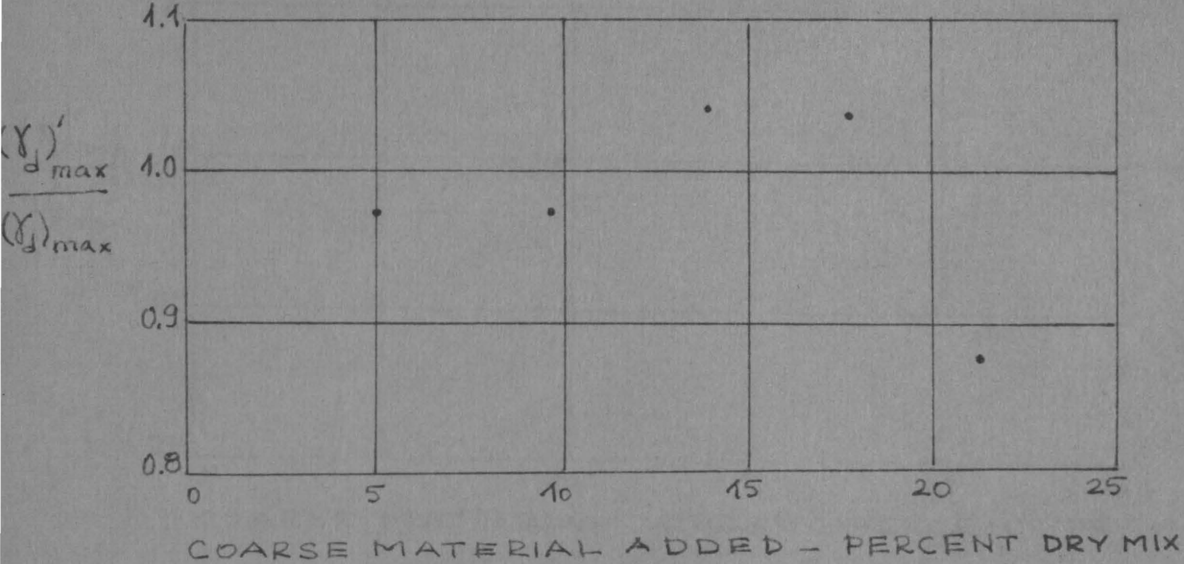
SAMPLE	COARSE SIZE ADDED	$(\gamma_d)'_{max}$	$\frac{(\gamma_d)'_{max}}{(\gamma_d)_{max}}$	w'_{opt}	$\frac{w'_{opt}}{w_{opt}}$	$(SF)'$	$\frac{(SF)'}{(SF)}$	d'_m	$\frac{d'_m}{d_m}$
A	–	113.5	1.000	17.5	1.000	5.57	1.000	0.200	1.000
B	4_8	110.2	0.988	14.0	0.800	5.35	0.960	0.255	1.275
C	4_8	113.8	1.001	13.6	0.778	4.84	0.868	0.325	1.625
D	4_8	116.6	1.027	16.0	0.914	4.57	0.820	0.394	1.970
E	4_8	114.5	1.010	14.6	0.835	4.32	0.755	0.471	2.360
F	4_8	117.0	1.030	16.0	0.914	4.09	0.734	0.561	2.800
G	10_20	109.9	0.968	18.4	1.050	5.30	0.951	0.236	1.180
H	10_20	110.0	0.970	17.8	1.018	4.98	0.894	0.306	1.530
I	10_20	117.8	1.038	15.0	0.856	4.50	0.807	0.417	2.085
J	10_20	117.3	1.035	16.0	0.914	4.18	0.750	0.520	2.600
K	10_20	99.5	0.877	14.2	0.811	4.03	0.723	0.570	2.850
L	60 -200	114.2	1.007	15.2	0.869	5.35	0.960	0.236	1.180
M	60 -200	114.0	1.004	15.6	0.893	5.18	0.928	0.256	1.275
N	60 -200	116.8	1.029	14.8	0.846	5.04	0.904	0.286	1.430
O	60 -200	117.0	1.030	13.8	0.789	4.83	0.866	0.325	1.625

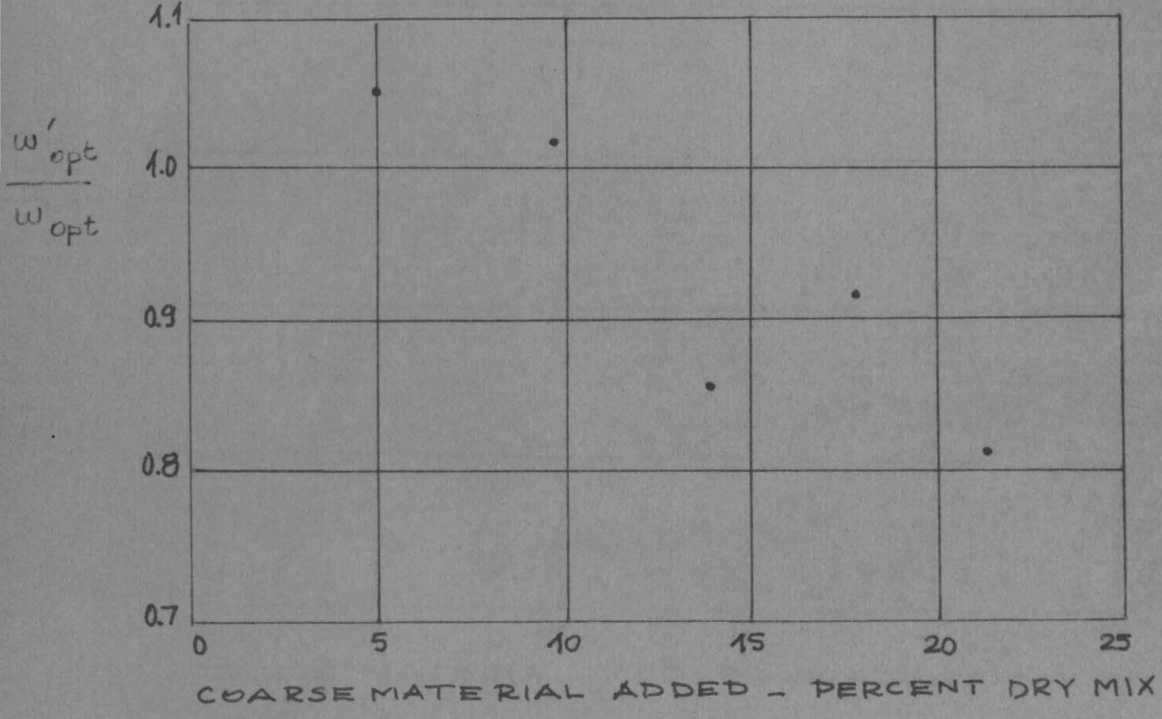
TABLE_XXI



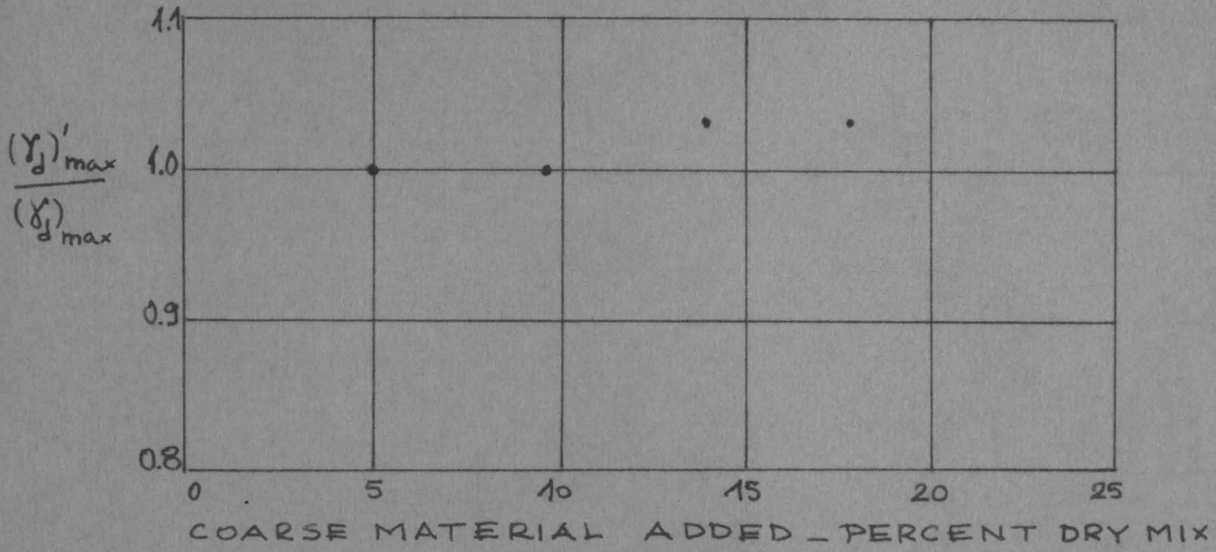


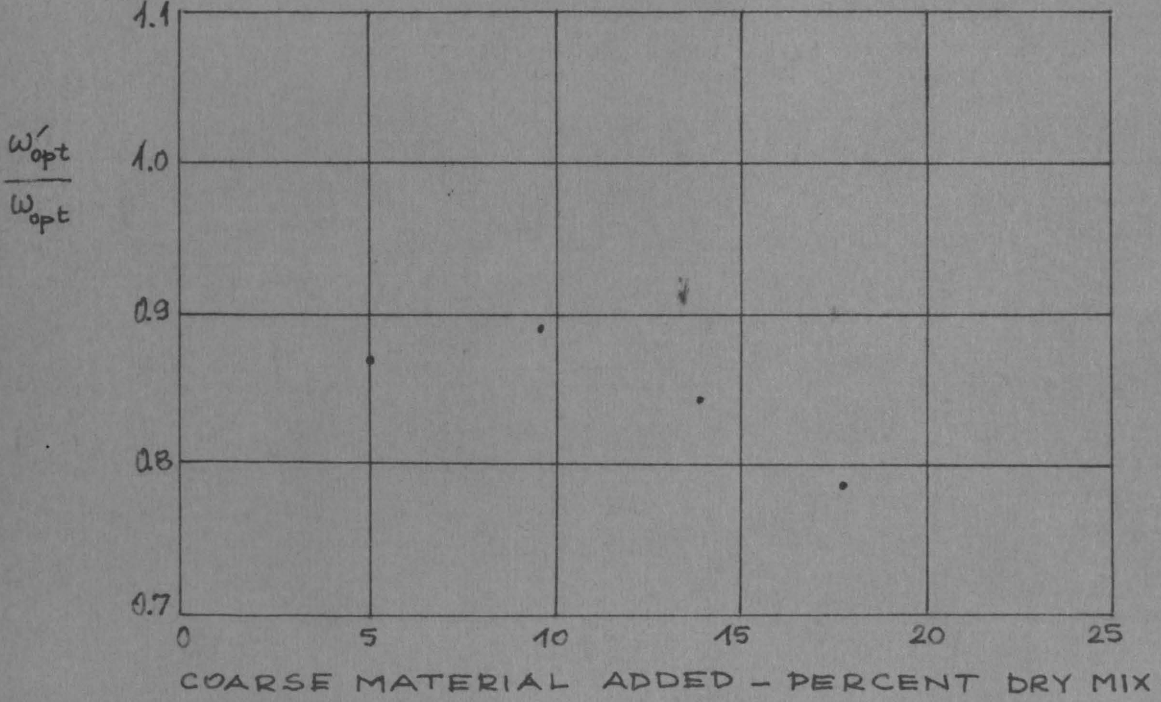
EFFECT OF COARSE MATERIAL PASSING No 4 SIEVE - RETAINED ON NO 8 SIEVE, ON MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT.



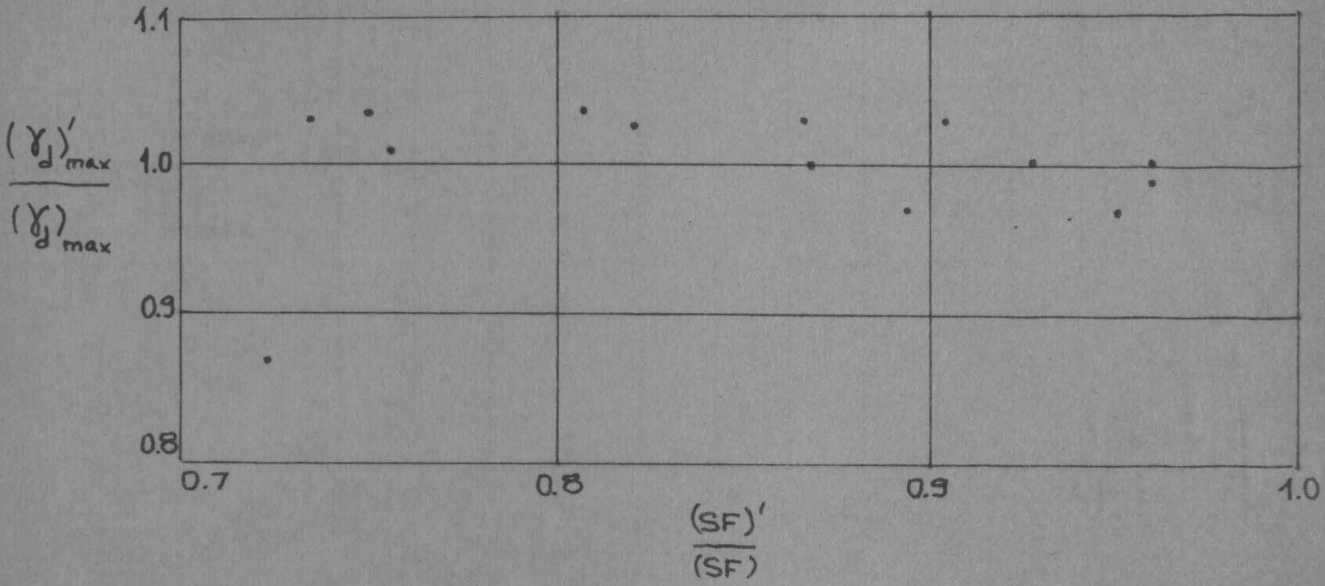


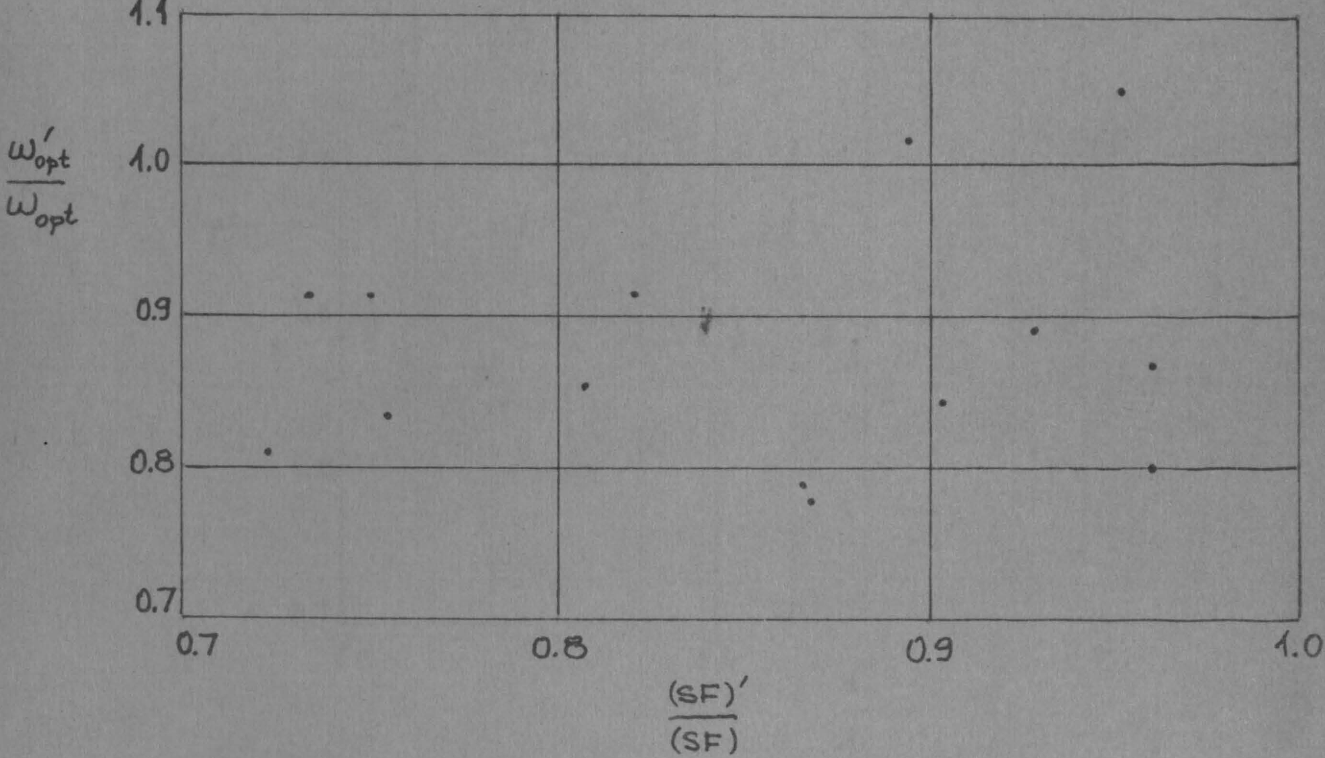
EFFECT OF COARSE MATERIAL PASSING NO. 10 SIEVE – RETAINED ON No20 SIEVE, ON MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT





EFFECT OF COARSE MATERIAL PASSING NO 60 SIEVE _ RETAINED ON NO 200 SIEVE, ON MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT.


$\frac{(\gamma_d)'_{max}}{(\gamma_d)_{max}}$
1.1
1.0
0.9
0.8
0.7
0.8
0.9
1.0
$\frac{(SF)'}{(SF)}$


$\frac{\omega'_{opt}}{\omega_{opt}}$
1.1
1.0
0.9
0.8
0.7
0.7
0.8
0.9
1.0
$\frac{(SF)'}{(SF)}$

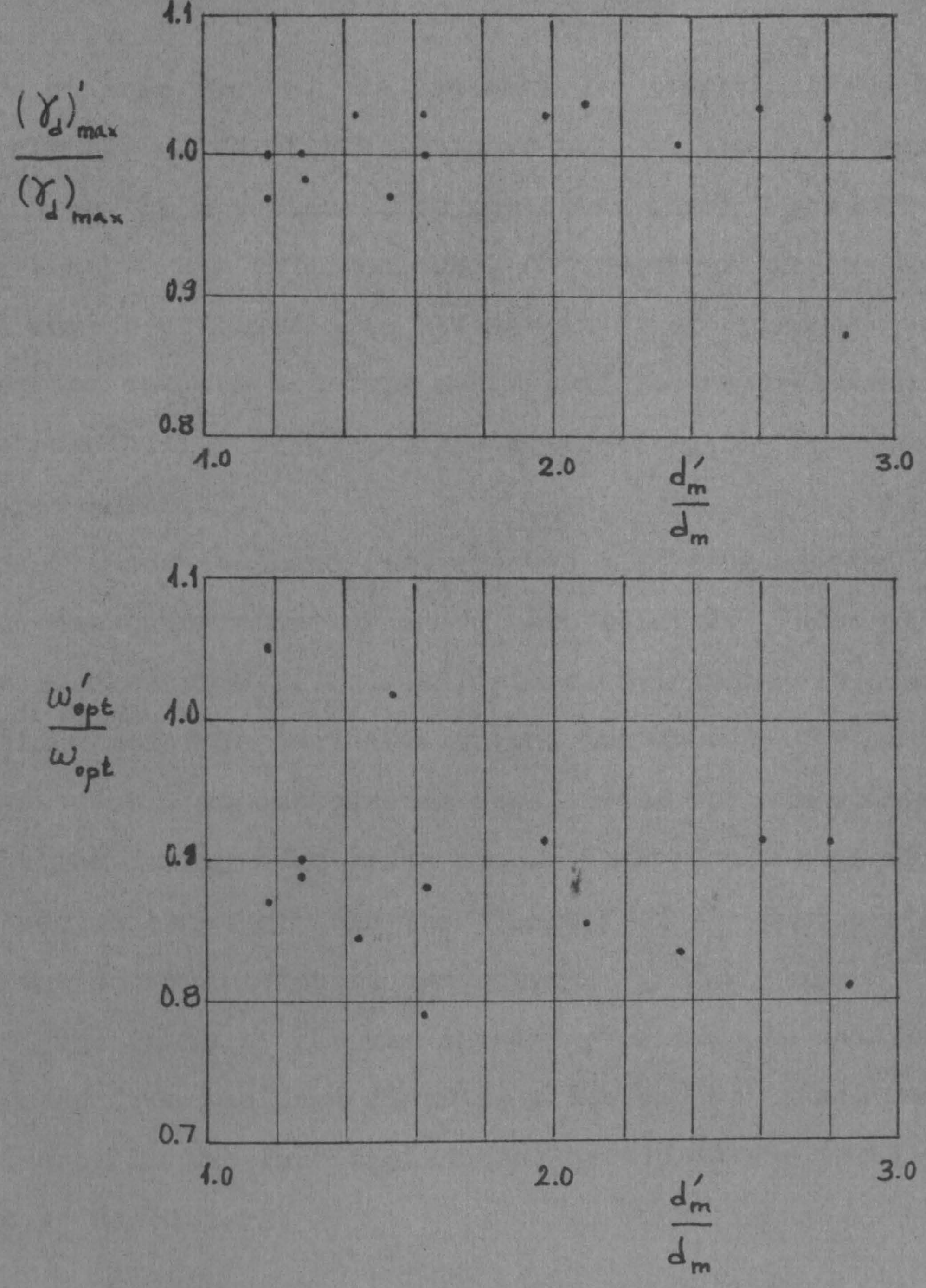

$\frac{(\gamma_d)'_{max}}{(\gamma_d)_{max}}$
1.1
1.0
0.9
0.8
1.0
2.0
3.0
$\frac{d'_m}{d_m}$
$\frac{\omega'_{opt}}{\omega_{opt}}$
1.1
1.0
0.9
0.8
0.7
1.0
2.0
3.0
$\frac{d'_m}{d_m}$

FIG - 64

more or less random. We can make one generalization however, namely the value of the ratio is most of the time, nearer to 0.9, which may be expected since the total surface area of the soil decreases. But this decrease, on the other hand is insignificant and may be considered in the margin of experimental error in locating the optimum moisture content of the natural sample suggesting the possibility of it being unaffected by the addition of the coarse material.

Figures (63) and (64) are two different ways of looking at the same thing since SF and d_m are related. These plots contradict the results of M.S. Youssef, since no increase is seen in the optimum moisture content with and increase in the size factor, and consequently no decrease is seen in the optimum moisture content with increasing mean grain size. This result suggests the presence of variables other than the diameter of the particle, determining of the optimum moisture content.

The ratio of the maximum dry densities is unaffected as can be seen from the same figures, which can be considered a second evidence to the fact that coarse particles are totally inert, and act as displacers.

CONCLUSION

The following conclusions can be drawn from the results of the tests made :

I. The compaction characteristics, such as the maximum dry density, optimum moisture content, shape of the compaction curve, of the Topser clay (LL 47.0 , PI 20.8) cannot be changed by the addition of coarse aggregate,(Of three groups, a- Passing No. 4 sieve , retained on No. 8 sieve. b- Passing No. IO sieve, retained on No. 20 sieve. c- Passing No. 60 sieve, retained on No. 200 sieve.) up to 2I % by dry weight. This result confirms the findings of Maddison (See p. 72) in Road Research Laboratory, England.

2. Thus this clay cannot be ~~stabilized~~ densified by adding coarse material within economic limits.

3. The results of Standard Proctor Compaction is not affected by the presence of coarse material, as long as the latter remains within 2I % by dry weight of clay, for this particular soil.

4. It is not possible to relate the compaction characteristics of this clay with properties derived from the grain size distribution, mean grain size and size factor. That is, these characteristics are independent of the mean grain size and the size factor.

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APPENDIX

PYCNOMETER CALIBRATION

PYCNOMETER NO. 20

THEORETICAL PROCEDURE

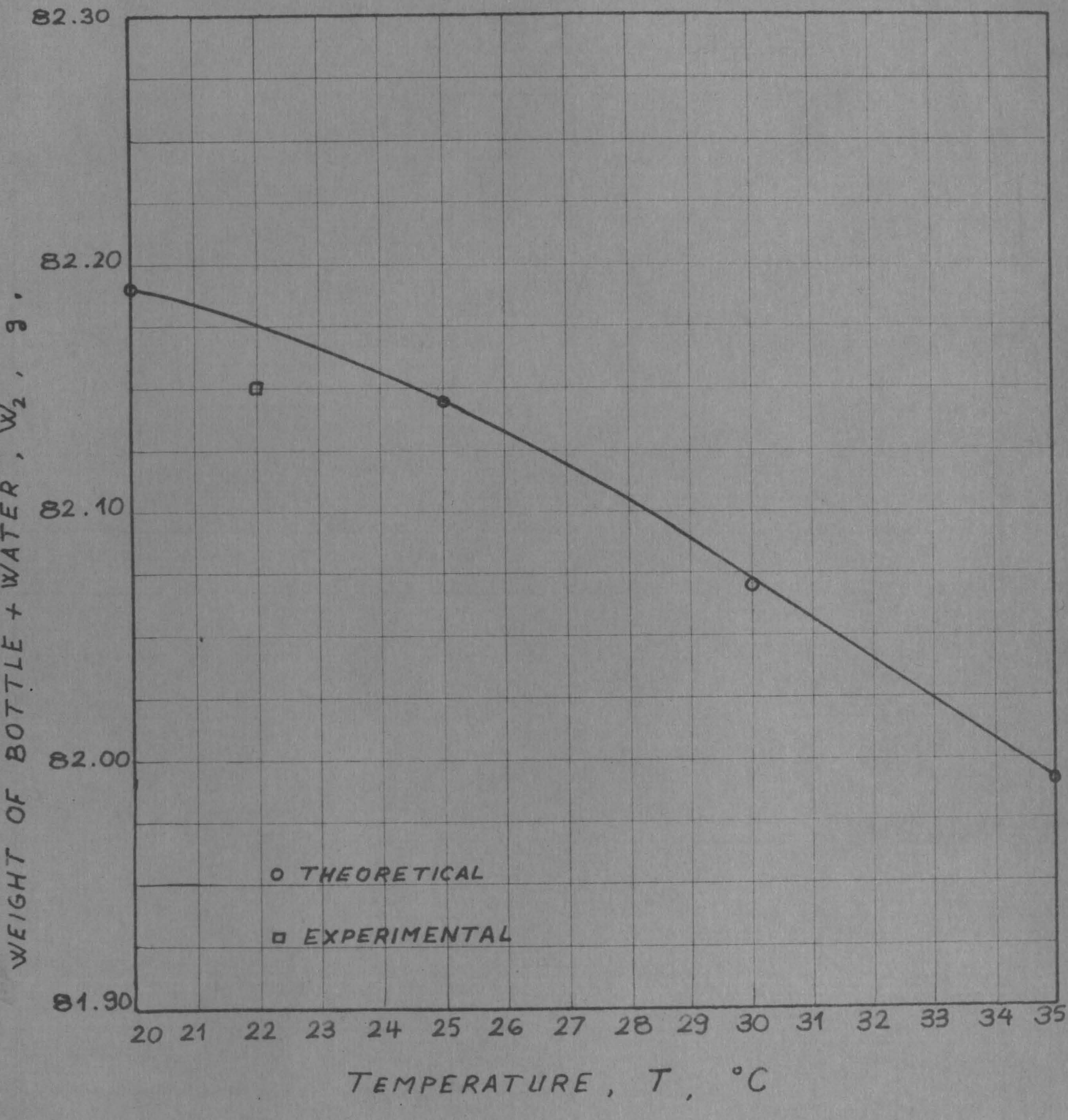
WT. BOTTLE W_B IN g	32.34	CUBICAL EXPANSION FOR GLASS IN 1/°C ϵ	0.1×10^{-4}
TEMPERATURE OF CALIBRATION T_C IN °C	20.0	UNIT WEIGHT OF AIR IN g/cc γ_a	0.0012
VOL. BOTTLE AT T_C V_B IN cc	50.0	$W_2 = W_B + V_B(1+\Delta T\epsilon)(\gamma_T - \gamma_a)$	

DETERMINATION NO.	1	2	3	4
TEMPERATURE T, IN °C	20.0	25.0	30.0	35.0
UNIT WEIGHT OF WATER AT T, γ_T IN g/cc	0.9982	0.9971	0.9957	0.9941
WT. BOTTLE + WATER AT T, W_2 IN g	82.19	82.14	82.07	81.99

EXPERIMENTAL PROCEDURE -

$T = 22.0$ °C $W_2 = 82.15$ g

PYCNOMETER CALIBRATION CURVE



CALIBRATION CURVE FOR HYDROMETER 23786