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MAGNITUDE OF FROST GRIP AS A FUNCTION OF WATER CONTENT, POROSITY, TEMPERATURE AND PARTICLE SHAPE

A THESIS

Submitted to the Faculty of Graduate Studies through the
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of the Requirements for the Degree of
Master of Applied Science at the
University of Windsor.

by

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ABSTRACT

The effect of water content on the magnitude of frost grip is investigated for two sands, an angular, uniform sand and a natural sand of better gradation. The influence of the material to which the frozen soil adheres is also checked.

The influence of porosity of the soil is investigated in the next stage of the work, and these results, in conjunction with the results of the initial stage, allow a mathematical analysis over the theoretical range and the development of a general equation in the practical range of porosity and water content.

The influence of the depth of the molds used in the laboratory work on the experimental results is investigated and analysed.

Using two soil mixes the influence of silt and clay size particles on the relationships previously deduced is investigated. A soil mix is also used to determine if the rate of freezing has any effect on the results.

ACKNOWLEDGEMENTS

The author wishes to express his appreciation to his advisor, Professor J. T. Laba, for his energetic and patient guidance, to the National Research Council, for sponsoring this project and to Miss M. Heynsbroek for typing this thesis.

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PREFACE

In areas suffering long and cold winters the problem of frost heave is a major concern. This problem manifests itself most markedly in roadway and airport works and consequently much energy has been directed to this area in an effort to understand and control this adverse phenomenon.

Generally overlooked, unfortunately, is the effect that frost heave has on footings and foundations or any other structure in contact with a frost susceptible soil. Since moist soil, when it freezes, adheres to a material in contact with it, when the surrounding soil heaves, an uplift force is exerted on the structure. For a lightly loaded footing this force may be enough to produce a harmful displacement.

It is the aim of this thesis to contribute to the understanding of this frozen soil force.

CHAPTER 1

INTRODUCTION

It is well known that when certain soils are subjected to below-freezing temperatures they will exhibit a phenomenon known as "frost heave". As the name implies the surface of the affected soil rises, sometimes several inches. It follows, then, that any structures located on or penetrating through this expanding soil layer will be subjected to forces tending to lift it upwards along with the surrounding heaving soil. Any footing or foundation resting directly on the ground surface will, of course, heave an amount equal to the heave of the supporting soil. It is to foil this unpredictable displacement that footings are never placed directly on the surface of a frost-active soil but are placed below the depth of local frost penetration. Although very helpful, this does not eliminate the problem since the footing must still pass upward through the frozen layer at the ground surface. This frozen layer, sometimes several feet thick depending on the depth of frost penetration, will still heave, and exert an upward pull on the footing due to the presence of a bonding force between the frozen soil and the footing. (Fig. 1a) It is the purpose of this thesis to determine

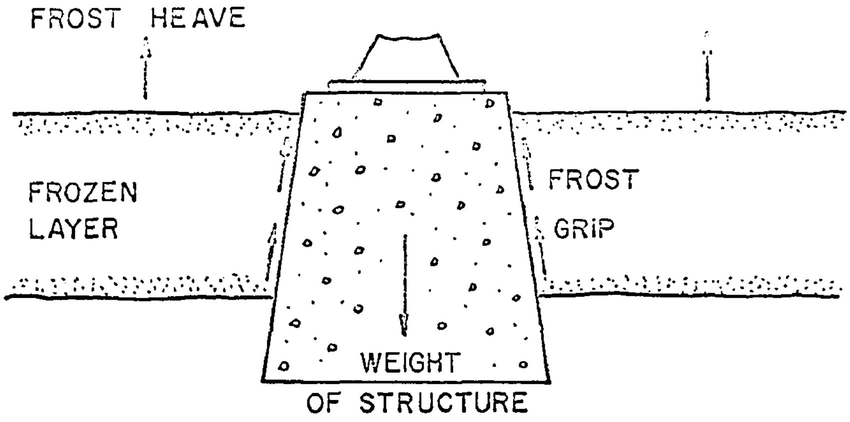


Fig. 1a Frost Grip Forces Contributing to the Uplift of Small Footing

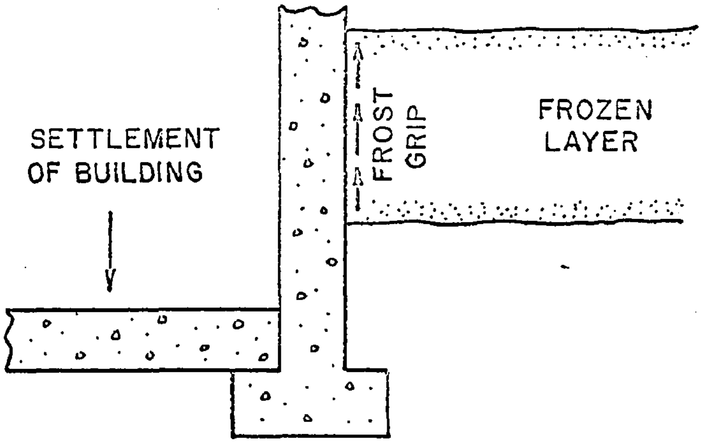


Fig. 1b Frost Grip Forces Resisting Settlement of Structure

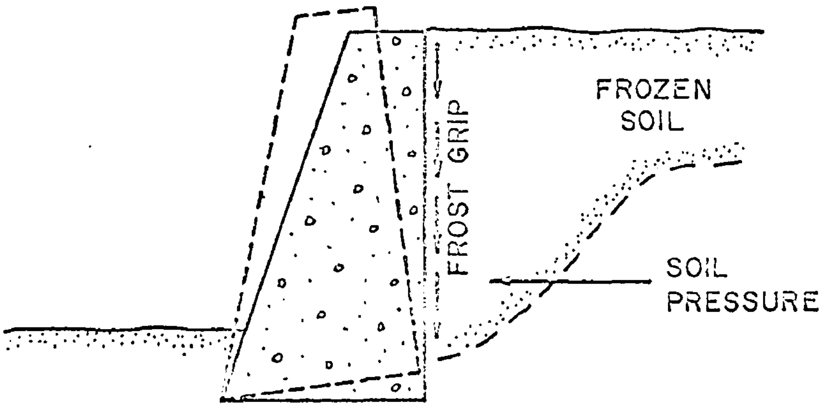


Fig. 1c Frost Grip Forces Contributing to Stability of Retaining Wall

the magnitude of this bonding force, hereafter refered to as the frost grip, and the factors influencing it.

On large footings where the downward forces are very great the upward force, as determined by the magnitude of the frost grip multiplied by the surface area of the footing in the frozen soil zone, will not be sufficient to overcome the weight of the structure and hence the frost grip will be broken, the soil will heave and the structure will remain stationary. If however the weight of the structure is insufficient to overcome the uplift due to the frost grip the structure will then heave. This is the case in hydro power transmission towers and transformer and distributing stations where heaving results in warped transformer pads, opened switches and distorted service boxes. These disturbances were reported by Trow(1) in a paper in which he develops a chart giving the bearing pressure required to overcome frost heave as a function of the footing parameter, perimeter/area. He uses a value of 400 p.s.i. for the frost grip, but reports that in his initial studies his values ranged from 304 p.s.i. to 495 p.s.i., a range of approximately ±24%. It is hoped that with a better understanding of the conditions influencing the frost grip a more reliable approximation may be made.

Frost grip is also one of the forces acting on retaining walls during the below freezing winter months. It is a beneficial force, since by bonding the frozen soil to the face of the retaining wall, it presents an additional stabilizing force which must be overcome before failure can occur through overturning. (Fig. 1c)

Frost grip will also retard settlement of structures in the winter. The frost grip will bond the frozen upper crust to the structure which will offer a measure of support. In the spring, during the thaw, the grip will disappear and soil consolidation will continue normally, causing settlement of the structure. (Fig. 1b)

CHAPTER II

REVIEW OF LITERATURE

Since the initial important work in soil freezing by Taber and Beskow in 1929 and 1935 respectively, there has been much energy directed into researching the many aspects of this field. Workers have investigated the rate of frost heave, the depth of frost penetration, the criteria for frost susceptibility of soils and the mechanism of frost heave. However, the subject of this thesis, the grip between frozen soil and materials, does not appear to have been investigated in depth at all.

W. A. Trow in his paper dealing with the effect of frost heave on small footings, investigated the grip between frozen soil and concrete in his initial studies. He reported that saturated soil frozen for 24 hours at -10° F gave frost grip values ranging from 304 p.s.i. to 495 p.s.i. He mentions that "these results confirmed measurements made in Siberia approximately eighteen years ago"(1). Although no mention is made of the types of soils used or their state of packing when tested, these values agree generally with those the author obtained in the present investigation.

1. THE PHENOMENON OF FROST HEAVING

To the layman, the explanation of frost heaving would appear to be simply the result of the natural expansion of 9 or 10% undergone by the water in the soil when it freezes. This is a gross error as is pointed out by Professor Stephen Taber(2) in his discussion of a paper by Benkleman and Olmstead, (1931). Professor Taber writes "Perhaps the strongest evidence that increase in volume is not a factor in frost heaving, when freezing takes place in open systems, is furnished by substituting for water, other liquids which solidify with decrease in volume. The results obtained from freezing a clay column that stood in sand saturated with Nitrobenzene is shown in Fig. 3." Fig. 3 shows one-half of the clay cylinder, with layers of solid nitrobenzene in evidence, and it is these layers or lenses which are the direct cause of frost heaving.

This was first shown in 1916 by the aforementioned Taber when contemporary opinion held that frost heaving was attributable solely to the water - ice volume change. These ice lenses grow perpendicular to the direction of heat flow and since the heat usually flows from the warmer ground water to the freezing surface, the ice lenses are usually parallel to the ground surface. The lenses vary from narrow hairline lenses to those several inches thick. The difference in thickness of the individual lenses and consequently the resultant total heave is due considerably to the presence of a favourable temperature gradient in the system.

> "When the rate of change in temperature with depth (temperature gradient) is very rapid, the zone of soil in which the pore water is unfrozen but below 32°F is narrow. The ice layers formed under such conditions tend to be thin and the amount of heave small. When the temperature gradient is small, the zone in which the pore water is unfrozen but below 32°F is wide and the ice lenses tend to be thick and the amount of heave great."(3)

There are three essential conditions required for frost heave:

1. Below freezing temperatures
2. Available supply of water
3. A frost susceptible soil.

If one or more of these conditions are absent, frost heaving will not occur.

The first point is self explanatory. The water held by the soil will not freeze until the temperature drops below 32°F. Even below this temperature only the bulk water in the large pore spaces freezes. Actually, soil freezes at temperatures slightly lower than 32°F.(4) In fine pores there is a freezing point depression dependent on the size of the pore(5) and there is also a thin film of adsorbed water around the soil particles having quite different properties from the free bulk water. In very fine grained soils like clays, the freezing temperature may be as low as 22°F or 23°F.(3)

The second requirement states that it is necessary to have a reservoir of water (ground water table) close enough so that the growing ice lenses have a sufficient supply of water to maintain their growth. "Experience has shown that if the water table is more than six feet below the ground surface, the growth of ice lenses is made difficult."(6) If new water cannot reach the growing crystals, their growth stops. This is the basis for one method of combatting frost heave in roads. By placing an impermeable membrane between the sub-base and the ground water table, the moisture movement may be prevented thus stopping ice lens growth and frost heave.

The third requirement concerns the soil itself. Since the driving force behind the movement of water from the ground water table to the freezing zone is a suction developed at the ice lens it is necessary that the soil have a high capillarity, that is small voids. Clay has suitable capillarity, however, the water flow is not optimum because of its low permeability. Sand, on the other hand, has much higher permeability but much lower capillarity. The most frost susceptible soil is therefore an intermediate one, ie., silt. The most frequently used criterion for speciflying the frost susceptibility of a soil is that of Casagrande. From tests at Massachusetts Institute of Technology during the winter of 1928 - 29 he concluded that: "Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than three percent of grains smaller than 0.02 mm., and in very uniform soils containing more than ten percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than one percent of grains smaller than 0.2 mm., even if the ground water level was as high as the frost line."(7)

(i) <u>The Means of Moisture Migration</u>

In the past decade much work has been done in trying to determine the mechanism of ice lens growth and the factors influencing it. There are of course, conflicting theories, but these should be resolved as a consequence of more and more data becoming available.

Depending upon the state of packing of the soil, soil moisture can be translocated upward through the porous medium of the soil upon freezing by one or another mechanism:

1. as a vapour
2. as a liquid (in bulk or film)
3. in a simultaneous combination of vapour and liquid.

Most researchers agree that most of the moisture is moved from the groundwater to the downward freezing ice lenses in the liquid phase, although in an unsaturated soil with a high porosity the vapour phase would assume increased importance as the transfer state. Where vapour diffusion is the mechanism the driving force is the vapour pressure difference between the vapour pressure at the warmer end of the freezing soil system and the vapour pressure in the upper region of the soil system just below the ice, where it can be very small, or even negligible as compared with that at the ground water table.

In a soil system where the packing is very dense (i.e., small voids) or where the voids are fully saturated with water, moisture transfer in the vapour phase becomes ineffective and it is then that the liquid transport mechanism becomes paramount. Much effort has been directed to this area of soil freezing and an understanding of this mechanism is difficult or impossible without an appreciation of the structure of liquid water in the soil system.

a) Soil Water

This water is comprised of two very different types. In the centres of the voids is the free or bulk water having all the familiar properties of ordinary water. But surrounding the soil particles is a different type of water known as "adsorbed water" and it is this adsorbed water with its unusual properties that contributes so much to ice lens growth. It is the result of what is called the "Electric Double Layer".

Under certain conditions, when a soil particle is immersed in a medium of water, it acquires an electrical charge, residing at its surface, usually the result of absorption of ions. The nature and magnitude of this charge is greatly dependent on the liquid used as the dispersing medium. Colloidal clay particles dispersed in water usually carry a negative charge. This charge results in the solid particle being surrounded by an electric double layer. The first layer is formed at the surface of the soil particle and consists of the aforementioned negative charge. The negatively charged soil particles tend to surround themselves with an ionic atmosphere, i.e., with ions of the opposite charge (cations) thus forming the second (outer) layer of the electric double layer. This is the original concept as formulated by Helmholtz, however Gouy and Chapman modified this to result in the diffuse electric double layer theory. They formulated that the positive ionic atmosphere surrounding the adsorbed negative ions at the soil particle surface while predominantly positive was not completely positive, that is, there are a few negative ions. In the immediate vicinity of the surface the ionic atmosphere is fairly dense, but at greater distances from the surface the ionic density decreases until the net charge density is zero. The diffuse electric double layer consists of a ridgid part that is immobile under induced physical stresses and consists of the first negative-positive layer at the soil particle surface and the mobile part extending through the diffuse layer into the homogeneous interior of the surrounding water. Dipolar water molecules tend to orient themselves around the cations in the diffuse layer, that is, with their negative ends clustered around the positively charged cation. We can visualize what would happen if one were to apply a direct, external electric potential across the two ends of a column of soil. Anions in the electrolyte (water) would be attracted to the positive end while the cations would be attracted to the negative end. The migrating ions would drag their attracted dipolar water molecules along with them tending to move the diffuse part of the electric double layer over the rigid part by viscous shear. This moving film would also influence the free or bulk water in the channels to move with it. The direction of flow depends on whether the liquid carries positive or negative charges. This principle of electro-osmosis is used in engineering applications to de-water excavations.

As mentioned earlier the adsorbed water film has properties quite different from free water. Its density is much higher, being of the order of 1.4 gm./c.c. next to the soil particle, and gradually decreasing to 1.0 gm./c.c. in the free water. The viscosity of this water, as measured by the diffusion of ions near the surface, indicates that at the surface the viscosity may be a hundred times greater than for bulk water. The dielectric constant near the surface is about one-tenth that of free water. The boundary of the adsorbed water layer is indistinct, however, Jumikis writes that, "the thickness of the double layer, according to Rutgers, is of the order of 10^{-6} or 10^{-7} cm.".(4) Yong and Warkentin(6) report it to be about 15A° or 1.5×10^{-7} cm. deep.

The importance of the adsorbed layer in the overall scheme of ice lens growth is pointed out by Penner.

> "The ice lens sits directly on top of the soil particles seperated only by the adsorbed layer of water. In the lens growing process, molecules of water from the adsorbed layer become attached to the ice (and become part of it) which reduces its thickness. This can be replaced from water in the soil pores. In turn, the water removed from the pores can be replaced from a high water table. The connecting link between the pore water and the ice lens is the adsorbed layer of water on the soil particles which is believed to have great significance in ice lens growth"(5)

(ii) Reasons For Moisture Migration To The Growing Ice Lens

"In a given soil water system, soil moisture movement occurs as a result of a variety of causes. These may include temperature, concentration, pressure and other physical and chemical gradients."(8) The most important appears to be the pressure difference existing between the growing ice lens and the unfrozen water below it. Penner explains it using an analogy.

"At the freezing plane, the water in the soil turns to ice. This is, in effect, a drying action and water in the unfrozen soil beneath moves toward the freezing plane in the same way that water will move from moist soil to dry soil."(9)

He expands on this point in another publication. "Liquid water moves from wet regions to dry regions in a homogeneous soil because a difference in 'suction' exists. The movement of water in soil due to a suction difference is not different from the action of dry blotting paper when brought in contact with a drop of ink. In case of ice lensing, the suction is brought about by the change of water to ice at the freezing zone. Water flows from the unfrozen soil to the freezing zone to equalize the suction but at the same time the ice lens is growing and the suction difference is maintained."(5) Broms and Yao(10) suggest that this negative pore pressure may be high enough to cause particle re-orientation during freezing with resultant loss of shear strength. They also point out that since the negative pore pressure is mainly dependent on the soil system temperature a smaller grain size will give increased soil suction. This follows since the freezing point of free water in the voids decreases with decreasing average grain size.

While Penner believes "the mechanism of ice lensing can be but understood in terms of a theory based on the dimensions of the pore structure",(5) Jumikis, on the other hand, says that his studies "have convinced him that the electric, diffuse double-layer theory may be considered as the basis for studying freezing soil systems".(4) This is probably the result of the importance that Jumikis attaches to the film transport mechanism in soil water migration. Although admitting that the rate of flow in the film phase is low, compared to bulk water movement, he cautions that over a freezing period lasting several months, a considerable amount of soil moisture can flow from the groundwater. His studies indicate that when a vertical column of soil is subjected to freezing from the top downwards, as in nature, a curvilinear temperature gradient, $\partial T/\partial x$, sets in across the freezing soil system, from the top down. There is a resultant heat transfer upward from a region of higher temperature (groundwater) towards a region of colder temperature (the frozen layer). The thermal energy in its turn, initiates the upward migration of soil moisture in the porous soil system.

In his book <u>Soil Mechanics</u>, T. H. Wu(11) discusses a third possible cause of soil moisture migration, i.e., the ionic concentration gradient. The pore water in a soil system usually contains ions of dissolved salts. When the temperature of the pore water is lowered below the freezing point, crystallization of the water into ice begins in the centre of the pore space. The growing crystal drives the ions into the surrounding unfrozen water, thereby increasing the ionic concentration there. Thus there exists an ion concentration gradient dN/dx between the water surrounding the growing crystal and the unaffected water immediately below. This gradient causes water to move to the ice crystal. The gradient remains, however, because the newly arrived water is quickly frozen, and the action continues until the freezing front advances. This explanation of frost heave was offered by Cass and Miller in 1959.

(iii) Growth of the Lenticular Ice Crystal

After crystallization and subsequent growth has converted the bulk water in a soil pare to ice, "the ice front will be temporarily prevented from propagating downward between the soil particles until the temperature has been lowered sufficiently"(12) to freeze the supercooled water in the narrow interstices. "Before this occurs part of the absorbed water above the particle will freeze. As water is being removed from the absorbed layer into the ice phase it is replaced from below and an equilibrium thickness of water is maintained around the soil particle by continual replacement with supercooled water molecules".(12) The water supplied to the growing ice crystal via the adsorbed layer is supplemented by the gain of free water from adjacent pores resulting from the suction developed at the ice front. Particle displacement occurs as a result of both water loss and crystal growth. Displacement increases as water is drawn from further sources and the crystal continues to grow. "Since heat transfer is essentially unidirectional i.e., vertically upward, the crystal begins to assume the shape of a lens perpendicular to the direction of heat loss".(6) When all the available water within the neighbouring area has been exhausted and more water cannot be drawn in because of the high energy requirements, then a new lens begins, lower down, and the process is repeated. The growth of ice lenses may also occur in closed systems, i.e., where there is no reservoir of water, however, the ice lenses will be small since only the original water content is available and there is little or no heave.

2. DEPTH AND RATE OF FROST PENETRATION

The depth of frost penetration will directly affect the uplift forces acting on a structure, by merit of its affecting the size of the area over which the frost grip per unit area can act. The rate of frost penetration will affect the magnitude of the resultant heave and in view of these considerations, these two aspects of soil freezing will be briefly investigated.

The depth to which the frost line will advance in a soil is dependent on several factors:

1. The soil type and grain size distribution

2. The freezing index and associated temperature factors
3. Thermal properties of the soil water system
4. Nature of the pore water

The freezing index is a measurement of the severity of a winter and is expressed in "degree - days". one degree - day occurs when the mean air temperature for a given day is one degree below freezing, that is, 31°F. The freezing index may be found graphically if a curve is plotted of the mean daily temperature for one year. The freezing index would be equal to the area above the curve but below the 32°F line.

The temperature in the soil is also influenced by the amount of snow cover. A blanket of snow tends to insulate the ground from the cold air and thus retard frost penetration. That is why the frost penetration is usually greated under roads than adjoining shoulders because the roads are kept clear while the shoulders benefit from the insulating snow layer.

The density or state of compaction also is a factor in frost penetration depth. The frost may penetrate up to two feet deeper under a compacted path or roadway than it would in the adjecent soil in its natural state.

The most widely used formula employed for calculating the expected depth of frost penetration is the Stefan equation or one of its modifications by Berggren or Aldrich and Paynter. From thermal considerations J. Stefan arrived at this simple formula for the formation of ice in calm water:

$$x = \sqrt{\frac{48\,k_f \cdot F}{L}} \qquad \ldots.(2\text{-}1)$$

where x = depth taken downward from top of ground surface in feet.

k_f = thermal conductivity for frozen soil in BTU/hr./ft./°F

F = freezing index in degree - days

L = latent heat of fusion in B.T.U.

This equation tends to overestimate the actual depth of penetration, however, the previously mentioned modifications, employing more sophisticated assumptions in their derivation tend to give more accurate results.

The rate of frost penetration is also a factor in frost heave. The necessity of a small temperature gradient for maximum frost heave has already been discussed. Not only is the magnitude of the frost heave affected by the freezing rate, but the rate of frost heave is also influenced. In a paper devoted to the importance of freezing rate in frost action Penner arrives at the conclusion:

> "Increasing the rate of heat flow away from the freezing plane in all cases increased the rate of moisture flow and, consequently, the heaving rate."(13)

3. PARTIAL SOIL FREEZING

When a soil freezes, it is not usual for all of the water in the system to be crystallized. Rather an amount will remain unfrozen, although supercooled, as a result of the properties of the adsorbed water films and the water in the very fine pore passages. Thus, to be technically correct, soil freezing should instead be called partial soil freezing. The percentage of the total water content remaining unfrozen is dependent on several factors:

1. original water content
2. percent water saturation
3. freezing temperature
4. percent of active clay particles in the soil - water system
5. charge density of the soil particles

6. electrolyte concentration

7. soil structure.

For active clay soils there will be unfrozen water even at temperature depressions of 20°C.

The unfrozen water content is also influenced by the freezing history of the soil specimen, that is, whether it has been cooled or warmed to the test temperature. This is a result of particle re-orientation during freezing and thawing which, in turn, affects the inter-particle forces acting on the adsorbed water.

Unfrozen water content determinations are usually made using calorimetric methods and thermal equilibrium equations.

Recently, Dillon and Andersland(14) published an equation for predicting the unfrozen water content of a partially frozen soil using soil parameters. Their equation is:

$$W_u = \frac{ST}{T_o} \cdot \frac{1}{A_c} \cdot L.k.100 \qquad \ldots.(2\text{-}2)$$

S = average specific surface area of soil particles M^2/gm.

T = temperature of frozen soil, °K

T_o = temperature of initial freezing of soil pore water, °K

$A_c = \frac{I_p}{\% < 2\mu}$ = activity ratio

I_p = plasticity index

L = 1 for non expandable clays, 2 for expanding clays.

k = a constant = 2.8×10^{-4}, gm. of water/M^2

CHAPTER III

EXPERIMENTAL TECHNIQUES AND PROCEDURES

To experimentally measure the frost grip between a frozen soil and a material the apparatus shown in Fig. 2 was used. The soil was placed into the hollow core of the mold, compacted, and then the mold was frozen in a freezer. After freezing, the mold was removed from the freezer, placed in a compression tester and the frozen soil core was loaded until freed from the mold. The ultimate force required to free the core from the mold was recorded, this being divided by the inside lateral surface area of the mold to give the frost grip.

This, basically, was the approach used by Trow(1) in his initial studies. The greatest difference in the two techniques was the fact that the present investigation used molds having circular cross - sectional cores whereas Trow's were square when viewed in plan. It was thought that the circular cylinder approach might avoid discrepancies due to corner effects, with a secondary benefit of allowing more uniform compaction over the area, since compaction was achieved using a manual cylindrical drop hammer.

The molds were made of concrete with two rings of

Fig. 2a Nominal Dimensions of Molds

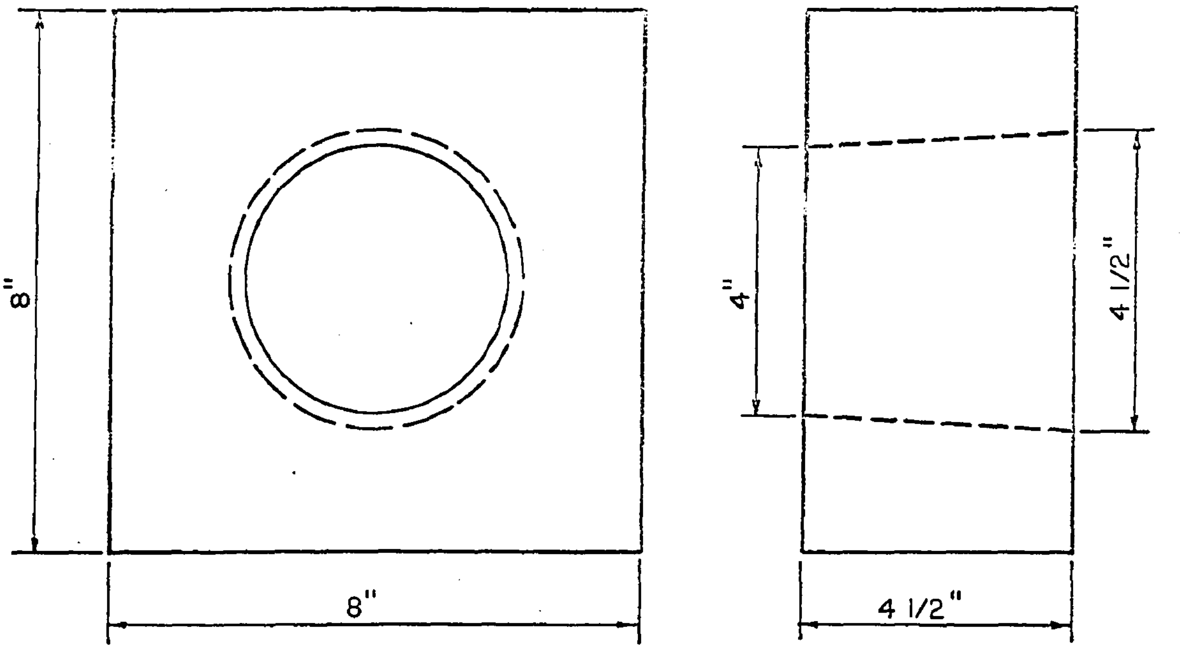
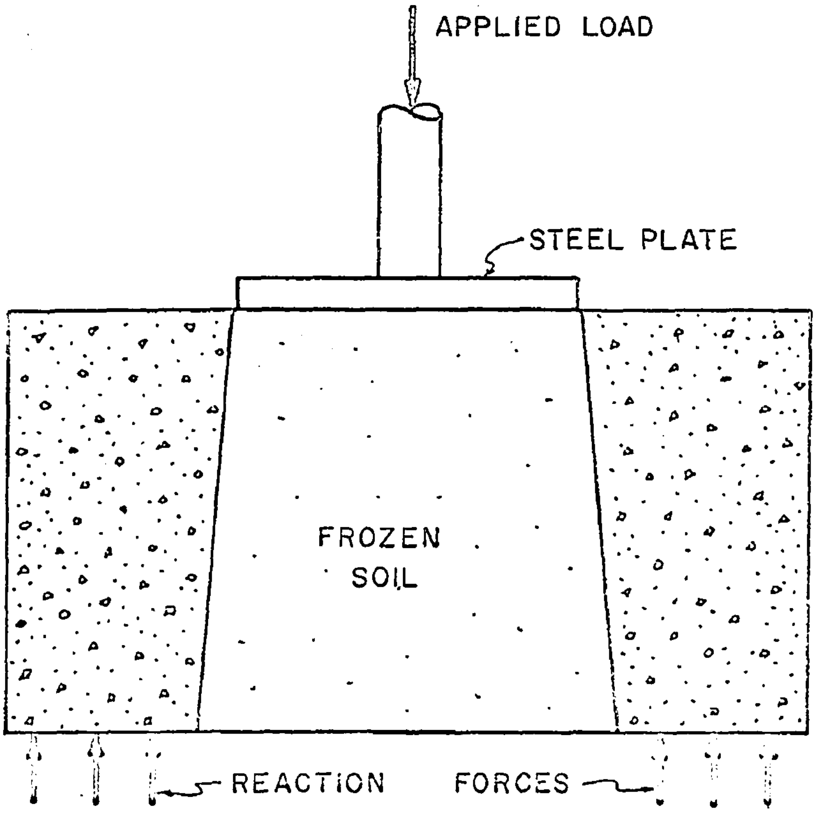


Fig. 2b Schematic of Testing Arrangement



reinforcing steel placed approximately one inch from the top and bottom surfaces. This reinforcing was instrumental in prolonging the useful life of the molds. Without the reinforcing the molds would have disintegrated from the cracking induced by:

1. the repeated stressing in the compression machine
2. the stresses set up by the expansion of soils frozen with a high degree of saturation
3. the deteriorating effects of the repeated freeze - thaw cycles on the concrete.

Two of the molds were poured around metal linings, one stainless steel, the other galvanized steel, to compare the properties of the frost grip on these materials with those on the concrete.

For increased strength and durability the molds were poured with a rich concrete mix, the proportions being 1 cement: 1 sand: 1 3/8" crushed stone with water being added to give good workability. This mix gave a compressive strength of 6,000 p.s.i. using 3" x 6" test cylinders cured under water at room temperature for 28 days.

The useful life of a concrete mold was approximately thirty or thirty-five tests, after which frost grip measurements were consistently lower than values obtained when the molds were fresh. This lessening of the frost grip was probably the result of the progressive smoothening of the mold walls thus lowering that fraction of the frost grip which can be attributed to the frictional resistance of the two solid bodies sliding past one another. This is supported by the fact that fresh, unused molds had a surface texture that felt like a very fine sand paper. After thirty or thirty-five tests the surface, although appearing unchanged to the naked eye, had taken on a smooth polished effect. The conclusion of smoother surfaces giving lower frost grip is also supported by the molds with the metal linings. The galvanized steel gave consistently lower results than the concrete, and the smoother stainless steel gave lower results still. (See Figs. 8 and 9)

Three soils were used in the course of the investigation. The first was a crushed Ottawa sand, designated Sand No. 1, having a uniformity coefficient of 1.50 and the grain size distribution curve shown in Appendix I. The second was a natural, well-graded sand, referred to as Sand No. 2. The third was a sandy clay from the

Essex County pit. The grain size distribution curves are also shown for both of these soils in Appendix I.

The work was begun using Sand No. 1, since being uniform and non-cohesive it was easy to work with and gave good results while the experimental techniques were being perfected. The bulk of the work was done using Sand No. 2, again because being granular it was easy to work with and also because selected granular material is usually specified as backfill material around footings.

Owing to the difficulty in handling and preparing the Essex County clay it was decided to test the effect of clay-size particles on the frost grip by preparing soil mixes. Soil Mix No. 1 consisted of 30% by weight of the Essex County clay and 70% by weight of Sand No. 2. Soil Mix No. 2 consisted of 50% by weight of Essex County clay and 50% by weight of Sand No. 2. The grain size distribution curves for both of these soil mixes are in Appendix I.

When preparing the specimens for freezing, the molds were placed in ordinary steel laboratory pans (Fig. 3) with a layer of heavy waxed paper between the bottom of the mold and the pan. The purpose of the wax paper was to prevent the wet soil, upon freezing, from

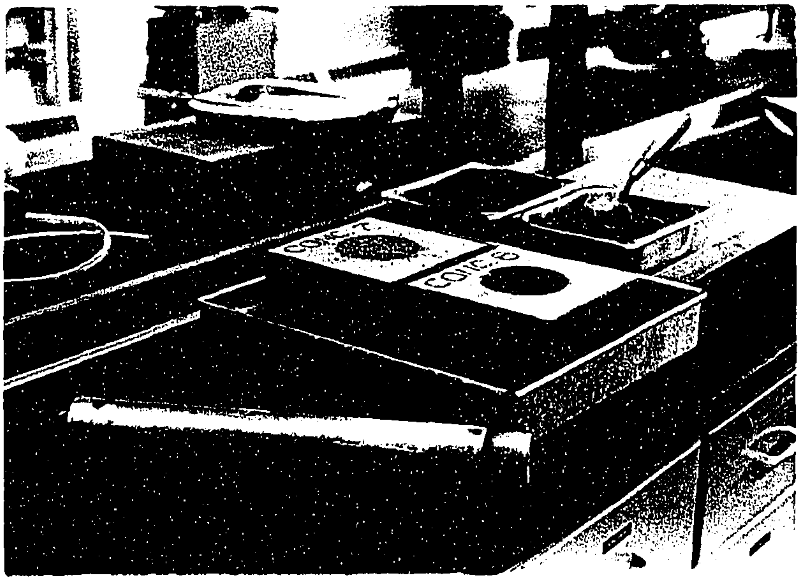


Fig. 3 Molds Being Filled, with Compacting Hammer in Foreground

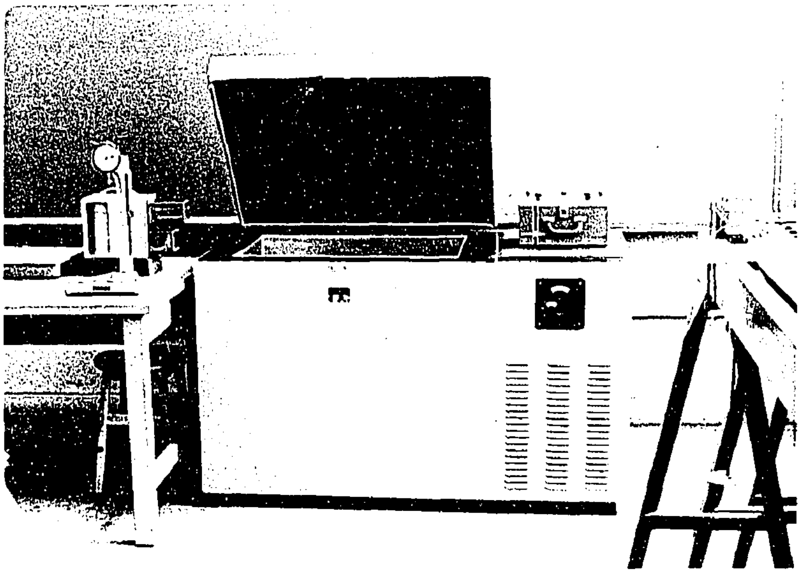


Fig. 4 Freezer

developing a frost grip on the pan and thus prevent easy removal of the molds from the pans at grip testing time. The waxed paper also helped to discourage water leakage when soils with a high degree of saturation were being used.

The dry soil to be used was weighed and placed in a large mixing pan. Sufficient soil was prepared so that all the molds could be filled from the common batch so as to facilitate correct moisture content selection and determination. Knowing the dry weight of the soil and the desired moisture content the amount of water required could be easily calculated. This was added to the soil in the large mixing pan which was then thoroughly mixed by hand with a trowel. The prepared soil was then distributed approximately equally among a number of smaller mixing pans, one pan for each of the molds being prepared at the time, usually five. Each pan was weighed before and after packing the molds so that the exact amount of soil packed into the molds could be calculated by subtraction. Compaction was affected by a manual hammer having a 5½ pound weight falling through 12 inches. (Fig. 3) The compactive effort was varied from zero blows (the soil being pushed in by a trowel) for maximum porosity to fifty blows, each of five layers being struck ten times for minimum porosity.

During compaction a representative soil sample was taken from each mold for the purpose of accurately determining the moisture content. The average of the moisture contents of the samples was used as the common moisture content of the entire batch for data purposes.

The filled molds were then placed in the freezer (Fig. 4) along with the steel punch-out plate. The idea behind having the punch-out plate cooled was to keep the specimen from thawing during the punch-out operation as much as was possible. This was further realized by having the freezer only ten feet from the testing machine. This kept exposure time at room temperature down to one or two minutes when punching out the plugs, and no thawing whatever was in evidence throughout the course of the work.

Throughout the tests the freezer temperature was kept constant at $0^{\circ}F$, except for a few comparison tests when the temperature was dropped in increments from 30° to $0^{\circ}F$. The temperature of $0^{\circ}F$, was low enough to ensure complete freezing in twenty four hours of all the water in the sand samples. This hypothesis is supported by the work of Dillon and Andersland(14) who concluded that there would be zero unfrozen water content for soils lacking particles finer than two microns. Referring to equation (2 - 2) of chapter II, Skempton's activity ratio,

$$A_c = \frac{I_p}{\% < 2\mu}$$

and the term $1/A_c$ approaches zero.

In the case of the soil mixes, where particles finer than two microns are present, the equation gives an expected unfrozen water content of approximately three percent. (See Appendix III)

The specimens were removed from the freezer approximately twenty-four hours later and tested one at a time. (Figs. 5 and 6) The specimens were loaded to failure quickly to avoid excessive plastic flow. The average time for failure was thirty seconds, with high capacity specimens taking longer and low capacity specimens failing more quickly.

After failure the extruded soil cores (Fig. 7) were dessicated in an oven at 110°C, pulverized and the material used again.

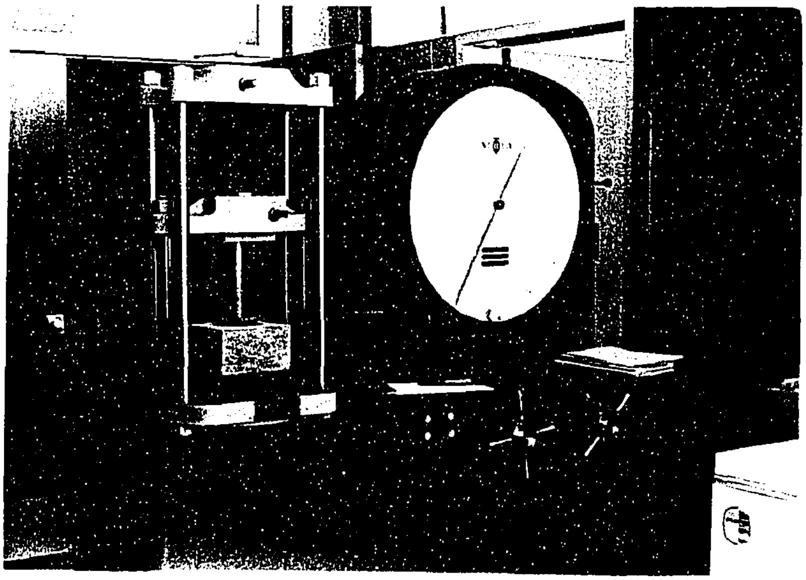


Fig. 5 Compression Testing Machine, with Mold in Position

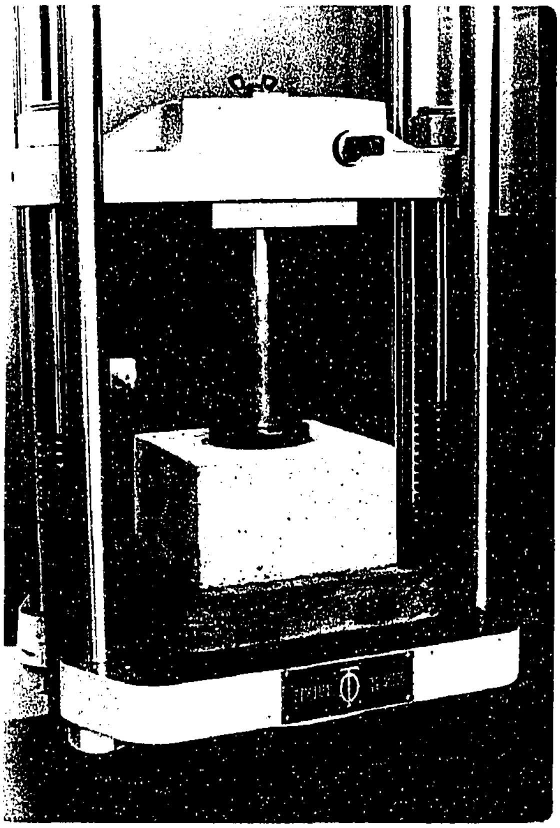


Fig. 6 Mold in Testing Position, Showing Steel Loading Plate and Wooden Support Ring

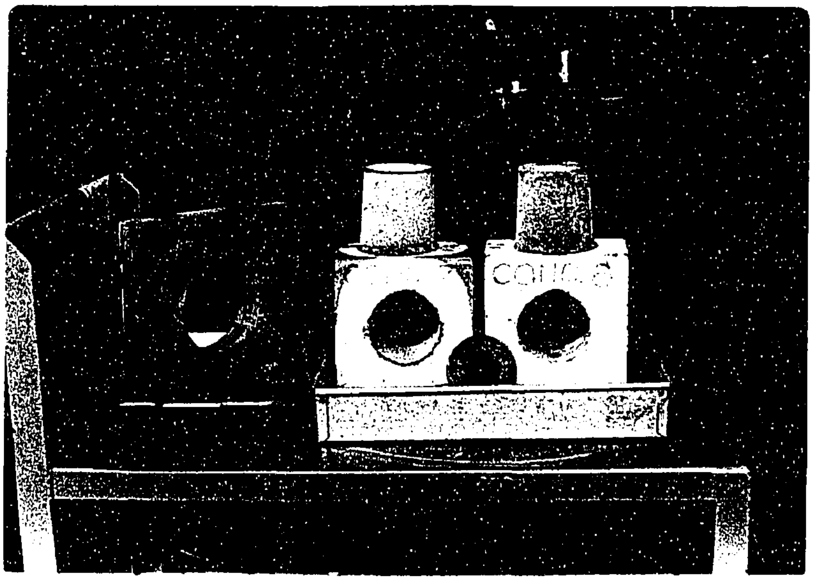


Fig. 7 Extruded Soil Cores After Testing

CHAPTER IV

DISCUSSION OF RESULTS

Probably the first factor to come to mind when analysing the factors influencing the magnitude of the frost grip would be the amount of moisture in the soil. It was therefore decided to begin the work with an investigation of the effect of water content on the frost grip.

Figures 8 and 9 show the not unexpected results of this initial phase of the work. Fig. 8 was derived using Sand No. 1 and showed a straight line variation of the frost grip with the moisture content (dry weight basis). For this series of tests the porosity was kept constant at 47% and the only variable was the moisture content. The heavy portions of the curves represent the limits of water content between which the work was carried out. These limits were imposed by the fact that at high moisture contents (beyond 19%) the water would drain under the force of gravity from the permeable sand and at low moisture contents (below 4%) the moisture could not be distributed uniformly. Also, at low water contents the absorption of water by the oven dry soil

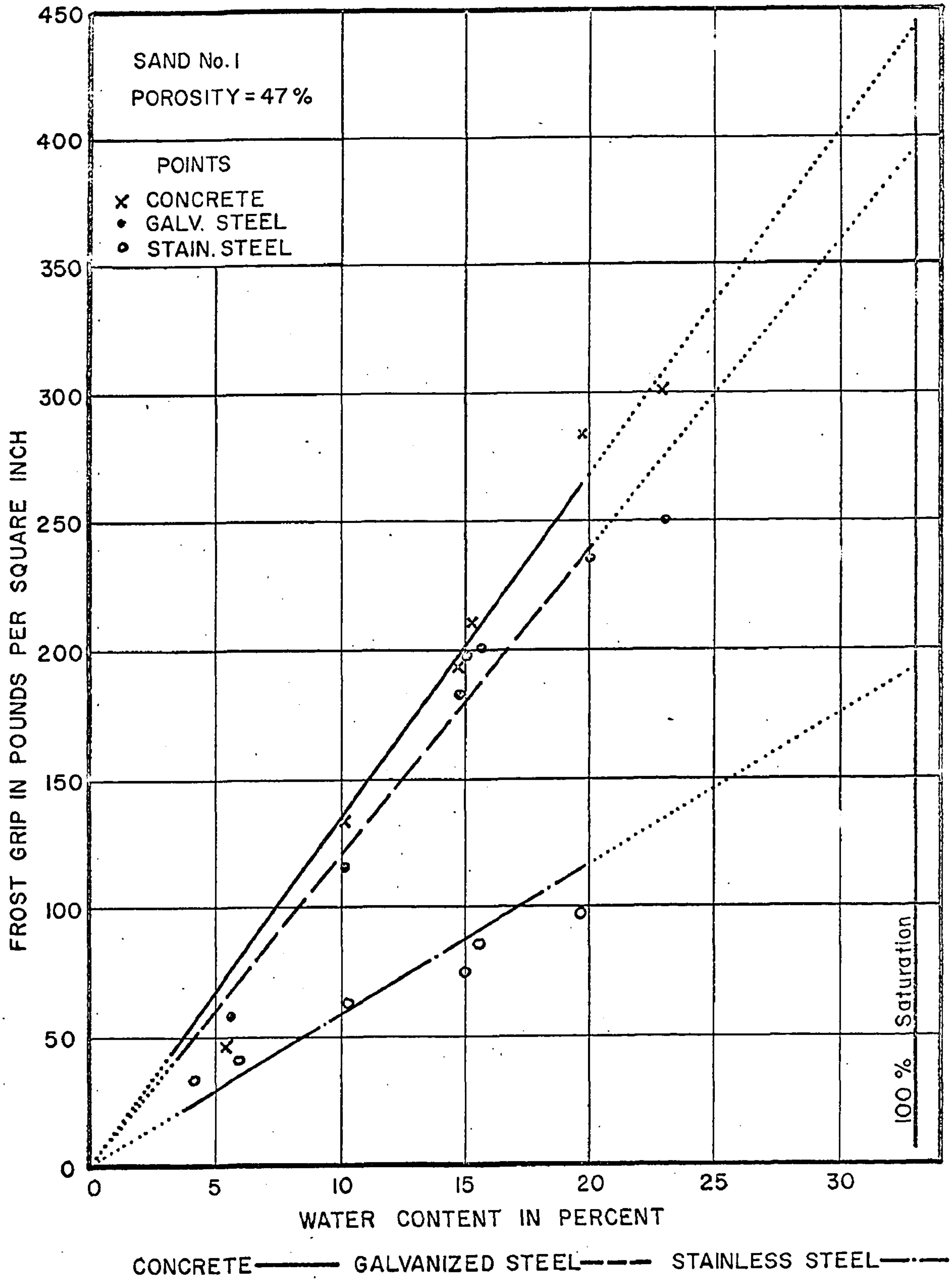


Fig. 8 Frost Grip vs. Water Content, Sand No. 1.

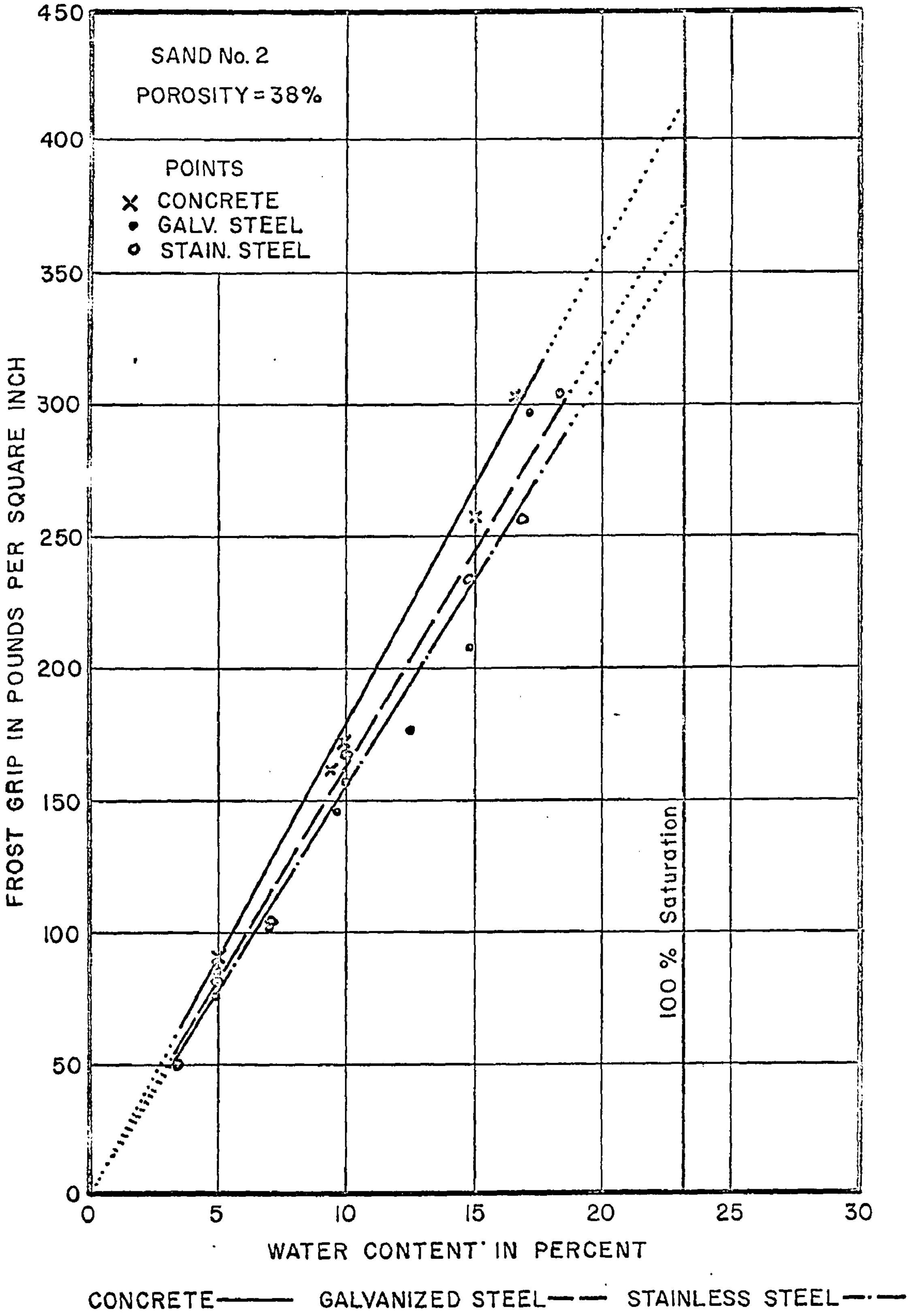


Fig. 9 Frost Grip vs. Water Content, Sand No. 2.

grains would mean there was insufficient water left to form an easily measurable frost grip. The dotted portions of the lines represent extensions to show:

(a) when the curves are produced backwards to zero water content they intersect the frost grip axis at zero, that is, they go through the origin.

(b) when the curves are produced forwards to intersect the line of one hundred percent saturation the value for concrete fell within Trow's(1) range of results of 395 p.s.i. to 495 p.s.i. Although Trow does not identify the soil he worked with, it was probably a frost susceptible silt and not a purely granular material as used in the present investigation.

Fig. 9 was derived using Sand No. 2, with the porosity held constant at 38%. This was the average porosity resulting from applying the same compactive effort as was applied to Sand No. 1, that is, two layers, both struck four times with the compacting hammer. The results were similar to those in Fig. 8, with the curves for all three materials extending back through the origin.

Besides showing the influence of moisture content Figures 8 and 9 also show how the frost grip is affected by the type of material to which the frozen soil adheres.

A pattern emerges from Figures 8 and 9 since for both soils it was found that concrete gave the highest value of frost grip, followed by galvanized steel* and then stainless steel** for a given water content. The influencing factor in this result would seem to be the contact surface texture of the material suffering the grip. The galvanized steel gave values ten percent lower than the rougher surfaced concrete for both sands, and the smoother stainless steel gave results approximately 61% less than concrete for Sand No. 1 and 13% less for Sand No. 2.

The equations of the straight lines in figs. 8 and 9 are of the type:

$$f = mW \qquad \text{....(4-1)}$$

where f = frost grip in pounds per square inch

m = slope of the line in p.s.i./percent

and W = water content in percent (dry basis)

Therefore, for water content, W_1, $f_1 = mW_1$ and for a second point on the same line $f_2 = mW_2$.

$$\frac{f_1}{f_2} = \frac{mW_1}{mW_2} \qquad \text{....(4-2)}$$

---

* - 22 Gauge Galvanized Steel Sheet
** - 20 Gauge, #302/304 Stainless Steel

or $$f_1 = \frac{W_1}{W_2} \cdot f_2 \qquad \ldots.(4\text{-}3)$$

The ratio W_1/W_2 could be called a moisture content correction factor and was used in subsequent work where it was desired to hold the water content at a specified percentage.

Having established the influence of the water content the next step was to determine the effect of soil porosity. During this stage of the project all the influencing variables were kept constant except the porosity. Unavoidable small deviations from the reference water content were compensated for by multiplying the experimental frost grip by the aforementioned ratio, (reference W.C.)/(actual W.C.). The porosity was varied by varying the number of blows given per sample with the compacting hammer.

The plotted data showed a linear decline in the frost grip as the porosity of the soil was increased. This same tendency was evident when the test water content was changed from a constant 10% to 5% and then 15% and the test series repeated. Figures 10 and 11 show the experimental points, each point representing the average frost grip and porosity of a batch consisting of three

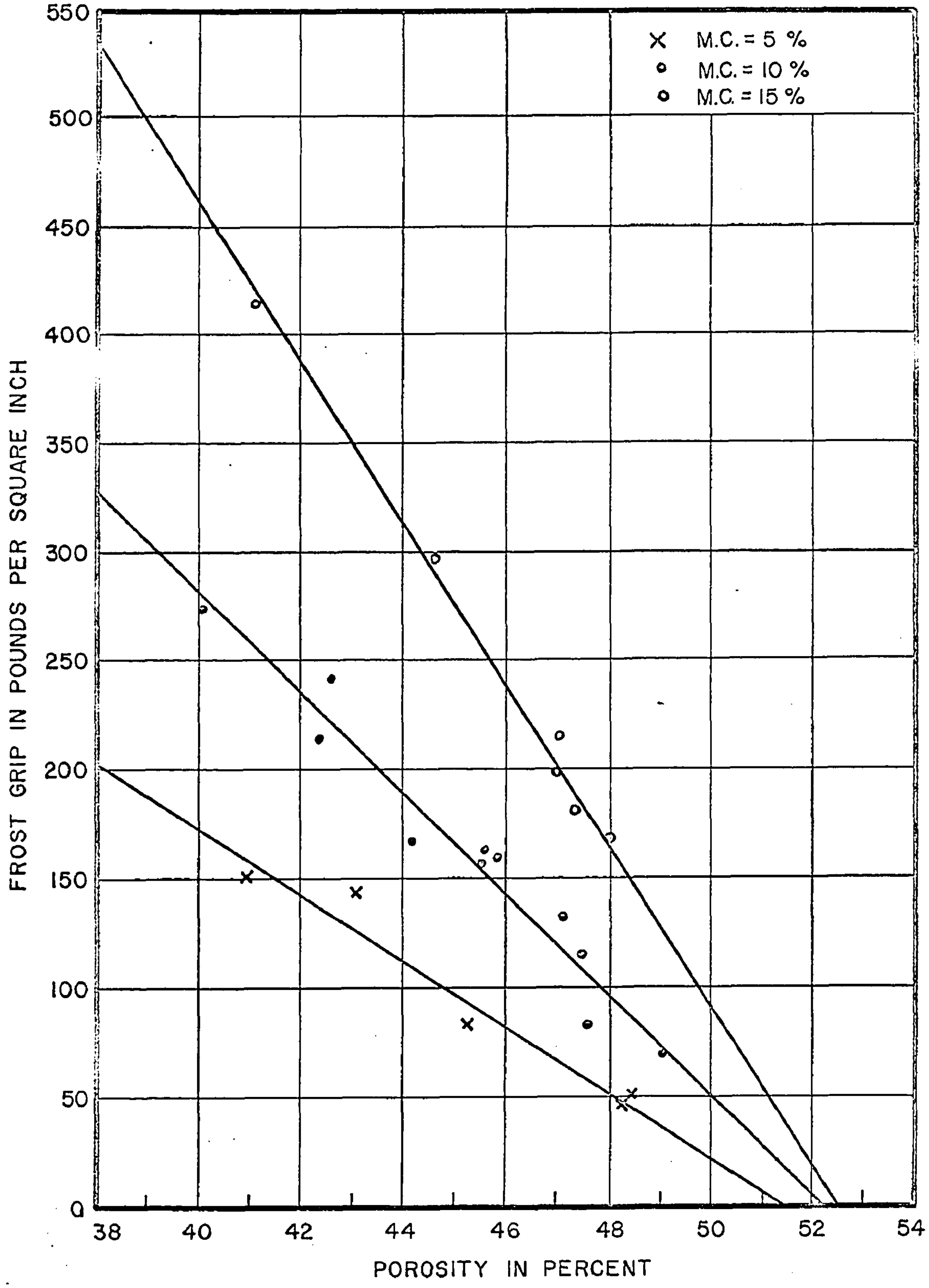


Fig. 10 Frost Grip vs. Porosity, Sand No. 1, Experimental Lines of Best Fit

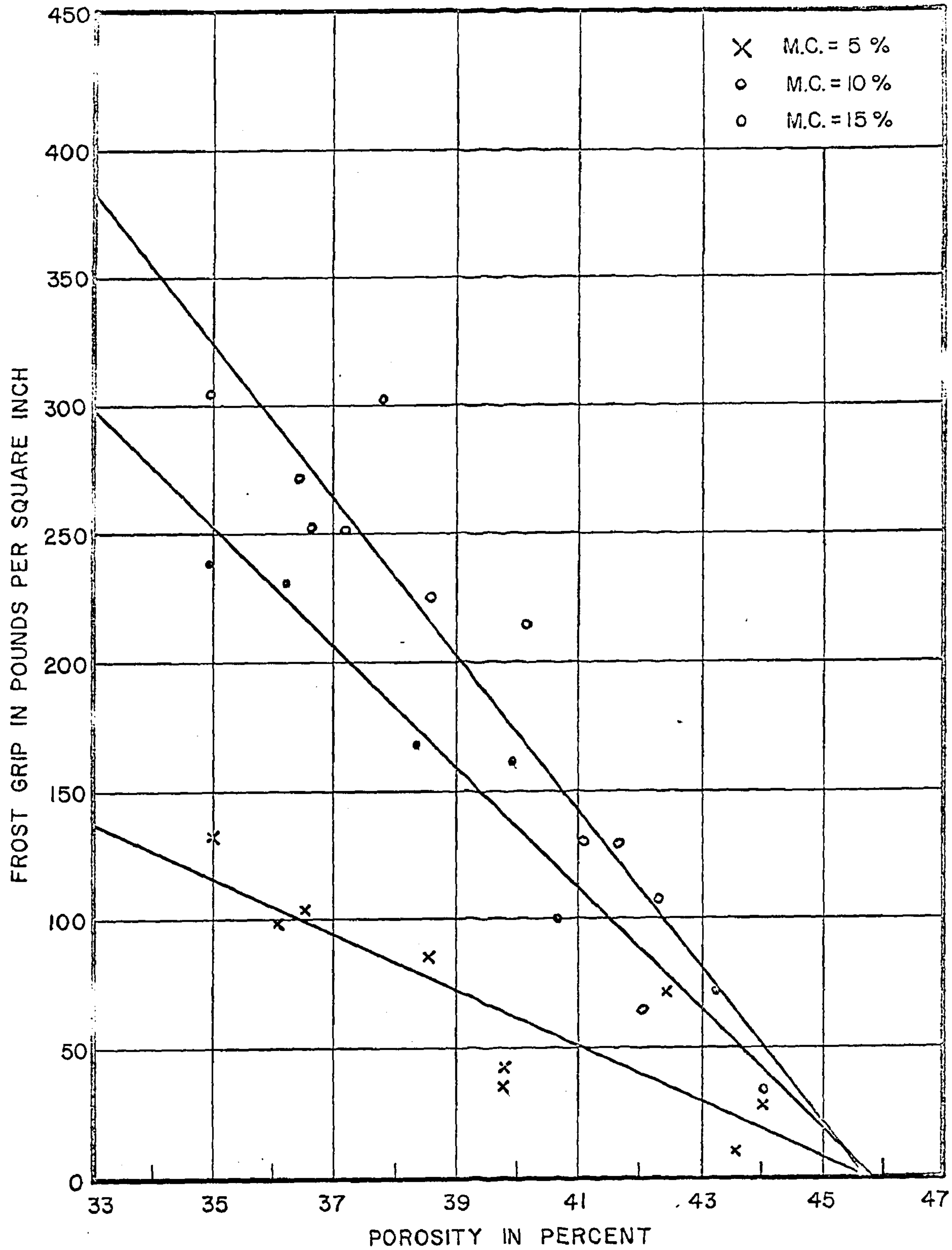


Fig. 11 Frost Grip vs. Porosity, Sand No. 2, Experimental Lines of Best Fit.

samples. This is consistent with all of the other figures where each point is an average derived from a sample of three. The lines through the points were established using the statistical line of best fit (Appendix III contains a sample calculation).

It is interesting to note that the scatter of the points about the regression line for Sand No. 1 (Fig. 10) is considerably less than that for Sand No. 2 (Fig. 11). This is probably due to the range in particle size for the two sands. Sand No. 1 is referred to as a uniform material, having a uniformity coefficient of only 1.50 and its particle sizes falling in the range 1.4 m.m. to 0.25 m.m. Sand No. 2 has a uniformity coefficient of 2.53. Thus the composition of random samples of Sand No. 1 would not be as likely to vary as much as would samples of Sand No. 2 where the percentage of some particle sizes may be higher or lower, and therefore affect the contact area between soil and mold.

Figures 10 and 11 show that the lines of best fit for each sand tend to converge at a certain point, although not the same point for Sand No. 1 as for Sand No. 2. It was found that the intercepts on the porosity axis (frost grip equal to zero) (Fig. 11) for sand No. 2

for the lines of best fit were 45.71%, 45.87% and 45.70% for water contents of 5%, 10% and 15% respectively. Considering the scatter of the experimental data about the regression lines, it appears highly significant that the intercepts should fall so closely on one another. With some degree of approximation, it may be seen that this converging tendency is also evident in the tests with Sand No. 1. In view of the fact that Figures 8 and 9 (and other selected graphs in Appendix V) show that the frost grip varies linearly with the water content, it was known that, ideally, the ordinates for a given abscissa in Fig. 10 and 11 should be in direct proportion to the moisture content, that is, at a given porosity, the frost grip for the M.C. = 10% line should be twice the value of the frost grip given by the M.C. = 5% line. Using this premise and assigning an average value for the common intercept on the porosity axes the idealized results of Figures 12 and 13 were constructed. The solid portions of these curves represent the range of porosity over which the tests were conducted. An interesting point raised by these idealized curves is the conjecture that if a porosity equal to the intercept porosity could be achieved, there would be no frost grip for the indicated

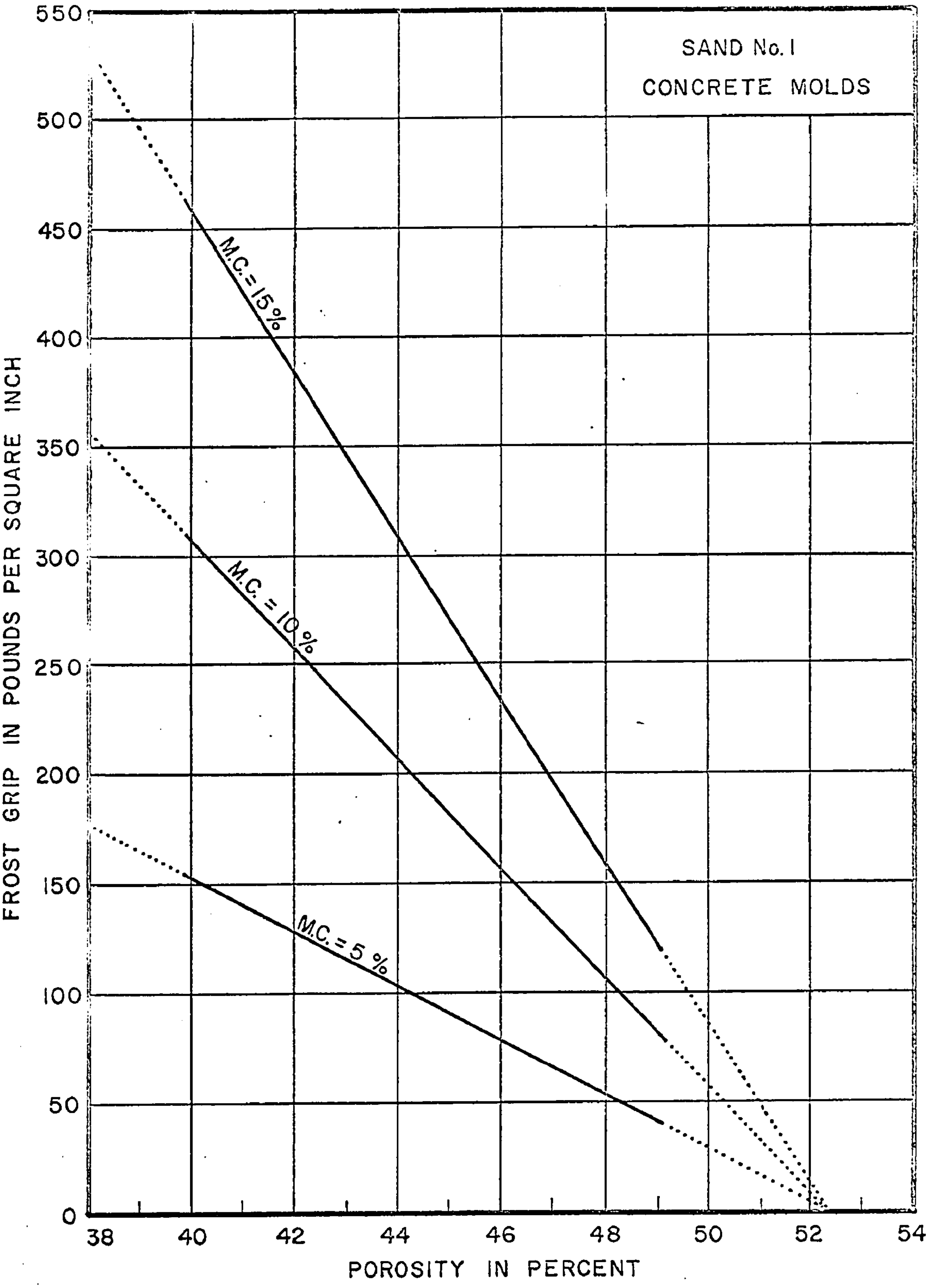


Fig. 12 Frost Grip vs. Porosity, Sand No. 1. Idealized Curves

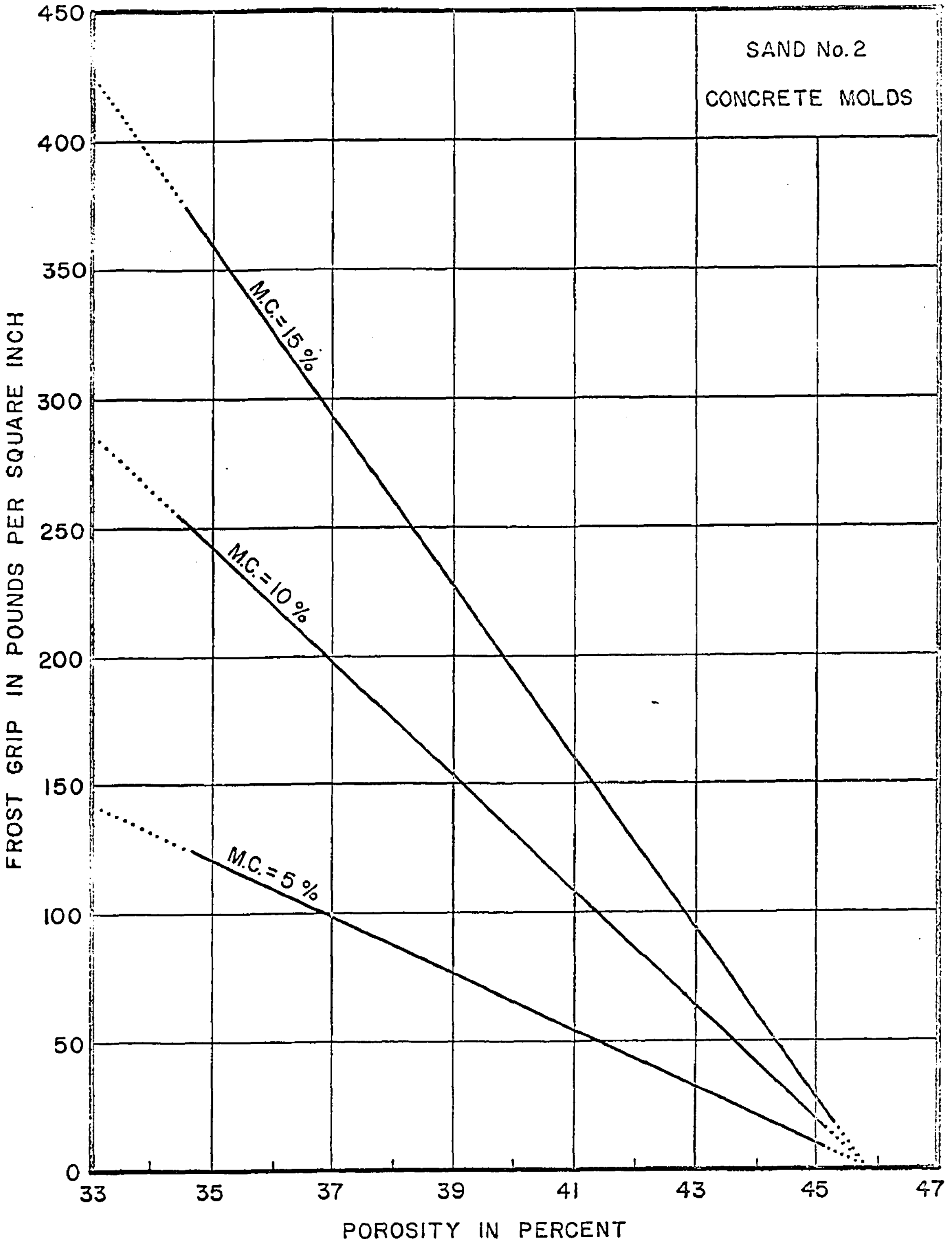


Fig. 13 Frost Grip vs. Porosity, Sand No. 2. Idealized Curves

values of water content.

A simple mathematical analysis can be made on the idealized "frost grip vs. porosity" curves.

To assist in the mathematical analysis Figures 14 and 15 were constructed from Figures 12 and 13, showing the porosity axis beginning at the origin.

For a straight line: $y = mx + b$ ....(4-4)

or $f = mn + F$ ....(4-5)

where: f = frost grip in pounds per square inch

m = slope of the line in p.s.i./percent

n = porosity in percent

and F = intercept on the frost grip axis at $n = o$, in p.s.i.

Eq'n. (4-5) may be re-arranged:

$$f = m(n + \frac{F}{m}) \qquad \text{....(4-6)}$$

However, the negative slope, $m = \frac{F}{-N}$ ....(4-7)

where N = the intercept on the porosity axis

From Eq'n. (4-7) $\frac{F}{m} = -N$

Substituting in Eq'n.(4-6) gives,

$$f = m(n - N)$$

$$\text{or} \quad f = - m(N - n) \qquad \text{....(4-8)}$$

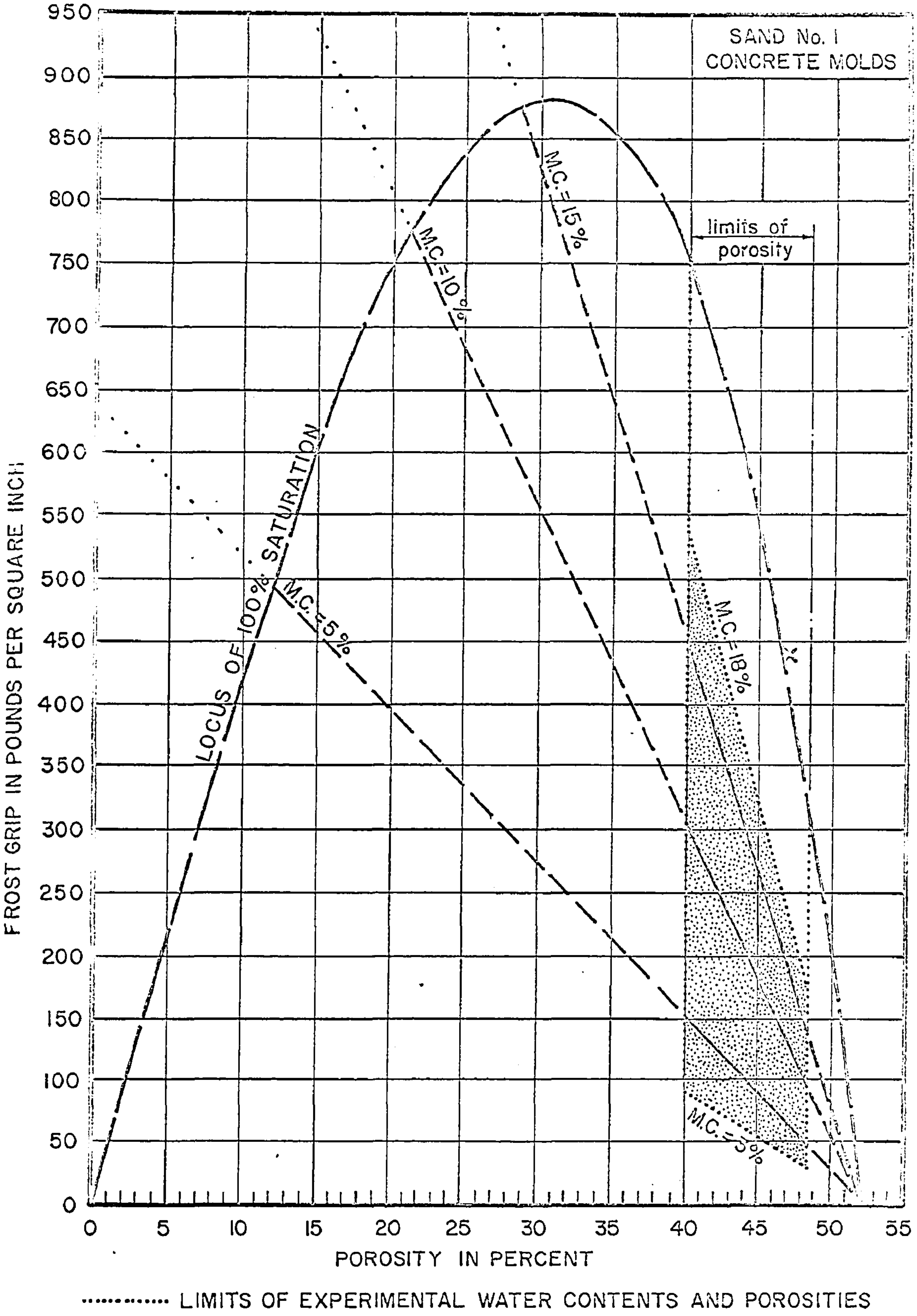


Fig. 14 Theoretical Limit of Frost Grip Compared with Experimental Range of Frost Grip, Sand No. 1.

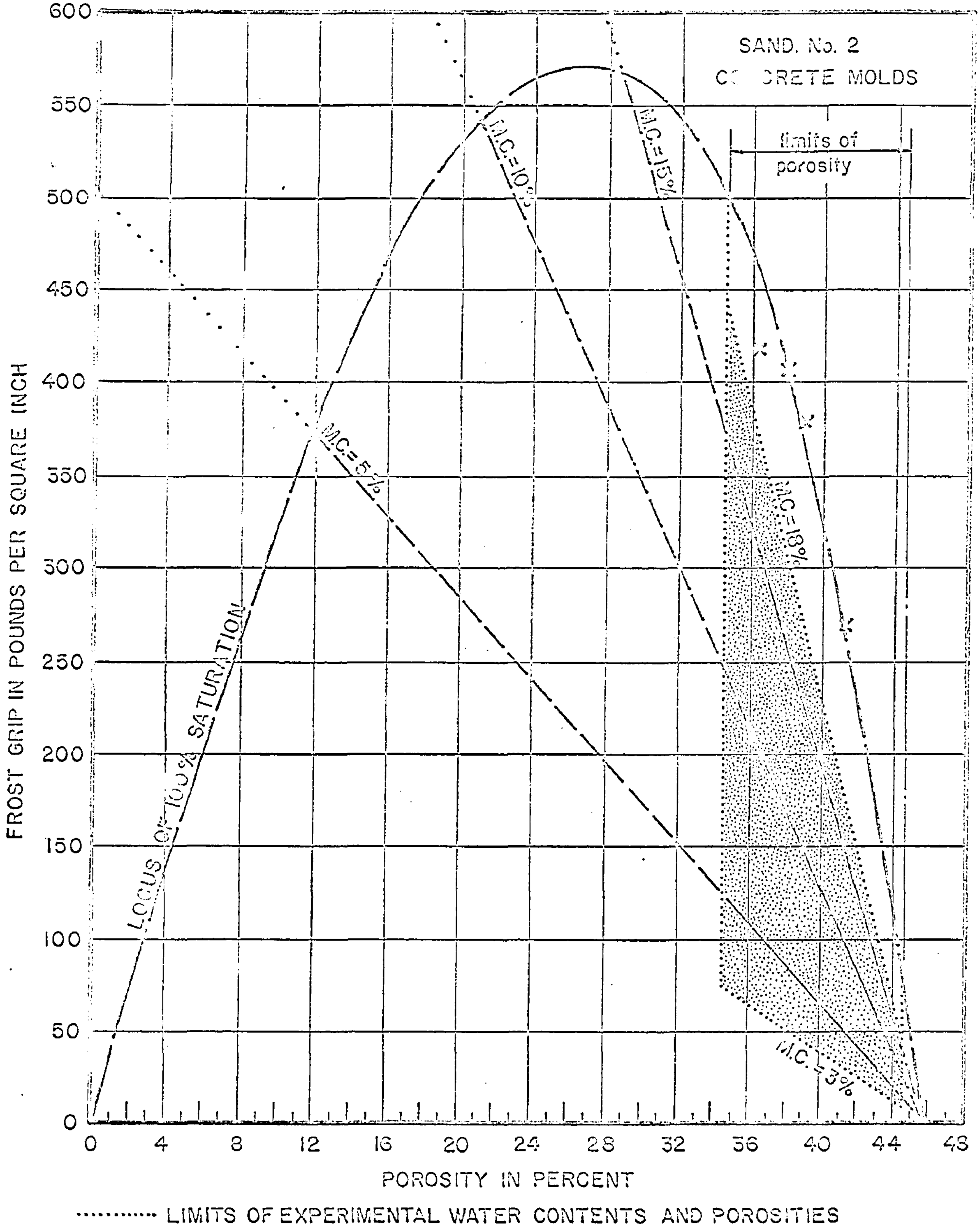


Fig. 15 Theoretical Limit of Frost Grip Compared with Experimental Range of Frost Grip, Sand No. 2.

Furthermore, the general expression for "m" from (4-7) may be inserted to give:

$$f = \frac{F}{N}(N - n) \qquad \text{....(4-9)}$$

This general equation holds true for any sand (by changing the intercept, N) and for any water content (by changing the intercept, F) within the investigated limits of $3\% < W < 18\%$ and $N_1 < n < N_2$ where N_1 and N_2 represent the practical range in porosity for the given sand.

Since, the slopes of the lines for a given sand are directly proportional to the water content, once the values of N and F are established for one water content the lines for different water contents may be found using the relationship:

$$\frac{m_1}{m_2} = \frac{W_1}{W_2} \qquad \text{....(4-10)}$$

If, for example, the line for W.C. = 5% is known then

$$m = \frac{W}{5} \cdot m_5 \qquad \text{....(4-11)}$$

or, using F and N:

$$\frac{F}{N} = \frac{W}{5} \cdot \frac{F_5}{N} \qquad \text{....(4-12)}$$

where F_5 is the intercept on the frost grip axis for the W.C. = 5% line. Eq'n. (4-12) may be substituted into the general equation (4-9) to give:

$$f = \frac{F_5}{N} \cdot \frac{W}{5}(N - n) \qquad \text{....(4-13)}$$

Of course, any workable water content may be used as the reference. As long as the values F_5 and 5 are changed accordingly.

The usefulness of Eq'n. (4-13) may be illustrated by substituting data from Sands No. 1 and 2.

For Sand No. 1, $F_5 = 640$, $N = 52.3$

$$\therefore f = 2.4474 \cdot W \cdot (52.3 - n) \qquad ...(4\text{-}14)$$

for $3\% < W < 18\%$ and $40\% < n < 48.5\%$.

For Sand No. 2, $F_5 = 502$, $N = 45.8$

$$\therefore f = 2.192 \cdot W \cdot (45.8 - n) \qquad ...(4\text{-}15)$$

for $3\% < W < 18\%$ and $34.5\% < n < 44.5\%$.

The limits given for water content and porosity present the water contents and porosities used in the lab and found in nature.

The straight lines of Figs. 12 and 13 do not extend indefinitely but are limited by the fact that for the given water content there is a porosity for which that water content provides 100% saturation. Furthermore, in nature, the porosity is also limited. The curved line shown in both Figs. 14 and 15 represents the locus of the points of 100% saturation and defines the theoretical limits of the frost grip.

Using Sand No. 2 as the example, the equation of this limiting locus may be found using:

$$f = 2.192 \cdot W \cdot (45.8 - n) \qquad \ldots.(4\text{-}15)$$

and $$Se = WG \qquad \ldots.(4\text{-}16)$$

where S = degree of saturation in percent

e = void ratio

and G = specific gravity of soil particles = 2.65 .

Since for every point on the locus S = 100%

$$\therefore \text{ from (4-16) } W = \frac{100}{2.65} \cdot e$$

$$\text{or } W = 37.736 \cdot e \qquad \ldots.(4\text{-}17)$$

Substituting this in (4-15) yields

$$f = 82.7215 \cdot e \cdot (45.8 - n) \qquad \ldots.(4\text{-}18)$$

however, since

$$e = \frac{n}{100 - n}$$ ("n" expressed in %)

equation (4-18) becomes:

$$f_t = 82.7215 \frac{(45.8n - n^2)}{100 - n} \qquad \ldots.(4\text{-}19)$$

The subscript "t" on "f_t" denotes the theoretical frost grip. This distinction is made because equation (4-11) was developed for use over the range $34.5\% < n < 44.5\%$ and does not necessarily hold for porosities outside this range.

Differentiation gives a maximum theoretical frost grip value of 575.6 p.s.i. at porosity n = 28.4% and water content W = 13.5%.

A similar deviation can be made for Sand No. 1 with the resulting equation of the locus being:

$$f_t = 92.0073\,\frac{(52.3n - n^2)}{100 - n} \qquad \ldots.(4\text{-}20)$$

and the maximum frost grip of 880.5 p.s.i. being achieved at porosity n = 30.95% and water content W = 16.85%.

The optimum porosities in both cases lie outside the limits of practicality and could not be physically realized.

These idealized graphs also allow the derivation of a term which may be applied as a correction factor for variations in porosity similar to that developed in Eq'n. (4-3) for water content. Using Eq'n. (4-8)

for case 1.: $f_1 = -\,m_1(N_1 - n_1)$

and for case 2.: $f_2 = -\,m_2(N_2 - n_2)$

If, in a test series, the moisture content and sand type are unchanging then $m_1 = m_2$ and $N_1 = N_2 = N$

and the ratio $\dfrac{f_1}{f_2} = \dfrac{(N - n_1)}{(N - n_2)}$

$$\text{from which } f_1 = \frac{(N - n_1)}{(N - n_2)} \cdot f_2 \qquad \text{....(4-21)}$$

The ratio $(N - n_1)/(N - n_2)$ may be used as a porosity correction factor. Thus, if it were desired to investigate some aspect of the frost grip keeping the porosity constant (say 40%) and the calculated porosity for the sample was 39%, the resultant experimental frost grip, f_2, at 39% could be corrected by substituting into Eq'n. (4-21) the values for the terms and deriving the theoretical frost grip at the desired porosity of 40%.

Fig. 16 is the result of the hypothesis that the depth of the frozen soil cores could influence the magnitude of the grip, that is, if the molds were 12" or 24" deep would the results differ from what was obtained using the 4½" deep molds? An attempt to answer this question was made by filling the molds to various depths and then noting how the grip was influenced. The scatter of the data made a firm conclusion difficult and so statistical methods were applied. Appendix III shows the calculation of the line of best fit (shown with a broken line in Fig. 16) and the application of a "t" test. This resulted in the conclusion that the depth of the mold had no significant influence, with the modifier that the

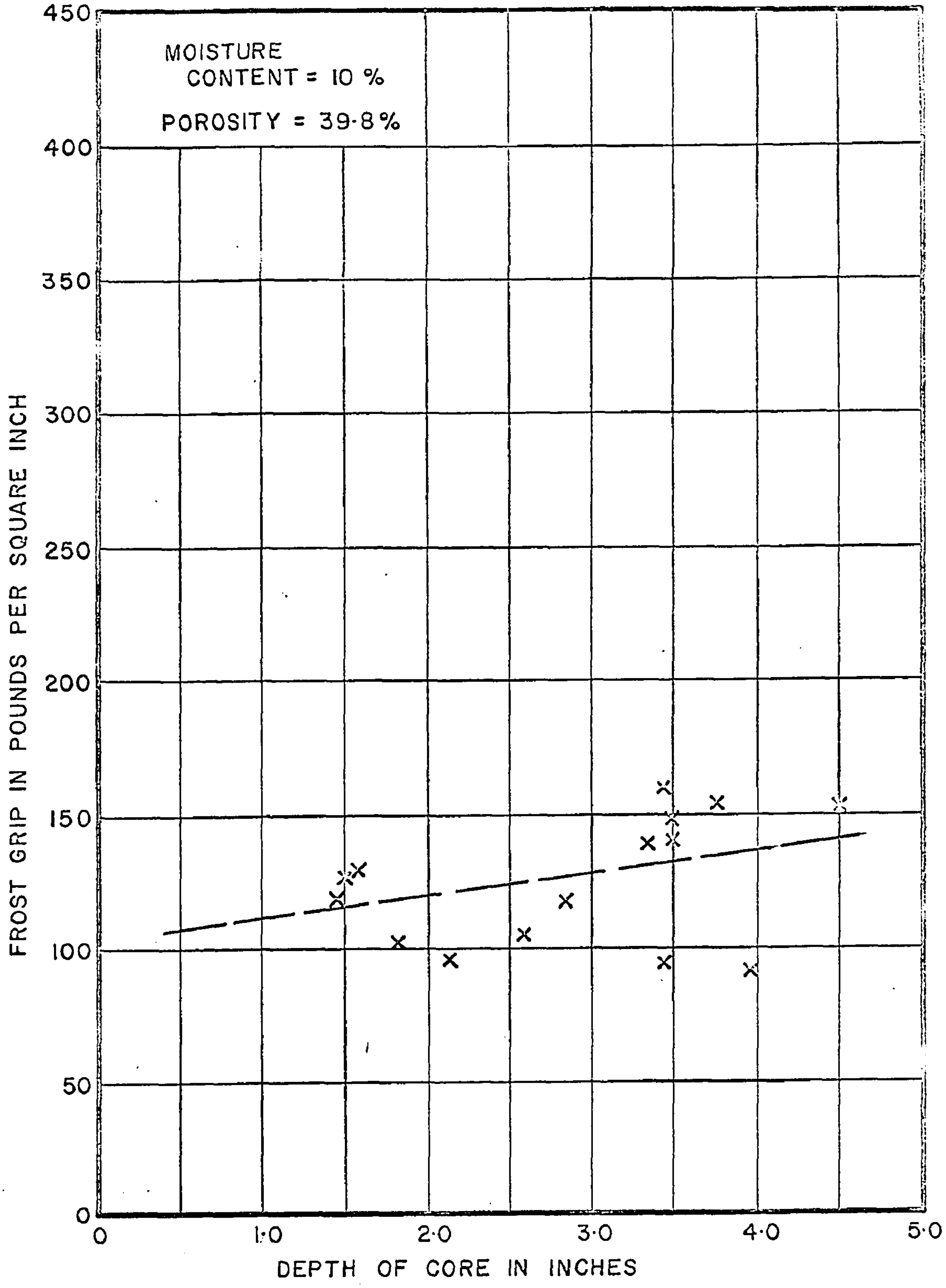


Fig. 16 Influence of Depth of Mold on Magnitude of Frost Grip Results.

possibility of this conclusion being erroneous was greater than 10%.

Fig. 17 shows the influence of clay and silt sized particles on the 'frost grip vs. porosity" relationship. This curve differs markedly from the pure granular soils in that the data do not suggest a straight line variation. Rather, there is a steep straight line portion at low porosities ($<$ 26%) where the soil is densely packed and then a gradual decrease at high porosities where the soil becomes, in effect, composed of large granules, due to the formation of soil "lumps", caused by the cohesiveness imparted by the clay particles.

The increased percentage of fines in Soil Mix No. 2 had practically no effect in the high porosity range, however, when the packing was dense the result was a decrease in frost grip from Soil Mix No. 1. This result conforms to the general pattern illustrated in Figure 18 where the results of Sand No. 1, Sand No. 2 and Soil Mix No. 1 are compared, all at the same moisture content of 10%.

We see that for a given porosity the resultant frost grip seems to depend on the amount of silt and clay sized particles in the soil. (This does not hold true at the

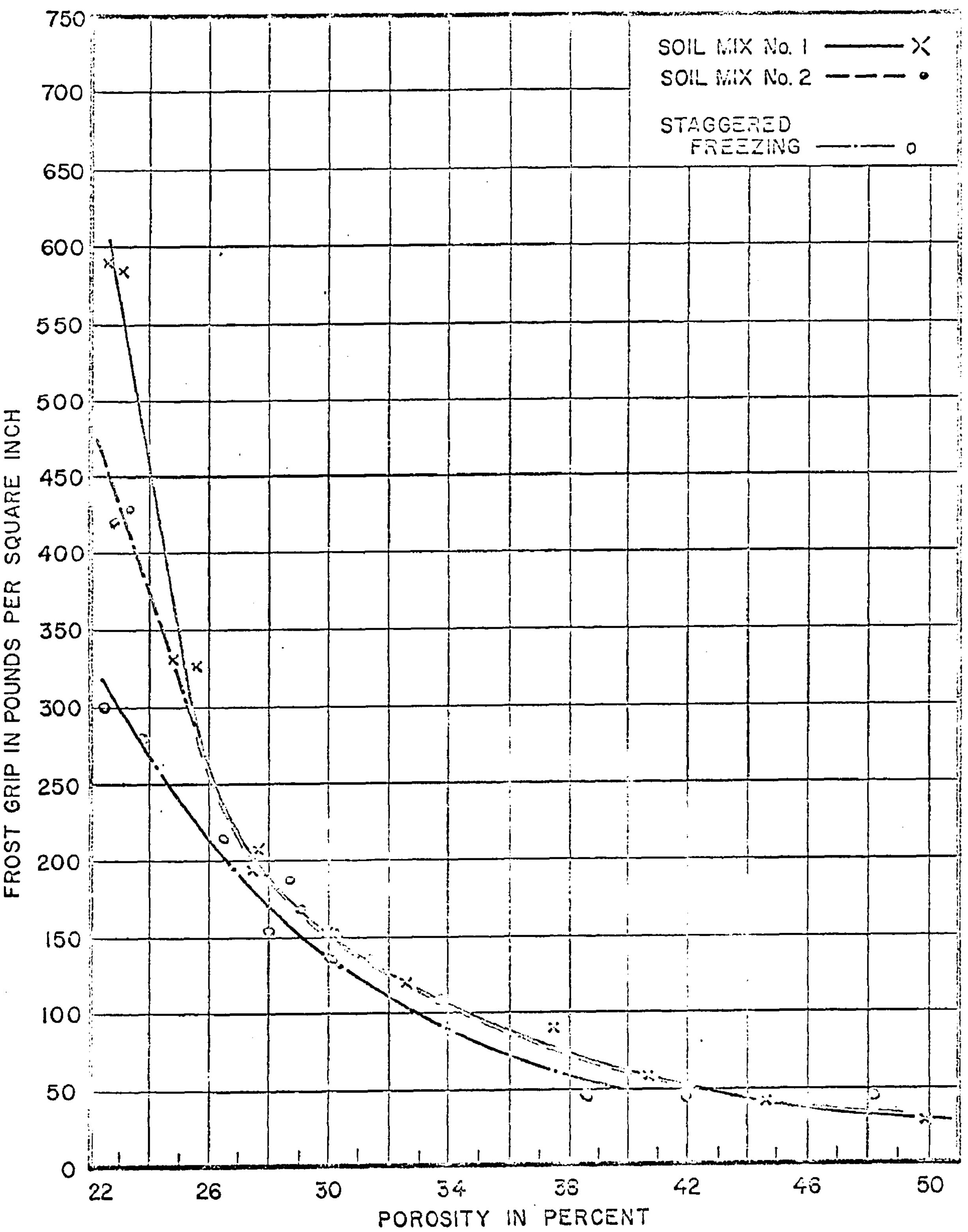


Fig. 17 Frost Grip vs. Porosity for Soil Mixes No. 1 and 2.

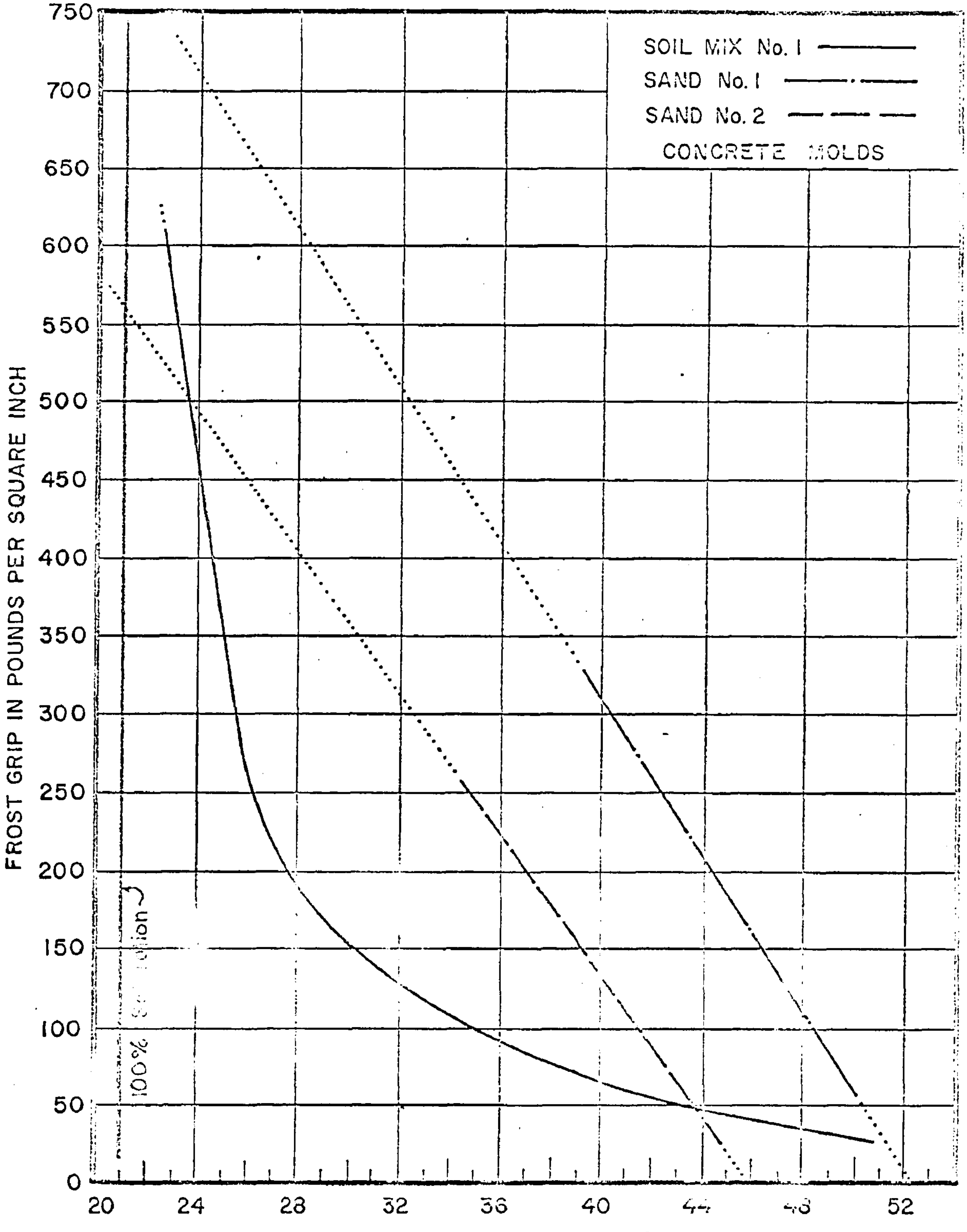


Fig. 18 Comparison of Results for Three Soils at Water Content of 10%

extremities of the soil mix curve.) The fines may act as a "lubricant" which lower the ability of the frost grip to withstand shear. This lessening in friction resistance is also evidenced when the soils are compared from the point of view of particle shape. The angular, manufactured, uniform Sand No. 1 gave higher results (for a given porosity) than the rounded, better graded, natural Sand No. 2. The soil mixes, containing relatively high percentages of clay platelets, gave lower results still (between $n = 23$ and 44%). It is interesting that, although more fines generally meant a lower frost grip, the maximum frost grip obtained during the investigation was with Soil Mix No. 1, giving a frost grip of almost 600 p.s.i.

Figure 17 also shows the results of a short series of tests conducted to determine the effect of gradually lowering the temperature by increments, instead of the standard procedure of freezing at a constant 0°F. Samples of Soil Mix No. 1 were prepared as usual but were placed in the freezer for 4 hours at 30°F, 4 hours at 20°F and 16 hours at 0°F. The temperature inside the soil core was checked using a thermocouple to ensure that at the time of testing the samples were indeed at

$0^{o}F$ and so the amount of unfrozen water content would be the same as for the standard samples.

It was found that these samples gave consistently lower results than were previously obtained, with the only difference being the rate of freezing. This may be explained by remembering that the moisture in the soil will migrate towards the freezing front. In this case the exposed top and bottom of the soil cores would begin freezing much sooner than the sides which were insulated by the thickness of concrete. Sufficient moisture would be drawn away from the sides before the freezing front reached them, to result in a noticeable drop in the frost grip.

Similar tests with the sands show no loss in gripping power. The sands, not being frost susceptible, would not support moisture migration.

CHAPTER V

CONCLUSIONS

The water content of a soil is one of the major factors influencing the resultant frost grip which can be exerted by that soil. The relationship is a linear one as demonstrated by Figures 8 and 9 and the graphs of Appendix V. For a water content of zero percent there is no frost grip and for water contents greater than zero the attendant frost grip is directly proportional to the water content, the slope of the straight line realtionship being determined by other factors such as porosity, particle shape and gradation or type of material to which the frozen soil adheres.

Soil porosity is the second major influence on frost grip for a given soil. As the porosity is increased, with all other factors maintained constant, there will be a corresponding decrease in frost grip (as shown by Figs. 10 and 11) which is again linear.

Particle shape and size also exert influence on the frost grip. An angular particle shape will give higher results than a rounded particle shape for a given porosity and water content as shown by Fig. 13.

The particle size distribution affects the frost grip in two ways. First, conducted tests indicate that under the same compactive effort a well graded soil will achieve a lower porosity than a uniformly sized material and hence will exert a greater frost grip since more frozen soil will be in contact with the structure. Secondly, a soil with relatively high percentages of silt and clay sized particles will have unfrozen water, with the percent of unfrozen water increasing as the percent less than two microns increases. Hence, less ice will form resulting in lower frost grip. This is shown in Fig. 17, most noticeably in the low porosity, high saturation range where Soil Mix No. 1 gives higher results than Soil Mix No. 2 by virtue of its having a lower percentage of these fine particles.

The surface texture of the material in contact with the frozen soil plays a part by providing roughness against which the frozen soil can key to resist shearing forces. This is shown by Figs. 8 and 9 where concrete gave the highest results followed by galvanized steel and then stainless steel.

The rate of freezing does not appear to affect purely granular soils (if the terminal temperature is constant). On the other hand, soils with silt and clay sized particles are affected. By gradually lowering the temperature to the freezing temperature frost susceptible soils will support moisture migration, as in nature, and the water will move from the warm sides to the initially freezing top and bottom of the sample. The drying of the sides means less ice will form and consequently the frost grip will be reduced. Subjecting the soil to lower temperatures will result in less unfrozen water content for soils containing silt and clay sized particles and thus will increase the grip. Theoretically, lower temperatures should result in higher frost grip even for complete crystallization of the soil water, since the shearing strength of ice increases with colder temperatures. Colder temperatures could also influence the laboratory test results depending on whether the coefficient of thermal expansion was greater for concrete or for the frozen soil.

Figures 14 and 15 summarize the results of the work with the granular materials. Using these two graphs curves may be constructed to show the idealized variation of frost grip with porosity for a given moisture content or the idealized variation of frost grip with moisture content for a given porosity. The latter may be accomplished by simply using the indicated value of frost grip on the locus of 100% saturation for the given porosity for one point and the origin for the second point. For the sake of comparison the values at 100% saturation from Figs. 8, 9 and 19 were plotted on Figs. 14 and 15 with 'M' 's. It can be seen that the differences between the experimental and theoretical values are relatively small, and the experimentally found lines, which were subject to experimental error, could be given a small adjustment to bring them into agreement with the theoretical values.

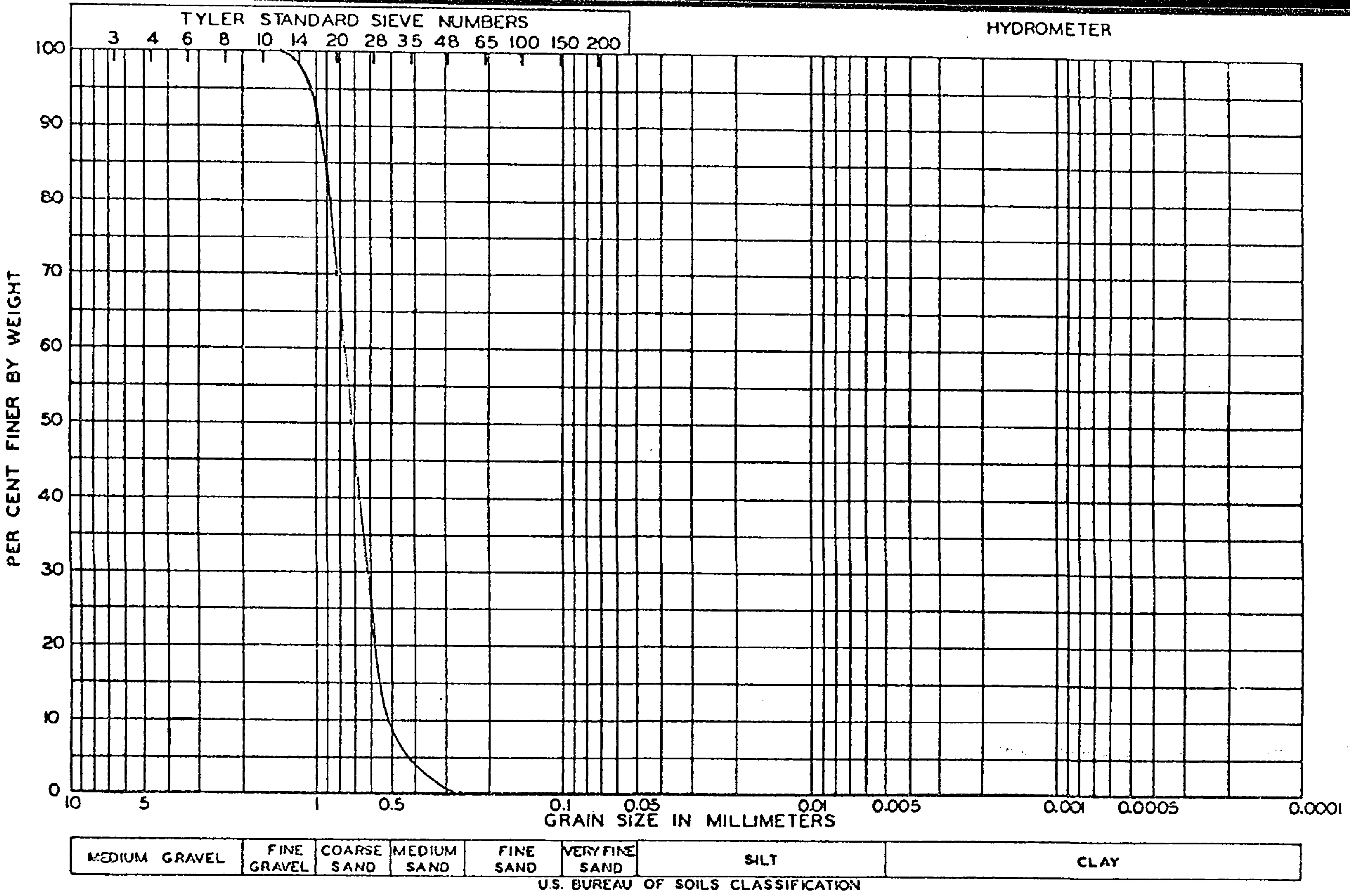
It might be noted here that, although frost grip is usually associated with frost heave and hence is undesireable, in practice, an artifically induced frost grip might prove to be very useful for some applications. If, for instance, after the construction of a light structure it was found that there was a high rate of differential settlement due to different rates of soil consolidation, the fast settling point may be temporarily supported by freezing the soil around it. During the temporary relief afforded by the grip of the frozen soil corrective measures such as underpinning or installing vertical sand drains may be carried out.

Finally, some recommendations as to the direction of effort in any future research in this field should be made in the light of experience gained during this work. Since the results of the "depth of mold" tests were not too conclusive it would be interesting to see if the frost grip would indeed be greater for deeper molds. Ideally, full scale model footings or walls would be tested to see if the results from the test molds corresponded to the grip in the field. A wider range of freezing temperatures might be used, resulting in different unfrozen water contents and, presumably, different values of frost grip for frost susceptible soils. Also, different soil types should be tested, sands with larger uniformity coefficients and soils with greater percentages of silt and clay, to verify the results for granular soils and establish master graphs for the silty and clayey soils. Ultimately, an equation would be developed consisting of soil parameters and conditions which would allow the engineer to predict the expected frost grip for any soil.

APPENDICES

APPENDIX I

GRAIN SIZE DISTRIBUTION CURVES



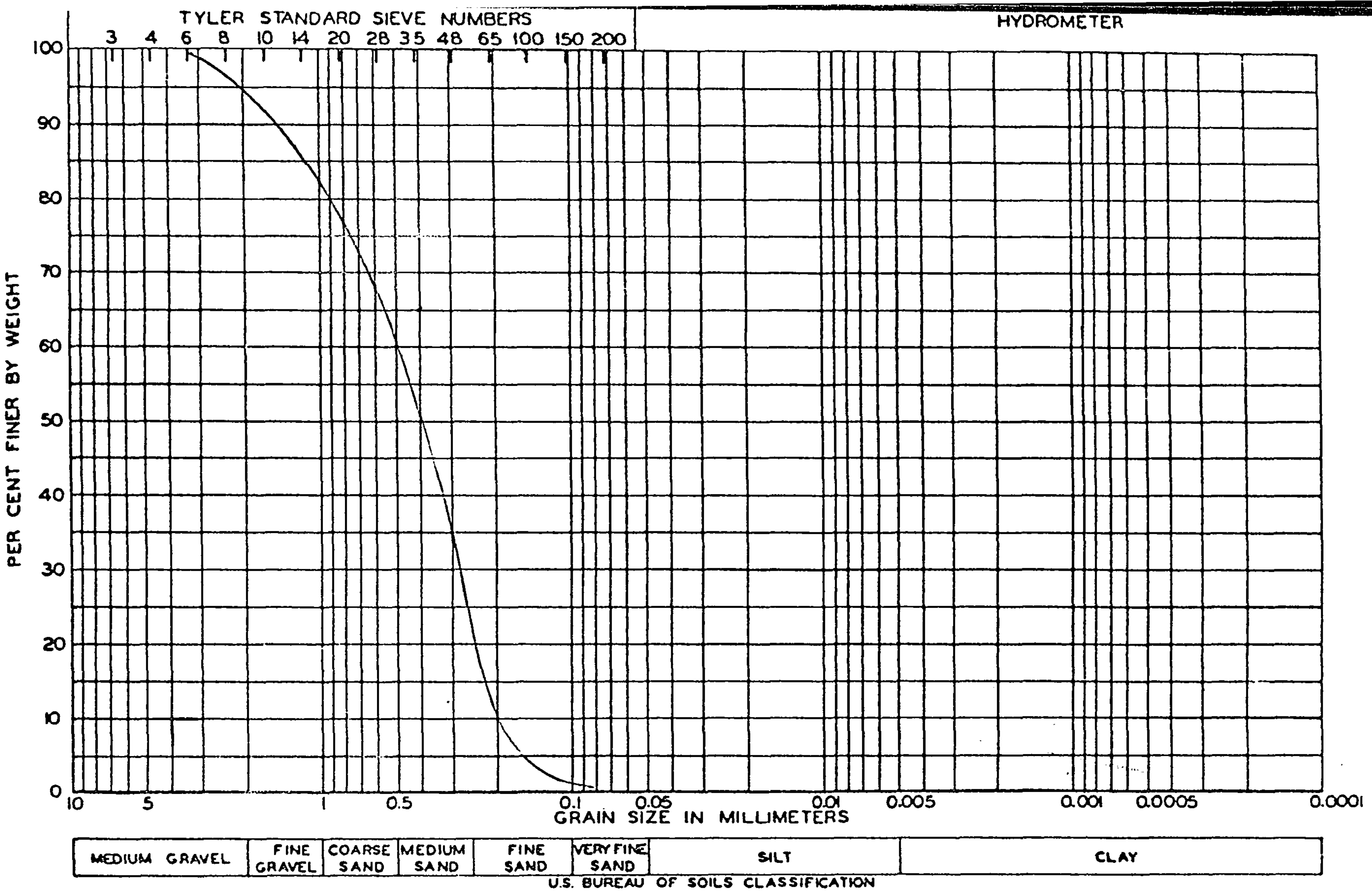
PROJECT Frost Grip Investigation BORING NO ________ SAMPLE NO. Sand No. 1

DEPTH ______ ELEVATION ______ REMARKS Uniformity Coefficient = 1.50

GRAIN SIZE DISTRIBUTION - DIAGRAM

DCL - 8

SOILTEST Incorporated 4711 W. North Ave. Chicago 39, Illinois, U. S. A.



PROJECT Frost Grip Investigation **BORING NO** ______ **SAMPLE NO.** Sand No. 2

DEPTH ______ **ELEVATION** ______ **REMARKS** Uniformity Coefficient = 2.53

GRAIN SIZE DISTRIBUTION - DIAGRAM

DCL - 8

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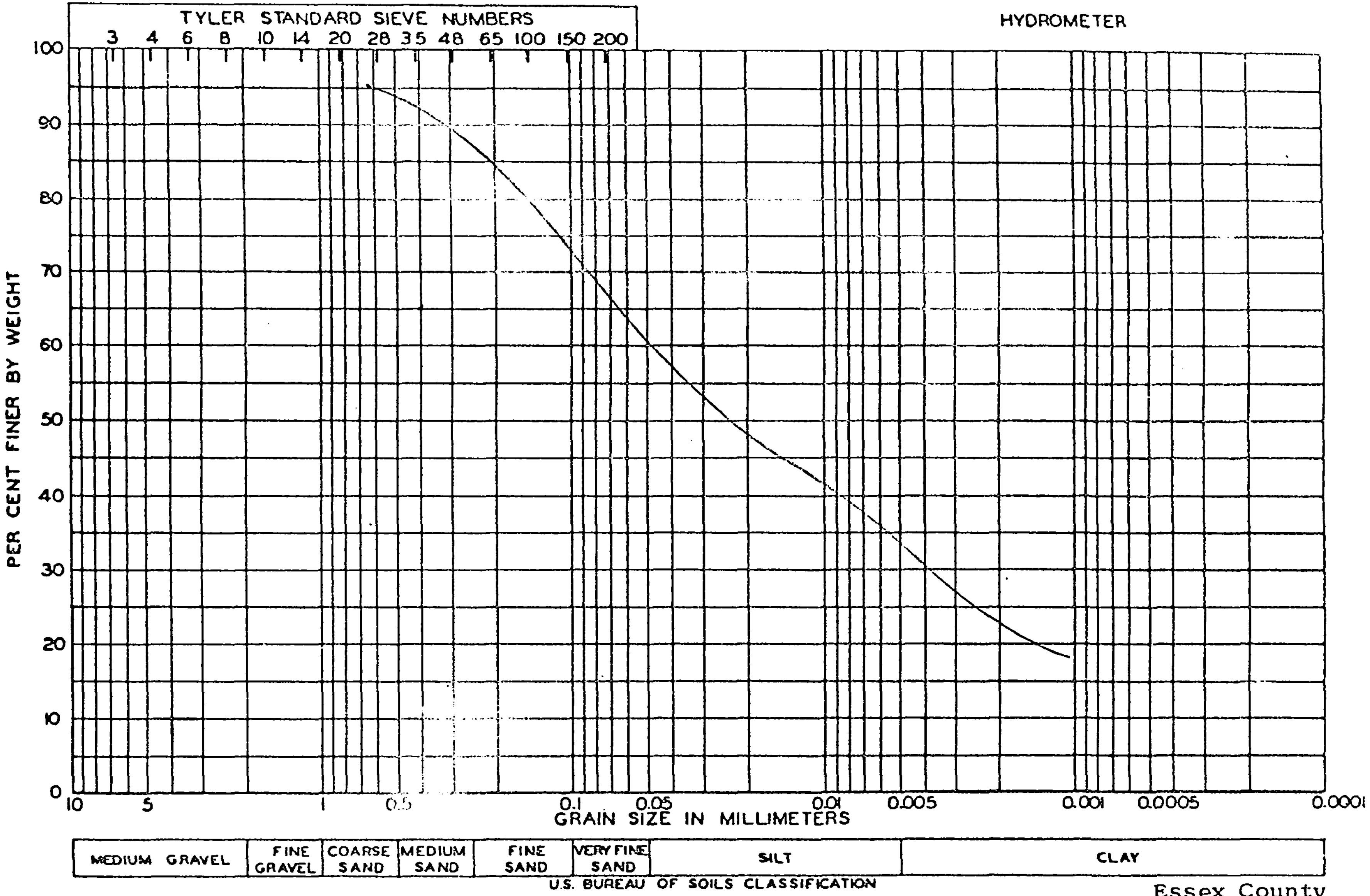

PROJECT Frost Grip Investigation **BORING NO** ______ **SAMPLE NO.** Essex County Pit Clay

DEPTH ______ **ELEVATION** ______ **REMARKS** U.S. Corps of Engineers Classification: Sandy Clay

Sand: 39%
Silt: 28%
Clay: 33%

GRAIN SIZE DISTRIBUTION - DIAGRAM

DCL - 8

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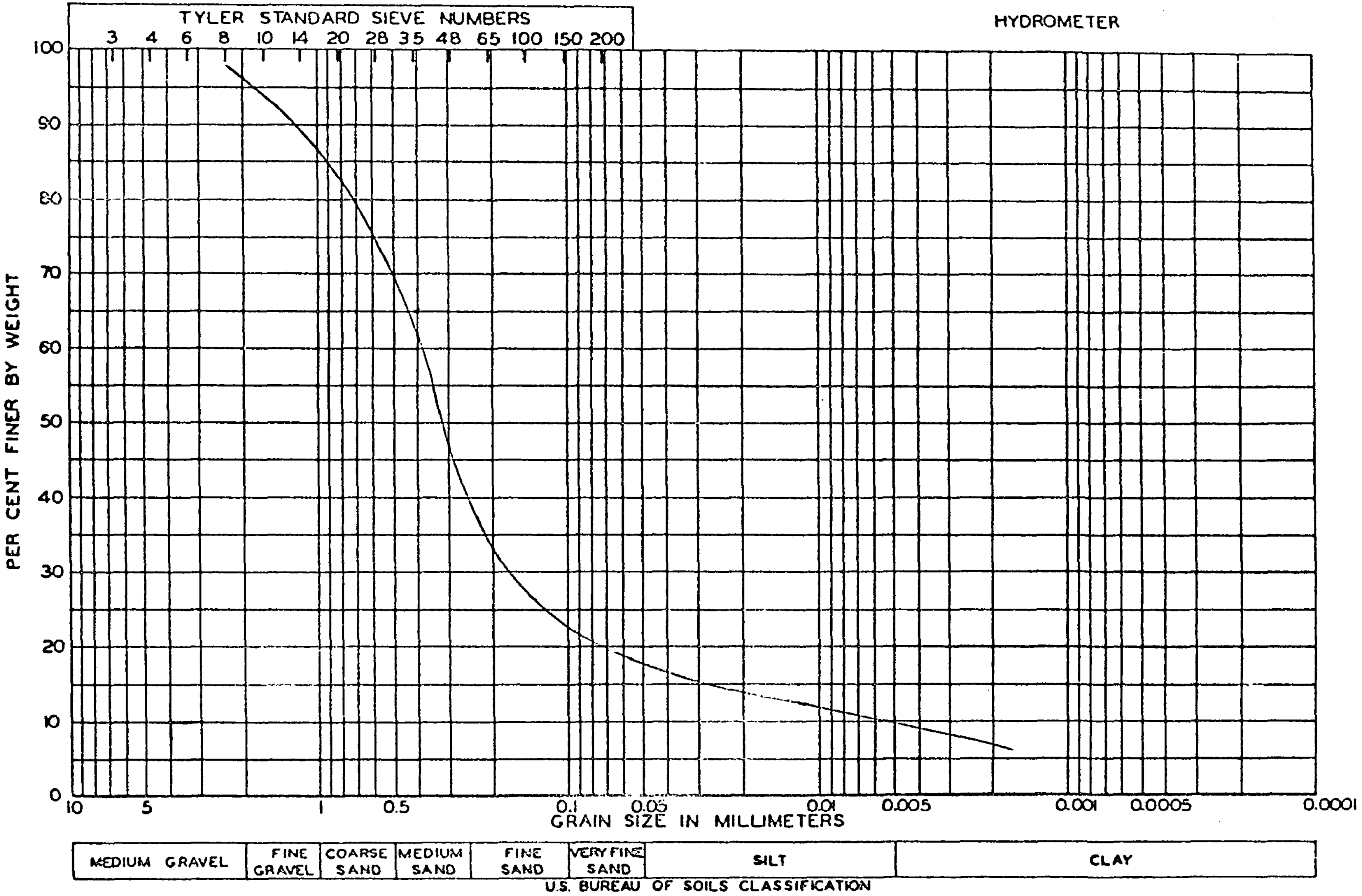

PROJECT Frost Grip Investigation BORING NO ________ SAMPLE NO. Soil Mix No. 1

DEPTH ______ ELEVATION ______ REMARKS U.S. Corps of Engineers Classification: Sand

GRAIN SIZE DISTRIBUTION - DIAGRAM

Sand: 82%
Silt: 8%
Clay: 10%

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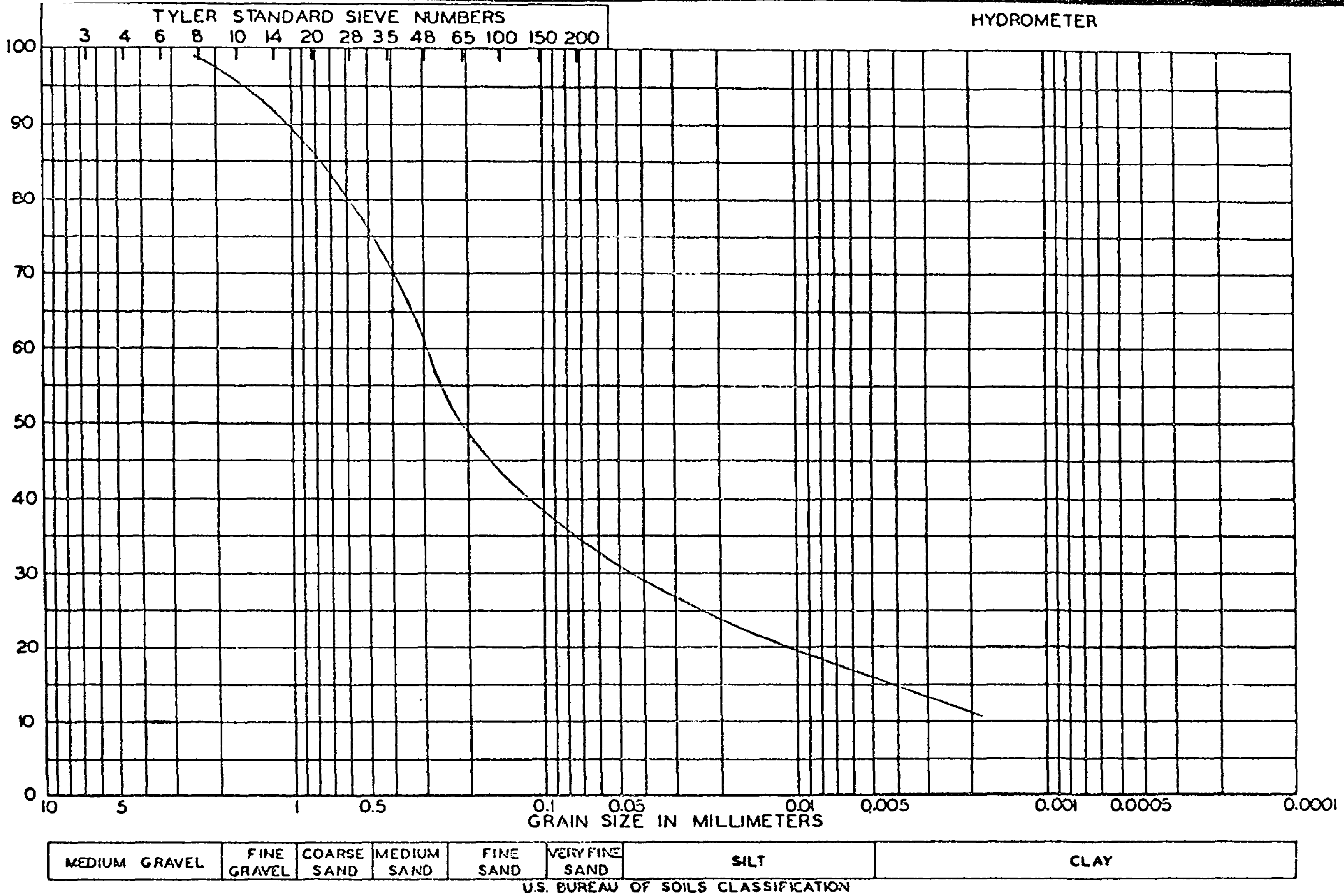

PROJECT Frost Grip Investigation BORING NO ________ SAMPLE NO. Soil Mix No. 2

DEPTH ______ ELEVATION ______ REMARKS U.S. Corps of Engineers Classification: Silty Sand

Sand: 70%
Silt: 14%
Clay: 16%

GRAIN SIZE DISTRIBUTION - DIAGRAM

DCL - 8

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APPENDIX II

SAMPLE CALCULATIONS

The method used for calculating the porosity, void ratio and degree of saturation was unchanging throughout the investigation. The sample calculations shown below are for Sand No. 2, Batch #36 for concrete mold #4.

Weight of wet soil in pan before packing = 3,203 gms.
Weight of wet soil in pan after packing = $\underline{1,221}$ gms.
Weight of wet soil packed into mold = 1,892 gms.

$$\text{Wet unit weight} = \frac{\text{wt. of wet soil}}{\text{vol. of mold}} = \frac{1,982 \text{ gm.}}{64.308 \text{ cu. in.}}$$

$$= 30.85 \frac{\text{gm.}}{\text{cu. in.}}$$

Converting to pounds per cu. ft.:

$$\text{Wet unit weight} = 30.85 \frac{\text{gm.}}{\text{cu. in.}} \times 1728 \frac{\text{cu. in.}}{\text{cu. ft.}} \times \frac{1}{454} \frac{\text{lbs.}}{\text{gm.}}$$

$$= 117.4 \text{ p.c.f.}$$

$$\text{Dry unit weight} = \frac{\text{wet unit weight}}{1 + \text{water content}}$$

Since W.C. = 14.73% for this batch

$$\text{Dry unit weight} = \frac{117.4}{1.1473} = 102.3 \text{ p.c.f.}$$

Porosity: $n = 1 - \frac{W}{VG\gamma}$

where n = porosity in decimal form

w = dry weight of soil

V = volume of soil

G = specific gravity of solids = 2.65

γ = unit weight of water = 62.5 p.c.f.

Considering a one cubic foot volume of soil; V = 1 cu. ft., and w = 102.3 pounds

$$\therefore n = 1 - \frac{102.3}{1 \times 2.65 \times 62.5} = 1 - 0.618 = 0.382 \text{ or } 38.2\%$$

Void Ratio: $e = \frac{n}{1-n}$

$$\therefore e = \frac{0.382}{1 - 0.382} = \frac{0.382}{0.618} = 0.618$$

Degree of Saturation: $S = \frac{WG}{e}$

where S = degree of saturation in percent

and W = water content in percent

$$\therefore S = \frac{14.73 \times 2.65}{0.618} = 63.2\%$$

APPENDIX III

CALCULATION OF UNFROZEN WATER CONTENT

Using the equation of Dillon and Andersland, (Equation 2-2 of Chapter II) The unfrozen water content of the soil mixes containing particles greater than 2 microns can be estimated.

For Soil Mix No. 2:

$T = 0^{o}F = -17.8^{o}C = 255.4^{o}K$

$T_o = 273.2^{o}K$ (since freezing point depression is unknown)

$$A_c = \frac{I_p}{\% < 2\mu} = \frac{2.3}{11.0} = 0.209$$

$L = 1$

$k = 2.8 \times 10^{-4}$ gm. of water/M^2

The average specific surface area "S" of the soil particle may be calculated using:

$$S = \frac{S_1 \frac{P_1}{d_1} + S_2 \frac{P_2}{d_2} + S_3 \frac{P_3}{d_3}}{\frac{P_1}{d_1} + \frac{P_2}{d_2} + \frac{P_3}{d_3}}$$ ...(Appendix of Ref. 14)

$$\therefore S = \frac{0.05 \times \frac{66}{1.0} + 1.0 \times \frac{23}{0.01} + 290 \times \frac{11}{0.0005}}{\frac{66}{1.0} + \frac{23}{0.01} + \frac{11}{0.0005}}$$

$$= \frac{3.3 + 2300 + 6{,}390{,}000}{66 + 2300 + 22{,}000} = 263 \text{ M}^2/\text{gm.}$$

$$\therefore W_u = \frac{262 \times 255.4}{273.2} \times \frac{1}{0.209} \times 2.8 \times 10^{-4} \times 100$$

$$= 3.28\%$$

<u>For Soil Mix No. 1.</u>, the term $\frac{L}{A_c}$ is taken as unity since it was not plastic enough for a plastic limit determination.

$$S = \frac{0.05 \times \frac{79}{1.0} + 1.0 \times \frac{14}{0.01} + 290 \times \frac{7}{0.0005}}{\frac{79}{1.0} + \frac{14}{0.01} + \frac{7}{0.0005}}$$

$$= \frac{4 + 1400 + 4{,}060{,}000}{79 + 1400 + 14{,}000} = 262 \text{ M}^2/\text{gm.}$$

$$W_u = \frac{262 \times 255.4}{273.2} \times 1 \times 2.8 \times 10^{-4} \times 100$$

$$= 0.69\%$$

APPENDIX IV

STATISTICAL INVESTIGATION OF DEPTH OF MOLD TESTS

(1) Calculation of Line of Best Fit

DEPTH OF MOLD x	FROST GRIP	$(x - \bar{x})$	$(y - \bar{y})$
2.156	95.01	-0.725	-31.40
3.313	140.70	0.432	14.29
3.406	163.72	0.525	37.31
3.438	147.58	0.557	21.17
1.781	103.46	-1.100	-22.95
1.438	123.52	-1.443	- 2.89
2.844	120.60	-0.037	- 5.81
2.625	107.73	-0.256	-18.68
3.469	143.52	0.588	17.11
3.438	93.57	0.557	-32.84
3.750	153.54	0.869	27.13
3.938	89.56	1.057	-36.85
1.625	130.85	-1.256	4.44
1.500	128.74	-1.381	2.33
4.500	154.00	1.619	27.59

$\leq x = 43.221 \qquad \leq y = 1{,}896.10$

$$\bar{x} = \frac{\leq x}{n} = \frac{43.221}{15} = 2.881$$

$$\bar{y} = \frac{\leq y}{n} = \frac{1896.1}{15} = 126.41$$

$\leq x^2 = 137.828525$

$\leq xy = 5{,}570.42076$

$$a = \frac{\Sigma x^2 \Sigma y - \Sigma x \Sigma xy}{n \Sigma x^2 - (\Sigma x)^2}$$

$$= \frac{137.828525 \times 1896.1 - 43.221 \times 5{,}570.42076}{15 \times 137.828525 - (43.221)^2}$$

$$= \frac{261{,}336.6568 - 240{,}759.1574}{2{,}067.428 - 1{,}868.055}$$

$$= \frac{20{,}577.4994}{199.373} = 103.21$$

$$b = \frac{n \Sigma xy - \Sigma x \Sigma y}{n \Sigma x^2 - (\Sigma x)^2}$$

$$= \frac{15 \times 5{,}570.42076 - 43.221 \times 1{,}896.1}{15 \times 137.828525 - (43.221)^2}$$

$$= \frac{83{,}556.312 - 81{,}951.338}{2{,}067.428 - 1{,}868.055}$$

$$= \frac{1{,}604.974}{199.373} = 8.050107$$

$$y = a + bx$$

$$\therefore\ y = 103.21 + 8.0501x$$

(2) Testing Significance of Slope

We may test whether the slope differs significantly from zero by applying a "t" test:

$$t = \frac{b}{S_b}$$ ...Eq'n. (15 - 32), Ref. (16)

where b = experimental slope

and S_b = standard deviation of "b"

also $$S_b = \frac{S_y}{\sqrt{\sum (x - \bar{x})^2}}$$ ...Eq'n. (15 - 24), Ref. (16)

and $$(n-2)Sy^2 = \sum (y - \bar{y})^2 - \frac{\left[\sum (x - \bar{x})(y - \bar{y})\right]^2}{\sum (x - \bar{x})^2}$$

...Appendix E, Ref. (16)

$$\sum (y - \bar{y})^2 = 8,199.6499$$

$$\sum (x - \bar{x})^2 = 13.291538$$

$$\sum (x - \bar{x})(y - \bar{y}) = 106.9982$$

$$\therefore (15 - 2)\ Sy^2 = 8,199.6499 - \frac{(106.9982)^2}{13.291538}$$

$$13\ Sy^2 = 8,199.6499 - \frac{11,448.6214}{13.291538}$$

$$= 8,199.6499 - 861.3465$$

$$\therefore Sy^2 = \frac{7338.3034}{13} = 564.48487$$

$$\therefore Sy = 23.757$$

$$\therefore S_b = \frac{23.757}{\sqrt{13.291538}} = \frac{23.757}{3.646} = 6.5159$$

$$\therefore t = \frac{b}{S_b} = \frac{8.0501}{6.5159} = 1.235$$

From Table 8, Ref. (16) for $\nu = (n - 2) = (15 - 2) = 13$

$$t = 1.771 \quad @ \quad \alpha = 0.10$$

Since our calculated "t" is less than 1.77 we cannot conclude that there is a significant difference.

The probability of our being wrong is greater than 10%.

APPENDIX V

MISCELLANEOUS GRAPHS

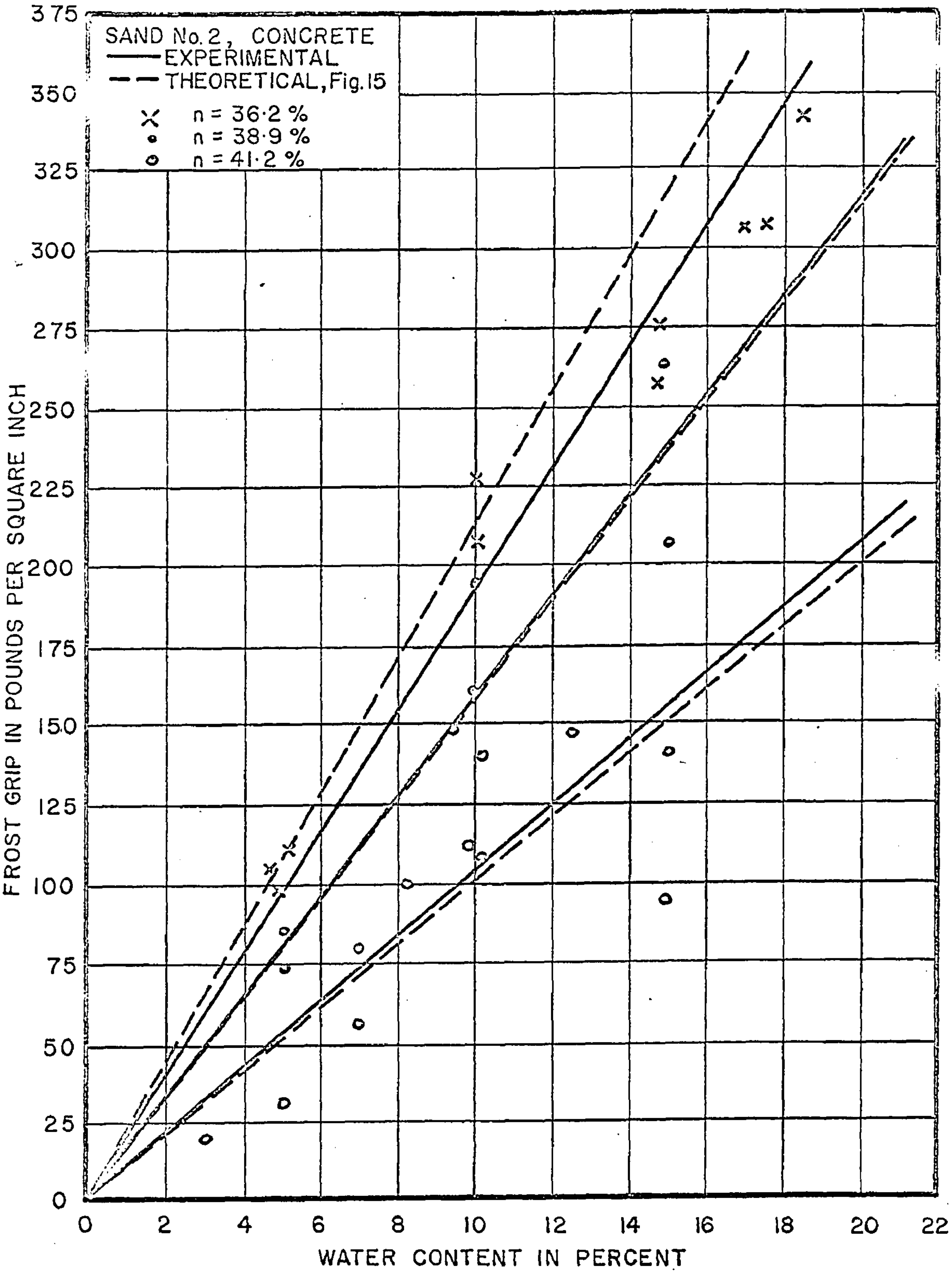

Fig. 19 Plot of Experimental Data for "Frost Grip vs. Water Content", for Various Porosities

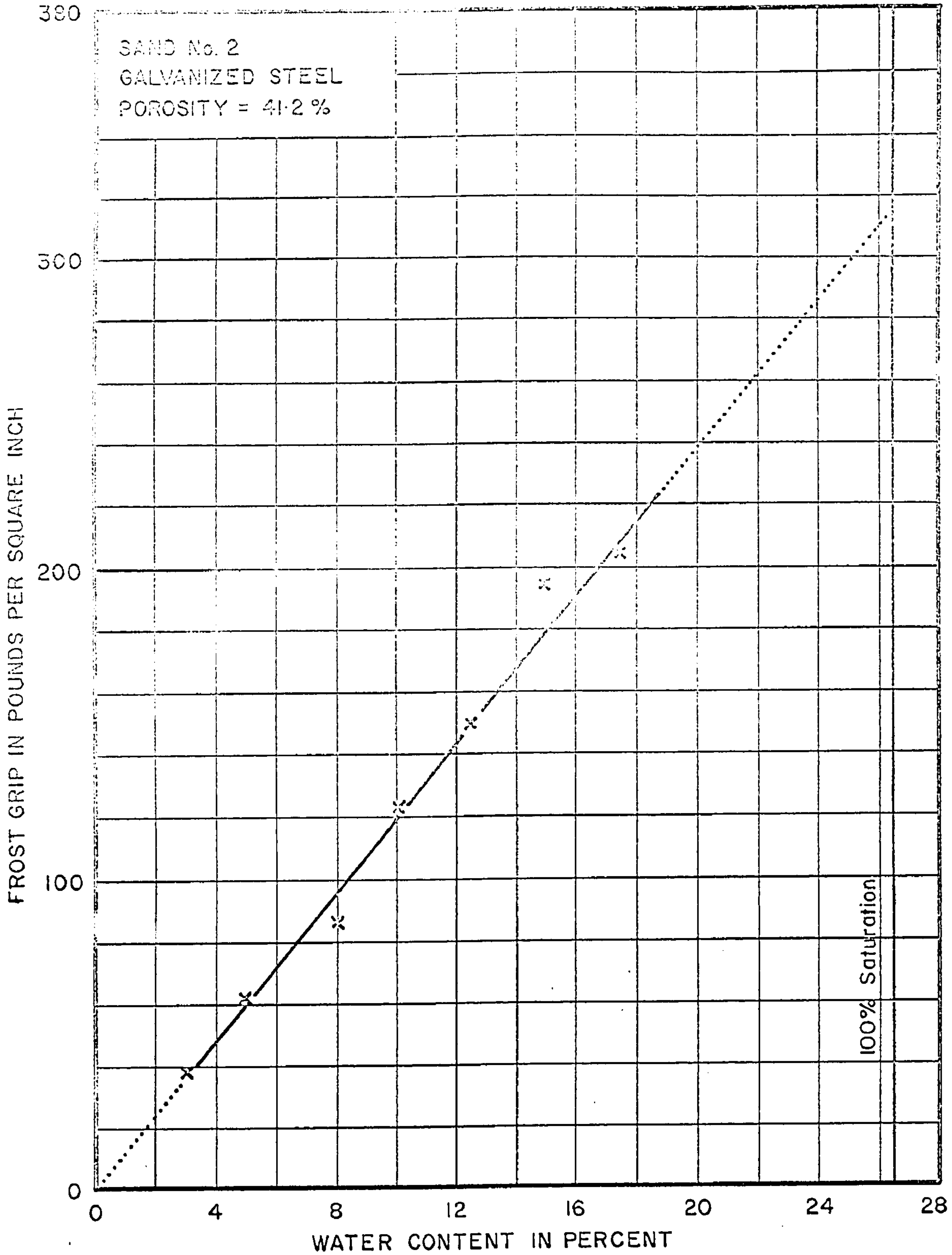

Fig. 20 Experimental Frost Grip vs. Water Content for Sand No. 2 and Galvanized Steel

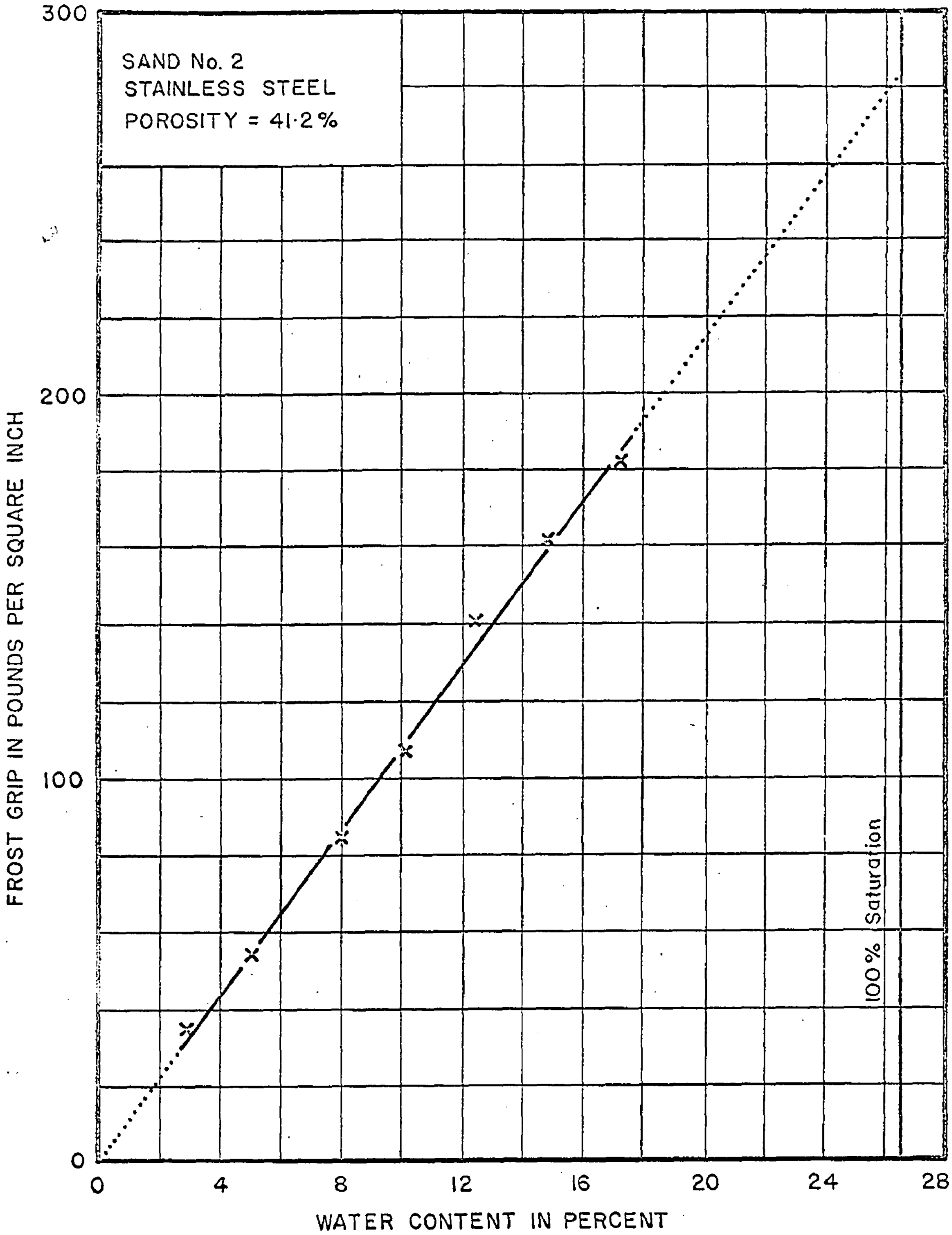

Fig. 21 Experimental Frost Grip vs. Water Content for Sand No. 2 and Stainless Steel

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