# SUBBASE DESIGN CRITERIA FOR SECONDARY ROADS

by

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# SUB-BASE DESIGN CRITERIA FOR SECONDARY ROADS

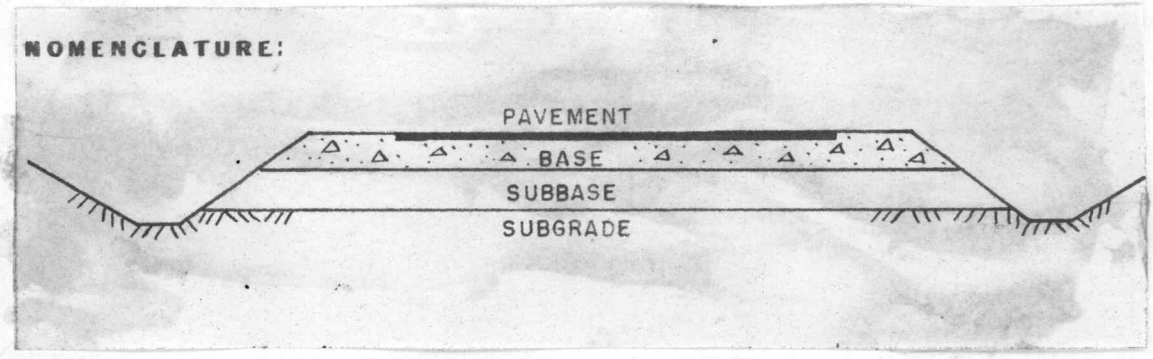
## INTRODUCTION

It seems somewhat out-of-place to present a thesis on Highway Soils, on which so much has been written, by men with decades of experience and study behind them. Yet it is my opinion that highway base problems are far from solved for all of the work that has been done, nor can it be said that highway literature is not useful at this time when highway and road building construction is occupying so much of United States labor and capital.

The intent of this work is to review Highway Soils design methods with the objective of determining a method that county and local engineers would find convenient to use. For even though literature is available, if it is not used, it is of little value. More specifically, a method of design is sought for use on those secondary roads where traffic is such as to require a pavement type surface.

Engineers who have studied the subject of roadway stability are aware that the high moisture season of the year is the critical time for road and highway failures. Also these same people must have concluded that the failures are a result of high moisture content in the subbase or subgrade. A considerable amount of work has been done toward determining the loss in strength. It is established

that the stability of roadways in the spring months is often less than 50 percent (14, p.28) of the fall season stability. It is apparent then, that the primary cause of roadway failures is in subbase deficiencies.




We could, perhaps, learn something by reading about the road builders of the Roman Empire, (5, p.106) where an arbitrary design was adopted; a design of such strength that some of these roads are still in use today.

When funds and technical knowledge are available, roads and highway development often include an adequate subbase and other basic requirements. It is when funds are very limited that the problem becomes acute of how to best use these funds. Mr. A. W. Campbell (5, p.152) Commissioner of Roads for Canada, once said: "The public may no doubt readily see that for every elaborate road

system the services of engineers are necessary; but they have yet to learn that whenever the possible funds are limited, there is all the more need for conserving to the utmost the energy available, and that for this reason skill and experience should be employed by every municipality."

It is futile to speculate on the possibility of having specialists available who are capable of conducting extensive soil surveys and evaluating each proposed road section, yet the cost of ignoring good design is such that many road officials are concerned.

A survey was conducted in the State of Oregon to determine what practice is now followed by county Engineers to evaluate subbase requirements. The replies are summarized as follows:

1. All reports indicated that local experience was the predominant guide, ie design by precedent.
2. Ten percent indicated that the Group Index served to assist in design.
3. About twenty percent used an estimated R factor (triaxial) with an occasional laboratory triaxial sample tested for the more important projects.

If, by observation, we could conclude that this was sufficient to produce roads that are adequate and economical, we would need to go no further. Yet, throughout the State, as elsewhere, we find innumerable instances of faulty road construction. A few of which are illustrated on pages 6 and 7 .

## INTRODUCTION TO DESIGN

Inadequate design can be eliminated, to a large degree but never completely because: 1. The element of changing traffic patterns or loads are intangible. 2. The growth of communities, and the resulting traffic density, often defies rational analysis.

With these thoughts in mind, the history of a particular road development is apt to be a stage construction process. Where conditions are known or can be forecast, the design process is abbreviated to that of selecting an adequate road design for known conditions.

A road section which will require complete repaving within the next few years. Until such time it will require extensive maintenance.

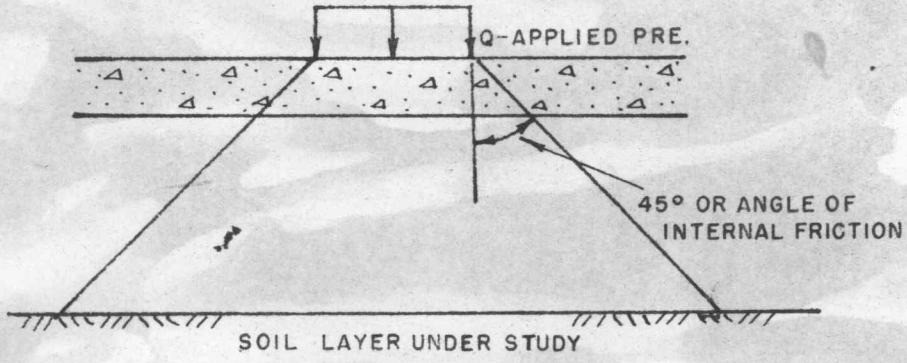
Surface appearance of subbase failure.

On any community project the intent of the project is to serve as much of the population as is possible with minimum cost to that population. There are, generally, minority groups who have special requirements that the project cannot satisfy. This same idea holds true for subbase design. If a maximum load is used for design purposes, there are those instances where this maximum will be exceeded. Yet, if the design is increased to include these heavier loads, part of the additional cost will be borne by those who have little use for the heavier road base. Therefore, the resultant maximum load (used for design) is a compromise and this compromise can only be resolved by legislation.

The American Association of State Highway Officials recommends a wheel load of 8000 pounds (10, p.305) for rural roads. A glance at AASHO bridge specifications (1, p.160) indicates that this loading is not excessive, as a matter of interest, the H-15-44 loading for highway bridges produce a concentrated load of 12,000 pounds. It would seem that the design load for road foundations should be equal to this. However, the stresses produced in a road foundation presents a different situation than that produced in bridge superstructures. In road foundations loads a short distance apart have no combined effect to increase the unit soil pressures. Spangler

(24, p.216) proved theoretically that dual wheels ten inches apart, each with a load of 4000 pounds, were no more severe on a four inch flexible pavement than a single wheel load of 4000 pounds and that a single load of 8000 pounds was considerably more severe. However, for heavy loads and thick subbases, a combination of loads closely spaced will produce higher combined stresses.

The distribution of soil pressures is a difficult problem to analyze, and no purely theoretical method exists today that will satisfactorily solve practical problems. To overcome these difficulties, many early designers made certain assumptions as to the stress distribution such as shown in the following sketch.

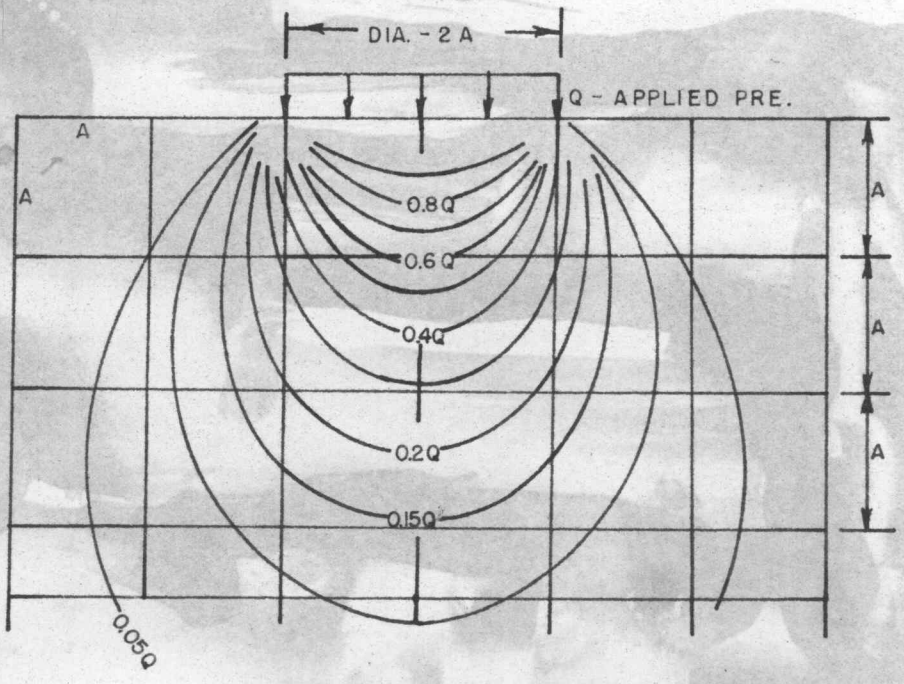



In the distribution shown, the unit stress was assumed to be evenly distributed at the level of the soil layer in question.

These assumptions are not now generally considered correct and other methods have replaced them which are accepted as being closer to the truth.

Boussinesq's theory of pressure distribution is widely used for soils and foundation work. Within the limits of its assumptions, it is mathematically correct. From the equations first developed by Boussinesq,

Jurgensen (17, p.206-241) has developed several pressure diagrams, the most applicable one is presented below:




BULB OF PRESSURE UNDER A UNIFORM CIRCULAR LOAD

In the above diagram the pressure lines shown are a trace of points of equal pressure in terms of the applied pressure.

The basic assumptions of the above method is such that it is correct for a uniform soil that behaves with a constant stress strain relationship. Quite obviously, this is not exactly true, since multiple layers of various soil types are generally encountered in practice.

In addition to these geometrical and analytical procedures, several agencies have approached the subject

through laboratory analysis, wherein they have attempted to actually measure the soil pressures (8, p.101-115). The problem is not a simple one. From the selection of elementary pressure gauges through the analysis of test results, it requires a considerable amount of effort and the ability to be satisfied with modest results.

All fine grained soils increase in supporting ability as the moisture content decreases. The tendency among design engineers is to use the soil stability that exists when that soil is in a compacted and saturated condition. (18, p.497-512) It seems apparent that this is a logical beginning point for soil stability evaluation (neglecting frost action) since (a) compaction can be obtained artificially and is often specified (b) It is difficult and often impossible (economically) to insure a condition less than saturated for the life of the roadway. (18, p.497-512).

Much has been said and written about drainage and while drainage is necessary, the author has concluded that if a roadway is dependent on surface drainage for stability, then at least part of that roadway will eventually fail. There are two sources of water available to the road foundation, that water that percolates into the foundation from the surface and that which is drawn from a lower elevation by capillary action. When a roadway is paved, much of the surface water is eliminated from the road structure. Perhaps this leads many to a false sense of security when thinking of subbase moisture, for capillarity in fine grained soils is a water carrier of real importance (21, p.91-92 and 28, 1-33). An illustration of the ability of capillary action is given

on the attached page 18.

In those areas where frost penetrates the subgrade soils the combination of capillary water and frost combine to produce heaving and loss of stability during the thaw period (15, p.139 and 146-157) as outlined in the following paragraphs.

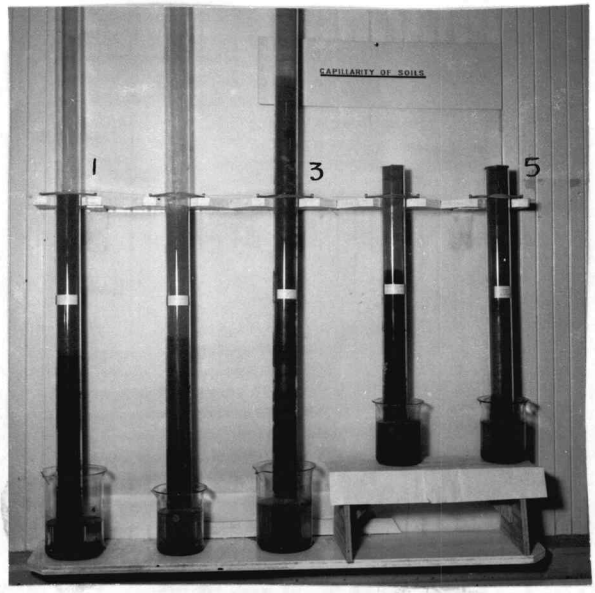
Since frost action produces a change both in moisture content and in the density of the soil mass, it can be said to be a major item in determining the stability of the soil mass. The formation of ice crystals and ice lenses in a soil mass, which subsequently produces an increase in the moisture content during the period of thaw is a product of three factors which must always be present simultaneously. These are (1) freezing temperature, (2) a supply of water readily available to the soil mass and (3) the presence of soil of a certain grain size. With respect to item (3) authorities (25, p.131) state that ice lenses develop only in fine grained soils. In perfectly uniform soil (soil grains all of one size) they state that ice layers do not form unless the grains are smaller than 0.02mm. Fairly uniform soils must contain at least 10 percent of grains smaller than 0.02mm, while in mixed-grain soils ice layers may form if as little as 3 percent of the soil particles in the mass are smaller than 0.02mm. (13, p.170-171) In soils with less than 1 percent of the grains smaller than 0.02mm, they state that ice layers are not formed under any condition found in the field. For convenience in determining grain size many agencies prefer to use the No. 200 mesh sieve which will pass soil particles up to 0.074mm in size. See sheet 43 for a typical correlation study.

If the size of the ice crystals and layers in the soil mass are considerable (representing water in excess of the plastic limit) sufficient water released during the frost melting period to cause a direct loss in the shearing resistance of the soil. If however, the amount is nominal and the soil mass has experienced but a slight drop in density, the shearing resistance of the material may remain high until consolidation occurs, which may produce a reduction in the stability value as described in the following paragraph.

Although the effect of frost may in itself weaken a subgrade by merely releasing an excessive amount of water into the mass, the reduction may be accentuated by the effect of compaction under traffic. This is explained briefly as follows: Due to frost action the soil may gain in moisture from very finely dispersed ice crystals distributed throughout the soil mass to the extent that its density is slightly reduced. If the gain in moisture and the loss in density is moderate, the soil structure can remain substantially intact and exhibit no loss in shear strength. It should be assumed that the soil in this condition is, for practical purposes, completely saturated. Since the density is less than that normally maintained by traffic, compaction tends to reconsolidate the soil component in question. This compaction breaks

down the existing soil structure as the particles attempt to move closer together. Because of the saturated condition of the soil, no volume change can take place without drainage. If drainage can not respond instantly to load applications, the load is transmitted through the pore water. Because water is without shear strength, the stability of the soil layer is reduced with this increase in the pore water pressure. An extreme case of this phenomenon is the quick livery appearance some times developed when a saturated silt or fine sand is puddled lightly under foot.

The photograph below illustrates the water carrying capacity of soils. The soil was inserted in an air dry condition after which water was put in the beaker at the bottom of each tube. Therefore, capillarity was the sole factor influencing water movement.




Results of the experiment were as follows:

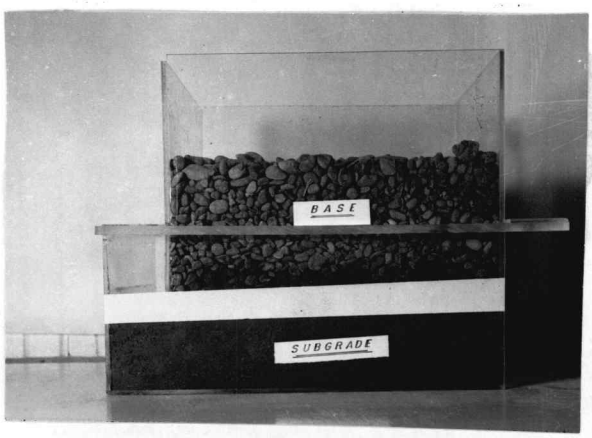
| Tube no. | Soil Type | Percent 200 | Total rise inches | Final percent moisture in top of tube |
|---|---|---|---|---|
| 1 | A-1-b | 7 | 16 | -- |
| 2 | A-4(2) | 44 | 18 | -- |
| 3 | A-6(9) | 94 | 55 (top tube) | 29.4 |
| 4 | A-6(9) | 95 | 27 (top tube) | 32.6 |
| 5 | A-7-5(20) | 99 | 27 (top tube) | 38.1 (slow rate of rise) |

A satisfactory select granular borrow material when placed over a silty subgrade may give excellent results with no distress evident for a period of years. Subsequently failure may develop. (Author's experience) Upon examination the cause of the distress may be found to be the infiltration of fine grained soil from the silt subgrade into the coarse granular material. In such cases the amount of soil fines can accumulate until they are detrimental, allowing the formation of ice crystals, retarding drainage, and consequently lowering the stability of the select borrow or base with failure resulting. The intrusion of subgrade soil into the subbase can occur when the subbase material is composed of very coarsely graded aggregate and the subgrade is composed of very fine grained silt or sand. Experiments have shown that the fine soil will not be washed into the coarse aggregate if the 15 percent size of the coarse aggregate is less than about 5 times as large as the 85 percent size of the fine grained soil. In this relationship the size of the material is expressed in millimeters. For example, assume that 15 percent of the aggregate for select borrow passes the No. 40 sieve and 85 percent of the subgrade soil passes a No. 200 mesh sieve. By reference to a grain size chart upon which the sieve numbers and their equivalent openings in mm are shown, it can be seen that a No.40

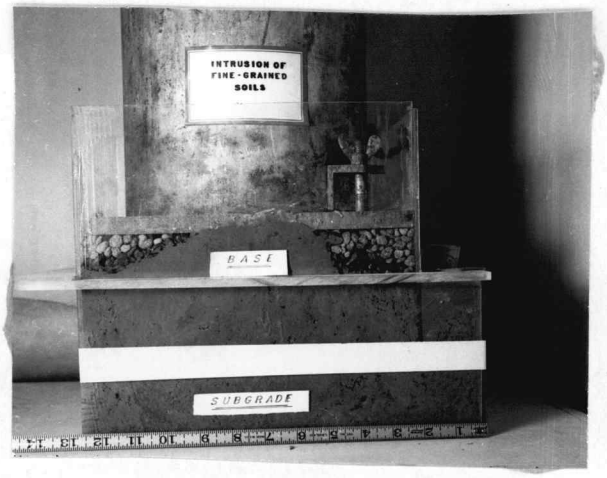
screen is equal to about 0.4 mm and a No. 200 sieve is equal to 0.074 mm. The quotient of 0.4/0.074= 5.4 and the example would be considered to be border line. As a rough rule therefore, a silt will enter a select borrow if (1) more than about 85 percent of the silt will pass a No. 200 sieve and (2) less than about 15 percent of the select borrow will pass a No. 40 sieve. An example of silt intrusion is shown on page 21.

From a practical standpoint during construction the finest grained portions of the select borrow pit should be utilized in making the first lifts over silty subgrades. In special cases the plans should provide for the placement of such a protective layer.

The above does serve to illustrate that the frost action problem is essentially one of excess concentrated moisture which results in supersaturated soil. Therefore, it is fundamental that a subbase depth adequate for 'normal' conditions, is not necessarily adequate where frost is a design factor.




The above photograph shows an A-4(8) subgrade overlaid with 6 inches of open graded gravel prior to loading.




Photograph number two shows the intrusion of the silt into gravel. (a) The moisture content of the silt was well above the liquid limit. (b) The load was about 5 psi.

Authorities well known in the field of soil mechanics have said that soils will never be an exact science (19, p. 438), but that it must depend on empirical tests and experience for development. With this idea in mind, it is obvious that the experience and testing knowledge of many men and organizations will produce advancement more quickly, yet to exchange ideas and information, a method of identification of soil types is necessary. Some men, proficient in soil work (26, p.26), prefer to deal with soil by its test characteristics; thus they evaluate the soil on this basis without going through the process of classification.

A few have attained this degree of proficiency and even these, it is supposed, often use classification systems.

Many classification methods have been developed somewhat independently, yet they are often based on the same tests, namely: (1) Atterberg limits, (2) mechanical analysis. The better known of these systems are:

1. Highway Research Board is a revised system based on the Public Roads Administration method which was developed in 1928.
2. Casagrande system was adopted in 1942 by the Corps of Engineers. This system uses letters to designate the predominate size range of the

soil and a second letter to denote a soil characteristic or a secondary type soil contained in the soil sample.

3. Pedeological classification is a system that is based on origin and soil history.

4. The above are frequently used for highway and airfield engineering work and are arranged in the numerical order of most frequent usage for highways (opinion of author). There are others such as:

    A. CAA system

    B. Agricultural systems

    C. Textural classification system which was used as early as 1890.

    D. Others

Note: For additional information, see (9, p.331-340)

The purpose of all systems is to have a name for a specific soil or combination of soils so that in various works, reference to a particular name will have the same meaning to all who are sufficiently interested to be familiar with the classification method.

It is believed that for highway work the HRB system is most frequently used and has become a part of the American Association of State Highway Officials Standard Specifications (2, p.48-51).

The discussion of soil and load variables reaches far beyond any similar narrative concerning such relatively simple construction materials as steel or concrete. To list some of them:

Load variables:

Maximum wheel load

Relationship of multiple wheel loads

Influence of impact and/or speed

Life expectancy and number of load repetitions

Tire pressure and contact area

Soil variables:

Soil stress distribution

Compressibility and deflection

Evaluation of critical deflection

Cohesion and internal friction

Evaluation of soil stability and correlation with field practice

Effect of moisture on soil stability

Determination of critical moisture contents from a practical standpoint

A method by which these variables can be resolved to definite and accurate information for use during design and construction has occupied the combined efforts of thousands. Many of these methods are briefly presented in the following paragraphs.

Empirical (E)

Those methods that are based on experience, ie history of past failures and successes without any tests or analyses of soil stability.

Empirical Arbitrary (E Ar)

Those methods that are based on empirical stability tests on the soil. The results are correlated with arbitrary thickness charts based on experience.

Empirical Comparison (E C)

Those methods that are analyzed by empirical tests, the results of which are compared to the strength of a standard soil.

Empirical Analytical (E An)

Methods that are based on empirical tests of soil stability and subsequently analyzed by stress-strain relationships.

Analytical (A)

Methods that use mathematical and elastic-stress ie similar to concrete design.

## METHODS ARRANGED ACCORDING TO THE PRIMARY TYPES

Empirical

- Civil Aeronautics Administration (CAA)
- Group Index
- Michigan Method

Empirical Arbitrary

- North Dakota Cone Bearing
- Plate Bearing Methods
- Triaxial Compression Methods

Empirical Comparison

- California Bearing Ratio

Empirical Analytical

- Golder Method (English)
- Kansas Department of Highways
- V. R. Smith (Triaxial and Boussinesq)
- Hveem Stabilometer

Analytical

- Burmister (2 layer)
- Burmister (3 layer)

EMPIRICAL

Group Index - This method was originated by a Highway Research Board sub-committee. The bases for soil stability evaluation is the soil index formula. The index value is then correlated with arbitrary design charts. Reference (6, p.402-403) states that the method gives satisfactory results in about 80 percent of the cases studied.

CAA Method - This procedure is very similar to the Group Index method. However it does consider the effects of frost action and drainage. It is based on the CAA soil classification system (27, p.1-56).

Michigan Method - The method was developed around the pedeological classification. It is based on the performance records dating back to 1925 (9, p.345-346).

EMPIRICAL ARBITRARY

Triaxial Methods - This system uses a soil stability apparatus which includes such variables as:

1. Vertical load
2. Vertical deformation
3. Lateral pressure
   A. May be constant during the test (open)
      This open system is used by:
      1. Kansas Highway Department

2. North Carolina Highway Department

B. May be closed, ie tends to increase lateral Pressure.

4. The results are correlated with empirical design charts which recognize such variables as:

A. Traffic volume

B. Traffic weight

C. Ground water

D. Rainfall

E. Frost penetration

5. Modified triaxial

A. Texas

B. Palmer-Barber

References (20, p.92-97) and (30, p.487-499)

Plate Bearing Methods - As the name suggests, there are several methods using statically loaded plates such as:

1. U. S. Navy, Flexural Pavement Method
Consists of constructing a trial section of approximately the correct design and then employing plate bearing tests to evaluate this preliminary design.

2. Canadian Department of Transport Method
This procedure was developed by correlating plate bearing tests with:

A. CBR (in place)

B. North Dakota Cone Method

C. Triaxial Compression

This method is based on the load required on a 30 inch circular plate to cause 0.5 inches penetration after 10 load repetitions.

References (6, p.412) and (28, p.1-87).

North Dakota Cone Method - A cone shaped penetration apparatus that converts test results (psi) to required thickness by formula $H = \frac{65.7}{b^{0.388}}$

H = thickness required

b = cone bearing value, psi

Developed from a study of past failures and successes and correlation with cone bearing values, based on an average load of 5000 lbs. with conversion factors to other loads.

Reference (6, p.411-412).

EMPIRICAL COMPARISON

California Bearing Ratio (CBR) - The many variations of this method precludes on exact outline of procedure for the general case. The CBR method is perhaps the most widely used of any method. The standard or modified procedure and graphs have been or are being used by

California, Wyoming, Kentucky, Australia, Canada, Bureau of Public Roads, England, Corps of Engineers, and others.

Perhaps one of the better highway CBR methods is that developed by the Wyoming Highway Department.

Origin - California State Highway Department.

Deflection - 0.1 to 0.2 in. penetration of the standard plunger.

Moisture - Most frequently the specimens are soaked for four days; hence, saturated soil.

Validity - Apparently these procedures give good results, perhaps overdesign in some cases.

References (3, p.281-282) and (9, p.346-350).

EMPIRICAL ANALYTICAL

Kansas Highway Department - The method involves determining the bearing capacity of the soil in psi and combines this information with the computed pressures derived from Boussinesq modified formula. Also includes an arbitrary deformation limit.

Reference (6, p.427).

V. R. Smith - Compares saturated shear strength (triaxial) with a shear strength curve derived from the Boussinesq Equations.

Reference (23, p.1-25).

Golders Method - This method is for soils that have a high cohesion value and little internal friction. It is based on Terzaghi's footing formula. The load pressures are projected through the subgrade from an assumed tire contact area at an assumed angle of 26 degrees with the vertical. This assumed soil pressure is equated to the allowable as derived from Terzaghi.
Reference (7, p.29-59).

Hveem Stabilometer - This is a procedure similar in some respects to the triaxial methods. It was developed by Mr. Hveem, for the California State Highway Department.
Reference (16, p.134-139).

ANALYTICAL

Burmisters two-layer system - An analytical method wherein the modulus of subgrade reaction is determined by the plate bearing test and then Burmisters analysis determines the stresses and strains in the subbase and subgrade with an arbitrarily selected surface strain of 0.2 inches.
Reference (4, p.126-148).

SUMMARY OF DESIGN METHODS

The previous discussions are an outline of the approximately fifty that exist today. However, it will be noted that many of the methods are a modified basic system.

The question of whether a design method exists that will give a correct answer for the general case may be answered by the fact that changes are constantly being made in the better design methods in current use. The Western Association of State Highway Officials undertook the WASHO road test in 1952 to 1954 in an effort to answer this question and others. A request to the State Highway Departments was sent out for a design that would apply to a given set of conditions. The replies to this request are tabulated below. (11, p.6 and p.18).

| STATE | DESIGN PROCEDURE | TOTAL DESIGN DEPTH INCHES |
|---|---|---|
| Alabama | CBRa, GI | 16 |
| Arizona | Based on MA & PI | 14 |
| California | R | 18 |
| Colorado | CBRo | 19 |
| Delaware | CBRa | 15 |
| Idaho | R | 17 |
| Kentucky | CBRo | 18 |
| Maryland | CBRo | 24 |
| Missouri | GI | 9 |
| New Mexico | CBRo | 20 |
| New York | CBRo | 20 |
| Puerto Rico | CBRa | 15 |
| South Dakota | CBRo | 14 |

| | | |
|---|---|---|
| Texas | Triaxial | 15 |
| Washington | RW | 15.5 |
| West Virginia | CBRo | 15 |
| Wyoming | CBRo | 13.5 |

Abbreviations:

CBRa uses Army impact compaction

CBRo other methods of compaction

R Hveem stabilometer

RW Hveem (Washington method)

GI Group Index

Two methods predominate, the triaxial and the CBR. The average total design depth for CBR was 17.3 inches, for triaxial it was 16.4 inches. One other factor is illustrated here and that is the comparative uniformity of the replies presented by those departments using triaxial equipment.

It may be possible to conclude that the triaxial method represents the best known procedure today. Certainly, triaxial has advantages for the evaluation of coarse grained soils (12, p.13) where the CBR method is not very effective.

An example of the ineffectiveness of CBR for coarse grained soils (author's experience) is where an A-1-a soil (sandy gravel) exhibited a CBR value of 270, which

according to the theory of the CBR method, is impossible.

The inherited disadvantage of the Triaxial procedure is the expenditure of funds for technicians, equipment, and the like, which the method requires. Apparently most small organizations find that it is beyond their individual financial means. County and urban road expenditures in 1953 represented 38 percent of the total road funds spent for that year (9, p.92). Therefore, it would appear that the best in design methods is what the situation suggests.

The English Road Research Laboratory (6, p.436) says this about secondary roads: "A development in the immediate future that would be of use to British Road Engineers is, for the design of less important roads, the development of a simpler, more approximate method than the CBR depending on soil classification alone, to suit British conditions, possibly something on the lines of the United States Highway Engineers' Group Index Method."

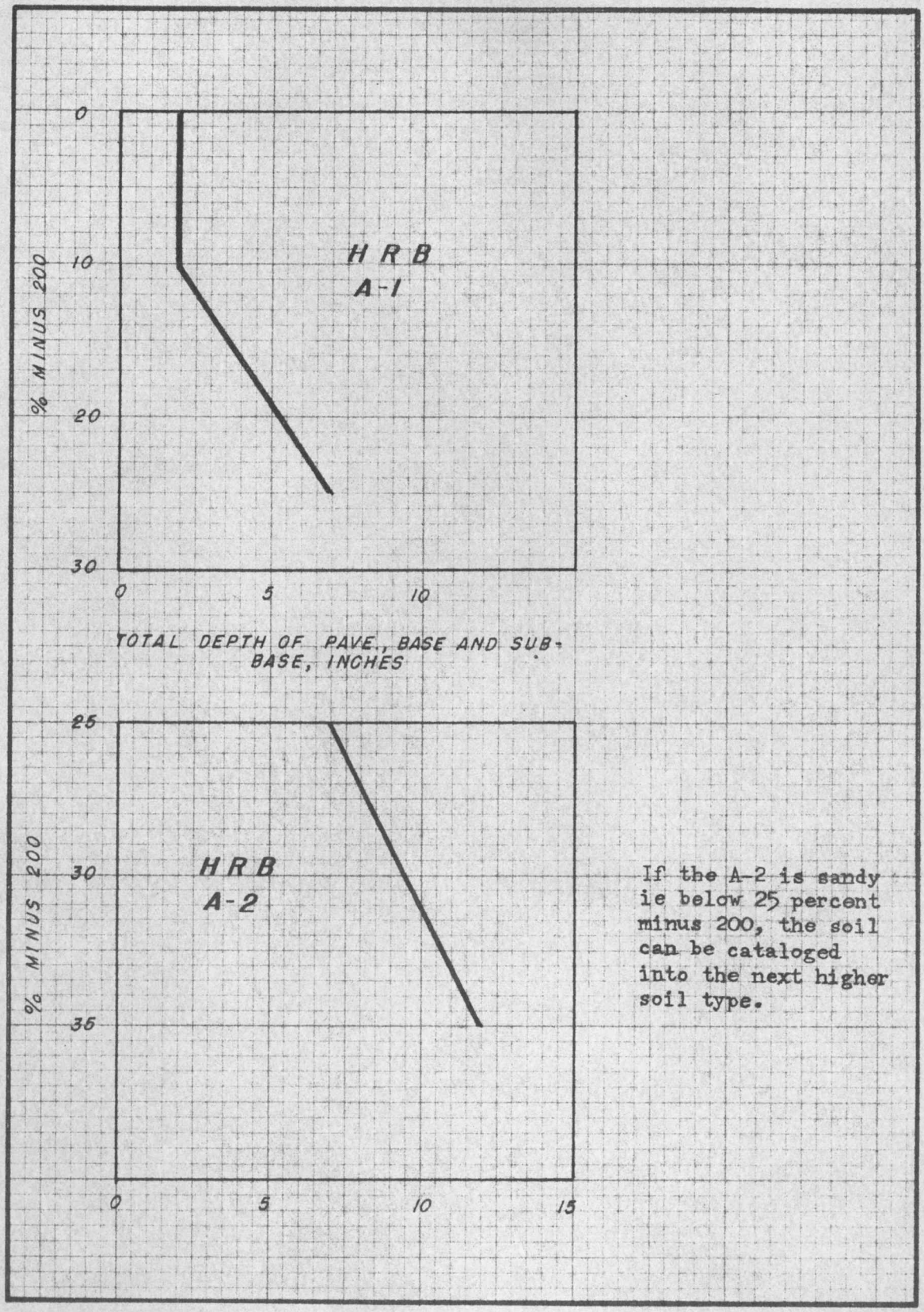
The basis of the Group Index Method is the soil formula that was developed. The soil rating is correlated with an arbitrary depth of subbase. The greater the G.I. value, the less desirable the soil is. This same idea was incorporated in the development stage of the Public Roads Administration Soil Classification system, which now under the name of HRB System has become standard throughout the country. It seems, therefore, that the HRB soils classification could

serve as an index of soil stability and in this way make the application of soils information more general since it would enable the individual to exchange ideas, information etc. with other agencies.

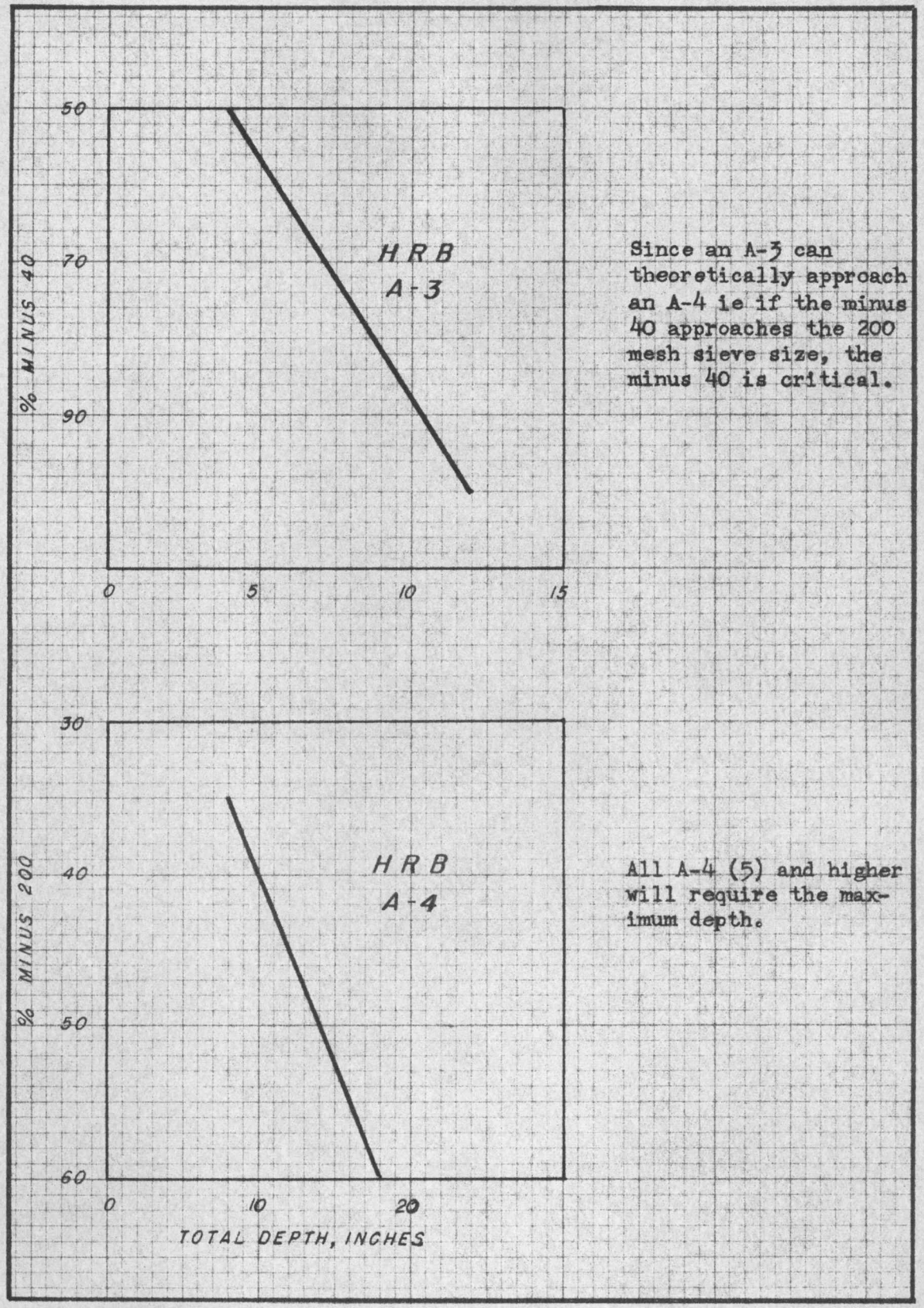
Standard HRB classification tables are attached so that references can be made to the various soil types. (See sheets 41 and 42.)

The following charts give design thickness for the various soil types, and represent a mean of available recommended thicknesses. A personal knowledge of the fallacies in a particular design system and the characteristics of the various HRB soil types served to separate a general recommendation which was inadequate or uneconomical from one that indicated closer correlation with the requirements of the soil type stability limitation.

The fine grained portions of a soil type are the materials that are affected by moisture. Therefore, in a general way, the stability of a soil combination is inversely proportional to the quantity of minus 200 included in that soil type (9, p. 313). This fact will be used to evaluate intermediate or subgroups for a particular main HRB soil class.

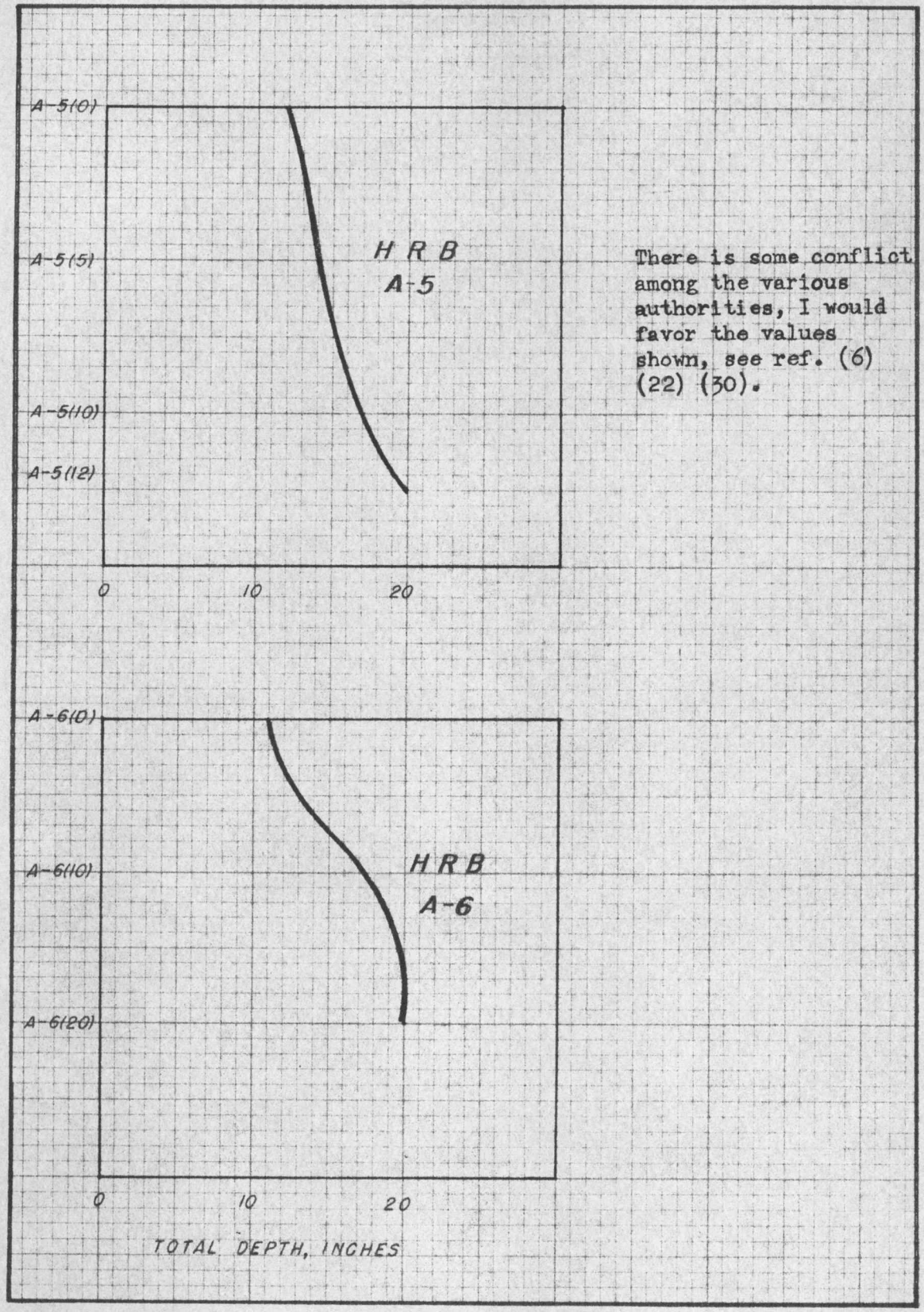



If the A-2 is sandy ie below 25 percent minus 200, the soil can be cataloged into the next higher soil type.

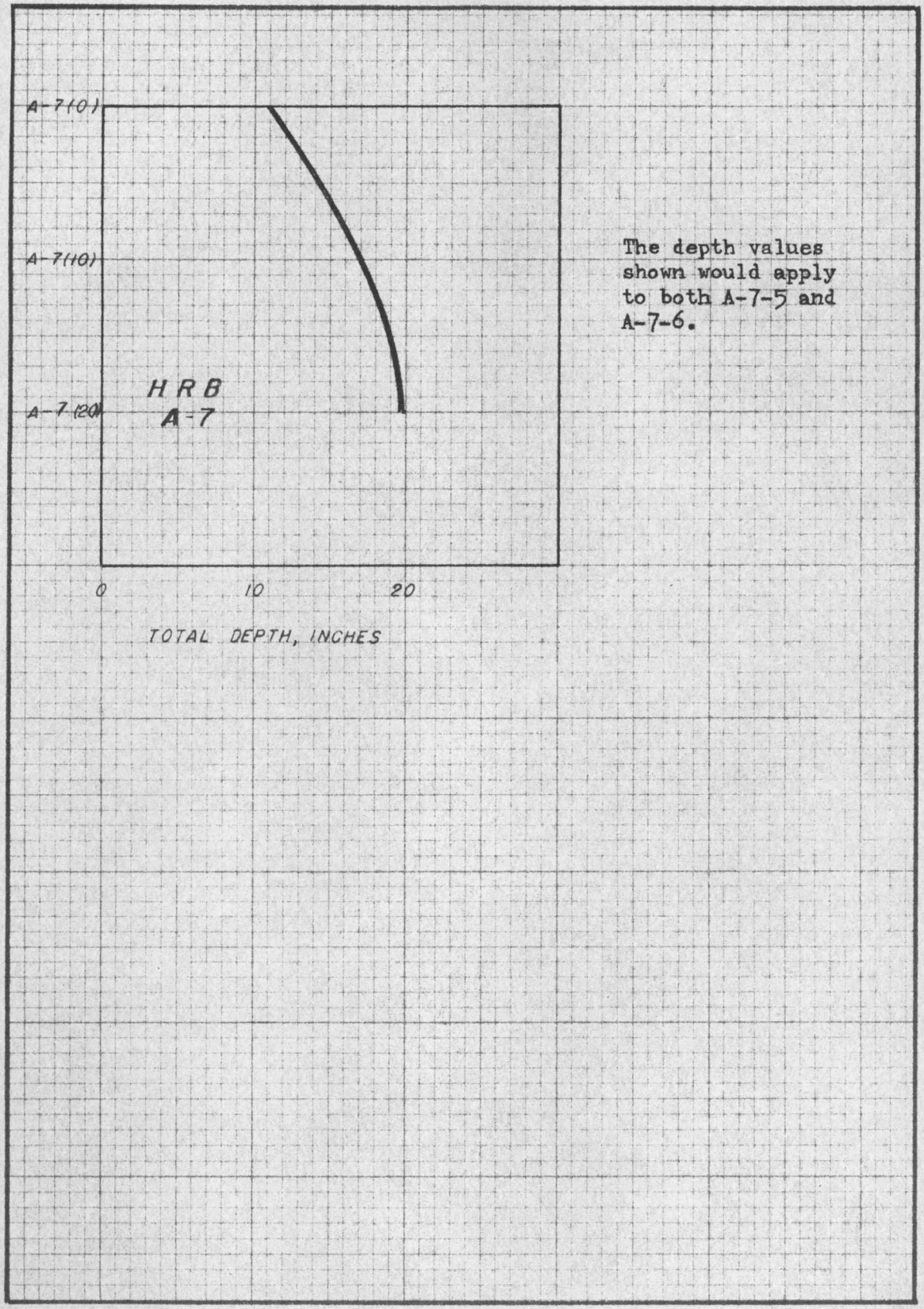



Since an A-3 can theoretically approach an A-4 ie if the minus 40 approaches the 200 mesh sieve size, the minus 40 is critical.

All A-4 (5) and higher will require the maximum depth.




There is some conflict among the various authorities, I would favor the values shown, see ref. (6) (22) (30).




The depth values shown would apply to both A-7-5 and A-7-6.

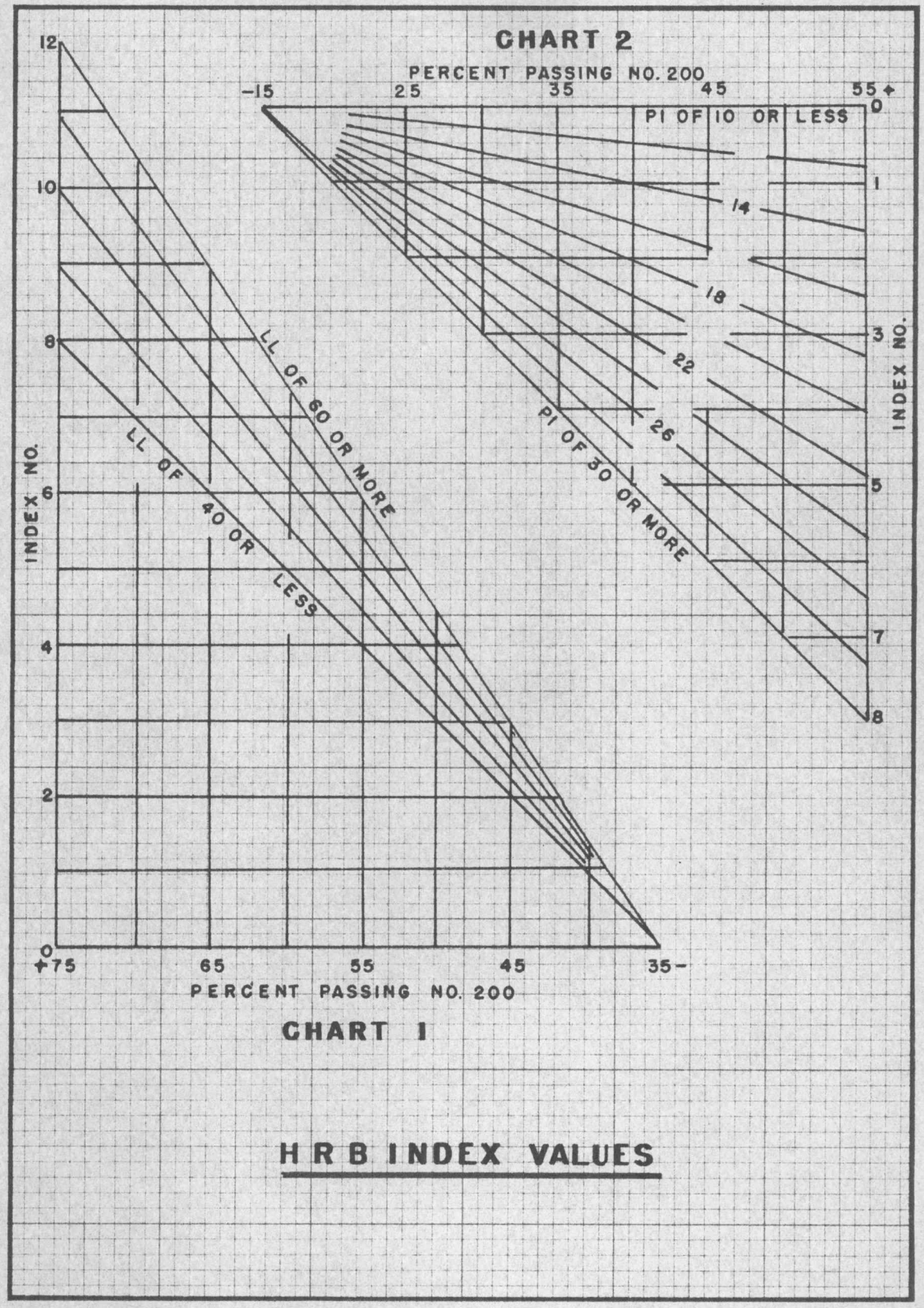
## CLASSIFICATION OF HIGHWAY SUBGRADE MATERIALS

| General Classification | Granular Materials (35% or less passing No. 200) | | | | | | | Silt-Clay Materials (> 35% passing) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Group | A-1 | | A-3 | A-2 | | | | A-4 | A-5 | A-6 | A-7 |
| Classification | A-1-a | A-1-b | | A-2-4 | A-2-5 | A-2-6 | A-2-7 | | | | |
| Sieve Analysis Percent passing: | | | | | | | | | | | |
| No. 10 | 50M | | | | | | | | | | |
| No. 40 | 30M | 50M | 51m | | | | | | | | |
| No.200 | 15M | 25M | 10M | 35M | 35M | 35M | 35M | 36m | 36m | 36m | 36m |
| Characteristics of fraction minus 40 | 6M | | N.P. | 40M 10M | 41m 10M | 40M 11m | 41m 11m | 40M 10M | 41m 10M | 40M 11m | 41m 11m* |
| Group Index | 0 | | 0 | 0 | | 4M | | 8M | 12M | 16M | 20M |

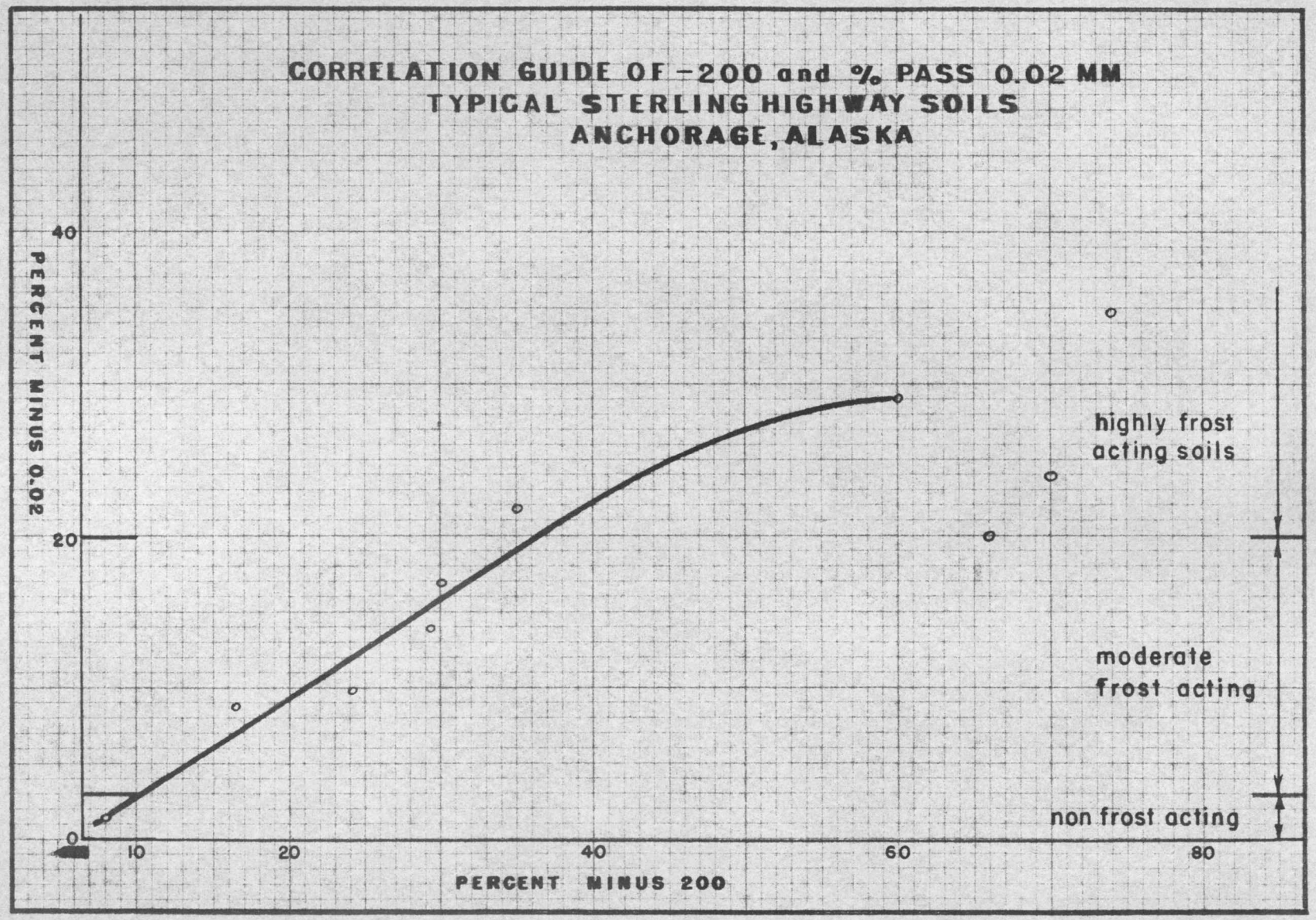
Note: M - Maximum
m - Minimum
N.P. - Non-plastic

Classification Procedure: With required test data available, proceed from left to right on above chart and correct group will be found by process of elimination. The first group from the left into which the test data will fit is the correct classification.

*Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.




## HRB INDEX VALUES


CORRELATION GUIDE OF -200 and % PASS 0.02 MM
TYPICAL STERLING HIGHWAY SOILS
ANCHORAGE, ALASKA
PERCENT MINUS 0.02
40
20
0
10
20
40
60
80
PERCENT MINUS 200
highly frost acting soils
moderate frost acting
non frost acting

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