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PERMEABILITIES & LOAD SUPPORT CHARACTERISTICS OF MATERIALS USED AS BASE OR SUBBASE COURSES IN NEW JERSEY HIGHWAYS

BRUCE COSABOOM, PRINCIPAL ENGINEER

VICTOR MOTTOLA, ASSISTANT ENGINEER

DEPARTMENT OF TRANSPORTATION
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16. Abstract

This report describes an investigation of the drainage and load carrying characteristics of natural, bank-run soils, and quarry processed materials, which are used as base and subbase layers in the current design sections of New Jersey highways. These characteristics are evaluated in light of the design assumptions held by the Department regarding the subject materials. They are further evaluated in the context of the constraints and/or requirements placed upon them by the highway "environment" and loadings. Insight regarding the effects of "blending" (the addition of coarse, crushed aggregate to bank-run soil) is provided, along with commentary on future availability of materials. Finally, some analysis and discussion of theoretical permeability prediction methods is presented.

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I. SUMMARY AND CONCLUSIONS

The load bearing and drainage characteristics of base and/or subbase materials as currently used by the New Jersey Department of Transportation, and specified thereby as Type 1, Type 2, or Type 5 materials, were investigated via laboratory CBR and permeability determinations. Samples of Type 1, Type 2, and Type 5 materials were selected to represent the various geological and mineralogical sources from throughout the State. Sample gradations were selected or artifically created to represent the extreme and intermediate gradations within each particular Type and Class specification band. The results were evaluated within the framework of the Department's design standards.

Currently, interstate highways in New Jersey basically consist of approximately 9 inches of pavement (bituminous or portland cement concrete) supported by about 6 inches of dense, well-graded base course material, which in turn is supported by 6-12 inches of slightly less-dense, relatively non-permeable subbase material upon the subgrade. Where underdrains are used, they consist of transverse outlet trenches placed to intercept infiltrated water traveling along the subbase, and located at approximately 350-foot intervals along the roadway profile.

Simultaneous field observations, conducted in the pursuit of this project and another, revealed important qualitative information regarding water in the base and subbase layers of the highway. Of significance was that base courses beneath portland cement concrete pavements, as well as bituminous pavements, were observed to be saturated following rain storms. Furthermore, these courses were observed to have remained saturated for two or three days in some cases. In addition, moisture measurements taken over a period of an entire winter revealed that for that winter, the moisture contents at most of the 31 sites monitored changed by only 2 or 3 percent, and represented a saturated, or near-saturated state of moisture.

In addition to the aforementioned, more directly applicable analysis, it was also possible to superficially study the effects of "blending", and to attempt to correlate measured permeabilities with results predicted by theoretical methods.

From the analysis of the test results, and in light of field observations, conclusions are offered as follow:

1. In a relative comparison of load bearing capacities, the various base and subbase materials, by Type and Class, generally conform to the assumptions held by the Department's Engineers. However, on an individual basis, i.e., from one source of material to another, there is little assurance of achieving the intended <u>relative</u> bearing capacities, given only the current gradation requirements as a control method. In this regard, the gradation-based specifications are inherently weak.

2. The current gradation-based specifications <u>allow</u> the use of materials which may have very little load stability under partial confinement and which have permeabilities ranging into thousands of feet per day. In practice, however, and on the basis

of field ovservations and laboratory testing, such instability is not frequently encountered, and materials having such large permeabilities do not exist in a natural state. It is necessary that the gradation bands of the current specifications be as "liberal" as they are. Further "tightening" or narrowing of the bands would probably cause the exclusion of more sources of "good" material than of poor material.

3. Materials used as base and/or subbase in New Jersey, when compacted by the Standard Proctor method (ASTM D-698), typically exhibit the highest CBR value when at less than the optimum moisture content (based on maximum density). There is no apparent further decrease in CBR values, as the moisture content is increased from optimum to saturation, while maintaining a sample at maximum (Proctor) density. This information, coupled with the aforementioned field observations regarding the state of moisture in base and subbases leads to the conclusion that the critical support layers of the pavements frequently lie in a weakened state for extensive periods of time. This conclusion is further reinforced in light of the dynamic nature of highway loading and resulting stresses.

4. Looking at the study data overall, Type 1, Class C subbase is generally a little more permeable than Type 1, Class A material, which is in general agreement with design assumptions. However, as with CRB results, there is no assurance that this relationship will be achieved in actual construction, given only the current gradation-based specifications. In reality, these two materials can be, and sometimes are,one and the same material.

5. In general, all of the base and subbase materials that comply with existing New Jersey Department of Transportation Type 1, Type 2, or Type 5 materials specifications and are available as naturally existing, "bank-run" materials should be considered for use as dense, non-plastic strength courses. With the relative performance observations of portland cement concrete pavements in mind, and in consideration of the requirements imposed upon the base courses by the highway environment, only a few of those courses in service can be described as drainable. Realistically, none of the materials allowed by the current specifications should be regarded as sufficiently drainable for pavement base courses, by any conservative estimate. In view of the foregoing statement, it follows that none of the materials currently used for base and subbase in New Jersey should be referred to, or used as a "drainage course" within the context of our highway design.

6. Generally speaking, the nature of the soils used in New Jersey base and subbase layers is such that these layers may normally possess in-situ moisture contents which are fairly close to saturation. This assertion is reinforced by the aforementioned statement regarding nuclear moisture measurements. While the exact cause(s) of the high moisture contents are debatable, and certainly vary from site to site, it may still be concluded that only very small amounts of water by seepage or surface infiltration are necessary to saturate, or super-saturate these layers. Ironically, it may be that the relative imperviousness of these soils prevents rapid saturation from occurring beneath a continuous expanse of impervious surface pavement, such as a new bituminous pavement having no cracks or joints, and a low void content.

7. On the basis of the analysis presented elsewhere herein, it is concluded that while the concept of using underdrains to remove water from the soil support layers is sound, the three basic components, i.e. the surface pavement, the soil support layers, and the underdrain, are incompatible. All pavements will allow some water to enter the base course. And the underdrain is capable of removing the water from the subsurface system, but <u>only</u> if the base course is capable of transporting the water to the underdrain. The soil support layers in use in New Jersey cannot <u>capably</u> transport excess water for even a few feet to an underdrain within the time frame thought necessary to prevent distress. Hence, the use of the outlet trench in conjunction with New Jersey's base and subbase materials can never be optimized as a complete subsurface drainage system. On the assumption that subsurface drainage is necessary, it is concluded that a complete subsurface drainage system must be incorporated into the design and construction of New Jersey pavements.

8. "Blending" as practiced in New Jersey is a method of "up-grading" a bankrun material via the addition of course aggregate, usually a crushed stone product. Although not originally planned, a number of blended samples were prepared and tested in the course of this project for two reasons: (1) some such samples were prepared to closely represent bases or subbases that had been constructed with blending; and (2) blending was used as a means of adjusting the gradation of a sample toward the course side of the applicable specification band.

With regard to load bearing characteristics, the results are inconclusive, with some samples yielding CBR increases, and others yielding CBR decreases. The most significant increases occurred with samples that had very low CBR values before blending. This tends to support construction practices, whereby blending has occasionally been used to "stabilize" some base courses. With regard to drainage capacity, it is concluded that blending to the extent currently practiced has no significant effect upon the drainage capacity of the bank-run natural soil aggregate. Blending by large amounts ($>$10% total sample weight) appears to have, at least superficially, a random effect upon achievable density. The research effort was not designed specifically to study blending. The variation of parameters such as specific gravities, particular shapes and gradations, and the relative proportions of added blending stone probably account for the "apparent" randomness in density results of blended samples. It is concluded however, that blending, particularly if achieved with the addition of material having a specific gravity significantly different from the bank-run soil aggregate, can yield highly erroneous results when density-control construction methods are used, and when the added quantities are around 10%, by weight, or more. If the added quantities are less than 10%, there is no sound evidence that the blending serves any beneficial purpose.

Blending a natural soil with a large proportion of crushed stone or gravel shifts the gradation curve of the total sample toward the coarse side of the specification bands where ultimately the obtained material is actually a very open-graded material having minimum permeabilities of a few hundred feet per day. However, the quantities of crushed stone required for this to happen are around 80% or more, of the total sample weight.

9. As an offshoot to this project, it was attempted to correlate the permeability test results of this project with results obtained via permeability prediction methods. Existing prediction methods are essentially empirically developed equations which consider those parameters thought to influence permeability. One of the latest methods is that of Lyle Moulton of West Virginia University, in which an equation is developed on the basis of three parameters: (a) P_{200}, the percentage by weight of sample that passes the #200 sieve; (b) D_{10}, the effective particle diameter for which 10% of the sample is smaller; (c) the dry density, or conversely the porosity of the sample.

Correlation between Moulton's formula and the project test results was not good, and substantial deviation occurred for P_{200} amounts of $\geq 6\%$, or generally for samples with very low measured permeabilities. Using the results obtained from the project tests, a multiple regression analysis was performed on the same parameters, and an equation evolved which differed substantially from that of Moulton. The large standard error of estimate, or confidence limits, associated with this new equation, as well as the poor correlation between the test results and Moulton's formula, indicate in the opinion of the authors, that these three parameters alone, are inadequate for prediction of permeabilities occurring over a range of several orders of magnitude.

Noting certain weaknesses in using P_{200} and D_{10} values to represent sample gradations, an alternative method of particle size representation was attempted through the use of a single, combined gradation modulus called the "$\overline{A}$-value". Although $\overline{A}$-values were developed for use in mix designs of portland cement concrete and bituminous concrete, they are essentially a representation of a complete gradation of a sample, with only a single value. After investigation of the method of computing $\overline{A}$-values, it was felt that the use of $\overline{A}$-values would be superior to the use of P_{200} and D_{10} quantities. The resulting analysis did not provide improved permeability prediction capability. However, the proper determination of $\overline{A}$-values requires a more refined sieve analysis than that which was routinely performed during this project. In spite of this consideration, the authors conclude that these results help confirm the foregoing conclusion regarding the inadequacy of only shape parameters and porosity to determine permeability. On the basis of the rather superficial investigation described elsewhere herein, it appears that $\overline{A}$-values are potentially more useful than D_{10} and P_{200} quantities in such empirically derived permeability prediction equations as presented.

In an effort to further resolve the deficiencies in predicting permeabilities, another variable was sought. In this project, samples were gathered from throughout the entire state. Since, generally, the southern portion of the state is former ocean-bottom material, and the northern portion of the state represents the leading edge of glacial advance, with a rather extensive conglomeration of soil materials having various origins, and therefore various particle shapes, it is highly probable that a particle shape parameter should be considered. The State Department of Transportation of New York has developed a permeability prediction method which indirectly incorporates a shape factor through the inclusion of the specific surface of solids as one variable. It was not possible to evaluate that method within the scope and constraints of this project. However, it is thought that the addition of this factor (i.e. particle shape) may provide the refinement necessary in order to predict permeability with sufficient accuracy. Conversely, by isolating particle shape and holding it constant for a group of samples with various densities and gradations, it may be possible to develop a series of the former type of equations, that would apply on an individual basis to any material depending upon the effective general shapes of the particles of that material.

II. Recommendations

This research has supplied data on the more rational engineering characteristics of the soil aggregate and crushed stone materials currently specified for use as base and subbase materials in New Jersey's state and interstate highway construction. Although there is little direct usage for such newly acquired CBR and permeability data in the design process in New Jersey, the investigation has shed new light on the individual, as well as the overall, relative character of the various aforementioned materials, and some interesting conclusions were thereby presented. On the basis of those conclusions, the following recommendations are offered:

1. It is felt that the bank-run soil aggregates currently being allowed by specification and used as base or subbase materials, are borderline materials i.e., they are not consistently adequate, nor are they consistently inadequate, for their intended use. The degree of adequacy depends upon several highway environment parameters, such as the magnitude and frequency of loads, water accessibility, localized natural drainability of the pavement bed, or subgrade, and others. Until a substantially more rational approach to pavement design (including sub-surface drainage) is adapted, there is little sound reasoning to support recommendations for changes to the current gradation bands of Types 1, 2 or 5 materials. In the absence of any implementation of the findings of this research project, it is likely that a permeability requirement in combination with stability and gradation requirements would only slightly improve the overall quality of base courses.

In the presence of a sub-surface drainage system consisting of a drainage base course and outlets, the existing specifications would be pertinent for the underlying subbase. While no changes to those specifications are recommended in an urgent fashion, it is recommended on an interim basis that the Bureau of Soils Design, in light of the range of CBR values attained in this work, give some consideration to the addition of a minimum load stability requirement for those materials which are to be used as dense supporting layers. A CBR test method is specifically <u>not</u> recommended for this purpose until the method is correlated with field results and another more applicable method such as a plate bearing method. In the event that only this approach (with or without incorporation of drainage) is taken by the Department, a general reevaluation of the specification is recommended since it is likely that only two (or three at most) different gradation bands would be necessary.

2. "Blending", or the addition of one size of aggregate to an existing, "bank-run" soil, is currently practiced in New Jersey for one of two reasons. Usually blending is performed for the purpose of "meeting" specifications requirements. A common example of this would be when a contractor adds 3/4 inch stone to a soil in order to theoretically reduce the percentage by weight passing a smaller sieve, i.e. a #4, or a #200 sieve. During the course of this project, samples of an "as constructed" blended base course were taken from beneath a pumping portland cement concrete pavement (Sections of I-295, south of Trenton), and it was found that without the blended stone, the samples had effective P_{200} amounts of up to 12%. Permeability tests on the blended, and on unblended samples of the basic material indicated that: (1) the permeability of the basic soil was around 10^{-4} ft./day, and (2) blending had no affect on the soil permeability. Therefore, it is seen that if the base course is selected for use as a drainable medium, the allowance of blending is indirectly detrimental.

The second reason for blending is in cases when a bank-run soil meets gradation requirements, but lacks unconfined stability. This occurs occasionally when the material is too sandy. Blending, via addition of coarse aggregate, can sufficiently stabilize such materials, and in fact, has been effectively utilized in that manner.

Blending for the second purpose is legitimately justified, whereas blending for the first purpose lacks rationale, and as indicated above, may actually be defeating the purpose of the gradation specifications. Therefore, it is recommended that the Department carefully reassess its position regarding the use of "blending" in base course materials.

3. In the course of this study, and another, related research effort, it has become apparent that sources of the good quality, granular material which typically comprises Type 1A base course in northern New Jersey, is quickly becoming virtually unobtainable. Fortunately, the highway construction program is at an ebb in New Jersey. It is strongly recommended that, during this period of grace, the Department investigate the performance and availability of alternative materials and/or highway design sections.

4. It is recommended that current and future attempts to develop theoretical permeability prediction methods give consideration to the use of a single, combined size parameter, such as the Gradation Modulus $\overline{A}$, and to the inclusion of a shape parameter, both in conjunction with a porosity parameter. The work cited herein as Reference #9 includes analysis of particle shapes and warrants further investigation with regard to permeability prediction.

The above recommendations are a direct result of this research. However, it is the opinion of the authors that the most outstanding, significant result of this work is its further accent upon the need for answers to questions that have existed too long. And basically, they all center about one issue: water in the highway section. Therefore, in New Jersey a comprehensive program of research is recommended which would encompass the following additional points:

5. A thorough investigation of the root causes of "failures" in portland cement concrete pavements, is sorely needed. These failures are usually well defined, and basically the causes are identified. However, there are no limiting criteria on these problem areas from which to proceed with a more rationale design for the pavement.

For instance, the limiting conditions for pumping must be determined. The effective porosity and drainage capability of the pavement system at which pumping cannot occur, or will no longer be detrimental, must be determined, after which engineers could apply appropriate judgements in the design of a system that would properly accommodate anticipated design quantities of water. Simultaneously, other aspects must be considered as well, such as the possibility of frost heave or deterioration, and subsequent differential settlements, or the relative load stability of various types of "drainable" base courses.

6. A similar, parallel investigation into the root causes of bituminous concrete pavement failures and design requirements is also urgently recommended. With the bituminous concrete pavements as currently constructed in New Jersey it is often difficult to find a clear definition of "failures". It is even more difficult therefore to recognize causes behind failures. For this reason the role that water may play in the failure of a bituminous roadway remains very

obscure and should be thoroughly studied. The phenomenon of "liquefaction" (described elsewhere herein) of granular materials is of particular interest in such a study.

7. It is interesting to note that the 9-inch thick bituminous pavements, as now used in New Jersey, have evolved from a much thinner pavement placed upon a macadam base course. In essence, the macadam pavements exhibited generally severe deterioration long before their intended longevity. The approach to this problem was to replace the macadam base with a very dense bituminous stabilized base having very low voids. This in effect is a "full-depth pavement", although it is not referred to as such in New Jersey. This kind of approach makes no attempt to solve the problems directly, but rather just pushes them into a more remote or obscure status, while perhaps introducing new problems of a different nature. Justification for this approach is very questionable, since many other users of bituminous pavement have not found it necessary to follow suit. Consequently, it is felt that this kind of approach may be denying the chance to attain the most economical overall solution.

Therefore, in conjunction with the above two recommendations, it is recommended that a reevaluation be performed of the entire pavement section, with particular emphasis on the accommodation of subsurface water. It is felt that the Department, through the continued adherence to only the gradation specifications, is fence-sitting. The Department must take a strong position on the matter of all sub-surface drainage, and then having established such goals, should react accordingly.

A positive reaction toward such drainage necessarily includes an overhaul of design and testing methods to reflect dynamic effects and related means of testing, including actual test tracks, as well as eventual use of analysis methods involving stress and strain criteria. This admittedly is a large program, but is never-the-less strongly recommended.

The merits of such a program would be an improved rationale on which to base the selection of all elements involved in the overall pavement section. Any comparative evaluation of different overall pavement sections should be conducted in light of the findings from the investigation recommended in items #1 and #2 above and should therefore consider load-carrying capacity, water-draining capacity, overall frost penetration and affects, long-term durability of the pavement, and "constructability" and economics, as well as future availability of materials.

III. INTRODUCTION

A. General

This report is the final report for New Jersey Department of Transportation Study #7775, "Permeability of New Jersey Base and Subbase Materials". The objective of the study is to determine permeability and load-bearing characteristics of materials that are used for base and subbase courses in New Jersey Interstate highways. On the basis of said determination, and in the context of the current design and construction of Interstate highways in New Jersey, the current method of specification, i.e., gradation requirements are evaluated. Conversely, given those materials allowed by the current particle size requirements (see Table 1) the subsurface drainage capabilities of standard pavement sections are investigated.

This study is basically an investigation of the status quo with regard to water in the base and subbase courses, and is therefore limited in scope in the sense that while problem areas may be identified, no provisions are included for the trial and/or evaluation of any gross changes in either the design of the "standard" pavement cross-section or in base-subbase material specifications. Nor is any consideration given to permeability and load carrying capacity of the subgrade materials, since there is evidence that water-related problems are substantial in the more critical base and subbase layers.

TABLE 1

NEW JERSEY DEPARTMENT OF TRANSPORTATION
STANDARD SPECIFICATIONS
FOR BASE AND SUBBASE AGGREGATES

Type	1			2		5
Class	A	B	C	A	B	A
4"	100		100			
2-1/2"						100
2"	70 - 100	100		100	100	
3/4"	50 - 95	65 - 100	60 - 100	70 - 100	70 - 100	55 - 90
No.4	30 - 60	40 - 75	30 - 100	35 - 75	30 - 80	25 - 60
No.50	5 - 25	5 - 30	5 - 35	15 - 30	10 - 35	5 - 25
No.200	0 - 7	0 - 7	0 - 5	*	*	3 - 12

*If the total of shale, slate, schist and soft and decomposed aggregate be five percent or less, the quantity passing the No. 200 sieve shall be not less than 4.5 percent and not more than 12 percent. If the total of shale, slate, schist, and soft and decomposed aggregate be more than five percent but no more than 25 percent, the quantity passing the No. 200 sieve shall be not less than four percent and not more than nine percent. The amount of shale, slate, schist and soft and decomposed aggregate shall not exceed 25 percent.

B. Background

1. Description of the Problems

The current New Jersey Standard Specifications for soil aggregates used as foundation materials in highways is primarily a set of gradation requirements. These specifications were originally formulated to encompass gradation of soil aggregates which were available at the time and appeared to have been effective in previous highway applications. Their formulation was therefore empirically based and also extremely subjective. The problems associated with this are essentially twofold: (1) there is little or no basis on which to evaluate even minor changes to the specification, and, (2) when in spite of adherence to the specifications, failures occur in the field, it is difficult to find a rational explanation for them. These problems have become increasingly prevalent in New Jersey with the construction of the Interstate Highway network. Other factors, as follow, further complicate matters.

New Jersey is heavily populated and highly urbanized and suburbanized. Use of the small "open" areas that still exist in this locale is extremely difficult for several reasons. Suffice it to say that in the midst of a probable abundance of material that meets our gradation requirements, "good" material that is actually available for use has become quite scarce. Base and subbase materials must often be transported over long distances, and are sometimes dredged from the sea or rivers. Compounding the problem is the fact that New Jersey lies directly between nearby, large metropolitan areas such as New York City, Philadelphia, Baltimore and Washington and therefore serves as a corridor for large volumes of through-state traffic. This further accentuates the demand for more highways and puts even more pressure on the "supply-demand" relationship for the base and subbase materials. Ultimately, this pressure has lead to serious questioning of the validity of the apparently strict, exclusive aspect of the gradation based specifications. In some cases of highway construction, materials are artifically blended to meet specifications. In such instances, the blending usually consists of adding coarse aggregate (crushed stone products) to a material which usually has an excessive percentage (by weight) passing the #200 sieve. The result is a soil aggregate which passes gradation requirements, but is costly to produce and has in general the permeability and stability of the unblended material.

Another consideration in this matter is the fact that a significant portion of New Jersey's Interstate highways have exhibited early "failures" which are often attributed to the presence of water in the soil layers directly beneath the pavement, i.e., in the aggregate base and subbase layers. Pumping, loss of bearing capacity, and frost heave are all directly attributed to water in the soil system, and at least the first two of the aforementioned are <u>known</u> problems in New Jersey pavements. The questions therefore follow: "Is the current specification really providing only the kind of material that is needed?" and "Does the specification exclude from use only the poor soil materials?" These two questions generate a host of other questions which all lead to the inescapable conclusion that very little concrete engineering information exists about the base and subbase aggregate materials. Therefore, in order to even consider changes to the specifications, or a more adept use of them, substantial data should first be gathered.

All of these things together, provided the spark that kindled this research project. The limited scope of the project evolved from a few considerations. The primary consideration, however, was that there were so many unanswered questions, that the need for a very large and comprehensive investigation was indicated. To intelligently formulate and determine the course of any such project, it was thought prudent to first obtain a deeper working knowledge of existing methods and materials. Therefore, the permeability and load carrying characteristics of our base and subbase soils were to be determined, and then these soils were to be evaluated in light of their use and their role in the overall pavement cross-section.

2. The Status Quo

The present design practice in New Jersey is to support a 9" - 10" surface pavement with a 6-inch thick densely graded base course, which is in turn supported by a less dense layer of soil subbase, generally of 6" - 12" thickness. The thicknesses vary from job to job depending upon anticipated traffic and numerous other factors. But the basic function of the base course is primarily to provide strength, whereas it is thought that the subbase layer need not be as strong and should serve to aid subsurface drainage somewhat. In addition, standard designs call for subbase outlet drains at 350 foot spacings along the pavement profile and at any low point in the profile, for removal of water from base and subbase layers. An outlet drain is a trench filled with coarse aggregate (usually 3/8" stone) that is cut transversely across the roadway subgrade. The top of the outlet drain is in contact with the bottom of the subbase and permits removal of subbase water which is flowing parallel to the road profile. Water intercepted by the drain is carried to an inlet basin or a pipe daylighted on an embankment slope.

The irony of this "design" is that these layers aren't really designed. They are more or less simply put together on the basis that it seems to work pretty good. Yet when failures occur, one is forced to look back at the design and reevaluate its adequacy or the adequacy of the materials involved to perform according to the design conditions. It is in this context that the following study is presented.

IV. STUDY PROCEDURES

A. Representative Sampling

Soil samples were gathered such that, as much as possible, a complete array of base and subbase course materials would be tested. The selection was based upon specification Class and Type, as well as geological origin. A statistical or weighted sampling approach was neither necessary nor practical to meet the scope and objectives of the project.

1. Selection by Class and Type

Table 1 is a condensed version of "Table 36 - Soil Aggregates Gradation" which is part of the New Jersey State Department of Transportation's "Standard Specifications for Road and Bridge Construction." In Table 1, only those materials which comprise bases and subbases are included. To help envision the gradation requirements, the "bands" are plotted in Figures 1, 2, and 3. As a matter of point, the material designated as Type 2, Class A has been deleted from the latest version of Table 36 because of the extreme scarcity of said material. Samples of each Type and Class, including "2-A", were gathered. Within the gradation band of a Type and Class, attempts were made to find or make some samples to represent the outer allowable limits, and others to cross the band. Figure 4 illustrates this concept.

2. Selection by Geological Origin

Upon examination of Table 1, it is seen that apparently small differences exist from one Type and Class to another. In practice, the differences are often very subtle if one only considers the sieve analyses in comparison. But considering load carrying capacity, substantial differences can exist.

Probably more so than anything else, these different specifications evolved from the fact that the State is sharply divided into greatly different geological facets. Generally in the northern half of New Jersey there is a conglomeration of shales, traprocks, limestones, and gneisses representing the edge of the glacial advance. In contrast, southern New Jersey has little or no rock, nor shale. It is practically

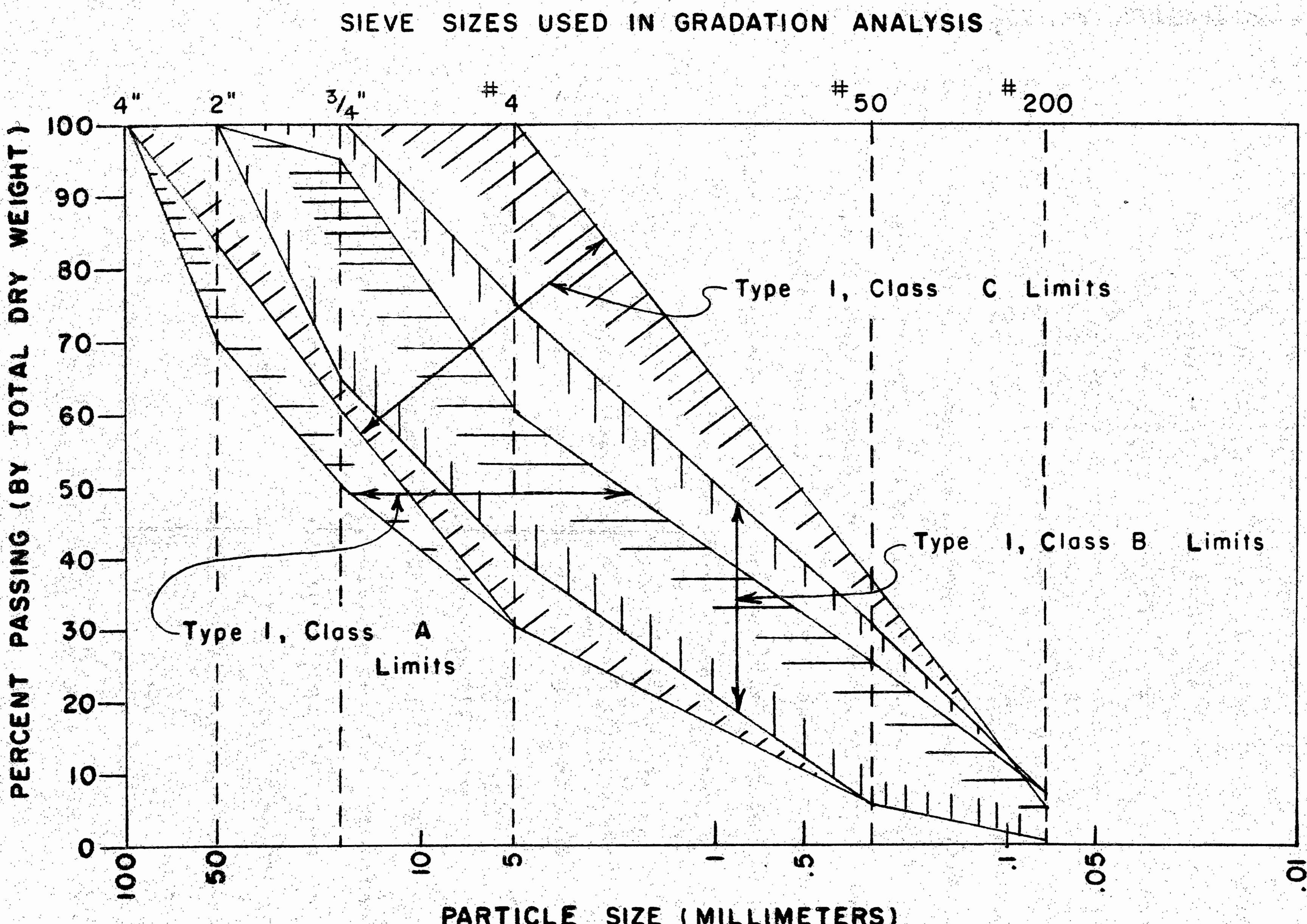


Figure 1 – SPECIFICATION BANDS FOR "TYPE I" SOIL AGGREGATE MATERIALS.

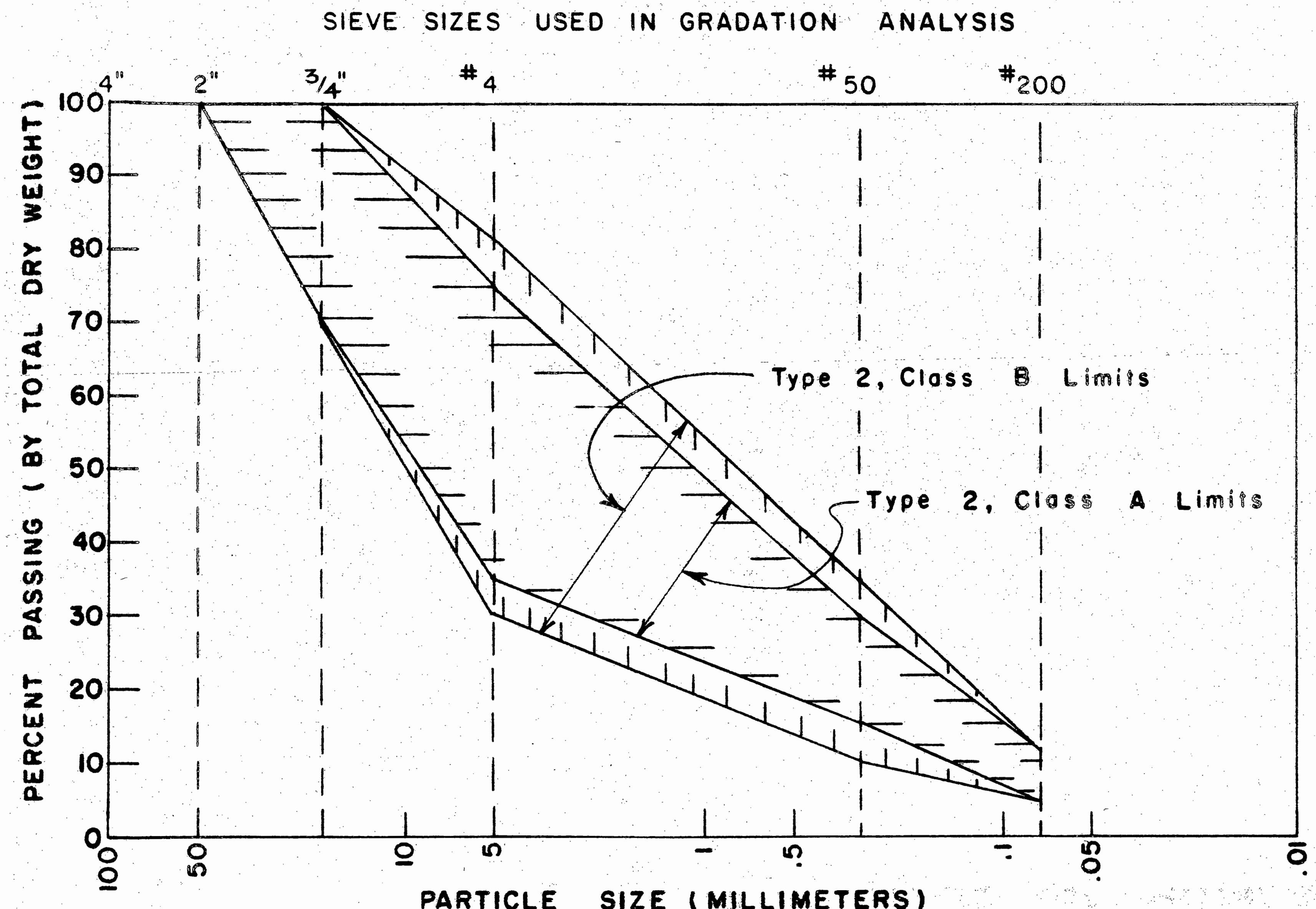


Figure 2 - SPECIFICATION BANDS FOR "TYPE 2" SOIL AGGREGATE MATERIALS.

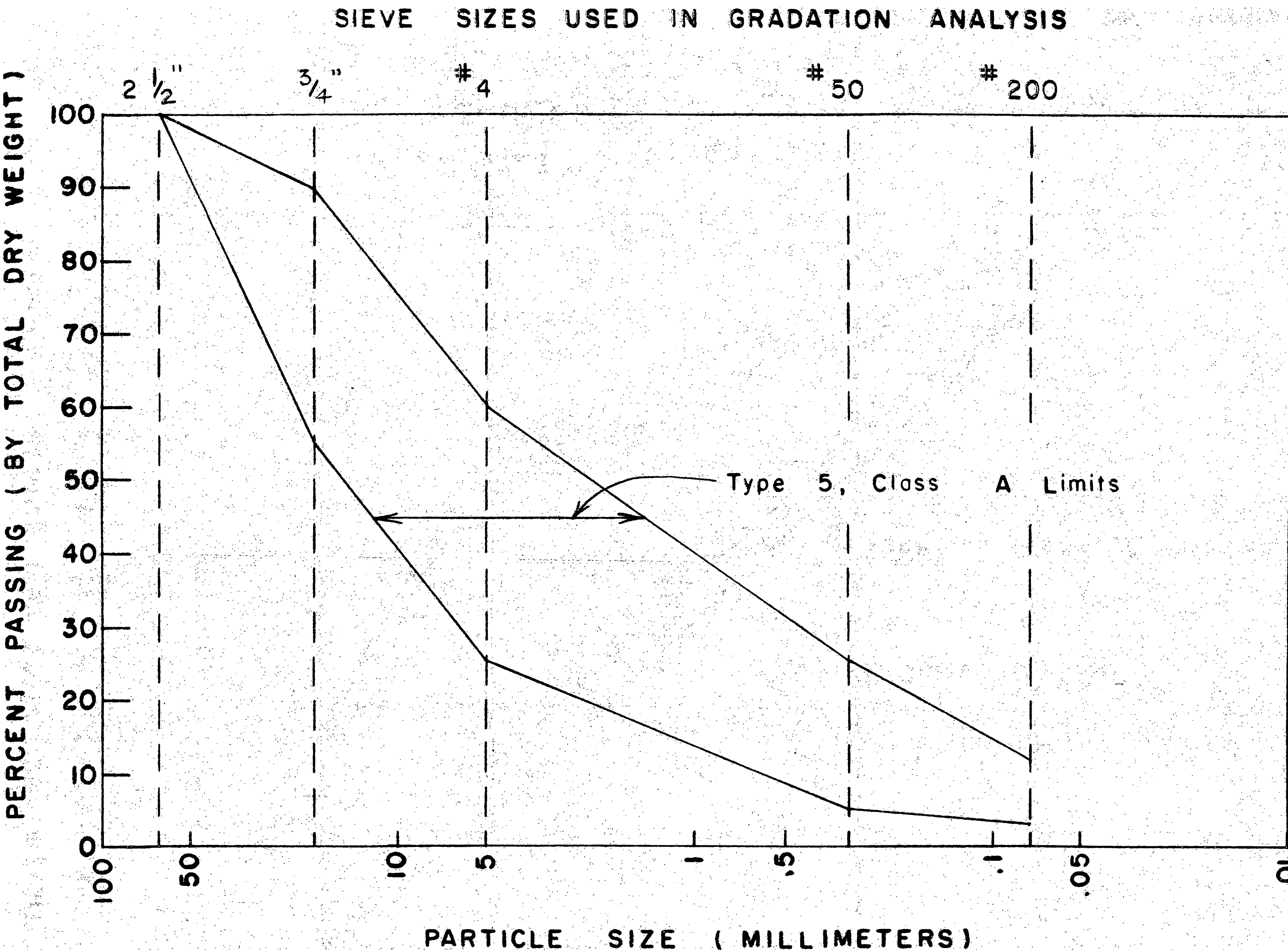


Figure 3 – SPECIFICATION BAND FOR "TYPE 5" CRUSHED STONE BASE COURSE MATERIAL.

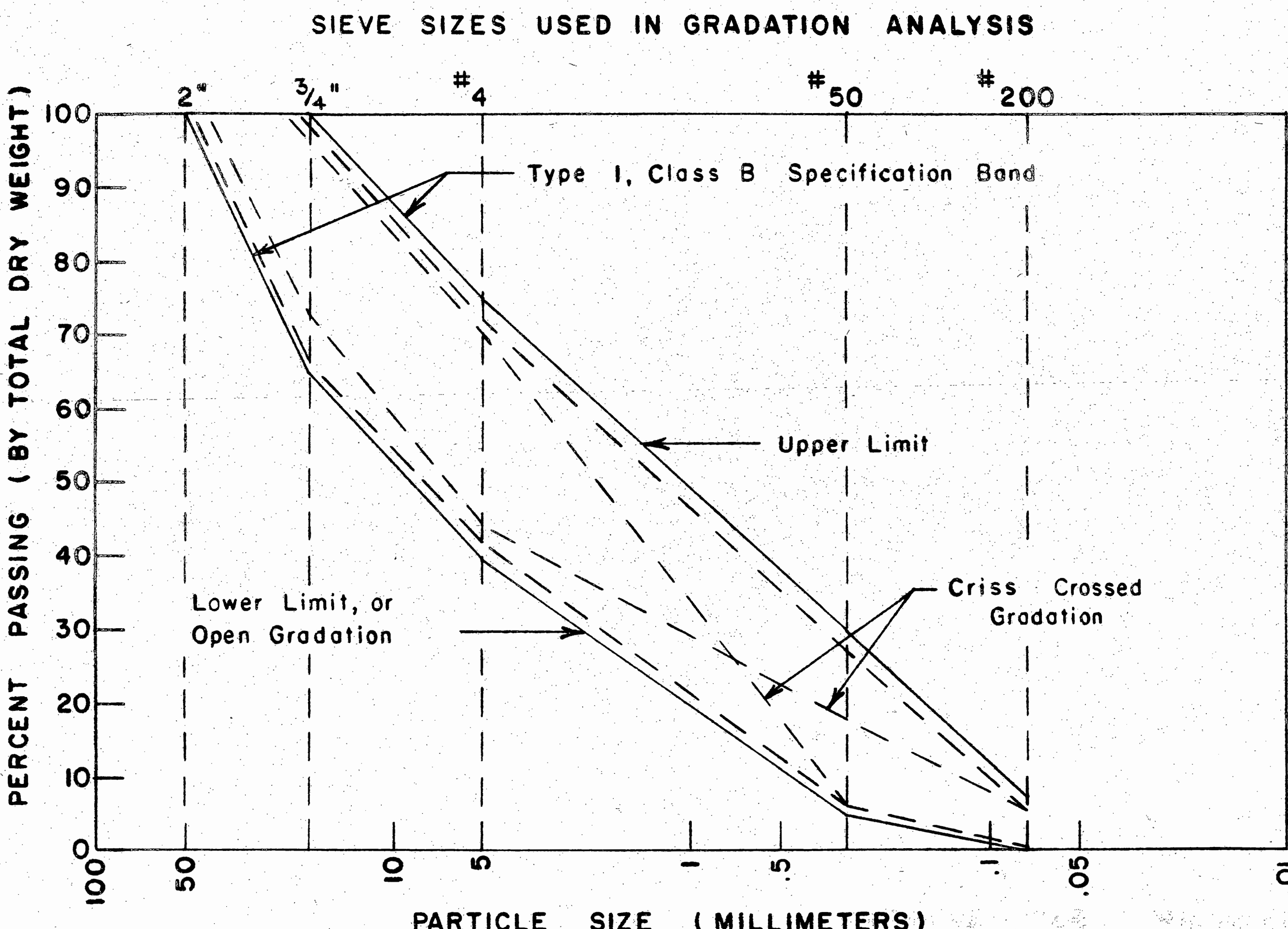


Figure 4 — DIFFERENT GRADATIONS WITHIN A SPECIFICATION BAND.

all ocean bottom material consisting of siliceous sands, clays, silts, and gravels. Therefore, in general, a gradation band for a particular Type and Class was derived according to the material which was available in a certain area of the State. As an example, Type 1, Class A material is commonly referred to as "North Jersey" base course, whereas Type 1, Class B material is commonly called "South Jersey" base or subbase.

For clarification, a very general description of the use of each material follows:

> Type 1, Class A (1A) - North Jersey base course placed directly beneath portland cement concrete pavements.
>
> Type 1, Class B (1B) - South Jersey base course placed directly beneath portland cement concrete pavement. Usually placed in thicknesses sufficient to serve as both base and subbase.
>
> Type 1, Class C (1C) - North Jersey subbase placed beneath 1A base course or beneath 5A material.
>
> Type 2, Class A, Class B (2A or 2B) - South Jersey material called "Gravel Base Course". Used where a very hard, dense course is desired, usually under the bituminous shoulders of a portland cement concrete pavement or as the soil subbase beneath a bituminous concrete pavement. Used in southern New Jersey because of a lack of 5A material.
>
> Type 5, Class A (5A) - Quarry processed crushed stone, used as a very dense, hard base course to support bituminous concrete pavements. Usually restricted to northern New Jersey pavements.

In order to encompass the majority of base and subbase materials in our testing program, meetings with the Department geologists and soils personnel were held. Using their expertise, the types of soil aggregates to be tested, and the sample selection sites were chosen.

New Jersey has, in general, seven geologic sources of material used as base and subbase. In south and central New Jersey, for testing purposes, the material was grouped into the Pennsauken Formation, Cohansey Sand, and Hydraulic Material. In northern New Jersey, 2 types of subbase and base sources were chosen, i.e., Limestone valleys and Gneiss and Quartzite sources. Also found in northern and central New Jersey are Trap Rock sources from which high quality crushed stone, "5A" material, is produced. The final source is limestone sources of crushed stone from Pennsylvania.

Material in the Pennsauken Formation consists of gravelly sand, mostly quartz with varying, sometimes large, amounts of clay. The formation is composed of heterogeneous Pleistocene alluvial deposits, and outcrops in a band from Sayerville southwest across the state to Salem. (See Figure 5) The Cohansey sands are comprised of quartz with varying amounts of clay and some small gravel; clay content is usually much less than in the Pennsauken formation. Cohansey sands are found throughout much of southern New Jersey. These two formations yield any of 1A, 1B, 1C, 2A and 2B materials. Hydraulic material is, for the most part, a clean quartz sand. It is a product of dredging and is obtained from coastal bays, and at times from the Delaware River. Dredging is a source for 1C material.

Limestone valley material (1A or 1C source) is heterogeneous glacial debris. Its composition varies widely from being larger carbonate rock, shale and slate fragments, to gneiss and quartzite.

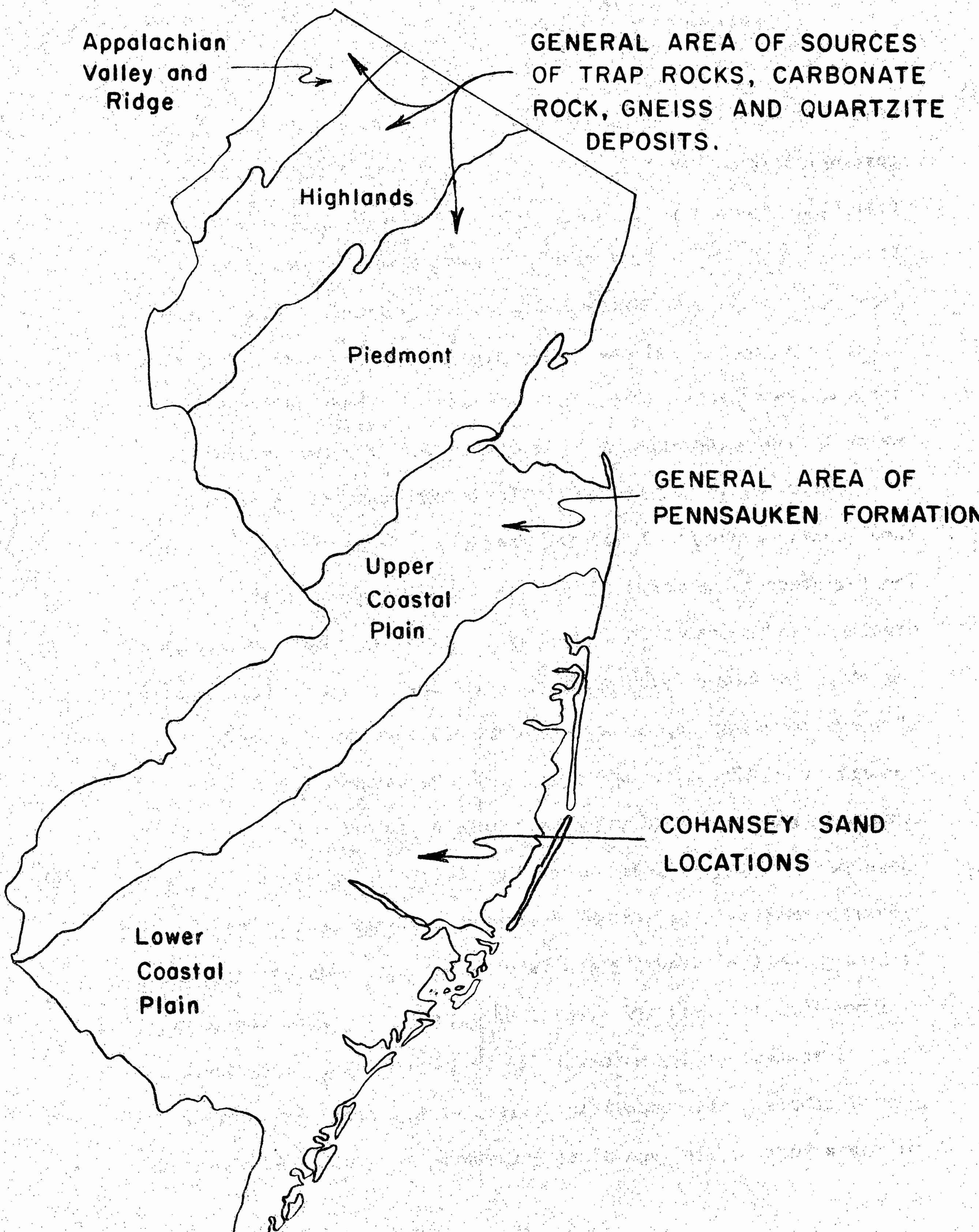


Figure 5 — GEOLOGICAL MATERIAL SOURCES.

This material, often in "bankrun" form, is scattered throughout the mountainous regions of northern New Jersey, notably, Hamburg, Sparta, Blairstown and Belvidere.

Gneiss and quartzite sources are usually "rock gravel"; hence they are often the source for Type 5A products via crushing operations. These are found in the Wharton, Hopatcong and Riverdale areas to mention a few. Located in northeast and central New Jersey are isolated igneous outcrops known commercially as "Trap Rock" which is very hard and durable diabase or basalt. It too is used to make quarry processed subbase (5A) -- a primary crusher product generally composed of all the material that passes the 2-inch screen. New Jersey also uses some Pennsylvania carbonate rock which is quarried in the New Hope and Norristown areas. The material is used in quarry processed subbase (5A).

B. Schedule of Sampling

In order to effectively study all of the aforementioned materials it was estimated that approximately 60 samples of soil material and blended aggregates should be tested. This figure was based on an estimated total of 15 sources which would include all Types and Classes in Table 1, and which would also afford general coverage of the various geological sources. For each of the 15 sources, it was intended to try to obtain 4 differently graded samples within a particular specification band, illustrated in Figure 4, and therefrom a total of 60 samples was derived. Table 2 summarizes the Types and geological sources actually sampled, and

TABLE 2

SAMPLING AND TESTING OF MATERIALS
USED AS BASE OR SUBBASE IN NEW JERSEY HIGHWAYS

Specification Type of Material	Actual Testing Source	No. Samples		No. of Replicates		Total No. of Tests
1A	1 Carbonate Source	2	x	3	=	6
	2 Granite/Quartzite Sources	5	x	3	=	15
	1 Pennsauken Formation Source	3	x	3	=	9
1B	4 Pennsauken Formation Sources	3	x	3	=	9
		7	x	2	=	14
1C	1 Carbonate Source	2	x	2	=	4
	1 Pennsauken Formation Source	4	x	2	=	8
	2 Hydraulic Sources	2	x	3	=	6
	2 Hydraulic Sources	5	x	2	=	10
	1 Granite/Quartzite Source	3	x	2	=	6
2A	2 Pennsauken Formation Sources	5	x	2	=	10
2B	2 Cohansey Sand Sources	2	x	2	=	4
	3 Pennsauken Formation Sources	3	x	2	=	6
5A	2 Carbonate Sources	2	x	2	=	4
		1	x	3	=	3
	1 Diabase Source	4	x	2	=	8
	1 Granite Source	1	x	3	=	3
	Totals	54				134

the total number of samples, including replicates, tested. At the start of the testing program, each field sample was divided and three replicates were prepared for tests. This procedure was reduced to two replicates after good repeatability in the testing procedure was demonstrated.

As a minor off-shoot of the project, a few samples of base course material were taken from two highways that were already in use and had demonstrated some particuarly interesting behavior such as "pumping", or in another case, excellent long-term stability.

C. Test Methods

1. General

Equipment for testing is detailed in Appendix A. To the extent practical, standard available testing apparatus and methods were utilized. Although permeabilities and bearing strengths were of primary concern, it was also necessary to perform a particle size determination for identification purposes and to determine some "appropriate" density at which to perform those tests. These are described hereinafter.

2. Incidental Tests

For density requirements the optimum moisture content and maximum density was determined using ASTM D-698-70, Method D (5.5 lb drop hammer and 12" drop). This test was performed on material passing a 3/4" sieve and employed a 6-inch diameter Proctor mold. Method "D" was chosen for two reasons. First, the mold size

(6" diameter) was the same size as that used in the compaction permeameters. Secondly, larger size material (maximum of 3/4") could be used with this mold. This was thought to be a better representation of the material actually encountered in the field.

In the 6-inch mold, the soil was placed in three equal layers, each layer compacted by 56 blows of the drop hammer. After compaction of the final lift, the excess material was "struck off" flush with the top of the mold using a straightedge. After the density was determined, the soil was remixed; increasing the moisture content by 2%. The procedure is repeated until a drop in density is noted.

The same material was used throughout the tests unless a particle breakdown was observed. In these cases, material was changed each time the moisture content was changed. In samples having individual aggregate larger than 3/4", the large aggregate was removed and replaced by an equal weight of material passing a 3/4" sieve and retained on a #4 sieve. Test results were plotted on standard moisture density forms and the optimum moisture content and maximum density were obtained from the graph. (See Figure 6)

Although there is general agreement that such "maximum" densities are less than in-situ field densities, it was felt prudent because of a lack of sufficient quantitative data in that regard, not to utilize any modified methods of achieving higher densities. This decision was further backed by two other factors:

(1) it was found that the maximum density (as per ASTM D-698, "D") could be very closely attained when preparing samples in a permeability or a CBR mold, and

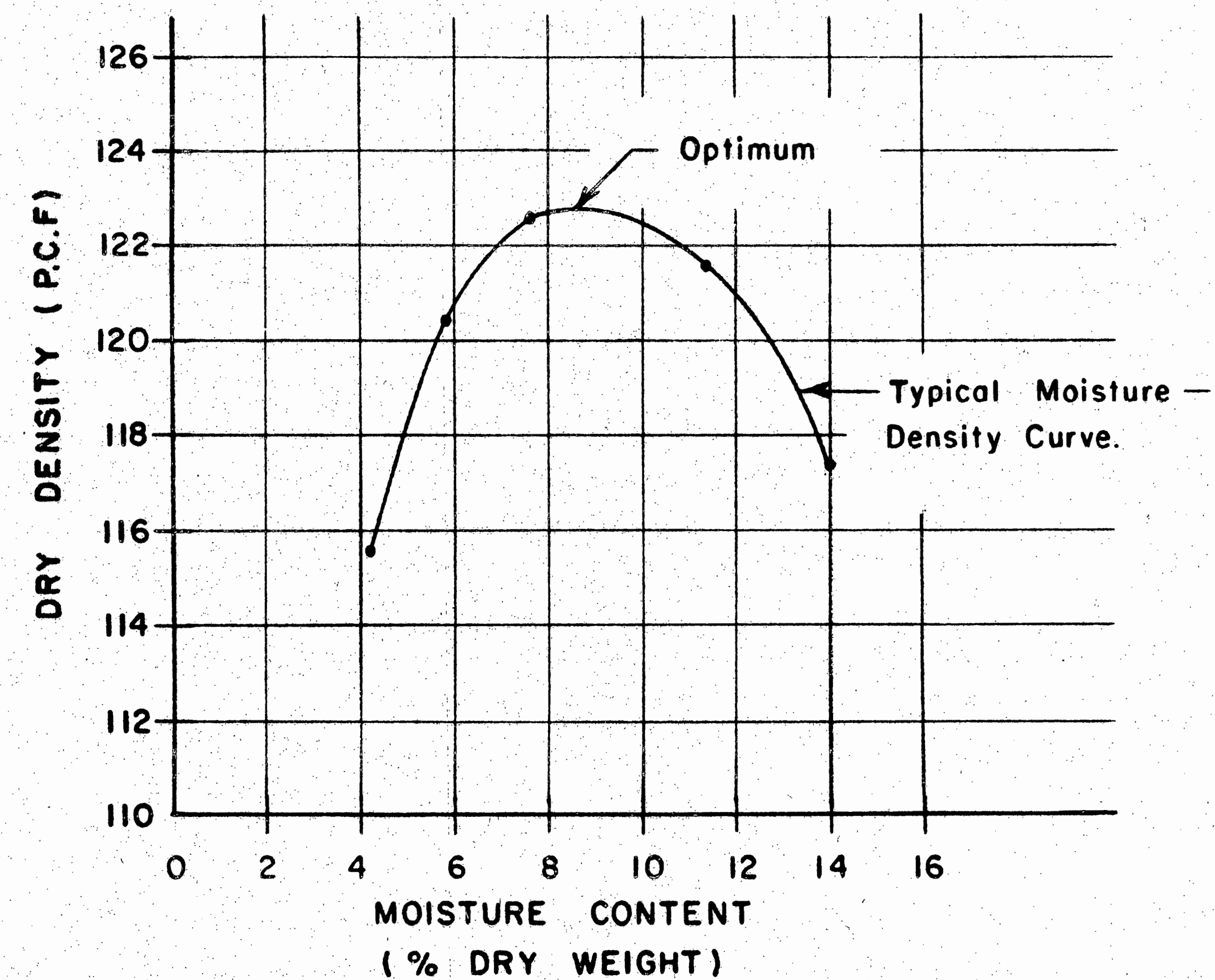


Figure 6 — OPTIMUM MOISTURE DETERMINATION BY STANDARD PROCTOR DENSIFICATION METHOD.

(2) the only other means of compaction that was available was the "modified" Proctor method and preliminary trials indicated some aggregate "breakdown" under the higher compaction effort. This was very undesirable since the amount of "fines" in a soil will influence that soil's permeability.

Particle size determination was required for all samples as a basic means of identification i.e., to determine whether or not the sample complied with the gradation requirements for the Type and Class for which it was intended, and also to determine where its gradation lay within the required band. Sieve analyses were performed in accordance with ASTM D-422 using a motorized sieve shaker.

Usually, the sieves employed were those required by the Type and Class specification, but occasionally additional sieve sizes were included. Sieves were checked for opening size compliance at six month intervals. For the soils tested, a hydrometer analysis was not thought to be necessary and samples were washed on a #200 sieve to accurately determine the amount of minus -200 material.

3. Load Bearing Capacities

Determination of load bearing capacity of the base and subbase soils was a subject of considerable controversy for a number of reasons. Although a California Bearing Ratio (CBR) test appeared to have drawbacks, it was selected because of a lack of a better, or more appropriate, test method. There are several variations of the CBR test which may be interpreted as evidence of its apparent inadequacies in a given situation. Having no prior load stability data on Types 1, 2 and 5 materials, it was opted to adhere as closely as practical to the method prescribed by ASTM D-1883.

All samples were compacted at maximum density (as per ASTM D-698,D) into 6-inch diameter molds. For each sample, four replicates were prepared. Two were tested immediately at their optimum moisture content; the other two replicates were submerged in a soaking tank for four days before testing at saturation. These samples were always instrumented and checked for swell during soaking; no swelling was ever indicated.

During all CBR tests a standard surcharge of 10 pounds was applied to the samples. Load was applied to the sample using a manually operated loading machine at the rate of 0.05 inches/minute. A load-penetration curve was obtained and the "Bearing Ratio" recorded for 0.10 inch, 0.20 inch and 0.30 inch penetrations. In the ASTM standard for CBR, the value to be reported is the one obtained at the 0.1" penetration. However, if the 0.2" penetration is greater than the 0.1" penetration, the test is to be repeated. If the results are unchanged, the 0.2" penetration is reported. It was found that all CBR tests run in this investigation had a greater value at 0.2" penetration as compared to the 0.1". Therefore, all CBR values reported are for the 0.2" penetration.

Some difficulty was encountered using the ASTM D-1883-73 method. Most notable was the tendency of certain materials (sands and clean base and subbase materials) to fall from the mold as it was inverted and reset on the base plate. This was corrected by the use of sheet polyethylene discs which were used in lieu of the required filter papers. The plastic discs reduced the tendency of the sample to adhere to the metal spacer and could easily be removed wihout disturbing the sample surface.

Figure 7 is a graph of the results of a number of CBR tests conducted on each of a Type 1A and Type 5A materials with varying moisture contents. The optimum moisture contents (for densification) for these materials were 7.5% and 9.2%, respectively. Notice that the maximum CBR values were obtained for moisture values of 5.5% and 4.5%, respectively. Therefore, it may generally be expected that a CBR result obtained at optimum moisture will provide a slightly conservative value, not considering such factors as dynamic loading during saturation, or super-saturation. The decision to run all samples at optimum moisture content provides a reasonable CBR value, i.e., one which could be quite frequently encountered in the base or subbase courses. As a matter of information, at the optimum moisture content most samples were about 90% saturated.

4. Soil Permeabilities

In New Jersey the Department of Transportation does not use permeability requirements in any aspect or phase of highway design. The Department has adopted no permeability equipment or test methods as "standard" for its use. Therefore, for the purposes of this project, it was necessary to completely develop the necessary laboratory, as well as adequate test methods.

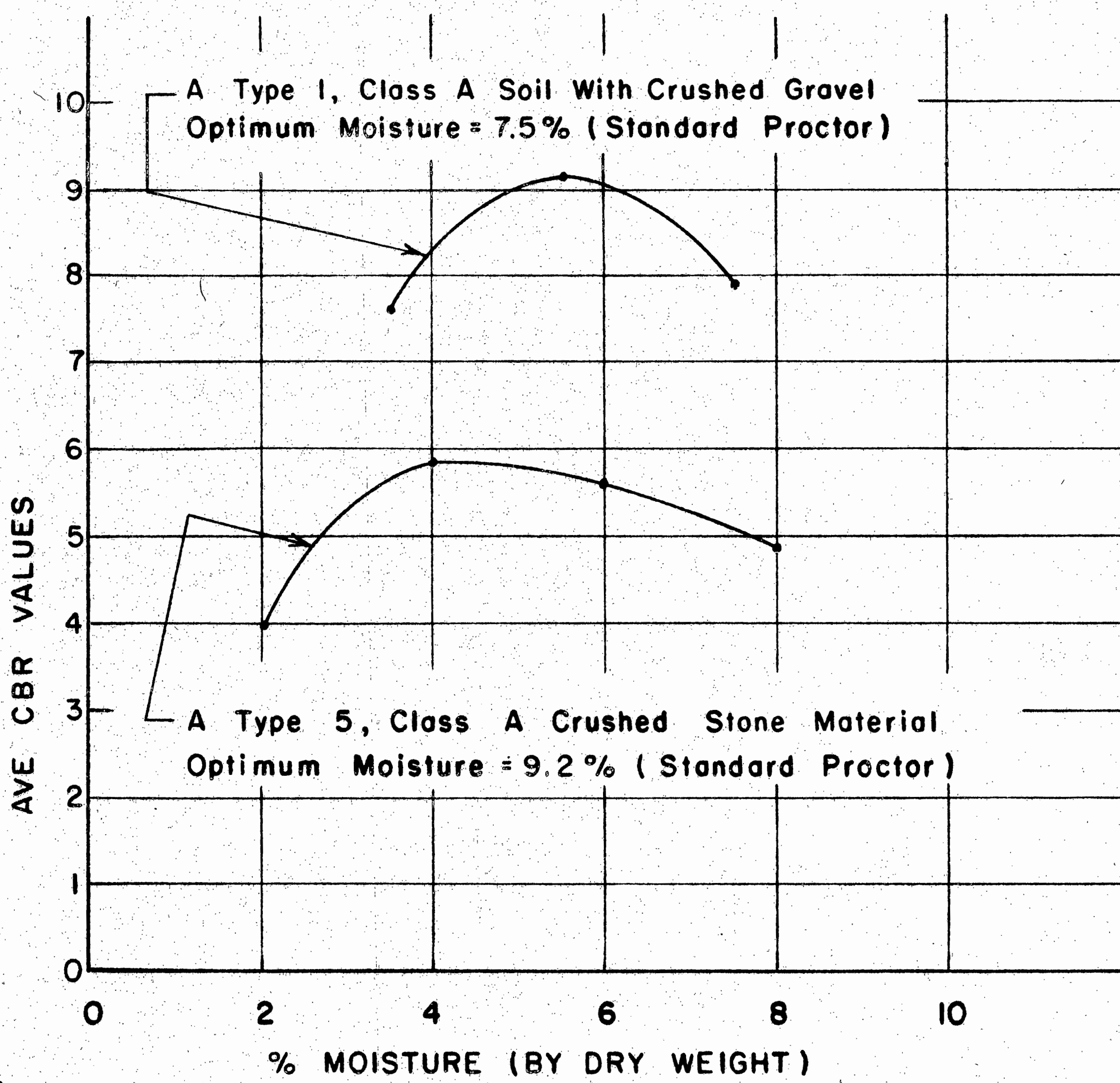


Figure 7 – EFFECT OF MOISTURE CONTENT ON CBR RESULTS.

Having no factual information regarding the permeabilities of the subject materials, the method of permeability determination was to be dependent upon the range of permeabilities obtained from the initial tests. Reference to a chart by A. Casagrande and R. E. Fadum[2] indicated that permeabilities of the subject soils may range from 10 to 10^{-6} cm/sec., particularly since it was intended to test soils meeting the "open side" of the specification bands. The same chart indicated that, both a constant head permeameter and a falling head permeameter would be required to achieve accurate results over this range. On the other hand, constant head permeability tests are supposed to yield a high degree of accuracy and repeatability when performed on soils with permeabilities of between 1 to 50,000 ft/day[3] and with less than 10% passing a #200 sieve. According to specifications, most of the soils to be tested would meet this #200 sieve requirement. Of the two methods, the constant head test requires slightly more sophisticated apparatus from which falling head apparatus is easily developed. Therefore, in the development of the laboratory, the primary emphasis was on constant head apparatus; although eventually, nearly all permeabilities were determined via the falling head method.

(4a) Constant Head Permeability Test and Apparatus

In the development of the constant head apparatus, reference was made to ASTM method of test D-2438-68, and a soil mechanics text[4] from which the theory and test requirements were obtained. The general setup is shown in Figure 8. Complete details, including theory, are provided in Appendix A, rather than here, since from the first

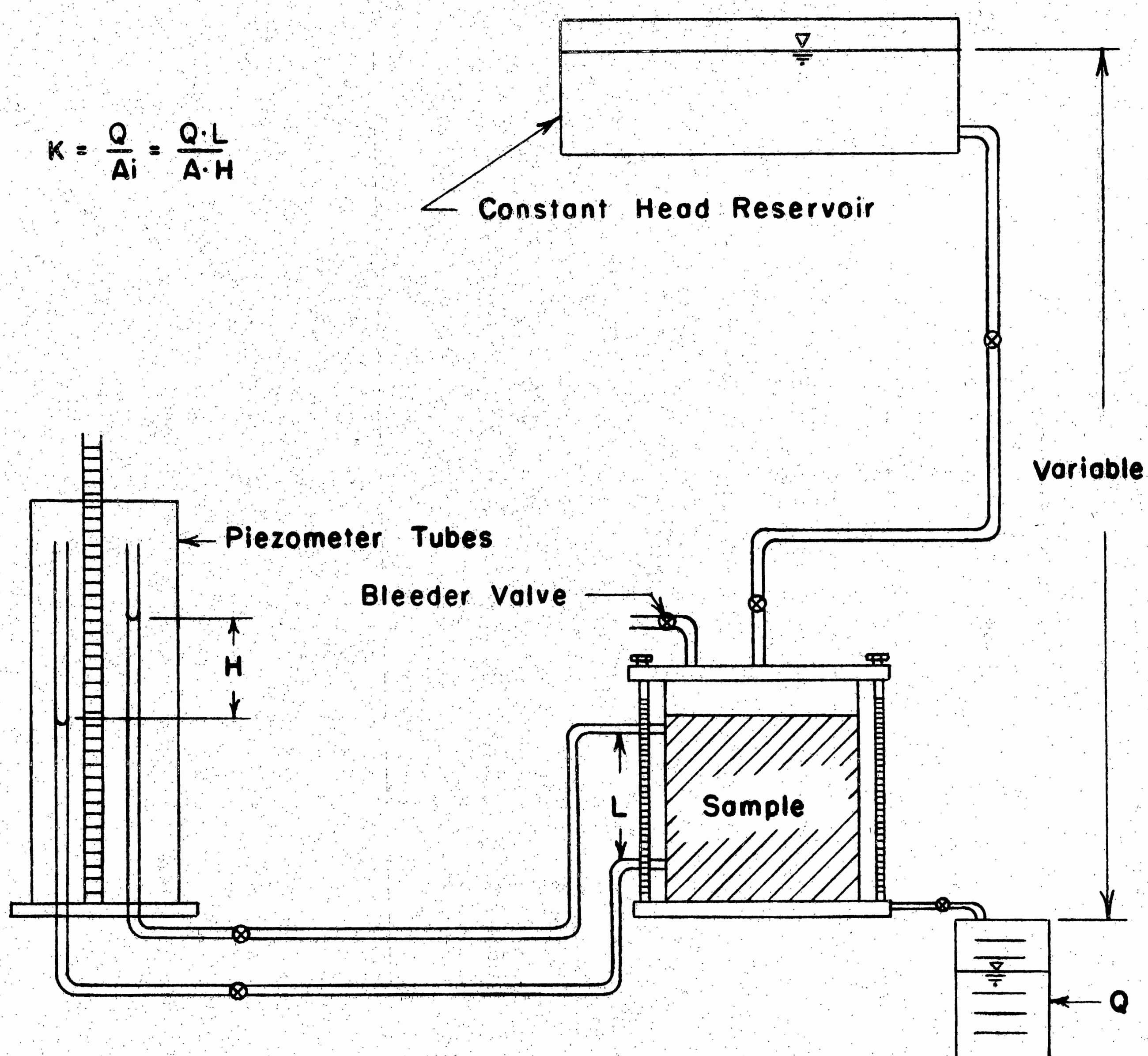


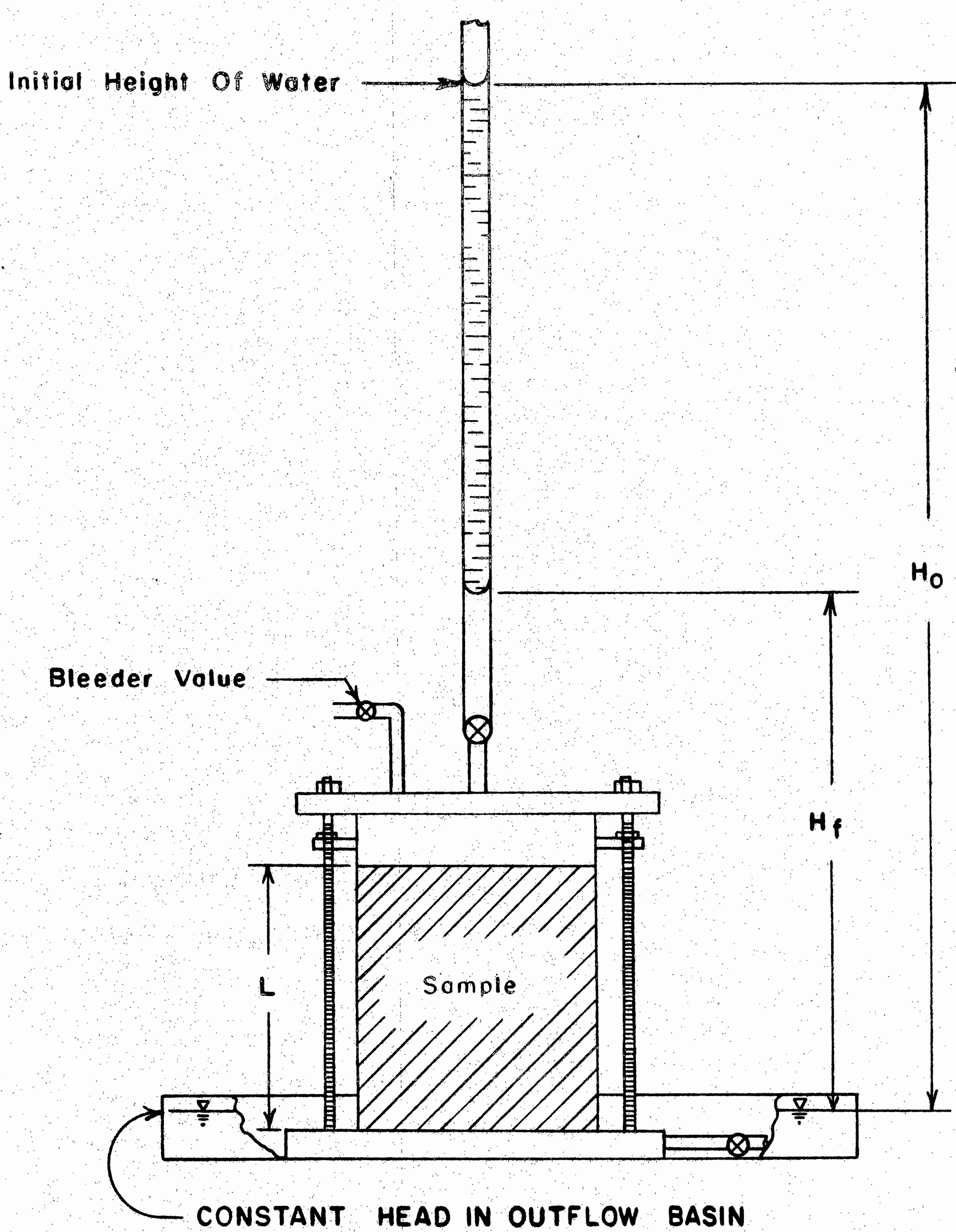
Figure 8 – CONSTANT HEAD PERMEABILITY APPARATUS (SCHEMATIC).

permeability tests conducted it became apparent that falling-head apparatus would be more appropriate for all subsequent permeability tests.

(4b) Falling Head Permeability Test and Apparatus

Figure 9 shows the falling head apparatus used in this project. The metal permeability molds were selected on the basis of long term durability and on the basis of commercial availability of the required size. A 6-inch diameter mold was selected on the basis of the ASTM D-2438 stipulation that the diameter be at least 8 to 12 times the diameter of the maximum particle size of the sample. The soil samples were to be prepared according to ASTM D-698, Method D. Thus, as cited earlier, all material retained on a 3/4" sieve was removed and replaced by an equal weight of material passing the 3/4" sieve but retained on the #4 sieve. This maximum particle size of 3/4" indicated the necessity , according to the aforementioned diameter stipulation, of a mold diameter from 6 to 9 inches. Therefore, the 6-inch diameter mold was used.

Although the 6-inch diameter metal molds were commercially available, some slight modifications were necessary. These included drilling and tapping the top cover plate to accept a 3/8" diameter water intake tube and installing a small shutoff valve at the base port. To purge the permeameter of air, a bleeder valve was also fitted directly to the top plate. The attachment of a standpipe and a head height measuring stick completed the permeameter.



$K = \frac{L}{t} \frac{a}{A} \ln \frac{H_0}{H_f}$ (t = Time For Head To Drop From H_0 To H_f).

Figure 9 — FALLING HEAD PERMEABILITY APPARATUS (SCHEMATIC).

Samples were compacted into the permeameters at optimum moisture content in five equal layers each layer being compacted by 56 blows by a 5.5 pound drop hammer. Each layer, or lift, was scarified before the next lift was added. Without scarification, there was a distinct unbound interface between layers, which could give erroneous permeability results. The top collar was then removed from the two-piece mold cylinder and the top surface of the sample was prepared by "striking off" with a steel straightedge. The collar was then replaced; a porous stone and spacer placed atop the sample; followed by placement of the top permeameter plate. The joints between cylinder components and top and base plates were all sealed with rubber gaskets.

The procedure for permeability tests requires evacuation of all air in the sample, followed by slow saturation with de-aired water from the bottom upward. In reality, however, it was established that this method of saturating was not feasible except in the most permeable base and subbase samples. Initial attempts at vacuum saturation literally tore the soil samples apart because of the pressure differential between the ends of the samples. This occurred in spite of the internal spacing collar (to hold the upper porous stone in place) and in spite of attempts to attain very slow water entry via small pressure differentials. Vacuum saturation attempts were abandoned out of necessity, and sample saturation was henceforth achieved simply by soaking in the CBR soaking tank. For this soaking method, a head of eight inches above the bottom of the sample was used and soaking was always achieved by water entering the samples through the base port and upward until free water was observed atop the sample for at least 12 hours. Some of the more dense samples required soaking times of up to seven days. Before removal from the tank, the base port was closed. The top porous stone, spring, rubber seal and top plate assembly were secured into place ready for the permeability test.

The entire assembly was placed in an outflow basin and the standpipe was filled with water. The assembly was purged of air using the bleeder valve. Then the assembly was gently tipped alternately from side to side to aid in this purging process. With this process completed, the bleeder valve was used to drop the head in the standpipe to just above the desired beginning head. This adjustment was only performed on samples with very low permeability. For samples with higher permeabilities, the standpipe was filled and the timing commenced as the standpipe water level passed the initial head height.

To standardize testing, water in the standpipe was allowed to drop from 105 cm. to 55 cm. above the level of the outflow basin for each test. Three drops were timed on each sample with a water temperature taken each time. During the initial tests, three molds were prepared for a soil sample. When it was shown that only minor differences occurred between molds, this testing was reduced to two molds per soil sample. If the permeabilities between molds differed by an arbitrary 10%, the sample was retested.

Permeability values, "K" are reported in units of feet per day and are calculated using the following formula:

$$K = \frac{2.3\ a\ L}{A\ (t_1 - t_0)} \times \log_{10} \frac{h_o}{h_f}$$

where $t_1 - t_0$ is the time required for the test head to drop in the standpipe from h_0 to h_f

a = cross sectional area of the glass tube

L = length of the specimen

A = cross sectional area of the specimen

In permeability calculations there is a correction which accounts for the change in water viscosity due to temperature changes. Permeability values are normally reported at 20°C. The following formula provides for temperature correction:

$$K_{20} = k_t \frac{u_t}{u_{20}}$$

where

K_{20} = Permeability at 20°C

K_t = Permeability at temperature, t

u_t = Viscosity of water at temperature, t*

u_{20} = Viscosity of material 20°C

*The ratio u_t/u_{20} is based on data from "International Critical Tables" Vol. V.[2]

Some tests were run to check the accuracy of this formula. Permeability tests were performed on soil samples with an initial temperature of 40°F. Subsequently, permeability tests were performed on the same samples at increasing temperatures to a maximum of 90°F, and the correction factor was applied to the results. The corrected values agreed with the calculated results very closely, as shown in Table 3. In all subsequent testing the above formula was applied, and all values of "K" reported hereinafter are for a water temperature of 20°C(68°F).

TABLE 3

CHECKS OF TEMPERATURE-VISCOSITY CORRECTION FACTORS

Sample	Temp.°F	Temp. Correction	Meas."K"(ft/day)	Adjusted "K"(ft/day)
Concrete Sand	40	1.53	5.44	8.32
	68	1.00	8.27	8.27
	90	0.76	10.79	8.20
1C-6	40	1.53	3.63	5.55
	68	1.00	5.51	5.51
	90	0.76	7.37	5.60
1A-3	40	1.53	.024	.036
	68	1.00	.038	.038
	90	0.76	.044	.033

V. Results and Discussion

A. General

Tables 4 through 9 group the test results of all samples by specification as follows:

Table 4	Type 1, Class A
Table 5	Type 5, Class A
Table 6	Type 1, Class C
Table 7	Type 1, Class B
Table 8	Type 2, Class A
Table 9	Type 2, Class B

Within a table, samples are grouped by source. From these tables, several interesting comparisons are investigated.

B. Effects of "Blending"

Blending is a method of combining a bank-run soil aggregate material with some predetermined size of aggregate for the purpose of meeting the gradation requirements for the intended material usage. For instance, for a particular construction project, the Contractor may be capable of obtaining a nearby source for Type 1, Class A base course material. However, this bank-run source may be running around 9% passing the #200 sieve (7% maximum allowed). Rather than use an alternate source several miles away from the job, the Contractor may find it cheaper to ship in 3/4 inch size crushed stone and mix it with the material from the nearby source. In terms of reducing the percentage by weight of material passing the #200 sieve, this practice appears to be sensible, and has become quite common. Yet the effects of blending upon the drainage and load support characteristics of the soils were essentially unknown. Table 10 provides a comparison of CBR values, "K" (Permeability) values, and Proctor dry densities for bank-run soils with and without blending.

Table 4 - TEST RESULTS OF TYPE 1, CLASS A SOIL AGGREGATE

					CBR (at .2" penetration)		Gradation (% Passing Square Mesh Sieve Sizes)					
Sample Number	Location	Perm. "K" ft/day	Dry Dens. (pcf)	Opt. M.C. %	Soaked	Unsoaked	2" 100-70	3/4" 95-50	#4 60-30	#50 25-5	#200 7-0	Comments
1A-1	Sayerville, NJ	0.019	129.9	8.7	--	0.45	100	95.0	59.3	14.4	5.9	Blend (29% Stone Added)
1A-2	Sayerville, NJ	0.058	133.3	9.0	0.65	0.65	100	95.0	45.0	10.8	4.3	Blend (43% Stone Added)
1A-3	Sayerville, NJ	0.038	118.6	11.0	0.16	0.17	100	95.6	88.3	21.4	8.8	
1A-4	--	--	--	--	--	--	--	--	--	--	--	Did not meet gradation
1A-5	Ogdensburg, NJ	0.570	129.4	8.8	0.58	0.76	100	93.8	58.2	18.7	4.8	Blend (35% Stone Added) (Treated as Non-blended
1A-6	Ogdensburg, NJ	0.260	131.0	7.8	0.61	0.66	100	95.3	57.8	20.1	6.1	Blend (33% Stone Added) (Treated as Non-blended
1A-7	G. S. Stone & Gravel	0.00043	126.4	11.0	0.32	0.28	100	86.6	59.0	10.7	5.1	Blend (5% Stone Added) (Treated as Non-blended
1A-8	G. S. Stone & Gravel	0.0021	132.0	9.0	0.40	0.22	100	93.4	58.8	13.7	7.6	
1A-9	Flanders S&G	0.0461	126.3	9.1	0.57	0.62	100	87.8	74.4	31.2	14.3	Failed - not used in CB
1A-10	Flanders S&G	0.342	119.5	11.8	0.29	0.48	100	64.0	58.7	24.7	7.5	Blend (24% Stone Added)
1A-11	Flanders S&G	2.130	106.7	13.6	0.29	0.29	100	64.1	59.0	27.2	3.5	Blend (35% Stone Added)
1A-12	Houdaille	0.250	132.2	7.5	0.91	0.79	100	93.9	58.1	17.7	4.3	

Table 5 - TEST RESULTS OF TYPE 5, CLASS A BASE COURSE

Sample Number	Location	Perm. "K" ft/day	Dry Dens. (pcf)	Opt. M.C. %	CBR (at .2" Penetration) Soaked	CBR (at .2" Penetration) Unsoaked	Gradation (% Passing Square Mesh Sieve Sizes) 2" 100	3/4" 90-55	#4 60-25	#50 25-5	#200 12-3	Comments
5A-1	Pennington TR	0.0086	135.9	11.1	0.76	0.59	100	83.1	55.1	16.8	8.9	
5A-2	W. Trenton Yd.	0.00438	137.8	9.2	0.48	0.49	100	81.5	50.6	16.4	9.3	
5A-3	W. Trenton Yd.	0.0103	144.3	9.2	0.51	0.40	100	92.9	51.1	20.0	12.9	
5A-4	W. Trenton Yd.	0.0056	139.9	10.2	0.44	0.39	100	74.3	40.9	16.0	10.3	
5A-5	Norristown, Pa.	0.0123	144.9	6.9	0.68	0.58	100	85.3	40.5	17.0	11.7	
5A-6	Norristown, Pa.	9.7	135.9	4.8	0.86	0.97	100	60.0	30.0	10.0	4.0	Blend (25% Stone Added)
5A-7	Mt. Hope	1200	125.0	--	--	--	100	70.0	21.3	5.7	3.0	(Falling Head)
5A-8	New Hope, Pa.	0.366	136.3	8.0	--	--	100	80.1	54.9	14.0	8.5	

Table 6 - TEST RESULTS OF TYPE 1, CLASS C SOIL AGGREGATE

Sample Number	Location	Perm. "K" ft/day	Dry Dens. (pcf)	Opt. M.C. %	CBR (at .2" Penetration) Soaked	CBR (at .2" Penetration) Unsoaked	Gradation (% Passing Square Mesh Sieve Sizes) 2" 100	3/4" 100-60	#4 100-30	#50 35-5	#200 5-0	Comments
1C-1	Hamilton Lakes, NJ	0.228	118.0	12.1	0.20	0.28	100	93.9	76.4	12.8	3.2	
1C-2	Hamilton Lakes, NJ	0.022	125.5	9.0	--	0.19	100	94.1	71.1	14.4	3.4	
1C-3	Hamilton Lakes, NJ	0.2256	122.6	7.7	--	0.42	100	95.9	71.4	23.8	3.6	
1C-4	Newark, NJ	2.85	114.5	12.1	0.46	0.45	100	100	65.0	36.5	1.0	Blend
1C-5	Houdaille, NJ	0.46	128.4	9.7	0.45	0.42	90.8	79.7	58.0	5.8	4.4	
1C-6	Houdaille, NJ	5.50	113.5	13.1	0.27	0.24	100	97.9	91.1	21.3	3.0	Hydraulic Material
1C-7	Ogdensburg, NJ	8.76	112.3	13.7	0.20	0.24	100	90.0	84.2	23.2	2.5	
1C-8	Ogdensburg, NJ	0.54	129.4	8.8	--	--	100	98.7	62.7	20	5.0	No CBR's
1C-9	Ogdensburg, NJ	22.11	112.4	14.0	0.22	0.20	100	100	99.9	30.2	2.0	
1C-10	G. S. Stone & Gravel	0.77	119.9	12.1	0.21	0.16	100	96.7	68.9	4.9	2.1	
1C-11	G. S. Stone & Gravel	3.99	125.5	10.8	0.18	0.18	100	100	82.2	12.1	1.6	
1C-12	Newark, NJ	4.32	112.4	13.3	0.38	0.42	100	100	65.0	37.6	1.0	Blend (35% Stone Added
1C-13	Hamilton Lakes, NJ	2.71	122.0	10.7	0.16	0.18	100	90.5	71.2	10.6	1.4	

Table 6 (Continued)

Sample Number	Location	Perm. "K" ft/day	Dry Dens. (pcf)	Opt. M.C. %	CBR (at .2" Penetration) Soaked	Unsoaked	Gradation (% Passing Square Mesh Sieve Sizes) 2" 100	3/4" 100-60	#4 100-30	#50 35-5	#200 5-0	Comments
C-14	--	--	--	--	--	--	--	--	--	--	--	Not Run
C-15	Hamilton Lakes, NJ	12.57	117.8	10.5	0.11	0.11	--	--	--	--	--	
C-16	Hamilton Lakes, NJ	2.24	132.9	7.3	0.57	0.44	100	69.0	45.0	5.0	0.0	Blend
C-17	Odgensburg, NJ	15.61	109.1	11.4	0.12	0.16	100	100	98.7	27.4	2.2	All material from Odgensburg
LC-18	Odgensburg, NJ	2.545	129.2	6.7	0.60	0.67	100	100	35.0	5.0	0.0	Blend
LC-19	Odgensburg, NJ	8.21	124.2	8.3	0.43	0.40	100	100	50.0	8.0	0.0	Blend
LC-20	Newark, NJ	3.05	112	--	0.27	0.27	100	100	99.6	56.6	1.5	

Table 7 - TEST RESULTS OF TYPE 1, CLASS B SOIL AGGREGATE

					CBR (at .2" Penetration)		Gradation (% Passing Square Mesh Sieve Sizes)					
Sample Number	Location	Perm. "K" ft/day	Dry Dens. (pcf)	Opt. M.C. %	Soaked	Unsoaked	2" 100	3/4" 100-65	#4 75-40	#50 30-5	#200 7-0	Comments
1B-1	Woodmansie, NJ	.0007	128.1	9.3	0.39	0.24	100	77.4	68.9	15.0	6.4	
1B-3	Rt. 295/Slab 844	0.0016	125.8	10.9	0.38	0.39	100	95.0	71.2	22.1	7.4	
1B-4	Crosswicks, NJ	0.0039	128.6	9.0	--	0.39	100	97.7	70.8	20.1	6.1	
1B-5	Rt. 295/Slab 407	0.057	126.0	9.8	0.43	0.41	100	77.8	51.6	13.9	6.2	
1B-6	Rt. 295/Slab 1004	0.135	123.2	10.9	0.28	0.30	100	90.5	68.6	11.6	5.7	
1B-7	Rt. 295/Slab 1004	0.441	116.3	9.8	0.29	0.31	100	93.8	79.6	11.7	5.7	
1B-8	Jamesburg, NJ	0.0025	126.3	10.0	0.37	0.36	100	97.5	85.2	18.1	8.7	
1B-9	Jamesburg, NJ	0.390	121.2	11.1	0.21	0.22	100	100	88.7	11.6	4.0	
1B-10	Jamesburg, NJ	0.01195	133.8	8.9	0.89	0.65	100	65.8	57.5	12.2	5.9	Blend (33.5% Stone Added)
1B-11	Jacobstown, NJ	0.024	128.2	-	-	-	100	97.9	79.5	24.5	8.2	Original Sample
1B-12	Jacobstown, NJ	0.026	126.5	-	-	-	100	98.4	60.0	18.6	6.4	Blended from 1B-11 (20%)
1B-13	Jacobstown, NJ	0.0021	125.8	-	-	-	100	100	89.3	39.7	12.3	Original Sample
1B-14	Jacobstown, NJ	0.0084	123.0	-	-	-	100	100	60.0	23.0	7.0	Blended from 1B-13 (30%)

Table 8 - TEST RESULTS OF TYPE 2, CLASS A SOIL AGGREGATE

Sample Number	Location	Perm. "K" ft/day	Dry Dens. (pcf)	Opt. M.C. %	CBR (at .2" Penetration) Soaked	Unsoaked	Gradation (% Passing Square Mes Sieve Sizes) 2" 100	3/4" 100-70	#4 75-35	#50 30-15	#200	Comments
2A-1	Rt. 295/Slab 1004	0.036	129.7	9.8	0.36	0.36	100	93.8	70.0	18.6	9.6	
2A-2	Rt. 295/Slab 844	0.019	125.1	10.4	0.38	0.30	100	93.7	65.5	26.0	5.9	
2A-3	Rt. 295/Slab 407	0.077	119.5	12.9	0.22	0.22	100	96.0	76.4	21.0	7.8	
2A-4	Rt. 295/Slab 239	0.254	121.9	11.4	0.28	0.32	100	98.1	66.5	13.1	4.9	
2A-5	Rt. 130 (Cranbury)	.0056	132.6	9.5	0.35	0.24	100	87.5	60.3	17.4	8.5	

Table 9 - TEST RESULTS OF TYPE 2, CLASS B SOIL AGGREGATE

					CBR (at .2" penetration)		Gradation (% Passing Square Mesh Sieve Sizes)					
Sample Number	Location	Perm. "K" ft/day	Dry Dens. (pcf)	Opt. M.C. %	Soaked	Unsoaked	2" 100	3/4" 100-70	#4 80-30	#50 35-10	#200 12-4	Comments
B-1	S. Jersey Maint. Yd.	0.00168	122.4	11.8	0.27	0.18	100	98.4	81.0	19.4	11.1	Blend (15% Stone Added)
B-2	Jamesburg	.0025	126.3	10.0	0.37	0.36	100	95	85	18.1	9.0	
B-3	Woodmansie	.0007	128.1	9.3	0.39	0.24	100	77.4	68.9	15.0	6.4	
B-4	Crosswicks	.0039	128.6	9.0%	0.36	.39	100	97.7	70.8	20.1	6.1	
B-5	Rt. 130	.0017	129.4	8.5%	0.60	.54	100	96.2	71.2	18.3	7.8	
B-6	Woodmansie, NJ	.0007	126.5	11.0	.03	.02	100	68.1	62.6	21.6	11.6	Originally sampled from a 1-B Source

TABLE 10 - COMPARISON OF BLENDED AND UNBLENDED MATERIALS

Case	Type & Source	"Bank-Run" Permeability, K (ft./day)	Permeability After Blending, K (ft./day)	"Bank-Run" CBR @ Opt. Moisture	Blended CBR @ Opt. Moisture	"Bank-Run" Proctor Density (pcf)	Blended Proctor Density (pcf)
1	1-A, Pennsauken Form.	0.038	0.019	0.17	0.45	119	130
2	1-A, Pennsauken Form.	0.038	0.058	0.17	0.65	119	133
3	1-A, Carbonate Shale	0.0021	0.0004	0.22	0.28	132	126
4	1-A, Quartz-Granite	0.046	0.342	0.62	0.48	126	120
5	5-A, Carbonate	0.012	9.7	0.58	0.97	145	136
6	1-C, Coastal Dredging	3.05	2.85	0.27	0.45	112	115
7	1-C, Coastal Dredging	3.05	4.32	0.27	0.42	112	112
8	1-B, Pennsauken Form.	0.024	0.026	-	-	128	126
9	1-B, Pennsauken Form.	0.0021	0.084	-	-	126	123
10	1-B, Pennsauken Form.	0.003	0.012	0.36	0.65	126	134

Avg. CBR before blending = 0.333
Avg. CBR after blending = 0.545
Avg. difference is 0.212 or 67% increase
Avg. unblended density 125
Avg. blended density 126

For each case shown in Table 10, the blended samples were, prior to the addition of the stone, precisely the same material as the unblended samples. The summary at the bottom of Table 10 reveals that overall, there was no significant change in average density with blending. A look at the individual density changes, however, reveals that individually, from one source to another, blending may substantially affect densities. For instance, the largest increase in density due to blending was 14 pcf. For that particular case, about 50% of the measured increase was due to the higher specific gravity of the added stone itself. Therefore in terms of the effective density change, or the change that occurred due to compaction, there was an increase of about 7 pcf due to blending.

The largest decrease occurred with Case #5, the Type 5, Class A sample, where the blended sample was 9 pcf less than the original sample. In this case, the stone that was added for blending was of the same source as the original sample. Therefore, the addition of the stone for blending caused a "real" density decrease of 9 pcf.

By similar reasoning in Case #4, a quartz-granite sample, blending by addition of traprock caused an apparent sample density decrease of 6 pcf. The stone that was added in blending had a higher specific gravity than the original sample material. Therefore, the "real" density decrease due to blending is around 8 pcf. For Case #3 the amount of stone added for blending was only 5% by weight. Therefore, the 6 pcf decrease is not adjusted and may be considered the "real" effect of blending.

Each of the aforementioned cases is from a different geological and mineralogical source. In Table 10, however there are five cases

(#1, 2, 8, 9, 10) which came from the same geological origin (Pennsauken Formation) although they came from 3 different pits. The 1-A samples from the first pit had an effective density increase of 5-7 pcf. Cases #8 and #9 from a second pit, had an effective density decrease of about 5 or 6 pcf. And Case #10 from a third pit had an effective density increase of around 2 or 3 pcf. Therefore, even materials that are geologically similar may demonstrate dissimilar density changes from blending.

In comparing CBR values, the summary at the bottom of Table 10 is representative of all but Case #4 shown therein. In general, the CBR values increased substantially with blending. How significant this is with regard to realistic load support of pavements and subsequent increases in pavement life is unknown.

It is interesting to note that all of the bank-run soils that had CBR increases with blending, originally had relatively low CBR values. The one sample (Case #4) that had a CBR decrease with blending, originally had a high CBR compared with the other samples of 1A material listed in Table 10. Although there are certainly not enough data on which to base absolute, sound conclusions regarding the affects of blending, there is the indication that blending with crushed stone will improve the strength of materials having very low CBR values, while the affect of blending upon materials having higher CBR values (from about 0.50 and higher) is very questionable.

Of primary importance in this project is the investigation of the permeabilities of the various base and subbase materials. During the course of the research a situation presented itself wherein a new PCC pavement, constructed on a 1B base course, exhibited substantial "pumping". A review of the material sampling and testing records showed that the base course had been blended to meet the gradation requirements. The performance of the base course raised a serious question pertaining to the affects of the blending. Samples taken by the researchers directly from the source of the material, revealed that the material was high in "minus #200" material. These samples are represented by Cases #8 and #9 in Table 10. Although the permeabilities (k) of these two cases varied by one order of magnitude, blending did not cause an appreciable change in K-values in either case. Of the ten cases shown in Table 10, only three show a change in measured K of one or more orders of magnitude.

Case #3 exhibited a decrease in permeability with blending. However, a separate set of tests conducted on this general sample of shale material revealed that substantial aggregate "breakdown" could occur by the drop hammer densification method. In all such tests, the minimum increase in minus -#200 material was 3% by weight. It is probable, therefore, that the observed drop in "K" was largely due to this factor.

In Case #4, the sample was visually noted during compaction to have lost stability with blending. This observation was verified by the CBR decrease as well as the drop in density. Therefore, the increase in "K" would appear to be a logical result of the blending.

Case #5 is a quarry processed material, designated as "5A". Based on other tests of this kind of material in related work but not specifically included within this study, the large increase in permeability, K, of this sample probably occurred because the material had been transformed into a "so-called" open-graded sample, i.e., larger size aggregate has been added to an extent sufficient to cause voids within the sample that can no longer be filled by the remaining finer aggregate sizes. This explanation also accounts for the large drop in density and the large increase in CBR value. Actually, at this point in such materials, the CBR test itself is probably inappropriate since a confined sample of this material can scarcely deform without crushing of the aggregate.

With the exception of the aforementioned three cases, "blending" appeared to make no change in the drainage characteristics, from those exhibited in the natural materials. Therefore, in summarizing Table 10, "Comparison of Blended and Unblended Materials," it may be stated that in general, blending has little effect upon the permeability of a material. If blending is used for the purpose of artificially reducing the percentage of minus -#200 material, the net effect is to allow the use of natural material, which even when blended, will be practically undrainable.

Furthermore, blending appears to substantially improve CBR values of soils having low CBR values in their natural state. These results are based on samples which are confined in a CBR mold. Just how well these results translate into reality, that is how closely they represent the behavior of the same materials in their semi-confined position within a pavement section, is questionable. It is noteworthy that the Department has used blending as a means of stabilizing soil base materials that met gradation requirements of the specifications. A recent case in point is on Route I-195, Section 6B, where Type 2B was placed. The material had little, or no unconfined stability under load. Blending improved this soil such that its replacement was not necessary. At this writing, no "before-and-after" CBR's are available on the above mentioned 2B material. But the fact that there is general agreement between laboratory and field experience implies that a fairly inexpensive load bearing test and load bearing requirements could be used to avoid, or at least to satisfactorily resolve, the disputes which arise over an "acceptable" material (by specification) that is not really acceptable.

If any increased CBR value is to be used (in a design analysis) for a blended material, it will be necessary to conduct appropriate tests using the actual materials that are to be utilized for the particular construction job. Particularly in northern New Jersey, the geological make-up is such that the soil composition and engineering properties can vary greatly over small distances within a source pit. Therefore, the importance of adequate and representative sampling and testing cannot be over-emphasized. Finally, the practice of blending can lead to large and important errors in any construction phase where density control is important. "Target" densities should always be established on the basis of the results of the blended material.

C. Load Bearing Characteristics

Tables 4 through 9 present two CBR values per sample taken at each of the optimum moisture content, and the soaked-saturated condition for each sample. These values represent, in the opinion of the authors, only a gross indication of the relative strengths of the various materials tested. In fact, during the planning stages of the project, it was never intended that the CBR results would be used for a more "refined" analysis with CBR data. Hence at that time, it was decided to perform one of the CBR tests at the optimum moisture content, on the basis of two main thoughts: 1. this water content would allow us to achieve maximum densification, somewhat representative of "field conditions" for a newly constructed highway, and 2. it was estimated that this moisture content would be generally representative of the moisture content in base and subbase courses at almost any time after construction of the highway. Tests were also conducted on soaked samples.

For most of the samples the permeabilities were so low that they took at least a day to become saturated in the soaking tank. For these samples, it is safe to say that a CBR test was conducted in a saturated condition. This saturated condition (or worse) is representative of the "field conditions" of some base courses immediately beneath PCC pavement slabs following rain storms. Other more permeable samples were tested in a soaked condition. That is to say that some drainage occurred immediately upon removal from the soaking tank and during the CBR test.

Table 11 summarizes the ranges and means for the soaked CBR values and the unsoaked CBR values for each of Tables 4 through 9. The mean, unsoaked CBR values for non-blended samples are also given. A few interesting aspects surface as one examines Table 11.

First, it is interesting to note that for each Type and Class of material, there is no significant difference between soaked and unsoaked CBR values. Scanning through any of Tables 4 through 9 shows that even on a sample by sample basis there are usually only small differences between soaked and unsoaked CBR values. Further, differences shown are not consistently higher or lower. The reason for this "closeness" of soaked and unsoaked values probably lies

Table 11 - SUMMARY OF CBR VALUES

Type and Class	General Use of Material	CBR Value at 0.2" Penetration, 10# Surcharge		
		Soaked Or Saturated	Unsoaked All Samples (Range)	Unsoaked Non-blended (Range)
1-A	Base Course Beneath PCC	0.48 (10 samples)	0.49 (0.17 - 0.79) (11 samples)	0.43 (0.17 - 0.79) (6 samples)
5-A	Base Course Beneath Bit. Concrete	0.62 (6 samples)	0.58 (0.39 - 0.97) (6 samples)	0.49 (0.39 - 0.59) (5 samples)
1-C	Subbase Beneath 1-A or 5-A	0.30 (16 samples)	0.30 (0.11 - 0.67) (16 samples)	0.23 (0.11 - 0.42) (13 samples)
1-B	Base and Subbase Combined	0.41 (8 samples)	0.36 (0.24 - 0.65) (9 samples)	0.33 (0.24 - 0.41) (8 samples)
2-(A&B)	Base Course	0.33 (11 samples)	0.29 (0.02 - 0.54) (11 samples)	0.30 (0.02 - 0.54) (10 samples)

in the fact that in general, for all of these soils, saturation is only 2% - 3% more water than optimum, i.e. if the optimum m.c. is 10% by dry weight, then saturation is usually reached at about 12% or 13%. Of course, this trend was not apparent until substantial testing on various Types and Classes had been performed. Nevertheless, the results obtained are significant as will be discussed hereinafter.

Upon further examination of Table 11, it is interesting to compare the relative strengths of the various Type and Class materials, particularly in view of their intended use. Type 5, Class A material is a crushed stone product. It is, by comparison to all the other materials, substantially stronger in terms of CBR values (0.58/0.49). Considering its origin and nature, these results are expected. Its use in the pavement section is to serve as a strong base upon which approximately a 9-inch thick bituminous pavement is usually placed. Of the six materials used for N.J. base and subbase, this is the strongest and therefore the one to use, barring any considerations of water problems.

Type 1, Class A (1-A) had a mean CBR of 0.49 (including blended samples) which lies about midway between the CBR's for 5-A and 1-C, a subbase material. This fits nicely into the pattern of design assumptions, i.e. that 1-A serves as a strong base, whereas 1-C has less strength (CBR = 0.30) and is used deeper within the pavement section. However, because of difficulties in obtaining 1-A material that actually met the gradation requirements, many of the 1-A samples were blended material. Samples 1A-5 and 1A-6 were taken from an old pit of glacial till that was no longer used as a 1-A source. It was found that the only way to obtain material meeting 1-A gradation was to blend the material from points within the pit. Since this pit had been used as a large 1-A source at one time, it was desired to make the 1-A samples entirely from the native material. Therefore, these samples, although listed as blended, are treated as non-blended. Because they were artificially made with their gradations so similar, the CBR results were averaged and taken as one CBR for computational purposes. With this consideration, the average CBR for unsoaked, non-blended 1-A material is seen as 0.43 which again falls nicely between the expected higher CBR for 5-A (0.49) and the anticipated lower CBR of 0.23 for 1-C materials.

The range of CBR values for 1-A material is somewhat revealing in that the lower end of the range is well into the range of values to be expected for 1-C materials. In other words, although it is intended that a 1-A material should be a stronger course than 1-C, there is absolutely no guarantee that this will be the case, through the application of the current specification.

Type 1, Class B (1-B) material is basically a southern New Jersey substitute for 1-A soils beneath PCC pavement. It is also used as a subbase sometimes beneath a 2-B base course. The mean CBR values lie about midway between those of 1-A and 1-C soils which is in general agreement with design assumptions. In addition, the range of CBR values (0.17, non-blended) is small in comparison to that of 1-A (0.62 non-blended) thereby implying that overall, the 1-B specification may at times be assuring a better material than the 1-A requirements. However, Figure 1 shows that the gradation bands for Type 1-A and

and Type 1-B soils, while shifted slightly from one another, are of about equal band width. It would therefore not be expected that the 1-B specification would assure better material, or exhibit less variability in CBR strength, than the 1-A specification.

In the authors' opinion, the small range of CBR values of the 1-B samples is due to the fact that, although the samples were selected from various sources, there is little change in geological-mineralogical makeup amongst these sources, since they are all in southern New Jersey (Refer to Figure 5). In contrast, the 1-A samples were selected from sources throughout northern New Jersey and thus represent materials of vastly differing geological-mineralogical origin. Therefore, at least for New Jersey, where the geological origin of materials varies substantially, a specification based primarily upon gradation characteristics is inherently weak with regard to load support characteristics.

Type 2 material is used where solid support is desired. It is used as a thick base beneath thin bituminous shoulders and as a base for bituminous pavement where 5A is prohibitively expensive or where lighter traffic is expected. Although 2-A is so scarce that it is no longer specified, some samples were obtained from highways already constructed. For practical purposes the results of Tables 8 and 9 may be considered together, particularly since 4 of the 5 samples of 2-A came from the same job.

Table 11 shows CBR means of 0.29 and 0.30 for the Type 2 materials. While the non-blended CBR of 0.30 is better than that of 1-C, it is not superior to that of 1-B. Densities are very comparable to 1-B samples. There apparently is no basis to assume that Type 2 material will provide more density or support than 1-B soils or than blended 1-C soils.

Of importance is the range for Type 2 samples. The highest value of 0.54 was obtained on a sample taken from a 30 year old PCC pavement. It was purposely selected because this particular pavement has performed (and still performs) excellently. By comparison the highest value of all other Type 2 samples was only 0.39.

On the other hand, the lowest CBR of the Type 2 range is 0.02. Because of the apparent abnormality of this value, this particular sample was checked and re-checked. Agreement in results was extremely good and the results must be accepted as valid. Of all of the non-blended samples, this sample (2B-6) possesses, by far, the most minus -#200 material. This sample also had one of the lowest percentages of material passing the #4 sieve. Records indicate substantial sloppiness of this sample at its optimum moisture content. While this explains the very low CBR value, it also serves as evidence of weakness in the specification. As hereinbefore mentioned, it is noteworthy that cases of extreme instability of 2-B material on construction jobs have occurred.

D. <u>Drainage Characteristics</u>

As stated in the introduction, the center of interest of this work is upon the drainage capacity of a material in <u>relation</u> to the requirements placed upon that material by the highway design and highway loads. Before getting into this discussion, however, a few other pertinent matters are presented.

1. Permeability Data

Data for all of the samples are presented in Table 4 through 9. A review of this data shows that in general the lowest permeabilities are found in Type 1B and Type 2 materials, and the highest are found in the 1C materials. Within each Type 1 Class, the permeabilities may vary from sample to sample by a few orders of magnitude. The general findings mentioned are somewhat consistent with the Department's design assumptions that the base courses are a little stronger, but a little less permeable than a subbase (1C). Type 1B material, which is sometimes used as both the base and subbase offers only moderate strength, combined with generally very poor drainage. A more detailed analysis of the data is presented later herein.

2. "Open" Gradations

The effects of blending upon permeability may or may not be favorable regarding drainage. For the most part, however, blending has a negligable effect upon permeability because the soils that are blended, initially have very fine gradations and consequently have very low permeabilities. It is possible to add enough stone of various sizes to eventually "open up" a sample, but this would require considerable laboratory testing and expense. During the course of this research, some soils were separated by sieving, and then recombined at predetermined percentages for specific sizes of the material. This was done to provide insight to the gradation specifications. Specifically, it was desired to make samples that followed the open side of the specification gradation band. While this is not really the same as blending as it is done in construction, these "artifical" samples represent the extremes that would develop if enough stone is added in various sizes to open a sample.

Several of the samples made in this manner are not included in the data tables because they were so unstable. These samples would never be encountered as a natural source of material and in that sense, the gradation band is excessively liberal on the lower limit of the various sieves. Because of their instability, it was felt that they would be totally impractical for use and no further tests were conducted. However, the key factor in this instability is that these samples were comprised only of the particular native material in recombined form. Since these particles are all fairly well-rounded, good stability can hardly be expected.

In contrast to these open materials, sample number 5A-7 (Table 5) represents the same concept of opening up a sample, but with 100% crushed stone. The effect in that case was to produce a material with a permeability of several hundred feet per day. The CBR test could not be conducted but would obviously be quite high. Sample #5A-6 represents the same affect, but not quite to the extreme of Sample #5A-7.

Therefore, it is seen that the current gradation specifications are liberal enough to allow substantial variation in permeability and stability; but, the area of the band where these gross changes occur, i.e. the lower limits of the bands, have no corresponding representative materials in use, either naturally occurring or man-made. The effects on stability of combining a natural soil with a crushed aggregate are hereinbefore documented, and there is a good possibility that the combination of crushed stone and a lesser portion of natural soil aggregate could provide a well-graded material with both strength and good drainage properties.

3. Prediction of permeability

A scan of each of Tables 4 through 9 shows clearly that only in the grossest terms could the permeability of a base or subbase be predicted, given only the gradation requirements of the Type and Class of soil to be used. In fact, given a set gradation for a number of different soil sources it would still be difficult to predict permeability.

Methods for predicting soil permeability are usually based on the densities and the particle size analyses to one extent or another. Characterization of filtration sand and gravels by the D_{10} size was attempted by Hazen as early as 1893.(5) One of the latest works in the field is that of Moulton,(6) in which a nomograph is presented for the estimation of permeability based on the D_{10} particle size, the dry density, γ_d, and the percentage by weight of material passing the No. 200 sieve, P_{200}. In Moulton's nomograph, the dry density, γ_d, is actually a convenient substitution for the sample porosity, n. The nomograph is represented mathematically by the following equation:

$$K_{Moulton} = \frac{6.214 \times 10^5 \, (D_{10})^{1.478} \, (n)^{6.654}}{(P_{200})^{0.596}} \qquad \text{(Eqn. \#1)}$$

To achieve maximum accuracy, the theoretical permeabilities for soil samples of this research were calculated via the above expression using the dry density, rather than the porosity.

Table 12 is a listing of the results and provides a comparison to the corresponding laboratory measured permeabilities. In Table 12, samples were grouped roughly according to their P_{200} amounts. Figure 10 is a plot of the logarithm of the measured permeabilities versus the logarithm of the calculated permeabilities whereby it is clear that the predicted values are consistently higher than measured values. Also, the accuracy of the expression decreases considerably as P_{200} increases. Generally, for $P_{200} \geq 6\%$, the predicted "K"-values are about 2 orders of magnitude too high. Because of this discrepancy, permeabilities were re-calculated using the actual porosities for those samples for which the specific gravity was accurately known. The results are plotted in Figure 11, whereon the dashed line indicates where the points should lie if there is one-to-one agreement between the calculated and measured results. Again, the predicted values are higher than the measured permeabilities.

As a matter of interest, a multiple regression analysis was performed, using only the test results from this research and the same basic expression of K: $C(D_{10})^x(n)^y/(P_{200})^z$.

The equation obtained is as follows:

$$K = \frac{1.633\text{x}10^7(n)^{13.37}}{(D_{10})^{0.092}(P_{200})^{1.347}} \qquad \text{(Eqn. \#2)}$$

This new expression is interesting because it appears to indicate that the D_{10} quantity is an insignificant factor in determining permeability, since with the small exponent, the quantity approaches a value of unity. In reality, this is not thought to be the case for all soil gradations. The reason that this has occurred, is that for all of the samples tested in this research, there was simply very little variation in

Table 12 - COMPARISON OF PREDICTED PERMEABILITIES (MOULTON) TO MEASURED PERMEABILITIES

Sample No.	D_{10} (mm)	P_{200}	γ_{dry} (pcf)	Measured Permeability (ft/day)	Calculated Permeability (Moulton, ft/day)
1C-13	0.31	1.4	122.0	2.71	17.0
1C-18	0.5	0	129.2	2.54	57.0
1C-17	0.12	2.2	109.1	15.6	9.93
1C-20	0.095	1.5	113.0	3.05	5.65
1C-19	0.35	1.0	124.2	8.21	11.1
1C-9	0.12	2.0	112.4	22.11	7.27
1C-10	0.38	2.1	120.0	0.77	14.75
1C-12	0.11	1.0	112.4	4.32	9.66
1C-11	0.19	1.6	125.5	3.99	2.78
1C-1	0.21	3.2	118.0	0.228	6.26
1C-3	0.12	3.6	122.6	0.226	1.34
1C-5	0.30	4.4	128.4	0.46	1.9
1C-8	0.13	5.0	129.4	0.54	0.427
1C-2	0.18	3.4	125.5	0.022	2.3
1A-7	0.30	5.1	126.4	0.0004	5.0
1A-5	0.11	4.7	129.4	0.57	0.5
2A-4	0.20	4.9	121.9	0.254	4.1
1C-7	0.13	2.5	112.3	8.76	11.0
1C-6	0.13	3.0	113.5	5.5	7.5
2A-1	0.10	9.6	129.7	0.036	0.29
2A-2	0.12	5.9	125.1	0.019	1.0
2A-3	0.10	7.8	119.5	0.077	1.5

Table 12 - (Continued)

Sample No.	D_{10} (mm)	P_{200}	γ dry (pcf)	Measured Permeability (ft/day)	Calculated Permeability (Moulton, ft/day)
2A-5	0.11	8.5	132.6	0.0056	0.2
1B-1	0.16	6.4	128.1	0.0007	0.78
1B-3	0.10	7.4	125.8	0.0016	0.55
1B-4	0.22	6.1	128.6	0.0039	1.4
1B-5	0.13	6.2	126.0	0.057	1.2
1B-6	0.23	5.7	123.2	0.135	4.0
1A-3	0.09	8.8	118.6	0.038	1.4
1A-6	0.11	9.1	131.0	0.26	0.30
1A-8	0.15	7.6	132.0	0.0022	0.39
1A-9	0.05	14.3	126.3	0.0461	0.015
1B-7	0.22	5.7	116.3	0.441	9.5
1B-8	0.10	8.7	126.3	0.0025	0.55
1B-9	0.24	11.1	121.2	0.390	2.9
1B-10	0.20	5.9	133.8	0.012	0.6
2B-2	0.08	9.0	126.3	0.0025	0.41
2B-3	0.15	6.4	128.1	0.0007	0.81
2B-4	0.13	6.1	128.6	0.004	0.52
2B-5	0.12	7.8	129.4	0.0017	0.49
5A-1	0.10	8.9	135.9	0.0086	0.08
5A-2	0.10	9.3	137.8	0.0044	0.06
5A-4	0.06	10.3	139.9	0.0056	0.01
5A-8	0.11	8.5	136.3	0.366	0.09

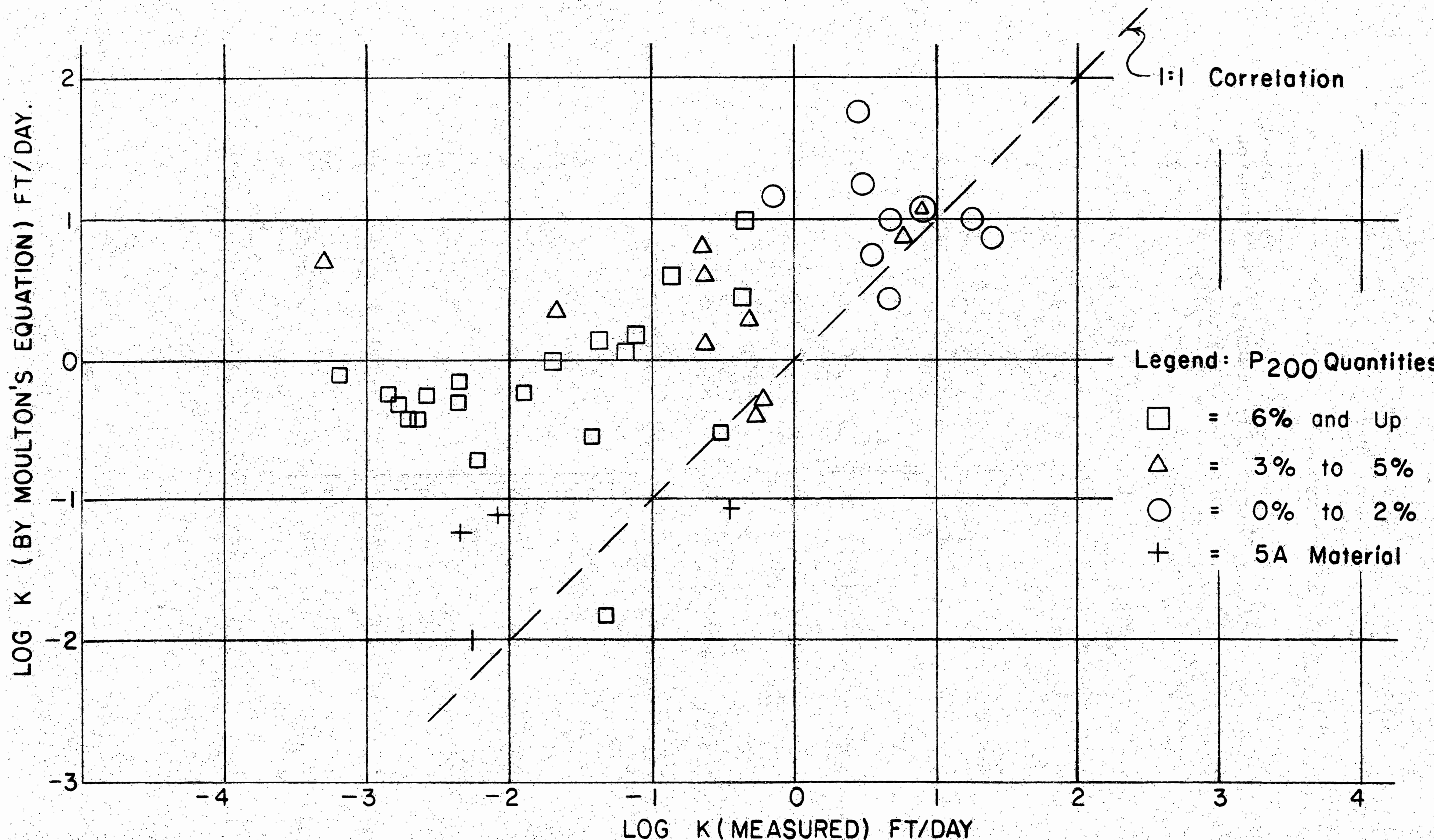


Figure 10 – PERMEABILITIES: CALCULATED VERSUS MEASURED, N.J.D.O.T. MATERIALS.

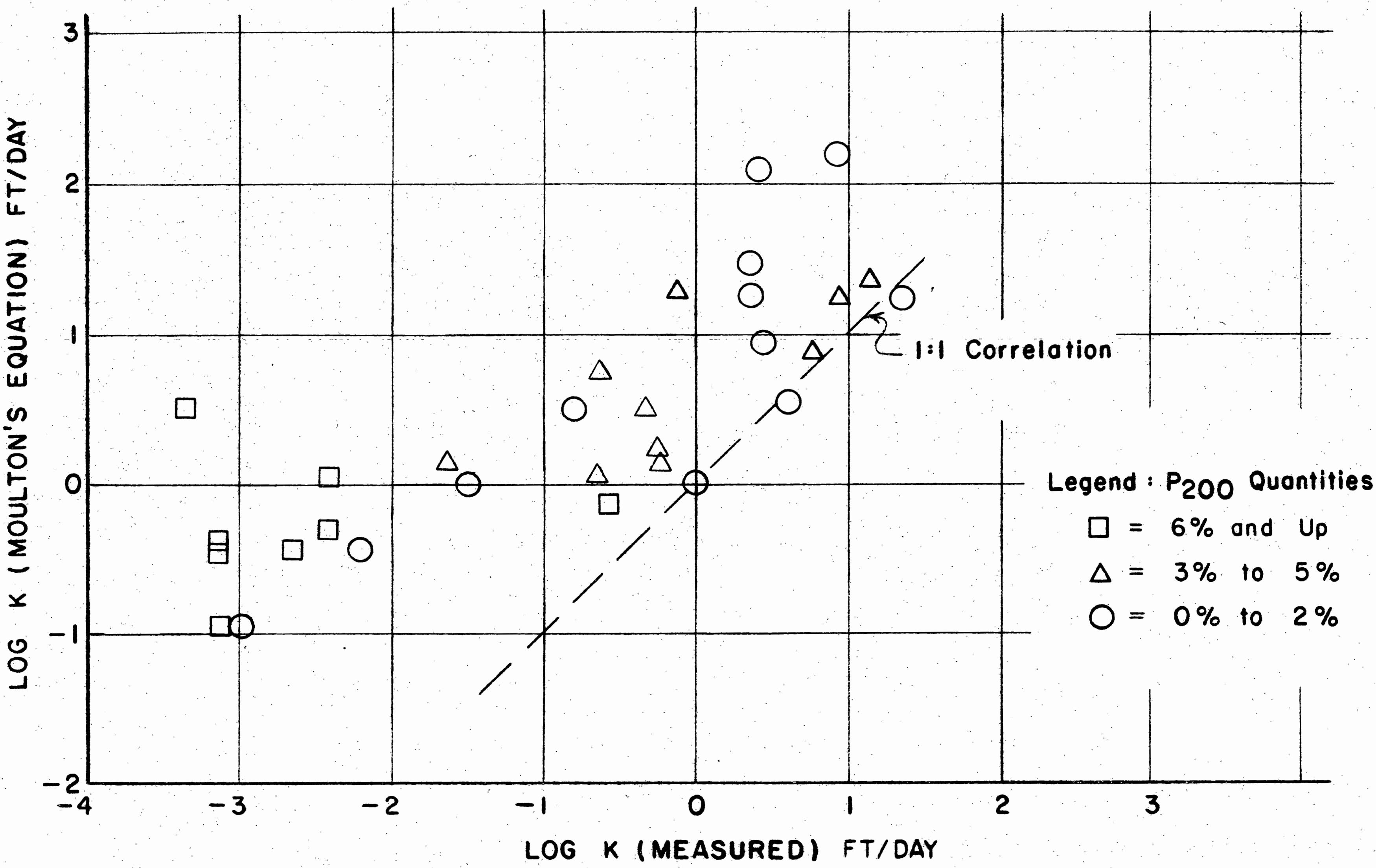


Figure 11 — PERMEABILITIES: CALCULATED VERSUS MEASURED (ADJUSTED FOR SPECIFIC GRAVITY).

the D_{10} parameter (See Table 12). For the naturally existing bank-run materials in New Jersey that comply with the Department's specification bands, this small variation is almost inevitable. But if samples having considerably more latitude in grain size distribution are included in such analyses, the D_{10} size would logically assume more significance. The poor agreement between the permeability results obtained in this research and Moulton's equation, as well as the significant differences between that equation and the new expression presented herein indicates, in the opinion of the authors, some basic, inherent deficiencies in the accurate prediction of permeabilities, using only these three parameters, namely, porosity, P_{200} and D_{10}.

In all fairness to the method, however, it is noted that the correlation coefficient obtained for Eqn #2 was 0.87 (standard error of estimate = 0.84 orders of mag.) indicating a fairly good fit of the data. This may be indicating that, although there is some apparent weakness in the method over a wide variation of the parameters, the method may be useful on a more limited basis, i.e. when one parameter shows little variation for a particular group of materials. In such an instance, it would, of course be necessary to develop, on the basis of test results, the equation that best described that group of materials.

In view of this, it may be conjectured that another method of representing the grain size characteristics of the subject soil may provide better results than reliance upon the D_{10} and P_{200} quantities. In an effort to study this possibility, Hudson A values were used in a combined form called "gradation Modulus $\overline{A}$,"[7] This method provides a single number to represent an entire gradation. For each consecutive pair of sieves used in the gradation analysis the mean diameter of the particles between those sieves is calculated. This diameter is multiplied by the percent fraction of aggregate retained between those sieves. The sum of these products is the combined "Gradation Modulus $\overline{A}$" hereinafter referred to as "$\overline{A}$" value.

The $\overline{A}$ value method was originally developed to compensate for deficiencies in some other aggregate gradation modulii and to provide a single modulus for use in the various applications of the other modulii. These applications were concerned with specific surface considerations in asphaltic concrete mixtures, soil-moisture mixtures, and portland cement concrete mixtures. However, basically the $\overline{A}$ value is still a function of average particle sizes. Superficially, $\overline{A}$ values appear to have an advantage over a D_{10}, P_{200} method because the parameter $\overline{A}$ would never approach zero regardless of the openness, or the fineness, of a particular gradation. Furthermore in the calculation of the $\overline{A}$ value, progressively more weight is given to the finer sieve sizes.

From the data obtained from the tests in this project, a multiple regression analysis was performed using measured permeabilities, "K", porosity, "n", and $\overline{A}$. The equation obtained is shown in Figure 12 and follows:

$$K = \frac{8.86(10^{16})\ (n)^{19.945}}{(\overline{A})^{9.798}} \text{ (ft./day} \qquad \text{(Eqn \#3)}$$

The correlation coefficient obtained for this expression is 0.787, with a standard error of estimate of 1.02 (orders of magnitude). Thus, on the basis of the same data, the use of $\bar{A}$ values did not provide a more accurate theoretical prediction method. One factor that should be noted is that while the $\bar{A}$ value method is designed to consider, in a weighted fashion, the amounts of soil particles of various sizes, the analysis performed in this research could not take advantage of this capability.

Unfortunately, these considerations involving $\bar{A}$ values were an afterthought. Therefore, the sieves used in the particle size analysis were for specification compliance determination and included only the 2", 3/4", #4, #50, and #200 sieves. The #50 and #200 sieves accurately provided D_{10} and P_{200} quantities. However, it is thought probable that additional sieve sizes between 3/4" and #200 would improve the prediction capability using $\bar{A}$ values.

As a final consideration regarding permeability prediction, the authors note that neither of the foregoing methods considers the shapes of the constituent particles. The New York State Department of Transportation has completed work in this regard(9) wherein a formula for the prediction of permeability of granular soils is presented. The formula itself considers only two parameters, i.e. porosity and specific surface. However, the method used in determining the specific surface includes "adjustment" of the specific surface of each sieve interval by a "shape factor". While determination of the "shape factor" is a subjective procedure, it may well provide the refinement necessary to obtain good permeability prediction. Shape factors were not determined in the course of testing in this project and no analysis is presented.

In summary, it appears that permeabilities of soils may be theoretically determined only within rather liberal limits of accuracy with methods that are based only on particle size characteristics and density or porosity. Some refinements in particle size parameters, and the addition of a "shape" parameter warrants further investigation. In this research, the number of samples for each particular geological origin was rather small. However, it may be possible to develop individual prediction formulii for various classifications of soil; said classification would isolate particle shapes and therefore eliminate the shape parameter requirement within each class. With the development of a more accurate permeability prediction capability, engineers would then possess a new tool which may become quite useful. Conceivably, the theory could be used to modify soils to attain a desired permeability for a particular situation. As is usually the case, the ultimate usefulness of a new "tool" or concept is realized only through years of trial and experimentation.

4. Considerations Relating to Needs of the Highway

The ensuing discussion largely departs from theoretical analysis and instead attempts to pull various facets and considerations together in a practical approach toward evaluating the permeability data in light of the highway system needs. In doing so, the discussion most appropriately begins with two very pertinent questions: a.) Do the base and/or subbase courses need to be drained, or to be drainable?; and b.) Assuming that drainage of said courses is necessary, what are realistic requirements and can they be achieved within the context of our current design?

The tests conducted in the course of this project are not considered "representative" of field conditions, nor were they intended to be. Therefore, it would be difficult, to accurately predict from the results of these tests the field behavior of the subject soils. And in that sense the CBR results of this project cannot be used to indicate the effects of water upon the base and subbase materials. However, the question of whether or not these materials need to be drained is answered directly by documented field performance in the case of PCC pavements. In bituminous pavements, water is not so obviously a culprit in failure, probably because the failure mode is so different from that of PCC slabs. It is known, however, that water does get into the supporting layers of bituminous pavements and in quantities that result in saturation of the base course. And it has been shown that base and subbase support values are lower during the presence of moisture varying from optimum to saturation. To provide maximum support, the base and subbase materials must be at a moisture content less than optimum. Hence, one can conclude that with bituminous pavements there is a need for drainage in these courses.

In determining the drainage requirements, several factors enter the picture.

a. Frost effects - Ideally, it would be desired that no water remain in the base course after an extended period of time, say 5 to 10 days after a rain. Based on experience in measuring frost penetration in New Jersey pavements, it would be extremely improbable that frost would penetrate the base course within that period of time following rain.

b. Stability with Water - For the granular materials used in New Jersey, the presence of water in the base courses is detrimental. For the most part, these materials are permeable enough to become saturated, but not permeable enough to drain themselves sufficiently to prevent damage under traffic loads. It has been noted elsewhere herein that CBR values are about the same for the optimum moisture content of these soils, as for the saturation point, and further, that both of these CBR values are lower than the CBR's in a "drier" state. This may be an indication that, particularly for bituminous pavement, the base course would remain with a semi-critical water content long after the "free water" had drained. However, given the type of soil gradations that exist, only the free water can be dealt with.

It is this free water that causes the more dramatic failure such as pumping and its associated problems, as well as the more obscure, long-term settlement related problems which may be occurring beneath bituminous pavements. This is explained by a phenomenon known as "liquefaction" which is described as follows:

> 'The liquefaction in fine, open-structure, and saturated sand, induced by a sudden shock, is explained as follows: the sudden shock means a suddenly applied shearing stress to the soil mass. Upon receiving the shock, the sand tends to decrease rapidly in volume. Simultaneously, the water in the voids of the sand - termed pore water - receives a suddenly applied pressure, or stress.

In soil mechanics, stress carried by water is termed neutral stress. Upon the increase in neutral stress, some of the weight of the soil mass, which might be considered as furnishing the normal effective or intergranular stress entering into the shearing process of soil, is transmitted to the pore water pressure. According to Coulomb, the shear strength of a non-cohesive soil, τ, can be expressed in an analytical form as

$$\tau = (\sigma_n - u)\tan\phi$$

where $(\sigma_n - u)$ = normal effective (intergranular) stress, in kg/cm^2, or in lb/in.^{2}, whichever system of units of measurements is used;
σ_n = total normal stress;
u = pore water pressure, or neutral stress, and
$\tan\phi$ = coefficient of internal friction of a non-cohesive soil which for one and the same material is considered to be constant.

From this equation, it can easily be seen that upon the increase in neutral stress, u, the effective or intergranular stress of the soil, $(\sigma_n - u)$ decreases, and the shear strength of the soil, τ, decreases. The decrease in shear strength means decrease in bearing capacity of the soil.'[4]

Thus, water must be removed from the base course as quickly as possible, since traffic loading continues regardless of the water's presence. The time to be allowed for removal of this water is a matter of judgement. Field investigations reveal that pumping continues for as much as 24 hours following a rain storm. Therefore, it would not be unreasonable to require removal of 10% - 20% of the available water within one or two hours -- even less time if practical and possible. This would (hopefully) relieve the soil support system for the hydrodynamic forces which occur due to traffic loads.

The U. S. Army Corps of Engineers utilizes this kind of approach, but with vastly different "boundary" conditions. The Corps requires that 50% of the available water be drained from a base course within 10 days.[8] In calculating the time for 50% drainage, the following formula is used:

$$t = \frac{N_e D^2}{2\ KH_o} \qquad \text{(Eqn \#4)}$$

where:

t is time in days for 50% drainage

N_e is the effective porosity of the soil

D & H_o are the dimensions as shown in Figure 12

K is the coefficient of permeability in ft./day

For purposes of illustration, Table 13 presents results using Equation #4 while varying parameters as listed.

In Cases #1, #2, and #3, the calculations are based on the current design practice of using an "outlet trench" every 350 feet along the profile. It is also assumed that the grade is 5%, a very generous assumption. Case #1 represents a clean sand and some of the more drainable 1C subbases. In reality such permeabilities are not encountered in the upper, or base, courses. Cases #2 and #3. are more representative of the majority of soil bases and subbases in New Jersey. In examining the results it should be remembered that the permeabilities listed are vertical permeabilities. Literature reveals that the effective permeability along the direction of stratification may be several times the value of the vertical permeability.[10] In Table 13 the ratio of horizontal to vertical permeability was taken as unity. Also, the calculations assume that no water escapes through any means other than the outlet trench. Admittedly, this is a severe assumption but it is used in the absence of any factual quantitative data to the contrary. The

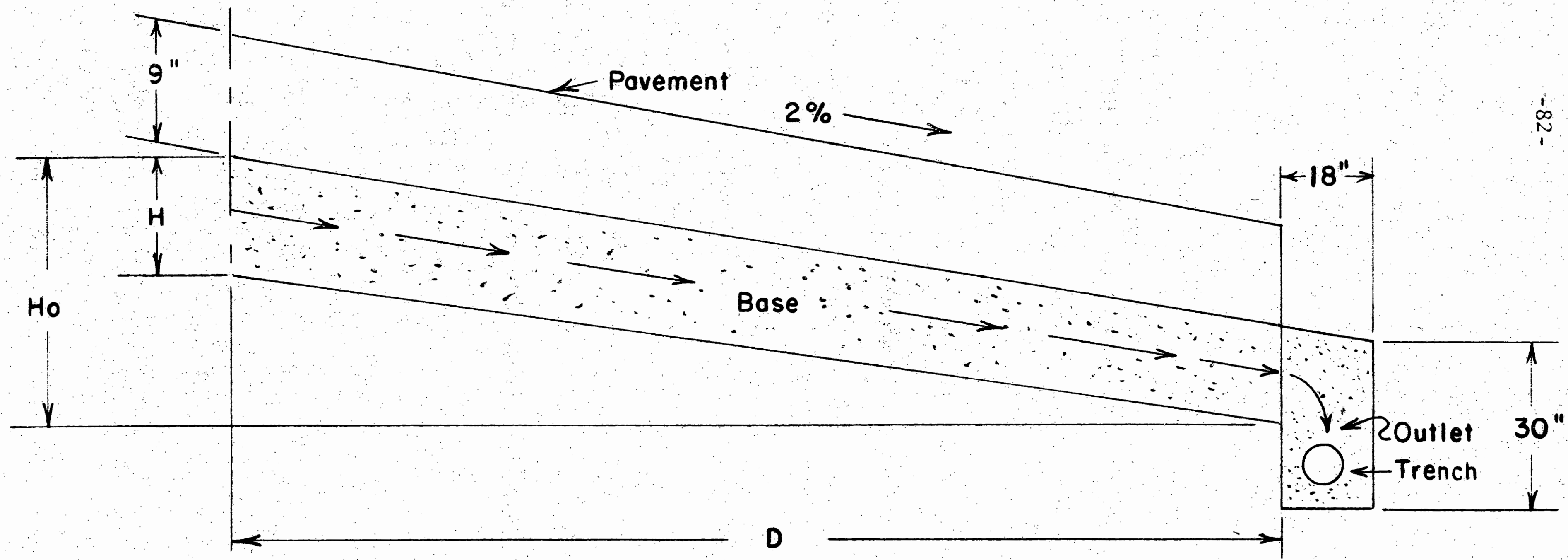


Figure 12 — SCHEMATIC OF PAVEMENT CROSS-SECTION WITH SUBDRAINAGE SYSTEM.

Table 13 - DRAINAGE EXAMPLES

Case No.	N_e	ft/day	D feet	H_o % grade x D	t Days	Comments
1	0.1	10.0	350	17.5 (@5%)	35.0	Clean sand base course
2	0.06	0.001	350	17.5 (@5%)	575 years	
3	0.06	0.1	350	17.5 (@5%)	5.8 years	
4	0.06	0.1	24	0.98 (@2%)	176 days	Underdrain along shoulder
5	0.06	0.01/ 10.0	0.25	1	25 days	Vertical drainage (a)
6	0.06	0.01	0.05	1	5 days	Vertical drainage (a,b)
7	.25	160	24	0.90 (@2%)	0.1 (2 1/2 hours)	Underdrain along shoulder (b)

Notes:

a) Equation #4 was not used. t (days) = D/K

b) In these cases, t was calculated only for 10% drainage of the free water severity of the assumption is borne out by the absurdity of the time required for 50% drainage, i.e. 575 years and 5.8 years for Case #2 and #3, respectively. The significance of this is simply that the pavement system on the whole does not drain itself via outlet trenches.

Case #4 exemplifies a base course with a "typical" permeability and assumes that there is an outlet trench placed parallel to the roadway and that water in the base course need only drain 24 feet transversely. The gradient is reduced to 2% which represents the maximum normal cross slope to be used. The total gradient used in the computation also includes an additional 0.5 ft. to account for the base course thickness. This case is an attempt to determine whether or not an underdrain can be practically used to drain typical bases meeting New Jersey's current gradation specifications. This example (Case #4) shows that even a base with a comparatively good permeability is undrainable by the use of underdrains.

Cases #5 and #6 assume that the base course drains vertically downward into an extremely permeable strata. Case #5 shows that for a base course "K" of 0.01 ft/day, 25 days would be required to drain one half of the free water. This means that theoretically after 25 days, the bottom half of a 6" thick base course would still be saturated. Similarly, Case #6 shows that the same course would require 5 days for 10% of its free water to drain out, although 10% drainage for such a material would most probably not be sufficient.

Case #7 is a hypothetical material which in reality would be an open-graded material. For this case, it was assumed that 10% drainage (of free water) occurs within 0.1 days. This assumption was based on a desire to attain 10% open voids within a minimum of time so that hydrodynamic forces are quickly alleviated. A material such as this would have an effective porosity of at least 0.20. Assuming that this layer would drain transversely into an underdrain, the required permeability would be 160 ft/day.

The various cases in Table 13 help to put the drainage characteristics of New Jersey bases and subbases into perspective. While it is known that these materials do not require 575 nor 5.8 years to drain, it is known that some materials remain saturated for 1, 2, or 3 days following rain. The analyses in Table 13 indicate that the current drainage, i.e. occasional subbase outlet trenches, is incapable of effectively draining the base and/or subbase layers. Because the permeabilities of most of these layers fall into the "low permeability" to "impervious" range, they cannot be effectively drained by an expansion of the underdrain network. Such an expansion exists on a section of Route I-78 where about 14 miles of underdrain were installed beneath the shoulder joint of a PCC pavement. This underdrain system was installed as an attempt to alleviate the severe pumping condition of the highway. Follow-up investigations revealed that after the installation of the underdrains, the base course simply pumped into the open-graded drain material instead of vertically upward and onto the pavement surface. It is also noteworthy that during the design of this underdrain system, anti-clogging criteria (relative percentages of particle sizes) was applied. In addition, it is suspected that the underdrain material was finer than that used in said analysis. This is very important because it demonstrates the severity of the hydrodynamic forces that exist beneath a pavement as well as the problems that result. In view of this experience, it is felt that the provisions of drainage outlets throughout such low-permeability base courses (as used in New Jersey) is a futile effort; conversely, the need for an entire, compatible, underdrain system, with filter design where necessary, is amply demonstrated. The hydrodynamic forces that occur due to traffic loading must be alleviated, and this can only occur through increased perviousness throughout the stressed and water-laden courses.

From this concept of increased perviousness throughout the base course, arises the question of how much permeability is needed. In New Jersey, basically the same PCC slab and expansion joint design has been used for almost 30 years. Some of those pavements perform excellently, while others begin to pump with their first years of traffic. The base course materials specifications have not changed appreciably in that 30 year period. Apparently, the base course materials allowed by the gradation specifications are "borderline" materials with respect to their susceptibility to pumping. In the case of bituminous pavements, as previously stated, failure modes are not so dramatic and the causes for failures, indeed the definitions of bituminous pavement failures, are not nearly so obvious. It is known that substantial water enters the base course of new and old bituminous pavement at least on a localized basis, if not in general. Therefore, it is known that at least some areas of bituminous pavement are sitting upon a weakened base course during wet periods. For these pavements the amount of improvement to be gained from a more pervious base course simply cannot be determined from the current extent of knowledge.

References

(1) "Standard Specifications for Road and Bridge Construction", with current supplemental specifications, New Jersey State Department of Transportation.

(2) A. Casagrande and R. E. Fadum, "Notes on Soil Testing for Engineering Purposes", Cambridge, Mass., Harvard University Publication No. 268, 1939/40, Figure 11, page 23.

(3) Department of the Army Technical Manual, TM5-820-2, August 1965, "Drainage and Erosion Control - Subsurface Drainage Facilities for Airfields", page 11.

(4) A. R. Jumikis, "Soil Mechanics", 1962, University Series in Civil Engineering and Applied Mechanics, Pub: D. VanNostrand Co., Inc.

(5) Allen Hazen, "Some Physical Properties of Sands and Gravels, with Special Reference to Their Use in Filtration", 24th Annual Report, Mass. State Board of Health, 1892, Public Document #34, Boston, page 553.

(6) Lyle Moulton, "Highway Subdrainage Manual", 1976, West Virginia University.

(7) NCHRP #69, 1969 "Evaluation of Construction Control Procedures, Aggregate Gradation Variations and Effects", Appendix A, Derivation and Use of the Gradation Modulus $\bar{A}$.

(8) Department of the Army Technical Manual, TM5-820-2, August 1965, "Drainage and Erosion Control - Subsurface Drainage Facilities for Airfields", page 10.

(9) "Test Procedure for Specific Surface Analysis", 1973, New York State Department of Transportation.

(10) "Soil Testing for Engineers", T. William Lambe, Mass. Institute of Technology, 1960, page 52.

APPENDIX A

EQUIPMENT USED IN SOIL TESTING LABORATORY

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EQUIPMENT USED IN SOIL TESTING LABORATORY

Apparatus Description and Modification

1. Particle-size analysis equipment

a.) Sieve Set from "Soil Test".

U. S. standard sieves 8" diameter. Sieve sizes included 2", 1 1/2", 1", 3/4", 1/2", 3/8", #4, #8, #10, #16, #50, #140, #200, and a pan with cover. Also #200 washing sieve 4" height were used in washing samples. Because of the large number of samples that were anticipated sieves ranging from #10 to #200 were purchased with stainless steel mesh to minimize wear.

b.) Sieve Shaker Set from "Soil Test".

Model 305 A-1, 110 volts 60 cycles AC. The shaker has a capacity of 7 sieves. The machine was secured to the concrete floor using lead "malz" anchors. A foam insulated cabinet housed the shaker to minimize noise emission in the working area. See Figure A1.

2. Proctor Density Equipment from "Soil Test".

Six inch diameter compaction molds with collars, model number CN-403-6 were used for moisture density determinations. Two compaction hammers, a CN-415 standard hammer (5.5 pounds x 12" drop) and a CN-416 modified compaction hammer (10 pounds x 18" drop) were used in the various tests.

3. CBR Equipment from "Soil Test".

Catalog number CN702. A complete set of laboratory CBR equipment was purchased as a unit. This included a mechanical loading

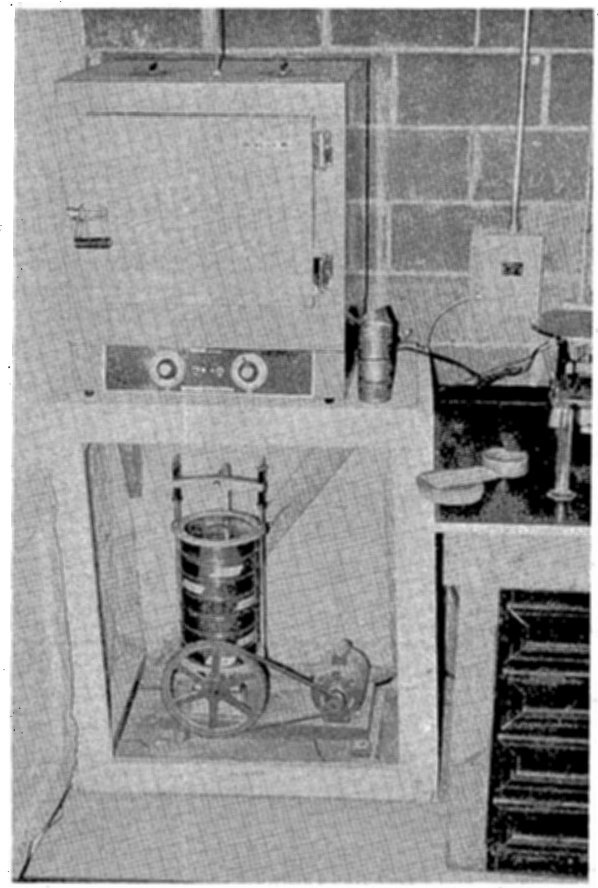

Figure A1
SIEVE SHAKER AND CABINET

press with 6000 lb. capacity, 3 CBR molds (6" dia.), spacers, filter screens, swell test apparatus, surcharge weights, straight-edge, cutting edge, and 10 pound compaction hammer. See Figures A2 and A3.

4. Permeability - Constant Head Equipment.

Compaction permeameters (6" diameter) were obtained from Soil Test, Inc., and modified to meet ASTM specifications D-2434-68 Permeability of Granular Soils. Figure A4 is a schematic drawing of the modified permeameter mold. To provide a means to take average pressure readings in the sample, a groove 1/8" x 1/8" was

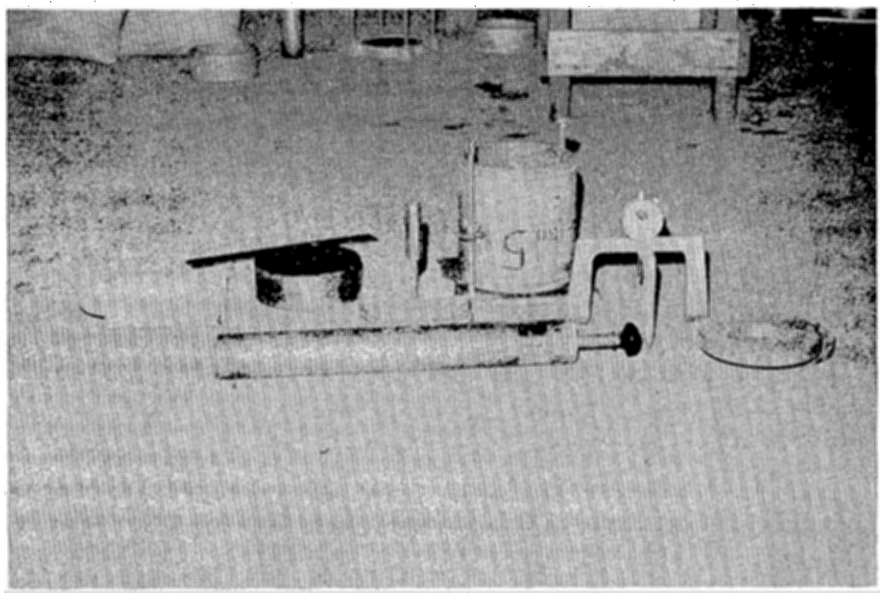

Figure A2
CBR APPARATUS

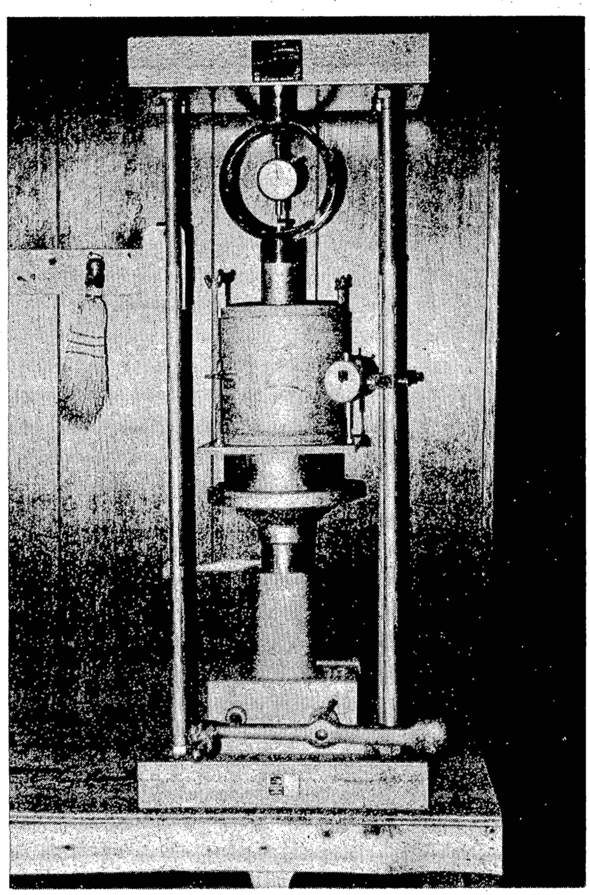

Figure A3
CBR LOADING PRESS

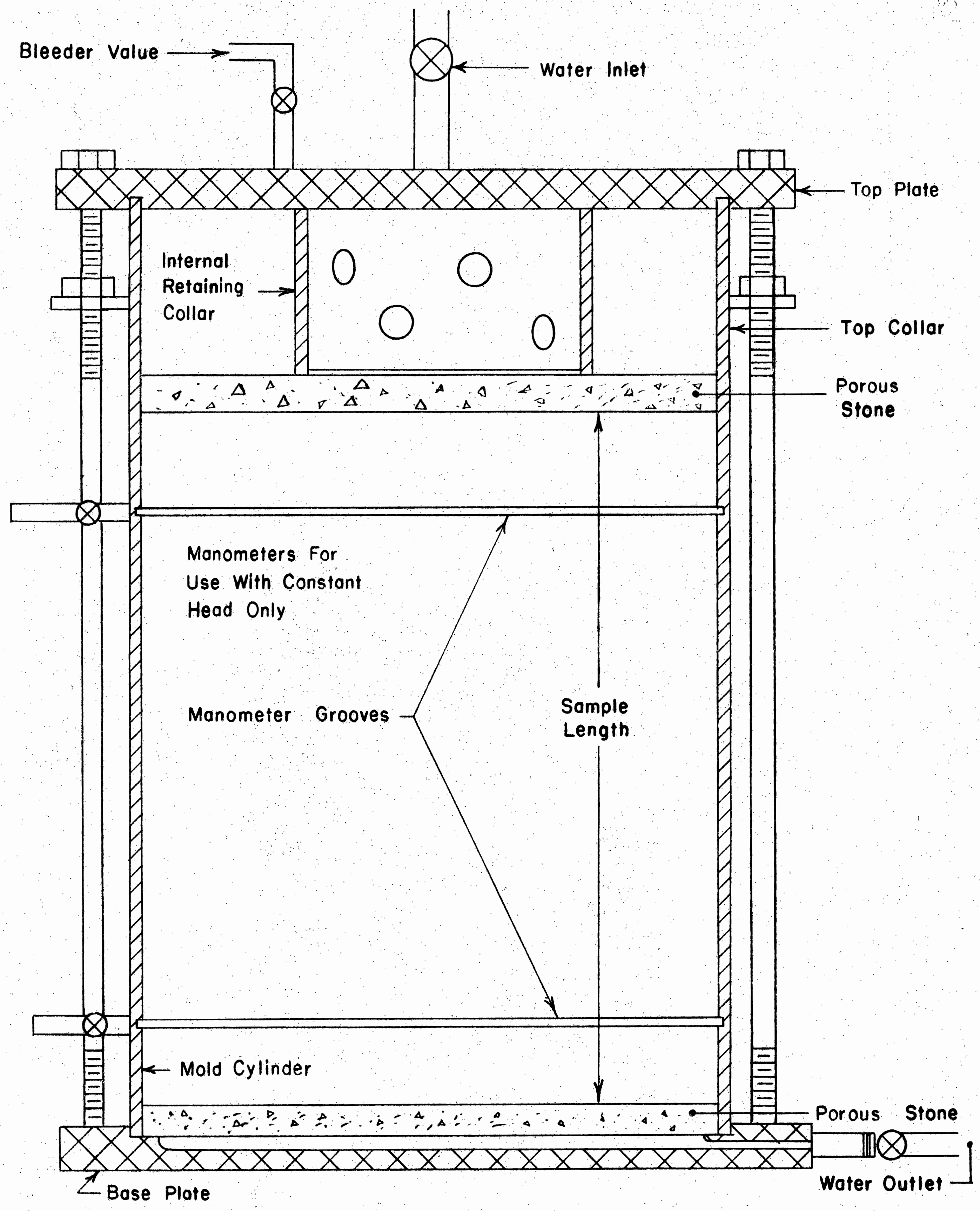

Figure A4 – SCHEMATIC DRAWING OF PERMEAMETER machined 1" from each end of the cylinders. With a 7" sample height this allows 5" of sample between manometer grooves, providing an adequate pressure differential. Nipples were tapped into the grooves for connection to the manometer tubes. No. 200 sieve screen epoxied across the grooves prevented fine soil particles from entering the manometer tubes. In the top plate of the permeameters, holes were drilled and tapped to accept a 1/4" metal pipe stud and inlet valve assembly. To remove trapped air, a small bleeder valve was also fitted to the top. On the outlet of the permeameter, a shut off valve was provided as a means to start and stop the tests.

Porous stones were obtained to fit tightly into the permeameter at the top and bottom of the soil sample. Rubber gaskets are used at the mold separations (top plate-collar-cylinder-base plate) creating a good seal for internal-external pressure differentials. During the vacuum saturation of the prepared soil samples, it was found to be impossible to keep the sample intact, despite the internal spring that is compressed between the top porous stone and the top plate. To maintain the integrity of the soil sample, a rigid, perforated collar was fabricated to fit into this same space. While this maintained the sample integrity, it was subsequently found to be totally impossible to vacuum saturate most of the subject soils and soak saturation was used henceforth.

The ASTM procedure for constant head permeability tests requires the use of either de-aired water, or a filter apparatus to remove most of the dissolved air from the water. Since constant head tests generally require large volumes of water, a constant head tank

20" wide x 24" long x 9" high was constructed and provided with a sand filter. See Figure A5. The tank was capable of delivering a maximum of 3 gallons/min., more than an adequate quantity for the base and subbase samples expected. A few tests on inlet and outlet water in the tank showed that very little air was being removed by the sand filter and tap water was used for subsequent tests.

On the very permeable samples, a vacuum pump was connected to the top bleeder valve and a vacuum gage attached to the lower part. At full vacuum, the gage was disconnected while the lower parts were opened slowly to allow the upward flow of water through the sample. After saturation, the lower port was closed, and the water supply was hooked to the inlet. The manometer tube valves were opened and tubes allowed to slowly fill with water.

The bottom part was opened and water run through the sample until a stable condition in manometer readings was observed.

Time, t, head, h (difference in levels in the manometer), quantity of flow, Q, and water temperature, T, were taken and recorded.

Coefficient of Permeability

Fundamental Test Conditions

The basic assumptions in constant head permeability calculations assume laminar flow through the specimens. The following ideal test conditions are prerequisites for laminar flow of a liquid through granular soils under a constant head condition.

a. Continuity of flow with no soil volume change

b. Flow with total saturation of soil voids

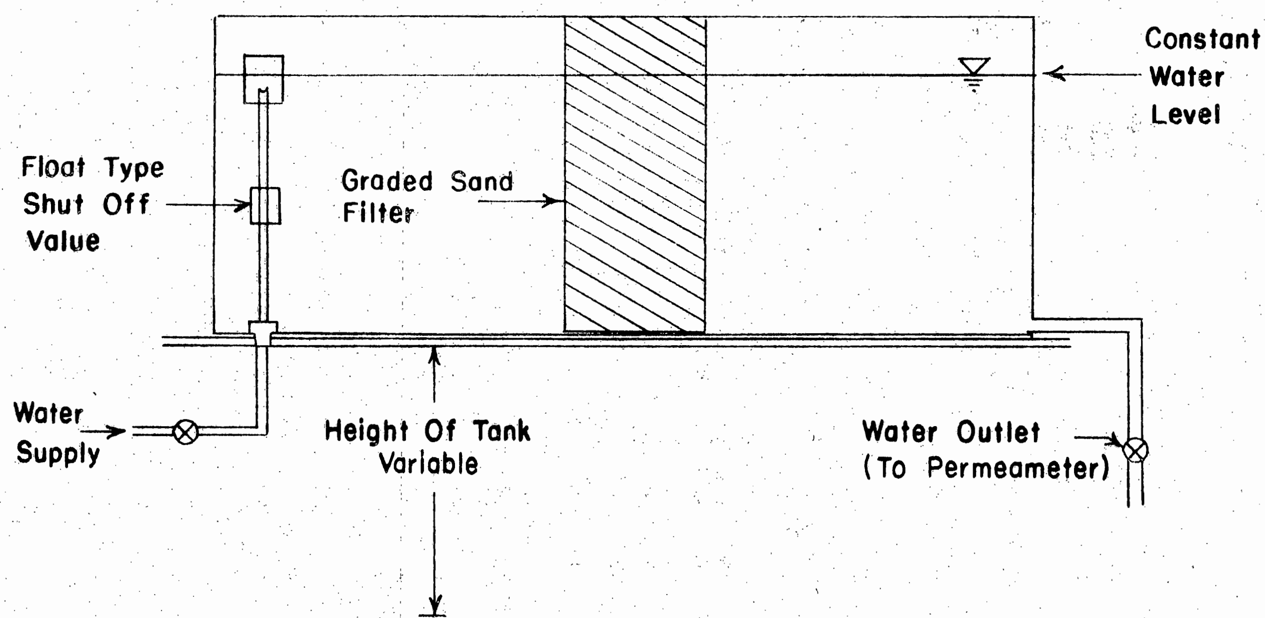

Figure A5 - SCHEMATIC DRAWING OF CONSTANT HEAD & DE-AIRING TANK

c. Steady state flow with no changes in hydraulic gradient

d. Direct proportionally of velocity of flow with hydraulic gradient below certain values at which turbulent flow starts.

Darcy's law for conditions of laminar flow states that Q the quantity of water flowing through a given cross-sectional area, A, of soil under a hydraulic gradient, i, in time, t, can be expressed by the formula

$$Q = K\, i\, A\, t \qquad (1a)$$

or solving for K

$$K = \frac{Q}{iAt} \qquad (1b)$$

where

$$i = \text{hydraulic gradient} \ \frac{h}{L}$$

$$K = \frac{QL}{hAt}$$

in which

Q = total quantity of water which flowed through elapsed time, t,

h = head loss (difference in manometer readings)

A = area of sample

L = length of samples between manometer ports.

5. Oven - From Blue M Electric Company

Temperature range from 100°F to 550°F, 230 volts Interior dimensions are 17" x 15" x 20".

6. Hydrometer Equipment - from "Soil Test".

Model CL-207B Hydrometer and CL-271 Hydrometer Jars were used in conjunction with the constant temperature bath at the department main soil testing lab since this equipment was not needed often enought to warrant the expense of a constant temperature bath at the Lexington Avenue lab.

7. Sample Washing Apparatus

A standard kitchen type spray hose was installed on the lab sink to facilitate sample washing. A settling tank of approximately 7 gallon capacity was installed beneath the sink to allow fine material to settle out of the water before reaching the building drainage system. This tank was equipped with a detachable outlet pipe and set on a dolly to facilitate dumping and cleaning.

8. Vacuum Pump - from Sargent Welch

Duo Seal pump Model No. 1399 110 volt AC powered.

9. Scales - a heavy duty solution balance from Ohaus with a 20 Kg capacity and a 1gram sensitivity was used to weigh large samples, molds and pans. A small scale from Will Corporation with a 1500 gram capacity and a .1 gram sensitivity was used in weighing moisture cans, small samples, etc.

10. Soak Tanks - a 55 gallon drum cut in half and heavily "red leaded" provided an adequate means of soaking the samples for both permeability and CBR tests.

11. Miscellaneous Equipment -

Towels, pans, sieve cleaning brushes, spatulas, sample cans, dial gages, rubber tubing, clamps, valves and other equipment was obtained by various means.