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Use of Aerial Color Photography In Materials Surveys

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•THIS PAPER summarizes the significant findings in the use of aerial color transparencies in construction materials surveys conducted by the Federal Highway Projects Office, Region 9 (Colorado, New Mexico, Utah, and Wyoming), U. S. Bureau of Public Roads, presents selected data and costs of aerial films and processing, as well as trends in aerial color films, and suggests areas in which research is needed.

The evaluation of aerial color photographs reported was a secondary objective of a comprehensive survey of construction materials. The project was not designed as a controlled research experiment and the results are representative of fairly restrictive climatic conditions, geographic area, and number of geologic materials. Aerial color transparencies (Kodak Ektachrome Aero Film) at a scale of 1:6,000 (500 ft to 1 in.), in single flight strips, were taken along the major road system and in other selected locations in Yellowstone National Park. The photographs were taken with a 6-in. focal length Pleogon lens from an average flight height of 3,000 feet. Over 400 flight-strip mi of color film were taken during the summers of 1961 and 1962. Selected segments totaling about 100 lin mi were taken with conventional panchromatic film at the same scale.

The color transparencies were examined in three dimensions by means of a mirror stereoscope and portable light source. Potential construction materials sources were annotated on transparent plastic envelopes containing the aerial photographs. Each source was then verified by ground inspection. Potential sources appearing to have the best construction materials were further investigated in the field with a backhoe after administrative approval for each potential site had been obtained. Representative samples were taken from each source investigated and laboratory soil tests were performed for engineering evaluation.

FINDINGS

As a primary result of the Yellowstone Park study and other materials surveys in which aerial color photography was used it was found that color photography had many advantages over black-and-white photography for use in materials surveys. Some of these advantages have been previously reported (1).

Use of black-and-white aerial photographs for interpretation of specific ground conditions, soils, or geologic materials involves the interpretation of various photographic tones. The number of such tones or shades of gray that can be differentiated is extremely limited, and many different types of soils or geologic materials may have about the same tonal expression. Inasmuch as the human eye can perceive about 20,000 shades and hues of color, interpretation of color photographs is considerably easier than that of black-and-white photographs. Identification of materials deposits can be based partially on the color of the deposits instead of relying on photographic tones as is done with the black-and-white photographs. Neither color nor photographic tone is used alone in identifying materials, but they do represent one of the most important elements used in photographic interpretation.

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An extreme example illustrating the deficiency of black-and-white photographic tones for use in photographic interpretation was noted in the Yellowstone Park study. Occasionally, it was extremely difficult to differentiate between certain low-growing types of vegetation and soil or rock on black-and-white photographs because both produced identical gray photographic tones. Differentiation was, of course, very simple on the color transparencies. This is an extreme example but it was paralleled by attempts to differentiate between types of materials.

Minard and Owens (2) in mapping soils and geology of the Atlantic Coastal Plain in New Jersey noted that certain rock and mineral types can be identified much more readily from aerial color photographs than from black-and-white photography. This also was found to be true in the Yellowstone Park study where it was possible to differentiate by using aerial color photography sands and gravels containing large percentages of obsidian or siliceous sinter from sands and gravels composed of the more common rock assemblages. Such a distinction was not as obvious on black-and-white photographs of the same areas.

Color differentiation also has proved superior to the use of black-and-white photographic tones for identification of engineering materials and geologic features on other projects undertaken in Region 9. For example, a comparison of color positive transparencies with black-and-white prints at a scale of 1:6,000 taken in Dinosaur National Monument, Colorado-Utah, where geologic formations are well exposed and of variegated coloring, showed the following: (a) differentiating between strata of sandstone and shale is easier on color than on black-and-white photographs, (b) flat erosional remnants of granular mountain outwash overlying dipping sedimentary formations are easier and more positively identified on color than on black-and-white photographs, (c) highly fractured light-colored limestone strata can be identified and differentiated about equally well on either type of photography, and (d) river terrace remnants are more reliably identifiable on color photographs.

Because of the distinctive greens and browns of organic soils, wet soils, boggy ground, and seepage zones, these features can also be more readily identified on color photographs than on black-and-white. Minor drainageways, poorly drained depressions and swales, and seepage areas in landslides show up quite clearly on color photographs. On black-and-white photographs such areas display generally darker photographic tones, but identification or delineation is not as positive and in some cases the conditions are not apparent even to an experienced interpreter.

Various vegetative types can also be recognized more readily on color than on black-and-white photographs. In some cases correlation can be made between type of vegetation and specific types of materials or ground conditions. In such cases the color photography proves superior to black-and-white.

Image identification is easier on color aerial transparencies than on black-and-white prints because of the color contrasts provided by the transparencies. Color transparencies are especially helpful in identification of cultural features such as trails and aerial targets.

While aerial color photography generally has been found superior to aerial black-and-white photography in terms of ease of interpretation and quality of results obtained, it does have certain disadvantages as compared to black-and-white photography.

The first, and most obvious, is cost. As a general rule, aerial color photography costs approximately two to four times as much as conventional black-and-white photography.

Another disadvantage of color photography is the difficulty in obtaining proper film exposure especially in areas where extreme lighting contrasts occur within the area of a single photograph. For example, in the Yellowstone Park study, hot spring deposits that were light in color and devoid of vegetation provided such sharp contrast with the surrounding dark vegetation that they often exceeded the exposure latitude of the color film. In such light-colored areas any colors present were often "washed out" in the photograph, and in general, images were not well-defined.

In the Yellowstone study considerable differences in quality of color reproduction were sometimes noted between flight strips of the same areas taken at different times.

Comparison of certain overlapping flight strips of aerial color photographs taken during the summer of 1961 with those taken during the summer of 1962 revealed that the latter had generally better color reproduction. This improvement was particularly noticeable in areas of dense vegetation where green hues were predominant. In these areas there was a tendency for the nongreen colors in the landscape to take on a more natural color in the 1962 photography than in the 1961 photography. This was particularly helpful in mapping old gray sandy beach deposits in heavily forested areas. In the 1961 photography the gray colors tended to be obscured by an over-all greenish cast making the detection of these deposits difficult. The improvement in the 1962 photography was attributed to more accurate film exposure.

Another deficiency of color transparencies as compared to black-and-white photographs is that production of duplicate transparencies is rather expensive; furthermore, color reproduction of the originals is difficult to attain without considerable experience. Therefore, if a color transparency is lost or damaged it cannot be replaced as easily as a black-and-white print. As a result, in working with color transparencies, considerable care in handling must be exercised.

Because color transparencies must be viewed by transmitted light, a suitable illuminating source is necessary if they are to be used successfully for materials surveys. This has not generally handicapped office use but can prove somewhat of a problem when used in the field. In the Yellowstone study, special portable light boxes provided with vibrator power supply which could be connected to an automobile electrical system were used for this purpose. They were fairly successful, but were awkward to work with in the field vehicle and of course were of no value when away from the vehicle.

The necessity for transmitted light when viewing color transparencies also means that a mirror stereoscope must be used because individual transparencies cannot be easily overlapped as black-and-white prints. In the field, a mirror stereoscope is relatively awkward as compared to the commonly used smaller lens stereoscope.

The Yellowstone study primarily utilized aerial color transparencies, and the comparisons have been with black-and-white photographs. If color prints are compared to black-and-white prints, it is obvious that some of the findings and conclusions from the Yellowstone study are no longer valid. For example, transmitted light would not be needed for viewing the color prints, and adequate duplicates of color originals would be available, thus eliminating the danger of loss or damage to the originals during interpretive work.

In one area of Yellowstone Park a number of short flight strips were taken using Agfacolor Negative Film CN17. Because these photographs were taken in conjunction with another project, only cursory examination was made. Contact color prints tended to have an excess of green, but had good resolution. It was later found that these photographs had been printed with a light source that did not provide a full spectrum. Contact prints subsequently made with an appropriate printing light source showed a great improvement in color balance. Black-and-white prints made from the same color film negative were of excellent quality.

PROCUREMENT OF AERIAL COLOR PHOTOGRAPHS

The distance between the project to be photographed and the airfield at which the photographic aircraft is based is extremely important in either a black-and-white or color photographic mission. Some delay in procuring color photography for the Yellowstone study was experienced because the aircraft and crew were based several hundred miles from the project area and could not take advantage of short periods of excellent photographic weather during prolonged cloudy and rainy periods.

Because the exposure latitude of most color film is narrower than for conventional black-and-white film, exposure difficulties are encountered where there is extreme contrast in the light reflectance within a photographed area. There is a tendency for the light-colored areas to be overexposed with a consequent "washing out" of colors and at times images are not registered distinctly. In flying over areas that are either light or dark, the photographer can properly adjust the exposure for the given scene.

In this connection, the ability of the photographer to think and use good judgment in photographing under varying conditions of light, haze, clouds, and changing subject matter is important in the procurement of quality color photographs.

Haze, which tends to cause color photographs to have an over-all bluish cast and thus subdues original ground colors recorded on the film, becomes an important factor as flight heights are increased, particularly in certain geographic areas and at certain times of the year. The effects of haze are less noticeable for relatively large scale photographs taken at fairly low flight heights. Various color films also have greater or lesser sensitivities to haze. Haze filters and compensation in the printing process are other ways in which the effects of haze are minimized. The tendency is toward the use of high quality shorter-focal-length lenses that permit lower flight heights for a given photographic scale. In addition to reducing the haze problem, such high quality short-focal-length lenses sharply reduce the amount of darkening of corners of photographs, a common problem with color aerial photography taken with inferior lenses. Antivignetting filters on the aerial cameras and automatic-dodging printers also can minimize corner darkening.

The use of filters reduces the amount of light reaching the film and sometimes may be a source of difficulty. For example, it was observed that in a number of flight strips on the Yellowstone project the images in the corners of the aerial photographs were blurred. It was later determined that this blurring was caused by a faulty color balancing filter that had inadvertently been exposed to excess heat. Fortunately, the blurring was not sufficiently serious to affect the utility of the photographs.

Color balance of an aerial photograph depends on the geographic latitude, season of the year, time of day, haze conditions, amount of cloud cover, film exposure, type and variation of emulsion, type of filters used, and film processing and printing.

TRENDS IN AERIAL COLOR FILMS

Through continued research by film manufacturers improvements in aerial color films have steadily been made. Many former objections and criticisms of color films are no longer valid. Significant trends, particularly within the last five years or so have been (a) a great increase in emulsion speed or light sensitivity, (b) reduction in granularity with consequent increase in resolution, (c) improvements in dye formulas resulting in a greater degree of color fidelity, (d) greater latitude of exposure, and (e) decreasing costs.

Film sensitivities and resolution have reached standards today that were hardly thought attainable by many only ten years ago. Costs of color film have been on the decrease, and this trend will continue as aerial color film comes into more general use.

Aerial color negative films of both domestic and foreign manufacture have recently been introduced in the United States. The advantages of aerial color negative film over color positive transparencies make it appear that negative film will come into greater use, particularly as improvements are made in emulsion speed and exposure latitude and as costs decrease. Color negatives afford greater flexibility in that both black-and-white prints and color prints can be made from the same negatives. The negatives can be saved and additional prints made as required. Compensation for incorrect exposure, haze, vignetting, and color balance can all be done in the photographic laboratory. Color balancing filters are not required on the aerial camera as they are with reversal-type film.

As a rule the paper base material on which color photographs are printed has many of the same characteristics as the base material used to print black-and-white photographs. At least one film company produces color print material which is tough, waterproof, and dimensionally stable (3).

SUMMARY OF SELECTED AERIAL COLOR FILM DATA AND CHARACTERISTICS

The following aerial color films are commercially available at this time and only selected characteristics are included.

1. Kodak Ektachrome Aero Film, Process E-3.—This is a color reversal film from which positive transparencies are obtained. This replaces Kodak Ektachrome Aero Film, High Contrast. This film has an "Aerial Exposure Index" (4) of 25, which is about three times as fast as the previous film with a speed index of ASA 40 daylight. Haze filters are recommended and color balance filters are used when necessary with particular emulsions.

2. Kodak Ektachrome Infrared Aero Film Process E-3.—This replaces Kodak Ektachrome Aero Film (Camouflage Detection). The new film is a false-color reversal film that has three layers sensitive to green, red, and infrared radiation, rather than blue, green, and red as in conventional color film. A yellow Kodak Wratten Filter No. 12 is used to absorb blue radiation to which all three layers are sensitive. Color compensation filters may be used for color improvement of transparencies as necessary. This film has an Aerial Exposure Index of 10 which takes into account the use of the yellow filter.

3. Kodak Ektacolor Professional Film, Type S.—This is a recently available color negative film with a speed index of ASA 80 daylight. Use of a haze filter is recommended. Color balancing is accomplished in the printing process. A recent experimental trial showed that good quality aerial color prints can be produced. This film appears to have reasonably wide exposure latitude without objectionable shifts in color balance. Although not designed for aerial photography, it has fairly high contrast and good haze penetration without the use of a haze filter.

4. Ultra-Speed Anscochrome.—A color reversal film from which positive transparencies are obtained. The speed index for this film is ASA 200 daylight. By forced processing the speed can be pushed to ASA 400-800. This film has a temperature-color balance of 6,000 K and has high resolution when exposed under normal conditions. The latitude of exposure is about plus or minus one-half lens stop for accurate reproduction and good color saturation. Color positive prints (Printon) can be made from this film.

5. Super Anscochrome.—This film is a color reversal film having a speed index of ASA 100. Exposure latitude and color balance are similar to Ultra-Speed Anscochrome. It has slightly higher resolution and provides somewhat less contrast than the latter.

6. Agfacolor Negative Film CN17.—This is a negative color film with a speed index of ASA 40 daylight. Typical exposures are 1/250 to 1/450 sec at f4 to f5.6 under average conditions of brightness. Satisfactory results can be obtained at one to two lens stops lower than for optimum exposure. Ultraviolet and light yellow filters can be used. Color balancing is accomplished in the printing process.

COST OF AERIAL COLOR PHOTOGRAPHY

As a general rule, aerial color photography costs about two to four times the cost of conventional black-and-white photography. This is not surprising since color films have three emulsions while panchromatic films have only one. Raw aerial color film costs approximately three to four times as much as panchromatic film. Chemical kits used for processing both types of film are about the same price.

One commercial aerial color processing photographic firm quotes a price of $90.00 for processing negative color film (9½-in. by 100-ft roll) and $115.00 for color reversal film of the same dimensions. Color paper prints that are not color balanced can be obtained for about $0.90 a print. Prints that have been color corrected and dodged for lens vignetting cost $3.00 each.

A price quotation obtained for 1:6,000-scale photography from a commercial photogrammetric firm was $55.00 per flight-strip mile. This includes one set of contact color prints, one set of black-and-white prints, and one photographic index. Additional color prints were obtainable at $15.00 per flight-strip mile and additional black-and-white prints at $1.00 each.

SELECTED ASPECTS OF COLOR FILM PROCESSING AND PRINTING

One of the more important phases, and often neglected, is the extreme care that is necessary in the photographic laboratory regarding general cleanliness and prevention

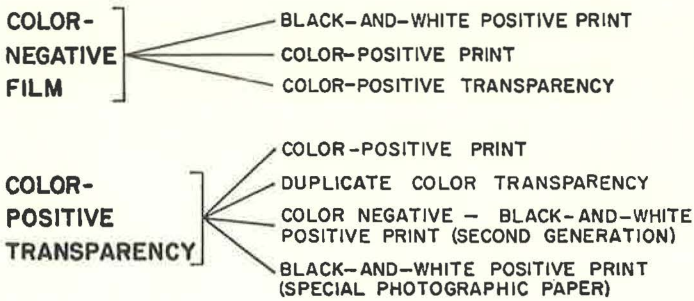

Figure 1. Possible products from color-negative and reversal-type film.

of contamination of chemical solutions. In addition, processing procedures must be rigidly controlled. The general attitudes, habits, and procedures followed in most black-and-white photographic laboratories are not good enough.

One phase of color processing that is often misunderstood is the time required in the photographic laboratory. For the most part, the total time required to process a roll of exposed aerial color film is only slightly longer than that required for black-and-white photography if the proper equipment is available. Color reversal film takes slightly longer to process than color negative film. Although color processing time does not differ significantly from the time for black-and-white, the complete attention of the laboratory technician is required throughout the color processing operation. This is not the case with black-and-white photography.

Two governmental agencies report that their production capacity for black-and-white prints is from two to two and one-half times greater than that for color prints. This comparison illustrates the additional time required to make color prints with the equipment used in these particular photographic laboratories. The type of aerial color film processing equipment used by most laboratories is of the so-called "wind and rewind" type. Continuous processing, as is used in the motion-picture industry, is not made to accommodate aerial color film. Continuous processing equipment is quite expensive, and at the present time is not justified by the quantity of aerial color film processed.

Contact printers, either conventional or automatic-dodging type, equipped with full spectrum tubes enable color prints to be made from either positive transparencies or color-film negatives. By means of special filters or by adjustment of the printing light source, the color balance of each photograph, can be regulated. Compensation for haze can also be made in the printing process by removal of some of the blue color by means of filters. Differences in film exposure and light quality (time of day) can also be compensated for in printing. Automatic-dodging printers enable a more uniform density to be obtained throughout a given photograph and tend to made the photographs in a flight strip more nearly uniform. Some objection has been voiced regarding the use of automatic-dodging printers that, with some films, cause serious color balance shifts due to reciprocity failure (longer exposure time in printing required). Attainment of perfect color balance in each print is still a costly and time-consuming process.

Figure 1 shows the possible products (contact size) that may be obtained from color-negative and reversal-type films. Enlargements from both types of film can also be made.

VIEWING AERIAL COLOR PHOTOGRAPHS

Optimum stereoscopic viewing of aerial color photographs presents a few special requirements. For optimum color perception it is necessary to have a light source that radiates energy over the entire color spectrum. Some modern fluorescent and incandescent lights and combinations of fluorescent and incandescent lights meet this requirement. Most of the light sources used for photographic interpretation of aerial color photography in the past have been of the "home brew" variety. Recently, special fluorescent tubes coated with selected phosphorus providing energy in the entire color spectrum have been made available. This light source is color balanced at 3,900 K and has intensities of 400 and 600 ft lamberts. The higher intensity is used for viewing dense transparencies. Diffusers provide an even distribution of light. This special light source has the added advantage of maintaining its color balance and intensity of output for long periods of time, something ordinary fluorescent light cannot do. As with other fluorescent light sources, heat dissipation is not a problem as it is with incandescent bulbs.

Double projection photogrammetric instruments currently in use have not been designed for use on aerial color photographs. Use of color aerial photography in these instruments, both qualitatively and quantitatively, has up to this time been experimental in nature. Color positive transparencies sandwiched between glass plates have been used in the Kelsh stereoscopic plotter by means of polarized light and a special platen with a surface of anodized aluminum. Despite its limitations, this approach appears to have considerable merit, particularly for interpretation purposes (5). Color transparencies have also been experimentally employed in optical-train photogrammetric instruments. Because these instruments do not depend on the anaglyphic principle for their operation, polarized light is not required for stereoscopic viewing. This approach has proved successful for qualitative use.

Predictions have been made, perhaps optimistically, that color diapositives for use in photogrammetric plotting instruments will be available within the next five years. Color diapositives would eliminate the dimensional instability problem of positive transparencies. However, there is still the problem of either coating or laminating color emulsions on glass plates. The latter procedure appears more promising. Although the commercial film companies and others have not seriously tackled this problem in the past, some token progress has been made and the results offer some encouragement. A breakthrough on this problem should result in more extensive use of color photographs in photogrammetric instruments both quantitatively and qualitatively.

Although not specifically designed for the 9- by 9-in. format used extensively in highway work, a stereoscopic-viewing instrument of interest to the military and others has recently been developed. This instrument permits the viewing of projected 70-mm strip color and conventional photography in three dimensions without the aid of polarized light or colored lenses. This is accomplished by a special projection system and viewing lens. Magnification is also provided.

RELATIONSHIP BETWEEN QUALITY OF COLOR REPRODUCTION AND PHOTOGRAPHIC INTERPRETATION

Establishment of a set of rigid criteria for evaluating the acceptability or nonacceptability of aerial color photographs for use in materials surveys is nearly impossible and at best not practical. General requirements applying to black-and-white photography procured for interpretation purposes are applicable to color photographs. The value of true color reproduction in interpretation of soils, locating sources of construction materials, and mapping geologic formations is questionable when considered in terms of the effort and cost of attaining this objective. Although it would probably be aesthetically desirable to obtain true color reproduction, the effort and cost is at present too great to warrant it. Actually, color distinctions and differences are of paramount importance, and it may sometimes be necessary to disturb good color balance to attain certain color contrasts.

Proper film exposure enhances color differences, and is particularly important with color reversal film because there is little leeway in the photographic laboratory to compensate for errors. Overexposure generally tends to "wash out" colors. Latitude of exposure varies considerably with various color films and with some films it may be desirable to underexpose rather than overexpose. The reverse may also be true for other films. For critical work, test exposures should be made with the same film (same emulsion number) as is to be used on the project. Laboratory procedures should be standardized.

Resolution in an aerial photograph is an important consideration. It may, however, be less important on a color than on a black-and-white photograph because color rendition and contrasts are probably more important in differentiating images. Aerial color films generally have much greater contrast than color films produced for ground photography. Aerial color photographs with high image resolution are certainly desirable, but other considerations may be more important for photographic interpretation. Where photographs are to be viewed under magnification or where enlargements are to be made, resolution assumes greater importance.

RESEARCH NEEDS

The available knowledge related to use of aerial color photographs for interpretive purposes is still relatively small. Because most state highway departments do not have the time, trained specialists, facilities, or funds to undertake comprehensive research projects of a basic nature, most research in this area has been conducted by universities and agencies of the Federal Government. Though considerable research in this field has been conducted by the military, the results of this work have not generally been made available to civilian engineers and scientists because of security restrictions.

The selection of the proper film and/or filter for use on specific projects having unique color combinations needs to be investigated. Some research of this type has been done on a trial-and-error basis using various film-filter combinations and film exposures. A limited amount of work has been done by an approach that eliminates most of the guesswork, by using spectral reflectance measurements (spectro-photometric studies) of the soils and rocks to be photographed. The results of these measurements show the dominant wave lengths of reflected light thereby enabling the photographer to select an appropriate filter that will give optimum color contrasts (6, 7).

Aerial photographic interpretation has in the past been largely qualitative and subjective in nature. Although the amount of information that can be obtained is quite large, the quantitative approach to interpretation may possibly be the approach that really "pays off." This approach has long been neglected and is in need of further study. By use of a densitometer or microdensitometer, color transparencies or prints can be scanned and the intensity of specific wave lengths of light either transmitted (transparencies) or reflected (prints) can be measured and recorded. Normally, measurements of the amounts of red, blue, and green light are made. Particular rock types, soils, and landforms can then be identified and differentiated. In this manner, distinctions can often be made among geologic formations and soils that are impossible to make visually (6, 7, 8).

Experimentation is needed with all types of aerial color film to determine the latitude of film exposure in relation to the value of the resulting photographs for interpretive purposes.

Research involving the use of various combinations of light sources, reflectors, filters, and light intensities in color-viewing systems for interpretation is needed. This phase of enhancing color differences for interpretation has been neglected.

The potential of false-color films for interpretive purposes needs to be evaluated. False-color films differ from ordinary color films in that one of the three emulsion layers is sensitive to infrared. Colors produced by this film after processing are not those of the objects (soil or rock) photographed. Color contrasts and differences depend to a large extent on the differences in infrared reflectivities of the scene photographed. This film has an advantage over conventional infrared film in which the record is in shades of gray (4, 9).

Controlled experiments in aerial color film deterioration are needed to ascertain the degree of stability of dyes under ideal storage conditions and in routine office and field use. The effect of certain chemicals used on color prints to retard the bleaching of colors should be ascertained. Normally, color prints are soaked in these solutions. When dried, the then invisible coating reduces the penetration of ultraviolet rays which cause bleaching of dyes.

Particular types of vegetation have been correlated with specific soils and ground conditions and can be used by trained interpreters to identify these conditions on aerial photographs. Recognition of vegetative types is greatly facilitated by means of aerial color photographs. Research to correlate identifiable vegetation on aerial photographs with specific conditions and materials on the ground is needed. False-color films may have some application and potential use. It is probable that this approach to interpretation will have its greatest success in semi-arid and subarctic regions. There is need for more test flights and use of aerial color photographs over a broader range of climate, topography, and geologic materials.

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Predictability of Certain Properties of Soil-Water Mixtures

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Results of consistency tests performed on cement pastes, cement mortars, and concretes show that the relationship between the change in consistency and the change in water content can be approached by a certain power function within practical limits. A similar approach for soil-water mixtures was recommended earlier by the Waterways Experiment Station as a "rapid" or "one-point" method for the simplification of the liquid limit test procedure. In this paper a simple physical interpretation of this approach by power function, called "assumption of independence," is presented. Also, three other approaches concerning the relationship between the consistency and water content of soil-water mixtures are analyzed mathematically, as follows: (a) assumption of a logarithmic relationship; (b) assumption of a point of convergence; and (c) assumption of a hyperbolic relationship.

The equations expressing the assumptions can be derived from each other; thus, the four methods are essentially four different mathematical descriptions of the same phenomenon concerning the soil consistency as a function of water content. Data are presented from the technical literature which indicate that the assumption of independence might also be used as a simple approach to soil properties other than the consistency.

•RESULTS of consistency tests on cement pastes, cement mortars, and concretes show that the relationship between the change in consistency and the change in water content can be approached by certain power functions within practical limits. A similar approach was previously recommended for soil-water mixtures. A simple physical interpretation of the approach by power function, called "assumption of independence," is presented. Three other assumptions concerning the relationship between the consistency and water content of soil-water mixes, are analyzed mathematically. A well-known utilization of any of these assumptions is to simplify the determination of the liquid limit, or more precisely, to predict the liquid limit of soils by so-called "one-point" or "rapid" methods. The approximation by power function, however, could be also used advantageously for soil properties other than consistency.

The consistency of a soil-water mixture will be characterized numerically in the usual way, i.e., by the number of drops of the cup of a standard mechanical liquid limit device at which the two halves of a soil cake will flow together for a distance of ½ in. along the bottom of the groove separating the two halves, when the test is performed according to the "Tentative Method of Test for Liquid Limit of Soils," ASTM Designation: D423-61T. The numerical characteristic of the consistency is called "consistency measure" in this paper.

ANALYSIS OF ASSUMPTIONS

The Relationship of a Power Function

The earliest assumption for the change in consistency, where change is due solely to the change in water content, is the assumption of the relationship of a power function.

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This was first proposed by the U. S. Waterways Experiment Station (1) for the prediction of liquid limit of soil-water mixtures. The pertinent power function can be written as

$$w = aN^{b} \tag{1}$$

in which

- w = water content of the soil-water mixture, percent by weight;
- N = number of drops of the cup of the standard device, i. e., the consistency measure;
- a = factor which is a function of the soil type; and
- b = factor which is constant for every soil but depends on the method of measuring consistency. (For the standard method for soils, the following value of "b" is recommended by ASTM D423-61T: b = -0.12.) It is characteristic of the rate of change in consistency.

The meaning of Eq. 1 can also be expressed as follows: The amount of (relative) change in consistency which is due solely to the (relative) change in water content is independent of the type and original consistency of the material.

This statement is called the "assumption of independence." Obviously, the assumption of independence is valid only to the extent that the assumptions expressed by Eq. 1 are valid.

To illustrate the assumption of independence, assume that a certain soil sample of 100 g and w = 77 g water requires N_1 = 20 drops of the cup to close the groove. Also, assume that a reduction of 2 g water (2.6 percent of the original water content) will increase the number of needed drops to N_2 = 25. If the assumption of independence is valid, then this 2.6 percent reduction of water content will result in the increase of the required drops from 20 to 25 for every soil.

In this example the relative value of the water reduction, and the relative value of the change of consistency were as follows:

$$\Delta w = 100\,(77 - 75)/77 = 2.6 \text{ percent decrease}$$

and

$$\Delta N = 100\,(25 - 20)/20 = 25 \text{ percent increase.}$$

The accuracy of the approximation by a power function, particularly its practical applicability to predict the liquid limit, has been checked by several investigators: the liquid limits of various soils were determined by the customary three-point method and predicted by the one-point method which is an approximation by a power function; then the results of the two methods were compared for significant differences. The analyses of the test results showed that the overwhelming majority of these differences were small within certain limits of validity (1 - 5) even though these differences also contained the experimental errors of the three-point method (6). Opinions differ slightly concerning the exact value of "b" power, the limits of validity, and the extent of accuracy of the approximation; nevertheless, it seems established that the application of a power function for the relationship between consistency and water content is, within certain limits, acceptable. However, any modification in the test method of measuring consistency can cause a significant change in the value of the power "b" (7).

The use of the approximation by power function on concrete consistency was explored in previous papers (7, 8) without knowledge of the proposition of the U. S. Waterways Experiment Station. In these papers the identity of the assumption of independence and Eq. 1 is mathematically shown.

Another simple proof is based on the fact that the values calculated from Eq. 1 with various "a" factors will give a family of parallel straight lines (so-called "flow curves") when plotted in a log-log system of coordinates (Fig. 1).

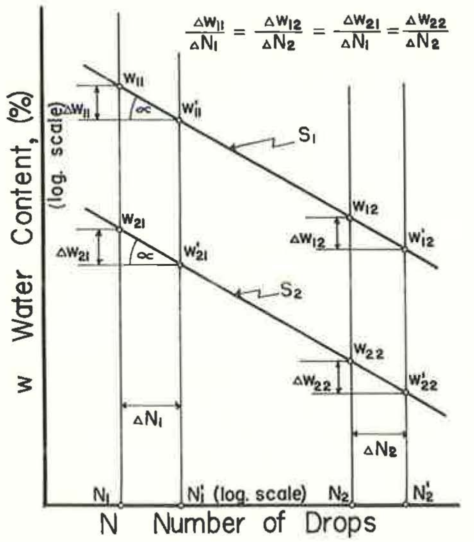

Figure 1. Consistency and water content relationship using a power function.

Proof:

1. That the relative change of consistency is independent of the N_i original consistency, follows from the fact that the flow curve gives a straight line in the log-log system; because if the following designation is used

$$\Delta w_{ij} = \log (w_{ij}/w'_{ij}) \tag{2}$$

and

$$\Delta N_i = \log (N_i/N'_i) \tag{3}$$

then the S_1 straight line of Figure 1 fulfills the equation as follows:

$$\Delta w_{11}/\Delta N_1 = \Delta w_{12}/\Delta N_2 \tag{4}$$

regardless of the value of N_i.

2. From the parallelism of any two S_1 and S_2 flow curves it follows that the relative change of consistency is independent of the type of the soil because (from Fig. 1):

$$\Delta w_{11}/\Delta N_1 = \Delta w_{21}/\Delta N_1 \tag{5}$$

regardless of the value of w_{i1} which value is indicative of the type of soil.

Various Forms for the Approximation by Power Function

Various forms can be derived from Eq. 1 for the approximation by a power function, i. e., for the assumption of independence. One such form is:

$$N_2 = N_1 (w_2/w_1)^n \tag{6}$$

in which

N_1 = consistency measure for the initial consistency;
N_2 = consistency measure for the changed (predicted) consistency;
w_1 and w_2 = water contents for the consistency measures of N_1 and N_2, respectively; and
$n = 1/b$ = test method constant.

Another form is as follows:

$$N_2 = N_1 (0.01\, w_{rel})^n \tag{7}$$

where $w_{rel} = 100\, w_2/w_1$ = relative water content in percent.

A further form is

$$N_{rel} = 100\, (0.01\, w_{rel})^n \tag{8}$$

where $N_{rel} = 100\, N_2/N_1$ = relative consistency measure in percent

And finally (10):

$$Y = 100\left[(1 + 0.01X)^n - 1\right] \tag{9}$$

where $X = 100(w_2 - w_1)/w_1$ = relative change in water content, percent, and $Y = 100(N_2 - N_1)/N_1$ = relative change in consistency measure which is due to the change of X in water content, percent.

Eqs. 6-9 provide further possibilities to visualize the extent of agreement between the experimental results and their approximation by a power function. For instance, the better the flow curves of various soils approach a family of straight lines in a log-log system, the better the approximation of Eq. 6. This principle has been used in soil mechanics to check graphically the accuracy of the approximation by a power function. The same principle is illustrated on consistencies of cement pastes in Figure 2 where results of Vicat penetration test (ASTM C187-58) are presented as a function of water content. The results were obtained by various investigators on a variety of portland cements (9, 11, 12). It can be seen that the Vicat penetrations form a family of parallel straight lines in fairly close agreement within the limits of 6 mm and 30 mm. Penetrations smaller than 6 mm or larger than 30 mm showed more considerable discrepancies from the parallelism, which gave a basis for the determination of the limits of validity for this approximation. The postulate for the parallelism of flow curves can also be used for the construction of a nomographic chart that makes easier the practical application of Eq. 6. Such nomographic charts for cement, mortar and concrete have been previously presented (13).

Results obtained on cement pastes indicate two other points: (a) if some kind of penetration test were used for the determination of liquid limits of soils (14) rather than the present standard liquid limit device, then a "one-point" method based on the assumption of independence could probably be developed for this new test also; (b) there is a good chance that a "one-point" method can be developed for the standard Vicat penetration test on cement pastes.

Eq. 7 provides another possibility for checking. If the water content related to the consistency of N_1 is w_{rel} = 100 percent, then the N_2 values calculated from Eq. 7 will form a single straight line in a log-log system, regardless of the original w_1 moisture content. Consequently, the approximation by a power function is accurate only to the

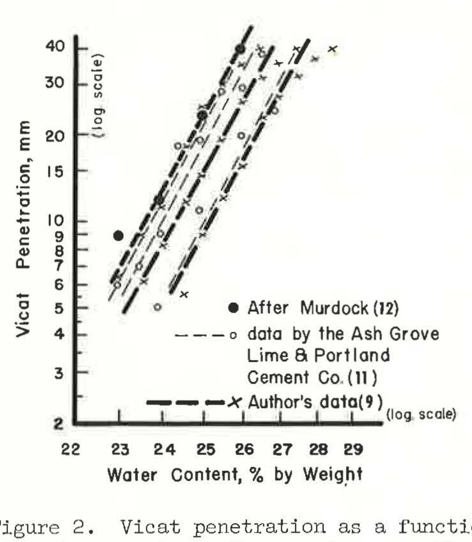

Figure 2. Vicat penetration as a function of water content for various portland cements.

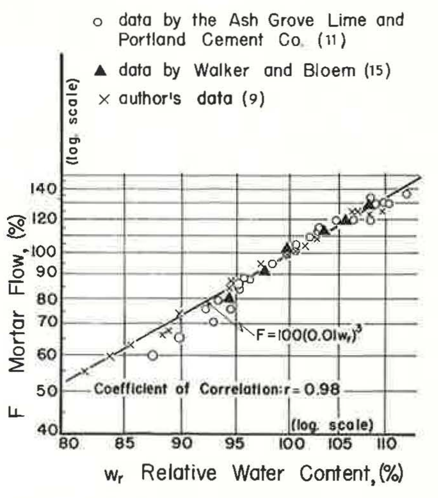

Figure 3. Flow of cement mortar as a function of relative water content for various portland cements (water content related to 100% flow is 100%).

extent that the experimental results approximate this straight line. Thus, from such a chart the value of the "n" power, the extent of approximation, and the limits of validity for the adequate approximation can be easily determined. This principle is illustrated in Figure 3 on consistencies of cement mortars where results of standard flow tests (ASTM C230-61T) are presented as a function of relative water content. The results were obtained by various investigators on a variety of cement mortars (9, 11, 15). Within the flow limits of 55 percent and 130 percent, the flow results form a straight line with a good approximation. This is also shown by the significantly high value of the coefficient of correlation. The discrepancies of flow values smaller than 55 percent or larger than 130 percent were considerable higher. Thus, Eq. 7 provides a comfortable visual method for checking the accuracy of the approximation by power function. In addition, the value of "n" power and the limits of validity for the adequate approximation can be determined easily from such a chart.

Eqs. 8 and 9 again reveal the identity of the approximation by power function and the assumption of independence. They show directly that a fixed relative change takes place in the consistency as a consequence of a given relative change in the water content, and this fixed change is independent of the original moisture content and original consistency. If, for instance, a Δw_1 percent reduction in water content makes a change in the N_1 consistency measure (for example, the number of blows changes from 15 to 16 which is an increase of $6\frac{2}{3}$ percent) then, according to the approximation by a power function or assumption of independence, the same Δw_1 percent reduction in water content will cause a $6\frac{2}{3}$ percent increase in the consistency measure when $N_1 = 30$. In other words, in the latter case the consistency measure will change from 30 to 32. The application of this principle is particularly useful for concrete consistency, and for several other soil properties.

Figures 2 and 3, as well as previous papers (8, 9), show that the presented approximation by power function, and at the same time the applicability of the assumption of independence, far exceeds the standard mechanical liquid limit device.

Assumption of a Logarithmic Relationship

A logarithmic form was recommended for the rapid determination, or more precisely, for the prediction of liquid limit by Cooper and Johnson (6), and later by Fang (7). The flow curve of this assumed logarithmic relationship gives a straight line in a w (linear) and N (log) semilogarithmic system of coordinates (Fig. 4). Also, Fang assumed that the slope of a flow curve in Figure 4, which he called "flow index" and designated by I_f, is a linear function of w_{i1} water content which belongs to any fixed N_1 consistency. (If $N_1 = 25$, then $w_{i1} = LL$ = Liquid Limit). The mathematical expression of this relationship is

$$w = w_{i1} + I_f \log N_1 - I_f \log N = A - I_f \log N \qquad (10)$$

and

$$I_f = Bw_{i1} + C \qquad (11)$$

where A is independent of N but is a function of the type of soil; and B and C are constants for every soil and consistency.

With a certain restriction, Eqs. 10 and 11 can be derived from Eq. 1. In other words, Eq. 1 and Eqs. 10 and 11 are saying essentially the same thing concerning the soil consistency as a function of water content—only the form of expression is different.

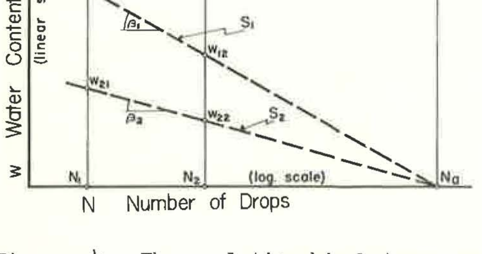

Figure 4. The relationship between consistency and water content approached by a logarithmic function.

Proof:

It can be seen from Figure 1 that for

any two fixed N_1 and N_2 values of consistency the following is valid:

$$w_{11}/w_{12} = w_{21}/w_{22} = \text{const.} \tag{12}$$

without respect to the type of soil, because N_1/N_2 = constant. On the other hand, if points w_{11} and w_{12} of the flow curve S_1 (Fig. 1) are represented in the semilogarithmic system of Figure 4, then the flow index, i. e., the slope of the straight line which passes through the points w_{11} and w_{12}, can be expressed, as follows:

$$\text{tg}\,\beta = I_f = \frac{w_{11} - w_{12}}{\log N_2/N_1} = w_{11}\,\frac{1 - w_{12}/w_{11}}{\log N_2/N_1} \tag{13}$$

In Eq. 13, however, the factor of the w_{11} term is a constant because, from Eq. 12, w_{11}/w_{12} is a constant; therefore,

$$I_f = Bw_{11} \tag{14}$$

It can be seen that Eq. 14 is a particular form of Eq. 11 when C = 0. Consequently, if and when C = 0, then Eq. 1 and Eqs. 10 and 11 are not independent of each other.

Although the assumption of a power function and the logarithmic assumption are "essentially" the same, the identity is not complete even when C = 0. A flow curve of a power function which gives a straight line in the log-log system of coordinates will resolve into a curve in the semilogarithmic system. The substitution of a straight line for this curve is tolerable because the logarithmic form represents the first approximation of the power function formula. The approximation is obtained by omitting the terms of second and higher degree from Taylor's series of the power function.

Proof:

$$w = aN^b = a\,\exp\,(b\ln N) = a\,(1 + b\ln N + R) \cong A\,(1 + c\log N) \tag{15}$$

where R is the remainder term of the Taylor's series; and c and A are constants. The other symbols correspond to those of Eq. 1. If the c term is written

$$c = -I_f/A \tag{16}$$

then Eq. 15 will give Eq. 10 what was to be proven.

Figure 5 shows that the agreement between the P flow curve of a power function and the L logarithmic flow curve is good within fairly wide limits, i. e., the R remainder term is small. The same agreement is also shown numerically in a paper by Joslin and Davis (18).

Assumption of a Point of Convergence

Olmstead and Johnston (19) showed that the flow curves of various soils converge toward a point in a semilogarithmic system of coordinates which point was near the zero moisture-content axis. By arbitrarily moving this N_a point of convergence to the zero axis for the sake of simplicity, the flow curves formed a family of straight lines having the following equation (Fig. 4):

$$w = w_{i1}\,\frac{\log N_a}{\log N_a/N_1} = \frac{w_{i1}\log N}{\log N_a/N_1} = D - E\log N \tag{17}$$

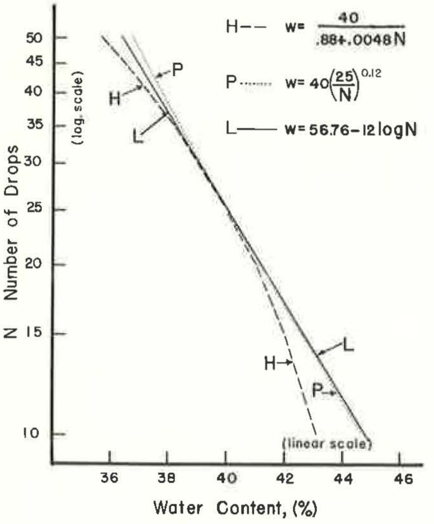

Figure 5. Comparison of three formulas for the relation of water content vs consistency.

where the values of D and E are determined by the position of the point of convergence, and the type of soil.

From Figure 4, however, the factor of the log N term in Eq. 17 is nothing else but the slope of the flow curve; therefore,

$$E = I_f = \frac{w_{il}}{\log N_a/N_1} = F w_{il} \qquad (18)$$

Thus, Eq. 17 is identical to Eq. 10, i. e., D = A; and Eq. 18 is identical to Eq. 11, i. e., F = B, if C = 0. This means that the arbitrary moving of the point of convergence to the zero moisture-content axis is equivalent to making arbitrarily C = 0 in Eq. 11; that is, in the case of a fixed N_a point, the slope of a flow curve in Figure 4 changes proportionally to the value of w_{il}.

Consequently, the assumption of a point of convergence expresses again essentially the same thing as the approximation by power function or the logarithmic relationship, only the form is different.

The identity of these assumptions makes it possible to convert from one to the other. For instance, with the knowledge of the I_f flow index and LL liquid limit, the location of the N_a point of convergence on the N-axis can be calculated by Eq. 18.

$$\log N_a = 1/F + \log 25 = LL/I_f + 1.4 \qquad (19)$$

Eq. 19 provides another possibility to check experimentally the validity of the assumption of a logarithmic relationship, or the assumption of a point of convergence.

Assumption of a Hyperbolic Relationship

The relationship between the soil consistency and water content can also be approached by a hyperbolic formula. After Mohan and Goel (20), by using the designation of Figure 4, this formula can be written

$$w = w_{il}/(d + eN) \qquad (20)$$

or

$$w = 1/(f + gN) \qquad (21)$$

where the values of d and e are the same for every soil; and the values of f and g depend on the type of the soil. If, for instance, w_{il} = LL, then d = 0.88 and e = 0.0048 (20).

The assumption of a hyperbolic relationship is related again to the previously discussed assumptions, because Eq. 20 was obtained from Eq. 1 by an expansion into binomial series and omitting the terms of second and higher degrees. Mohan and Goel found that this hyperbolic approximation is acceptable when the number of blows is within 20 and 30. The agreement between the hyperbolic approximation (curve H) and two other approximation (curves F and L) is shown within wider limits in Figure 5.

FURTHER APPLICATIONS

The approximation by power function has an important interpretation—the assumption of independence which is always applicable whenever the approximation by power function is applicable. The significant point is that the exploitation of the assumption of independence can simplify to a great extent the prediction of certain soil properties. Thus, it is interesting to examine whether the approximation by power function is applicable for properties of soil-water mixtures other than consistency, and if so, to show the simplifying influence of the assumption of independence on the prediction. For this purpose results of four test series, taken from the technical literature, are analyzed. This presentation is confined to test series which were readily available; no thorough study of the literature was undertaken.

Penetration Resistance

It appeared logical to try to apply the approximation by power function for the relationship of soil strength versus moisture content because this can be considered as an extension of the consistency test for soil-water mixtures that are too dry for the standard liquid limit device.

Figure 6 shows the so-called Ohio typical moisture content-penetration curves in a log-log system. The original set of Ohio typical curves was based on the results by Woods and Litehiser. The set of curves presented is that part of the Ohio typical curves shown in a study by Johnson and Sallberg (21, Fig. 101) which is within the limits of 50- and 1,000-psi penetration resistance. The curves form a family of parallel straight lines with a fairly good approximation; i. e., the approximation by power function is fairly good. The discrepancy is the largest in the first curve, but it is not serious. For instance, where the experimental value is about 110 psi, the value from the power function is 100 psi. Consequently, it seems possible to develop a "one-point" or "rapid" method for the prediction of penetration resistance as a function of the moisture content only. For instance, by using the form of Eq. 7 and choosing the water content as 100 percent relative water content which is related to the penetration resistance of 500 psi, the following formula can be applied for the prediction of the presented penetration resistance of Ohio soils:

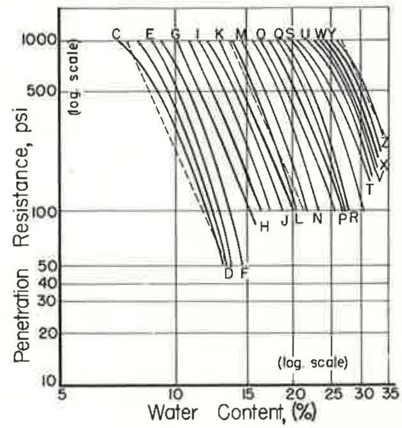

Figure 6. Ohio typical moisture content-penetration resistance curves (21).

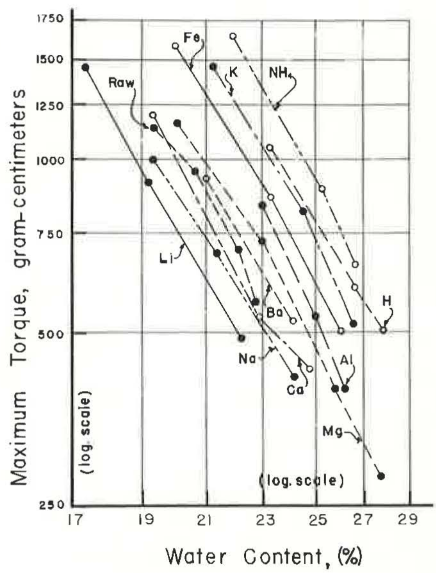

Figure 7. Effects of various cations on the shearing strength of a clay (22).

$$P = 500\,(0.01\,w_{rel})^{-5} \qquad (22)$$

in which

P = penetration resistance, psi; and
w_{rel} = relative water content, percent.

Also, by applying the assumption of independence, this whole set of typical curves can be substituted, in a sense, by the following rule of thumb: a fixed relative change takes place in the penetration resistance as a consequence of a given relative change in the water content, and this fixed change is independent of the soil type, original moisture content, and original penetration resistance. This simple rule is expressed mathematically by Eq. 8 or 9. For example, a 20 percent relative increase in moisture content (e. g., from 15 to 18 percent) will reduce the penetration resistance to about one-third of the original value for the presented Ohio soils, regardless of the original strength, and original moisture content.

The approximation by power function is a simple method. In addition, the application of the assumption of independence produces very simple, practical rules that not only visualize the conclusions of the test series but, through the elimination of the irrevelant variables, contribute to the better understanding of the mechanism of the phenomenon.

Shearing Strength of Clay

In Figure 7 the shearing strength of a clay is plotted in a log-log system as a function of water content and type of absorbed cation. The data, originally obtained by Sullivan, were taken from Tschebotarioff (22, Figs. 7-29). The test results form a family of parallel straight lines with a good approximation, i. e., the approximation by power function is good. In this case, the various cations influence the shearing strength of the clay differently; nevertheless, as the rule of thumb based on the assumption of independence states, the same relative increase in the water content causes the same relative decrease in the shearing strength for every cation, every original moisture content, and original shearing strength. For example, a 5 percent relative increase in the water content will reduce the presented values of the shearing strength by about 20 percent regardless of the other variables involved.

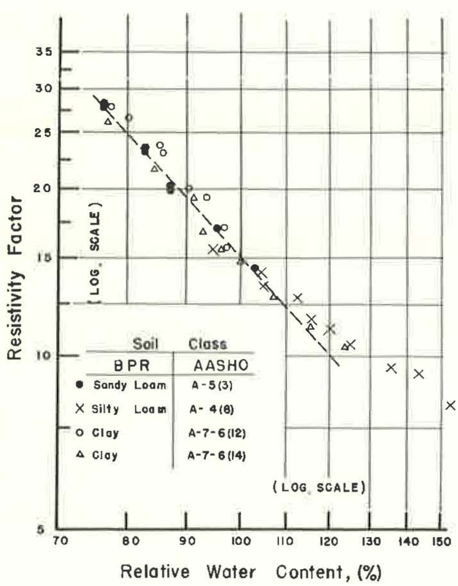

Figure 8. Electrical resistivity factor as a function of relative moisture content for four soils (23).

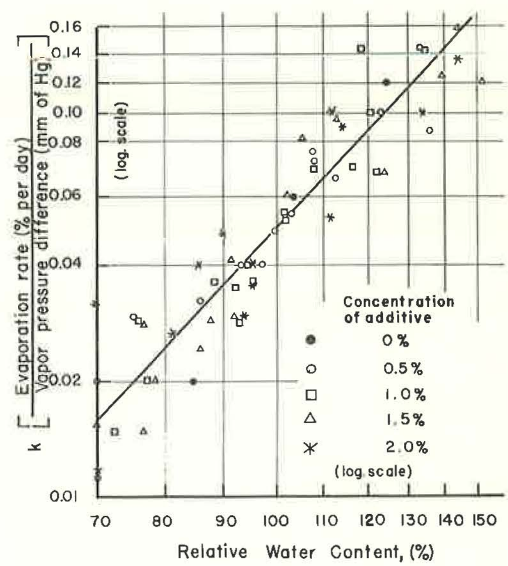

Figure 9. Index of moisture retention effectiveness for various additives as a function of relative moisture content (24).

Electrical Resistivity

Figure 8 shows the electrical resistivity factors of four soils mixed with brines of two concentrations as a function of relative moisture content. The data were taken from Sheeler et al. (23, Fig. 17). The resistivity factor is defined as "the ratio of the resistance of the soil-brine sample to the resistance of the brine." Figure 8 shows that the resistivity factors, within the limits of 75 and 120 percent relative moisture content, form a straight line with good approximation. Thus, within these limits, there exists a well fitting approximation by power function. Furthermore, a fixed relative change takes place in the resistivity factor as a consequence of a given relative change in the moisture content, original resistivity factor, and the concentration of brine.

Moisture Retention

Figure 9 shows the "k" index of moisture retention of various soil mixtures as a function of relative moisture content. The data were taken from Gow et al. (24, Figs. 9-14). The purpose of the test series was to investigate the effects of the treatments with calcium chloride, sodium chlorides, lignosulfonates, and molasses in various concentrations on the evaporation rate of soil-aggregate mixtures. Inasmuch as the test results again form a straight line with good approximation within the limits of 70 and 140 percent of relative moisture content, both the approximation of a power function and the approximation of the assumption of independence are good for the data presented. Thus, the conclusion of this test series can be summarized very simply; for example: the relative change in the evaporation rate, due solely to a given relative change in the moisture content, is practically the same for all six additives and is also independent of the original moisture content and the concentration of the used solution.

CONCLUSIONS

1. The four methods express essentially the same statement concerning the soil consistency as a function of water content; only the form of expression is different. Thus, it is understandable that the values calculated by the various formulas show good agreement within practical limits (Fig. 5).
2. From a practical point of view, it is difficult to give preference to any of these methods. It might well be that the form of the logarithmic approximation recommended by Fang has wider limits of validity because Eq. 11 contains two experimental constants, whereas the corresponding equations of the other methods contain only one. This is, however, at the expense of simplicity.
3. The approximation by power function is advantageous in principle because of its significant physical interpretation, the so-called assumption of independence.
4. When the results of a test series on a soil-water mixture, plotted in a log-log system, provide a family of parallel straight lines as a function of water content, or a single straight line as a function of relative water content, several of the tested variables are irrevelant in a sense. Such variables may be the original moisture content, original consistency, and type of soil. Therefore, the tested relationship can be expressed in a form where the number of variables is reduced. Consequently, the application of the assumption of independence produces simple practical rules that not only visualize the conclusions of the test series but, through the elimination of the irrevelant variables, contribute to better understanding of the mechanism of the phenomenon.
5. Figures 6 through 9 seem to indicate that the approximation by power function and the assumption of independence might be used for the prediction of various properties of soil-water mixtures with an accuracy that is satisfactory for many practical purposes. Therefore, it would be worthwhile to undertake more thorough investigation to see to which soil properties this assumption is applicable, what the degree of the expected accuracy is, and what the limits of validity are.

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Study of the Reproducibility of Atterberg Limits

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The Atterberg limits have become some of the most extensively used soil tests in highway engineering. In addition, numerous earthwork design and construction specifications are set up on the basis of these limits. Frequently, the reproducibility of the Atterberg limit tests is questioned. Nevertheless, little work has been done to investigate the various aspects of operator variability.

A statistically controlled experiment was performed to investigate how well an operator can reproduce the Atterberg limits and the effects of an operator's experience on the test results. Two operators, one with considerable experience and the other with practically no experience, performed a series of liquid and plastic limit tests on three different soils. The statistical analysis of the test results revealed the following:

1. There are variations in the Atterberg limit values. However, the magnitude of the variations are relatively small. The Atterberg limits can, therefore, be regarded as reproducible from the engineering standpoint.
2. The amount of experience an operator has does affect the variations of the Atterberg limit values.
3. The plasticity index values are most variable, and the liquid limit values are least variable with the plastic limit values occupying an intermediate position.

This paper includes a discussion of the specific numerical values on which these general conclusions are based. A method based on quality control techniques is also proposed for the training of technicians in performing Atterberg limit tests.

•IN CASAGRANDE'S Atterberg limits paper (1), several factors that may cause irregularities in Atterberg limits were discussed. Shook and Fang (2) conducted an investigation on the variability that might be expected among several relatively untrained operators. They found that there was a significant difference between operators for both liquid and plastic limit values. This paper presents a study on the range and degree of operator variation in Atterberg limits values as well as the effects of operator experience on the test results.

Materials Tested

In order to cover a range of liquid limit values that are more commonly encountered, three Illinois soils were selected: a glacial till, a loess, and a glacial lake sediment. For designation convenience, the three soils are referred to as sandy silt, silt, and silty clay, respectively. The grain size distributions for these soils are shown in Figure 1 and the soil classification data are given in Table 1.

Test Procedure and Experimental Design

To provide uniform soil samples and to reduce variations among the samples, the processing and sample preparation of the three soils were undertaken with great care. After each soil was air dried, it was pulverized and thoroughly mixed in a Lancaster

Paper sponsored by Committee on Surveying, Mapping and Classification of Soils.

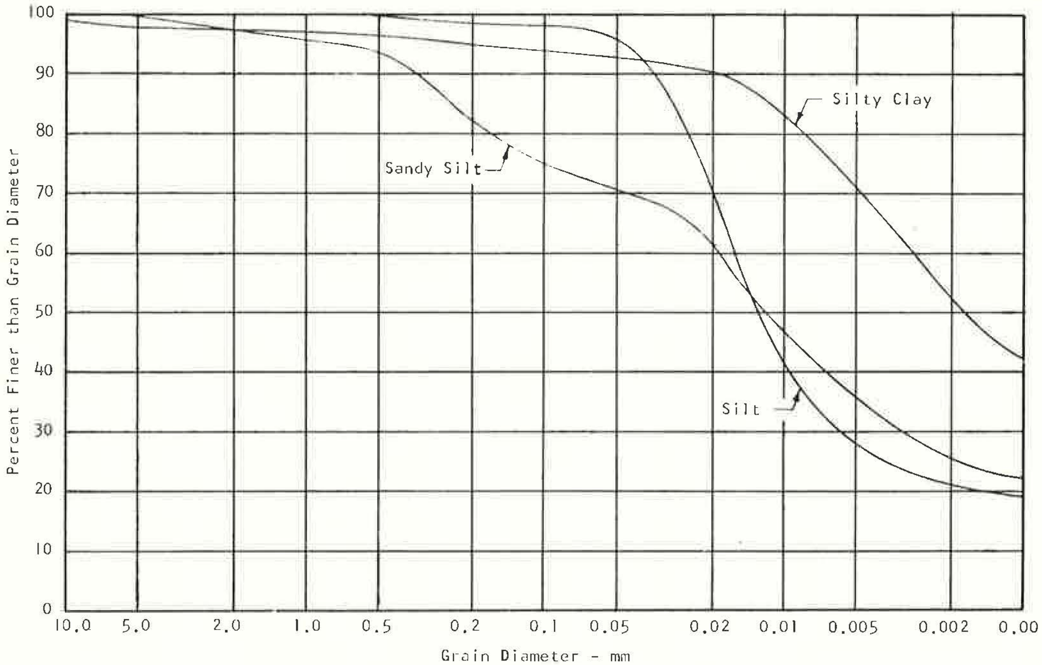

Figure 1. Grain size distribution curves.

TABLE 1

SOIL CLASSIFICATION

Soil Type	Passing No. 200 Sieve (%)	Liquid Limit (Avg.)	Plastic Limit (Avg.)	Plasticity Index (Avg.)	AASHO Group Classification	Group Index
Sandy Silt	73	27.6	16.2	11.4	A-6	8
Silt	98	34.5	23.0	11.5	A-6	9
Silty Clay	94	45.0	21.0	24.0	A-7-6	15

PC Mixer. Each soil was sieved through a No. 40 sieve. Using the sample splitter and standard quartering method, twenty 110-gm samples for each soil were obtained from the portion passing the No. 40 sieve. Each sample was placed in a covered paper carton.

In order to investigate whether the experience of the operator would have significant effects on the reproducibility of Atterberg limits, two operators were chosen to perform the tests. One operator had performed several hundred Atterberg limits tests while the other operator had no prior experience. Prior to the testing program, both operators were provided with copies of "AASHO Standard Specifications" (3) and "Introductory Soil Testing" (4). The inexperienced operator was also given three samples of a silty soil that he could use in familiarizing himself with the test equipment and procedure.

To eliminate the effects of day-to-day variations in conditions and operator behavior and to avoid personal bias in the interpretation of the results, the experiment was designed according to a statistical procedure. Each operator numbered his 30 samples (ten of each soil) from 1 to 30. The order of testing was determined from a table of random numbers. The liquid limit, plastic limit, and plasticity index were determined in accordance with the AASHO Standard Methods Designation T89-60, T90-56, and T91-54, respectively. A Casagrande grooving tool, however, was used in lieu of the ASTM grooving tool. Both operators used the same liquid limit device (with hard rubber base) and all the tests were conducted at the same location in the laboratory.

TABLE 2

TEST RESULTS

Soil Type	Liquid Limit		Plastic Limit		Plasticity Index	
	Experienced Operator	Inexperienced Operator	Experienced Operator	Inexperienced Operator	Experienced Operator	Inexperienced Operator
Sandy Silt	27.8	27.7	16.1	16.9	11.7	10.8
	27.6	28.0	16.0	16.7	11.6	11.3
	28.1	27.1	15.9	16.1	12.2	11.0
	27.2	28.7	15.8	17.4	11.4	11.3
	27.6	27.6	15.7	16.7	11.9	10.9
	27.2	28.0	15.0	15.6	12.2	12.4
	26.9	27.4	17.1	15.3	9.8	12.1
	27.2	27.4	16.2	16.3	11.0	11.1
	27.2	29.2	16.0	17.0	11.2	12.2
	27.6	27.2	16.5	15.3	11.1	11.9
Silt	35.0	33.8	22.3	22.5	12.7	11.3
	35.3	33.5	23.7	23.0	11.6	10.5
	35.0	33.6	23.3	23.7	11.7	9.9
	35.2	35.3	22.6	25.1	12.6	10.2
	34.5	33.2	21.3	21.9	13.2	11.3
	34.9	35.5	22.9	23.0	12.0	12.5
	33.9	32.6	24.2	21.8	9.7	10.8
	35.2	35.2	23.0	23.8	12.2	11.4
	35.3	34.0	23.2	23.8	12.1	10.2
	35.0	34.1	22.9	22.6	12.1	11.5
Silty Clay	44.7	44.5	21.8	19.9	22.9	24.6
	45.8	44.7	21.2	20.6	24.6	24.1
	44.6	44.5	21.3	20.3	23.3	24.2
	45.3	45.4	21.0	21.7	24.3	23.7
	45.8	45.1	21.1	20.6	24.7	24.5
	45.4	43.8	21.0	21.9	24.4	21.9
	45.3	45.0	20.8	20.5	24.5	24.5
	45.2	44.8	20.9	21.8	24.3	23.0
	45.8	44.6	20.7	21.8	25.1	22.8
	46.6	43.8	20.8	19.6	25.8	24.2

Analysis and Results

The complete test results are given in Table 2. The data were analyzed by the University of Illinois' Digital Computer Laboratory IBM 7094 System using various statistical analysis programs.

An analysis of variance was conducted on all the data that were grouped into the three test categories (liquid limit, plastic limit, and plasticity index). The results of this analysis (Table 3) show that (a) for each test highly significant differences exist among the three types of soil (F value for soil types is significant at the one percent level in all three instances); (b) operator experience had a highly significant influence on the mean liquid limit values (F value for operators is significant at the one percent level for liquid limit); (c) operator experience did not cause significant differences in the mean plastic limit values (F value for operators is not significant at the five percent level for plastic limit); (d) operator experience caused significantly different results in the mean plasticity-index values (F value for operators is significant at the five percent level); (e) the magnitude of variation in all three tests for each operator and each soil is relatively small (The variance for replicates is small in all three cases); and (f) the degree of variability of the tests increases from liquid limit to plastic limit to plasticity index (The variance for replicates increases in the aforementioned order. The variance for replicates is also referred to as the residual error that is an estimate of the inherent variation for the specific test. In other words, the the standard deviation (the square root of variance) for the liquid limit, plastic limit, and plasticity index is 0.62, 0.74, and 0.80 respectively).

The following basic statistics were calculated in order to study the magnitude of variations associated with the Atterberg limits tests for each soil type and operator.

TABLE 3

ANALYSIS OF VARIANCE

Source of Variation	Degrees of Freedom	Sum of Squares	Variance	F
(a) Liquid Limit				
Total	59	3100.8325		
Soil types	2	3072.2520	1536.1260	3995.12[a]
Operators (within soil types)	3	7.8175	2.6058	6.78[a]
Replicates (within operators and soil types)	54	20.7630	0.3845	
(b) Plastic Limit				
Total	59	524.4658		
Soil types	2	493.8863	246.9432	447.68[a]
Operators (within soil types)	3	0.7925	0.2642	0.48
Replicates (within operators and soil types)	54	29.7870	0.5516	
(c) Plasticity Index				
Total	59	2160.3733		
Soil types	2	2118.4843	1059.2421	1658.13[a]
Operators (within soil types)	3	7.3930	2.4643	3.86[b]
Replicates (within operators and soil types)	54	34.4960	0.6388	

[a]Significant at the 1 percent level.
[b]Significant at the 5 percent level.

1. Mean

The mean is the arithmetic average of all the individual values. For calculating the mean, $\bar{x}$,

$$\bar{x} = \frac{\Sigma X}{N} \tag{1}$$

in which

X = individual value, and
N = number of individual values.

2. Variance

The variance is the sum of the squared differences between the individual values and the mean divided by one less than the number of individual values. For calculating the variance, S^2,

$$S^2 = \frac{\Sigma (X - \bar{x})^2}{N - 1} \tag{2}$$

3. Standard Deviation

The standard deviation, S, is the square root of variance.

4. Coefficient of Variation

The coefficient of variation is the ratio of the standard deviation to the mean. For calculating the coefficient of variation, V, in percentage form

$$V\ (\%) = \frac{S}{\bar{x}} 100 \tag{3}$$

These values are given in Table 4. The standard deviation expresses the range of variation with respect to the mean. For the purpose of comparing the degree of variation associated with mean values considerably different in magnitude, it is often more helpful to use the coefficient of variation that expresses the standard deviation as a percentage of the mean. For ease of comparison, the coefficients of variation are shown in a bar diagram (Fig. 2).

From Table 4 and Figure 2 the following observations can be made:

1. Liquid limit
   a. Experienced operator. —The range of variation in liquid limit increases with the increasing plasticity of the soil. (Standard deviation increases with in-

TABLE 4

SUMMARY OF STATISTICAL DATA

Soil Type	Operator	Index Property	Mean	Standard Deviation	Coefficient of Variation
Sandy Silt	Experienced	Liquid Limit	27.4	0.36	1.31
		Plastic Limit	16.0	0.54	3.38
		Plasticity Index	11.4	0.71	6.20
	Inexperienced	Liquid Limit	27.8	0.67	2.41
		Plastic Limit	16.3	0.74	4.52
		Plasticity Index	11.5	0.59	5.15
Silt	Experienced	Liquid Limit	34.9	0.43	1.24
		Plastic Limit	22.9	0.79	3.43
		Plasticity Index	12.0	0.94	7.81
	Inexperienced	Liquid Limit	34.1	0.96	2.83
		Plastic Limit	23.1	1.01	4.35
		Plasticity Index	11.0	0.79	7.20
Silty Clay	Experienced	Liquid Limit	45.4	0.59	1.29
		Plastic Limit	21.0	0.32	1.52
		Plasticity Index	24.4	0.82	3.37
	Inexperienced	Liquid Limit	44.6	0.52	1.16
		Plastic Limit	20.9	0.86	4.12
		Plasticity Index	23.7	0.90	3.78

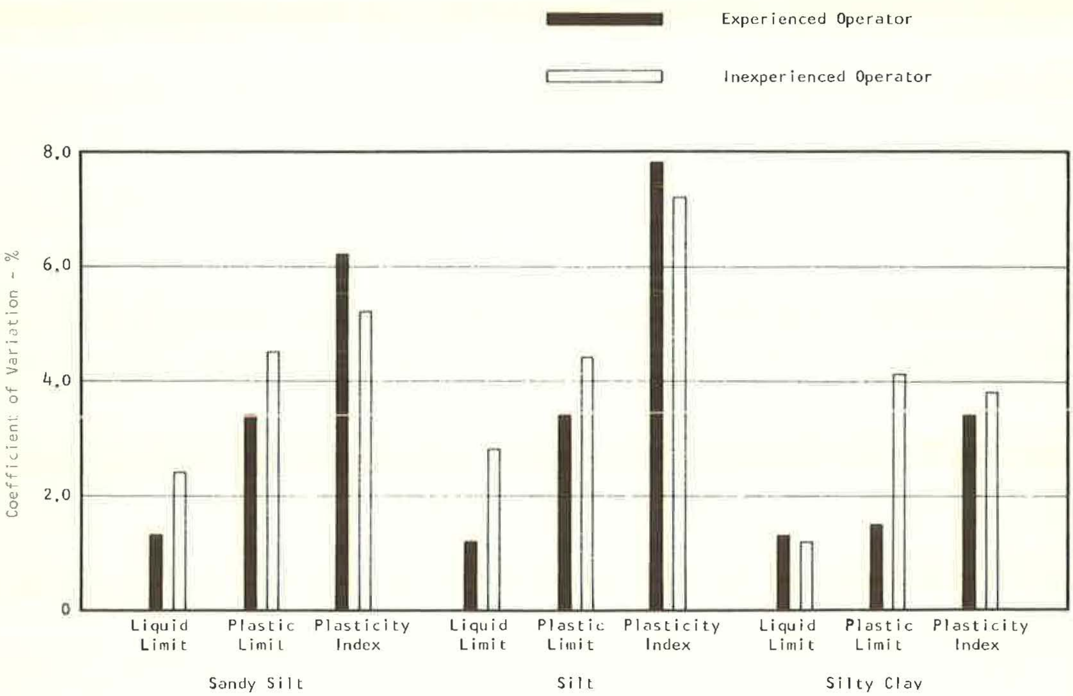

Figure 2. Coefficient of variation.

creasing mean values.) However, the relative degree of variation in liquid limit is practically the same irrespective of the plasticity of the soil (the coefficients of variation remain unchanged for all three soils tested). In other words, an experienced operator can be expected to produce a nearly constant degree of variation in liquid limit values for soils varying from low to moderately high plasticity.

b. Inexperienced operator. —Both the range and relative degree of variation in liquid limit for the silty clay are smaller than that for the other two soils.

This indicates that an inexperienced operator can reproduce the liquid limit value of a moderately highly plastic soil better than he can reproduce those of soils with low to moderately low plasticity.

c. Experienced operator vs inexperienced operator. —For soils with low to moderately low plasticity, the variation in liquid limit for the inexperienced operator is higher than that for the experienced operator. However, the amount of experience seems to have little effect on the variation in liquid limit of moderately highly plastic soil.

2. Plastic limit
   a. Experienced operator. —Both the range and relative degree of variation in plastic limit for the silty clay are smaller than that for the other two soils. In other words, an experienced operator can reproduce the plastic limit value of a moderately highly plastic soil better than he can reproduce those of soils with low to moderately low plasticity.
   b. Inexperienced operator. —The relative degree of variation in plastic limit is about the same for all three soils. Thus, for any soil within the range of plasticity investigated, an inexperienced operator can be expected to produce an approximately similar degree of variation in plastic limit.
   c. Experienced operator vs inexperienced operator. —Operator experience affects the variation in plastic limit because the experienced operator has a narrower range and a smaller degree of variation for all three soils. Consequently, an experienced operator can be expected to reproduce the plastic limit values better than an inexperienced operator.

3. Irrespective of operator experience, the variations in the plastic limit test are generally larger than those in the liquid limit test.

4. The plasticity index values are generally more variable than either the liquid limit or plastic limit values.

5. A detailed examination of the magnitude of standard deviation and coefficient of variation reveals the following:
   a. Range of standard deviation. —Liquid limit, 0.36 to 0.96; and plastic limit, 0.32 to 1.01.
   b. Range of coefficient of variation. —Liquid limit, 1.16 to 2.83; and plastic limit, 1.52 to 4.52.

Actually, these values are relatively small. This fact is shown in Figure 3 (a plot on the AASHO plasticity chart of the range of test results obtained by each operator for each soil). It is obvious that the results of the two operators show a small amount of variation. Thus, the liquid limit and plastic limit can be regarded as reproducible from the engineering standpoint.

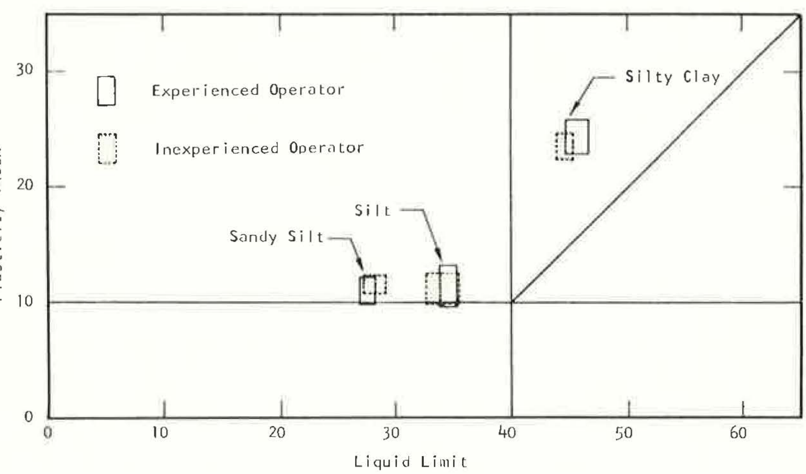

Figure 3. Range of test results.

TABLE 5
"T" VALUES

Soil	Liquid Limit	Plastic Limit	Plasticity Index
Sandy Silt	1.62	1.04	0.31
Silt	2.54[a]	0.45	2.66[a]
Silty Clay	3.37[b]	0.65	1.66

[a]Significant at the 5 percent level.
[b]Significant at the 1 percent level.

In the analysis of variance, it was found that there are highly significant differences in mean liquid limit values as determined by an experienced and an inexperienced operator. No significant differences are found in mean plastic limit values obtained by the two operators. However, the results of the analysis of variance will not reveal whether the significantly different mean values are caused by one soil or all three soils. In order to answer this question, the "t-test" (5) can be used to determine whether there is significant difference between two means—such as the mean liquid limit values of the sandy silt determined by two operators. Various "t" values comparing the mean values determined by the operators in each soil type were calculated. These data are given in Table 5. It can be seen that the mean liquid limit values obtained by these two operators are most significantly different for silty clay, significantly different for silt, and not significantly different for the sandy silt. No significant difference between the mean plastic limit values determined by the two operators is indicated for any of the three soils. The mean plasticity index values are only significantly different for the silt.

CONCLUSIONS

A statistically controlled experiment was performed to investigate an operatior's ability to reproduce the Atterberg limits and the effects of an operator's experience on the test results. Two operators, one with considerable experience and the other with practically no experience, performed a series of liquid and plastic limit tests on three different soils. The statistical analysis of the test results revealed the following:

1. There are variations in the Atterberg limit values. However, the magnitude of the variations as determined on the same equipment and according to the specified method and procedure is relatively small. The Atterberg limits can, therefore, be regarded as reproducible from an engineering standpoint.
2. The operator's experience does affect the variations of the Atterberg limit values. An experienced operator can be expected to reproduce the test results better than an inexperienced operator.
3. The plasticity index values are generally most variable, and the liquid limit values are least variable. The plastic limit values occupy an intermediate position.

ACKNOWLEDGEMENT

Gratitude is expressed to Eugene R. Wilkinson and Richard Miller who performed the two series of tests. Appreciation is also expressed to Samuel G. Carmer, Assistant Professor of Biometry, University of Illinois, for his assistance on the statistical analysis.

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Discussion

ROBERT D. KREBS, Associate Professor of Civil Engineering, Virginia Polytechnic Institute—It was found in our laboratory that a single operator could obtain appreciable differences in liquid limit. This was dependent on whether the soil was moistened or dried between successive trials and on the amount of spatulation employed. This was checked with an air-dried kaolinite soil that was mixed with water to above its liquid limit and cured for three days. As shown in Figure 4, a flow curve was established by successively drying the cured soil between blow-count determination until fifty blows were required to close the groove. Water was added at this point, between blow-count determinations and a second flow curve resulted. It was also found that after prolonged mixing, the cured soil appeared to gain strength with spatulation without changing water content. This phenomenon is illustrated by the horizontal line in Figure 5. The four points were determined within one minute at a nearly constant water content. Finally, with additions of water, the determinations progressed along a flow curve with a normal appearance. This strength gain phenomenon may be responsible for the effect shown in Figure 4.

Even though the soil was vigorously mixed with water (more mixing than one might routinely employ), cured several days, and thoroughly remolded before determination of the liquid limit, it is evident that the soils propensity to adsorb water increased during the course of the determination. This may be due to the progressive breakdown of structural "domains" within the soil. These domains, as described by Aylmore and Quirk (6), consist of clusters of clay particles. As the soil and water are mixed, it may appear that thorough mixing has occurred; however, if spatulation is continued, the domains may break down thereby allowing individual clay particles to adsorb more

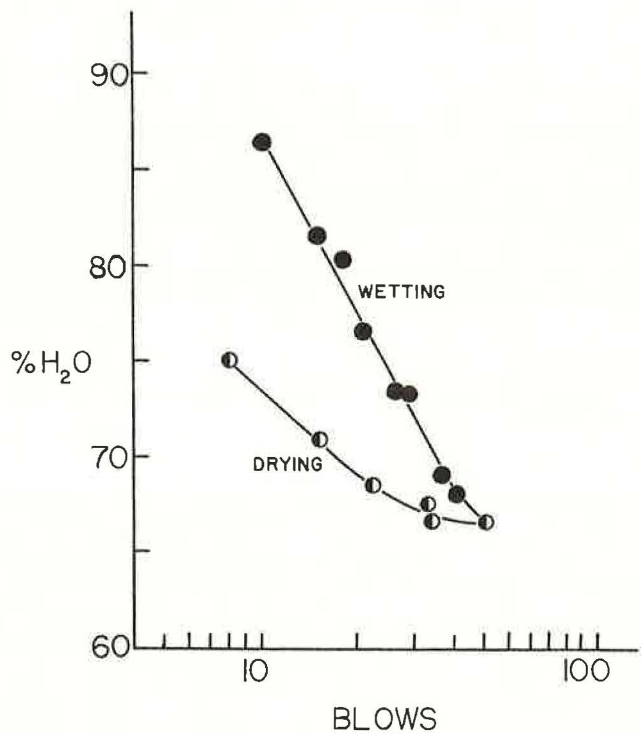

Figure 4. Flow curves for a kaolinitic soil for conditions of drying and wetting during liquid limit determination.

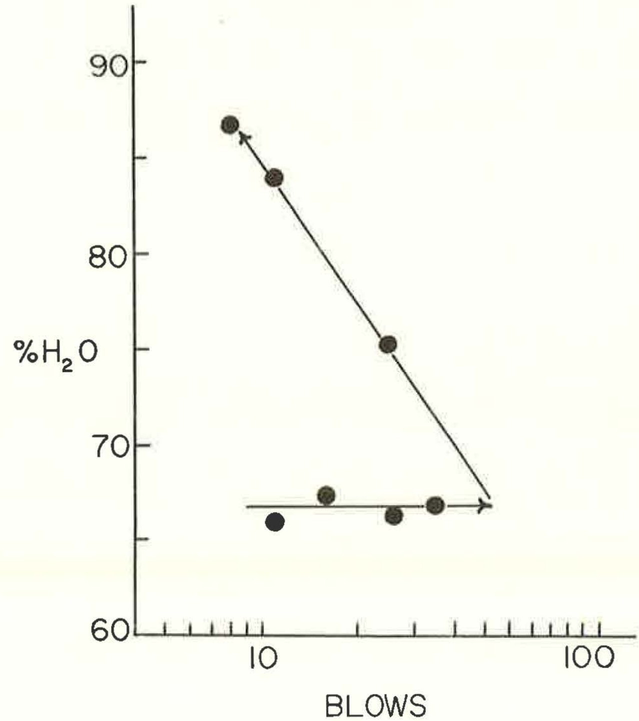

Figure 5. Flow curve for a kaolinitic soil showing the effect of spatulation without drying during liquid limit determination.

water. In this way, the amount of effective lubricating water surrounding each particle or domain is decreased. Because there was no significant change in liquid limit of the kaolinitic soil with curing times varying from one to fifty days, spatulation is apparently an important part of this process. Air drying soil prior to testing probably enhances the formation of domains and increases their resistance to breakdown.

It seems probable that error from this source may account for much of the operator variation found by the authors. If so, the variation can be reduced by curing the soil at near the liquid limit, spatulating with or without slight drying until well below the liquid limit, and finally establishing the flow curve with successive additions of water.

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THOMAS K. LIU and THOMAS H. THORNBURN, Closure—Mr. Krebs has presented some interesting and unusual results on the influence of testing procedure on the liquid limit values of a kaolinitic soil. Our purpose was to study the operator variation in Atterberg limits values associated with the current AASHO Standard Procedures.

Further investigations of the test procedure appear to be in order on the basis of Mr. Krebs' data. If his findings are confirmed for other clay minerals and especially for soils with lower liquid limits, the standard procedures may need revision. Even with such revisions, however, it may be anticipated that a certain amount of operator variability will remain. If the testing procedure becomes more involved, the operator variability, very possibly, will become greater. Only investigations such as those reported in this paper provide a valid basis for the estimation of inherent variability of a procedure—regardless of the reasons for this variability.

New Tests for Measuring Elasticity of Micaceous Subgrades and Their Application to Flexible Pavement Design

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This report summarizes a study (1) of flexible pavement performance conducted by Clemson College for the South Carolina Highway Department and the U. S. Bureau of Public Roads. The research was initiated in July 1960 to determine the cause of poor flexible pavement performance on several primary highway sections and to recommend procedures for the improvement of performance in future construction.

Routine classification, moisture and density tests of the subgrade as well as tests of the pavement and base were performed on sites of widely differing performance ratings. In addition to the routine tests, other special tests were also performed. Test results and pavement performance were then correlated with Benkelman beam deflection measurements taken at all test sites at different seasons for a period of two years.

• TWO HIGH-TYPE flexible pavement sections located in the Piedmont area of South Carolina (US 29 near Spartanburg and US 123 near Clemson) were selected for the study. Each section was about five miles long and consisted of a 10-in. uniform macadam base course surfaced with high-type asphaltic concrete when the study began. Both highway sections carried heavy truck traffic through an area where elastic, micaceous subgrades are commonly found.

After visually rating the pavement performance along the entire length of both highway sections, eighteen test sites (summarized in Table 1) were chosen to include areas of poor, good, and excellent performance.

The net pavement deflections (maximum and residual) in Table 1 were measured with a standard 12-ft Benkelman beam. A semi-trailer tractor unit with the single rear axle loaded to 18,150 lb supplied the load for these measurements.

SPECIAL TESTS

There was no indication of instability or poor quality in the pavement or base material. It became apparent early in the investigation, however, that the sites of poor performance and high deflection were associated with subgrade soils containing a high percentage of mica. The presence of mica in some of the resilient soils tested is revealed by high liquid limit, low plasticity index, and A-5 classification. However, the coarser grained soils of this type will have a liquid limit of less than 40 and a classification of A-4 or A-2-4, the same as the highly desirable top soils of the Piedmont. The test method and design procedure that came closest to indicating poor performance and subgrade resilience was the Georgia Highway Department method discussed by Abercrombie (2). In this method, higher density soils having low volume change and low clay content are considered superior, whereas lighter soils with high volume change are resilient soils of inferior performance.

Paper sponsored by Committee on Surveying, Mapping and Classification of Soils.

TABLE 1

SUMMARY OF TEST SITES

Test Site	Performance Rating	Station	Profile	Deflection (in.)	Base and Pavement Thickness (in.)	Pavement Thickness (in.)
US 29						
1	Excellent	25 + 80	15-in. Fill	0.021	12.0	3.6
3	Poor	34 + 30	5-ft Cut	0.062	12.6	3.6
5	Poor	58 + 69	9-ft Fill	0.067	12.2	4.8
7	Excellent	210 + 50	4-in. Fill	0.023	13.7	3.8
8	Excellent	21 + 73	10-in. Fill	0.010	14.2	3.8
9	Poor	48 + 68	8-ft Cut	0.034	12.8	3.8
10	Good	109 + 18	3-ft Cut	0.019	12.5	4.3
11	Poor	184 + 08	12-ft Cut	0.080	13.0	3.9
US 123						
21	Poor	291 + 40	30-in. Fill	0.065	11.0	3.2
22	Excellent	269 + 55	10-ft Cut	0.016	13.0	3.5
23	Good	254 + 20	3-ft Cut	0.049	13.0	3.5
24	Good	207 + 95	4-ft Cut	0.047	14.0	4.0
25	Poor	154 + 65	18-ft Cut	0.113	13.0	3.7
26	Excellent	150 + 10	1-ft Cut	0.020	11.6	3.5
27	Poor	154 + 80	26-ft Cut	0.077	12.0	3.2
28	Poor	177 + 10	10-ft Fill	0.071	13.0	3.2
29	Poor	299 + 10	18-ft Cut	0.070	11.5	3.5
30	Excellent	126 + 30	5-in. Fill	0.014	12.9	3.5

None of the test methods and design procedures, however, gave a very accurate evaluation of the coarser grained, micaceous subgrades that were associated with sites of poor performance. As the grading of the soil becomes coarser, response to conventional testing is lower, yet the performance of the coarse grained subgrades was as bad or worse than all other subgrades investigated.

Comments by Yoder (3), Dehlen (4), and Ritter and Paquette (5) emphasize the lack of some truly rational method of flexible pavement design. It is thought that the poor soils of the Piedmont area are essentially elastic and that some new rational design procedure to limit flexure should be applied to these soils. It is felt that design methods that utilize ultimate shear strength will, when properly applied, prevent lateral displacement and permanent rutting of plastic subgrades, but this is not the answer to the problem of elastic deflection that fatigues and ultimately cracks the pavement structure. Thus, the search for new, easily performed testing procedures to measure soil elasticity was started.

IGNITION LOSS TEST

The crystalline and chemical structure of a number of common soil minerals including clays and micas have been determined (6). The formula for muscovite mica, $KAl_2(AlSi_3O_{10})(OH)_2$, is relatively fixed; for other micas, the formula is more variable permitting the substitution of several possible metallic ions. Muscovite decomposes between 600 C and 700 C into orthoclase, corundum, and H_2O. The loss in weight of H_2O from the decomposition of a local sample of muscovite averaged 4.5 percent in laboratory tests and agrees quite closely with the 4.52 percent computed from the atomic weights of the formula. Biotite averages 3.9 percent crystal water while Paragonite is 4.71 percent H_2O.

Mechanical and chemical weathering of micas result in alteration products that have much higher amounts of combined water. The degradation of mica is thought to proceed through a hydrous mica, in which some of the alkali ion has been replaced by water, to a moderately plastic clay mineral. There is no sharp line of demarcation between hydrous mica, illite, and the final clay product as mica disintegrates.

Thus, two forms of combined water may be present: (a) hydrate water (H_2O) or water of crystallization found in the hydrous micas and lost at moderate temperatures (200 to 300 C), and (b) water of constitution or the hydroxyl ion part of the crystal lattice lost at higher temperatures (600 to 1,000 C). Pure hydrous micas often contain as much as 10 percent total combined water.

With the exception of mica and clay none of the common Piedmont soil minerals contain appreciable crystal water. Therefore, if the clay is first removed from the sample, the water of crystallization present in the remaining portion of the sample will be an indication of the amount of mica responsible for the elastic behavior of the soil. The following test procedure was developed to measure the weight of crystal water in the +200 fraction as a percentage of the total sample.

The sample to be tested for mica is washed through a No. 200 sieve following wet mechanical analysis. The +200 portion of the sample is then oven-dried at 105 C to remove all free water and hygroscopic moisture. It is then heated in a small crucible to approximately 1,000 C until constant weight is reached. The crystal water (percent ignition loss) as a percentage of the total sample is computed from the weight loss and mechanical analysis as percent ignition loss = (wt loss) (100 - percent passing the No. 200 sieve) ÷ (wt of the +200 material tested).

The sand fraction samples of this investigation tested for ignition loss were usually about 20 gm in size and were usually heated about thirty minutes in a small electric furnace capable of continuous operation at 1,000 C.

REPEATED-LOAD CONSOLIDATION TEST

Another new test procedure to measure the elasticity of the soil directly was also developed as part of this investigation. In this test, the consolidation apparatus (Soiltest Model C-280) was used to measure the confined modulus of elasticity of the subgrade soil. Although the consolidation apparatus is generally not used in the study of highway subgrades, it was felt that useful information that could be correlated with ignition loss and pavement deflection could be obtained. The basic principle of the test is not unlike that of the resiliometer developed by Hveem (5) to measure the resilience of soils.

In the repeated-load consolidation test, the vertical stress increment for the repeated loads was based on expected stress conditions in the subgrade about 15 in. below the surface of the pavement, assuming a 9,000 lb dual-wheel load and no bridging action on the part of the pavement. Stresses from which the modulus of elasticity was computed were an initial stress of 1.2 psi (due to overburden) and a final stress of 15.25 psi (due to overburden plus wheel load).

For each subgrade sample approximately 120 gm, representative of the material passing the No. 10 sieve, was pulverized and tested for elasticity in the following manner. A known weight of pulverized material passing the No. 10 sieve, sufficient to nearly fill the apparatus, was placed in the apparatus dry. A small 0.75 lb weight producing 1.2 psi on the sample is placed on the weight hanger and left there throughout the entire test. The dry sample is then compacted with 100 quick applications of three additional 8.8 lb weights raising the pressure on the sample to 43.4 psi with each application. The sample is then saturated with water added through the porous sandstone disk at the base of the sample while confined with the small weight only. Again 100 applications of 43.4 psi are applied to further compact the sample—allowing a few seconds for each application cycle. In a similar manner, applications of one 8.8 lb weight (15.25 psi) are repeated at a rate of 1/cpm until dial readings at the end of each 30-second period indicate that equilibrium has been reached. Both high and low dial readings are recorded to the nearest 0.0001 in. The deflection for the load increment attributed to the give in the apparatus is determined by calibration with a 1-in. steel blank. The final net rebound for the test-load increment is obtained by subtracting the apparatus deflection from the difference between the two observed dial readings recorded for the test on the soil sample.

The confined modulus of elasticity, E_S, is computed from the net rebound, δ, and the height of the sample, h, for the subgrade sample using the following relation:

$$E_S = \frac{14.05\ h}{\delta} \tag{1}$$

The modulus of elasticity, as measured by the repeated-load consolidation test previously described, was obtained while the sample was confined and lateral displacement restricted. A lower modulus of elasticity would have been obtained had a triaxial or unconfined compression test been used. However, it was felt that the results obtained by the test method used were still abnormally low, particularly for those samples containing a high percentage of clay and silt because of the relatively long time period allowed for deformations to occur in the test. Therefore, a correction based on the percent retained on the No. 200 sieve was applied in the following manner to obtain the corrected modulus of elasticity for the samples in the saturated condition: Corrected E_s = Measured $E_s/1 - 0.01$ (percent passing the No. 200 sieve).

MODULUS OF ELASTICITY VS IGNITION LOSS

For a number of samples, the corrected modulus of elasticity was compared with results of the ignition-loss test. It was found that an equivalent modulus of elasticity yielding good correlation could be computed from the ignition loss using the following inverse relationship: Equivalent $E_s = (8{,}000/I_g)$ psi. The equivalent modulus of elasticity, computed from the ignition loss, was compared with the corrected modulus of elasticity of the same subgrade sample, and the comparison of these two values for 182 samples from the US 29 and 123 highway sections yielded a coefficient of correlation of 0.75.

The next step in the evaluation of the two new tests was to compare actual pavement deflections, as measured with the Benkelman beam, with deflections computed from the test results using deflection analysis.

DEFLECTION ANALYSIS

If the Boussinesq equation for vertical stress is multiplied by "dz" and divided by the modulus of elasticity, E_s, of the semi-infinite continuum and then integrated between the limits of "d" and infinity, the result will be the total elastic deflection, δ, of a point at radial distance, r, and depth, d, from the point of application of load, P, indicated as follows:

$$\delta = \int_d^\infty \frac{3Pz^3\,dz}{2\pi E_s (r^2 + z^2)^{5/2}} \qquad (2)$$

The above equation was integrated and various values of P and r were introduced to accomodate the geometry and dimensions of the Benkelman beam and to approximate an area load produced by dual tires loaded to 9,000 lb. To predict the net deflection of the flexible pavement as measured by the Benkelman beam, the following equation is used:

$$\delta = \frac{4{,}300}{E_s}\left[\frac{d^2}{6(5.4^2 + d^2)^{3/2}} + \frac{1}{3(5.4^2 + d^2)^{1/2}} + \frac{d^2}{6(8.7^2 + d^2)^{3/2}} + \frac{1}{3(8.7^2 + d^2)^{1/2}} - \frac{0.97d^2}{(108^2 + d^2)^{3/2}} - \frac{1.93}{(108^2 + d^2)^{1/2}} + \frac{0.63d^2}{(165^2 + d^2)^{3/2}} + \frac{1.27}{(165^2 + d^2)^{1/2}}\right] \qquad (3)$$

If the eight terms of the deflection equation are multiplied by 4,300 and the product designated as the coefficient of deflection, $C\delta$, the equation may be written in the following simplified form:

$$\delta = \frac{C\delta}{E_s} \qquad (4)$$

Figure 1 shows a plot of the coefficient of deflection for various depths below the pavement surface.

Thus, the expected deflection resulting from the deformation of any soil layer can easily be calculated if the depth range and modulus of elasticity of the soil layer is known. The expected deflection for the layer, $\Delta\,\delta$, is obtained by dividing the difference between the coefficient of deflection at the top and the bottom of the layer by the modulus of elasticity as follows:

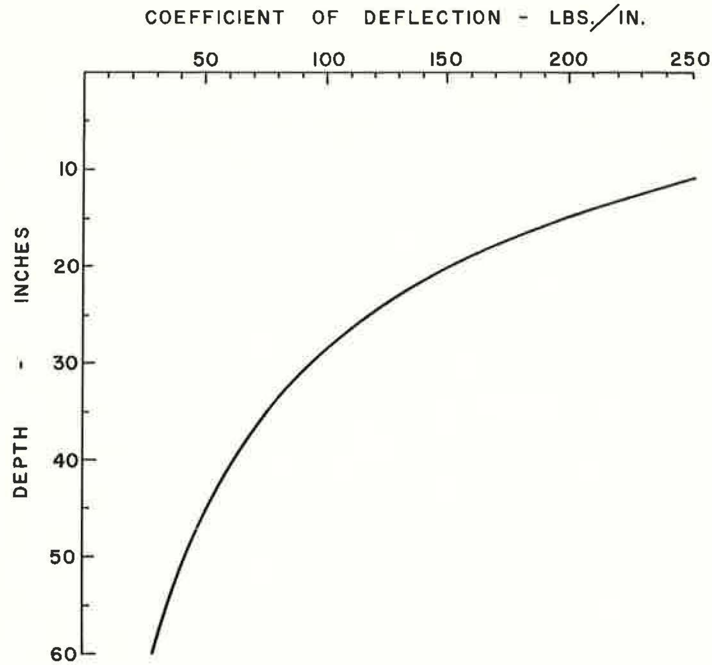

Figure 1. Coefficient of deflection plot for various depths below pavement surface.

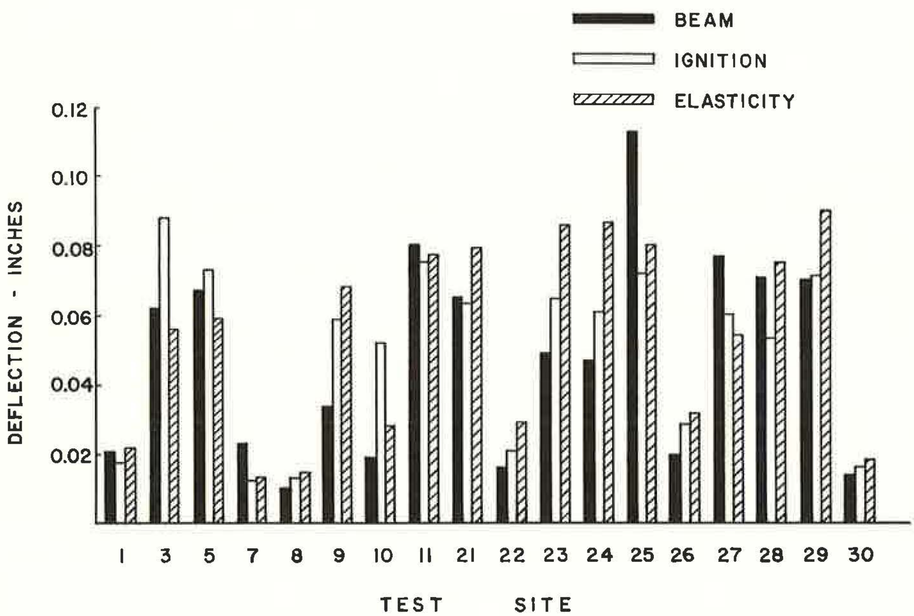

Figure 2. Observed pavement deflections.

$$\Delta \delta = \frac{\Delta C_\delta}{E_S} \qquad (5)$$

The total expected deflection will be the sum of the expected deflection for all subgrade layers to a depth of about 10 ft and the deformation of the pavement and base course.

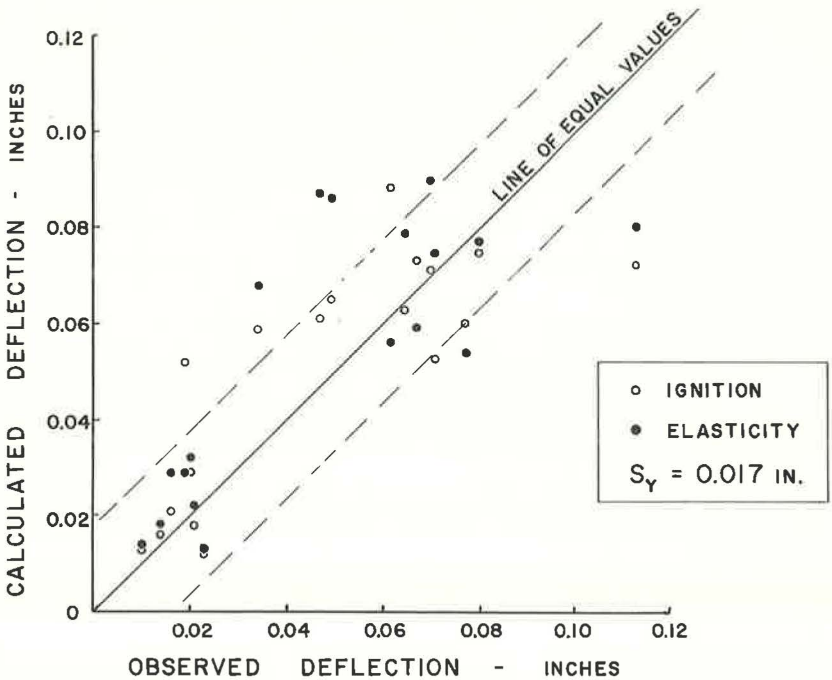

Figure 3. Observed and calculated pavement deflections.

A deformation of 0.004 in. was estimated for the pavement and base studied in this investigation.

The accuracy of the previously mentioned analysis depends on the truth in the assumption that flexible pavements and untreated bases have little or no beam strength and that the vertical stresses at various depths are as predicted by the Boussinesq equation. Also, it must be assumed that deflection is a result of elastic compression rather than lateral displacement of the soil layer. Experimental studies by Sowers and Vesic (8) confirm the first assumption. A study of these deflection curves indicates that vertical rather than lateral displacement predominates.

To check the validity of the theory, the deflection of each of the eighteen sites was computed using the deflection equation. The modulus of elasticity was determined by both ignition loss and elasticity, and the deflection from the deformation of each soil layer was computed in the manner explained previously. Calculated and observed deflections are graphically compared in Figures 2 and 3.

The coefficient of correlation between calculated and measured deflections was nearly the same for both methods of calculation. In the case of the ignition-loss method, the coefficient of correlation was 0.70. For the elasticity method, which is based on the repeated-load consolidation test, the coefficient of correlation was 0.68. There is, therefore, no strong evidence favoring one method of calculation over the other.

DESIGN PROCEDURE

The calculation of elastic deflection by the ignition-loss method and the modulus of elasticity method proved to be reasonably accurate in this study; either test procedure in combination with conventional classification procedures is considered suitable for evaluating the performance of upper South Carolina micaceous subgrade soils.

The typical cross-section shown in Figure 4 is designed to limit the total deflection as measured with the Benkelman beam to about 0.030 in. This is accomplished by providing compacted select-material subbases of sufficient rigidity and thickness so that the total deformation of these materials will be no more than 0.030 in. less the expected deflection of the subgrade on which the subbase is placed. The design curve, subgrade ignition loss vs combined thickness of pavement, base, and subbase (Fig. 5), was developed to satisfy the previously mentioned criteria and can be used to determine the thickness of select material required for the given Piedmont subgrade.

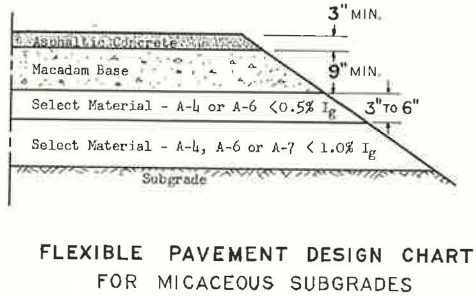

Figure 4. Typical pavement and subbase cross-section.

If a given subgrade cannot meet the minimum requirements of the top layer of select material shown in Figure 4, three to 6 in. of A-4, A-6, or some granular select material having an ignition loss of less than 0.5 percent shall be provided as a subbase to support the macadam base. If the required total combined thickness exceeds 18 in., a lower layer of select material must be provided having an AASHO classification of A-7 or better and less than 1.0 percent ignition loss. A-5 soils of the Piedmont area usually have high mica content and high ignition loss and, therefore, are not suitable as a subbase material.

Good subbase materials meeting the previously mentioned requirements are generally found in the top 3 ft of the normal Piedmont soil profile. If this material is stockpiled from the cuts within the right-of-way, enough subbase material should be available to meet the needs of the construction.

It is obvious that failure of flexible pavements is primarily related to the thickness of the asphaltic concrete surface and the sharpness of bending rather than the magnitude of total vertical deflection. Therefore, the permissible deflection from which the design curve was calculated was increased slightly as the required depth of combined thickness increased. It was assumed that for a 3.5-in. asphaltic concrete pavement and a combined thickness of 15 in., the pavement performance would be satisfactory if 0.025-in.

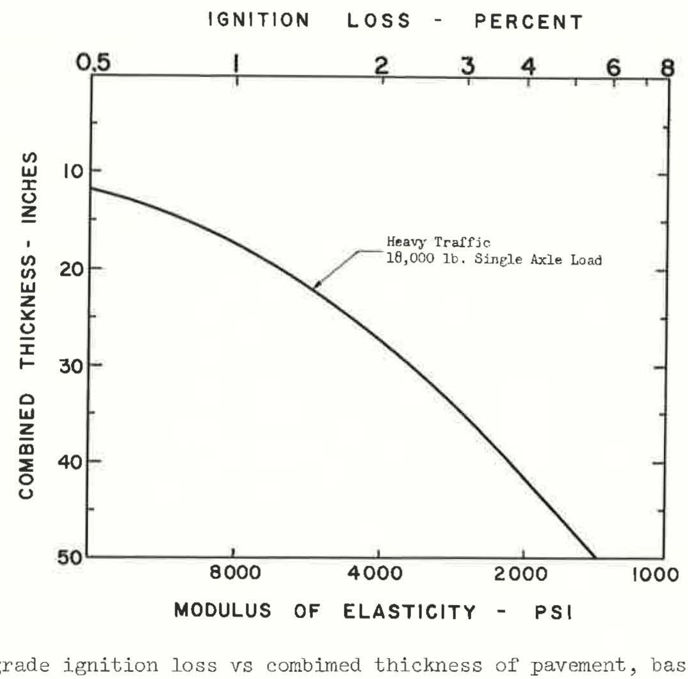

Figure 5. Subgrade ignition loss vs combimed thickness of pavement, base, and subbase.

deflection is not exceeded. A 0.035-in. deflection would not be excessive if the combined thickness of the roadway structure is greater than 36 in.

Even though there is good correlation between the results of the ignition-loss test and pavement performance, it should be pointed out that the study of the US 29 and 123 sections was limited to a few square miles of area located about five miles west of Spartanburg and five miles west of Clemson, South Carolina. Further study of micaceous soils from other areas of the piedmont will be necessary to establish the relationship between elasticity and ignition loss for a greater variety of soils.

CONCLUSIONS

1. Benkelman beam deflection measurements were correlated with pavement performance of the test sites studied.
2. The ignition-loss test is simple and economical. The results of this test were an indication of the mica content and elasticity of the subgrades tested.
3. Direct measurement of elasticity by means of the repeated-load consolidation test is also relatively easy to perform; test results were inversely related to the ignition loss of the subgrade soils tested.
4. The calculation of elastic deflection of a flexible pavement system by the ignition loss (or repeated-load test) method was reasonably accurate in this study and is the basis for a recommended design procedure.

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