were pumped up by the tires and thrown backwards. If the forces had been constant the material would be regularly pushed back and would remain spread all over the road. As the tires have a certain elasticity the tangential forces increase or decrease together with the bouncing movement of the vehicle (this having nothing to do with the springs) so that the material is pushed back irregularly, causing the formation of waves.

I am sure that if Mr. Tschebotarioff or Mr. Relton could see our films, they would agree with our explanation, and with the practical lessons we drew from our tests.

Stabilization is not recommended for dry country, where it is to be used as a surface.

It is better to spread out clay, marl, or silt and roll it down. You will have dust but you will avoid the waves, and that means much for your equipment and for yourself.

RELATION BETWEEN THE PLASTIC INDEX AND THE PERCENTAGE OF FINES IN GRANULAR SOIL STABILIZATION

F. L. D. Wooltorton, *Public Works Department, Maymyo, Burma*

SYNOPSIS

Specifications for granular soil stabilization usually give maximum permissible values for the plastic index of the fraction passing the No. 40 sieve without adducing logical reasons for this limitation.

Without a basic underlying theory, such specifications represent more or less localized experience which cannot be utilized in regions of different soils and different climates. The bad performance in periodically desiccated soil regions of granular soil stabilization which followed U. S. specifications, indicates the great practical need for such an underlying theory which would permit a general specification applicable to all soils and climates.

While it is not claimed that this problem has been solved by the writer, it is felt that a definite advance has been made toward its solution by analyzing the physical meaning of specifications limiting the percentage and the PI of the minus 40 sieve portion, and by evaluating the different rules of thumb which are being employed by a number of experienced engineers. These rules possess the general form of:

$$\text{PI} \times \text{Percent passing No. 40 sieve} = C$$

where C is either a constant or the difference between a constant and a function of the percentage of soil fines.

A number of problems confront the engineer constructing low cost roads in a country where they have not been previously tried on a scientific basis. Amongst these problems appear a number of questions concerning the plastic index and the clay content, such as:

1. Can or must we work to the American recommendations for PI and clay content?
2. What is meant by the plastic index?
3. Why should there be maximum and minimum values?
4. How does climate affect these limits?

It is felt there is no clear basic conception of the meaning of the plastic index requirements which, in application to roads. will satisfy an analyst or soil scientist. It is known that, for some reason, the PI permissable is dependent on the prevailing yearly temperature values and range and that a design satisfactory in Central or South America would not necessarily be satisfactory in the U. S. A. if the soil were transported to that country. It is thought that a higher PI or clay content may be permissable for a sodium clay than for a calcium clay, and that, in some way, the soil chemistry plays a part in determining these values. It is known that the plastic index and fines requirements differ amongst the States of the U. S. A. and that some successful designs are based on values well outside the standard ranges.

The plastic index is determined for the material passing the No. 40 sieve presumably because:

1. That fraction is most suitable for the performance of the liquid limit and the plastic limit tests.

2. The liquid and plastic limits are supposed to represent certain capillary moisture relationships.

Actually:

3. The plastic index has been shown by C. A. Hogentogler and H. F. Winterkorn to lie in the colloidal fraction and hence part of the clay, and any silt, fine and coarse sand etc. are dilutants which may be expected to give, by some unknown equation, lower plastic indexes for the whole soil material.

4. The capillary hypotheses are perhaps misleading. They may represent the respective capillary capacities of the fines in one sense—the disturbed sense—but not in the structural sense of an undisturbed soil, or even of a densified soil, in which the fines form but a fraction of the total material.

Logically, there appears to be no reason why the plastic index requirements should not be based on the whole soil as many United States engineers firmly believe. It is the behaviour of the whole material in the presence of moisture in which the engineer is really interested.

In formulating a method for the logical evaluation of the plastic limits, one line of thought was based on:

1. Unpublished data in the U. S. A. and England suggested that for soils containing a particular clay mineral, the ratio of the clay content to the plastic index appeared constant; and

2. It seemed reasonable to presuppose that for unlike soils containing any clay mineral, the product of some power of the plastic index (representing clay activity) and the clay content should be constant. This formula obviously involved the climatic factor.

Attention was concentrated on the latter till it was realised that the expression sought was:

> "For the stability of a granular or non-cohesive material—which depends on its shear resistance for stability—the amount of moisture absorbed over the plastic range should not exceed the pore space available within the structural system or the structure will be ruptured by the swelling pressure with loss of frictional stability".

This gives one condition for determining the maximum permissable value of the plastic index or of the maximum permissable amount of fines.

As the PI refers to the minus 40 material, which includes the capillary fine sand fraction, the expression (PI × clay fraction) is not an obvious selection. A more fitting formula for investigation is:

$$\text{PI} \times (-40)^{1} \rightarrow \text{some value.}$$

When applying the discussion to cohesive subgrade materials or to slightly cohesive wearing courses, whose stability depends wholly, or in part, on cohesion, it must be remembered there is no such rigid and permanent structure and volume changes will occur. The formula involving PI × (−40) must then be modified to allow for some permissable swelling.

The PI represents the moisture content range over which swelling in the minus 40 fraction occurs in excess of that small amount represented by the PL which, for a plastic "fines" fraction, is said to equal the critical moisture content (CMC) beyond which the stability of the fines rapidly decreases with increase of moisture content. This CMC, which is presumably close to the upper limit of osmotic hydration, appears to be a most desirable moisture content in the field as far as the fines are concerned.

On the wetting and drying, or swelling and shrinkage, curves used in England, the PI

[1] Hereafter, in this report, "(−40)" is used to signify "the fraction of material passing the No. sieve."

range gives a direct measure of that swelling in cu. cm. per 100 g. of fines.

PI × (−40) is therefore a direct measure of the maximum swelling of the fines within the structure of a coarse granular mix between their PL and LL.

For non-swelling coarse aggregate mixtures, this swelling of the fines must not, for base courses, exceed the volume available to accomodate the swelling and hence must not be sufficient to disturb, on swelling, the coarser material making up the structure. This means that PI × (−40) → some value equal to the available pore space. In such a system the available pore space can only be supplied by the volume of the entrapped air when the amount of fines is the ideal amount for maximum density and the amount of entrapped air plus semi-entrapped air when the percentage of fines is less than the ideal value for maximum density. Under the peculiar conditions enabling the fines to absorb moisture up to the LL, it may perhaps be assumed that even the entrapped air is free to escape. Under such conditions the permissable swelling would be regulated by the pore space at the PL.

For cohesive materials the expression PI × (−40) must be modified so that the difference between it and the pore space shall not exceed some permissable degree of swelling.

The volume available to accomodate the swelling of the fines will depend upon the soil type, the grading, and density and is therefore not a simple constant. For uniformly graded materials complying with standard specifications and compacted by equipment giving results in accordance with the standard compaction test, it would appear that this available volume should be based on air-content of about 4 percent.

However, in trying to analyse this "constant", it must be remembered that the mix need not necessarily be ideally graded according to the accepted uniform grading specifications because, according to Hveem (*1*)[2], a Prof. Kriege has shown that the maximum possible density does not occur under uniform grading but when 50 percent of the coarsest size is combined with 50 percent of the finest, i.e. skip-gradations may be better than uniform gradations from the point of view of density.

Further, in considering the problem in general, it must be remembered that the total material need not necessarily be ideally graded, from the point of view of density, below the coarse sand fraction so that the fines content may vary down to zero when the PI of the non-existant fines could be infinite without worrying anyone.

From an examination of data on the OMC determined by the standard compaction test, it appears that the OMC for cohesive soils, examined in the U. S. A. and U.K., lies within the approximate limits of (PL ± 4)[3]; and that the OMC for coarse aggregate mixes lies within the limits of the proportionate PL and the proportionate (PL + 4). The PL appears significant. Whether the difference of 4 is incidental, due to experimental errors, or is some significant figure, was not known, but the same figure appeared to occur in another formula—that connecting the PI of the fines with the PI of the binder.

THE PLASTIC INDEX AND THE PERCENTAGE OF FINES

If the data quoted by R. W. Miller (*3*) is based on experiment then, for the soils used by him, the PI of the fines is apparently connected with the PI of the binder material by the approximate formula:

$$\frac{\text{PI (fines)} + 4}{\text{PI(binder)} + 4} = \frac{\text{Percent binder in fines}}{100}\ ; \textit{ and}$$

$$\frac{\text{PI(binder)} + 4}{\text{Percent binder}} = \text{a constant for any particular binder where the constant increases with the activity of the binder.}$$

Markwick's (*4*) average data curves, when transposed to this form, give a formula of the type:

[2] Italized figures in parentheses refer to list of references at the end of the paper.

[3] Markwick (*2*) says the OMC is generally rather less than the PL.

$$\frac{\text{PI} + c}{\text{percent clay}} = \text{a constant for any particular soil; where PI refers to the fines.}$$

If these formulae be generally correct then there is no simple relationship connecting PI_B and the percentage of binder which was, at first sight, a little disconcerting, and disconcerting it was till once again consideration was given to the drying and wetting curves.

Now, the particular drying and wetting curve, for the fines, to be consulted depends on the quantity of air entrapped during a normal field moisture cycle. The many possible curves will not have a common (PL, vol.) point but they will, under the peculiar conditions where the LL is reached, have an approximately common (LL, vol.) point since at the LL, involving manipulation, it may be assumed the air content of the fines approaches zero. Though the PL is determined for a powdered sample with no structural density it may, I feel, be assumed that the manipulation is sufficient to expel most of the entrapped air or, in other words, the PL and LL may both be considered to be virtually on the basic drying-wetting curve obtained by drying a powdered sample from the LL.

Hence for a densified cohesive mass, as a cohesive subgrade or the cohesive fines within a granular non-cohesive coarse aggregate mix, the in situ volume change should equal the experimental PI minus the volume of entrapped air which, at the OMC, is usually about 4 percent.

Assume, as in Wartime Road Problems, No. 11 (5), that the dry density remains constant for changes in moisture content. This may be true for granular base courses but does not appear true for cohesive plastic subgrades which swell on absorbing moisture. How does the density vary with changes in the percentage of fines?

Little work appears to have been done on ascertaining how the density varies with the gradation down to the fines and with variation in binder content. Information available concerning non-cohesive coarse granular mixes may be summarised as:

1. The percentage of fines must be less than 50.

2. For high densities the percentage of fines should be about 30. Shaw's (6) figures for 1-in. max. size, give a volume somewhat less than 36 percent.

3. For high densities the PI of the fines should be low.

4. Variations in the percentage of binder material affect the density and the variations in density may be of the order of minus 6 percent for a sand-clay mix and about minus 3 percent for a coarse aggregate mix where the variations are relative to the optimum density.

5. For variations in stone admixtures up to 50 percent there is little variation in density. For increasing quantities of stone the soil becomes non-cohesive when grading is the most important factor controlling density.

6 For such non-cohesive materials the weight of the compacting equipment and the number of passes has presumably little effect on the density other than in their effect in distributing the fines evenly amongst the coarse material. For cohesive subgrade materials there is no 'rigid' structure and the weight of the compacting equipment and the number of passes becomes all important.

For the purpose of the following studies, the calculations are based on 100 g. of whole mix and the specific gravity of the particles is taken as 2.7. No allowance is made for frost effect.

Preliminary Studies in Swelling.—(See Figure 1)

1. The starting point P, representing the condition of the soil fines at the time of compaction, may be assumed to be at the most desirable moisture content for the fines equal to their PL and of some volume denoted by V_4.

2. Considering 100 g. of fines

$$\begin{aligned}\text{Swelling above the PL} &= (V_5 - V_4) \text{ cu. cm. per 100 g.} \\ &= (\text{PI} - V_a)\end{aligned}$$

where V_a = vol. of entrapped air and assuming all air is located within the fines. (This

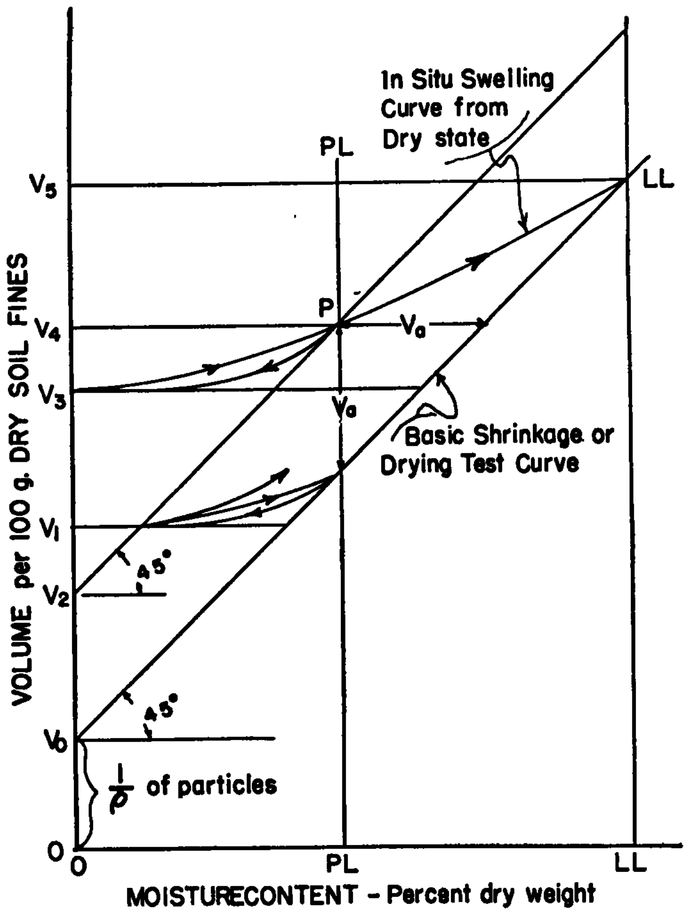

Figure 1 (Not to scale)

entrapped air will not be free to escape until the soil be manipulated).

3. If the percentage of fines in 100 g. of total mix is A, then the weight of fines = A g. per 100-g. total mix. Hence swelling of A g. of fines

$$= \frac{A}{100}(\text{PI} - V_a) \text{ cu. cm. per 100-g. total mix} \quad (1)$$

when entrapped air is located within fines.

This expression is also that for the proportionate effective PI of the whole mix.

The volume of air entrapped in 100-g. total mix is the volume of air entrapped in A g. of fines and equals $\frac{A}{100} \times V_a$.

Therefore;

$$\text{percent air} = \frac{\text{Vol. air}}{\text{Vol. 100-g. total mix}} \times 100 = \frac{A}{100} \times V_a\sigma$$

where σ = density of compacted mix = 2 approx.

$$\text{or } \frac{A}{100} \times V_a\sigma = 4 \text{ approx.}$$

so that swelling in the fines may be represented by:

$$\frac{A}{100}(\text{PI} - V_a) = \left(\frac{A\text{PI}}{100} - \frac{4}{\sigma}\right) \quad (2)$$

where $V_a = \frac{4}{\sigma} \times \frac{100}{A} = 4 \times \frac{3}{2}$ approx. cu. cm. per 100 g. total mix.

4. Swelling in A g. of fines

$$= \frac{A}{100} (\text{PI} - V_a) \text{ in cu. cm. per 100-g. total mix}$$

when entrapped air is located within the fines. If the entrapped air be not located within fines and is still free to escape, the swelling above PL becomes

$$\frac{A\text{PI}}{100}$$

5. If the OMC be taken as the effective proportionate plastic limit (PL_E) for the whole mix, then it would seem that as an approximation:

$$\text{OMC} = \text{prop. } (\text{PL}_E),$$

when entrapped air is located within the fines.

$$= \text{prop. } (\text{LL}_E - \text{PI}_E)$$

$$= \frac{A}{100}\text{LL} - \frac{A}{100}(\text{PI} - V_a)$$

$$= \frac{A}{100}(\text{PL} + V_a) \tag{3}$$

$$\text{or} = \left(\frac{A\text{PL}}{100} + \frac{4}{\sigma}\right) \tag{4}$$

$$\text{or} = \frac{A}{100}\left(\text{PL} + \frac{4 \times 3}{2}\right) \tag{5}$$

which is similar to one of the limits previously stated for coarse graded aggregate mixes. The figure "4" would thus appear to represent some function of the entrapped air content in the fines. If all contained air is not located within the fines, then equation (5) becomes

$$\frac{A}{100}(\text{PL} + c) \text{ where } c \text{ is less than } \frac{4 \times 3}{2}.$$

If the entrapped air be not truly entrapped but held in part or in whole in the interstices in the coarse fraction or within voids between the fines and the coarse fraction, then the OMC would, on the previous assumption, take up some value between

$$\frac{A}{100}\left(\text{PL} + \frac{4 \times 3}{2}\right) \quad \text{and} \quad \frac{A\text{PL}}{100}$$

giving the other apparent extreme limit for coarse aggregate mixtures.

6. For cohesive soils $\frac{A}{100}$ approaches unity and, on the above analogy, it would appear that the OMC should vary between PL and $\left(\text{PL} + \frac{4 \times 3}{2}\right)$.

The two conditions are however not truly comparable. In a coarse granular mix, mechanical equipment features mainly in distributing the fines within the voids of the granular structure; whereas with a cohesive soil compaction and not distribution is the function of the equipment. In a coarse granular base course no swelling is to be permitted; whereas in a cohesive mixture some swelling will occur.

In the (PL − 4) limit the figure "4" appears to depend more on the equipment used than on the value of entrapped air. If, for example, a lighter equipment be used for compacting, then the OMC would be expected to be higher than when heavier equipment be used.

Markwick (*2*) says, on the assumption that standard equipment be used, that the presence of more than 50 percent stone implies a higher OMC for the soil mortar than when percentage of stone is less than 50.

7. The above assumes that the air entrapped at the PL is free to escape before the LL is reached. Should some air, volume = V_b, remain at a moisture content equal to the LL, then the swelling in the fines will be greater and will equal ($\text{PI} - V_a + V_b$) so that when the air is located within the fines, the swelling, given in equation (1), becomes

$$\frac{A}{100} - (\text{PI } V_a + V_b).$$

If the air be located between the fines and the coarse material, then the swelling above the PL remains $\frac{A}{100}$ PI as in Par. 4.[4]

The effect of air retention on the OMC, as discussed in Par. 5 is to reduce the $\left(\frac{4 \times 3}{2}\right)$ by the amount of air retained at the LL.

Approximation for Ascertaining the Maximum Permissable Value of the PI for a Coarse Granular Base Course in which No Overall Swelling is Permitted—Assuming that the OMC of mix. produces a moisture content in the fines of: (1) their PL or (2) their PL plus volume of air entrapped within fines, and for some reason it is possible for the fines to absorb moisture up to their LL and for the entrapped air to escape. (If more moisture be forced into the mix then presumably this extra water will drain away as there is no capillary force to retain it.)

1. Since the only space available for water in excess of the OMC is that occupied by air entrapped during compaction, it must be assumed that, under the conditions enabling moisture to be absorbed up to the LL, this entrapped air will be free to escape.

Hence the swelling of the fines over the effective plastic index range must not exceed the volume of the voids available; or, in the first example

$$\mathrm{PI} \times \frac{A}{100} = \frac{B}{100} \times \frac{100}{\rho}$$

where A = Percent fines in 100 gm. total mix
B = Percent entrapped air between fines and coarse material on a volume basis and under standard compaction methods for well graded materials

[4] Note: If all the air entrapped at the PL be retained by the time the LL is reached, then the swelling in the fines is not only greater but there will be no room available for it and in consequence the total mix will swell. Hence for certain soils, as some dispersed soils, the PI must be kept lower than for others, as aggregated soils, i.e. lower for some A-6 than for some A-7 soils.

and ρ = specific gravity of compacted soil

$$\text{or} \quad \mathrm{PI} \times A = \frac{400}{\rho}$$

$$\text{or} \quad \left(\frac{A}{100}\mathrm{PI} - \frac{4}{\rho}\right) = 0 \qquad (6)$$

which is expression (2) equated to zero though (2) refers to fines containing entrapped air. This formula is but another way of saying that for no swelling of the whole mix the proportionate effective PI of the whole material must not exceed zero.

2. If the air be entrapped entirely within the fines then the swelling within the fines will be

$$\left(\frac{A}{100}\mathrm{PI} - \frac{4}{\rho}\right)$$

and will occur within the fines at the expense of the entrapped air which must be free to escape at the LL as there is no other space available for the swelling.

Hence for no swelling of the whole mix

$$\left(\frac{A}{100}\mathrm{PI} - \frac{4}{\rho}\right) = 0 \quad \text{as in (6).}$$

So that as far as swelling is concerned, the necessary condition, assuming air free to escape, is the same whether the air be entrapped within the fines or be held between the fines and the coarser material as voids.

3. When ρ = 2.16 (density = 135 lb. per cu. ft.)

$$\mathrm{PI} \times A = 185 \qquad (7)$$

When ρ = 2.00 (density = 125 lb. per cu. ft.)

$$\mathrm{PI} \times A = 200 \qquad (8)$$

Consider the average value of ρ = 2.08 and assume that specific gravity remains sensibly constant over the permissable range of fines content

$$\text{Then } \mathrm{PI} \times A = 192 \qquad (9)$$

(For a more general approximation see "*Another Method for Obtaining the Approximate Maximum PI*" later.) Hence when:

$$\left.\begin{array}{l} A = 10 \text{ percent, PI may not exceed } 19 \\ A = 20 \text{ percent, PI may not exceed } 10 \\ A = 30 \text{ percent, PI may not exceed } 6 \\ A = 40 \text{ percent, PI may not exceed } 5 \end{array}\right\} \quad (10)$$

where a PI of 6 is the standard maximum value permissable and 30 percent fines gives approximately the maximum density.

$$\text{When } \rho = 2;\ \text{PI} \times A = 200$$

so that when $A = 5$ percent, PI might approach 40 and when $A = 35$ percent, PI might approach 6. When $A = 50$ percent, as permitted in Texas, the PI should not exceed about four against the value of 15 which, from the above, corresponds to a fines content of 13 percent as compared with their minimum value of 15 percent.

4. When a fines percentage of about 30 percent is exceeded in a uniformly graded mix there apparently will be some resultant swelling unless the compacting equipment be reinforced to give higher densities.

Note that if higher than normal densities can be obtained and (or) the percentage of air content can be reduced below about four then presumably the maximum permissable PI must be reduced.

5. If some air, Y_b, be still retained when the LL is reached, then equation (6) becomes:

$$\text{PI} \times \frac{A}{100} = \frac{(Y - Y_b)}{\rho}$$

where Y_b = percent air retained.

Similarly for the condition when the air is originally entrapped within the fines the equation becomes:—

$$\text{PI} \times \frac{A}{100} - \left(\frac{Y - Y_b}{\rho}\right) = 0, \quad \text{as above}$$

when $Y = Y_b$, the PI must be zero unless the air be still compressible.

Another Method for Obtaining the Approximate Maximum PI—

1. The maximum amount of permissable swelling in the soil fines when air is free to escape equals the volume of the voids when the fines content in an otherwise well graded mixture is zero, less the volume of the added fines (A weight) when dry less the volume of moisture at the OMC working on a density of 130 lb. per cu. ft. This gives the formula:

$$\text{PI} \times A = 48\,(27 - 0.77x) \quad (11)$$

(As a particular example, when $A = 30$ percent approximating ideal grading:

$$\text{PI} \times A = 192 \text{ as in equation (9).)}$$

When

$$\left.\begin{array}{l} A = 10, \quad \text{PI} \times A \\ \quad = 960 \text{ when PI may approach } 96 \\ A = 20, \quad \text{PI} \times A \\ \quad = 576 \text{ when PI `` `` } 28 \\ A = 30, \quad \text{PI} \times A \\ \quad = 192 \text{ when PI `` `` } 6 \\ A = 35, \quad \text{PI} \times A \\ \quad = 0 \text{ when PI} = 0. \end{array}\right\} \quad (12)$$

2. The above assumes detrimental swelling occurs over the range (LL − OMC) where ODC = proportionate PL_p. If however the deterimental swelling is to be determined over the range (LL − OMC) where OMC = proportionate (PL + 4) then the formula becomes:

$$\frac{\text{API}}{100} - \frac{4}{\rho} = (23 - 0.77x)\frac{1}{\rho}$$

which is the same as (11) above.

A Study Involving the Liquid Limit—

Let weight of total mix = 100 g.

Percentage of fines = A
Wt. of fines = A (See Fig. 2)

From the curve in Figure 2: Moisture intake between PL and LL producing detrimental swelling per 100 g. of fines = volume change in cu. cm. per 100 g. fines over same range

$$= (V_4 - V_3) = \text{PI in cu. cm.}$$

So that swell per 100 g. fines = PI in cu. cm. For no swelling in total mix, volume change of fines within 100 g. of total mix must not exceed voids available or $\frac{\text{API}}{100}$ → total volume —volume of solids when fines are at their

PL assuming entrapped air located between fines and coarse material and is free to escape before LL is reached.

$$\rightarrow \frac{100}{\rho} - \left(\frac{A}{100}\mathrm{PL} + \frac{100}{2.7}\right) \qquad (13a)$$

$$\text{or } \frac{A\mathrm{LL}}{100} \rightarrow \frac{(270 - 100\rho)}{2.7\rho} \qquad (13)$$

when $\rho = 2.08$ and $A = 30$ percent

$$\mathrm{LL} \rightarrow 36 \qquad (15)$$

If the air between the fines and coarse material is not free to escape and is considered incompressible then formula (13a) breaks down as the righthand side of the expression is zero. The appropriate formula would then be

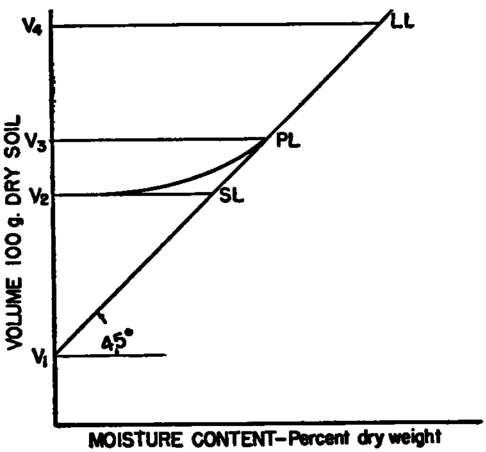

Figure 2. Fines only (Not to scale)—For no entrapped air in fines; $V_3 = (V_3 - V_1) + V_1 = \mathrm{PL} + \frac{100}{2.7}$

$$\frac{A\mathrm{PI}}{100} = \text{permissible swelling.}$$

When the entrapped air lies within the fines and is free to escape before the LL be reached, the formula becomes

$$\left(\mathrm{PI} \times \frac{A}{100} - \frac{4}{\rho}\right) \rightarrow \frac{100}{\rho} - \left(\frac{A\mathrm{PL}}{100} + \frac{100}{2.7}\right)$$

or

$$\frac{A\mathrm{LL}}{100} \rightarrow \left(\frac{104}{\rho} - \frac{100}{2.7}\right) \qquad (17)$$

For $\rho = 2.08$ and

$$A = 30 \text{ percent}, \qquad \mathrm{LL} \rightarrow 42 \qquad (19)$$

When air is not free to escape, then:

$$\frac{A\mathrm{PI}}{100} - \frac{4}{\rho} = \text{permissible swelling.}$$

When air lies equally within fines and between the fines and coarse material and the air between is not free to escape but is compressible, then:

$$\frac{A\mathrm{PI}}{100} \rightarrow \frac{100}{\rho} - \left(\frac{A\mathrm{PL}}{100} + \frac{100}{2.7}\right) - \frac{2}{\rho}$$

for which, when $\rho = 2.08$ and $A = 30$ percent

$$\mathrm{LL} \rightarrow 33 \qquad (20)$$

Comparisons—

1. A PRA officer was met in California who used a formula

$$\mathrm{PI} \times \text{Percent passing No. 200} \rightarrow 60$$

or if the percentage of binder be taken 0.66 × (−40) this formula becomes

$$\mathrm{PI} \times \text{Percent passing No. 40} \rightarrow 90$$

so that from equation (12) he should be prepared to use a fines content of up to 33 percent having a PI of up to 3 which appears reasonable.

2. A PRA officer in Texas, where the climate is different, quoted a formula

$$\mathrm{PI} \times A \rightarrow 375 \text{ to } 400$$

Texas should therefore, on the basis of equation (12), be prepared to use a fines content up to about 23 percent having a PI of up to about 20. The above, however, assumes that there will be no resultant expansion whereas Texas seems to permit of some expansion when the above fines could be increased.

3. Successful Texas Caliche Base Courses give on test (7)

$$\mathrm{PI} \times A \rightarrow 750$$

when subject to frost action which suggests a maximum percentage of fines of 18 per cent having a PI → about 40 whereas they limit the PI to 15 and allow fines up to 50 percent.

When PI = 15, then, from equation 11 the percentage of fines for no swelling = 30.

When not subject to frost action they would presumably be prepared to use 100 percent fines with a PI → 12.

4. From De Klotz's (*9*) formula and data it appears that for his soils to give the required average of ratio of bearing value to standard value for:

$$\textit{Sub-base};\ \frac{\text{PI}}{\text{LL}} \times \text{Percent fines} \rightarrow 13.5$$

or

$$\text{PI} \times \text{fines percent} \rightarrow 13.5 \times \text{LL} \rightarrow 13.5 \times 40 \rightarrow 540$$

so that he should be prepared, on the basis of equation (12) to use up to 21 percent fines having a PI → 26.

$$\textit{Base Course},\ \frac{\text{PI}}{\text{LL}} \times \text{Percent fines} \rightarrow 9 \times 25 \rightarrow 225$$

giving a fines content of up to 27 percent and a PI → 8.

$$\textit{Surface Course};\ \text{PI} \times \text{Percent fines} \rightarrow 9 \times 35 \rightarrow 315$$

giving a fines content of up to 25 percent and a PI → 12.

UPPER LIMIT OF PI

Design Requirements—

Maximum moisture content (no manipulation)

$$\text{MC max} \leqq \text{PL} \qquad (1)$$

Field moisture equivalent

$$\text{FME} > \text{PL} \qquad (2)$$

A factor of safety for the compressive strength of the fines of, say, 2 (3)

when:

$$\frac{\text{FME} - \text{PL}}{\text{LL} - \text{PL}} \rightarrow 0.5 \quad \text{(see shrinkage curve)} \qquad (4)$$

$$\begin{matrix} \text{Volumetric change} \rightarrow 28 \text{ percent} \\ \text{(Lineal Shrinkage} \rightarrow 8.5 \text{ percent)} \end{matrix} \qquad (5)$$

From shrinkage curve study and study of data[5]

$$\text{MC max} = \text{FMC}$$

$$= 0.75 \text{ FME (approx:)} \qquad (6)$$

$$\text{VC} = \text{volume change}$$

$$= R(\text{FME} - \text{SL})$$

$$\text{or:—} \quad \text{FME} = \frac{\text{VC}}{R} + \text{SL} \qquad (7)$$

$$\textit{From (1) \& (6)}\ \text{PL} \geqq 0.75 \text{ FME}$$

$$\text{or FME} \leqq 1.33 \text{ PL} \qquad (8)$$

$$\textit{From (7)}\ \frac{\text{VC}}{R} + \text{SL} = \text{FME} \leqq 1.33 \text{ PL}$$

$$\text{or PL} \geqq \frac{1}{1.33}\left(\frac{\text{VC}}{R} + \text{SL}\right) \qquad (9)$$

$$\textit{From (4) \& (8)}\ \frac{\text{FME} - \text{PL}}{\text{PI}} \rightarrow \frac{0.33 \text{ PL}}{\text{PI}}$$

$$\text{or } \frac{0.33 \text{ PL}}{\text{PI}} \leftarrow \frac{\text{FME} - \text{PL}}{\text{PI}} \rightarrow 0.5$$

$$\text{or PI} \rightarrow 0.66 \text{ PL} \qquad (10)$$

when PL = 10; MC max ≦ PL,

then PI → 6.6) (11)

when PL = 20: MC max ≦ PL,

then PI → 13.2)

[5] F.L.D. Wooltorton, "Report on Low Cost Roads, Appendix 1," Government of Burma Press, Rangoon (1948).

From (4), (5) & (9)

$$PL \geqq \frac{1}{1.33}\left(\frac{VC}{R} + SL\right)$$

when VC = 28; SL = 10; R = 2.0, then
then $PL \geqq 18$
so that min. value of PL = 18 when
$PI \rightarrow 12$
(FME = 24 = 1.33 PL)
when VC = 28; SL = 20; R = 1.7 then
$PL \geqq 28$
so that min. PL = 28 when $PI \rightarrow 18$ (12)
(FME = 31 which is less than
1.33 PL = 37)
when VC = 28; SL = 60, R = 1.0, then
$PL \geqq 60$
so min. PL = 66 when $PI \rightarrow 43$
(such soils are found in Hawaii;
their LL = 66
FME = 60 but they are said
to be non plastic.)

There appears to be a very good correlation between R and SL given approx: by

$$R \times SL^{0.41} = 10^{0.738}$$

Further Comparisons—

Liquid Limit.

For base courses specifications normally call for a maximum value not exceeding 25.

LL could vary between 33 and 42 when the percentage of fines is 30 percent.

Texas permits a maximum LL of 45 with a fines percentage between 15 and 50.

Plastic Index.

If maximum LL be fixed at 25, then

$$PL = 25 - PI \text{ or}$$

$$25 - PI = PL \qquad \text{or}$$

substituting from equation (10)

$$25 - PI = PL \leftarrow \frac{0.50}{0.33} PI$$

$$\text{or } PI \rightarrow \frac{25 \times 0.33}{0.83} \rightarrow 10 \text{ for a base course.}$$

Similarly for a surface course, LL = 35 and $PI \rightarrow 14$.

Texas Soils.

SL appears to be either 11 or 30. So that from equations (12) above

$$PL \geqq 18 \text{ or } PL \geqq 38$$

$$\text{and } PI \rightarrow 12 \text{ or } PI \rightarrow 25 \text{ respectively.}$$

Where Texas limits its PI to 15.

Now Texas limits lineal shrinkage to 8½ percent or its C_f to 28 percent.

Hence equating for swelling

$$\frac{API}{100} - \frac{4}{\sigma} \rightarrow 0.5 \times \frac{28}{100} \times \frac{100}{\sigma}$$

$$API \rightarrow \frac{3200}{2\sigma} \rightarrow 800$$

which is the value quoted for Texas Caliche Bases, or when PI = 15; $A \rightarrow 53\%$ against Texas maximum permissable value of 50 percent.

Frost.

On the assumption that frost destroys the soil's structure, the volume change must be based on the LL when allowable values of the PI are lower.

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THE EFFECT OF CALCIUM CHLORIDE ON THE COMPACTIVE EFFORT AND WATER RETENTION CHARACTERISTICS OF SOILS

ELDON J. YODER, *Joint Highway Research Project, Purdue University*

SYNOPSIS

The results of the tests reported herein were obtained in the soils laboratory of the Joint Highway Research Project, Purdue University. The greater portion of the tests were made on four local materials consisting largely of fine-grained glacial drift soils from both the Wisconsin and Illinoian ages. The remaining tests were performed on 21 soil samples from nine southern states.

The soils were tested with an admixture of calcium chloride ranging in amount from ¼ percent to 1½ percent by weight of dry soil. In the compaction tests the compactive effort was varied from 5 to 90 blows of the Proctor hammer.

A total of 156 compaction tests were made; 81 of these tests were made on the raw soil and 75 were made on various combinations of the soil plus calcium chloride. In most cases the compacted densities of the soil-calcium chloride mixtures were higher than those of the soil alone. However, a portion of the increase in density was attributed to the weight of the admixture that was added to the soils.

The test results indicated that the compactive effort required to produce a given density of soil was decreased, for most of the soils, by the use of calcium chloride and also that the calcium chloride was most effective at low compactive efforts.

Tests were made to determine the effect of calcium chloride on the pH of soils. The results showed that the pH of the soil-calcium chloride mixtures were less than those of the same soils with no calcium chloride.

Several of the soils were tested to determine the effect of calcium chloride on their plasticity. The results indicated calcium chloride lowered both the liquid and plastic limits of some of the soils.

The results of load-penetration tests showed that the penetration resistance of the specimens containing the admixture were somewhat less than the penetration resistance of the soil alone.

To determine the effect of calcium chloride admixture on the water retention characteristics of soils, a series of drying tests was performed. The results of these tests indicated calcium chloride retarded the drying out of the soils when subjected to accelerated drying. The results of drying and re-wetting tests showed that the moisture contents of the specimens containing calcium chloride were lower after the drying and wetting cycle than those with no calcium chloride.

Calcium chloride has been used on highway work as a dust palliative and as a stabilizing agent due to its moisture retention and surface tension properties. However, the available data have not been conclusive as to its effect on the various physical properties of soils.

In the past few years some work has been done along this line. The data indicate that