HIGHWAY EXPERIENCE WITH THIXOTROPIC VOLCANIC CLAY

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SYNOPSIS

The soils on the eastern slopes of Mauna Kea on the island of Hawaii were formed by the laterization of volcanic ash under conditions of continuous moisture. Because of thixotropy, chemical composition and exceedingly high natural moisture contents, these soils possess unusual properties. In the natural state, they possess the stable properties of a solid although the moisture contents are in excess of the plastic limit and many times in excess even of the liquid limit. When remolded, these soils become plastic or even semi-liquid (at the same moisture contents) resulting in difficult construction problems. Another property is that of self-agglutination so that upon complete drying the material turns granular. When used as highway subgrades, slow consolidation takes place so that after a period of years the material becomes decidedly more tufaceous. The peculiar properties of these soils are believed to be due to the presence of silica gel.

The islands comprising the Territory of Hawaii are of volcanic origin. Most of the land area is covered or underlain by lava flows. In addition to lava flows there have been eruptions of volcanic ash. The majority of Hawaiian soils have been derived from lava or ash by the soil forming process of laterization.

Bedrich Fruhauf (*1*)[1], has pointed out that, from an engineering standpoint, lateritic soils possess properties different from those of non-lateritic soils with comparable test constants such as grading, liquid limit, plasticity index, etc. In the discussion of Fruhauf's paper by Edward A. Willis of the Public Roads Administration, mention was made of an unusual soil from the Papaikou-Pepeekeo Section of the Hawaii Belt Road on the island of Hawaii. This particular soil is the result of the laterization of volcanic ash. An account of some of the construction difficulties encountered and experience gained in the handling of this unusual soil is given herewith.

GEOLOGIC AND CLIMATIC BACKGROUND

Figure 1 is a map of the island of Hawaii, the largest of the Hawaiian archipelago, from which the Territory of Hawaii takes its name.

As may be seen from the map, the island of Hawaii is roughly triangular in shape and consists of the mountain masses of the Kohala Range, Mauna Kea, Mauna Loa and Hualalai. Of these four mountain masses, Mauna Loa, altitude 13,680 ft., is still active volcanically, Hualalai has been active within historic time (last eruption in 1801), while Mauna Kea, altitude 13,784 ft., and the Kohala mountains are extinct.

The lavas of the Kohala mountains, because of their age, are now covered with a fairly thick layer of red lateritic soil, the result of the weathering of the underlying lava rocks. Much of the lava flows of Hualalai and Mauna Loa are fairly recent, and are barren of soil. On the other hand, the lavas from Mauna Kea are covered with varying thicknesses of volcanic ash. The source of this ash was Mauna Kea.

There seem to be geologic evidences that the ash covering some of the Mauna Loa lavas, even as far away as Pahala, originated from Mauna Kea (*2*) (*3*).

Volcanic ashes are aeolian deposits and so the wind velocity and distance from the source undoubtedly affected the nature of the ash deposit in any given locality. As distance from the source increases, the ash deposits become finer. An air-deposited mass tends to be exceedingly porous since the particles fall directly into place without shuffling and readjustment into more stable (and hence more dense) positions. Some ash deposits in the Pahala region (semi-arid) resemble loess and have indeed been described as such (*4*).

The prevailing moisture-laden winds blow inland from the northeast (the northeast trade winds) and striking against the cold mass of

[1] Italicized figures in parentheses refer to the list of references at the end of the paper.

Mauna Kea, their moisture is precipitated. The eastern slopes of Mauna Kea are, therefore, generally rainy. From Hilo north to under conditions of continuous moisture. The result is a type of soil with unusual engineering characteristics and the latter together with

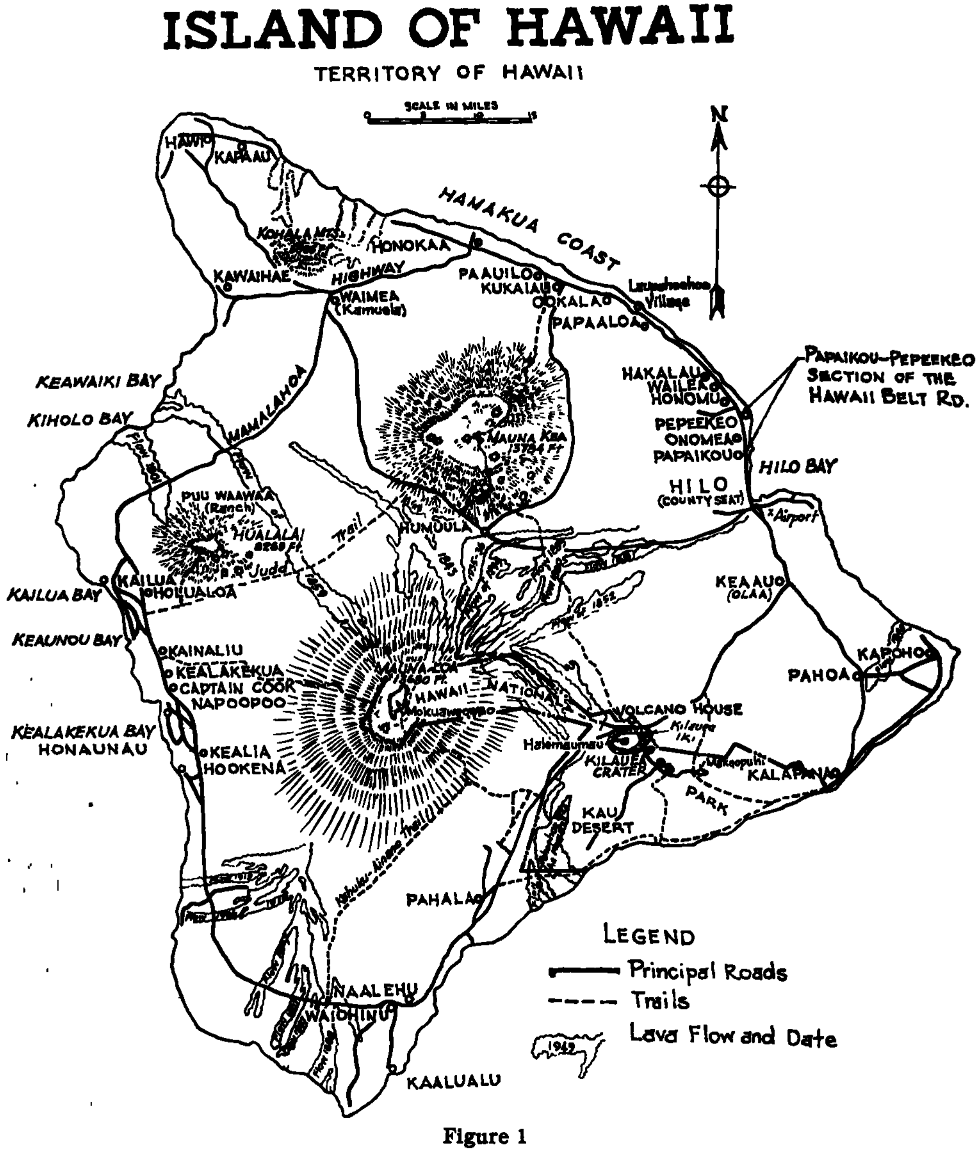

Figure 1

Hakalau, the annual rainfall exceeds 200 in. The Papaikou-Pepeekeo section of the Hawaii Belt Road, now under construction, is located in this region of heavy rainfall.

From the Wailuku river in Hilo north to about Laupahoehoe, the ash has weathered exceedingly rainy conditions have given rise to difficult construction problems.

Due to the great amount of rainfall, the weathering of the ash along the eastern slopes of Mauna Kea has been markedly different from the weathering of the ash of semi-arid regions such as Pahala. Because of a common origin, as stated previously, many writers, including Mr. Fruhauf in the aforementioned paper (*1*), refer to the ash along the eastern slopes of Mauna Kea as "Pahala ash". In our studies, which are directed mainly towards the attainment of engineering objectives, we have preferred the term "Pepeekeo ash" for that particular kind of weathered volcanic ash found along the route of the Papaikou-Pepeekeo section of the Hawaii Belt Road. The use of local place names seems preferable since it helps to recall and correlate certain construction experiencs associated with projects in those localities.

Pedologists, on the other hand, have described this particular soil as "hydrol humic" since, as mentioned, it has been formed under weathering conditions of continuous moisture (*5*).

One of the great difficulties in connection with highway construction in this part of the island of Hawaii is the presence of numerous gulches. To cross these gulches in a direct line, in many instances calls for viaducts 100 ft. or so high and several hundred feet long. In order to avoid expensive structures, the present highway, built in the days before the advent of the modern high-speed automobile when sight-distance, alignment and width were of minor importance, follows a tortuous path in and out of these gulches. The winding alignment called in general for shallower cuts and fills.

Present highway standards call for a straighter alignment and, in general, heavier cuts and fills. Experience shows that deep cuts in Pepeekeo ash are practicable but building embankments with this ash soil presents formidable construction difficulties.

Figure 2, is a view looking along the centerline of the new alignment. The photograph serves to give a good idea of the rugged nature of the terrain.

In order to obtain information on the nature and the relative ease or difficulty of excavation, one naturally turns to an examination of existing road cuts. Figure 3, is typical. Note the relatively steep slopes. Cut slopes of $\frac{1}{2}$ to 1 and even $\frac{1}{4}$ to 1 are obviously stable and since hard rock is not in evidence, one naturally (and correctly) comes to the conclusion that excavation will be relatively easy; that is, drilling and blasting will be negligible.

Soil tests.—Soil samples as received from the field were very moist and highly plastic. Pre-

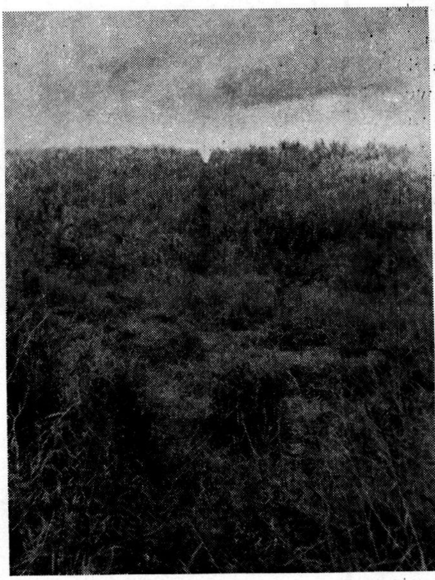

Figure 2. View Looking Along Center Line Of Project

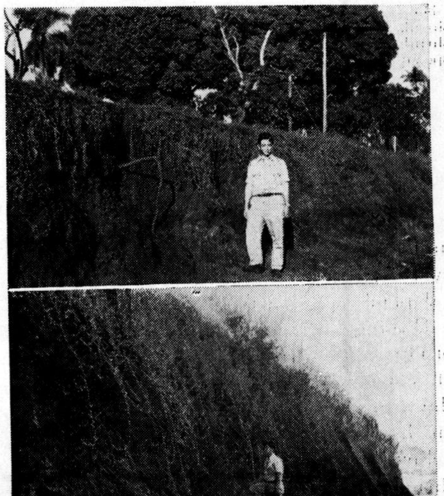

Figure 3. Cut Slopes in Pepeekeo Soil

paratory to making the usual tests for liquid limit, plastic limit, grading, compaction, etc. the samples were air-dried in the laboratory as called for by standard AASHO procedures.

The air-dried samples when tested for plastic limit were found to be granular and no amount of wetting would restore their original plastic properties. The soil in its original plastic state when wet-sieved showed as much as 80 percent or more passing the No. 200 sieve. The same soil when dried showed less than 15 percent passing the No. 200 sieve. It was, therefore, concluded that in the drying-out process, the clay particles must have cemented themselves together into larger particles. Examination showed that these cemented particles resembled volcanic tuff. As a matter of fact, these cemented particles are artificial tuff. Samples kept immersed continuously under water in the laboratory for over 2 years show no indication that the original plastic state will ever be regained.

TABLE 1
LABORATORY TEST RESULTS FOR PEPEEKEO VOLCANIC ASH SOIL

Test	Hawaii Highway Dept		P R. A.			California Research Corporation	
	(a)	(b)	(c)	(d)	(e)	(f)	(g)
Liquid limit	179	136	217	245	230	170	168
Plasticity Index	92	33	71	110	95	61	57
Field Moisture Equivalent	113	120	204		248		
Centrifuge Moisture Equivalent	155	137	170		206		
Shrinkage limit	89	102	44	44	46	86	71
Shrinkage ratio	0 74	.70	1.17	1.17	1.19	0 8	0 9
Specific gravity	3 10				2 80	2.90	2 90

(a) Sample No. 190 B
(b) Sample No 190 C
(c) Sample No. 71730 (moist)
(d) Sample No. 71730 (wet)
(e) Sample No. 71731 (wet)
(f) Sample No. P-791 (wet)
(g) Sample No P-792 (wet)

Having established the fact that drying produces an irreversible change in the nature of the material, it was decided to wet-sieve the sample over the No. 40 sieve and then to air-dry the fraction passing through to just below the estimated liquid limit, plastic limit, shrinkage limit, etc. Then the samples were re-wetted in the usual way. If the estimated liquid limit, for example, proved too high, the test was repeated with a lower estimated liquid limit. This partial drying, however slight, undoubtedly affected the plastic nature of the samples, but the results obtained were obviously closer to their true values than if the samples had been dried completely.

Due to the unusual nature of the soil, samples were sent to a number of other laboratories for check purposes. The results of our own tests together with those of the Public Roads Administration and the California Reseach Corporation of Richmond, California, are given in Table 1.

It is evident that there is considerable variation in test results from sample to sample due probably to the extent of air drying prior to testing. In spite of such variations, the tests clearly reveal a type of soil which in its natural state has high moisture holding qualities.

The P.R.A. results have been reported previously. In connection with the P.R.A. tests, Edward A. Willis has already described the special procedures that were used (*1*).

The hydrometer analysis for the determination of the amount of silt and clay size particles proved difficult because of pronounced flocculation. The clay including colloids, amounted to as much as 56 percent of the total and possibly more in some samples. From a highway construction point of view, however, the physical nature of the clay and its engineering properties are of more immediate importance. These points will be dealt with later.

Borings taken during the course of the soil survey showed that the ash cover was as much as 30 ft. thick and that except for a few short sections, the underlying Mauna Kea lavas were far below subgrade.

Numerous moisture samples taken from various depths indicated an exceedingly high natural moisture content. The data which have been plotted as Figure 4, seem to indicate some correlation between depth and moisture content. Thus the material below a depth of 15 ft. appears, in general, to have a lower moisture content than the material in the zone above.

Some exceedingly high values of moisture content were encountered in a few samples. The highest was 560 percent but moisture contents in excess of 275 percent, occur in pockets and are believed to be exceptional. Hence they have been omitted from Figure 4.

Moisture contents were determined according to standard methods by oven-drying at 230 F. and are expressed in terms of dry weight.

Samples taken approximately 1 in. inside the face of the cut shown in Figure 3, showed a moisture content in excess of 180 percent. Farther in, the moisture content increases to well over 200 percent and since there is no seepage of water, the contained moisture cannot be free gravitational water.

From the foregoing, it is evident that the natural moisture content of this soil is far above the plastic limit and in many cases beyond even the liquid limit; and yet as can be seen from Figure 3, the material in its natural, undisturbed state has the characteristics of a solid rather than of a plastic or liquid mass. The reason for this stability is that the material has a cemented structure, although the cementation is relatively weak. Wentworth has indeed called it palagonitic tuff (2).

Thixotropy—Thixotropy in clays is defined as that property which manifests itself in a loss of consistency upon manipulation or working with a subsequent regain of consistency after a period of rest.

The material in cuts such as shown in Figure 3, appears solid and relatively dry. Small pieces when taken in hand and examined look crumbly. Upon repeated kneeding and molding, the material becomes plastic and begins to feel and appear moist. After a period of rest, it regains its former crumbly and solid appearance. Pepeekeo ash, therefore, possesses thixotropic properties.

It is obviously impossible to take earth from a cut and deposit it in an embankment without some "working" of the material. Due to thixotropy, this "working" of the material causes it to lose some or all of its natural stability. It was, therefore, early recognized that the most difficult problem in highway construction in the Pepeekeo region would be to build stable embankments out of material that could be rendered unstable by the mere act of handling it. Allowing the soil to dry out offers only a partial solution as will be shown hereafter. Furthermore, there are no deposits of better quality soil within economical hauling distance. The nearest approach, is the top soil of the region which, as will be shown later, has desirable engineering properties. But using top soil as borrow is impractical because it occurs only as a very thin layer over the undersoil.

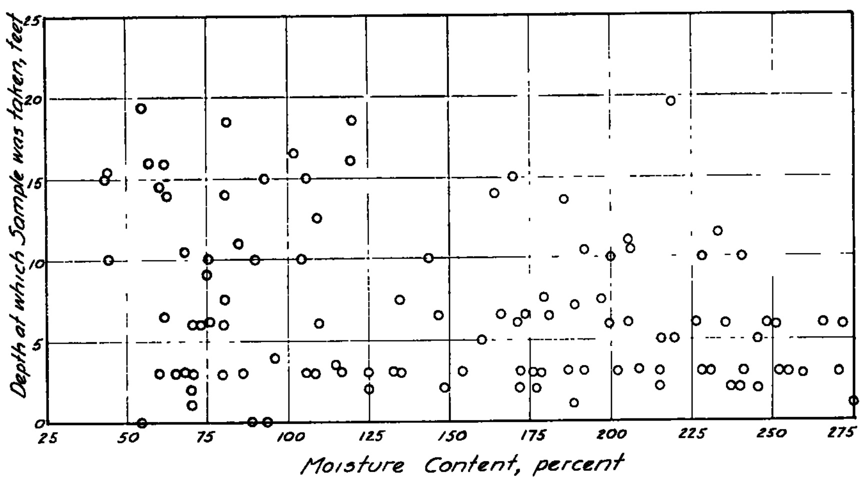

Figure 4. Scatter Diagram Showing Moisture Contents of Samples and Depths at Which Taken

In the determination of optimum moisture and maximum compacted density, if Pepeekeo ash is first air-dried as called for in the standard procedure, AASHO method T 99-42, it is evident that the results would not be representative, for the reason previously given, viz. air-drying effects an irreversible change in the characteristics of the material from one that is highly plastic to one that is less plastic or entirely non-plastic depending on the extent of drying.

The compaction curve was, therefore, determined by air-drying the soil in increments, making a compaction test and determining the wet density, moisture content, and dry density for each increment. The solid (lower) line curve of Figure 5 was thus obtained.

Edward A. Willis has already given a compaction curve determined in this manner (*1*).

If after allowing the sample to air-dry to a certain extent, say down to 75 percent, it is re-wetted in increments and compacted in the usual way, a series of points will be obtained which will plot as a curve that is concave down, such as curve B in Figure 5. Similarly if the sample is air-dried further and again re-wetted and compacted in increments, we obtain another curve with a slightly higher maximum such as curve C, Figure 5. Thus the optimum moisture and maximum density is dependent on the amount of drying that

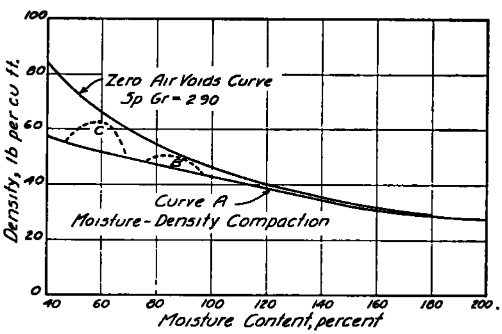

Figure 5. Compaction Curves for Pepeekeo Soil. Below approximately 40 percent moisture the soil becomes increasingly granular.

took place prior to re-wetting. What probably happens is that each increment of drying induces some slight irreversible change in the character of the material at the exposed surface.

In practice re-wetting to attain any theoretical maximum is out of the question because of the naturally high moisture content. When it is considered that precipitation amounting to 40 in. per month and as intense as 10 in. in 2 hours is not uncommon in this region, it is evident that, however desirable, too much reliance cannot be placed on drying out the soil. And since the contained moisture is not free water drainage is impossible.

Experience has shown that if the moisture content is known the solid line curve A, Figure 5, furnishes a reasonably close estimate of the density that can be expected. Table 2 shows a comparison of the actual densities obtained during construction and the densities as determined from curve A of Figure 5 for the moisture contents given. The data have also been plotted as Figure 6.

A dry density of 50 lb. per cu. ft. has been arbitrarily adopted as the standard or Proctor density. This density is the lowest of the maxima in Figure 5.

The compaction requirements of embankments on this project were set at 90 percent of standard for the top 6 ft. and 60 percent of standard below that. Experience has shown that 90 percent compaction is difficult to obtain while 60 percent is a reasonable requirement. The latter is equivalent to a compacted dry density of 30 lb. per cu. ft.

The natural in-place density of the undisturbed material varies and is invariably low compared to the usual standards. The data are given in Table 3. The average density from Table 3 is 38 lb. per cu. ft.

The ash soil found in drier regions can be compacted by laboratory methods to a density of from 60 to 75 lb. per cu. ft. Pepeekeo ash when sufficiently dried can also be compacted to a density of around 75 lb. per cu. ft. It is interesting to consider what this means in terms of construction quantities.

Consider, for example, material coming out of a cut at a wet density of 80 lb. per cu. ft. and a moisture content of 200 percent. These values are fairly representative. We will then have the following:

Wet density	80 lb. per cu. ft.
Moisture content	200 per cent

$$\text{Dry density} = \frac{\text{wet density} \times 100}{\text{percent moisture} + 100}$$

$$= \frac{80}{200 + 100} \times 100 = 26.67 \text{ lb. per cu. ft.}$$

Required density in fill = 75 lb. per cu. ft.

Shrinkage from cut to fill

$$= \frac{75 - 26.67}{26.67} \times 100$$

$$= 181 \text{ percent}$$

It is thus seen that if the material were to be completely dried out, 2.8 cu. yd. in cut will be required to make 1 cu. yd. of fill. Aside from the practical impossibility of complete drying as previously pointed out, the economics of the situation forbid making such a big increase in excavation quantities.

Excavation methods—What has been said applies to the undersoil. The top soil, which varies in depth from 2 in. to about a foot and more but is generally about 6 in., is of a distinctly less plastic nature than the undersoil. It can be distinguished from the latter by its darker color and slightly granular appearance. The undersoil is generally of a red, brown, or buff color.

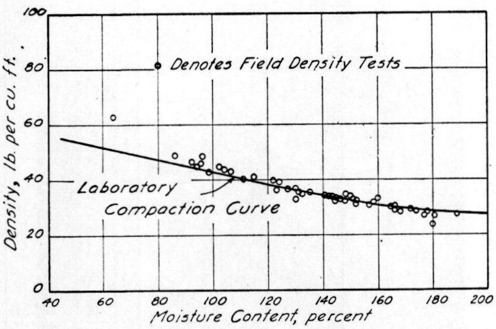

Figure 6. Relation of Field Densities to Laboratory Compaction Curve.

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When freshly exposed, the undersoil is ol a cheese-like consistency. The surface wil be waxy and slippery like wet soap. The bearing power is low and rubber tired vehicles cut deep ruts in it and soon bog down. See Figure 7.

It was previously stated that the undersoil when dried in the laboratory turned irreversibly to a non-plastic tuff. A similar change takes place in the field. The distinctly clayey surface slowly turns granular and becomes noticeably so in a few weeks' time. The granular surface is then subject to secondary weathering and traffic abrasion.

The natural top soil of the region was probably formed in the manner described. The rate of formation is slow. Thus on one highway project where the newly excavated roadway surface had been left continuously exposed for 5 years, (work was stopped on account of war) the thickness of the granular layer formed by this method of natural atmospheric drying amounted to at most 2 in.

One sample of top soil gave the following test results:

Results of Tests on Top Soil at Pepeekeo

Liquid limit	64
Plasticity index	5.6
Centrifuge moisture equivalent	50
Field moisture equivalent	63
Shrinkage limit	45
Shrinkage ratio	1.11
Specific gravity	2.79

Grading

Sieve No.	*Passing %*
4	100
10	98
20	93
40	83
60	74
150	57
200	52
Clay	15

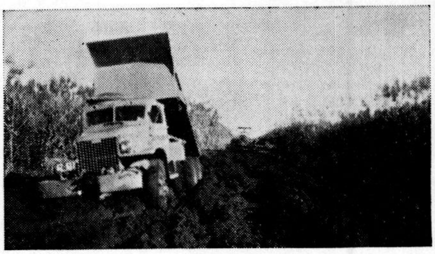

Figure 7. Trucks Bogging Down

These values give some idea of the differ ence in characteristics between top soil and undersoil. Test values will vary considerably from sample to sample depending upon extent of atmospheric drying, secondary weathering, degree of mixing with undersoil, etc.

From a construction standpoint, the most important characteristic of the top soil is its high bearing power when contrasted with the plastic undersoil. Field experience shows that a layer of top soil of 6 in. apparent thickness, as distinguished by its darker color, is sufficient to support the weight of construction trucks without the latter bogging down.

One successful method of excavation is to use a drag line. The latter will then always be moving backwards over the topsoil and will, therefore, not bog down. The trucks to carry the excavated material must also be kept travelling on topsoil.

The conventional type of rubber tired carryall is extremely vulnerable. One or two passes will remove the top soil and the equipment then bogs down. Providing carryalls with tracks improves performance. Figure 8 shows a carryall fitted with Athee tracks. This arrangement has proved successful, but even tracks are no guarantee against bogging down. See Figure 9.

Another successful method evolved by the contractor on this project is to excavate a half-section at a time using any convenient equipment—bulldozers, dragline, etc. The topsoil on the unexcavated half-section serves as an access road for trucks to haul away excavated material. In the case of deep cuts, it is not generally possible to excavate the

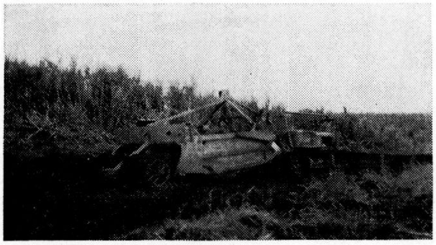

Figure 8. Carryall Fitted with Athee Tracks

half-section to grade in one lift. After excavating the half-section to as near grade as possible, it (the half-section) is covered with select material. The incompletely excavated half-section then serves as a base of operations from which to excavate to grade the other half-section. The select material on the first incomplete half-section is then salvaged by bulldozing it over to the completed half-section. After the completed half-section is thus covered with select material, it in turn is used as a base of operations to excavate to grade the incomplete half-section. Covering this with select material then provides an effective all-weather surface for construction purposes.

For shallow cuts and fills bulldozers and carryalls (fitted with tracks) have proved fairly successful. As stated previously, continued passage of equipment over the same area eventually works the clay into a semi-fluid mass. With a little experience, one learns to gage the state of the clay and get his equipment off before it bogs down.

Embankments—The greatest difficulties in the handling of this volcanic ash soil lie in the construction of embankments. Due to its naturally high moisture content and its thixotropic nature, the ash is an extremely difficult material to compact. This is because the process of spreading and compacting necessarily manipulates the material so that the ash, which in cut sections, such as Figure 3, is in a solid state, is worked into a plastic state. Upon further working, the plastic state is changed to the semi-liquid state, all without additional moisture.

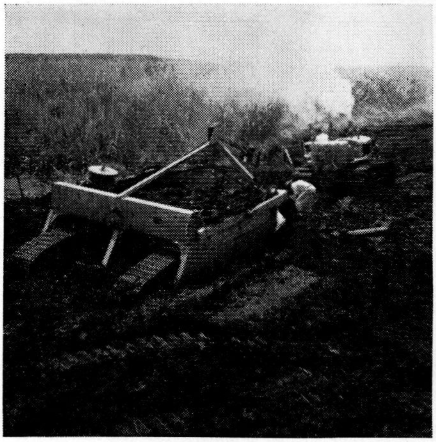

Figure 9. Track-fitted Carryall Bogging Down

The usual types of compacting equipment such as flat-wheel, sheepsfoot and pneumatic tired rollers have proved impractical. Compaction with bulldozers has given the best results thus far.

In the case of shallow fills up to about 5 ft., or if the material has a moisture content of less than 100 percent, the difficulties are not too great. In the case of deeper fills or if the moisture content is 200 percent and over, the difficulties are greatly increased.

In general, it is not possible to compact in thin layers. With thin layers, a given volume of material is subject to a more intense working than in the case of thick layers, the weight of compacting equipment remaining the same. A thickness of layer of about 3 ft. appears to give the best results. With such a thick layer, the compacting equipment does not impart plasticity throughout the entire thickness of the lift. Hence, the upper part of the lift is compacted in a plastic state while the lower part is compacted in a solid state. If bogging down of equipment is imminent, it is imperative to cease operations and transfer the equipment to some other location. During this period of rest, the material will not only gradually compact due to its own weight, but the part that is plastic will set and regain its lost consistency. This setting is not due to dehydration but is the result of the jellifying of the mass. The embankment can then be further compacted and additional material spread on it. The above process is slow but it is the most reliable method thus far evolved.

TABLE 2
DENSITIES OBTAINED IN COMPACTING EMBANKMENTS OF PEPEEKEO ASH

Test No.	Station	Moisture Content %	Compacted Density lb. per cu. ft.	Approx. Density from Compaction Curve
23	34 + 75	132	35.5	38
44	34 + 90	157	31.5	31
72	34 + 90	127	37.0	37
10	35 + 00	181	27.3	28.5
45	35 + 10	172	29.6	29
47	35 + 10	152	32.5	32
63	35 + 10	146	33.7	33
29	77 + 00	92	46.8	44.5
28	90 + 00	94	45.6	44
53	96 + 17	157	31.6	31
6	143 + 95	111	40.5	40.5
17	148 + 00	150	34.0	32.5
5	150 + 00	145	33.8	33
19	150 + 00	124	39.5	37.5
71	150 + 00	130	37.3	36.5
20	151 + 00	115	41.5	39.5
21	152 + 00	269	19.7	[a]
27	152 + 00	106	43.3	41.5
68	170 + 75	143	34.1	34
64	171 + 00	174	28.4	29
2	171 + 03	178	28.0	28.5
64	171 + 03	174	28.4	29
70	171 + 03	180	24.0	28.5
33	187 + 50	86	49.1	46
75	35 + 00	98	43.5	43
76	96 + 17.7	189	27.2	28
78	35 + 00	149	34.8	32.5
79	171 + 00	142	34.7	34
80	182 + 50	95	46.1	43.5
81	182 + 50	102	44.8	42
82	20 + 07	141	35.0	34
84	96 + 17	123	36.9	37.5
85	127 + 00	135	35.7	35
90	20 + 50	122	40.0	38
92	76 + 75	177	27.1	28.5
94	76 + 75	168	28.6	29.5
95	126 + 75	165	30.5	30
96	126 + 75	144	32.7	33.5
98	91 + 00	96	49.0	43.5
101	182 + 50	131	35.8	36
102	76 + 50	104	44.3	42
103	76 + 50	64	63.1	51
109	90 + 50	148	32.7	32.5
111	30 + 18	160	33.8	31
112	76 + 50	166	29.6	30
116	11 + 75	166	30.2	30
117	11 + 50	130	33.2	36.5
118	12 + 50	152	31.8	32
119	30 + 18	151	33.0	32
120	76 + 50	151	31.6	32
121	90 + 50	158	31.5	31
Average...		142	35.5	

[a] Beyond limits of compaction curve.

The densities obtained in the construction of embankments have already been given in Table 2.

Due to the nature of the material, it is virtually impossible to use compacting equipment clear to the edges of embankments. Hence, the shoulder areas and the side slopes will be in a somewhat loose state with air pockets. The latter in times of heavy rains collect water, which tends to lessen the stability of side slopes. Experience, however, shows that if Pepeekeo soil is not in a plastic state, and if it has acquired the right degree of set, it will not slake in water and will resist the erosive action of rain falling directly on the embankment. This is illustrated in Figure 10 which shows two pieces of Pepeekeo soil which had been continuously immersed in water for over two months with no sign of slacking or loss of stability. The moisture content was 180 percent.

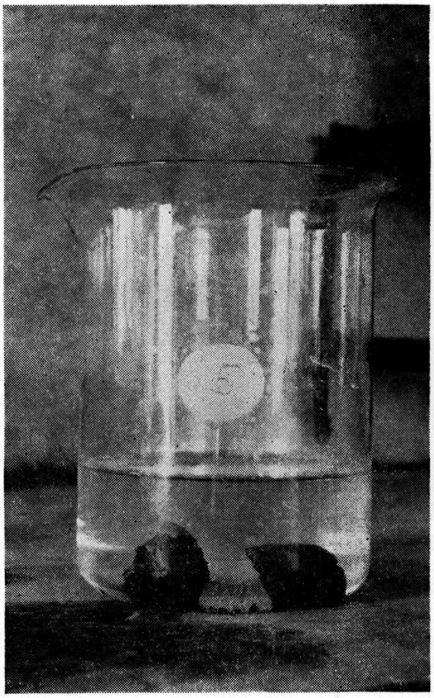

Figure 10. Showing Impermeable, Non-slacking Qualities of Pepeekeo Soil.

still reflect the original high values of moisture content.

The intrusion of the ash into the base above has been practically negligible. This observation applies even to open graded material such as a Telford base

To explain this lack of intrusion, it is only necessary to recall from Figure 10 that the ash does not absorb any additional moisture. Hence any change in moisture must be downward. The pressure applied by traffic does not result in a working of the clay provided the base is of a thickness that results in sufficient distribution of load over the subgrade.[2] Hence, a slow consolidation takes place. In addition, the material sets, that is changes from a plastic to a solid state even though the moisture content may be above the plastic limit. In these respects, Pepeekeo ash differs from the usual type plastic clays which readily absorb additional moisture.

The bearing value of Pepeekeo ash when freshly remolded is low due to plasticity and high moisture content. Because of complications due to varying degrees of plasticity, the results are likely to be erratic if expressed in terms of a single variable such as moisture content or density of compaction. In general, the initial C.B.R. for the usual range of moisture contents from 175 percent up is approximately 2 percent for 0.1-in. penetration. As the material sets, the C.B.R. for the same moisture content may increase to as much as 5 percent. Upon consolidation through a period of years, the C.B.R. values increase still further.

Table 5 gives some data regarding C.B.R. values of ash subgrades. The "inplace" C.B.R values have been plotted in Figure 13. The plotted data seem to point to a hyperbolic relationship between moisture contents and C.B.R. values. However, the data being limited, the relationship has not been investigated further.

TABLE 5
TEST DATA ON BEARING VALUE OF ASH SUBGRADES

Project	Location of Sample	Moisture Content of Sample	Dry Density, lb. per cu. ft.	C.B.R. for 0.1-in. Penetration	Sample from Cut or Fill Area	Remarks
		%		%		
14B	13 + 00	172	27	5	Cut	6 in. cores cut from subgrade; soaked
Pepeekeo Road		230	22	2	Cut	6 in. cores cut from subgrade; soaked
Pepeekeo Road		178	28	2+	Cut	6 in. cores cut from subgrade; soaked
Pepeekeo Road		208	24	5	Fill	
Pepeekeo Road		255	20	2	Fill	6 in. cores cut from subgrade; soaked
Honolii Gulch Road		65	58	9	Cut	6-in. cores cut from subgrade; soaked
		72	35	8 to 10	Cut	
14C	320 + 50	66		12	Fill	
14C	303 + 50	45		16	Cut	Shoulder area
14A	48 + 50	61		7	Cut	Shoulder area
14G	600 + 00	57		26	Fill	
14G	600 + 00	56		5	Fill	Shoulder area
14G	555 + 00	119		9	Cut	
14G	555 + 00	87		6	Cut	Shoulder area
14H	623 + 75	131		10	Cut	
14H	623 + 75	181		6	Cut	Shoulder area
14I	636 + 00	43		43	Cut	Ash here is tufaceous
14I	636 + 00	47		23	Cut	Shoulder area
26A	705 + 00	79		13	Cut	
26A	705 + 00	78		3+	Cut	Shoulder area
26A	755 + 00	67		14	Cut	Subgrade
26A	755 + 00	88		11	Cut	Shoulder area

Note: The bearing values for projects 14C, 14A, 14G, 14H, 14I and 26A are "in-place C.B.R." values.
Unless otherwise shown under "remarks", the C B R. values are of the subgrade under pavement areas.
The in-place C.B R values for shoulder areas were taken 1 ft below ground level All shoulders untreated.

CHEMICAL ANALYSIS AND NATURE OF CLAY MINERALS

It remains now to consider briefly the chemical analysis of Pepeekeo ash and the nature of its clay minerals.

Numerous analyses of Hawaiian soils are available (*1*), (*5*), (*6*), (*7*). To begin with the parent materials of Hawaiian soils, except those of marine, or alluvial origin are

[2] Specifically, it must be thick enough to prevent pumping action.

either volcanic lava rocks or volcanic ashes. Hawaiian lava rocks and ashes are characterized by a rather low silica content, which seldom exceeds 50 percent or so.

In the formation of soil through the process of weathering and chemical decomposition, vegetation and climate are factors of great importance. The soils of Pepeekeo were formed under conditions of luxuriant vegetation and a very humid tropical climate. Most of the silica and the bases have been leached out, resulting in a soil rich in the sesquioxides of aluminum and iron. In a soil that is continually wet such as the present one, the aluminum tends to be more stable than the iron, whereas in a drier climate the aluminum tends to leach out, resulting in a more ferruginous laterite. Moreover, much of the aluminum is believed to be in the form of hydroxides or hydrates instead of as aluminum silicates in colloidal form as in true clays.

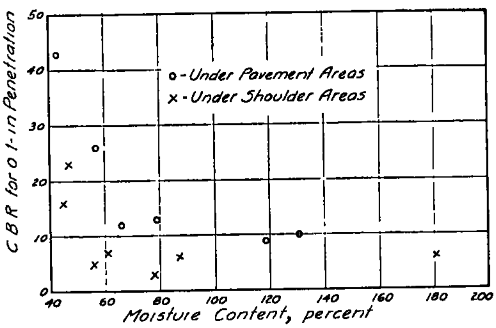

Figure 13. In-Place C.B.R. Values of Consolidated Ash Subgrades.

The usual methods of chemical analysis report the various chemical elements present as oxides whereas actually they may be present in the soil as oxides, hydrates or hydrated silicates. Thus, chemical analysis fails to explain the peculiar physical characteristics of Pepeekeo ash.

Chemical analyses of Pepeekeo soil as given by various investigators differ in only minor respects from that already given by Willis (*1*). The outstanding characteristics are high organic content, 20 percent and over, the highest in any Hawaiian soil; the low percentage of silica, 4 to 12 percent; and the relatively high percentage of combined aluminum and iron oxides, 40 to 65 percent.

On the assumption that Al_2O_3 remains constant, Wentworth shows that as much as 93 percent of the original silica is leached out, while the Fe_2O_3 content increases to three and one-half times the original (*2*). Thus, the weathering process is that of laterization.

The cation exchange capacity of Pepeekeo ash is high, 20 to 60 milliequivalents per 100 grams and 50 to 90 percent of the exchange capacity is due to the organic content. The buffering capacity is also high. The soil is approaching its ultimate pH (*5*) (*12*).

Research has identified clay minerals as kaolinite, montmorillonite, or hydrous micas (*8*), (*9*), (*10*). Dean (*11*) has established by differential thermal analysis, that the clay minerals in Hawaiian soils are kaolinite. He has also shown that the kaolinite content of Pepeekeo soil is low. Thus, it is concluded that the clay in this type of soil is an oxide clay.

The authorities quoted above directed their investigations mostly to the attainment of certain objectives in the field of agriculture. The data given, however, have possible application to engineeering problems involving the properties of clays (*10*).

Turning now to the geologists, we find that Wentworth (*2*) gives various modes of cementation of ash particles. The first consists of agglutination of particles while still hot and plastic. Another is by means of calcium carbonate in the form of calcite or aragonite. Still another is by the formation of gelatinous silica compounds with a subsequent dehydration similar to the setting of Portland cement. Lastly, there is the process of cementation through the formation of palagonite.

According to Wentworth (*2*) and Stearns (*3*) the ash we are considering is a palagonitic ash. Palagonite is a waxy yellow silica gel produced by the hydration of basaltic (volcanic) glass. The peculiar thixotropic and setting up properties of Pepeekeo ash are undoubtedly due to the presence of this silica gel in the form of palagonite. The yellow color mentioned above is many times oxidized to other colors such as red, brown, buff, and lavender.

CONCLUSION

The usual soil tests fail to adequately predict the service behavior of highly hydrated and thixotropic volcanic clay soils. Compaction tests are of some value in predicting densities that will be attained during construction. Just as important as compaction is the degree of plasticity imparted to the soil by construction processes.

ACKNOWLEDGMENT

In addition to the authors quoted, the writer wishes to extend his thanks for helpful discussions to Messrs. E. S. Kaaua and E. F. Morrison, engineers and fellow employees of the Territorial Highway Department. He is also greatly indebted to Dr. Chester K. Wentworth, geologist of the Honolulu Board of Water Supply for information on the geology of the island of Hawaii. The photographs have been taken from the files of the Territorial Highway Department of which Robert M. Belt is Territorial Highway Engineer. The project is being undertaken jointly with Federal and Territorial funds. R. W. Hendry is Highway Construction Engineer and Jack C. Myatt is Highway Design Engineer, both of the Territorial Highway Department while Frank R. Carlson is District Engineer for the Public Roads Administration. George Pollock and Company of Sacramento, California are the contractors.

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DISCUSSION

MR. EDWARD A. WILLIS, *Public Roads Administration:* Mr. Hirashima's most interesting paper serves a dual purpose. Primarily, it is a valuable contribution to the general fund of knowledge on soils. In addition, because of the unusual nature of some of the facts presented, it arouses the interest of soils engineers generally and tends to divert their thoughts, at least temporarily, from the more conventional approaches to the solution of soils problems.

As noted in the paper, the Public Roads Administration tested two samples of soil from the Papaikou-Pepeekeo section of the Hawaii Belt Road in 1946. It was of considerable interest to the writer, therefore, to read Mr. Hirashima's description of the behavior of the soils in their natural environment and to learn of the construction procedures which were resorted to in building a road under unfavorable climatic conditions from a material whose properties were continually changing as the work progressed.

It is encouraging to note that the fundamentals of soils engineering, if intelligently applied, can be of assistance to the highway engineer even when dealing with a material composed of 5 to 14 cu. ft. of water to every cubic foot of soil particles (moisture contents from 200 to 500 percent by weight).

The problem of controlling the compaction of embankments when laboratory tests indicated a series of optimum moisture contents and maximum densities depending on the degree of drying before the test was started, was successfully overcome by Mr. Hirashima and his co-workers. Having established the relationship between natural moisture content at the time of compaction and maximum attainable density (see Fig. 5) it was possible to estimate in the field the densities that were to be expected.

Advantage was taken during construction of the known thixotropic properties of the soil. Thus, when the soil began to lose its stability through manipulation, it was allowed to "rest" and regain its stability. While this procedure undoubtedly slowed up the work, it did permit the construction of stable embankments from a material which by all generally accepted criteria would be entirely unsuitable.

Some small measure of encouragement can be found for the proponents of the triaxial compression tests as a tool for designing pavement thicknesses even when dealing with so unusual a soil as this one. Triaxial tests were made in the Public Roads laboratory at a number of different moisture contents ranging from the driest to the wettest that might normally be anticipated. While Mr. Hirashima feels that the values for the coefficients of cohesion and friction obtained were too high, the suggested range in pavement thicknesses of $11\frac{1}{2}$ in. to 31 in. based on the triaxial test values, did bracket the pavement thicknesses of at least 18 in. in cut sections and 24 in. in fill section which have been found necessary to support vehicular traffic over these soils.

K. B. Hirashima, *Closure*—As stated by Mr. Willis, the fundamentals of soils engineering, when properly applied, are applicable to the type of soil under discussion. Nevertheless, as pointed out previously, the peculiar properties of this "Pepeekeo" type of volcanic ash call for certain modifications in testing procedures and in the criteria for judging field performance.

The outstanding characteristics of this soil are its unusually high natural moisture content, its thixotropic properties, and its property of changing irreversibly from a plastic to a non-plastic soil upon complete drying.

In addition to the above, this soil possesses another very important property not previously mentioned, viz., its property, while in the natural state, of resisting erosion. This property is easily seen to be highly valuable. During the construction of this project, there have been numerous rainstorms, many as severe as 8 to 12 in. in 24 hrs. At such times, heavy streams of water have been observed coming down certain cut slopes. It is hard to believe that no erosion takes place, and some erosion undoubtedly takes place even if merely to substantiate a principle, but examinations of cut slopes after heavy storms have failed to show any visible evidence of erosion. Of course, there is a limit to everything and so if the quantity or velocity of the storm waters is sufficiently great, erosion will take place. Figure A shows the discharge from a culvert. The photograph was taken shortly after a storm of 8 in. in 24 hrs. which taxed the culvert to capacity. Notice that the discharge water is clear and that it has not yet cut a distinct channel.

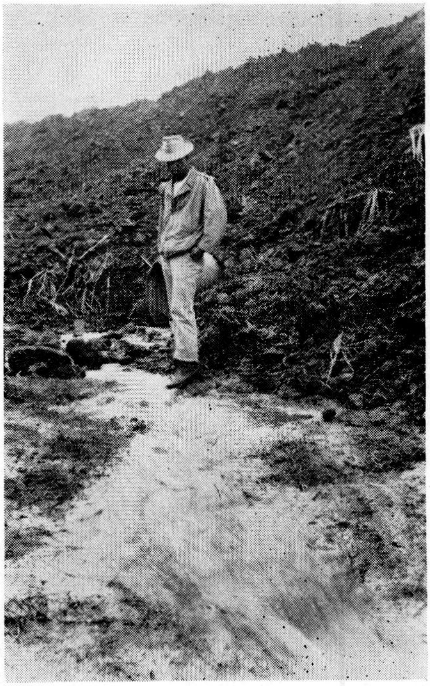

Figure A. Heavy Discharge from a Culvert Fails to Erode Pepeekeo Soil in Its Natural State.

The original ground here is very soft (because of its high moisture content) so that it is possible to push a steel rod into it for a distance of 10 ft. or so with very little effort.

The disturbed soil on the other hand, as previously stated, is easily eroded, which is one of the difficulties in the construction of embankments of this soil. If the soil is intensely manipulated, as with a Sauerman bucket, it takes on the property of viscous flow and becomes absolutely unstable. Again, because of thixotropy if the soil is allowed to rest, it will regain its property of resisting erosion. (The above described phenomena have actually been observed on this project.)

While in principle it is possible to build high embankments with this material, in practice there are certain difficulties that are almost overwhelming. One is the almost constant rainfall. Another is the limitations of compacting equipment which makes it impossible to compact the lower lifts to a density high enough so that there is no further consolidation as a result of the dead weight of material above. Hence unless a high embankment is built very, very slowly, sudden settlements are likely to occur, especially along the side slopes resulting in slides because of unbalanced horizontal forces.

The most expeditious method of constructing high embankments (in excess of 20 to 25 ft. is to build only the central core of Pepeekeo ash, using some type of stable borrow material for the sides. This type of embankment construction has proved successful.

The various tests, physical and chemical, made by the PRA proved invaluable throughout the construction of this project, and the writer wishes to acknowledge his indebtedness to the results reported by Mr. Willis and his co-workers. While not used directly, the triaxial test results were especially valuable in providing at least some estimate of stability where previously there was none.

A last thought. Judging from the literature "Pepeekeo" ash must be similar in some respects to the volcanic clay of Mexico City. It is hoped that a direct comparison of the two can some day be made.

LABORATORY EXPERIMENTS WITH LIME-SOIL MIXTURES

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SYNOPSIS

This paper reports the results of laboratory research conducted by the Engineering Experiment Station of Purdue University and sponsored by the National Lime Association. Mr. Jean E. Hittle, formerly soils engineer, on the staff of the University, planned the details of the project and directed most of the laboratory work reported here.

The experiments were organized in three broad parts on the basis of soil types used. The tests in Part I were run on fine-grained soils by Mr. Lu I Cheng, a graduate student. Tests in Part II were performed on naturally-occurring gravels and those in Part III on synthetic gravel-binder mixes.

Atterburg limit tests were run on 25 fine-grained soils. The plastic indexes of silty soils were increased slightly by additions of 2 and 5 percent of hydrated lime but the PI's of clay soils were lowered appreciably by the lime additive. Comparable tests on a few soils with lime dust as the additive gave higher values for the PI than were obtained with lime. Tests were run on several of the soils at 7 and 14 days after the lime and water were mixed with the soil but the curing time appeared to have little consistent effect on the results.

Proctor compaction curves were plotted for 11 of the fine-grained soils with 0, 2, and 5 percent of lime. With but two exceptions, the addition of lime was accompanied by decreases in maximum density. The changes varied from an increase of 1.2 percent to a decrease of 5.5 percent of the corresponding maximum density obtained with the raw soil. Plots of penetration resistance quite consistently showed increased resistance to penetration with successive amounts of lime.

Compaction curves were plotted for five natural gravels to determine optimum moisture contents, and CBR specimens were molded at or near this moisture content with 0, 2, and 5 percent of lime. CBR tests were made on each combina-