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Volcanic Ash and Laterite Soils in Highway Construction

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VOLCANIC ASH AND LATERITE SOILS IN HIGHWAY CONSTRUCTION

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HIGHWAY CONSTRUCTION PROBLEMS INVOLVING PLASTIC VOLCANIC ASH

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Synopsis

Along the eastern shores of the island of Hawaii are soils derived by weathering of volcanic ash under conditions of continuous moisture. This has resulted in a very plastic type of soil with natural moisture content generally close to 200 percent The contained moisture is not free water and hence cannot be drained. Due to the almost continuous rainfall, the deeper layers maintain their high moisture content. The surface layers, measuring 6 to 18 in. thick, are subject to partial drying. Drying effects an irreversible change in the soil from plastic to relatively non-plastic. Thus passenger cars and light trucks can travel over the topsoil. On the other hand, the undersoil, due to its high plasticity, will not support rubber-tired traffic; equipment with caterpillar tread, however, can be used successfully within certain limitations. These limitations stem from the fact that the undersoil is thixotropic, so any undue working or manipulation of the soil causes it to lose all stability, with resultant bogging down of equipment and sliding of embankment slopes.

Excavation can be handled expeditiously by using equipment such as a dragline, taking the material out in one lift as close as possible to required grade. The freshly exposed undersoil must be quickly covered with 18 in of select rocky material to form a temporary surface (which later becomes the sub-base) for trucks hauling away the excavated material. Carryalls fitted with Athee tracks have also proven successful under certain conditions.

Embankments are best constructed in alternate layers of 5 ft. of ash and 18 in. of stable rocky material. The latter serves as a temporary stable surface over which equipment such as trucks and trac-fitted carryalls can bring up material. Compaction is dependent upon construction traffic entirely. This layer method achieves satisfactory compaction without danger of too intensive working of the material. End-dump methods can be used for relatively low and short embankments. For long embankments and for heights over 15 ft., end-dump methods result in too intensive working of the material with consequent danger of slides

In a previous paper (1) an account was given of the difficulties encountered in highway construction involving a highly plastic clay soil, derived by weathering, from volcanic ash occurring on the island of Hawaii, which is the largest of the group of islands comprising the Territory of Hawaii. The present paper is a progress report on methods that have been devised and have proven successful in overcoming these difficulties.

Hawaii (see Fig. 1) is a volcanic island with several high mountains: Mauna Kea (13,784 feet), Mauna Loa (13,680 feet), Hualalai (8,269 feet), Kilauea (4,090 feet), and the Kohala Mountains (5,505 feet). Of these, Mauna Kea and the Kohala Mountains have long been volcanically extinct, Hualalai has been dormant since 1801, while Mauna Loa and Kilauea are still active. The climatic conditions and rainfall are greatly influenced by these high mountains.

Although there have been recorded ash eruptions from Kilauea volcano within recent time (last ash eruption in 1924), most of the ash deposits found throughout the island appear to have originated from Mauna Kea (2) and for the purpose of this paper will be so regarded.

The series of ash eruptions was the last phase in the volcanic activity of Mauna Kea. Previous to this it erupted lava. Hence, along its flanks, the ash everywhere appears at the surface and is underlain by older lava flows.

In contrast, along the slopes of

Mauna Loa, Kilauea, and Hualalai subsequent lava flows have covered much of the original ash beds.

The prevailing moisture-laden winds blow inland from the northeast. Striking against the high peaks of Mauna Kea and Mauna Loa, their moisture is precipitated. Since these northeasterly, or trade, winds blow practically the year around except for comparatively short periods of southerly winds, it follows that the east side of the island is a region of heavy rainfall, which, on the whole, is well distributed throughout the year. For the purpose of this paper, the region east of the Mauna Kea-Kilauea axis (Fig. 1) will be regarded as the wet side of the island and the region west of the axis the dry side.

The rainfall in the wet region varies from 100 to well over 300 in. per year. Accordingly, the ash deposits here have weathered under conditions of practically continuous moisture. This has resulted in a type of volcanic clay soil with exceedingly high natural moisture content, as high as 560 percent based on dry weight, to quote an extreme example. In general, as it affects the problem of highway construction, the natural moisture content of this plastic type of volcanic ash may be regarded as somewhere between 100 and 200 percent.

The Wailuku River in Hilo is the dividing line between the lava flows of Mauna Kea and Mauna Loa. North of the river are thick beds of ash, measuring 20 to 30 feet in thickness along the Hilo coast. Further north, along the Hamakua coast, the thickness of the ash beds lessons to somewhat less than half this amount. South of the river the ash, although still highly plastic, seldom measures more than about 10 ft. in thickness. Moreover, large areas of it are covered by later Mauna Loa lavas.

In the dry region west of the Mauna Kea-Kilauea axis, the ash has weathered under semi-arid conditions. As a result, its physical properties differ markedly from those of the ash found in the wet region. It occurs as a fine sand or silt deposit. At least one writer has referred to it as loess (3). The material is practically non-plastic and does not call for any unusual construction techniques. This paper is confined to the plastic type of volcanic ash occurring in the wet region east of the Mauna Kea-Kilauea axis.

REVIEW OF CONSTRUCTION DIFFICULTIES

The outstanding physical properties of this plastic type of volcanic ash affecting highway construction are exceedingly high natural moisture content, high liquid limit, high plasticity index, thixotropic properties, and irreversible change from a highly plastic clay to a non-plastic sand-silt upon drying. Data concerning test results were given in the previous paper referred to above and so will not be repeated here (1).

In spite of its exceedingly high natural moisture content, cut slopes in this soil as steep as 1/4 to 1 (horizontal to vertical) have been found to be stable, but because of its thixotropic properties, it loses its consistency once it is disturbed and worked and becomes increasingly unstable the more it is worked. Cases are on record where the soil was worked so intensely, with sauerman bucket of long travel that the soil started to flow like a viscous liquid (see Fig. 2). Since it is impossible to move earth from excavation to embankment without some working of the soil, the most difficult problem was how to build stable embankments with a material that could be rendered unstable by the mere act of handling it. Adding to the difficulties are the frequent, heavy rainstorms of the region which at times amount to as much as 12 in. in 24 hr. and occasionally even more. Because of these difficulties, highway construction projects in the past were invariably behind schedule. Completion dates were as much as 12 to 18 months behind contract time. That progress is being made is attested by the fact that recent projects have been completed months ahead of contract time.

EXCAVATION METHODS

The surface of a freshly opened cut in this plastic volcanic ash is so soft and slick that rubber-tired construction

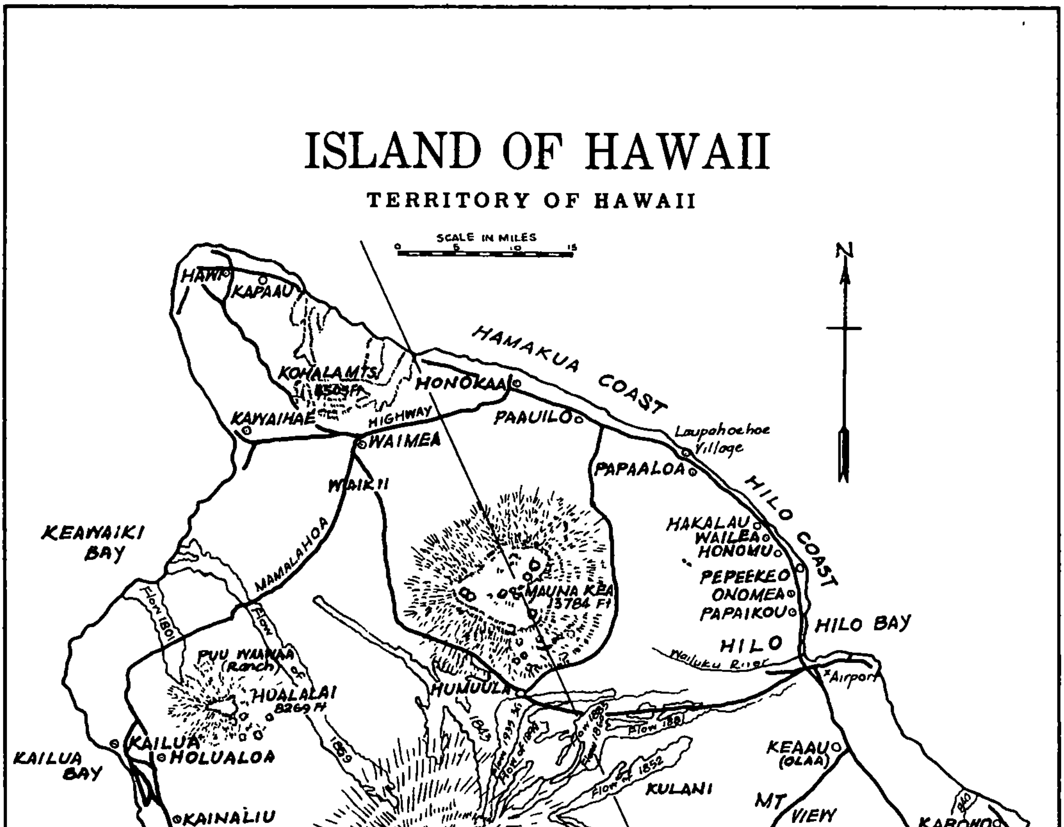
ISLAND OF HAWAII
TERRITORY OF HAWAII
SCALE IN MILES
0 5 10 15
N
HAWI
KAPAAU
KOHALA MTS.
KAWAIHAE
HIGHWAY
HONOKAA
HAMAKUA COAST
PAAUILO
WAIMEA
Laupahoehoe Village
PAPAALOA
HILO COAST
WAIKII
KEAWAIKI BAY
MAMALAHOA
HAKALAU
WAILEA
HONOMU
PEPEEKEO
ONOMEA
PAPAIKOU
MAUNA KEA 13784 Ft
HILO BAY
HILO
Wailuku River
Airport
Flow 1801
PUU WAAWAA (Ranch)
HUALALAI 8269 Ft
HUMUULA
Flow 1881
Flow of 1852
KAILUA BAY
KAILUA
HOLUALOA
KEAAU (OLAA)
KULANI
MT VIEW
KAINALIU
KEALAKEKUA
CAPTAIN COOK
NAPOOPOO
MAUNA LOA 13680 Ft.
Mokuaweoweo
PAHOA
KAPOHO
KEALAKEKUA BAY
Kilauea
Makaopuhi
KALAPANA
HOOKENA
KAU DESERT
MAUNAKEA-KILAUEA AXIS
PAHALA
Flow 1919
Flow 1926
LEGEND
Principal Roads
1950
Lava Flow and Date
NAALEHU
WAIOHINU
KAALUALU

Figure 1.

equipment, e.g., carryalls and trucks, cannot immediately travel over it. Over a period measured in weeks the surface dries partially, due to aeration. As mentioned above, this partial drying causes the ash to become less plastic. Upon complete drying, which does not occur to any great extent in the field because of the frequent rains, the originally highly plastic ash turns irreversibly to a non-plastic sand-silt. This

layer exposed, the equipment bogs down. If an effort is made to get the equipment out, the tractor, which must be of the caterpillar type, will necessarily work the soil more intensely. This intense working soon renders the soil almost semi-liquid and the entire assembly becomes immobile. Cases are on record where equipment has thus remained bogged down for days and weeks at a time. It is evident that

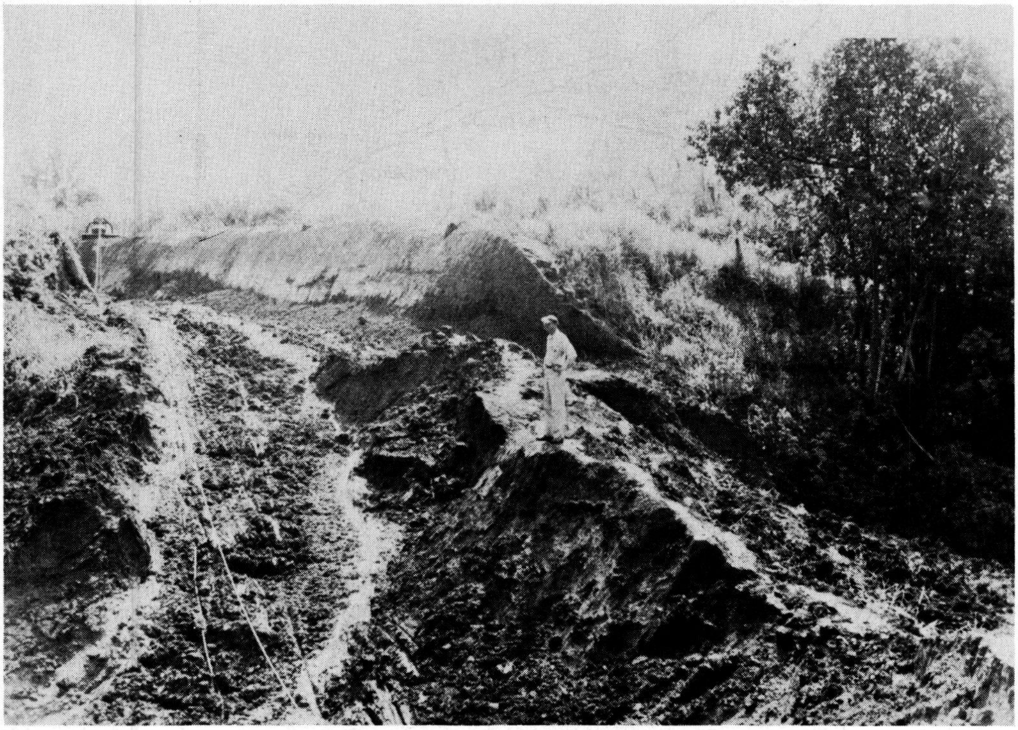

Figure 2. Embankment Slide Due to Intensive Working of Material Using a Sauerman Bucket. Note contrasting stability of cut slopes in background.

phenomenon has resulted in the formation of a surface layer measuring 6 to 18 in. in thickness, of slightly granular texture, and decidedly less plastic than the original ash. The most valuable characteristic of this topsoil is that, except in very rainy weather, it will readily support rubber-tired construction equipment.

As far as appearances go, conditions look almost ideal for a carryall job, but experience has shown that immediately after the thin, slightly granular topsoil layer is removed and the plastic under-

carryall operations, except in special circumstances noted hereafter, are impractical.

For any sizable depth of cut, the only satisfactory method is to take the material out in one lift. For the deepest cuts a dragline is the preferred piece of equipment (see Fig. 3). By staying on high ground and traveling at all times on the slightly granular topsoil it does not bog down. For moderate depths of cut a backhoe or shovel can be used. The latter necessarily has to travel over the freshly exposed surface, but experience indicates that if its movement is limited it will not bog down, unless conditions are extremely bad.

An important requisite is to follow close behind with a layer of select material. This must be spread, usually with a bulldozer, in one solid lift of 18-in. thickness. Any attempt to lay material in thinner layers is futile, since mere weight will cause construction trucks to punch through clear to the axles.

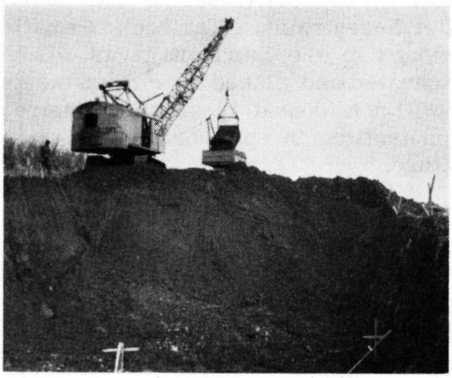

Figure 3.

The 18-in. layer of select material protects the freshly exposed soil against being worked and also serves as a base for construction operations. The best insurance against bogging down in excavation operations is, therefore, to follow very closely behind the shovel or dragline with the select material layer. This means close coordination on the part of the contractor of excavation and borrow operations for select material. Provided the two operations are properly scheduled, the method has proven well-nigh foolproof.

During construction operations spillage is more or less inevitable. Traffic churns up such spilled material, and the select material layer soon becomes covered with almost liquid mud. Fortunately, the select material being used is of such a nature that construction operations are practically unaffected. When the project is ready for fine grading the surface mud is bladed away, leaving only a thin surface film. In such thin layers the ash dries in a remarkably short time. Thus, due to rains, the surface may be very muddy in the morning but if the sun is shining it will be dusty by noon. In this dusty condition the plasticity index drops to around 6 or lower so that with a little sprinkling it serves as an excellent binder.

The conditions under which carryalls may be used are as follows: (a) the rubber tires must be removed and Athee type tracks fitted on; (b) the capacity of the carryall must not be too great (about 8 to 12 cu. yd.); (c) the depth of cut must be moderate (say about 15 ft. at the most); (d) locally the moisture content of the ash must not be too high (say below 150 percent); and (e) weather conditions must be favorable. If the conditions are as outlined above, then carryall operations can be carried on successfully (see Fig. 4). Occasional showers and streaks of material with moisture content in excess of 150 percent will not greatly affect operations. Of course, the motive power must be

Figure 4.

furnished by a large unit, such as a caterpillar D-8. Also, if the unit is of the type that has to come to a complete stop before gears can be shifted, then the engines will have to be "soupped up" to reduce the necessity of such shifting.

For very shallow cuts, bulldozers can be used to advantage. But if the unit has to move back and forth a great deal, there is some danger of the equipment bogging down.

CUT SLOPES

The fact that the material is stable in cut is indeed fortunate and as long as the ash retains its characteristic moisture content it resists the erosive action of water remarkably well. As the face of the cut is exposed to the drying influence of the sun and wind, the originally plastic ash slowly turns granular and drops onto the road shoulder, from whence it is removed by maintenance forces.

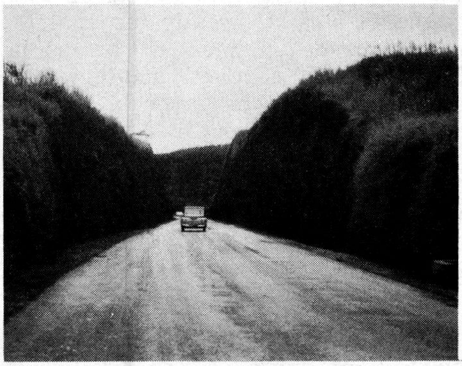

Figure 5.

The process described takes place very slowly, involves only small amounts of material at any one time, and so does not result in a major slide.

Along the sides of gulches the above described loose material from the weathering of the ash tends to accumulate and come down all at once during periods of heavy rains causing major slides. The somewhat flatter slopes of the gulch sides and vegetation tend to hold back the loose material. Hence, sidehill slopes are very vulnerable to slides, and whenever possible such locations should be avoided.

Of course the cut slopes can be benched as a precaution against slides. Figure 5, which shows a newly completed road cut, gives a good indication of the rugged nature of the terrain. How greatly construction costs will be increased by benching can be readily surmised. It was in the interests of economy, therefore, that 1/4-to-1 cut slopes were adopted as standard. Examination of old railroad cuts in the vicinity shows that such steep slopes have withstood the test of time remarkably well.

The ash cover in Figure 5 varies from 15 to 20 ft. or so. Below the ash cover is a conglomeration of cinders, lava rock of various sizes up to 5-ton boulders, rock ledges, and tufaceous material. All of the material was so highly weathered that very little blasting was necessary. The rock formation is extremely porous and there is considerable seepage from the face of the cut, especially during and shortly after heavy rains. The rock formation belongs to the Hamakua lavas. When excavated and placed in embankments, it will not adequately support construction traffic, in spite of its very rocky nature.

EMBANKMENT CONSTRUCTION

The most difficult problem in connection with this soil is the construction of stable embankments. Due to its high moisture content and its thixotropic nature, this volcanic ash cannot be compacted in the usual manner. Also, the very size and weight of modern construction equipment adds greatly to the difficulties. The use of conventional rolling equipment, e. g., sheepfoot tampers, flatwheel power rollers, pneumatic rollers, has proven absolutely futile.

First was the problem of getting the excavated material there. As long as the equipment was traveling on the original topsoil no great difficulty was encountered. Hence, placing the first layer was relatively easy. With successive lifts the difficulties increased. At times it was possible to discharge the material at the near end and depend on bulldozers to push the material forward and to level it off; but this, in effect, constituted constant working of the material, so that after awhile even the bulldozers showed signs of bogging down. Operations were then discontinued and shifted elsewhere, to be resumed when the material regained sufficient consistency.

The above end-dump method of embankment construction was heretofore a sort of standard procedure and, although feasible, was time consuming and unreliable in the case of embankments exceeding 15 to 20 ft. in height. Not only did equipment bog down frequently, but because of lack of compaction, trouble was experienced with sliding of the side slopes. So serious were embankment slides at times that it was proposed that all ash material be wasted and embankments built entirely of select rocky material. The proposal had to be rejected because of the cost.

The most expeditious and satisfactory method of embankment construction is that which has come to be known as the layer method. In this method the embankment is built up in alternate layers of 5 ft. or more of ash soil and 18 in. of special borrow material. First, the excavated material is transported to the far end of the embankment in trucks (or carryalls) and discharged there. Bulldozers then level off the material into a layer approximately 5 ft. high. When the 5-ft. layer of ash soil has been placed all the way to the near end, other dump trucks begin spreading the 18-in. layer of special borrow material beginning at the near end. This special borrow layer accomplishes two things. First, it acts as a temporary surface to provide reliable traction for construction equipment in laying down the next 5-ft. layer of ash. Second, by judicious routing of heavy construction traffic, very effective compaction is imparted clear to the edges; something that was not possible in previous end-dump methods. Also, the side slopes are carefully trimmed as the embankment is built up.

Using the layer method, embankments up to 60 ft. in height have been successfully completed and higher ones of 100 ft. and over are contemplated.

Since the adoption of the above layer method of construction, lost time due to rainy weather and to equipment bogging down has been negligible.

Some question may be raised as to the amount of compaction that can be obtained by spreading material in such relatively thick layers. Due to the high moisture content of the soil, the amount of compaction that can be imparted to it even in thin layers is limited. Beyond a certain relative compaction, a greater compactive effort merely causes a plastic displacement instead of added compaction. Numerous tests show that the compaction actually attained in 5-ft. layers is approximately equal to the compaction obtained by AASHO Method T-99 for the actual moisture content of the soil. Of course, if there should be a loss of moisture from the present natural values of close to 200 percent down to, say, below 40 percent, there will be a great shrinkage and consequent settlement, but all available data indicate that the moisture content below the first foot or so remains relatively constant so that the mass of the embankment as a whole maintains uniform moisture conditions. The best proof that compaction is reasonably adequate lies in the fact that settlement of embankments has been negligible or slight.

The top of all embankments is finished off with an 18-in. layer of select material as in cut sections.

SELECT MATERIAL AND SPECIAL BORROW

The select-material layer forms the sub-base on which the regular base and pavement are laid. Both select material and special borrow are obtained from the same pit and the two differ only in that the former is better graded and laid more carefully to grade. The special borrow is more of a construction expedient; therefore, strict requirements as to particle gradation and quality are unnecessary.

The Mauna Loa lava flows at the southern edge of the city of Hilo near the airport are just about ideal for use as select material and special borrow. For one thing, there is practically no overburden. The material is a loose fragmental form of lava resembling furnace clinker (4), known as aa (pronounced ah-ah). It is only slightly weathered, occurs in 8- to 10-ft. layers, requires no blasting, and is easily loaded with a shovel. Placed on the ash subgrade in an 18-in. layer, it not only protects the subgrade but provides the contractor with a temporary surface that assures positive traction for all his construction equipment through all kinds of rain and mud, making possible all-weather operations.

North of Hilo are the Mauna Kea lavas. These are practically everywhere covered by varying thicknesses of volcanic ash from later eruptions. In general, the older Mauna Kea lavas are not as suitable for select material or special borrow layer as the Mauna Loa lavas already mentioned. This is due to more advanced weathering. Many exposed vertical faces of apparently solid rock are seen along road cuts and sides of gulches, but these are deceiving. At the exposed face all earthy material is washed away by the frequent rains so that the rock looks clean. Back of the face the rock is mixed with earthy material, the result of weathering. The weathering is anaerobic which, together with the high organic content and continuous moisture, gives the whole a somewhat slimy consistency. Grading analyses show that the amount of fines passing the No. 200 sieve usually does not exceed 15 percent be weight. Under ordinary conditions such rocky materials make good bases and, indeed, were thus used in the old days of hand labor and light construction equipment. What makes such material unsuitable for base use under present conditions is that, due to its saturated and slightly slimy consistency, it will not initially support modern heavy construction equipment. If the material is allowed to dry out, which takes time, it makes a fairly satisfactory base, but such drying out procedures are cumbersome and unsatisfactory on contract work.

The lava flows of Mauna Kea have been classified in two great series: an older series of flows, known as the Hamakua series, and a later, less weathered series of flows, known as the Laupahoehoe series (5). The latter are the only Mauna Kea lavas that are suitable for use as select material or special borrow layer.

There is a correlation between the depth of ash cover and the age of the lava flow as expressed by the above two main series. Thus if the ash cover is less than about 5 feet or so, it is possible that the lavas belong to the later Laupahoehoe series. If the depth of ash cover is great, the chances are practically certain that the lavas, no matter how promising they appear to be, are of the older Hamakua series and therefore unsuitable for select material or special borrow use.

Satisfactory borrow pits are strictly limited to lavas of the Laupahoehoe series. Requirements are that the lava be fragmental (for easy working) and that it be only slightly weathered. However, experience shows that some drilling and blasting is necessary.

The location of suitable sites for select material and special borrow assumes greater and greater importance for projects beyond economical haul distance from the Mauna Loa lavas of Hilo.

OLDER CONSTRUCTION METHODS

In the old days of hand labor and wheelbarrows no trouble was encountered with bogging down of equipment. Sub-grades were carefully dressed to exact line and grade. It was then more or less standard practice to lay an 8-in. Telford base followed by a 2-in. rock layer to smooth off the Telford and a 2-in. asphalt-macadam surface. Construction was, in a great many cases, by stages, and the final asphalt macadam surface was not laid until many years after the first layer of rock was placed. Hence, variations from the simple 8-2-2 mentioned above are encountered but, to the best of the author's knowledge, none of the roads built prior to 1935 has a total thickness exceeding 12 in. or so.

Because both traffic and construction equipment were light in weight, it was possible to make extensive use of the above-mentioned Hamakua lavas, obtained both from road cuts and roadside quarries, as base material. Although our recent experience showed that such materials are not initially stable under heavy wheel loads, they proved successful under the then current light loads. As time passed the material gained stability, due to added compaction of both the base and subgrade, enabling the road to carry increasingly heavier loads. Such increase in load capacity by increased compaction due to traffic and time have been reported by others (6) (7). Pavement failures were remedied by superimposing additional layers, so in many sections the normal 2-in. asphalt surface measures a good 4 to 6 in. or so. Within the last two years construction traffic and regular commercial traffic has increased both in weight and density so that many sections of the older roads have deteriorated badly.

Coming down to more recent years, just before the start of our heavy construction program, it was the practice to fully complete grading operations before laying any base material, forcing all construction equipment to travel over the ash surface. Equipment was frequently bogged down and work could only be resumed after a week or so of good, dry weather which dried the surface sufficiently to turn it slightly granular as previously mentioned. Completion dates were invariably months behind contract time.

In embankment construction, end-dumping methods were standard. This was satisfactory for low embankments, but for heights exceeding 15 feet it was not only slow and difficult but often resulted in troublesome slides.

SUMMARY

The methods described in the foregoing have greatly facilitated construction operations. In particular, provision for a layer of rocky material to provide positive traction for construction equipment during grading operations has enabled the contractors to bid with more confidence.

If modern construction equipment were not so heavy, the select material layer could probably be reduced to about 12 in. For example, a D-8 caterpillar tractor is in many cases much too heavy and bogs down during unfavorable weather, while a D-6 appears to be the best all-around motive power, but it does not seem practical to specify that the contractor shall use only light equipment. For one thing, more units would be required and contractors would need to be re-equipped, since they are already stocked with the heavy equipment.

As it is, during the early stages of construction, there is a most pronounced heave of the subgrade at times, even with 18 in. of select material under the wheel loads of construction traffic. This heaving action gradually subsides as time goes on. Prior to laying the base, all weak spots are noted and the sub-base built up to a compacted thickness of 24 in. This is followed by a waterbound macadam base 4 in. thick. The whole is then finished off with a 2-in. surface course of asphalt concrete.

That our construction methods have proven successful is shown by the fact that lost time due to unfavorable weather and muddy field working conditions has been reduced to negligible proportions. As a direct result, projects are now being completed well ahead of contract time.

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LATERITE SOILS AND THEIR STABILIZATION

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Synopsis

Origin, occurrence, and correct identification of laterite rock and soil are briefly discussed. The peculiar properties of laterite soils are outlined and suggested methods of dealing with them are indicated.

Results are reported of experimental studies on four laterite soils and one intrazonal non-laterite soil, with special attention to their susceptibility to stabilization. The latter varied over a wide range Of the stabilizers used, Portland cement gave best results in some cases and aniline-furfural (2: 1) in others. The presence of organic matter seems to play a detrimental role in many cases.

The possible use of "oiled earth" properly fortified with anti-oxidants and bactericidal substances for low-cost roads in laterite areas is briefly discussed

Of the important groups of the tropical and subtropical soils of the world, the laterite soils occupy a unique place, in regard to both their extensive occurrence and peculiar properties. They are widely distributed in such areas as India, Indonesia, Indo-China, Malaya, Burma, Western Australia, Madagascar, Central Africa, the Guianas, Brazil, and Cuba (Fig. 1). From a world-wide political and economical point of view[1] study of these soils is of vital interest because: (1) they normally possess good tilth and excellent drainability, with plenty of solar energy and water available; with an adequate supply of fertilizers they are capable of excellent yields and may well be destined to contribute in a major degree to the food supply of the world; and (2) a great need exists for a suitable network of low-cost roads in these areas already in their present under-developed condition and even more so if their proper agricultural development is to proceed.

From a purely scientific point of view, the peculiar engineering properties of laterite soils as extreme products of soil genesis call for an extensive investigation and possible elucidation, not only for their own sake but also for a better understanding of the properties of less extreme soil types. The present work is confined to the engineering characteristics of these soils, especially those of importance in low-cost road construction.

The development of the science of soil stabilization has given a scientific footing to an understanding, though as yet more or less qualitative, of soil behavior in highway and airport structures (31). Most of the available information, however, pertains to soils of the temperate zones. That soils of the different climatic zones vary extensively in their properties is well-known. The necessity for a possible special approach to the study of the tropical and subtropical soils has been emphasized by many authors (13, 27, 32, 33).

PREVIOUS INVESTIGATIONS

Ever since the word laterite was introduced, first in geology and later

[1]See Point IV of the Truman Program, and the British plans for assisting in the economic development of Southeastern Asia and of Africa.

in pedology, there has been a sea of controversy over its correct nomenclature and identification. Laterite owes its origin to the Latin word later meaning "brick," and was first discussed in 1807 by Buchanan (3) purely on its field occurrence: "It is one of the most valuable materials for building. It is full of cavities and pores, and contains a large quantity of iron in the form of red and yellow ochres. In the mass, while excluded from the air, it is so soft that any iron instrument cuts it, and is dug up in square masses with a pick-ax, and immediately cut into the shape wanted with a trowel or a knife. It soon after becomes hard as brick, and resists the air and water much better than any brick I have known in India. . . The most proper name would be Laterite from Lateritis." [2]

In 1911, Fermor (9), in a most explanatory paper, defines laterite as being formed by a process which causes the superficial decomposition of the parent rock, removal in solution of combined silica, lime, magnesia, soda and potash, and accumulation of hydrated iron, aluminum, titanium, and rarely, manganese. The latter were termed by him "lateritic constituents." A residual rock with 90 percent or more of lateritic constituents is termed a true laterite. This true laterite is to be distinguished from the lesser groups, viz., lithomargic laterites and lateritic lithomarges with 50 to 90 percent and 25 to 50 percent of lateritic constituents respectively. The controversy over the identification of bauxite as a variety of laterite was answered by Clark (7) in 1916 to the effect that there is no dividing line between bauxites and laterites - one shades into the other. Later, the term bauxite came to be used only for commercial aluminum ores. Campbell (4) in 1917 called a material laterite only if it contained uncombined alumina in the form of the hydroxide.

In 1926 Harrassowitz (14) first characterized laterites as red soils enriched on the surface with oxides of iron and aluminum. According to him the European red beds are of fossil laterite origin. Marbut's (21) "normal" laterites are those formed under the influence of good drainage free from the action of high ground water. He calls ground-water laterites the soils falling within Fermor's first group. Mohr (23) believes that the laterite crust is the result of eluviation followed by erosion. He mentions five stages in the full development of a typical laterite, viz., fresh ash soil, tarapan (juvenile soil), brownish yellow lixivium, red lixivium, and laterite. Periodic variation of ground-water level (descent and ascent) appears to be a most important factor in laterite formation.

LATERITIC SOILS

One of the important reasons for the confusion regarding the correct identification of laterites is the large area of the tropical and subtropical zones involved and the different degrees of laterization encountered in the various parts. Martin and Doyne (21) used the silica-alumina ratio as a classification criterion:

Soil Type	SiO_2/Al_2O_3
Laterite soil	1.33 or less
Lateritic soil	1.33 - 2.00
Non-lateritic soil	2.00 and over

The SiO_2/Al_2O_3 rather than the SiO_2/R_2O_3 ratio was employed for analytical reasons since iron may sometimes be in the form of ferrous oxide. This leads to the paradox that formalistically the presence of iron is nonessential for a soil to be termed laterite; this, of course, is in direct contradiction to the original definition of Buchanan and overlooks the important role which

[2]This term used by Buchanan, meaning literally "brick disease", throws a peculiar light on the problem whether "laterite" refers primarily to the use of the material in construction or to its brick-like appearance. In several Indian dialects laterite is called "brick-stone" (itica cullu), in the Tamil language it is called shuri cull or "itch-stone" because its surface appearance resembles certain cutaneous disorders. The Latin-derived English term happens to cover both appearance and function.

iron oxides play in the rock named laterite by him. Moreover, the presence of iron in laterite soils is one of the most important factors that influence their engineering properties. It certainly is more appropriate to employ as classification criterion the silica sesquioxide ratio:

Soil Type	SiO_2/R_2O_3
Laterite soil	1.33 or less
Lateritic soil	1.33 - 2.00
Non-lateritic soil	2.00 and over

(4) Non-accumulation of organic matter
(5) Distinctive red color.

B. Characterization of the residual weathering product "laterite".

(1) A tropical climate subject to alternations of dry or wet seasons or monsoons.
(2) A level, or very gently sloping, elevated land surface which is not subject to appreciable mechanical erosion (abrasion by rain and wind)
(3) The chemical and mineralogical com-

Laterite and Lateritic Soils
(AFTER BOL'SHOI SOVETSKI ATLAS MIRA: JOFFE)

Figure 1. Distribution of Laterite and Lateritic Soils in the World.

It should be useful at this point to juxtapose the presently accepted characterization of lateritic and laterite soils and the conditions stipulated in the revised theory of C. S. Fox for the tropical residual weathering product "laterite" (12, 19).

A. Characterization of lateritic and laterite soils.

(1) Disintegration and decomposition of the parent material in the direction of the end products of weathering.
(2) Release and removal of silica.
(3) Separation of sesquioxides and fixation in the profile.

position of the exposed rocks to be suitable for a supply of the lateritic constituents, alumina and ferric oxide.
(4) The texture of the rock to be (or rapidly become during weathering) sufficiently porous for the entry of percolating water, so that the conditions for chemical action will be at a maximum.
(5) The infiltrating water to remain in the interstices of the rock for long periods annually, i. e., during the wet monsoon, but eventually to drain away in the dry period thus giving maximum play to chemical erosion
(6) The infiltrating water to contain

either an acid or alkaline substance with which to react on the rock components as well as to constitute an electrolyte and allow electro-kinetic phenomena to operate.

(7) These annual processes to be in operation for at least a geological epoch of roughly a million years

This comparison shows that it is relatively easy for a rock to become a lateritic or laterite soil, but that the chances to become a "laterite" are severely restricted.

The general acceptance of absence of organic matter due to intense biologic and inorganic oxidation assumed for the red soils of tropical areas has been questioned by some authors who testify to high organic content of some of these soils. Joffe (19) states that a red soil with organic content either is an immature specimen of laterite or falls in a category between laterites and podsols. This explanation, however, hardly touches the nucleus of the problem or organic matter in laterite soils.

PROPERTIES OF LATERITE SOILS

As has been excellently dealt with elsewhere (13), laterite soils in their natural state are granular in structure and are possessed of low plasticity and excellent drainability. In this state they can carry heavy loadings. When remolded in the presence of water they often become clayey and plastic to the depth disturbed (34). An extraordinary influence of the change in the natural moisture content on the plasticity and density characteristics of a certain Hawaiian lateritic clay of volcanic origin has been reported; depending upon the natural moisture content, the plasticity index of this clay varied from 245 to zero (13). Thus, it is easy to see that the conventional subgrade soil test results may not at all reveal the true characteristics of members of the laterite soil group.

POSSIBLE METHODS OF APPROACH

Plasticity, density, and structural characteristics of soils are only the external manifestations of the granulometry and the inherent affinity of the individual particles or aggregates to one another under different conditions of moisture and mechanical stress. Consequently, the nature, mechanical strength, and the permanency of cementation in these soils must be understood.

The granulating effect of organic material is well known. However, the resulting aggregations are usually soft and friable, rather than mechanically strong. It is felt that the organic matter in laterite soils may be more important for cementation because of its reducing effect giving the iron greater mobility than for its specific cementing or water-repelling power. Baver (1) makes the following statement concerning organic matter and cementation in laterite soils:

"The only group of soils in which a correlation has not been observed between organic matter and aggregation is the lateritic soils, where dehydrated oxides of alumina and iron are responsible for stable aggregate formation."

In connection with the problem of organic matter it is of interest that the previously discussed Hawaiian clay which exhibited such unique consistency properties contained about 20 percent of organic matter (15).

The existing situation indicates a great need for a fundamental study of the structure of laterite soils with special emphasis on the actual type and amount of organic matter present including the role of micro-organisms (1, 13, 28, 34). An integral part of such a study must be concerned with swell and shrinkage properties, with cracking patterns, as related to the degree of laterization and to the type and amount of organic matter, as well as with the effects of remolding (13, 27, 28) and of prevailing moisture conditions.

PROBLEMS IN SUBGRADE COMPACTION AND GRANULAR SOIL STABILIZATION

The engineering soil problems encountered in laterite areas as related to the nature of the soil material as well as to the environmental conditions have been thoroughly studied and ably presented by Wooltorton (33). He was the first to point out the importance of swelling and shrinking for tropical soils which by many "temperate-zone soil scientists" were believed to possess great volume constancy. His work combined with concepts normally employed in Portland cement concrete design led to Winterkorn's work on volume relationships in soil stabilization (27). Application of the volume principle leads, for granular stabilization of laterite soils, to clay contents greater than those given by the ASTM Standard specifications. Obviously, the latter represent a specific solution of the general problem; but being specific they are applicable only for the average illite clay soils and for the average climatic conditions of the United States.

Specifically, Wooltorton (34) suggested: For no over-all swelling of a coarse granular system: Plastic Index x Fines Content > Volume of voids available between granular aggregates to accomodate swelling, or a modification thereof when some swelling is permissible. In place of the plastic index, shrinkage volumes may be employed as indicators of suitability of the binder portion, viz., (a) liquid limit-shrinkage limit, where exposure conditions permit slaking, or (b) field moisture equivalent-shrinkage limit where environmental conditions do not favor slaking.

It is necessary to caution here that the fines content as determined by the ordinary standard method of mechanical analysis for soils may be greatly misleading; well developed laterite soils with their granular structure are not easily dispersed by the common dispersing agents. Secondly, the consistency limits may show large variations in particular cases, depending upon the moisture content of the sample and the extent to which it was remolded during the tests. Special tests or modifications of the standard tests may have to be developed to serve as indices for the plasticity and swelling characteristics that form the basis for proper design.

With respect to compaction, both in the laboratory and in the field, the task is to search for and adopt suitable mixing and compaction methods that would not destroy the granular structure and yet yield sufficient density.

In the field extra care is needed if heavy machinery is employed. Experience has shown that primitive manual compaction of certain laterite soils has yeilded better airfields than compaction of the same soils with standard heavy-compaction machinery and procedure.

So far consideration has been given mainly to the problems encountered in the engineering use of laterite soils. This might give the impression that all laterites and laterite soils give trouble. This is far from the truth. The physical and chemical properties of the group of materials encountered range from excellent to poor for engineering purposes. The problem is one of recognizing the good, eliminating the poor, and improving the intermediate ones. This is well stated in a recent paper by Christophe (6) treating specifically with the utilization of African laterites as road construction materials.

Many of Christophe's statements are so pertinent from a road building as well as from a scientific point of view that they are given in the following as abstracted from the original French.

The term laterite is applied to any rock colored red to dark maroon which is either in the hardening or in the decomposition stage as a result of environmental variations. Laterites range from friable soils to hard rock similarly as lime stone may range from marl to marble The use of laterites must be preceeded by a study of their properties.

The silty parts of the foundation soils must be eliminated. In granular soil stabilization higher amounts of lateritic clays than of other clays are allowed The pH influences the clay content as determined by sedimentation analysis. The interpretation of sedimentation curves requires great care. The strength of compacted laterites may go up to 2,000 kg per sq. cm. and higher

At the Ivory Coast a very excellent laterite rock is found of strength properties comparable to those of medium porphyry In equatorial Africa and especially at L'Oubangui-Chari pudding stones predominate In Togo amorphous laterites abound, in the Haute Volta strong lateritic gravels are found. In the Sudan, under the present dry climate, hills of laterite rock are eroded to a considerable extent. In laterite soils one can always find more or less rock-like aggregations.

The affinity of hard laterite to asphaltic bitumen is good, similar to the affinity of bitumen to Basalt and lava, but the presence of silt hinders the proper contact between the hard laterite and bitumen Laterite used in bituminous construction should not contain too much loam or clay Design must be based on the results of preceding tests Pougnaud, chief engineer of Abidjan, has treated laterite by impregnation and subsequent surface treatment The oldest job is 13 years old and still in good condition. Treatment with cut-backs yeilds good results if it is thoroughly mixed with the soil Improvement of granulometry is recommended to obtain greatest strength. This can be done by breaking amorphous laterite down to sand size The expense of this is compensated by the resulting smaller bitumen requirements, for the base and surface treatment.

Laterite clays make excellent binders for stabilized soil roads. In the case of soils composed of pisoliths (lateritic gravel) and fines, compaction alone is sufficient, and results in bearing values of 60 or more. Hydrocarbon binders have good adhesion to laterite rocks Excellent bituminous concrete has been made with such rock. Amorphous laterites should be comminuted and well-mixed in order to obtain a homogeneous material. Compact laterite rocks have sufficient strength to be used as a construction material for highways as broken stone, gravel, etc. However, it is indispensable to judiciously choose the borough pit.

The statements by Christophe obviously refer to actual "laterite" or material coming very close to it. The experiments described in the following deal with the more troublesome laterite - lateritic and non-lateritic soils occuring in tropical regions.

EXPERIMENTAL INVESTIGATION

Five soils, two laterite, two lateritic and one non-lateritic, were employed in the experimental investigation (Table 1). The latter was concerned with: (1) physical and physico-chemical tests and (2) stabilization tests with external additives.

Consistency limit and thermal analysis tests were made on all materials. The Matanzas soil was studied in greater detail, with respect to mechanical analysis, Proctor compaction, permeability, base exchange capacity, etc. Only a limited number of tests were performed on the other soils.

The stabilization tests consisted of preparation of 2-by 2-in. cylindrical soil-stabilizer specimens of Proctor density, moist or dry curing for 7 days, exposure to four cycles of freezing and thawing[3], four cycles of wetting and drying[4], and 7 days immersion in water, respectively, with subsequent determination of the compressive strength in a Carver hydraulic press. The susceptibility to stabilization is rated on the basis of compressive strength values, combined with a qualitative judgment of the nature of failure etc.

STABILIZERS USED

The stabilizers used and the nomenclature employed are given in the relevant tables of results of the stabilization tests. Only Portland cement, MC-3 cut-back, and aniline-furfural (2:1) were used with the Havana and Catalina soils while these and a variety of other stabilizers were employed with the Matanzas soil. The tests with the Nipe soil were confined to the respective soil-cement system. The data reported for the Guinea soil were obtained during an investigation undertaken for Portuguese Guinea, which had as object the utilization of local low-cost vegetable materials in low-cost road and airdrome construction. The common stabilizers, Portland cement and asphaltic cut-backs, had been employed originally only for purposes of comparison; however, the failure of these materials to stabilize the soil in question prompted a search for the underlaying reasons.

PREPARATION OF THE SPECIMENS

Soil (passing through No. 10 sieve) was mixed with predetermined quantities of water and stabilizer in a Hobart mixer and compacted in a Dietert machine. Solid stabilizers such as

[3] Freezing and thawing: 4 cycles, each cycle consisting of 8 hours of freezing and 15 hours of thawing.

[4] Wetting and drying: 4 cycles, each cycle consisting of 3 hours of wetting and 20 hours of drying.

TABLF 1

DESCRIPTION OF THE SOILS USED FOR EXPERIMENTAL INVESTIGATIONS

Soil Name Given	Description of the Soil	Profile Characteristics	Organic Matter Present	Parent Material	Clay Mineral from Thermal Analysis
Matanzas	Dark red, well granulated laterite soil from Cuba	A_2 horizon 0 15 to 1 10 ft below ground level	Yes (rather high)	Lime Rock (coco')	Hydrated Halloysite with some gibbsite
Havana[a]	Ashy gray, clayey tropical soil from Cuba (Probably similar to the ''C'' horizon soil of the Matanzas series)	B_3 horizon 0 50 to 2 75 ft below ground level	Yes (rather low)	Lime Rock (coco')	Mixture of illite and montmorillonite
Catalina	Yellowish-red lateritic soil from Puerto Rico, not well granulated	B and C horizons 1-5 ft below ground level	Yes (rather low)	Andesitic tuff & tuffaceous shale	Kaolinite
Guinea	Light red, sandy lateritic soil from Guinea Africa	--	Yes (very high)	--	Kaolinite
Nipe	Dark red, well granulated laterite soil from Puerto Rico	B_2 horizon--2 to 5 ft below ground level	Yes (rather low)	Serpentine Rock	Kaolinite with gibbsite

[a] Contains more than 50 percent $CaCO_3$

Portland cement and lime were first mixed dry with the soil; subsequently, water in an amount calculated to satisfy the moisture requirement of the soil and the hydration needs of the stabilizer was added. In the case of semi-solid and liquid stabilizers, the concept of optimum liquid content was used. The reported percentages of the stabilizers are based on the dry weight of the soil.

The preparation of soil-cement specimens at the standard optimum moisture content in the earlier attempts posed a problem due to the fact that the soil-water-cement system presented a rather sticky consistency, hardly conducive to obtaining well defined samples. A close observation of the mixing process showed the existence of a gradual change of the mix from the granular texture to a rather sticky consistency. This was much pronounced with an increase in the speed and time of mixing.

Best specimens were obtained by employing the lowest speed of the mixer and molding the specimens when the mix presented a very loose granular appearance with uniform moisture distribution.

Observations of the behavior of the laterite and lateritic soils in the preparation of specimens with a large number of stabilizers brought out the importance of avoiding over-mixing, not only for soil-cement, but also for the other systems. Excessive mixing destroys the desirable granular structure of the soil. However, with bituminous materials the dispersion was poor and did not improve significantly, even with continuous and intimate mixing.

The non-lateritic Havana soil alone showed a uniform mix with the bituminous stabilizer.

Aniline and furfural were added separately in the respective order to the soil-water system and the mixing continued after the addition of each.

TABLE 2

PHYSICAL CHARACTERISTICS OF THE MATANZAS SOIL

A Granulation Characteristics

1 Dry Sieve Analysis Characterizing Natural Granulations

Sieve No.	Percentage Passing (by weight of total)
4	77 1
10	59 1
40	23 95
200	2 50

2 Aggregate Analysis in an Elutriator

Size of Aggregates	Percentage by Weight
> 0.1 mm	84 45
0 1 - 0 05 mm	7 85
0 05 - 0 02 mm	3 73
< 0.02 mm	3 97

3 Mechanical Analysis on Soil Passing No 200 Sieve, Sedimentation Method

Size and Description of Particles		Without Treatment With H_2O_2	After Treatment With H_2O_2	After Removal [a] of Fe and Al Oxides
Sand	>50 μ	6 9	31 4	37 4
Coarse silt	5μ - 50μ	28 3	61 97	38 64
Fine silt	5μ - 2μ	12 8	3 17	15 30
Clay	< 2 μ	51 9	3 46	9 59

B Consistency Limits

	Fresh Soil Moisture Content 20.5 Percent		Soil air-dried for a long time (Moisture Content 4 7 Percent)	
Description of test	Remolded	With Minimum Handling	Remolded	With Minimum Handling
Liquid limit	53 0	46 2	52 6	45.9
Plastic limit	31 2	31 2	32 1	32 1
Plasticity Index	21 8	15 0	20 5	13 8
Shrinkage Limit	20 8	31 5	20 32	26 8

C Other Tests

1 Specific gravity	In Water 2 90	In tetrahydronaphthalene 2 94
2 Compaction tests	Proctor	Modified AASHO
Optimum moisture content	30 %	25 5 %
Maximum dry density in lb per cu ft	88	95.25

3. Permeability coefficient from consolidation test at 4 8 tons per sq ft 3×10^{-3} cm per sec

[a]Jeffries "Rapid Method for the Removal of Free Iron Oxides in Soil Prior to Petrographic Analysis " Proc Soil Science Society of America 211-212 (1946)

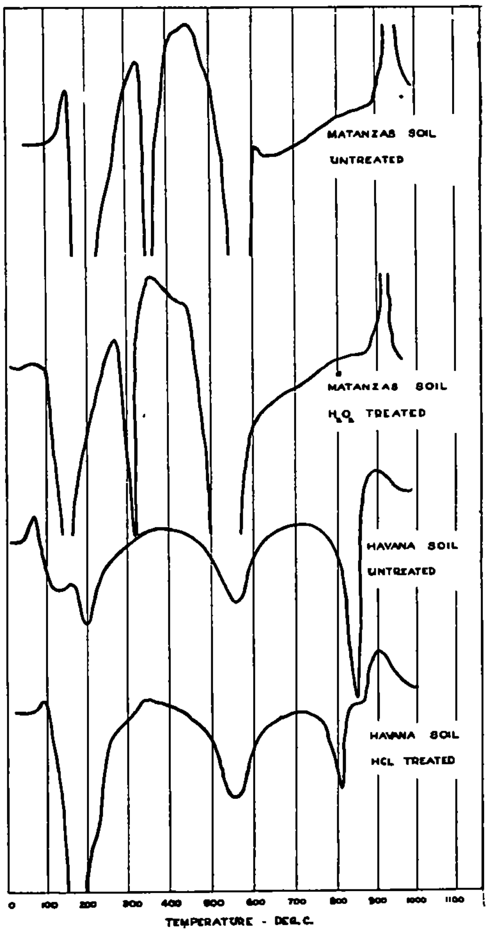

Figure 2. Differential Thermal Curves of -200 Fractions of the Soils.

CURING

All the Portland cement and lime specimens were moist cured. One-half of the number of specimens containing organic cementing or waterproofing agents were moist cured, the other half were dry cured: the same procedure was followed with the sodium-silicate specimens.

EXPOSURES

Separate sets of the lime and Portland-cement specimens were subjected to the three kinds of exposure tests mentioned previously. The only exception was the Guinea soil which failed to set with moist curing for 7 and 14 days, respectively.

All the other specimens were immersed in water for 7 days before testing for compressive strength.

COMPRESSIVE STRENGTH TESTS

On completion of the exposure tests the compressive strength tests were performed at room temperature and the nature of failure noted.

PHYSICAL AND PHYSICO-CHEMICAL PROPERTIES

The results of the tests are given in Tables 2, 3, and 4.

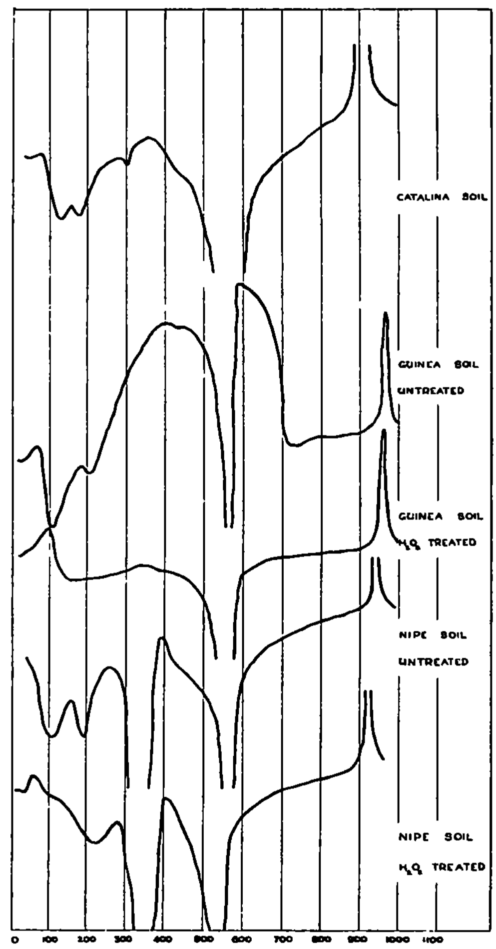

Figure 3. Differential Thermal Curves of -200 Fractions of the Soils.

MATANZAS SOIL

The results of the sieve analysis and aggregate analysis signify a very high degree of stable aggregation of the primary particles of the soil in its natural condition.

No proper dispersion could be obtained employing agents such as sodium silicate, sodium hydroxide, sodium carbonate, ammonium hydroxide, or a mixture of sodium hydroxide and sodium oxalate.

CONSISTENCY LIMITS

A study of the effect of change in the initial moisture content of the soil and the effect of remolding on its consistency properties was made. The change in the initial moisture content did not affect the values to any significant extent. On the other hand, the role played by the structure is indicated clearly by the increase of the plasticity index by 40 to 50 percent in the case of the remolded soil, as compared with the value obtained when structural disturbance due to the test itself was held at a minimum.

The other tests conducted on this soil show a high specific gravity of 2.94, high permeability of 3×10^{-3} cm per sec., a low affinity for water, and a low base exchange capacity of 6.6 me per 100 grams. The thermal curve

TABLE 3

PHYSICO-CHEMICAL PROPERTIES OF THE MATANZAS SOIL

		3.3% H_2SO_4	30% H_2SO_4	50% H_2SO_4
1	Hygroscopicity at 25 C	22 4	5 7	2 8
2	pH value of soil-water slurry (1 1)	6 30		
3	Base exchange capacity by conductometric titration	6 6 m e 100 gms		
4	Carbon content by combustion analysis	2 0 Percent		
5	Clay mineral from thermal curve	Hydrated halloysite with some gibbsite		

TABLE 4

A Physical and Physico-Chemical Characteristics of Havana, Catalina, Guinea and Nipe Soils

No	Name of Soil	L L	P L	P I	S L.	Clay Mineral From Thermal Curve
1	Havana	62 2	36 7	25 5	26.5	Combination of illite and montmorillonite (non-lateritic)
2	Catalina	72 0	53 7	18 3	25 9	Kaolinite (lateritic)
3	Guinea	--	--	--	--	Kaolinite (lateritic)
4	Nipe	48 3	38 0	10 3	25.7	Kaolinite with some gibbsite (laterite)

B. Results of Additional Tests on the Guinea Soil

1	Wet Sieve Analysis	Passing Sieve No	Percent by Weight
		4	100
		10	95
		40	38
		200	12
2	Maximum dry density of Proctor Compaction	113 5 lb. per cu ft	
3	Optimum moisture	10 3 Percent	

shows presence of gibbsite in an essentially hydrated halloysite, the presence of gibbsite probably indicating a fairly high degree of laterization (Fig. 2, Table 3).

HAVANA, CATALINA, NIPE AND GUINEA SOILS

The limíted number of tests performed on these soils shows reduced values of plasticity indices with higher degrees of laterization as seen from the thermal curves. The Guinea soil had only 12 percent passing the No. 200 sieve, and an optimum moisture content of 10. 3 percent.

The thermal analysis shows the Havana soil to contain a combination of illite and montmorillonite clay minerals, which are typical for non-laterite soils. The Catalina and the Guinea soils show typical kaolinite clay minerals indicative of their lateritic character. The Nipe soil shows kaolinite with gibbsite, the latter an indicator of a higher degree of laterization (Figs. 2 and 3).

SUSCEPTIBILITY TO STABILIZATION WITH ADDITIVES

A number of investigators have reported good stability characteristics of systems of red and yellow soils stabilized with Portland cement in amounts as low as 6 to 10 percent (5, 30). Studies have revealed that soils with high iron and aluminum contents are easily stabilized by means of bituminous materials and by silicate of soda, the latter to be supplemented with a waterproofing agent (20, 29).

The results of the present series of stabilization tests are given in Tables 5, 6, 7, 8, and 9. In the following discussion, only the most significant features are pointed out.

MATANZAS SOIL (TABLE 5)

Fairly good compressive strength values were obtained at comparatively high percentages of cement. Steady and optimum strength conditions seemed to be reached around a cement content of 14 percent. Further increase to 16 and 18 percent did not improve the strength values appreciably. It is even possible that a lesser quantity of cement (12 to 14 percent) may show satisfactory stability characteristics.

Stabilization with lime was a total failure; in fact, decreasing values of compressive strength accompany increase in lime content. It is possible that for lime stabilization a longer period must be allowed for the carbonation of the calcium hydroxide. Furthermore, if a soil contains acid organic matter, lime may stimulate the growth of bacteria with detrimental action on mechanical resistance of the system.

The dry-cured specimens failed completely while the moist cured ones showed very poor compressive strengths. It is, of course, realized that field performance of a soil-bitumen system cannot be judged correctly from compressive strength values alone; however, such data are important for comparison of relative effectiveness. The complete failure in one case and the very poor strength values obtained in the other, show the ineffectiveness of the bituminous stabilizer used or the method adopted.

Of the dry cured specimens, those containing 10 percent of tar alone withstood the immersion exposure, but showed rather poor strength. The moist cured specimens showed fairly satisfactory values. Though in general, stabilization with tar alone may not be adequate, treatment with the latter resulted in distinctly better stability characteristics than treatment with corresponding quantities of MC-3 cutback.

Both the dry cured and moist cured specimens showed satisfactory values, though a feeble tendency for slaking was noticed during immersion of the dry-cured specimens. This, however, did not significantly affect the strength characteristics. The poor results, amounting practically to total failures with the use of the other stabilizers listed in Table 5 do not warrant separate or detailed discussion. Noteworthy tendencies, if any, are brought

TABLE 5

RESULTS OF THE STABILIZATION TESTS WITH MATANZAS SOIL

Type of exposures F-T 4 cycles of freezing and thawing
W-D: 4 cycles of wetting and drying
IMM 7 days immersion in water

Description of Soil-Stabilizer Specimens		Percent of Stabilizer	Compressive Strength in P S I	Remarks
I Inorganic Stabilizers				
(a) Portland cement - moist cured	F-T	8	258	Shear failures
		14	481	
		16	506	
		18	550	
	W-D	8	132	Shear failures
		14	296	
		16	299	
		18	314	
	IMM	8	150	Shear failures
		14	313	
		16	351	
		18	354	
(b) Lime - moist cured	F-T	8	30.0	Crumbly shear failures
		14	15.7	
		16	12 0	
		18	8 0	
	W-D	8	--	Slaked and failed on wetting for the second cycle
		14	--	
		16	--	
		18	--	
	IMM	8	40 0	Crumbly shear failures
		14	52 0	
		16	31.0	
		18	41 0	

Note All compressive strength values listed in the following parts of Table 5 were obtained after 7 days immersion of the specimens in water

Description of Soil-Stabilizer Specimens	Percent of Stabilizer	Compressive Strength in P S I	Remarks
(c) Silicate of soda (N-Brand 40 % Concentrated Solution) - dry cured·	4	--	Stabilizer added directly to soil
	4	--	Stabilizer added in an aqueous medium
Silicate of soda - moist cured	4	12	Stabilizer added directly to soil
	4	--	Stabilizer added in an aqueous medium

TABLE 5 (Cont'd)

Description of Soil-Stabilizer Specimens	Percent of Stabilizer	Compressive Strength in P S I.	Remarks
II Organic Stabilizers			
(a) MC-3 cut-back - dry cured	6	--	Swelled, cracked and failed
	8	--	
	10	--	
MC-3 cut-back - moist cured	6	10	After exposure all specimens were wet but did not show failure
	8	12 5	
	10	13	
(b) Tar RT-6 - dry cured	6	--	Both 6 and 8 percent samples swelled, cracked and failed
	8	--	
	10	8	Cracked slightly and showed some compressive strength
Tar RT-6 - moist cured	6	30	Plastic failures
	8	33	
	10	33	
(c) Aniline-furfural (2 1) - dry cured	2	46.0	Very feeble slaking was noticed, failed in crumbly shear
	3	54 0	
	4	43.0	
Aniline-furfural (2 1) - moist cured	2	46 0	Crumbly shear
	3	52.0	
	4	70 0	
(d) Resins (abietic acid) - dry cured	0.5	17.0	Considerable slaking was noticed in all cases. The 1 percent specimens were of poor mold
	0 75	18 0	
	1 00	7 0	
Resins (abietic acid) - moist cured	0 5	8	Crumbled
	0 75	8	
	1.0	14	
(e) Abietic salt Resin 321 - dry cured	0 5	0	All the specimens slaked considerably
	0 75	3	
	1 0	5	
Abietic salt Resin 321 - moist cured	0 5	11	Crumbled
	0 75	11	
	1.0	10	

TABLE 5 (Cont'd)

Description of Soil-Stabilizer Specimens	Percent of Stabilizer	Compressive Strength in P.S.I	Remarks
III Bitumen and Admixtures			
(a) cut-back MC-3 + 10% gasoline -			
dry cured	8	--	Slaked and failed
moist cured	8	17	Plastic failure
(b) 8% cut-back MC-3 + 10% kerosene + 0 8% aniline-furfural (2 1) -			
dry cured	-	0	
moist cured	-	35	Crumbly shear failure
(c) 8% cut-back MC-3 + 10% gasoline + 0 24% aniline-furfural 2 1 + 0.08% pentachlorophenol -			
dry cured	-	12	Specimens slaked considerably in both cases
moist cured	-	44	
(d) 8% tar RT-6 + 0.8% aniline-furfural 2 1 -			
dry cured	-	0	Specimens slaked completely and failed
moist cured	-	30	Plastic failure

forward.

Silicate of soda was a complete failure in immersion for both dry and moist cured specimens. Both abietic acid and abietic salt (Resin 321) showed very poor strength values. The addition of gasoline as a dispersing aid for the MC-3 treatment gave no improvement. However, definite improvement resulted from the addition of a small percentage of aniline-furfural to the MC-3. Further addition of pentachlorophenol in the combination gives even better results. These observations are in line with previous findings on the influence of these activators. The addition of aniline-furfural to the RT-6, however, did not show any marked change in the strength values. In view of the excellent compatibility of aniline-furfural with coke-oven pitch, the observed behavior with the RT-6 may be due to the character of the cutting oil.

The results obtained with soil of the Matanzas series show a fairly good susceptibility to stabilization with Portland cement, and a rather satisfactory one with aniline-furfural; the general results are, however, quite below the level that would normally be expected of this well developed laterite soil.

HAVANA SOIL (TABLE 6)

The specimens failed almost completely in the wetting and drying tests with Portland cement. In the other exposures, the specimens showed relatively poor strength values considering the high percentages of cement employed.

The stabilization with the MC-3 was a total failure for both moist cured and dry cured specimens. The fact that moist cured samples with 6 percent of cut-back were relatively more stable than those with 8 and 10 percent is noteworthy. The soil contained a considerable amount of lime.

Only the moist cured samples withstood the exposure tests, with aniline-furfural showing better strength values than the cut-back MC-3. Optimum performance possible with this resin was not achieved, however, since the calcium carbonate in the soil neutralized the acid catalyst and slowed down the setting and hardening time of the resin. This is illustrated by the fact that the lesser percentage of aniline-furfural, viz., 3 percent gave better results than 4 percent. In the meantime, methodology has been developed to successfully stabilize calcium carbonate-containing soils with aniline-furfural resin.

The Havana soil with its illitic and montmorillonitic clay mineral and high calcium content presents extreme difficulty in stabilization with the conventional stabilizers.

CATALINA SOIL (TABLE 7)

The specimens failed completely in wetting and drying in the Portland cement test and gave extremely poor values in freezing and thawing. They did better, though still poorly, in the immersion test. It is significant that in the supposedly semi-rigid or rigid system of soil-cement, the specimens with as much as 16 percent of cement exhibited plastic failures. This brings home the fact, observed separately on pure soil-water specimens, that partially developed lateritic soils may have a greater water affinity and consequently more pronounced swelling

TABLE 6

RESULTS OF STABILIZATION TESTS WITH HAVANA SOIL

Description of Soil-Stabilizer Specimens	Test	Percent of Stabilizer	Compressive Strength in P S I	Remarks
I Portland cement - moist cured	F-T	8	114	Shear failures
		12	131	
		16	164	
	W-D	8	0	Failed on second wetting
		12	0	Failed on second wetting
		16	70	Considerable slaking was noticed
	IMM	8	128	Shear failures
		12	175	
		16	182	
II Cut-back MC-3 - dry cured	IMM	6	0	Swelled, cracked and failed within one day of immersion
		8	0	
		10	0	
Cut-back MC-3 - moist cured	IMM	6	5	The 6 percent samples were definitely better than the 8 and 10 percent ones by both appearance and strength
		8	0	
		10	0	
III Aniline-furfural (2 1) - dry cured	IMM	3	0	Cracked and failed after 1 day
		4	0	Cracked and failed after three days
		5	0	
Aniline-furfural (2 1) - moist cured	IMM	3	30	Plastic failure
		4	19	Crumbly shear failure
		5	20	

TABLE 7

RESULTS OF STABILIZATION TESTS WITH CATALINA SOIL

Description of Soil-Stabilizer Specimens		Percent of Stabilizer	Compressive Strength in P S I.	Remarks
I Portland cement - moist cured	F-T	8	0	Plastic failures
		12	3	
		16	25	
	W-D	8	0	Slaked and failed on second wetting
		12	0	Slaked and failed on second wetting but less fast
		16	0	Slaked and failed on third wetting
	IMM	8	31	Plastic failures
		12	30	
		16	80	
II Cut-back MC-3 - dry cured	IMM	6	0	Swelled, cracked and failed
		8	0	Swelled, cracked and failed but retained the shape, showed no strength
		10	0	
Cut-back MC-3 - moist cured	IMM	6	33	Plastic failures, cracks commenced with a pressure of 10 to 20 psi
		8	33	
		10	40	
III Aniline-furfural (2 1) - dry cured	IMM	3	97	Crumbly shear failures
		4	68	
		5	143	
Aniline-furfural (2 1) - moist cured	IMM	3	78	Plastic failure
		4	70	Plastic to crumbly shear
		5	90	

and shrinkage characteristics than normally expected in view of the kaolinite type of clay contained in such soils.

The dry-cured specimens failed completely with MC-3 cut-back. The moist-cured specimens showed fairly good compressive strength for such plastic systems.

The results obtained with aniline-furfural are in sharp contrast to those obtained with Portland cement and MC-3. Excellent compressive strength was exhibited by both the dry- and moist-cured specimens. The dry-cured specimens, for the first time, showed better strength characteristics than the moist cured ones for any soil with any organic stabilizer. The nature of failure of the moist-cured specimens was crumbly shear. The strength and water-resistance obtained with 3 percent of stabilizer were sufficiently good to suggest lowering of the stabilizer quantity to 2 percent. The good values obtained here have an added significance because of the poor showing with this soil of the other stabilizers, even at high percentages.

GUINEA SOIL (TABLE 8)

The size composition as well as the lateritic character of the soil under investigation would indicate that it could be easily stabilized by means of any of the conventional materials, Portland cement, asphalt and tar.

The low cohesion of this soil makes it imperative that the organic stabilizers possess cementing in addition to waterproofing properties. The proto-

TABLE 8

RESULTS OF STABILIZATION TESTS ON GUINEA SOIL (23)

Precent of Portland Cement	Compressive Strength of the Soil-Cement Specimens			
	Without additive	10% $CaCl_2$[a]	2% Hydrous $AlCl_3$[a]	2% Hydrous $Al_2(SO_4)_3$[a]
A After 7 days curing				
4	22	--	--	--
6	40	6 4	0	19
8	25	9 1	0	12 7
10	29	12 7	0	19
B After 14 days curing				
6	--	--	9 6	27
8	--	--	8	24
10	--	--	12 8	30

[a]Percentage based on weight of Portland cement

TABLE 9

RESULTS OF STABILIZATION TESTS ON NIPE SOIL

Description of Soil-Stabilizer Specimen	Moisture Content During Molding	Compressive Strength in P S I	Remarks
Portland cement used 8% - Moist cured			
F-T	23 4[a]	27 5	
W-D	23 4	77	
IMM	23.4	53 3	
F-T	27 0	--	Samples were bruised during molding
W-D	27 0	77 0	
IMM	27.0	--	Samples were bruised during molding
F-T	30.0	33 3	
W-D	30 0	43 0	
IMM	30 0	43.3	

[a]Optimum moisture content as of Proctor

The results obtained with Portland cement were quite disappointing because the cement failed to set and harden. This condition was not helped by the admixture of calcium chloride as suggested by the Portland Cement Association, nor by the admixture of aluminum chloride or aluminum sulfate.

type of a stabilized system to be achieved with this type of soil is "sand-bitumen" (8).

The real problem in the stabilization of this specific soil was the presence of organic matter ranging in size from easily recognizable fractions of peanut shells and roots to colloidal dimensions and viable matter ranging from seeds to microbes.

The seeds sprouted lustily in the soil-cement specimens; the microbes proved to be especially active in the immersion test and provided visual and kakodylic evidence of their existence by reducing the red, iron-oxide color of the soil to a dirty bluish-green and by polluting the atmosphere. Subsequently, some oil-treated specimens were immersed in water contained in an iron bath well covered with rust. The microbes not only changed the color of the soil but, in addition, so thoroughly reduced and dissolved the rust of the bath that the latter showed a clean and shiny metal surface after the seven day immersion test.

The problem of the organic matter could be overcome by chemically sterilizing makro- and microbial agents present in the soil. The relative proportion of organic matter in the fraction passing the No. 200 sieve can be gleaned from a comparison of the thermal curves obtained on the unoxidized and oxidized soil fraction.

NIPE SOIL (TABLE 9)

The tests with this soil were concerned mainly with the influence of mixing moisture on the properties of soil-cement. The results are not conclusive; but there is a tendency for a decrease in strength with increase in moisture content at constant compactive effort. The data do not permit decision as to whether this apparent effect is a primary one of the moisture, or a secondary one either caused by easier destruction of structure during mixing at higher moisture contents or by a resulting change of the density of the specimens. See reference (30).

CONCLUSIONS

The work described and documented in this paper was concerned with a limited number of tests on a limited number of soil systems. Despite these limitations, the following conclusions of general importance could be drawn:

(1) The susceptibility to stabilization of laterite and lateritic soils may vary over a wide range from excellent to poor. As a general rule, this susceptibility increases with increasing degree of laterization as evidenced by the silica-sesquioxide ratio. This confirms conclusions drawn from previous work performed on temperate zone soils in which iron and aluminum ions had been substituted for the exchange ions of the natural soils.

(2) In the absence of the analytical data required for calculation of the SiO_2/R_2O_3 ratio, clay-type determination by differential thermal analysis may be used as an index to the degree of laterization. Highly laterized samples contain aluminum and iron oxides, possessing typical thermal curves, in addition to the Kaolinite mineral which possesses a silica-sesquioxide ratio of 2.

(3) Soils possessing a low degree of laterization may possess more pronounced swell and shrinkage characteristics than normally expected of red lateritic clay soils.

(4) Fairly high quantities of organic matter may be present in red tropical and subtropical soils. This organic matter, especially if biologically active, may prevent a soil from properly responding to stabilizing treatment, even though the soil be granulometrically well suited for such treatment. In such cases, chemical sterilization preceding or simultaneous with the stabilization is indicated.

It may be appropriate to append a few considerations to this paper. Depending upon environment, parent rock and time, soil forming processes may, in the tropical and subtropical zones, produce materials ranging from hard laterite "rock" to non-laterite soils similar to those found in temperate regions.

From the point of view of road use, either as an aggregate, or as a binder, or as a material to be treated with cementing or waterproofing agents, the material becomes more favorable the farther it has progressed on the way to laterization. This indicates that the greatest problem in the road use of laterite soils arises from the effect on the soil systems of the severity of climate, especially the desiccation followed by the monsoon, rather than from the inherent soil properties which at the worst come to resemble more and more those of soils from the temperate regions.

It stands to reason that granular soil stabilization has been more widely employed in laterite areas than the other stabilization methods. Granular stabilized soils suffer most from climatic severity especially from long desiccation periods. This is true not only for tropical climate but also for our own gravel roads in the great plains and for the more frigid climate of Argentine Patagonia, as judged from the wind erosion in these areas.

Considering these general facts, and also the specific properties of most soils in the tropic regions, i. e., cementation and structure which are best left undisturbed, it would appear that the most promising method for low-cost road construction is "earth oiling" as described in the Current Road Problems Bulletin on "Soil Bituminous Roads." Of course, the oil to be employed - and it may be any type of oil, asphaltic, pyrogenous from coal, vegetable or animal origin, or any other, must be well fortified with antioxidant and with bactericidal additives.

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