From February 24, 1943 to September 6, 1943, an average of 4.7 lb. per sq. yd. of calcium chloride was used, or a total of 245 tons on the 7.4 miles treated.

In mid-September this roadway had developed into an excellent stabilized base. It presented a dense surface, was well compacted and of uniform section. The cost of this work averaged 36½ cents per sq. yd. As the next step, a prime coat of one half gallon of R. T. 2 tar and a seal coat of ⅓ gal. of R. T. 6 tar was applied and covered with chips, passing the ¾-in. sieve. This surface treatment cost 17½ cents per sq. yd. The total cost for gravel surfacing, calcium chloride and surface treatment amounted to 54 cents per sq. yd.

In judging this cost, it should be remembered that it is made up of a combination of construction and maintenance, that construction was complicated by a heavy traffic flow using the road, and that this traffic flow was so important that cost was not considered significant in keeping the road in service.

Had asphalt been obtainable, it is likely we would have considered the maintenance aspect only, and used some form of mixed patch, similar to hot or cold bituminous concrete. As this could not be done, we used the next best scheme possible, making use of the available local materials, our own maintenance organization, and methods familiar to our organization. Although economy was not our object, we feel the cost to be extremely low for the work accomplished, even when consideration is given to the usual cost differential of work done by contract against the actual cost of work done by a maintenance organization.

GRANULAR STABILIZED BASE CONSTRUCTION OF ACCESS AND RELOCATION ROADS BY T. V. A.

By F. W. Webster and F. H. Kellogg

General Problems. The problems encountered during the highway construction program of the T. V. A. have made those operations somewhat unique. This has been due to a number of factors, some of which fall outside the scope of this paper, but which will be mentioned to give a background of the conditions under which stabilized base roads have been built.

Most of the activities of the T. V. A. have been located in hilly or mountainous terrain, where the logical road locations follow valleys. With the construction of dams, and consequent impounding of water in these valleys, the relocated highways must either "take to the hills," or be placed on high fills which are periodically or permanently submerged. In either case, numerous problems involving slope stability, settlement, wave erosion and drainage are encountered.

Furthermore, most of the highway construction is subject to rather severe time limitations, either to provide access to dam sites as soon as possible after construction has been authorized, or to replace existing roads before they are inundated by rising reservoirs.

The regional geology of the Tennessee Valley area presents a further problem in that, in general, there are few natural deposits of granular or well graded materials. In the eastern section of the region, the large masses of micaceous schist which constitute much of the material in the Great Smokey Mountains yield highly elastic soils of the A-5 group which lose much of their strength in the presence of water. In the rest of the area, the valleys contain mostly similar materials which become progressively finer to the west, while the bulk of the other soils in the region consists of A-7 clays resulting from the decomposition of limestones and shales *in situ*. Relatively minor exceptions occur in the loess, chert and sand beds of western Kentucky and Tennessee, in alluvial deposits derived from the Cumberland Mountains and in the lower sections of large river flood plains.

The access roads to the various dams must be capable of supporting heavy loads and dense traffic for two or three years, after which time they usually become light to medium traffic roads.

Design. The only complete design studies required of T. V. A. forces are in connection with access roads. Relocations are intended to replace existing facilities, so the type of road to be replaced is the dominant factor affecting the design of a relocated highway. (Exceptions to this occur when on primary interstate roads, the states desire to participate in order to provide a replacement of higher standards.) Moreover, the T. V. A. Highway Division is not considered as a permanent organization, nor one engaged in long-term highway construction. For these reasons, no attempts have been made to standardize designs or to undertake any extensive program of research. Design is not based entirely either on set empirical procedures like the California Bearing Test, or on theoretical considerations involving stress distributions and mechanics. Thicknesses of base, and of sub-grade stabilization are determined largely on the basis of experience, and particularly of experience in the general area involved. Sub-grade treatment and drainage are problems usually handled by the construction engineer according to the local conditions encountered, and are discussed here under separate headings. Extensive studies are made on stability of major fills, but these are outside the scope of this paper. On the other hand, specifications for materials for the stabilized base and seal course have been standardized.

Table 1 gives the grain size distributions, liquid limits and plasticity indices specified for a large part of the T. V. A. construction of granular stabilized bases. The dust ratio (ratio of percentage by weight passing U. S. Standard Sieve No. 200 to percentage passing Sieve No. 40) has been changed for later specifications. This change will be discussed under the heading of performance.

Sub-Grade. For a stabilized base road, which is not expected to bridge over soft spots, the character of the sub-grade is a controlling factor. An inferior sub-grade may be improved by drainage, by replacing the poor material with something better, by increasing the thickness of the stabilized base, or by some combination of these measures. The next few sections will discuss these various measures.

Drainage. The primary function of drainage is considered as that of keeping water from entering the pores of the sub-grade, rather than as removing water that has already entered. The latter function is achieved, for practical purposes, only in sands. The subject of rate of drainage of pore water is being studied by the T. V. A. in connection with investigations of the behaviour of earth dams. Cells have been used to read pressures exerted by pore water in earth embankments. The rate of drop of such pressures as the reservoir level is lowered indicates the speed of drainage. The results to date indicate that rate of drainage can be analyzed by the same methods used

TABLE 1
GRADING OF STABILIZED BASE MATERIALS

Passing U. S. Standard Sieve Size	Stone or Slag, % by Weight	Chert or Gravel, % by Weight
2 -in.		100
1½-in.		90 to 100
1 -in.	100	75 to 100
¾-in	80 to 100	60 to 95
½-in.		30 to 80
⅜-in.	50 to 90	
No. 4	35 to 65	25 to 60
No. 10	22 to 50	20 to 45
No. 40	15 to 30	15 to 35
No. 200	5 to 15	5 to 20
Fraction Passing No. 40 Sieve		
Dust Ratio	Not to exceed 0.5	Not to exceed 0.5
Liquid Limit	Not to exceed 35	Not to exceed 35
Plasticity Index	Not to exceed 8	Not to exceed 8

for the analysis of heat flow, flow of pore water in settlement problems, or any other case of flow in the non-steady state. The test value expressing rate of drainage is indicated as the permeability coefficient, divided by the sum of the specific yield (quantity of gravity water in a unit volume of soil) and the "coefficient of volume change" (quantity of water pressed out of a unit volume of laterally-confined soil by a unit vertical stress). Thus, the efficacy of drainage of a given wet soil could be evaluated quantitatively in advance. Regardless of the outcome of these studies, however, the observations already made concur, with many noted during highway construction, in indicating that the moisture contents of fine soils cannot be reduced in any reasonable time, if at all, by simply lowering the water level.

The policy has been followed in the T. V. A. of locating wet spots in the field, intercepting the ground water which caused such spots with ditches and drains, and replacing the wet soil with dry, compacted material. In fills with crests only 4 or 5 ft. above water level (causeways crossing portions of reservoirs), where capillary water might conceivably saturate the sub-grade, compaction rather than drainage has been used.

Selection and Admixture of Sub-Grade Materials. When studies of the natural sub-grade indicate low potential bearing capacity, it is either removed and replaced with better material, or mixed with sufficient sand and crushed rock to stabilize it. The depth to which such treatment is carried follows no set rules. Usually the minimum depth is 6 in. Occasionally, depths of 1 to 2 ft. in cuts and 2 to 4 ft. in fills which are to be submerged have been excavated and backfilled with selected material. The distribution of stresses in an elastic medium under a circular loading area has been used as a rough guide. Thus, at a depth below the top of the completed pavement equal to the diameter of the loaded area, the vertical stress might be considered as slightly less than 0.3 of the stress at the top of the slab, while at a depth of two diameters, it would be less than 0.1 of the stress at the top of the slab. This question has generally been left to the judgement of the construction engineer.

In the Tennessee Valley, soils of classes A-1, A-2 and A-3 are scarce, so the practice of replacing existing sub-grade materials by such soils has seldom been followed. Usually replacement has involved merely taking out water-logged material and replacing it with material of the same soil type, compacted at controlled moistures. Flocculent lateritic clays of class A-7, compacted at controlled moistures, have frequently been used as replacement material.

In connection with the construction of Alabama Highway No. 1 in Guntersville Reservoir, sufficient sand was found in river alluvium deposits to permit mixing with the natural soil, which was a very plastic A-7 clay. Sufficient sand was used to reduce the plasticity and shrinkage of the mixture to normal values (figures are not available at this time). A 6-in. stabilized base was placed here, with a bituminous prime and a 1-in. mixed-in-place bituminous surface. The traffic density in this section was 1000 or more vehicles per day, including heavy trucks. The road was opened to traffic in the summer of 1938. The winter of 1939–1940 was one of the hardest ever known in that section. Since severe winters are infrequent there, facilities for snow removal were inadequate, and the road was blanketed with snow for considerable periods. Most of the flexible-slab pavements in the area were so badly torn up by disintegration of bituminous surfaces, ravelling and sub-grade failures, that they had to be rebuilt almost completely. However, except for a few scattered spots, no disintegration or sub-grade failure was noted on this particular 3⅓-mile relocation.

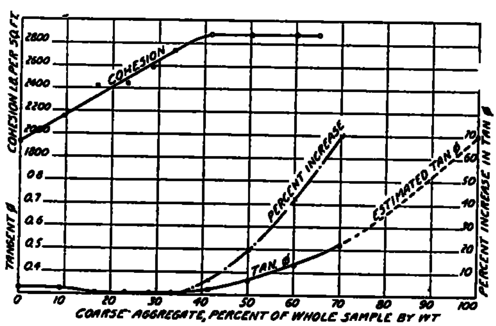

Figure 1. Variation in Strength Due to Additions of Coarse Aggregate to Clay

The use of crushed rock or chert to stabilize local weak spots of moderate depth in a sub-grade has been widespread. As far as is known no sub-grade failure has occurred in such spots. However, attempts to make a stabilized crust on top of a considerable depth of soft plastic material, usually made in connection with construction roads, have been considered failures. The coarse aggregate merely sinks down into the soft soil. Each grain of coarse material is completely surrounded by a matrix of soil, and little is accomplished. Figure 1 shows the results of various admixtures of crushed rock on shearing strength tests of the mix, considering shearing strength as the sum of cohesion and frictional resistance. This figure indicates that as long as the soil content of the mixture is appreciably greater than the normal porosity of the rock, the frictional resistance is the same as if no rock at all had been added.

We have found that calcium chloride is effective in increasing the density to which some soil may be compacted. At the same time, optimum moisture for the soil is decreased. Hence, soil that is nearly at optimum moisture becomes wetter than optimum when calcium chloride is added. This effect has been pronounced when dry calcium chloride has been inadvertantly spilled on the sub-grade. The sub-grade quickly took a wet and soft appearance. However, if the moisture content of the soil is well below optimum, the addition of calcium chloride results in an increased density and therefore in increased strength. While calcium chloride has been used extensively by the T. V. A. in construction of stabilized bases, no large scale efforts have been made to use it to increase the strength of the sub-grade, since cheaper measures are usually available.

Compaction of Sub-Grade. A great deal of reliance has been placed in compaction as a factor in increasing the strength of the sub-grade. All fills that are to be submerged have been compacted in 6-in. layers (loose) at controlled moistures with sheepsfoot rollers. All other fills were placed in 12-in. layers and compacted by the hauling equipment. Where the natural sub-grade material was replaced by another soil, this soil has been compacted by one method or another. All sub-grades have been compacted at least by traffic.

Sub-grade conditions in Cherokee reservoir led to a considerable study and use of compaction at controlled moistures. In that area there was practically no soil avaliable except somewhat treacherous A-7 clays. Moreover, time was not available to do extensive stabilization of the sub-grade, since the dam was an emergency war-time power project, the construction time of which broke all existing records. The roads in this area had to be built before the dam impounded water above the level of existing highways. For this reason, much of the road construction had to be completed in one summer. Therefore, compaction of the existing sub-grade to depths of 6 to 12 in., at controlled moistures, was considered the only measure that could be applied generally. Stabilization with chert, sand, crushed rock and interception of ground waters with ditches and drains was the treatment applied locally in the worst spots, usually in deep cuts.

The utility of compaction at controlled moistures is indicated by Figures 2 and 3. Figure 2 shows the moisture-density diagram for a certain soil as a solid line, with points

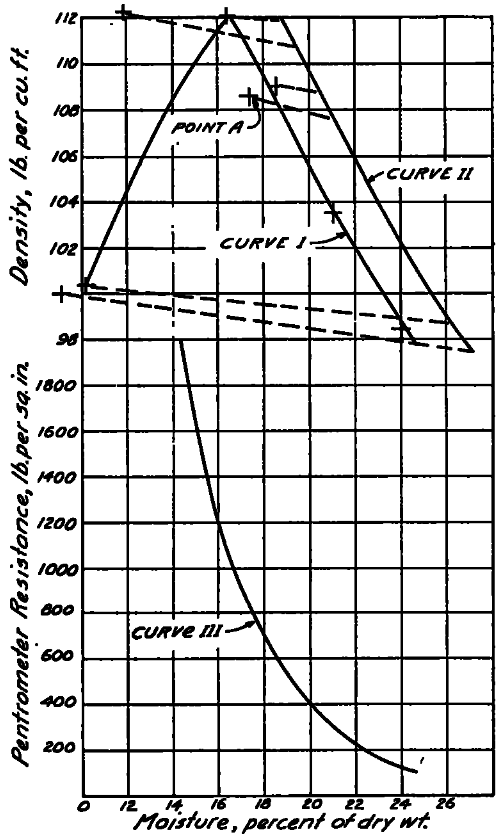

Figure 2. Effect of Compacting Moistures on Volume Changes and Softening

Curve I shows effect of moisture on maximum compaction of sheepsfoot roller rated at 300 lb. per sq. in.

Curve II shows moisture required to saturate various densities.

Curve III shows effect of moisture on resistance to penetration. At 300 lb. per sq. in. resistance, ruts begin to form. At 150–200 lb. per sq. in. the road becomes impassable.

Crosses show moistures and densities of samples as actually compacted.

Dashed Lines show changes in moisture and density when samples are saturated under load of 115 lb. per sq. ft.

Soil is A-4, with PL of 19.

Example

Point A represents: compaction to 108.6 lb. per cu. ft. at 17.4 per cent moisture. Penetrometer resistance at 17.4 per cent is 800 lb. per sq. in. When saturated, moistures and densities follow dashed line from point A to Curve II, where density is 107.6 lb. per cu. ft. and moisture is 21.0 per cent. The penetrometer resistance corresponding to 21 per cent is found from Curve III and is 280 lb. per sq. in. Hence, the fill has changed from a hard, compact mass at 800 lb. per sq. in., to one which is just beginning to rut at 280 lb. per sq. in.

indicating the actual moisture and density of various compacted samples, and dashed lines indicating the change in density as these samples were subsequently saturated under a load of 160 lb. per sq. ft. Note that samples compacted at moistures lower than optimum lost density with saturation, indicating swelling. Samples compacted at or somewhat above optimum moisture showed no appreciable volume change, while those compacted at very high moistures actually increased in density, showing settlement. At these latter moistures, the soil would hardly support construction equipment. Figure 3 shows the moisture-density curve for a given soil, together with shearing strengths for the same soil compacted at various moistures and then saturated and submerged before testing. These figures indicate the desirability of compacting at, or somewhat above, optimum moisture, to avoid appreciable swelling and softening from wetting of the sub-grade, and to obtain maximum strength after saturation.

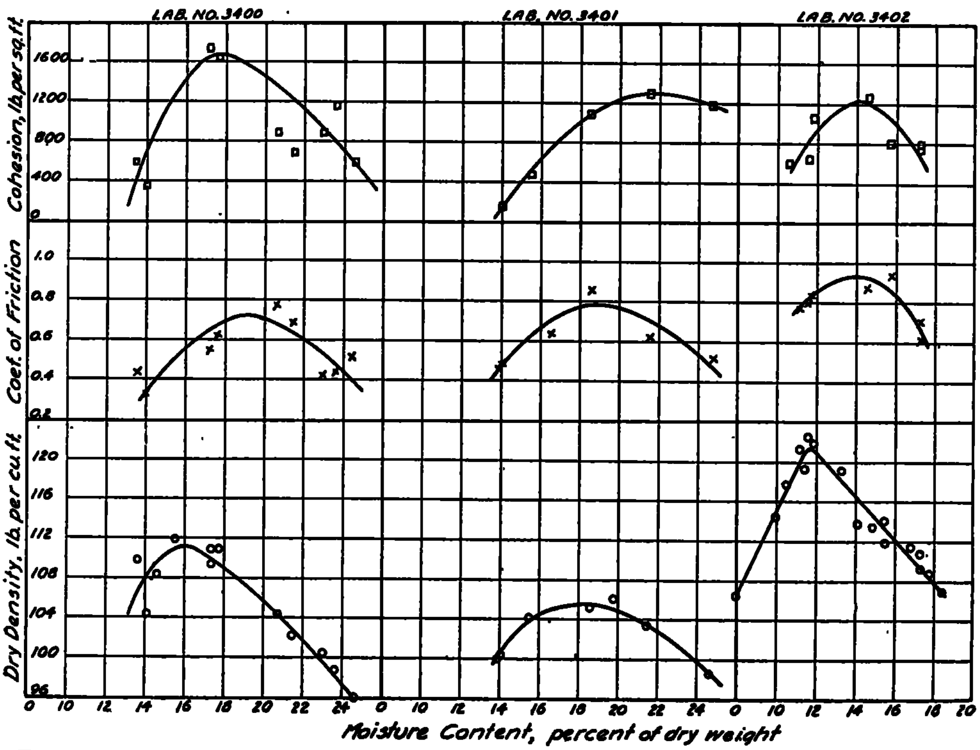

Figure 3. Shear-Compaction Relations. All samples compacted with 13½-lb. hammer (sand ballasted rollers)

The effects of sub-grade compaction alone as a stabilization measure for inferior soils have been illustrated by subsequent experiences, of which two will be described.

The Cherokee Dam access road, a 5-in. crushed stone stabilized base resting on a flocculent clay sub-grade, was subjected to a traffic density estimated at well in excess of 1500 vehicles per day, including loads of 70 lb. per sq. in. (gross load 70 tons). Local conditions of excess ground water were corrected by removing wet soil, replacing it with compacted cherty material, and draining the area affected. Elsewhere, the sub-grade was compacted to a depth of 6 in. From subsequent tests, it is believed that the entire sub-grade became saturated to a depth of 6 in. or more, during the following winter and spring. In the spring, equipment from the dam was moved over this road, giving maximum loads of at least 70 tons, distributed over 24 wheels at tire inflation pressures of 70 lb. per sq. in. Hence, the equivalent diameter of the loaded area would be about 10½-in. Imagining the slab and sub-grade as a homogeneous elastic medium, the stress on the top of the sub-grade would be about 45 lb. per sq. in., and at the bottom of the 6-in. compacted sub-grade, less than 20 lb. per sq. in. No sub-grade failures were noted in the center third of the 4½-mile stretch of road. Five or six sub-grade failures were noted along the edge of the road on the west side, indicating the effects of south-bound loads of heavy equipment. No sub-grade failures are recalled on the east edge of the road.

An interesting speculation is called to mind by this experience. Considering resistance to penetration as an index of soil strength, the maximum penetration force for a soil compacted with a standard, water-ballasted sheepsfoot roller (200 lb. per sq. in.), and subsequently saturated is about 400 lb. per sq. in. This figure holds for a T. V. A. penetrometer and for soils containing no appreciable material that would be retained on a No. 4 sieve. For a soil compacted by the heavier rollers used in dam construction (300 lb. per sq. in.) the maximum penetration resistance after saturation is 1000 lb. per sq. in. Since compaction with heavier rollers cost the same or less than compaction with standard rollers, it is suggested that the "vest pocket" rollers used in highway work could well be replaced by more effective units. T. V. A. costs on compaction at controlled moistures were usually less than 5 cents per cu. yd. Considering the additional strength obtained from this measure, and the cost and time loss involved in searching for selected sub-grade materials, such a procedure might be worth while.

U. S. Highway No. 25, from Morristown to Bean Station, Tennessee, is a road that had to "take to the hills" to avoid flooded areas. It passes through deep cuts, and over partially submerged fills 60 to 65 ft. high, along a terrain consisting entirely of shale and limestones. The soils are classes A-7 and A-5-7, with high liquid limits and shrinkage limits. A 10-mile stretch of this highway was built with a 6-in. stabilized base, with sub-grade treatment essentially similar to that on the Cherokee Access Road. In certain sections, there was an essential difference. The sub-grades were compacted by traffic for about two months. Portions of this traffic-compacted sub-grade were subsequently plowed 6 in. deep and attempts made to re-compact with sheepsfoot rollers at controlled moistures. However, the soil had lost its flocculent texture through compaction, and water added to it merely coated small chunks of hard, compacted clay. During the following winter and spring, the road carried considerable traffc including south-bound loaded coal trucks, applying loads estimated at 25 tons. A number of sub-grade failures occurred along those portions of the road where the sub-grade had not been stabilized with granular material and had been subjected to traffic compaction. All failures were found near the edges of the road, and nearly all along the west side, showing the effects of the coal trucks. Sections of the slab were removed and the sub-grade was tested with a penetrometer. Within about 5 ft. of either edge of the pavement, penetration resistances of less than 100 lb. per sq. in. were noted. The resistances increased to a maximum of about 400 lb. per sq. in. at the center. These figures compare with readings of 400 to + 2000 lb. per sq. in. on the traffic-compacted sub-grade before the stabilized base was put down. The effects of swelling and softening of dry, compacted soil due to waters entering the sub-grade from either side are those that would be suggestsd by Figure 1.

The possible effect of saturation of sub-grades in clay fills with crests only 4 to 5 ft. above water level has already been mentioned. It is hard to say whether such saturation has actually occurred, as submerged fills have been compacted at moistures higher than optimum. Therefore, nearly all the air has been squeezed out of them and they are usually 95 to 98 per cent saturated as built. Errors inherent in testing make it hard to distinguish between such high degrees of saturation and complete saturation. Theoretically, capillary water could rise 50 ft. or more in most of the T. V. A. road fills. Actual observations indicate that the capillary rise seldom exceeds 8 ft. (*U. S. Geological Survey Water Supply Paper*, 489, pp. 32–38). In any case, the submerged fills have shown practically no sub-grade failures, in spite of the large number of them that have been made of supposedly inferior soils. This is considered further evidence of the value of compaction.

Construction of Stabilized Bases. The sources of material for the stabilized base roads constructed by the T. V. A. have been largely local crushed rock or natural gravel and chert banks. On one access road, some crushed slag was used. Soil admixtures have usually been obtained from natural sand banks, although in one case, dolomite tailings from local zinc mines were used. The films of flotation oil on the grains of these tailings reduced cohesion to a negligible factor, but seemed to have no deleterious effect on frictional resistance. The specifications for stabilized base materials have been given under the heading of design.

In designing mixes, field tests are checked by the central laboratory, which recommends mix proportions for a first trial. Subsequent modifications have been made in the field. During construction, field tests are made as required, and central laboratory check tests are made on samples representing each 1000 linear feet of base. For field control, a trailer laboratory has been found useful.

A great deal of trouble was encountered in keeping the dust ratio below 0.5 when crushed rock was used. Either crushing to a maximum size of $\frac{3}{4}$-in., or use of an auxiliary crusher was required, with preference given to the latter. The presence of quarry dirt from seams and overburden was particularly troublesome. A certain amount of care was required in eliminating such material when loading the crusher. The crushing was controlled by checks on the material coming from the bins in the crushing plant. Maximum and minimum percentages of material passing the $\frac{3}{8}$-in. sieve were worked out from the grain size distribution of the soil admixture. Usually, rock from the bins, approximately 50 per cent of which would pass the $\frac{3}{8}$-in. sieve, could be combined with the soil to give a mix passing specifications.

The natural gravel banks used for much of the stabilized base construction in Kentucky Reservoir usually came within specification limits on grain size distribution without admixtures. However, the plasticity indices were generally too high. For this reason, it was necessary to locate sources of fine sand which could be added to the mix to reduce plasticity without bringing the grain size distribution outside of the specified limits. Since a dependable determination of the plasticity index requires an oven with temperature control, a trailer laboratory is particularly useful when this factor is subject to considerable variation in the field.

Batch mixing was used during the construction of the Hiwassee Access Road. All other projects employed mixing in place. This was done because most of the construction involved numerous, widely scattered short stretches of highway, and because of time limits. Designed mixes were kept on the coarse side because of the addition of fines from the sub-grade during blading that usually accompanies mixed-in-place methods.

Calcium chloride has been used in the proportion of 1 lb. per sq. yd. per 3-in. compacted layer. We have found it effective in increasing compacted density, holding moisture and increasing bonds. The experiences with the use of calcium chloride in the early construction work of the T. V. A. have been described by Moreland[1].

Base thicknesses up to 4 in. are constructed in one layer. Thicker bases are built in layers having a compacted thickness of 3 in. It has been found difficult to obtain proper mixing with thicker layers. Both pneumatic tired rollers and road rollers have been used for the compaction of the base. The latter is preferred for finishing, since it gives a smoother surface. The base material is compacted until it has less than 7 per cent air voids and over 85 per cent solids. Densities of the order of 150 lb. per cu. ft. have been obtained on properly designed mixes with specific gravities of grain of 2.7 to 2.8. Densities are checked by the use of calibrated Ottawa sand. Much of the stabilized base has, of necessity, been built under traffic. It is recognized that, if the road is kept rolled until a bond is obtained, and then sealed, before opening to traffic, a better job can be obtained.

The handling of drainage and sub-grade conditions has already been described. Particular care is taken where the highway passes through cuts.

Performance. In general, the performance of the stabilized base roads constructed by the T. V. A. has been surprisingly good. This is especially true in view of the heavy loads some of them have been required to support, of scarcity of the best sub-grade materials, and

[1] "Economics of Stabilization with Calcium Chloride," *Proceedings*, Highway Research Board, Vol. 19, p. 565–569.

of the heavy rainfalls characteristic of the region.

Mean annual rainfall in the Tennessee Valley area varies from about 57 in. in the east to about 50 in. in the west, with certain areas showing over 70 in. At Knoxville, Tennessee, a rainfall of 8 in. was recorded in 24 hours, in September 1944. Under these conditions, with existing designs, it is hard to keep the sub-grade from becoming saturated. It has been suggested that a stabilized base having a bituminous seal which would prevent evaporation, would become softened by water penetrating the base from the sub-grade. Actually, where local pockets of disintegration have been observed, there are no data to give support to such a conclusion, provided the base conforms to specifications. Where tests have been made on cases of disintegration, the plasticity indices, liquid limits, or both, have been higher than the specified limits of 8 and 35, respectively. Although the performances of some bases with plasticity indices of more than 8 have been satisfactory, experiences indicate that the best results are obtained if the liquid limit does not exceed 25 and the plasticity index is kept within the limits of 2 to 4.

The difficulties encountered in obtaining a dust ratio of 0.5 or lower led to an investigation of some of the access roads of known performance under heavy loads. Table 2 gives the test results. Of the six roads investigated, Hiwassee Access Road is considered as having shown the best performances, and Apalachia the poorest. All of the roads have been considered satisfactory, however. On the basis of the results shown in Table 2, the specifications for dust ratio have been amended as follows:

"If as much as 40 per cent of the mixture passes the No. 10 sieve, the fraction passing the No. 200 sieve shall not exceed 50 per cent of the fraction passing the No. 40 sieve. If less than 40 per cent of the mixture passes the No. 10 sieve, and the plasticity index of the mixture is low, the fraction passing the No. 200 sieve may not be more than 65 per cent of the fraction passing the No. 40 sieve."

This amendment has effected tangible savings in cost and time, and has resulted in roads which, on the basis of two to five years of performance records, are eminently satisfactory.

Figure 4 shows the densities to which the portions of various stabilized base materials passing a No. 4 sieve can be compacted by a Proctor compaction test. Densities are plotted against optimum moisture, roughly delineating a curve parallel to the zero air voids curve. Any deviation of these test samples from the specifications listed in Table 1 is noted. The specified grain size limits have been re-computed for material, 100 per cent of which passes a No. 4 sieve. The figure

TABLE 2
ANALYSES OF BASE MATERIALS IN ACCESS ROADS OF KNOWN PERFORMANCE

Project	Dust Ratio	Liquid Limit	Plasticity Index	Percentage by Weight Passing U. S. Standard Sieve Size							
				1½-in.	1-in.	¾-in.	⅜-in.	No. 4	No. 10	No. 40	No. 200
Apalachia	0.549	21.7	1.7	100.0	90.1	85.5	67.4	47.5	38.3	29.3	16.1
Apalachia	0.527	24.2	1.2		100.0	95.3	72.7	47.6	32.5	24.1	12.7
Apalachia	0.633	33.2	6.9	100.0	96.8	88.1	72.5	64.1	58.9	52.0	32.9
Cherokee	0.419	14.5	1.8		100.0	88.3	62.6	44.1	32.5	23.6	9.9
Cherokee	0.500	15.0	1.2		100.0	93.8	66.4	47.2	35.3	24.6	12.3
Cherokee	0.335	13.9	0.7		100.0	82.7	58.6	44.2	34.7	26.0	8.7
Chickamauga	0.615	23.9	9.9		100.0	99.0	72.7	55.8	40.8	25.2	15.5
Chickamauga	0.605	17.2	3.0	100.0	80.3	59.8	46.9	40.2	28.0	15.2	9.2
Chickamauga	0.659	21.0	6.6	100.0	95.0	77.0	53.8	42.6	32.0	21.4	14.1
Fort Loudoun	0.545	27.0	12.2	100.0	98.1	86.2	63.1	51.0	44.3	21.3	11.5
Fort Loudoun	0.473	26.5	12.7	100.0	96.0	74.8	47.1	38.9	33.4	18.6	8.8
Fort Loudoun	0.579	30.5	13.1	100.0	97.8	84.4	57.5	46.5	40.6	20.2	11.7
Hiwassee	0.500	22.4	1.8	100.0	90.3	67.5	41.1	30.3	24.7	20.4	10.2
Hiwassee	0.453	23.6	1.7	100.0	74.3	54.3	38.4	30.5	25.9	21.4	9.7
Hiwassee	0.481	22.8	2.5	100.0	87.9	79.1	55.0	43.5	36.1	29.5	14.2
Ocoee No. 3	0.709	25.1	7.0	100.0	94.4	83.4	56.4	40.7	29.3	19.9	14.1
Ocoee No. 3	0.631	26.5	9.8	100.0	96.6	90.1	71.0	57.9	44.1	29.3	18.5
Ocoee No. 3	0.700	26.9	10.8		100.0	93.8	72.9	56.9	41.9	30.0	21.0

shows that if the fine aggregate (—No. 4) of any base material can be compacted to densities greater than about 125 lb. per cu. ft. by standard Proctor compaction, it will have liquid and plastic limits conforming to the specifications in Table 1, but will not necessarily have the required dust ratio. It also suggests that a dust ratio of 0.5 has no relationship to density. Hence, if density is an indicator of strength, this dust ratio is not. This is in accordance with the data of Table 2. This figure also suggests a way of checking in the field, specifications of the Atterberg limits,

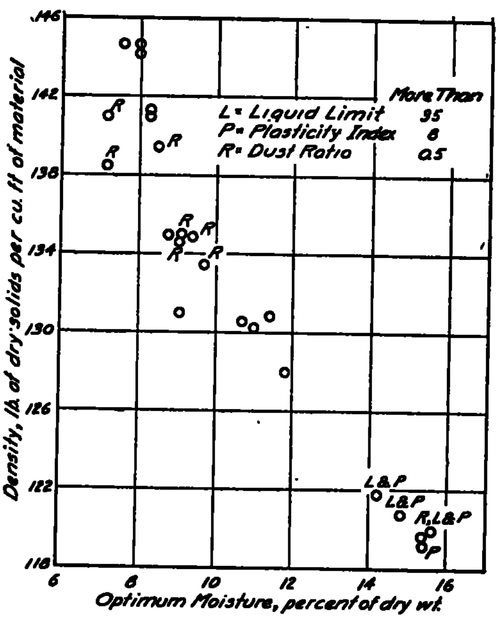

Figure 4. Stabilized Base Roads Compacted density and optimum moisture of fractions of base mixes passing a No. 4 sieve, with mixes not conforming to specifications noted.

when facilities are not available for running the limit tests.

SUMMARY

The experience of the Tennessee Valley Authority in the construction of granular stabilized base course up to and including the year 1939, is outlined in the paper which was presented to the Highway Research Board at its annual meeting in 1939, by Mr. James E. Moreland. Since the presentation of this paper, there has been opportunity to determine by experience the performance not only of roads described in that paper but also of many additional miles of the same type road which have been constructed by the Authority since then. This experience has, to a very marked degree confirmed and established the truth of every conclusion which is outlined in Mr. Moreland's paper.

The Authority has constructed approximately 128 miles of granular stabilized base course, of which 47.5 miles have been access roads and 80.7 miles, relocations. The methods of construction as described in Mr. Moreland's paper have been continued with little change. Complete stabilization of sub-grade, except for some stabilization by compaction as described herein, has not been undertaken. However, local weak spots in the sub-grade have been fairly carefully stabilized, as described herein. Immediately following the completion of all granular stabilized base construction, a bituminous prime coat and a bituminous surface treatment have been applied to the completed base course.

The maintenance cost on the completed surface has been extremely low and has averaged less than $100.00 per mile per year. There have been very few failures, and such failures as have occurred have been of small extent and of local character. These failures have occurred only at places where the roads have been subjected to excessive loads, as described in the case of the Cherokee access road. The performance of the crushed stone stabilized base has been considerably better than that of the gravel base. The performance of the roads constructed by the Authority and described in Mr. Moreland's paper was so favorable that since that report the Authority has used the granular stabilized base with a light bituminous surface as a standard for its access roads, and also for the replacement of such state and county roads as before flooding were composed of crushed stone or gravel base with a bituminous surface.

The Authority has constructed stabilized granular base courses with light bituminous surfaces to provide access to 11 dams. The traffic on these access roads has averaged from 1000 to 1500 vehicles per day for periods from 2 years to 5 years. The average interval of construction use, however, has been about 2½ years. The majority of the workmen employed on the dams have used these access roads in getting to and from work; and in addition, there has been a large volume of heavy truck hauling, since generally the cement used in the dams and a great deal of the construction machinery, supplies and materials have been transported over these roads. The maximum loaded truck using these roads regularly has weighed approximately 20 tons, and on some roads there have been as many as 50 vehicles of this weight per day for long periods. In very rare cases, heavy construction equipment weighing as much as 70 tons has passed over these roads. The passage of such extreme weights, however, has been limited to not more than one or two times for any road.

Stabilized granular base and light bituminous surfacing has also been used by the Authority on both state and county highway relocations on roads constructed to replace existing roads that were flooded by the various reservoirs. A thickness of 6 in. of stabilized base has been used on all State highway replacements. Thicknesses of 4 in., 4.5 in., and 5 in. have been used for county highway replacements. Traffic on the sections of state highways which have been replaced has averaged from 500 to 2000 vehicles per day, and most of these sections have had a large volume of heavy truck traffic. Traffic on county highways constructed as replacements has been light, varying from 100 to 400 vehicles per day, and with little heavy truck traffic.

As indicated in Mr. Moreland's paper, the cost of the first stabilized base construction will average approximately 15 cents per inch of depth for crushed stone, and approximately 8 cents per inch of depth for gravel. In the years 1940 and 1941, there was a slight decrease in cost to an average of about 12 cents per inch of depth for crushed stone. For roads which have been contracted for or constructed by force account after January 1, 1942, the cost has been almost double the cost which was being obtained in the years of 1940 and 1941. In fact, the cost of roads contracted for or constructed since 1942, has been between 21.5 cents and 30 cents per square yard per inch of depth for crushed stone.

MECHANICS OF CALCIUM CHLORIDE TREATMENTS

By J. F. Tribble, *Assistant Construction Engineer, Alabama State Highway Department*

The predominating type of road construction in Alabama is a bituminous surface carried by a carefully designed and controlled soil bound base course. Due to climate and the local materials available, double bituminous surface treatments on soil bound aggregate base courses are used principally in the northern half of the State. In the southern and coastal sections, sand clay and top soil base courses are used to carry single bituminous surface treatments with hot plant mixed wearing surfaces.

In general the base courses are compacted and consolidated in two layers with no specific waiting period between layers or between completion of base course and laying the bituminous surface.

On such base courses a density of at least 100 per cent AASHO is required. Experience has shown that something more is needed. To this end, several experimental roads have been constructed, are under observation, and have given from 3 to 5 years of strenuous service without serious fault.

Problem:

During the hot dry part of the construction season there is a tendency to rush the bottom layers of base courses to the required density by heavy rolling of materials containing too little moisture. The top layers, of necessity, get more thorough processing and consequently, enough moisture for clay slaking, particle lubrication and consolidation before priming. Then the bituminous surface is laid. The wet season arrives. The bottom layer goes through a softening and reconsolidating stage. The entire overlying road structure is disturbed. This internal disturbance not only causes an uneven riding surface but breaks up and shatters the primed crust of the base course. In brittle sand clay base courses