# WEATHERING STUDY OF SOME AGGREGATES

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## SYNOPSIS

An investigation of distressed sections of a surface treated water-bound macadam pavement brought to light the existence of a plastic layer immediately below the surface treatment. The formation of this layer was attributed to the rapid breakdown of an unsound rock. Yet the aggregate had passed all the requirements of the current Virginia *Road Specifications*. This prompted a study of the weathering process for some typical aggregates. Past and present tests on highway aggregates have been concerned with a restricted number of physical or chemical properties considered almost one at a time. Such are the Deval abrasion test, the Page impact test, the Los Angeles rattler test, freezing and thawing tests or petrographic investigations. Each one of those tests allows an evaluation of a given property which may or may not be reflected in the field performance. Experience has shown that a satisfactory test result on the resistance to abrasion or strength or any other given property does not always mean satisfactory durability. In fact, individual tests do not duplicate the combined field conditions which cause the weathering of the aggregates.

Though it is obviously impossible to recreate in the laboratory the exact conditions found in nature, an attempt was made at creating an accelerated weathering test for aggregate. The process consisted on the one hand of a wet grinding of the stone in an ordinary ball-mill and computing the percent loss (i.e., passing a US No. 40 sieve), and on the other hand of the testing of the fines as a synthetic soil. Thirteen different aggregates were investigated. Five were of igneous, four of metamorphic and four of sedimentary origin representing a variety of formations and locations throughout the Commonwealth. The initial gradation and the time of grinding were varied to study their influence on the results. Soil tests included the determination of Atterberg limits and mechanical analysis. Field performance was obtained from typical road projects.

It appeared that the wet-grinding test is a much more strenuous one than the Los Angeles rattler. A tentative standardization was made using the AASHO standard "C" grading and 48 hr. of grinding at 69 rpm. Results showed that all stone suspected of poor performance had a loss greater than 65 percent and the variation in percent lost was of the same order as the variation in performance rating. Most granitic rocks were found to furnish non-plastic synthetic soils. All other rocks gave soils having a high liquid limit which was clearly highest for aggregates with unsatisfactory field performance record. A fair to poor relation was obtained between the fineness of the soil and the performance of the parent material since only some aggregates with poor rating furnished some of the soils with the highest amount of clay-size particles. Some of the non-plastic soils turned out to be coarser than some of the plastic ones. As an illustration the test results obtained on a sample of pure silica (quartz) and pure mica (muscovite) are included in the tabulations. Satisfactory checks were obtained in duplicate tests and the results were well grouped. An increase in the time of grinding or variation in the initial gradation affected the results in an orderly and logical manner. This accelerated weathering process is offered as a potential method to study durability characteristics of rocks. The major item of difference between laboratory and field conditions is that in nature the water used would have been more active since the distilled water was practically void of oxygen, carbon dioxide, organic and inorganic salts and acids found in atmospheric and subterranean waters.

In the spring of 1947, the alarming condition of a section of route 250 located west of Charlottesville and 1.6 mi. east of the Nelson-Albemarle County line prompted a special investigation of the project. The pavement is a surface treated waterbound macadam. The terrain is rolling to hilly. The underlying rock formations are pre-Cambrian granitoids and have weathered fairly uniformly to form silty to sandy clay soils. The striking feature brought out by the survey was the variations in performance since the distress appeared to be limited to certain small areas which had "shoved and pushed" while the remaining of the pavement was apparently in good condition. Since the climate, the traffic, the soil, and the design were practically the same throughout the project, an attempt was made at correlating the failure to the fifth factor of the "performance equation": material.

"The sampling of the road surface treatment in the Spring of 1947 brought to light the existence of a plastic layer between it and the macadam and firmly attached to the former. The thickness of the plastic layer has been found variable up to ½ in."[1] Everywhere the macadam base was found in excellent condition. The investigation was focused on the surface treatment and the formation of the plastic layer.

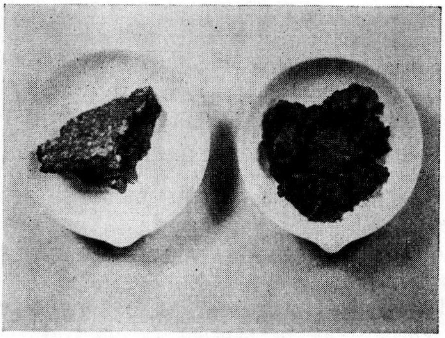

**Figure 1. Left: Sample of Surface Treatment as Obtained from a Failed Section of US 250 (Albemarle County). Right: Similar Sample Disintegrated After 96 Hr. Immersion in Cold Tap Water.**

The surface treatment from the failed areas had retained little of its stability after two years. This was confirmed experimentally by removing a piece of the failed surface treatment to the laboratory and immersing in cold tap water. The sample is shown on Figure 1, right side, after 96 hr. in water; on the left is a similar specimen kept air dry for reference. As observed this average sample had slaked into a muddy mixture of broken aggregate particles and plastic, oily mud, with practically no cohesion. It appears that in a matter of two years under traffic, the aggregate had weathered sufficiently to surround the stone particles with a plastic film, yet this aggregate had at the time of construction passed all the specifications set up by the Commonwealth.

This is a typical example but not a unique one. A few years ago a similar condition (see Fig. 2) was noticed on a new section of US 60 in Powhatan County near the Cumberland County line, and also on a third location on US 29 in Amherst County. Again it was believed that the aggregate in the surface treatments weathered rapidly to form a soil layer above the macadam. The aggregates used were granitoids from local quarries which had been inspected and approved for use in macadam construction and surface treatment on the bases of present requirements.

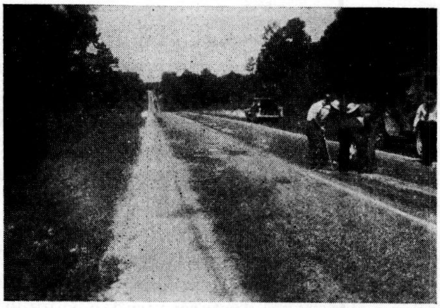

**Figure 2. Localized Failures of the Bituminous Surface Treatment on Rt. 60 in Powhatan County**

The problem turned out to be a fundamental one: how can such an aggregate be detected and rejected before its being used in highway construction and maintenance?

PREVIOUS WORK

This is by no means a new problem. Back in Roman times the first attempts were made at selecting satisfactory rocks to be used as flag stones, the surfacing material of the time. One might venture to say that the choice was not always bad since many a highway on which the Legions of Caesar marched some 2000 years ago is still in excellent condition.

[1] Furgiuele, A. W. and Melville, P. L., "Performance Survey of Projects 723F,B-3, 4-G" *Report* of the Research Section (Unpublished), Virginia Department of Highways, Richmond, 1947.

A milestone in the use of highway aggregates was the invention by the pioneer Scotch engineer and road builder John MacAdam of the type of pavement named after him. The need for aggregate testing was then greater than ever. The Laboratoire des Ponts et Chaussées in Paris developed the first abrasion test for stone in 1870. In 1878 Deval invented the now world famous test used to determine the resistance of mineral aggregates to abrasion. It was one of the first tests approved by the American Society for Testing Materials in 1908 and, with minor modifications, was used in the United States for many years to determine the percent of wear and the French coefficient of wear.

Testing in this country dates back to 1893 when L. W. Page was placed in charge of the first American highway laboratory by the Lawrence Scientific School of Harvard University. Page became afterwards the first director of the U. S. Bureau of Public Roads which has been testing stone since 1905. He also introduced a test for the cementing value of the broken stone dust and a test for toughness by the Page impact machine.

As aggregate testing became of age, a number of shortcomings became apparent with the Deval method. In particular the Deval (as well as the Page) test required a sample of ledge rock which might not have represented the material actually used. For a number of years rounded particles could not be tested. "A criticism of the standard Deval abrasion test frequently made is the comparatively small range in values found in testing rock of the quality ordinarily used in road construction. The test is essentially an abrasion rather than an impact test and for this reason certain types of materials which are very low in toughness but at the same time quite hard will show relatively low abrasion losses in this test. Certain granitic materials fall in this class. Such materials are frequently reported as giving unsatisfactory results in service even though the percentage of wear by the Deval test may be quite low".[2]

To offset these handicaps "the Los Angeles abrasion machine was devised by employees of the Los Angeles City Engineer's office in 1916. Subsequently it was adopted by the California Division of Highways"[3] and after several years of investigation was also adopted by the Virginia Department of Highways in its 1938 *Road Specifications*. This was a progressive step and a remarkably reliable relationship has been established between losses in the so-called Los Angeles rattler test and service records of materials. Yet once in a while, with all its advantages, and its definite superiority over the Deval, the rattler test has given satisfactory results for aggregates which did not perform adequately. Such was, for instance, the case of the stone used in the surface treatments previously described.

It may be stated that many an aggregate will pass all requirements on wear, impact, and toughness and yet may not be durable. Concrete pavement investigations have shown over and again that strength does not mean *per se* durability. It is logical to expect that for surface treatment or macadam aggregates, a high initial strength (whatever the criterion to measure it) is no panacea.

A number of soundness tests have been devised. Freezing and thawing as well as salt tests have given valuable information on the soundness of a number of aggregates. In addition, petrographic investigations will permit identification of suspicious minerals. All these test methods contribute to the evaluation of the probable durability of aggregates. But whatever the methods, the final opinion can only be derived in the light of the service record of the material where and when it has been subjected to weathering rather than any combination of artificially created laboratory tests.

Weathering is not a clear-cut process and to try forecasting the durability of an aggregate subjected to natural degradation—in addition to man-made forces such as traffic—is no simple problem. The investigation reported in this paper may be considered as a tentative method for furthering the prognostication of the durability of aggregates and as a suggestion for an accelerated weathering test.

## THEORY

Webster's *International Dictionary* (Second Edition) defines weathering as "the action

[2] Woolf, D. O. and Runner, D. G., "The Los Angeles Abrasion Machine for Determining the Quality of Coarse Aggregate", *Proceedings*, ASTM, Vol. 35, 1935.

[3] Woolf, D. O., "Results of Cooperative Tests Using The Los Angeles Abrasion Machine", *Proceedings*, Highway Research Board, Vol. 16, 1936.

of the elements in altering the color, texture, composition or form of exposed objects". The elements are usually understood to mean such natural conditions as rain, snow, sleet, hail, wind, temperature, lightning and minor phenomena such as the aurora borealis or cosmic rays. They may be static (viz. a constant humidity) or dynamic (viz. a change in temperature) and act separately or concurrently. But in engineering it is indeed difficult to separate clearly the so-called natural phenomena from those which are man-made. For example there is in Virginia a sand known by the nickname of "Black Jack". From a geological point of view it is a broken Triassic diabase which withstands the so-called natural weathering but quickly turns into a plastic fine soil when used in highway work. In particular it has performed quite poorly as an aggregate for surface treatment. Of course such factors as traffic or crushing of the aggregate are man-made, but they will create new conditions for the courses of alteration by the elements.

Man-made conditions may speed up or slow down the processes of nature but it can be stated that the actions of man are mere "accidents" on the overall trends of natural events. The tendency is toward an equilibrium which is to be reached after an infinite time and an infinite number of alterations. The alterations are efforts toward equilibrium with changing surrounding conditions, whether truly natural or man-made.

Thus it seems that for the engineer, weathering is more a common concept than a so-called scientific truth. For this reason weathering should be construed in this paper in its broader meaning involving changes directly or indirectly related to the natural processes caused by elements.

For the sake of clarity such weathering will be oversimplified and divided into three main types of actions: chemical, physical or a combination of both. Under chemical action shall be included processes resulting in changes in the composition of rock. The major ones are listed in Table 1. Under physical weathering shall be included processes resulting in the disruption of rock and shown in Table 2. The result of either process will eventually be the production of a material which is called soil.

If we investigate that soil (and by this we hereafter mean the material passing a US Standard Sieve No. 40) from a mineralogical point of view, we shall find a number of particles varying in composition, shape and size. The soil technician will recognize in them clay, silt and some fairly unweathered parent material. In addition, some organic matters are usually found in soils.

From an engineering point of view the rock parent material will weather into silt and clay if we discount minor colloids and chemicals. The differentiation between silts and clays is commonly made by their grain sizes: silt particles between .053 and .005 mm. in diameter and clay ones smaller than .005 mm. in diameter. Clay is commonly identified by its plasticity which may be recognized in a soil by its shrinking and cohesibility upon drying. Whether grain size and plasticity can be scientifically correlated is open to speculation. Investigations into the mineralogy and behavior of clay have shown the problem to be a complex one. For practical purposes the Atterberg limits have been found convenient means to investigate and classify soils.

TABLE 1

CHEMICAL ACTIONS ON ROCK

| |
|---|
| Oxydation and Reduction |
| Hydration and Dehydration |
| Carbonation and Decarbonation |
| Silification and Desilification |
| Sulphidation and Desulphidation |
| Solution and Precipitation |

TABLE 2

PHYSICAL ACTIONS ON ROCK

| | | |
|---|---|---|
| Wear | Abrasion | Attrition |
| Impact | Fatigue | Crushing |
| Splitting | Shearing | |

In the investigation reported in this paper a ball mill was used to grind stone into what may be considered as a synthetic soil. The actions taking place in the mill were mostly physical, but the fact that water was always present had a definite effect on chemical compounds. In addition, the comminuting resulted in an output of heat. Measurements indicated that the water in the jars was raised from 1 to 5 deg. C. within half an hour after the beginning of the test and was maintained thereafter at a fairly uniform temperature approximately 2 deg. C. above room temperature. Since distilled water was used, organic and inorganic acids and salts as well as dissolved oxygen and carbon dioxide were the major natural weathering factors absent in the laboratory tests. But during the comminuting the water formed with the minerals a number of chemical compounds which often had a definite expediting action on the weathering processes. Thus conditions were not basically different from those in the field except for intensity and duration of the actions.

### EQUIPMENT AND MATERIALS

To try to evaluate the weathering characteristics of some Virginia aggregates, typical stone samples were ground to a fine powder in a ball mill. The apparatus used was manufactured by Paul O. Abbe, Inc., at Little Falls, New Jersey and is shown in Figure 3. It is powered by a $\frac{1}{6}$-hp. AC. motor and the porcelain jars rotated at the rate of 69 rpm. The inside of the jars is glazed and tops are ground to fit exactly. Rubber gaskets are provided to prevent leakage. No loss in weight of the jars was noticed throughout the investigation. The balls used in grinding were carefully selected commercial flint pebbles. They are smoky gray or brown to nearly black nodules. Flint is composed of quartz ($SiO_2$) and opal ($SiO_2 + n\,H_2O$) and has a hardness according to Moh's scale of 6.5 to 7. This is harder or as hard as any of the aggregate tested since feldspar has a hardness of about 6, limestone 3, mica 2, and only the quartz found as a cementing agent in granitic rocks has a hardness approaching that of flint pebbles. Thus in the process of grinding it is natural to expect that the softer stone will be ground and the harder ones only slightly worn. Moreover, the flint pebbles being rounded and homogeneous will have less of a chance to wear than a material of equivalent hardness when angular and heterogeneous such as the siliceous cement of granitic rocks.

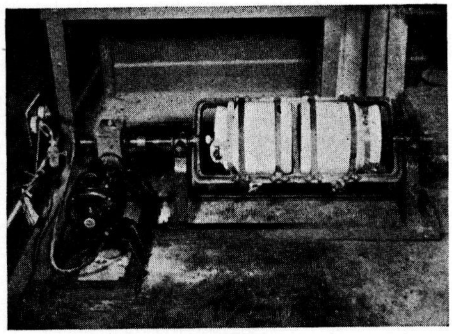

**Figure 3. Ball Mill Used in Investigation**

A detailed record was kept on the wear of the flint pebbles and it was found that for the standard charge of 3,000 g. of pebbles in the mill, the loss had been about 15 g. after 120 hr. grinding at a speed of 69 rpm. This loss will of course vary somewhat with the type of stone being ground, but the variation is small and of the order of 1 or 2 g. It may be stated that the ball loss is 0.5 percent and may be neglected in comparison with the aggregate loss.

For the tests each jar was charged with 3000 ± 1 g. of clean pebbles and 1000 ± 1 g. of distilled water and 500 ± 1 g. of clean, dry stone. The stone specimen may be a piece of ledge rock broken to the required size in the laboratory with a hammer or an average sample out of the crusher. The gradations of the test samples followed the requirements set up by the AASHO for the Los Angeles Test (Designation T 96) and are shown in Table 3.

TABLE 3
GRADINGS OF TEST SAMPLES

| Sieve Size (Square Openings) | | Percent Weight | |
|---|---|---|---|
| Passing | Retained on | Grading B | Grading C |
| $\frac{3}{4}$″ | $\frac{1}{2}$″ | 50 | 0 |
| $\frac{1}{2}$″ | $\frac{3}{8}$″ | 50 | 0 |
| $\frac{3}{8}$″ | No. 3 | 0 | 50 |
| No. 3 | No. 4 | 0 | 50 |

The time of grinding was varied from 120 hr. in series A, to 72 in series B and 48 hr. in series C. Upon completion of a test, the jars were taken from the mill and emptied into large glass dishes. The pebbles and the unground rock were removed. The slurry of ground rock and water were placed in the oven to dessicate and then weighed after cooling to the nearest gram. The percent loss by weight was computed and recorded.

The ground stone was then considered as a synthetic soil and was tested accordingly. The liquid limit (AASHO Designation T 89—using the modified grooving tool), the plastic limit (AASHO Designation T 90) and the plasticity index (AASHO Designation T 91) were used to determine the physical properties of the synthetic soils. Besides Atterberg's limits, the testing included determination of the specific gravity and the mechanical analysis (AASHO Designation T 88). In addition chemical analyses were performed on some selected samples derived from sedimentary rocks. Results are shown in Table 4. The investigation was centered around the aggregates produced by 14 Virginia quarries located throughout the Commonwealth and representing a number of different geological formations. Quarries 6 and 12 were dropped from the investigation because of erratic results obtained which were attributed to the large lithological variations between the various sections of the outcrop. On the other hand quarry 3 was subdivided into two parts as explained in the following discussion.

Quarry 1 is located in Loudoun County. The material is of Triassic Age and known locally as the Prince William Shale. The misleading name of shale comes from the badly fractured and foliated appearance of what

TABLE 4

CHEMICAL ANALYSIS OF SYNTHETIC SOILS FROM LIMESTONE AND DOLOMITE PARENT MATERIAL

| Quarry | CaO | MgO | Iron & Aluminum Oxide | Insoluble | Loss on Ignition |
|---|---|---|---|---|---|
| | % | % | % | % | % |
| 5 | 49.34 | 1.31 | 4.06 | 0.40 | 40.62 |
| 9 | 29.82 | 22.70 | 0.60 | 0.06 | 46.82 |
| 14 | 28.52 | 15.22 | 6.82 | 7.76 | 38.52 |

actually is a basalt. Quarry 2 in Culpeper County contains a Triassic Conglomerate (trap phase). Quarry 3 is in Albemarle County and was the source of the aggregate used on the first project described early in this paper. It is interesting to note that though the major part of the stone is a quartz-amethyst gneiss, the outcrop is distinctly cut by a large dyke of pyroxenite. This latter material has been identified by the index 3 A. Both formations are pre-Cambrian in age. A few miles away is quarry 4. The material is a pre-Cambrian quartz-monzonite which shows some schistosity.

Quarry 5 in Augusta County is an outcrop of arenaceous limestone. It is massive, light to dark grey in color and characterized by thin beds of sandstone which upon weathering stand out as ribs on the surface. It is Cambrian in age. Quarry 7 in Nelson County contains a pre-Cambrian gneiss with large amount of blue quartz and chlorite. In Amherst County is quarry 8. The formation is the pre-Cambrian Lovingston hornblende gneiss with large white phenocrysts of feldspar. Also to be noted is the pyroxene.

Quarry 9 is in Botetourt County. The rock is the shady dolomite, a light grey uniform high magnesium limestone. Quarry 10 is in Grayson County. The material is the pre-Cambrian Grayson granite gneiss with large phenocrysts of feldspars and small amounts of chalcopyrite. Quarry 12 is an outcrop of pre-Cambrian mica-schist in Grayson County. It is remarkable for its extreme schistosity and large amount of mica. Quarry 13 in Dinwiddie County contains a pre-Cambrian medium to coarse grain variety of Petersburg granite with large amounts of light blue quartz and biotite. Finally quarry 14 located in Botetourt County contains a high magnesium limestone.

## RESULTS

If we consider first the tests reported in Table 5 where the time of grinding was 48 hr. and the initial gradation was the AASHO (Designation T 96) "C" grading, the percent loss (i.e., passing a US No. 40 sieve) varied from a low of 42 for the dolomite (No. 9) to a high of 99 for the mica-schist (No. 11). The average value was 62 percent. An increase in the time of test increased the amount of fines produced. A coarser gradation resulted in a lower loss in weight after comminuting. When check tests were performed the maximum variation was 28 percent and the minimum zero.

The liquid limits were uniformly high and averaged 50. The highest was 72 for the mica-schist (No. 11) and the lowest 37 for the dolomite (No. 9). It must be noted that it was practically impossible to obtain satisfactory tests results with most igneous formations which were classified as non-plastic. As will be seen later this so-called non-plasticity could not be correlated with the grain size of the soil. The plastic limits were also rather high with a maximum value of 53 for the mica-schist (No. 11), a minimum of 25 for the dolomite (No. 9), and an average of 40. The indexes of plasticity were rather low and averaged 10 with a low of 1 for the Triassic conglomerate (No. 2), and a high of 19 for the mica-schist (No. 11). An increase in the time of grinding resulted in a slight increase of

TABLE 5
TEST RESULTS

| Quarry | Type of Rock | L.A. Loss | Gradation | Grinding Time | Weight Loss | | | Liquid Limit | | | Plastic Limit | | | Plastic Index | | | Percentage Silt Size | | | Percentage Clay Size | | | Field Perform. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Max. | Min. | Avg. | Max | Min | Avg. | Max. | Min. | Avg. | Max. | Min. | Avg. | Max. | Min. | Avg. | Max. | Min | Avg. | |
| | | | | hr. | % | % | % | | | | | | | | | | | | | | | | |
| 1 | Triassic "Shale" | 14[a] | C | 48 | 71 | 64 | 67 | 57 | 51 | 54 | 44 | 43 | 44 | 13 | 7 | 10 | | | 22 | | | 21 | Poor |
| 2 | Triassic Conglomerate | 14[a] | C | 48 | 60 | 59 | 60 | | | 42 | | | 41 | | | 1 | 21 | 21 | 21 | 43 | 38 | 41 | Good |
| 3 | Gneiss | 26[a] | C | 48 | 56 | 53 | 55 | | | NP | | | NP | | | NP | | | 22 | | | 38 | Good |
| 3 A | Pyroxenite | 40[a] | C | 48 | 74 | 67 | 71 | 51 | 45 | 48 | 46 | 40 | 43 | 5 | 5 | 5 | | | 20 | | | 36 | Very Poor |
| 4 | Quartz Monzonite | 35[b] | C | 48 | 69 | 63 | 66 | | | NP | | | NP | | | NP | | | 29 | | | 50 | Fair |
| 5 | Arenaceous Limestone | No Test | C | 48 | 60 | 54 | 57 | 45 | 44 | 45 | 36 | 32 | 34 | 12 | 9 | 11 | | | F | | | F | Fair |
| 7 | Gneiss | 24[a] | C | 48 | 48 | 48 | 48 | | | 55 | | | 48 | | | 7 | | | 30 | | | 30 | Good |
| 8 | Granite Gneiss | 40[a] | C | 48 | 76 | 74 | 75 | | | NP | | | NP | | | NP | | | 28 | | | 40 | Very Poor |
| 9 | Dolomite | 18[b] | C | 48 | 48 | 36 | 42 | 39 | 35 | 37 | 25 | 24 | 25 | 15 | 9 | 12 | 22 | 21 | 21 | 17 | 15 | 16 | Very Good |
| 9 | Dolomite | 18[b] | C | 72 | | | 60 | | | 36 | | | 29 | | | 7 | | | | | | | Very Good |
| 9 | Dolomite | 18[b] | C | 120 | | | 65 | | | 40 | | | 26 | | | 14 | | | 38 | | | 4 | Very Good |
| 9 | Dolomite | 18[a] | B | 120 | | | 36 | | | 44 | | | 32 | | | 12 | | | 22 | | | 3 | Very Good |
| 10 | Granite | 25[a] | C | 48 | | | 49 | | | NP | | | NP | | | NP | | | 28 | | | 32 | Very Good |
| 11 | Micaschist | No Test | C | 48 | | | 99 | | | 72 | | | 53 | | | 19 | | | 30 | | | 45 | Very Poor |
| 13 | Granite | 40[b] | C | 48 | 63 | 60 | 62 | | | NP | | | NP | | | NP | | | 30 | | | 40 | Good |
| 13 | Granite | 40[b] | C | 72 | | | 51 | | | NP | | | NP | | | NP | | | | | | | Good |
| 13 | Granite | 40[b] | C | 120 | | | 83 | | | NP | | | NP | | | NP | | | | | | | Good |
| 13 | Granite | 40[a] | B | 120 | | | 48 | | | NP | | | NP | | | NP | | | | | | | Good |
| 14 | Dolomitic Limestone | 18[b] | C | 48 | 51 | 40 | 46 | 46 | 45 | 45 | 35 | 31 | 33 | 15 | 10 | 13 | | | F | | | F | Very Good |
| | Ottawa Sand | No Test | No. 30 | 100 | | | 100 | | | NP | | | NP | | | NP | | | 10 | | | 88 | |
| | Muscovite | No Test | No. 30 | 100 | | | 100 | | | 103 | | | 64 | | | 39 | | | 28 | | | 70 | |

[a]—L.A. Test on "B" Grading
[b]—L.A. Test on "C" Grading
NP—Non Plastic
F—Analysis Interrupted by Flocculation

both the plastic and the liquid limits with little change in the index of plasticity.

The results of the mechanical analysis are somewhat conflicting. This may be due, in part, to the fact that difficulties were encountered in the performance of the tests.

Most synthetic soils had a tendency to flocculate and though many dispersing agents were tried, several of the tests could not be completed. There was a fair to poor relation between the plasticity of the synthetic soil and its fineness. A number of the more plastic soils had a large amount of clay size particles but so many exceptions were noted that no conclusion could be drawn.

In order to facilitate comparison of test results, the percent abrasion loss in the Los Angeles rattler test has been included in the tabulation. There is a fair to good agreement between percentages lost by the two different tests though, as is to be expected, the ball mill is undoubtedly a much more strenuous one. Under standard practices an upper limit of 40 percent loss in the Los Angeles rattler has been used to select satisfactory aggregates. In the ball mill grinding test, it appears that a loss of more than 65 percent could be correlated with a fair to very poor field performance. This can be offset or accentuated—as the case may be—by the plasticity of the resulting fines.

Of special interest is the material coming from quarry site No. 3 since it has been considered the cause of the road failure previously described. The outcrop had to be divided into two parts since the formation is cut by a massive pyroxenite dyke. The latter was classified under the index 3 A. The test results between 3 and 3 A were drastically different. While gneiss No. 3 gave 55 percent of non-plastic fines, under the constant conditions, the pyroxenite No. 3 A furnished 71 percent of fines having a liquid limit of 48 and a plasticity index of 5. This striking finding may be extrapolated to other quarry sites where pyroxene and feldspar were among the major minerals identified in the stone. Such is, for example, the case of site No. 8 from which came a gneiss which lost 75 percent (though the fines were non-plastic). This aggregate when used in construction on US Rt. 29 in Amherst County gave an alarmingly poor performance similar to that of aggregate 3 A on US Rt. 250 in Albemarle County.

Only two of the aggregates gave somewhat conflicting results. No. 2 had a fairly high loss in grinding although it has a low Los Angeles abrasion loss and a satisfactory road performance. This may be explained from the fact that it is an heterogeneous conglomerate on which comminuting may have an especially serious degrading effect. The other one is the granite from site No. 13 which had a high loss both in the ball mill and the Los Angeles rattler test. The fines were non-plastic and the performance reported as "good" by field forces. The fact that some aggregates were reported as having a satisfactory field performance when the contrary was suspected from the grinding test results may be explained as follows: Wet grinding in the ball mill was a much more severe test than that of the natural conditions to which those given aggregates have been subjected so far. On the other hand there may have been other factors, in the road projects where they have been used, which, for performance, overshadowed their weathering characteristics. It would be pointless to investigate weathering processes of the aggregate when—for some other causes—the pavement has too short a life to permit the natural physical and chemical degradation of the stone.

Finally, as a matter of comparison, two tests were performed on pure minerals. One on silica (quartz) and the other on Muscovite (mica). The former gave an ideal non-plastic rock-flour and the latter a highly plastic clay (though coarser than the rock-flour).

Thus, it appears that the wet-grinding gave worthwhile prognostics which could be related to field performance. As a potential test it should be noted for its rather good reliability. Results could be satisfactorily duplicated and were well grouped. Some of the tests were performed on different samples from one given quarry site, and a number of months apart without conflicts. Since the tests were performed using the AASHO "C" grading as most practical, this method of investigation would be especially suited for surface treatment aggregates.

The results do not show how close to natural the synthetic soils were. A similarity undoubtedly exists, since laboratory tests on soils neighboring the quarry sites gave results corresponding to those herewith reported. The use of a more active water in the ball mill (similar to atmospheric and subterranean waters) would no doubt emphasize that situation.

## SUMMARY OF RESULTS AND CONCLUSION

In the light of the investigational tests described and of the data reported the following may be summarized:

1. It is possible to test a stone aggregate by wet-grinding in a ball mill and to obtain the following information:
   a. The percentage lost (i.e., passing a US No. 40 sieve).
   b. The physical properties of the synthetic soil thus produced.
2. The results were correlated to field performance as follows:
   a. Aggregates with unsatisfactory field performance lost more than 65 percent under standardized conditions.
   b. High plasticity of the fines corresponded fairly well to poor field performance.
   c. Rocks with large amounts of feldspars and especially of pyroxene had poor field performance and high grinding losses.
3. As a potential test to forecast the weathering characteristics of aggregates, the method investigated has the following advantages or disadvantages.
   a. It is a more strenuous quality test than common standard tests such as the Los Angeles rattler or freezing and thawing tests because it consolidates several contributing weathering agents.
   b. The results are reliable and can be duplicated.
   c. The test appears more efficient on smaller particles and would thus be especially suited to test small size aggregates such as those used in surface treatment.
   d. It requires more attention and time than present soundness tests.
   e. It is likely that this accelerated weathering test may in certain cases be more severe on the stone than natural conditions encountered during the normal life of the pavement.

As a conclusion it may be stated that this imited series of exploratory tests has indicated the potential value of a wet-grinding test followed by an investigation of the synthetic soil produced in the comminuting. It is hoped that this study may be a contribution to a more adequate laboratory test which may permit a satisfactory prediction of the weathering characteristics and field durability of stone aggregates.

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## REFERENCES

1. Casagrande, A. and Fadum, R. E., *Notes on Soil Testing for Engineering Purposes.* Graduate School of Engineering, Harvard University, 1940.
2. Clarke, F. W., "The Data of Geochemistry", *Bulletin* 770, U.S.G.S., Washington, D. C., 1924.
3. Clemmer, H. F., "Abrasion Tests for Aggregates", *Report on Significance of Tests*, A.S.T.M., 1943.
4. Deval, "Abrasion Test for Crushed Rock", *Annales des Ponts & Chaussées*, 1879.
5. Erkblaw, G. E. and Grim, R. E., "Some Geological Relations Between the Constitution of Soil Materials and Highway Constructions", Ill. Geological Survey, *Report of Investigation* 42, 1936.
6. Furgiuele, A. W. and Melville, P. L., "Performance Survey of Projects 723 F, B-3, 4-G", *Report* of the Research Section (Unpublished), Virginia Department of Highway, Richmond, 1947.
7. Goldbeck, A. T., Gray, J. E. and Ludlow, L. L., "A Laboratory Service Test for Pavement Materials", *Proceedings*, Am. Soc. Testing Mat., Vol. 34, 1934.
8. Grim, R. E., "The Clay Minerals in Soils and their Significance", *Proceedings*, Purdue Conference on Soil Mechanics and Its Applications, Purdue University, 1940.
9. Grim, R. E., Bray, R. H. and Bradley, W. F., "The Mica in Argillaceous Sediments", *Am. Min.*, Vol. 22, 1937.
10. Grim, R. E., "Relation of the Composition to the Properties of Clays", *Journal* Am. Cer. Soc., Vol. 22, 1939.

11. Hauser, E. A., "Silicic Chemistry in Highway Research", *Proceedings*, Highway Research Board, Vol. 27, 1947.
12. Hewes, L. I., *American Highway Practice*, Vol. I, John Wiley, Inc. New York, 1942.
13. Johnson, A. L. and Davidson, D. T., "Clay Technology and its Application to Soil Stabilization", *Proceedings*, Highway Research Board, Vol. 27, 1947.
14. MacNaughton, M. F., "Physical Changes in Aggregates in Bituminous Mixtures Under Compaction", *Proceedings*, Assoc. Asphalt Paving Technologists, 1937.
15. Melville, P. L., "Proposed Studies of Weathering Characteristics of Virginia Rocks", *Proceedings*, Va. Academy of Science, 1947.
16. "Methods for the Determination of the Physical Properties of Road Building Rock", *Bulletin* 348, U.S.D.A., 1916.
17. Parrott, W. T., "Notes on the Abrasive Quality of Some Virginia Aggregates," *Proceedings*, Va. Academy of Science, 1947.
18. "Relation of Mineral Composition and Rock Structure to the Physical Properties of Road Materials," *Bulletin* 370, U.S.D.A., 1916.
19. von Rittinger, P. Ritter, *Lehrbuch der Aufbereiturgskunde*, Berlin, 1867.
20. Rockwell, A. P., *Roads & Pavemenst in France*, John Wiley, Inc., New York, 1896.
21. Shelburne, T. E., "Crushing Resistance of Surface Treatment Aggregates," *Bulletin* 76 (Research Series), Purdue University, 1940.
22. *Standard Specifications for Highway Materials and Methods of Sampling & Testing*, Vol. II, Am. Assoc. State Highway Officials, Washington, 1947.
23. Walker, S., "Soundness of Aggregates," *Report on Significance of Tests*, A.S.T.M., 1943.
24. Wood, W. H., "Significance of the Los Angeles Abrasion Tests as the Measure of the Service Value of Coarse Aggregates," *Report*, Texas State Highway Dept., Austin, Texas, 1938.
25. Woolf, D. O., "Correlation of Los Angeles Tests of Aggregates with Service Behavior," *Proceedings*, Highway Research, Board, 1937.
26. Woolf, D. O. and Runner, D. G., "The Los Angeles Abrasion Machine for Determining the Quality of Coarse Aggregates," *Proceedings* Am. Soc. for Testing Mat., Vol. 35, 1935.
27. Woolf, D. O., "Results of Cooperative Tests Using the Los Angeles Abrasion Machine," *Proceedings*, Highway Research Board, Vol. 16, 1936.

## DISCUSSION

D. O. Woolf, *Senior Materials Engineer, Public Roads Administration*—There are quite a number of items in Mr. Melville's report which warrant discussion. These include considerations of the standardization of the proposed test method, and of the reasons for the behavior of the rock types which were found to give poor results. In connection with the use of a method of test involving the abrasion of rock in the presence of water, mention should be made of the "wet shot rattler test" used by the California Division of Highways in the selection of coarse aggregates for waterbound, bituminous, or portland cement concrete pavements or bases.

The reported behavior of the stone used on U. S. Route 250 brings to mind another investigation which involved considerations applicable to the present subject. In 1916, Dr. E. C. E. Lord[1] prepared a report of the effect of the mineral composition on the physical properties of rock, with particular attention to the loss in the Deval abrasion test, the hardness and toughness, the cementing value, and the slaking of rock powder.

In Table 8 of this report, Dr. Lord presented a summary of a large number of samples of rock showing the average mineral composition and the slaking properties of the powdered rock. Two entries in this table are of particular interest. In a study of basic crystalline rocks, Dr. Lord compared 83 samples which did not slake with 48 samples which did slake in water. The nonslaking rocks contained 89 percent of primary, that is, unaltered, minerals, including 35 percent of plagioclase feldspar and 32 percent of augite. The rocks which did slake contained only 69 percent of primary minerals, including 34 percent

[1] "Relation of Mineral Composition and Rock Structure to the Physical Properties of Road Materials," by E. C. E. Lord, U. S. Department of Agriculture Bulletin No. 348, April 4, 1916.

of feldspar and 15 percent of augite. It should be noted that in these two groups of rock the feldspar content is almost the same but the rocks which slaked have less than one-half of the augite contained by the nonslaking rocks. The rocks which slaked contained appreciable percentages of kaolin, chlorite, epidote, calcite, and limonite, all of which are secondary minerals produced by alteration. Wahlstron[2] states that augite alters to hornblende, epidote, chlorite, antigorite, calcite, diopside, and biotite.

From the above, it is believed that the rock identified by Mr. Melville as pyroxenite is a highly weathered material. Suspicion of questionable material for use in bituminous construction should be directed not to rocks containing varieties of pyroxene, but to rocks containing altered or weathered minerals of this type.

Over 30 years ago, Dr. Lord showed that certain types of rock might be unsuitable for use in highway construction. He was primarily concerned with waterbound macadam and the significance of his remarks apparently has been forgotten. In calling the attention of the highway engineer to the undesirable service record of highly weathered rocks containing appreciable quantities of ferromagnesian minerals, Mr. Melville has made a notable contribution to the study of the durability of bituminous pavements.

Phillip L. Melville, *Closure*—The writer appreciates the interest shown by Mr. D. O. Woolf in the preceding paper. It is indeed true that several attempts have been made in various states at creating wet abrasion tests. Yet such tests have found little universalization. Though they appear to have a very sound and logical cause for their existence, their popularity is somewhat restricted if judged by the amount of published data on the subject. Mr. Woolf's recall of Dr. Lord's excellent work is very timely. Yet the author is inclined to believe that such a theoretical approach on the identification of potentially deleterious rocks, as enlightening as it may be, is better suited for use by well-trained geologists or laboratory technicans than by field engineers. The need for a test which would permit the rejection of potentially deleterious rocks remains if we want to avoid most of the human element and the always arguable results of a so-called visual "inspection".

As far as the possibility that the rock identified as pyroxenite be a highly weathered material (sic), the author is inclined to disagree with Mr. Woolf. Lacking a detailed mineralogical analysis it is impossible to state the amount of weathering to which the rock under discussion has been subjected since it was formed millions of years ago. Yet when a fresh sample was carefully selected at the quarry site for the investigation reported in this paper, everything led one to believe that it was practically non-weathered. Since the same could not be stated as far as the aggregate found in the surface treatment was concerned, suspicion might not only be directed against rocks "containing altered or weathered" varieties of pyroxene but also against minerals which will be altered fast enough to form a deleterious road aggregate.

The writer agrees that it is unfortunate that Dr. Lord's publications have somewhat fallen into oblivion and that some of the rather old and hard-to-get government publications do not receive the interest which is sometimes due to them. But if the author has done some work which may overlap the outstanding investigations of Dr. Lord, he feels indeed fortunate to find himself in such a worth-while company. As the great thinker La Bruyère once said (and the significance of his remarks has certainly not been forgotten): *"Tout est dit, et l'on vient trop tard depuis plus de sept mille ans qu'il y a des hommes, et qui pensent."* ("Everything has been said and one comes too late after more than 7000 years that there have been men who have been thinking." de La Bruyère, (1645–1696) *Les Caractères*)

[2] "Igneous Minerals and Rocks," by Ernest E. Wahlstrom, John Wiley and Sons, Inc., 1947.