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THE APPLICATION OF GEOLOGY IN THE BENEFICATION OF AGGREGATES

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The demand for suitable aggregates for use in construction is ever increasing. In highway construction alone, it has been estimated that 750 million tons are being required annually. Along with the increased need, suitable sources are rapidly being depleted and, in the case of the "urban sprawl," zoned out of existence. To meet these demands, highway engineers are seeking new sources and the better use of existing sources.

Efforts to provide adequate quantities of aggregates have taken many directions. These include materials surveys for locating and identifying new sources, and the upgrading of existing sources. New sources, such as synthetic aggregates, are being considered and, in some instances, used. Improved exploration techniques and recent developments in aerial photography are aiding in material surveys. At the same time, stabilizing additives and better identification of aggregate particles are aiding in upgrading existing sources.

Although beneficiation is a term more frequently applied to the removal of unwanted fractions by such methods as heavy media separation, jigging, rising current classification, and elastic fractionation, it also may be applied to the improvement of aggregates through a better understanding of their characteristics. The latter viewpoint is expressed in this paper.

The selection of an aggregate source and the suitability of the aggregates depend on the task the aggregates must perform. After the tasks have been clearly identified, suitability may be determined by service records, in the case of existing sources, and laboratory tests where new sources are being considered.

The basis for evaluating aggregate suitability where adequate service records do not exist is related to the aggregate properties and in-service tasks. The properties are encompassed in physical and chemical characteristics, and grading. An understanding of these properties and techniques for their evaluation are essential to the selection and the assignment of an aggregate for a particular use. It must be recognized that a "perfect" aggregate to meet demands under all situations does not exist, so it becomes a matter of establishing a desirable performance level and finding an aggregate suitable for that level. To require aggregate properties beyond those needed for a particular performance level is uneconomical.

Although materials engineers frequently refer to aggregate as an inert material in concrete and bituminous mixes, it is becoming recognized that they do have "active" properties and are, in fact, an important constituent of the mixture. In judging the "worth" of an aggregate, empirical means, such as the Los Angeles abrasion and the sodium sulfate soundness tests, are frequently used. Yet these tests fail to represent any in-service condition, and in fact provide little information about aggregate characteristics. For aggregate selection and beneficiation, it istics. For aggregate selection and beneficiation, it is essential to understand their physical and chemical properties.

The role the geologist can play in aggregate selection and beneficiation is readily apparent from a listing of physical and chemical properties such as given in Table I. This becomes obvious when one also recognizes that "all of the mineralogic, physical, and chemical properties of rocks, including those important to the suitability of aggregate, stem from (1) their geologic conditions of orgin and (2) from the later processes of weathering and alteration to which they have been subjected during their geologic history" (*1*). The adoption by the American Society for Testing and Materials of such standards as ASTM C294 on "Descriptive Nomenclature of Constituents of Natural Mineral Aggregates" and C295 on "Petrographic Examination of Aggregates

*The opinions and conclusions expressed or implied are those of the Author. They are not necessarily those of the Highway Research Board, the National Academy of Sciences, the Bureau of Public Roads, the American Association of State Highway Officials, nor of the states participating in the Program.

for Concrete" further serves to point to the geologist's role. In recent years petrographic and geologic know-how has been brought to bear on highway problems and significant contributions have been made to provide a better understanding of aggregate constituents and their influence on degradation, soundness, alkali activity and skid resistance (*2, 3, 4*).

The importance of geology in aggregate beneficiation was recognized at the outset of the National Cooperative Highway Research Program. One of the initial six areas of research was on aggregate beneficiation. This area was allocated $250,000 to contract research on the "benefication of poor aggregate and the manufacturing of suitable aggregate from local materials including the development of new and improved stabilizing agents.

The Program's Advisory Panel divided these funds to support projects deemed most pressing. They include:

Project 4-1, Development of Appropriate Methods for Evaluating the Effectiveness of Stabilizing Agents

Project 4-2, A Study of Degrading Aggregates in Bases and Subbases with Production of Excessive Amounts of and/or Harmful Types of Fines

Project 4-3(1), Development of Methods to Identify Aggregate Particles Which Undergo Destructive Volume Changes When Frozen in Concrete

Project 4-4, Synthetic Aggregates for Highway Uses

Project 4-5, A Study of the Mechanism Whereby the Strength of Bases and Subbases is Affected by Moisture and Frost

In analyzing these problems from the viewpoint of subject matter, it may be observed that volume change, degradation, frost and moisture, and stabilizing agencies, were considered major areas of need. The importance of geopogy is recognized in at least four of the six studies.

As further support of the thesis that geology does have a role in aggregate beneficiation, a brief description of four of the above NCHRP projects and their status follows:

The investigation of synthetic aggregates, conducted at Battelle Memorial Institute, has been completed (*5*). Its objectives were directed towards exploring the feasibility of utilizing artificial aggregates. The study was, in effect, a state-of-the-art report on the existence or potential existence of synthetic aggregates and their potential uses. Sources of pertinent information found in geologic literature aided materially in the conduct of the study.

Among other things, the selective use of materials on the basis of their contribution to the layered structure of the roadway is suggested as a possible means of obtaining maximum economic utilization of construction aggregates. For example, high-quality aggregates are needed in surface mixtures, whereas materials of lower quality may be used in base and subbase work. At present, economic considerations restrict the use of many potential sources of synthetic materials other than those existing for lightweight aggregates.

The research on degrading aggregates (*6*) continues at Purdue University. It is expected that this study will show not only the mechanism of degradation, but also what aggregate properties to measure to predict degradation. The researchers, after reviewing the history of aggregate degradation and reassessing the problem, concluded that the degradation problem was widespread geographically and that numerous rock types were affected.

In the research, the Los Angeles abrasion test is being used to characterize aggregate degrading. Laboratory tests include differential thermal (DTA) analysis, X-ray diffraction, insoluble residue, and petrographic analysis. The petrographic work has been by examination of hand-picked specimens, polished sections, and thin sections. In the case of thin sections, the following characteristics were observed:

1. Grain size
2. Grain size distribution
3. Grain interlock
4. Void characteristics
5. Weathering

The research is continuing and the final report is due in July 1965.

Two studies just completed relied heavily on petrographic examination for aggregate identification and classification. This sections as well as polished sections were studied. These were the projects at Virginia Polytechnic Institute headed by Dr. Richard Walker (*7*) and the one at Pennsylvania State University under the direction of Dr. Thomas Larson (*8*). Both projects were assigned the same objective of developing a quick method of test to distinguish deleterious particles and predict their behavior under various degrees of exposure in concrete subjected to freezing and thawing.

The research approach by Dr. Walker at VPI utilized a variety of coarse aggregates ranging from traprock and limestone to a variety of gravels. Concretes were made from the whole coarse aggregates as well as from their constituents. The specimens were exposed to alternate cycles of freezing and thawing and at the end of selected cycles were measured for length, weight, and dynamic modulus. A rather detailed petrographic description of each type aggregate and, in some instances, individual particles was reported. Dr. Walker noted that an experienced petrographer was used in the study.

In the Penn State study, Dr. Larson reported that the research included evaluation of pore characteristics, aggregate particle expansion, petrographic examination, and the Powers freeze-thaw test. In summarizing the petrographic studies, it was noted that the analysis explained and predicted the behavior of the test aggregates in freeze-thaw testing, but that petrography alone can not isolate all possible effects of the several potentially deleterious features in most aggregates.

Although only four of the six NCHRP studies assigned by the panel on aggregate beneficiation are herein discussed, the studies at the University of Illinois and Michigan Technological University pertain to aggregate beneficiation, but geology plays only a minor role in the research.

Another area of importance to the highway and materials engineer in which geologists can assist is that of skid resistance. It is apparent that types of aggregates and their mineralogical constituents play a major role in pavement slipperiness. Research has pointed to the influence of geologically different limestones on skid resistance (*9*). Petrographic studies coupled with laboratory and field testing of concrete and bituminous mixtures may eventually lead to selected use of limestone in surface mixtures.

Probably the greatest influence of geologic techniques has been in the studies of alkali-aggregate reaction. The petrographer has isolated such minerals as opal, chalcedony, and different types of rocks and gravels that react with cement alkalies to cause concrete deterioration. Rhoades and Mielenz (*1*) noted that: "The physical and chemical properties of particles critical to the suitability of aggregate arise from their petrographic and mineralogic composition, texture, and internal structure. Therefore, the serviceability of aggregate under anticipated conditions can be predicted accurately only if the petrographic and mineralogic characteristics are known and evaluated."

In summary, the geologist has an important part in the beneficiation of aggregates in highway construction. His greatest contribution can be through providing a better understanding of the physical and chemical properties as they relate to the aggregates desired performance.

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TABLE I
Aggregate Properties

Physical	*Chemical*
Strength	Solubility
Elasticity	Stability
Hardness	Alkali resistance
Toughness	Organic impurities
Volume stability	Combustible and Volatile material
Porosity	Coal and lignite
Permeability	
Absorption	
Density	
Wear resistance	
Soundness	
Thermal	
Soft fragments	
Silt and clay	
Gradation	

FREEZE-THAW CHARACTERISTICS OF AGGREGATES

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INTRODUCTION

Premature deterioration of concrete under freezing and thawing conditions is often attributable to the aggregate fraction. Past research has shown that the freeze-thaw characteristics of aggregate are related, in a general way, to such properties as: (*1*) porosity, (*2*) absorption, and (*3*) bulk specific gravity. Actually it is the pore system—that is, the size, shape, arrangement and continuity of the pores—that governs the freeze-thaw characteristics. Distress in aggregate particles arises from hydrostatic pressure induced when a portion of its absorbed pore-water is frozen. The degree of distress or damage manifested is dependent upon the amount of permeable porosity, the degree of saturation, the severity of freezing, and the rupture strength of the rock particle and the restraint imposed upon it.

Porosity, an index to the pore system, is expressed as the ratio of the void or pore volume to the total volume. Absorption, an index to the pore system, is expressed as a ratio of the weight of absorbed water to the weight of the solid component. Since this ratio is dependent upon the specific gravity of the solid component, it may be quite variable. Bulk specific gravity, another index to the pore system, is also quite variable since it is dependent upon the specific gravity of the solid component. It is evident, of course, that porosity is an independent parameter. If absorption, which is easily determined, were expressed as the ratio of volume of water to the total bulk volume of the rock particle, then this index would also be independent of other parameters.

The exact limits that should be placed on these physical properties to control aggregate quality remains controversial. This is due not only to the variables just mentioned but also to the methods used for determining aggregate durability. Present methods employ composite samples for tests; and, therefore, the results of such tests are composite or average values. For example, the average value of absorption obtained from a composite sample may exhibit a low value—indicating a sound aggregate; however, if each particle were analyzed, it might be found that a portion of the aggregate is so highly absorptive as to be detrimental to concrete. It is the percentage of these deleterious particles in aggregate that is so important.

A second factor in freeze-thaw testing is the antecedent moisture condition of the aggregate—which heretofore has not been duly considered. For instance, a sample of aggregate which is in a highly saturated condition in its natural environment or a stockpile may be oven-dried in preparation for laboratory freeze-thaw testing; many of the standard procedures require that this be done routinely. Even subsequent re-soaking often fails to restore the original moisture condition; and, since aggregate must be critically saturated to be vulnerable to damage, the duration of such freeze-thaw testing may merely reflect the time required for the aggregate to become critically saturated. Ideally, in such testing, consideration should be given to: (1) from a given antecedent moisture condition, the time required for the aggregate to acquire critical saturation in the environment to be imposed and, (2) once critically saturated, whether the aggregate can withstand the stresses accompanying freezing.

Heretofore, the discrete conditions previously mentioned have not been compensated for by methods of test in determining aggregate soundness. Logically, in determining the soundness of an aggregate sample, the freeze-thaw testing should be conducted on a per particle basis. Each particle should be saturated at the onset of test and kept saturated during testing. For study purposes the degree of saturation may be varied. However, maximum saturation definitely establishes the ultimate susceptibility of aggregate to damage from freezing and thawing.

In order to obtain objective data pertaining to the freeze-thaw characteristics of aggregate—that is, to establish more definitive relationships between the effects of freezing and thawing of an aggregate and its physical properties of porosity, absorption and bulk specific gravity—a method of test was devised whereby the discrete conditions previously mentioned were fulfilled. A freezing medium was sought, whereby each particle could be frozen quickly. By doing this, the quickly frozen surface would form a seal or shell about the particle and thus retain the pore water. Also, the medium should not contaminate the pore water. Chilled mercury was chosen as the freezing medium—it has high thermal conductivity; it is nonmiscible with water; and it has a low freezing-point. The test consisted of submerging the aggregate particle in the prechilled mercury. From preliminary testing, it was found that if a particle did not show visual distress at the end of four cycles—which could be performed in a matter of minutes—it would withstand innumerable cycles. For analysis, it was desired to have a saturated gravel which would represent a variety of rock types and possess a wide range of physical properties. To meet these conditions, a sample was secured from a glacial outwash deposit and kept inundated.

MATERIALS AND PROCEDURES

The primary constituents of the aggregate sample were dolomites, cherts, limestones, sandstones, siltstones, and various igneous and metamorphic rock particles. Although usually acceptable for concrete, this gravel contained both sound and unsound particles and, thereby, provided an assortment of particles needed for the study.

The sample was graded into the following sieve sizes:

Passing 1-1/2-inch and retained on 1-inch,
Passing 1-inch and retained on 3/4-inch,
Passing 3/4-inch and retained on 1/2-inch,
Passing 1/2-inch and retained on 3/8-inch, and
Passing 3/8-inch and retained on No. 4.

After grading according to size, each particle was numbered. The saturated surface-dry weight and bulk volume were obtained for each particle prior to the freeze-and-thaw tests. The weight was obtained by weighing each surface-dry particle in a capsule, and the volume was obtained by weighing the amount of mercury displacement.

Calculations for the bulk specific gravity were made for each particle by the use of the following expression:

$$G_{SSD} = \frac{W_T \ \gamma_M}{W_M \ \gamma_W}$$

where:

G_{SSD} = bulk specific gravity (saturated surface-dry),
W_T = saturated surface-dry weight of particle,
γ_M = density of mercury at temperature of test,
W_M = weight of mercury displaced by particle, and
γ_W = density of water at temperature of test.

The freezing of the aggregate was accomplished by submerging each particle in cold mercury (-30 C to -35 C) for a period of five minutes. The freezing apparatus is shown in Figure 1. The mercury was contained in a glass cylinder enclosed in an insulated container of dry ice (solid CO2). A low-temperature thermometer was used to observe the temperature of the mercury.

At the end of each freezing cycle, each particle was removed from the freezing medium and returned to a container of water and allowed to thaw. At the end of each thawing cycle, a visual examination was made of each particle; and any distress resulting from the freezing and thawing cycle was recorded.

For obvious reasons the absorptive values for each particle were not determined until the conclusion of freeze-and-thaw testing. Also, since any losses due to chipping and spalling would vary the weight of the particle, the saturated surface-dry weight of each particle (or major pieces thereof) was determined for the second time. Each particle (or major piece) was then dried (110 C) to a constant weight.

Calculations for the absorption of each particle were made by the use of the following expression:

$$\omega = \frac{W_T - W_S}{W_S} \times 100$$

where:

ω = absorption expressed as a per cent,
W_T = saturated surface-dry weight of particle (or major pieces), and
W_S = oven-dry weight of particle (or major pieces).

Porosity calculations were made by the use of the following expression for saturated aggregate:

$$\eta = \frac{\omega}{100 + \omega} G_{SSD} \times 100$$

where:

η = porosity expressed as a percent,
ω = absorption expressed as a percent, and
G_{SSD} = bulk specific gravity (saturated surface-dry).

It should be emphasized that the above equation applies only for saturated aggregate.

After all other testing was completed, a fresh surface of each particle was examined by use of binocular and petrographic microscopes. Where necessary, the petrographic examination was supplemented by chemical tests. Mineralogical and textural features were recorded.

RESULTS

The accumulated number of fractured particles at the end of each freeze-thaw cycle, expressed as a percentage of the total number of particles, is presented graphically in Figure 2. The slope of the curve indicates the increase in the percentage of fractured particles at the end of each cycle; and, as would be expected, the largest increase occurred at the end of the first cycle. Of the total number fractured, approximately 70 per cent occurred during the first freezing cycle. The small increases in the accumulated percentage of fractured particles which occurred with succeeding cycles are probably due to the fact that many of the particles had undetected fractures at the end of the first freezing cycle and additional cycles were needed before the fractures became visible. The condition of the particles at the end of the fourth cycle of freeze-and-thaw testing was used for correlation.

The percentage of fractured particles at the end of the fourth cycle of freeze-and-thaw, for each particle size tested, is given in Figure 3. The largest percentage of fractured particles occurred in the 1/2-inch size; whereas, the smallest percentage of fractured particles occurred in the 1-inch and No. 4 sizes. The average percentage of fractured particles in all sizes combined is indicated by the dashed line, at 53.6 percent. The 1-inch size particles consisted mostly of igneous and metamorphic rock particles which, by their mode of formation, are less porous than the water-lain sedimentary rocks. The bulk of the 1/2-inch particles consisted of porous cherts and porous dolomites. The bulk of the No. 4 size particles were mostly quartz.

The relationship between the soundness of the test particles and their adjusted porosity values is shown in Figure 4. As expected, particles in the higher porosity range were less durable than particles of the lower range. All particles having a porosity of more than 11 per cent fractured; whereas, less than 25 per cent of the particles having a porosity of less than 2 per cent fractured.

A graph depicting the relationship between the percentage of fractured particles and the absorption values of the test particles is shown in Figure 5. All particles having absorptions of 4 per cent or greater failed when subjected to freezing and thawing. However, very few specimens having absorptions of less than 1 per cent failed. Between these extremes, the percentage of particles failing increased as the absorption increased.

The percentage of fractured particles at the end of 4 cycles of freeze-and-thaw for different bulk specific gravity values is shown in Figure 6. A definite trend is established, indicating that aggregate in the lower specific gravity ranges has lower resistance to freezing and thawing than aggregate of the higher specific gravity ranges. However, no specific gravity level can be classed as critical—except the level below 2.40. The increase in fractured particles in the heavier bulk specific gravity range is due to highly porous, highly absorptive dolomites. For comparison, suggested curves for cherts and dolomites are included.

The durability of the various types of aggregate particles is presented in Table 1. Sedimentary rock particles had a much larger percentage of fractures than the igneous and metamorphic particles. Within the sedimentary classification, limestone contained the least percentage of fractured particles, whereas dolomite contained the greatest.

It is interesting to note that chert, which is usually thought of as being very unsound under freeze-and-thaw action, did not have the greatest percentage of fractured particles. Since porous, absorptive chert and porous, absorptive dolomites may be similar in appearance, it is possible that, in the past, some concrete deteriorations which were thought to be due to unsound chert were, in reality, due to unsound dolomite.

DISCUSSION

Porosity, Absorption and Bulk Specific Gravity

Figure 4 and Figure 5, depicting relationships between porosity and per cent fractured particles and between absorption and per cent fractured particles are similar. This is because calculations are based on a two-phase system—i.e., water and solids. This being the case, porosity may be related directly to absorption. With a given porosity and assuming that all voids permeable, the corresponding saturated absorption value may be computed by assuming a value for apparent specific gravity. Figure 7 shows such a relationship. These absorption values were calculated by taking the specific gravity of the solid components to be 2.65 and 2.94.

If critical porosity is taken to be two per cent, critical absorption could range from 0.695 for chert having a specific gravity of 2.65; to 0.77 for dolomite having a specific gravity of 2.94.

Bulk specific gravity and its apparent relationship to the percentage of fractured particles may or may not reflect the true durability of an aggregate. Even though all natural aggregate having bulk specific gravities below 2.40 are normally considered to be deleterious, aggregates in the heavy bulk specific gravity ranges may be just as deleterious. This is well illustrated by the dolomites shown in Figure 6. Moreover, in quarrying a limestone deposit which has a large proportion of granoblastic dolomite facies, the bulk specific gravity of the highly porous, highly absorptive dolomite would correspond with the bulk specific gravity of the high quality limestone facies.

Theoretical Considerations of Freezing Effects

The ability of a rock fragment to withstand internal pressures accompanying freezing is controlled by certain inherent properties of the fragment. Insight into these properties is gained through Timoshenko's (23) explanation of Lame's solution for the radial and tangential normal stresses in a thick-walled, spherical container under internal and external pressures is considered (Figure 8). According to Timoshenko, the radial normal stress is given by the expression:

$$\sigma_R = \frac{P_o b^3 (R^3 - a^3)}{R^3 (a^3 - b^3)} + \frac{P_i a^3 (b^3 - R^3)}{R^3 (a^3 - b^3)}$$

and the tangential normal stress is obtained by the equation:

$$\sigma_T = \frac{P_o b^3 (2R^3 + a^3)}{2R^3 (a^3 - b^3)} - \frac{P_i a^3 (2R^3 + b^3)}{2R^3 (a^3 - b^3)}$$

where:

a = inner radius of the sphere,

b = outer radius of the sphere,

P_i = the internal pressure, and

P_o = the external pressure.

If the exterior confining pressure (P_o) is zero, the equations for the normal stresses at the extreme outer fiber are as follows:

$$\sigma_R = 0$$

$$\sigma_T = - \frac{3P_i a^3}{2 (a^3 - b^3)}$$

and by multiplying and dividing the latter equation by $4/3\ \pi$ and substituting:

$$V_v = 4/3\ \pi\ a^3,$$

$$V_t = 4/3\ \pi\ b^3, \text{ and}$$

$$V_v = \eta\ V_t$$

where:

V_v = volume of voids,

V_t = volume of sphere, and

η = porosity as defined above,

the equation reduces to

$$\sigma_T = 3/2\ P_i \frac{\eta}{1 - \eta}$$

The radial normal stress, σ_R, along the outer edge of the spherical container is zero and the tangential normal stress, σ_T, at the same point, which is a tensile stress, is dependent upon the porosity and tensile strength of the container and is independent of size or total volume.

If the tensile strength, σ_u, is inserted in the above equation for σ_T, the equation becomes

$$P_u = 2/3\ \sigma_u \frac{(1 - \eta)}{\eta}$$

P_u = maximum allowable internal pressure, and

σ_u = tensile strength.

Tensile strengths of the gravel particles were not determined, but similar materials have tensile strengths ranging from 100 to 1000 psi (8). With a known tensile strength, the maximum allowable internal pressure for various porosity values may be calculated. Three porosity-pressure curves for tensile strengths of 300, 600, and 900 psi are shown in Figure 9.

The determination of the internal pressure accompanying freezing of water within a hollow sphere may, likewise, be approached for a theoretical standpoint. If the temperature-volume changes of the sphere, water, and ice are neglected, the volume of ice within the cavity, assuming all the water freezes, is as follows:

$$V_i = 4/3\ \pi\ a^3 S (1+\beta)(1-\frac{P_a}{K})$$

where:

V_i = volume of ice,

S = degree of saturation,

β = volume increase accompanying freezing of water,

P_a = available internal pressure, and

K = bulk modulus of ice.

The tangential strain at any point within the sphere is given by the expression:

$$\varepsilon_T = \frac{\delta}{R} = \frac{\sigma_T}{E} - \mu\frac{\sigma_T}{E} - \mu\frac{\sigma_R}{E}$$

where:

ε_T = tangential strain,

δ = radial displacement,

E = Young's modulus of elasticity, and

μ = Poisson's ratio.

Assuming $P_o = 0$, the increase in the radius of the internal cavity or void due to an internal pressure, P_a, is

$$\delta_a = \frac{(4\mu - 2)\ a^3 - (1+\mu)\ b^3}{2E\ (a^3 - b^3)} \cdot aP_a$$

and, by multiplying and dividing the latter equation by $4/3\ \pi$ and substituting,

$$V_v = 4/3\pi a^3 = nV_t, \text{ and}$$
$$V_t = 4/3\pi b^3,$$

the equation reduces to

$$\delta_a = \frac{1+\mu+2n-4\mu n}{2E(1-n)} \cdot aP_a$$

The volume of the cavity, V_c, under internal pressure will, thus, be

$$V_c = 4/3\pi\ (a + \delta_a)^3$$

or

$$V_c = 4/3\pi\ a^3(C^3P_a{}^3 + 3C^2P_a{}^2 + 3C\ P_a + 1)$$

where:

$$C = \frac{1 + \mu + 2n - 4\mu n}{2E(1 - n)}$$

The volume of the ice within the cavity must be equal to the volume of the cavity and by equating the two, it follows that:

$$4/3\pi a^3 S(1 + \beta)(1 - \frac{P_a}{K}) =$$
$$4/3\pi a^3(C^3P_a{}^3 + 3C^2P_a{}^2 + 3C\ Pa + 1)$$

or

$$P_a{}^3 + \frac{3}{C}P_a{}^2 + \frac{3CK + S + S\beta}{C^3\ K}P_a + \frac{1 - S - S\beta}{C^3} = 0$$

From the above relationship, it is seen that the respective dimensions disappear and that the internal pressure created by the freezing of water within a closed, spherical container is dependent upon the volume increase accompanying freezing of water, the bulk modulus of ice, and the porosity, degree of saturation, Poisson's ration, and modulus of elasticity of the sphere.

The available internal pressure, Pa, was calculated for various porosity values by selecting, from the indicated references, the following values:

$$\mu = 0.25\ (8),$$
$$E = 2 \times 10^6 \text{ psi } (8),$$
$$K = 0.392 \times 10^6 \text{ psi } (3),$$
$$S = 1.00, \text{ and}$$
$$\beta = 0.09\ (3).$$

This plot is superimposed on the maximum allowable internal pressure curves in Figure 9.

The intercept between the available-pressure curve and the appropriate allowable-pressure cure is the maximum allowable porosity value. For porosity values greater than this, the available pressure is greater than the allowable pressure and the container will rupture. Conversely, for porosity values less than this, the available pressure is less than the allowable pressure, and the container will withstand the pressure.

Critical porosity may be calculated by another method. An equation (10) expressing the force generated when particles are undergoing freezing, shows that the force developed is proportional to pore pressure, porosity and the effective spherical area encased within the ice shell. The equation is

$$F = P_p \times n \times a$$

where:

F = force in pounds,

P_p = pore pressure in psi,

n = porosity expressed as a decimal, and

a = effective area in $in.^2$

By arranging the equation so the porosity is the dependent variable, taking the maximum value for pore pressure (30,000 psi, -7.6 F), assuming a force of 600 pounds per unit area to equal the flexural strength of concrete, and taking the area of the sphere encased by the ice shell as unity, then the critical porosity is two per cent. If the flexural strength of the concrete is greater than 600 psi, then critical porosity increases; if the flexural strength is lower, then critical porosity decreases.

Applicability of Theory

The values of critical porosity obtained from the test data correspond closely to the theoretical values. It appears that the range over which critical porosity occurs is a manifestation of the inherent tensile strength of the aggregate. Whereas it must be presumed that tensile strength varies in inverse proportion to porosity, the inherent tensile strength varies within a wide range depending on the mineralogical nature of the aggregate. Hence, the force opposing the expansion accompanying freezing—and therefore the limiting pressure—is governed by the restraining strength of the aggregate or the confining vessel. Logically, any encasement medium—such as mortar or concrete—will provide additional restraint against the forces emanating from a dilating particle of aggregate. Thus, restraint increases with dept of embedment. A saturated particle of aggregate in concrete may be viewed as being a center of compression, and the surrounding concrete may be viewed as a thick-walled shell or vessel. Of course, at near-surface locations, the restraint is unbalanced; and "pop-outs" or cracking will result if the dilating pressures are critical.

The pressures accompanying confined freezing cause an attendant depression of the freezing-point of the water—approximately 1750 psi per degree centigrade. Therefore, if the freezing-point of the water in an aggregate particle or in a cavity in concrete is monitored during a cooling cycle, the attendant active pressures may be deduced. To illustrate this mechanism, a small capsule of water was cast into a concrete block at a specific distance from the surface; a thermocouple was placed at the center of the sphere of water; and the block was frozen. A "pop-out" resulted as shown in Figure 10. A typical thermogram is shown in Figure 11. There, the temperature at rupture is seen to be -4.2 degrees centigrade (7200 psi). Following rupture, the pressure subsided; and the freezing-point returned to normal. Thereafter the freezing was isothermal. This same type of thermogram has been obtained previously (4) by embedding thermocouples at the center of 6-in. by 6-in. cubes of concrete, but the events apparent in Figure 11 were displayed over a number of successive cycles rather than within a single temperature-excursion. This was interpreted to be a manifestation of progressive damage.

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TABLE 1
DURABILITY OF AGGREGATE
ACCORDING TO CLASSIFICATION

Classification	Per Cent Fractured Particles at End of 4 Cycles of Freeze-and-Thaw
(1) Sedimentary	66
(a) Dolomite	77
(b) Chert	59
(c) Limestone	41
(d) Sandstone & Siltstone	69
(2) Igneous and Metamorphic	23

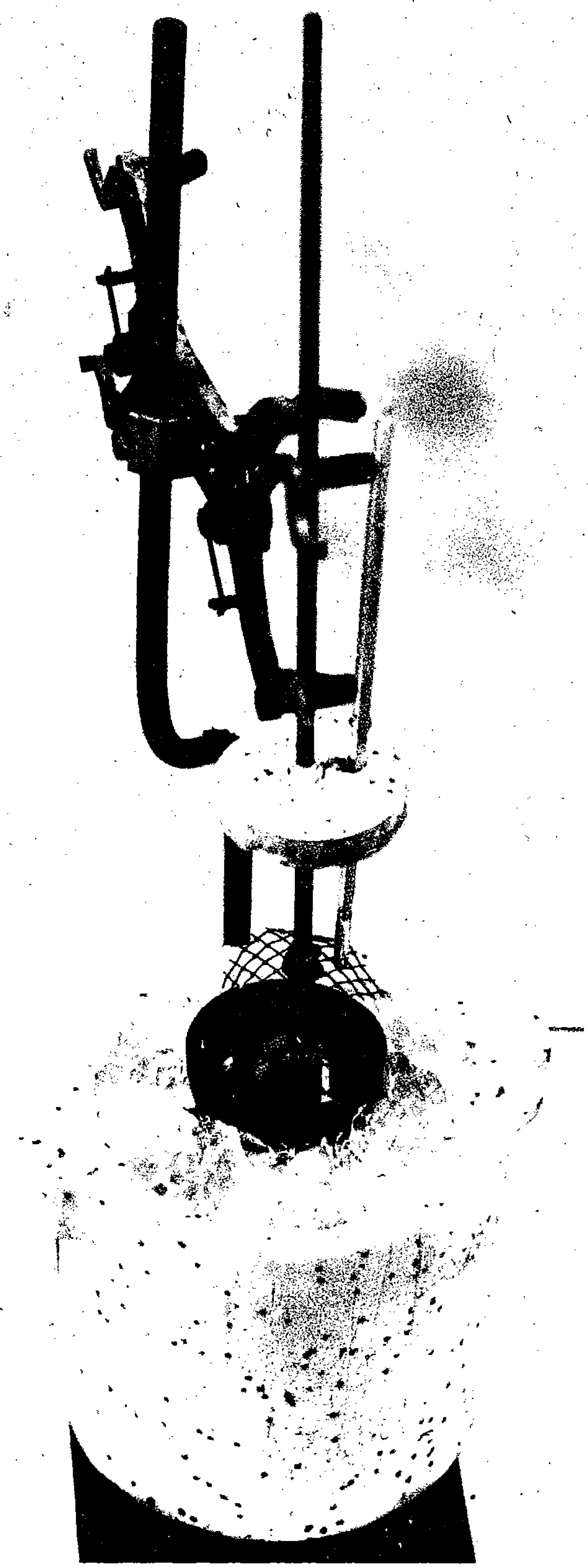

Fig. 1.— Freezing Apparatus.

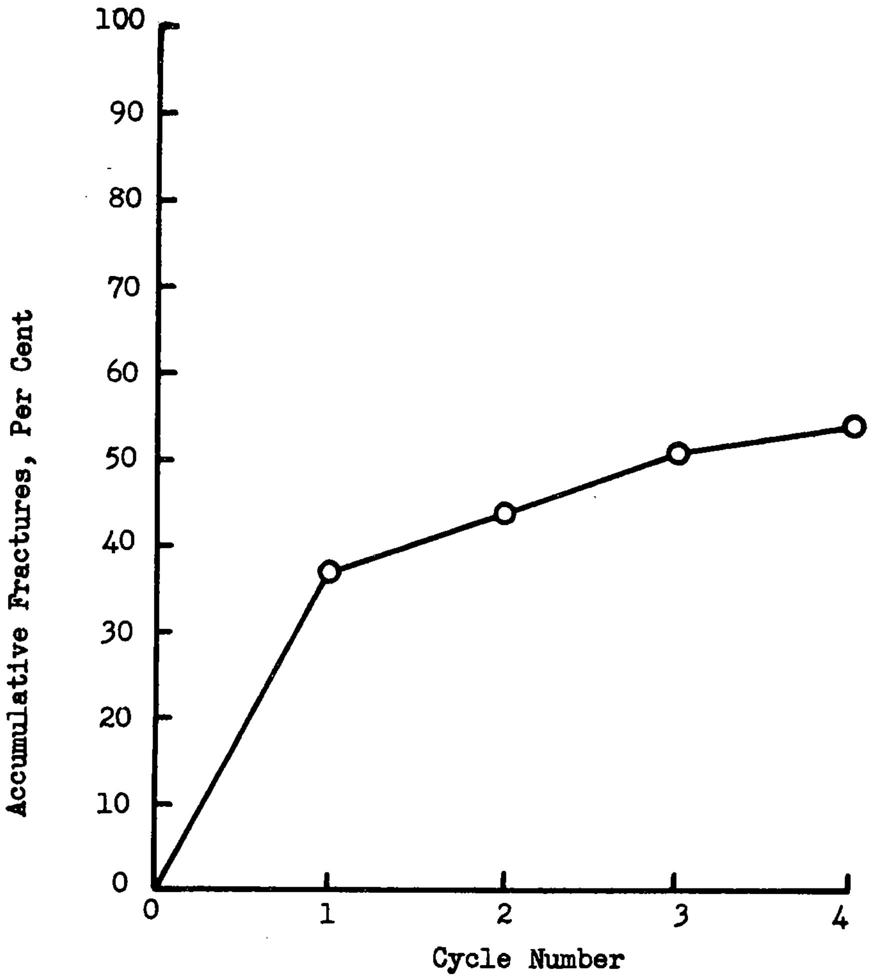

Fig. 2.— Accumulative Percentage of Fractured Particles for Each Cycle of Freeze-and-Thaw.

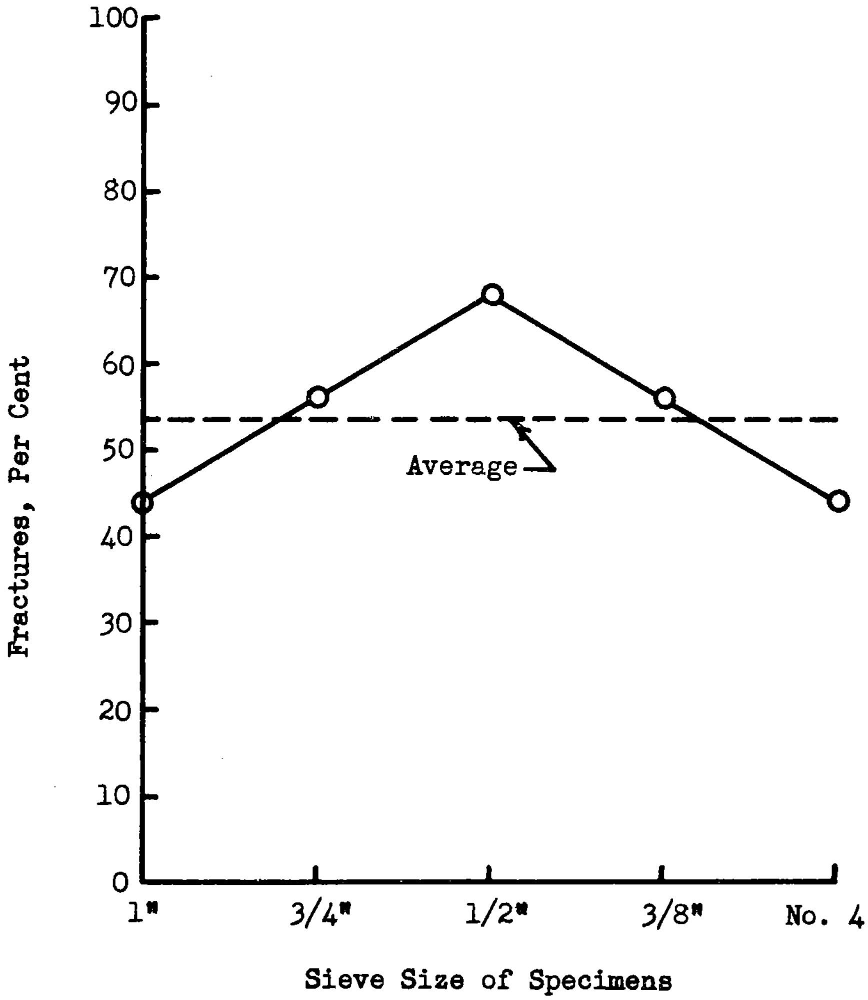

Fig. 3.— Relationship between Particle Size and Percentage of Fractured Particles after Exposure to 4 Cycles of Freeze-and-Thaw.

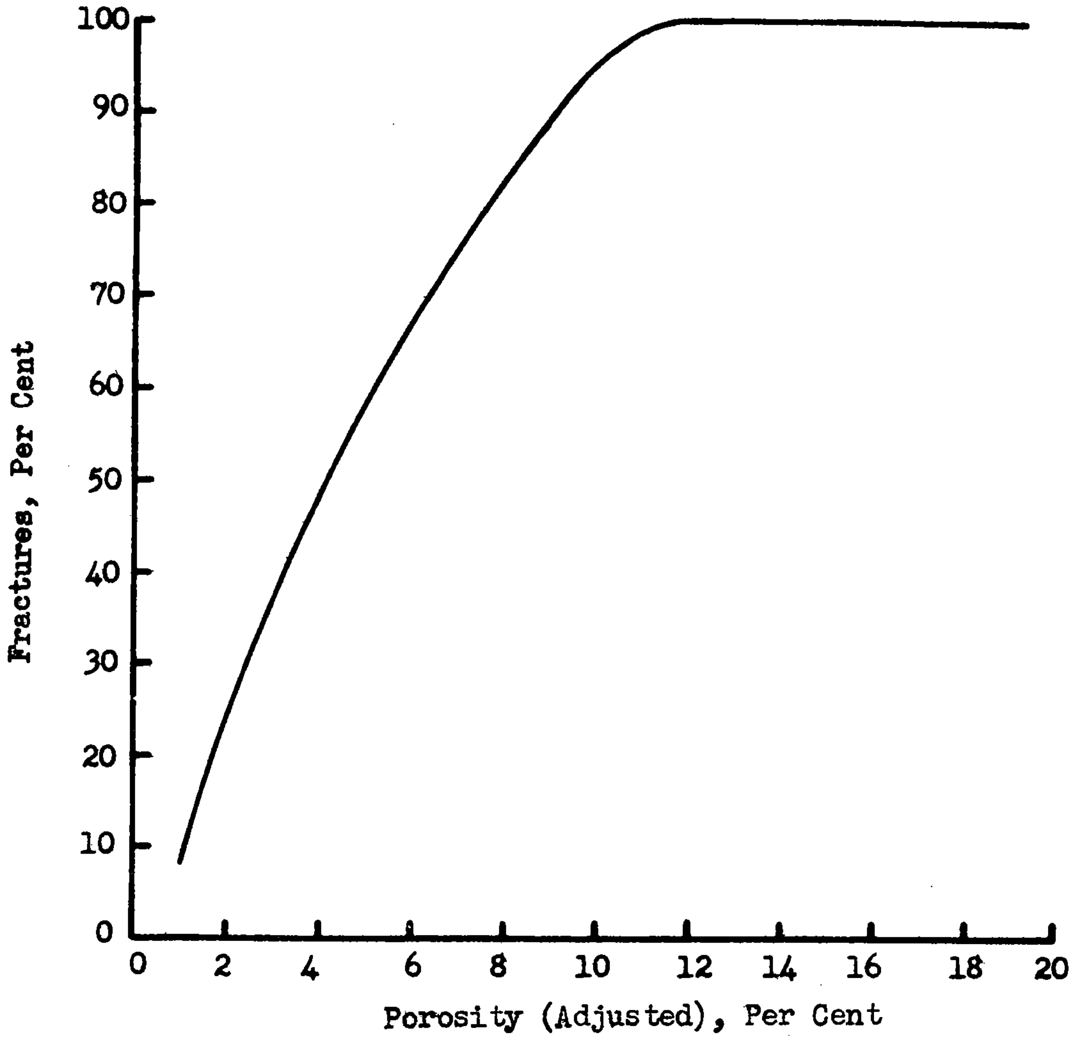

Fig. 4.—Relationship between Porosity (Adjusted Value) and Percentage of Fractured Particles after Exposure to 4 Cycles of Freeze-and-Thaw.

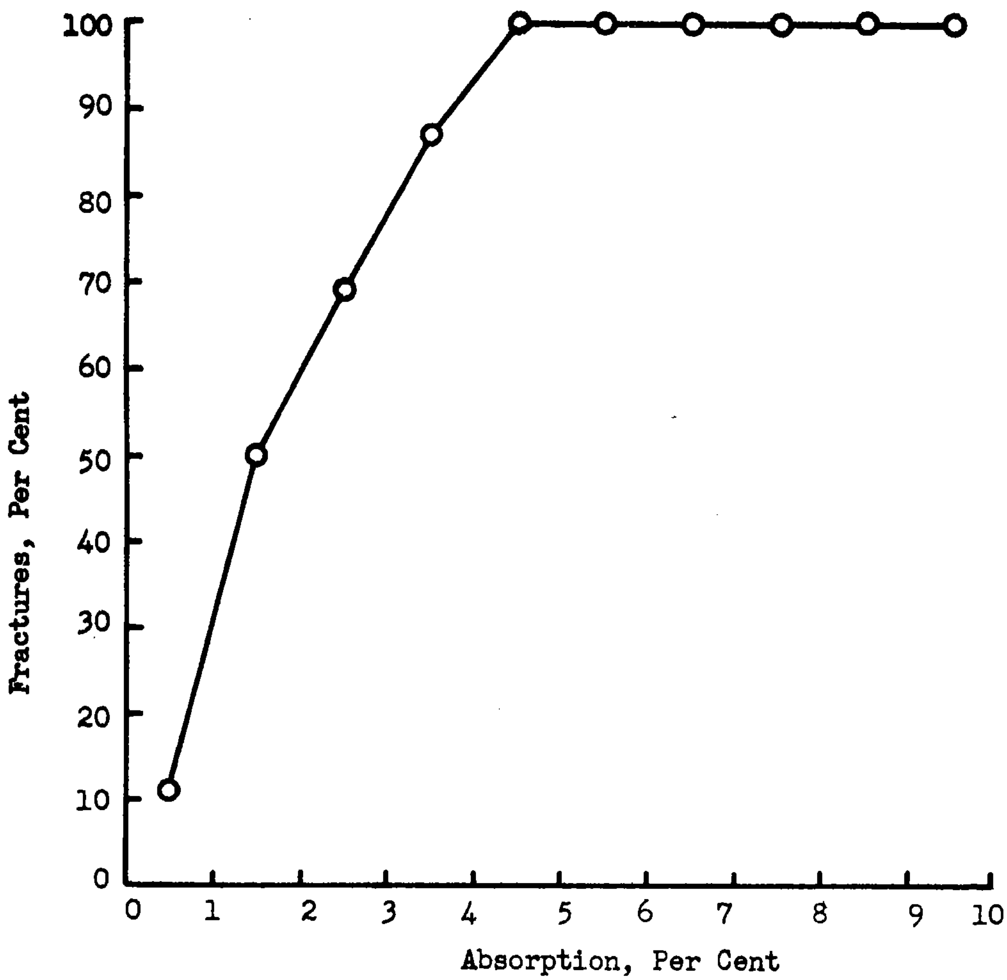

Fig. 5.— Relationship between Absorption and Percentage of Fractured Particles after Exposure to 4 Cycles of Freeze-and-Thaw.

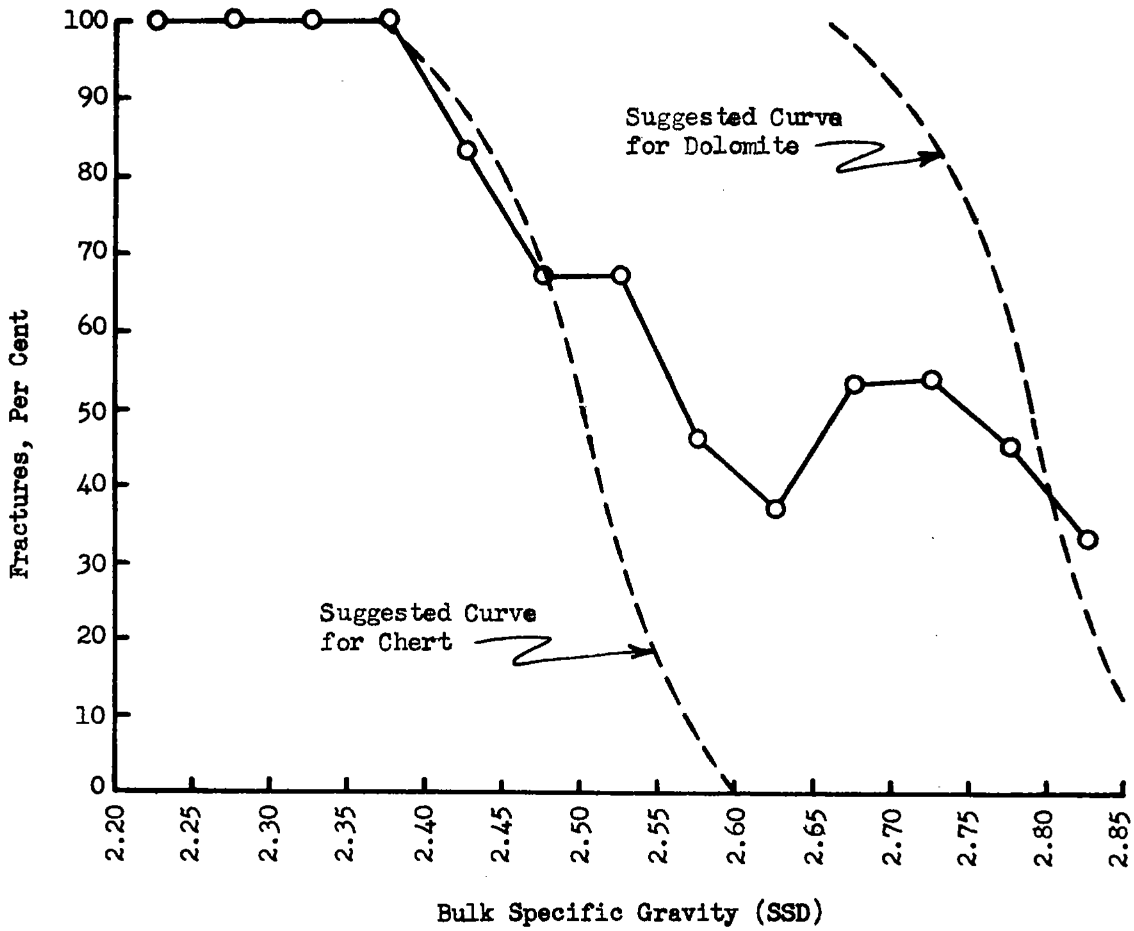

Fig. 6.— Relationship between Bulk Specific Gravity (Saturated Surface-Dry Condition) and Percentage of Fractured Particles after Exposure to 4 Cycles of Freeze-and-Thaw.

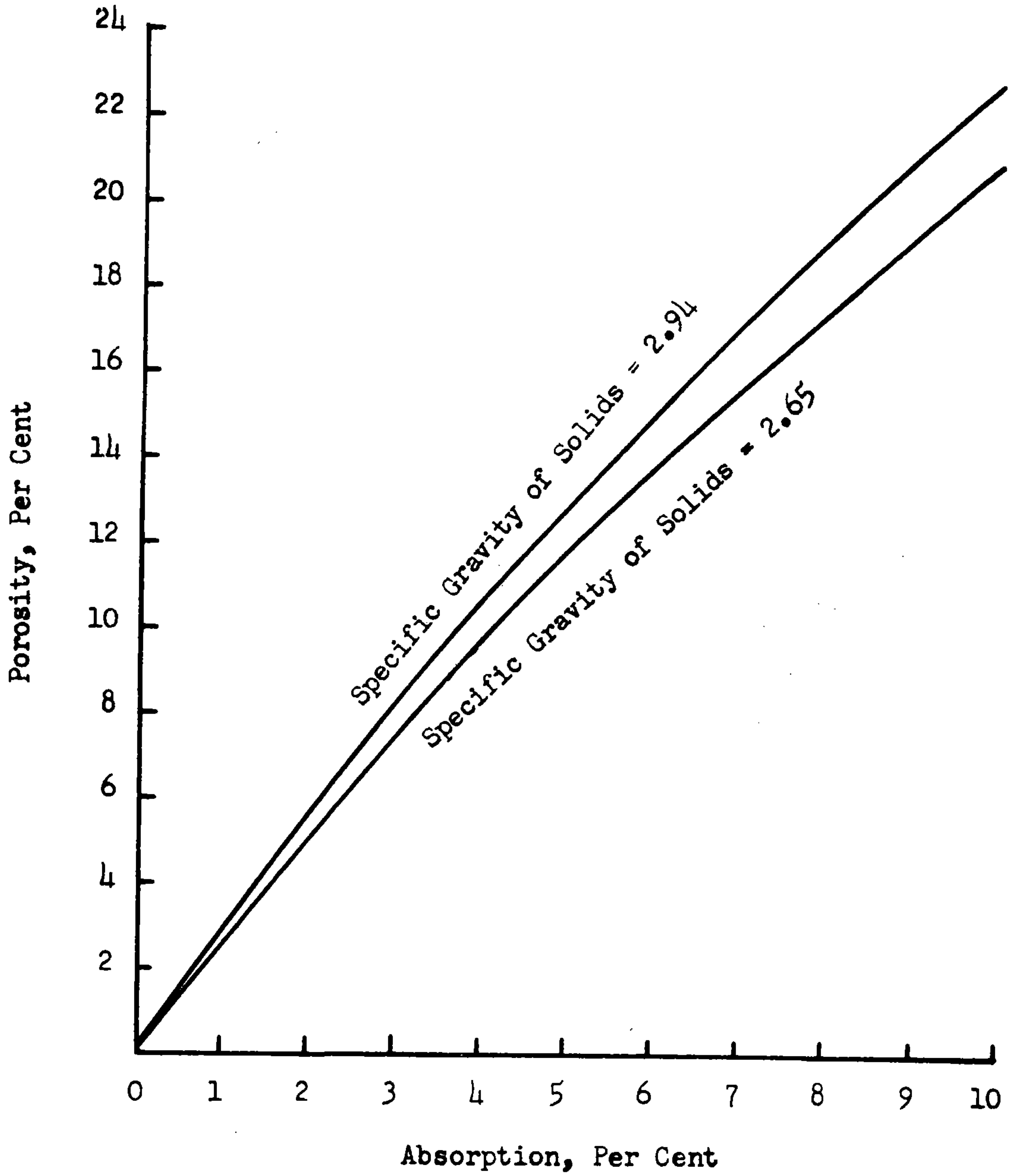

Fig. 7.— Theoretical Relationship between Porosity and Absorption when all Pores are Permeable and Saturated.

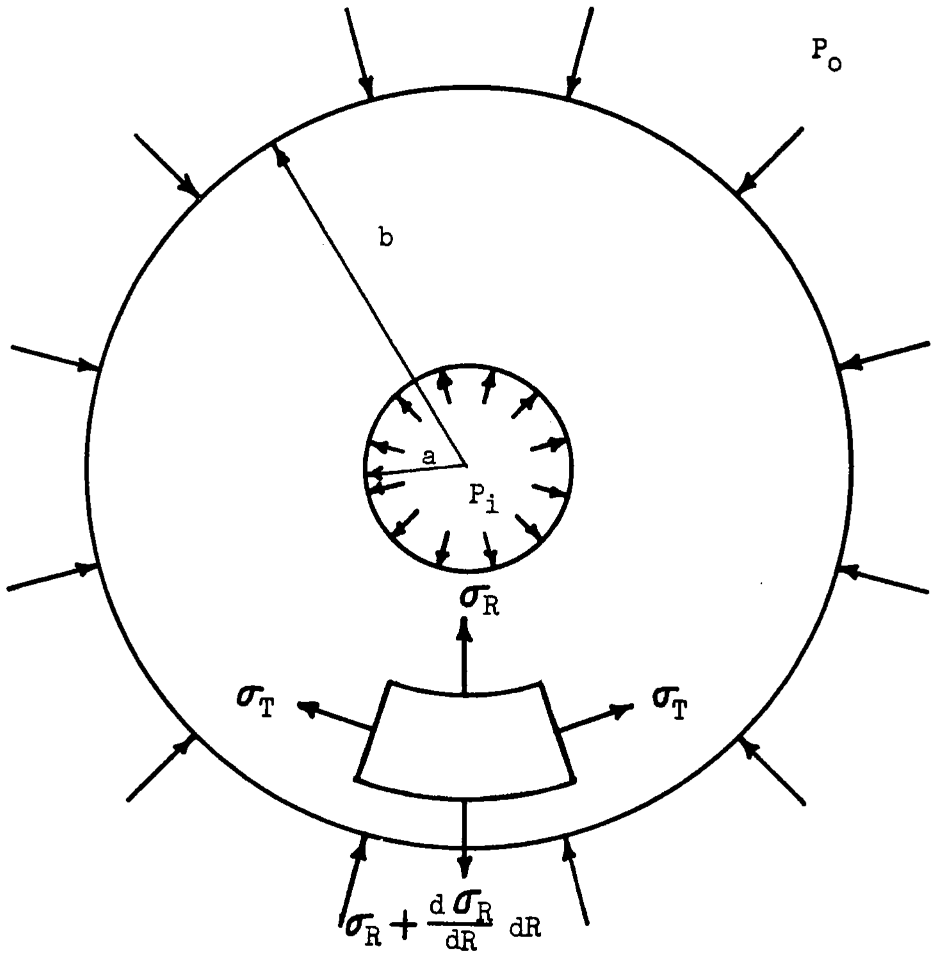

Fig. 8.— Thick-Walled Sphere under Internal and External Pressures (cf. Reference 23).

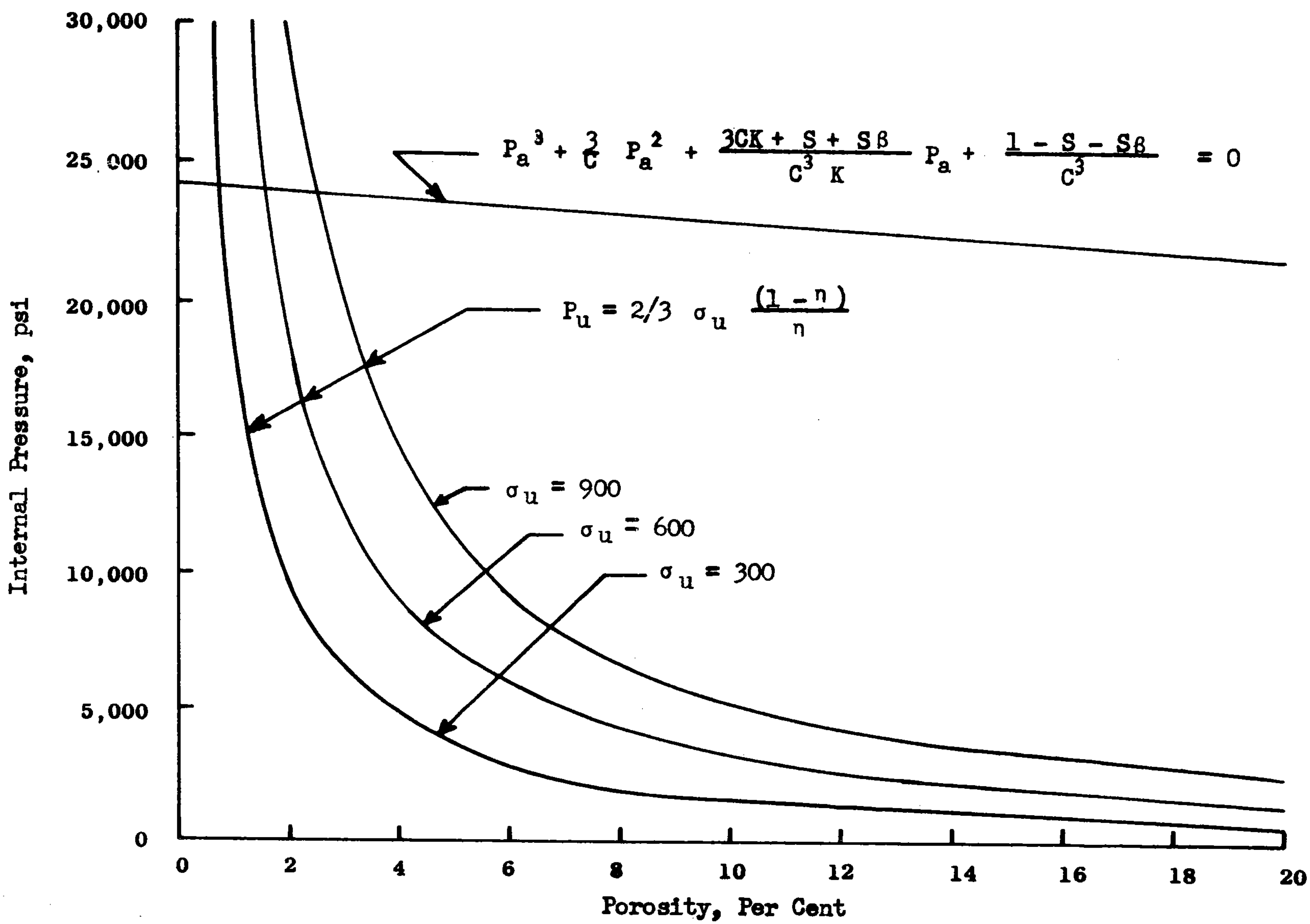

Fig. 9.— Suggested, Theoretical Relationship between Allowable Internal Pressure, P_u, and Available Internal Pressure, P_a, for Varying Porosities and Tensile Strengths.

Fig. 10.— Concrete "Pop-out" Resulting from the Freezing of an Internal Sphere of water.

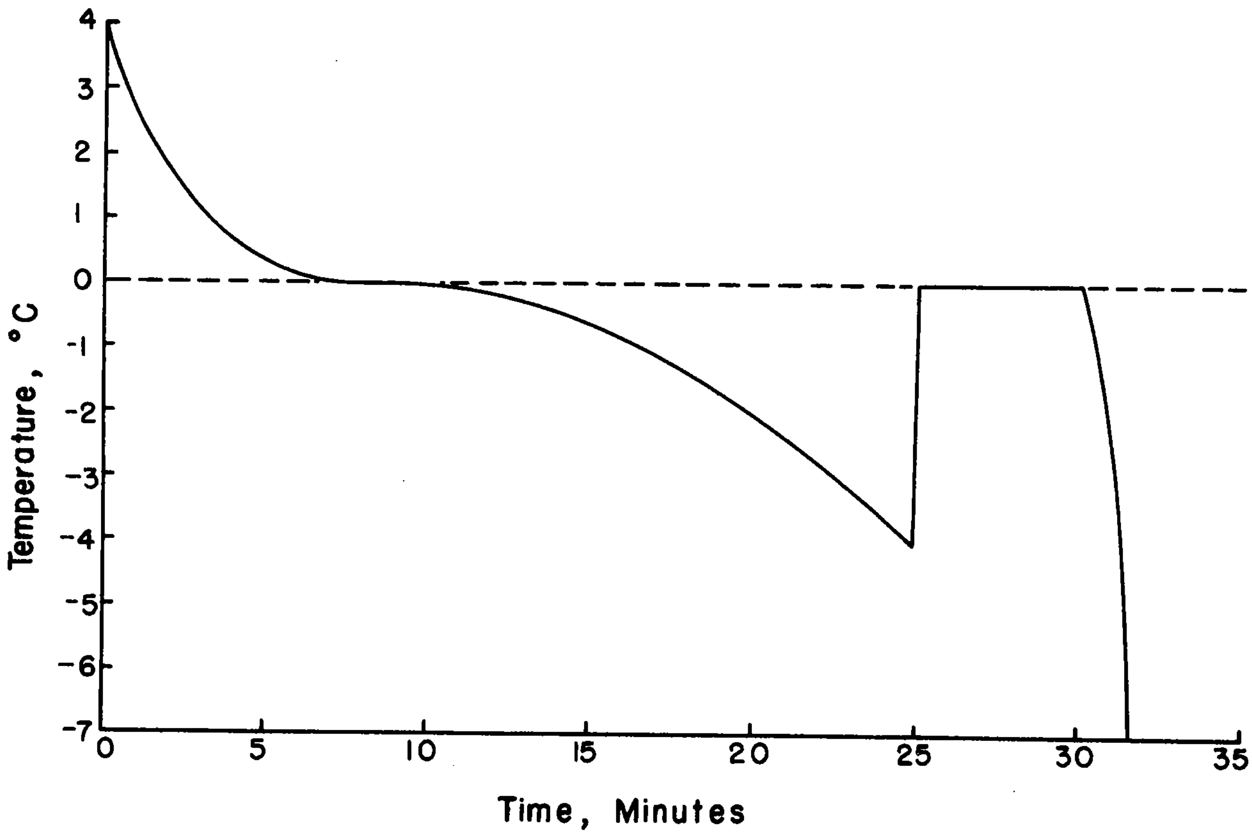

Fig. 11.—Typical Thermogram of a Sphere of Water Encased in Concrete Undergoing Freezing.

PETROGRAPHY OF SOME INDIANA AGGREGATES IN RELATION TO THEIR ENGINEERING PROPERTIES

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INTRODUCTION

The evaluation of aggregates through petrographic analysis is being conducted by a number of state or provincial highway departments and governmental agencies in this country and in Canada. Commonly the aggregates are analyzed prior to approval for construction use, but in some cases petrographic examination is made only after an aggregate problem has been recognized.

The petrographic techniques applied vary considerably among the different agencies. In some cases only megascopic examination is used to ascertain rock type, gross mineralogy and weathering aspects. In other instances microscopic examination, accomplished through the use of the polarizing microscope, is performed in addition to the visual observation. This yields additional information about the aggregate grain characteristics and generally about the overall aggregate quality.

There is a present trend to include several laboratory techniques in addition to visual inspection and thin-section analysis in the general category of petrographic methods (Lounsbury and Schuster, 1964). These include differential thermal analysis, x-ray diffraction, insoluble residue determination and possibly electron microscopy and infra-red studies. It is in this latter, more comprehensive sense, that petrography is applied in this paper.

Indiana aggregates are derived from gravels of glacial origin and from crushed stone from carbonate quarries. The glacial gravels are associated with recessional moraines and glacial sluiceways, and are obtained through the working of gravel pits or by river dredging operations. This discussion, however, is limited to the petrographic aspects of selected quarried stones in Indiana, as the variation in heterogeneous gravels is such that microscopic examination and further petrographic refinements cannot be realistically performed on the numerous rock types present.

The purpose of this study was to relate engineering test data to aggregate properties obtained from petrographic measurements. The petrographic techniques used were essentially those developed in an unpublished report by West et al (1964) in conjunction with research on aggregate degradation of base and subbase material. In previous unpublished studies by Aughenbaugh et al (1963) and by Schuster (1961), petrographic techniques were employed in addition to the major research activities. This earlier work served as a basis for the development of the petrographic analysis used in this study. At present, further refinements are being made as one part of the current research on NCHRP Project 4-2, Degradation of Aggregates, which is being conducted at Purdue University. This paper, however, does not include the results of the continuing research done for the project on this subject.

HISTORY OF AGGREGATE PETROGRAPHY

With the increasing use of aggregates in our expanding construction program, materials engineers and those concerned with the building of highways and other structures seek to assure that quality materials are used on these projects. Standard acceptance tests are generally employed to assure the use of quality aggregates.

There has been a growing cognizance of the fact that these tests may not always detect a potentially poor aggregate, and as far back as the late nineteen thirties, petrographic examination was applied to some extent as an additional tool to predict the possible performance of aggregate behavior in concrete.

Excellent laboratory studies such as that of Sweet and Woods (1942) showed the correlation between such properties as bulk specific gravity, saturated surface dry, of cherts and performance in freezing and thawing tests. Other examples could be cited.

Yet some aggregates may pass standard tests, and still have poor performance records in concrete. It was felt by active workers such as R. Mielenz, B. Mather, and K. Mather that petrographic examination of aggregates might disclose relations between composition and structure of aggregates and their performance in tests and in service in concrete. Their work and that of others helped to establish in 1954 the recommended practice of petrographic examination of aggregates for concrete in the A.S.T.M. standards (1954) on concrete and mineral aggregates. Mielenz (1954) presented his recommendations for the petrographic examination of concrete aggregates, and in 1962 he reported the advantages to be gained through the use of petrography applied to Portland cement concrete.

Meantime in Canada, Bayne and Brownridge (1955) described the petrographic analysis for determination of quality of coarse aggregates. The problem of the Kingston dolomite in Ontario, which passes standard A.S.T.M. tests, but has long had a poor service record as aggregate in concrete, has been discussed in a series of papers by Swenson and his coauthors (1957, 1960 and 1960).

Lemish (1961) reported distress in some concretes made with certain Iowa carbonate aggregates a few years after construction. Some of the development of the use of petrographic examination of aggregates has been summarized by Lounsbury and Schuster (1964). There is every reason to believe that this method of studying concrete aggregates will be utilized even more widely in the future.

INDIANA ENGINEERING TESTS

The requirements for coarse aggregates used for highway construction in Indiana are indicated by limiting specifications set by the Indiana State Highway Commission. As is the common practice of state and federal highway agencies, specifications are developed which set the limiting value of aggregate quality as measured by specified tests. Table 1 is a summary of the coarse aggregates specifications which were of particular interest in this study. It has been condensed from the Standard Specification of the Indiana Highway Commission, 1963.

In reference to Table 1, soft and non durable particles are those which are structurally weak such as soft sandstone, shale, limonite concretions, coal, weathered schist or cemented gravel.

Only Class A stone is used for exposed portland cement concrete and for asphaltic concrete. Class B is acceptable for use in a limited number of situations where the portland cement concrete will not be exposed to the air. Base, subbase, and surface road material coarses can be made from Class A, B or C material. In no case shall aggregates be used which have given unsatisfactory performance in previous construction of a similar nature.

QUARRY SELECTION

The selection of the quarries for study formed the initial step in this research problem. The authors wished to make selections such that a considerable range of properties was obtained but retaining the restriction that the aggregates were either being used currently or had been acceptable sometime in the past. The number of quarries that could be included was limited by the time involved, so that a select number of representative quarries were chosen. In order to solve this problem the Indiana Highway Commission was consulted for advice.

A concerted effort was made to choose quarry sites which would yield examples of differing Los Angeles abrasion percentage wear values, sulphate soundness losses, specific gravity and absorption values, and if possible, contrasting performance records. In addition, engineering data had to be available for the selected quarries.

Ten quarries were chosen to represent the diversity of engineering test data that was desired. The location of these sites varies both stratigraphically as well as geographically with nearly state wide coverage accomplished. The sample sites are shown in Figure 1.

The engineering data for the quarries were obtained from the Indiana Highway Commission. The stratigraphic position in the quarry of the samples used for testing was noted when available. Information was generally available for each prominent lithologic ledge or zone in the quarries, through the courtesy of a testing program currently being carried out by the Commission.

Following the collecting of engineering data, the quarries were visited and ledge samples were taken corresponding to the zones tested by the highway engineers. The engineering data supplied were augmented, when applicable, by additional information from research work performed by the School of Civil Engineering, Purdue University which was exclusive of work done by the authors. The summary of engineering data obtained is compiled in Table 2.

GEOLOGIC SETTING

The formations which were sampled range in geologic age from Silurian through Mississippian. Most of the quarries chosen for sampling are stratigraphically in middle and upper Silurian units. These quarries are found chiefly in the northern third of Indiana, but some lie in the eastern part of the state, extending down into the southeastern counties. Two are located in west-central Indiana.

Along the eastern border of the state the structure of the bedrock is controlled by the Cincinnati arch. This arch bifurcates in the north into the Findlay arch trending northeast into Ohio and the Kankakee arch to the west. Ourcrops in the north are sparse owing to glaciation.

The units which were selected for study include the Liston Creek, Huntington, Kokomo and Kenneth mite and Jeffersonville limestone, and the Ste. Genevieve and St. Louis formations of Mississippian age.

Most of the Silurian units are dolomites or dolomitic limestones. Reef or biohermal facies as well as nonbiohermal rocks are represented in sampled lithologies. The nonbiohermal facies tend to be somewhat porous, crystalline-granular carbonates, and the reef and biohermal rocks are usually aphanitic and vugular. Among the Silurian stones are found some of the best aggregates in the state, but one of the poorest Indiana aggregates, with respect to service record, also occurs in this part of the gologic section. Somewhat argillaceous dolomites of this age also have poor performance records.

Rocks of geologic age younger than Silurian are chiefly crystalline granular limestones with varying porosity and absorption valves. The service records of these quarry stones vary from marginal aggregates to very good ones.

LABORATORY TESTING

A series of laboratory tests were performed on the quarry ledge samples. Insoluble residue determinations, differential thermal analyses (DTA) and x-ray diffraction studies were made. In addition, megascopic and microscopic examination of the aggregates of all the ledge samples was accomplished.

The insoluble residue fractions or total non-carbonate material was determined initially. The amounts of insoluble material ranged from about 2% to 31% of the total material for the aggregate sources studied. X-ray diffraction analysis of the insoluble fractions indicated that quartz and illite were the primary mineral constituents. These findings are in keeping with previous work done by the authors (West et al 1964).

Differential thermal analysis of the carbonate quarry stones was performed to determine the predominance of calcite or dolomite. In the differential thermal analysis, chemical reactions of the sample constituents are recorded with increasing temperatures. Exothermic (heat-loss) and endothermic (heat-gain) reactions occur at specific temperatures for individual mineral species, and these become the basis for mineral identification. Differential thermal analysis is particularly amenable to carbonate differentiation, and it was used semi-quantitatively in this research. A typical DTA thermogram is shown in Figure 2.

X-ray diffraction analyses were made on both the total rock constituents and the insoluble material. The total rock samples were prepared on porous ceramic tiles from a rock-powder slurry. A vacuum aspirator was used to draw the dispersal liquid through the tile leaving the solid constituent on the tile surface. X-ray samples of the insoluble residues were prepared in a similar fashion except that the acid insoluble fraction of the rocks were used to prepare the slide material rather than the total rock constituents. The x-ray analysis was used in a semi-quantative manner to identify the total rock and the insoluble residue constituents. More accurate methods have been described by Diebold et al (1963), but for the purpose of this study the qualitative, more expedient method was employed. It was felt that a combined DTA, x-ray and thin-section analysis was sufficient to recognize the predominant mineral species. A typical x-ray diffraction pattern is shown in Figure 3.

The laboratory results for insoluble residue percentages, DTA results and x-ray analyses of the total rock samples are shown on the extreme right of Table 2. In this table the laboratory results are round adjacent to the engineering test information, which permits a convenient comparison of data.

The analysis of the grain properties and their mutual relationships or indeed, the analysis of aggregate textures, formed the final phase of laboratory testing. The relationship of grain texture and aggregate properties has been noted previously by Rhodes and Mielenz (1948), again by Mielenz (1956) and more recently by K. Mather, in a U. S. Army Engineer Waterways Experiment Station report (1958), and by Gillott (1963). Although the importance of aggregate texture had been recognized, a procedure was not developed as a true laboratory technique for evaluating aggregate characteristics. West and Aughenbaugh (1964) also recognized the importance of texture in relation to aggregate properties in their review of the aggregate degradation problem, and it was from work in this area that ideas were formulated. The techniques used to measure grain relationships for this research problem, as stated in the preceeding introductory remarks, are presently being refined in order to find more meaningful procedures for determining aggregate grain parameters as a measure of overall aggregate properties.

The petrologic analysis of the aggregate samples was accomplished in three steps, (1) hand-specimen examination, (2) polished-section examination and (3) thin-section analysis. This procedure was followed on each of the ledge samples from the selected quarries.

Hand-specimen analysis is helpful in recognizing surface weathering features, depth of weathering and gross characteristics of the rock such as veins, cracks and large fossils. Each rock specimen was named and described on the basis of hand-specimen appearance using the usual simple hardness tools, acid bottle and magnifying glass.

Polished sections were made of each ledge sample. A smooth surface was obtained on the specimens by use of a diamond saw. Following this operation, the cut face was polished by use of a polishing wheel apparatus with consecutively finer grinding powders.

The polished sections were examined first megascopically and finally microscopically using a stereographic binocular microscope. Textural properties and weathering characteristics were noted and compared with the hand-specimen descriptions.

Aggregates with coarse texture or with structures of megascopic proportion were particularly amenable to polished-section analysis. The carbonate rocks with a bedded appearance and those of an argillacious nature were well delineated by this procedure. Quite commonly the thicker strata are more easily determined using polished-sections than by thin-section analysis with its increased magnification.

Thin sections were made for the ledge samples. Thin sections are thinly-ground rock slices about 0.03 millimeters thick which are mounted on glass slides by Canada balsam or similar optical cement. Light transmitted through non-opaque minerals of this thickness yields diagnostic properties for mineral identification in addition to indicating the shape and orientation of original minerals or grains and the texture of the rock as a whole.

In rock thin-sections the following characteristics were observed:

1. Grain roundness
2. Average grain size
3. Grain-size distribution
4. Grain interlock
5. Void content, size and interconnection
6. Weathering characteristics

The individual grain outlines were noted first in the thin-section analysis. The degree of angularity of the grain corners was observed and described accordingly. The possible grain descriptions used were very angular, angular, sub-angular, sub-rounded and rounded. A pebble roundness chart was used for comparison in an attempt to give consistent descriptions for similar grain shapes. A pebble roundness chart is shown as Figure 4. Although this chart was devised for particles much larger (16-32 mm) than those examined, it was found to be quite workable.

The rhombohedral form of euhedral dolomite crystals was a form that caused some consternation. This grain outline is fairly common in the carbonate rocks studied, as recrystallized dolomites make up important aggregate sources in Indiana. Although this straight-edged, polygonal form has no true counterpart on the pebble roundness chart it was decided to regard it as very angular or angular depending on the actual situation.

The average grain size was also ascertained. This value was taken as that grain size that most commonly occurred (modal grain size). No attempt was made in this research to find an average statistically, but this evaluation is planned for future work. The average grain size varied from about 0.7 mm to 0.01 mm in diameter.

The grain size distributions of the aggregate sections were determined next. This data was obtained by noting the largest grain diameter that was present in a significant amount and also the smallest diameter that was significantly present. This gives, on the surface, only a range of grain sizes but with the addition of the average size, a rough measure of the distribution is obtained. Statistical determination of grain size distribution in relation to aggregate behavior is another fruitful area of research, and is a part of other current unreported research.

Grain interlock determination played an important role in this study. It seems apparent that a greater resistance to shearing between grains as indicated by interlock should yeild a greater resistance to abrasion in the rock aggregate. For this reason the degree of interlock or irregularity of mutual grain boundaries was deemed important, and was carefully ascertained in the rock thin-sections.

The possible grain interlock descriptions which were envisioned prior to the actual work, were as follows:

(a) grains seldom touching (floating)
(b) grains with point contacts only
(c) grains with point contacts to straight contacts
(d) grains with straight contacts
(e) grains with straight to concavo-convex contacts
(f) grains with concavo-convex contacts
(g) grains with concavo-convex to sutured contacts and
(h) grains with sutured contacts alone.

The sutured contacts refer to grooved interlocking surfaces similar to decussate textures of metamorphic rocks, which when observed in section, take on a serpentine or S-shaped appearance. This type of contact is normally found only in metamorphic rocks and hence was not commonly observed in the sedimentary carbonate aggregate in this study. The interlock types in the previous list are assumed to increase from top to bottom with point contact being the poorest common interlock (example - friable sandstones) and sutured the best.

The interlock observed in these Indiana aggregates ranged from point contact - straight interlock to concavo-convex. Several examples of interlock are shown on the photomicrographs which appear as Figures 5, 6, and 7.

In Figure 5 an example of straight interlock is shown. These mineral grains are euhedral dolomite rhombs. Figure 6 shows the concavo-convex interlock which is more common, particularly in crystalline, calcareous rocks. There are some straight interlocking grains in this photomicrograph as well. Figure 7 is a photomicrograph of an organic limestone with a fairly large grain size.

The void content and weathering characteristics were observed in the final analysis. Most of the aggregates were unweathered, so that this feature was of little importance in this study. The void content, however, varied considerably. In most of the aggregates the void content was extremely low, but in others the amount of voids was 15% or more. There is an inherent limit to the pore size that can be resolved using the petrographic microscope. The limit in this study was about 0.001 mm or 1 micron. Pores smaller than this size could not be recognized.

The data found from thin-section measurements are summarized in Table 3. In addition to the parameters described above, a mineralogic description is also given.

RESULTS AND INTERPRETATIONS

The selection of quarries for this study, as indicated in a preceeding discussion, was not made without careful consideration. The quarries were chosen to give wide variation of engineering properties and overall aggregate quality.

A comparison of the Indiana Highway Commission Specifications in Table 1 and the engineering data in Table 2 serves as an indication of quality according to the highway personnel.

Ind. 1 ledge 3 and the ledges above it have one of the best performance records in the state. Ledge 3, as seen in the able of results, has a low percentage of loss by abrasion, a low sulphate soundness value and absorption value. In contrast to this, Ind. 5 has a relatively high percentage of loss by abrasion and commonly shows too high a soundness loss to be accepted as class A or B stone. By comparison it is not considered to be as reliable an aggregate source as Ind. 1. In this manner the relative reliability of the aggregate sources can be ascertained for the quarries studied.

Ind. 1 and Ind. 10 were included for a particular reason. These two stones represent the extremes of aggregate quality which were used for concrete pavements in Indiana in the mid 1940's. Ind. 1 is still operating today, but Ind. 10 has long since been abandoned.

As reported by Woods et al (1945) pavement blowups were a problem of considerable importance prior to 1945. A blowup is the sudden upward movement of a concrete slab section at the joint location when the pavement expansion has become so great that adjacent slabs can no longer contain it. The study showed that most blowups occur on spring afternoons following a shower. The problem was traced to the coarse aggregate supply, and Ind. 10 was found to be the worst offender. It yielded as high as 50 blowups per mile in some sections. In contrast Ind. 1 (bed 3 and above) yielded no blowups at all.

In this study Ind. 1 (bed 3) and Ind. 10 were compared petrographically along with the other quarry samples. The grain parameters looked favorable in both aggregates. In all cases the grain size was small and the void content low which should yield mechanically good stones. The fairly low insoluble residue values (10%) suggested a fairly low sulphate soundness value. The key to the problem lies in the absorption and degree of saturation of the two aggregates. Ind. 10 is well beyond the upper limit of the 3% absorption as set by present specifications, and has an excessive degree of saturation. Ind. 1 is in the low range for both tests. This suggests that the problem involved chemical stability. Also the small size pores, which are obviously present in the Ind. 10 material giving rise to the high absorption, cannot be observed using microscopic techniques.

In a more complete unpublished study by West et al (1964) numerous mid-continent carbonate aggregate samples were analyzed in a method similar to the petrographic approach used here. The petrographic descriptions were transposed into numerical form and statistically compared from which a number of tentative conclusions were reached. The number of samples in this present study was not sufficient to make a similar comparison, but the same general (and tentative) conclusions appear to hold. These can be summarized as follows:

Porosity—The greater the porosity in thin section normally, the greater absorption and degree of saturation, the lower freeze-thaw resistance and the higher Los Angeles abrasion loss.

Grain Interlock—The greater the degree of interlock, the less the Los Angeles abrasion loss.

Grain Size—Excessively large grains yield high Los Angeles abrasion loss.

Argillaceous Content—The greater the argillaceous content as measured by insoluble residue or x-ray diffraction, the lower the freeze-thaw resistance, the greater the absorption and degree of saturation and the higher the sulphate soundness loss.

Lime-Magnesia Ratio—A complex relationship, because the grain interlock improves with magnesia content (dolomite), but porosity commonly increases as well.

In this paper no attempt was made to make implications about the relationship between petrographic parameters and performance. The comparison was made instead between petrographic tests and some standard engineering tests. Therefore whatever relationship is felt to exist between standard engineering tests and actual in-service behavior can be transferred to the petrographic measures through their relationship to the engineering laboratory tests. The comparison between grain characteristics in addition to other petrographic measures, and field performance, is yet another problem which remains to be solved.

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TABLE 1

Coarse Aggregate Specifications

Indiana Highway Commission, 1963

	Classes A	B	C
Percent of loss (by weight) Los Angeles abrasion 500 revol. (T96) not to exceed	40.0	45.0	50.0
Soundness (Sodium Sulphate) T103, T104 percent not to exceed	15.0	20.0	20.0
Absorption (T85) percent not to exceed	3.0	3.0	--
Freeze-Thaw (ASTM) 50 cycles, percent not to exceed (used to approve samples if failing soundness)	15.0	20.0	20.0
Deleterious Materials (percent by weight) not more than			
Clay Lumps	0.2	0.2	
Ocher	1.0	1.0	
Shells	0.7	1.0	
Soft or non durable particles	4.0	4.0	
Sum of all above (excepting clay lumps)	5.0	7.0	
Chert less than bulk specific gravity (T85) of 2.45, percent not more than	3.0 for cement concrete		

TABLE 2
Engineering and Laboratory Test Data

Designation	Percentage of Wear Los Angeles Abrasion Grading A	Sulphate Soundness Loss %	Specific Gravity	Absorption % Degree of Saturation	Insoluble Residue %	Mineralogy	Geologic Unit
Ind 1							
Ledge 3	24	5.3		0.8*, 55%*	16	Calcite	Ste. Genevieve
Ledge 4	25	29.8			31	Dolomite, illite, quartz	St. Louis
Ind 2							
Ledge 2	27	6.8	2.70	0.6	8	Calcite, a little illite, quartz	Ste. Genevieve
Ind 3							
Ledge 2	27	11.6	2.65	1.4			
Ledge 3	30	11.5	2.46	2.8	5.5	Dolomite	Liston Creek
Ledge 4	27	8.3	2.63	2.0	7.9	Dolomite	
Ind 4	37	6.5	2.64	2.7	3.0	Dolomite	Huntington
Ind 5							
Ledge 3	34	20.6	2.40	5.3		Dolomite	Geneva
Ledge 4	29	2.5		2.7	6.4	Dolomite	
Ind 6							
Ledge 3	30	17.7		1.7	8.7	Calcite	Golconda
Ledge 4	28	24.3		1.4	12.2	Calcite	
Ind 7							
Ledge 2	25	7.0	2.69	1.2	3.1	Dolomite	Huntington
Ledge 3	35	5.0	2.59	1.6	3.7	Calcite	Liston Creek
Ind 8							
Upper					6.4	Dolomite	Jeffersonville
Middle	30	22.1	2.59	1.3	5.6	Dolomite	Geneva
Lower					15.2	Dolomite, a little calcite	Louisville
Ind 9							
Top	28	27.8		7.7	12.5	Dolomite	Jeffersonville
Middle	29	2.5		2.7	6.4	Dolomite	Jeffersonville
Lower	33	1.4		2.4	3.4	Dolomite	Geneva
Lower level					4.3	Dolomite	Geneva

TABLE 2 (Con't.)
Engineering and Laboratory Test Data

Designation	Percentage of Wear Los Angeles Abrasion Grading A	Sulphate Soundness Loss %	Specific Gravity	Absorption % Degree of Saturation	Insoluble Residue %	Mineralogy	Geologic Unit
Ind 10							
Pit 1							
5' from top					7.3	Calcite	Kenneth
10' from top				8.3*, 98%*	5.0	Calcite	Kenneth
15' from top					7.9	Calcite	Kenneth
Pit 2							
5' from top					7.9	Calcite	Kenneth
10' from top					9.4	Calcite	Kenneth
30' from top					2.0	Dolomite plus calcite	Kokomo
40' from top					2.6	Dolomite plus calcite	Kokomo

* Vacuum Saturation valves

TABLE 3
Results of Thin-Section Analysis

Designation	Avg. Grain Diameter (mm)	Void Content	Grain Size	Grain Interlock	Grain Roundness	Description
Ind 1						
Ledge 3 sec a	.10	Low (<5%)	6-.05	Concavo-convex to straight	Subrounded	Calcite, fossiliferous
sec b	.4	Low (<5%)	1-.2	Concavo-convex	Subangular	Calcite, fossiliferous
Ledge 4 sec a	.03	Low (<5%)	.05-.015	Straight	Angular	Dolomite
sec b	.02	Med to High (10%)	.06-.01	Point contacts to straight	Subangular	Dolomite
sec c	.75 .02 (bimodal)	High (~15%)	1.5-.01	Straight contacts	Angular	Fossil fragments plus dolomite
Ind 2						
Ledge 2	.45 (bimodal) .01	Low (<5%)	3-.005	Concavo-convex	Subangular	Calcite
Ind 3	.045	Low-Medium (~5%)	.12-.025	Straight to concavo-convex	Angular	Dolomite
Ind 4	.15	Medium (5-10%)	.25-.05	Straight to concavo-convex	Angular	Dolomite
Ind 5	.03	Medium (5-10%)	.12-.01	Straight	Angular	Dolomite
Ind 6						
Ledge 3	.5	Low (<5%)	1.5-.3	Point contacts to straight	Subrounded	Calcite
Ledge 4	.7	Low (<5%)	2.0-.25	Point contacts to straight	Subrounded	Calcite

TABLE 3
Results of Thin-Section Analysis

Designation	Avg. Grain Diameter (mm)	Void Content	Grain Size	Grain Interlock	Grain Roundness	Description
Ind 7						
Ledge 2	.1	Med-High (~10%)	.5-.02	Straight to concavo-convex	Angular	Dolomite
Ledge 3	.06	High (~15%)	.15-.01	Straight to concavo-convex	Angular	Dolomite
Ind 8						
Upper	.02	High (~15%)	.04-.01	Straight	Angular	Dolomite
Middle	.04	Med-High (~10%)	.085-.020	Point contacts to straight	Angular	Dolomite
Lower	.05	Low (<5%)	.5-.01	Straight	Angular	Dolomite
Ind 9						
Top, sec a	.01	High (~15%)	.025-.005	Straight	Angular	Dolomite
sec b	.015	Med (5-10%)	.025-.005	Straight to concavo-convex	Angular	Dolomite
Middle	.03	Low (<5%)	.065-.015	Straight to concavo-convex	Angular	Dolomite, little calcite
Lower	.03	Low (<5%)	.06-.005	Straight	Angular	Dolomite, little calcite
Lower level	.035	Low (<5%)	.10-.01	Straight to concavo-convex	Angular	Dolomite, calcite wgs.
Ind 10						
Pit 1						
5' from top	.04	Low (<5%)	.15-.02	Concavo-convex	Subangular	Calcite
20' from top	.03	Low (<5%)	1.0-.01	Concavo-convex	Subangular	Calcite crystals with large calcite fossils
30' from top	.1	Low (<5%)	.04-.005	Straight to concavo-convex	Angular	Calcite

TABLE 3
Results of Thin-Section Analysis

Designation	Avg. Grain Diameter (mm)	Void Content	Grain Size	Grain Interlock	Grain Roundness	Description
Ind 10 (Con't.)						
Pit 2						
5' from top	.02	Low (<5%)	1.0-.005	Straight to concavo-convex	Angular	Calcite
10' from top	.01	Low (<5%)	.015-.003	Concavo-convex	Subangular	Calcite
30' from top	.04	Low (<5%)	.060-.001	Straight	Angular	Dolomite some calcite
40' from top	.04	Low (<5%)	.10-.020	Straight	Angular	Dolomite some calcite

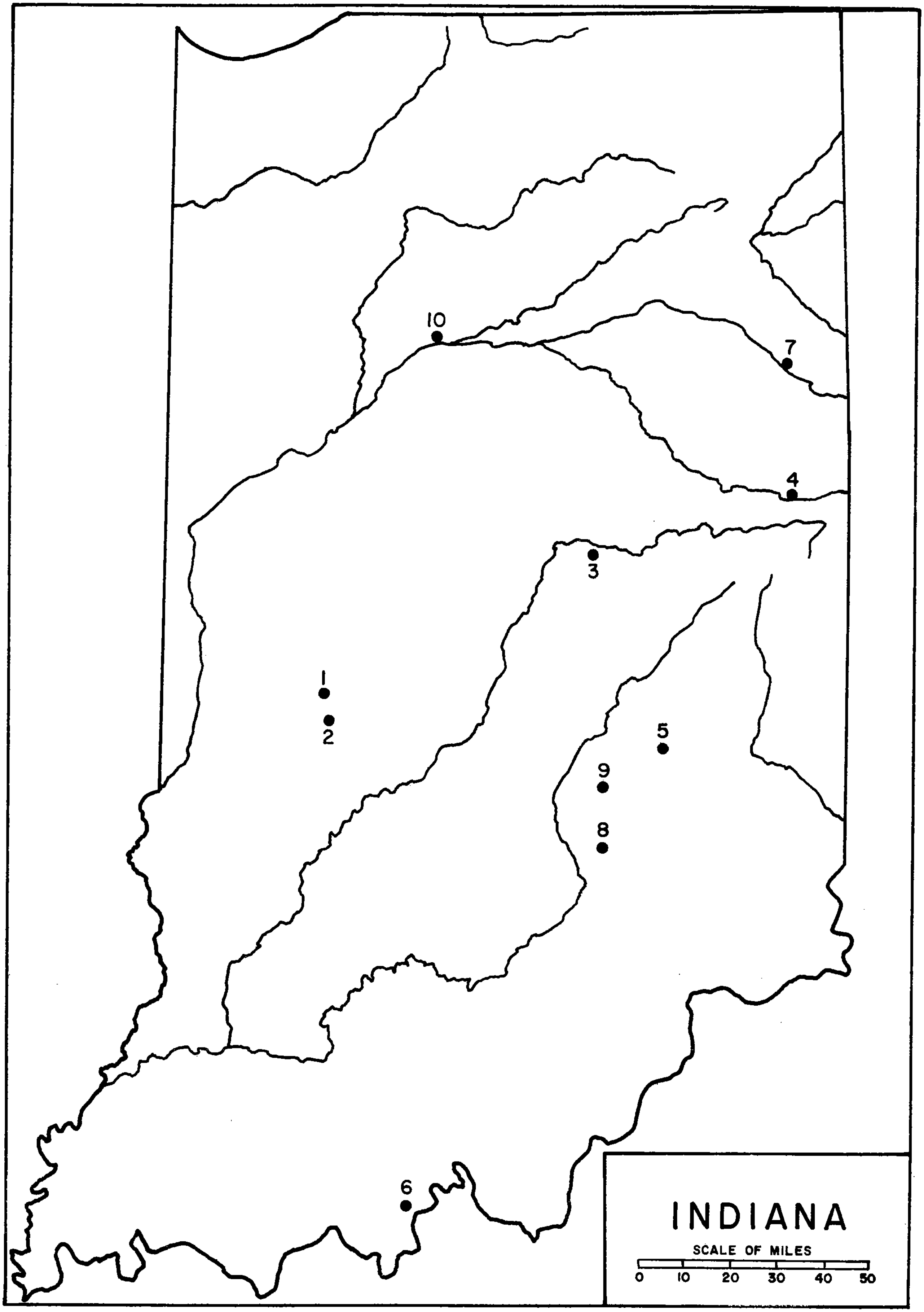

Fig. 1.— Location of Sample Sites.

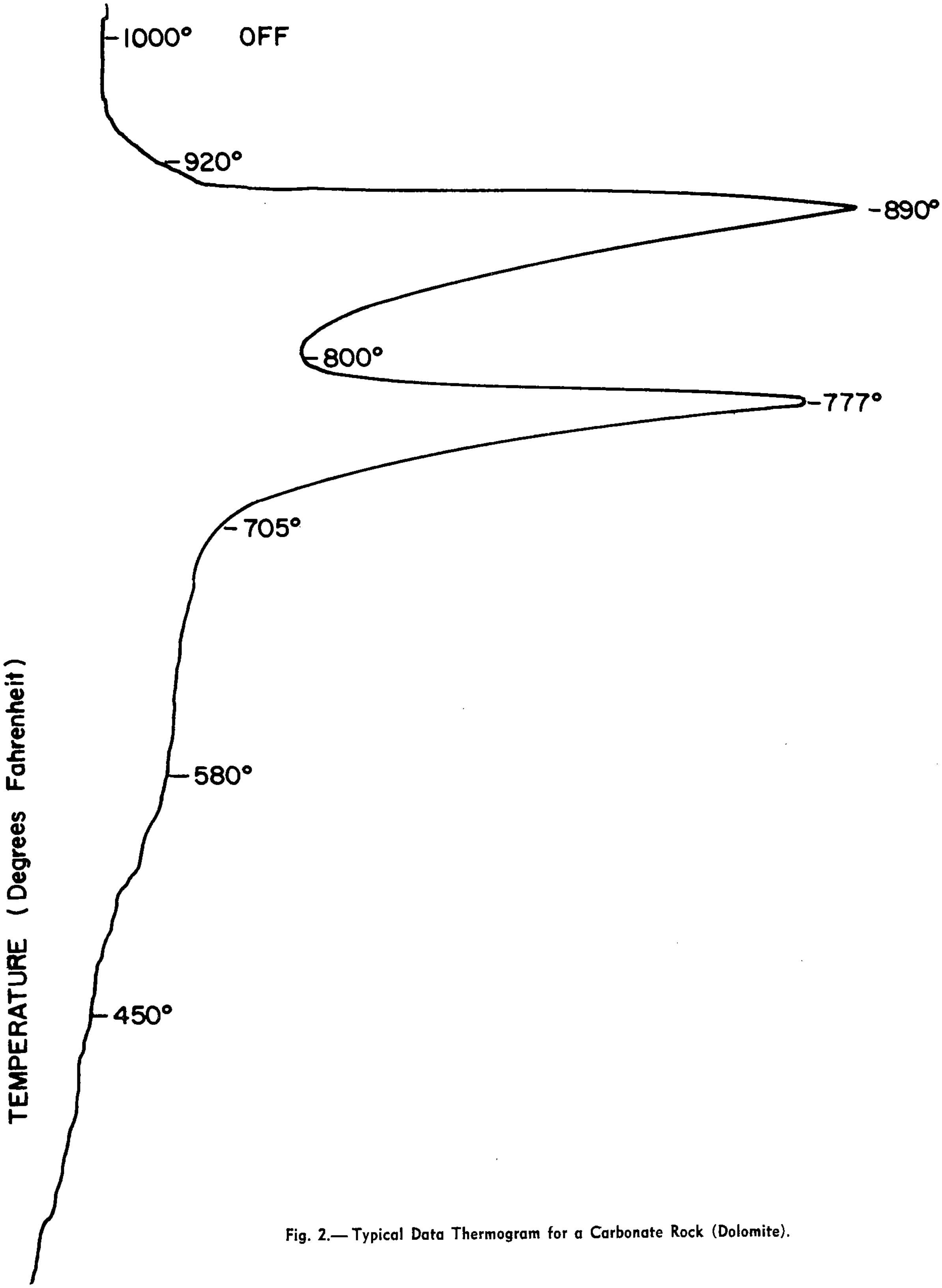

Fig. 2.— Typical Data Thermogram for a Carbonate Rock (Dolomite).

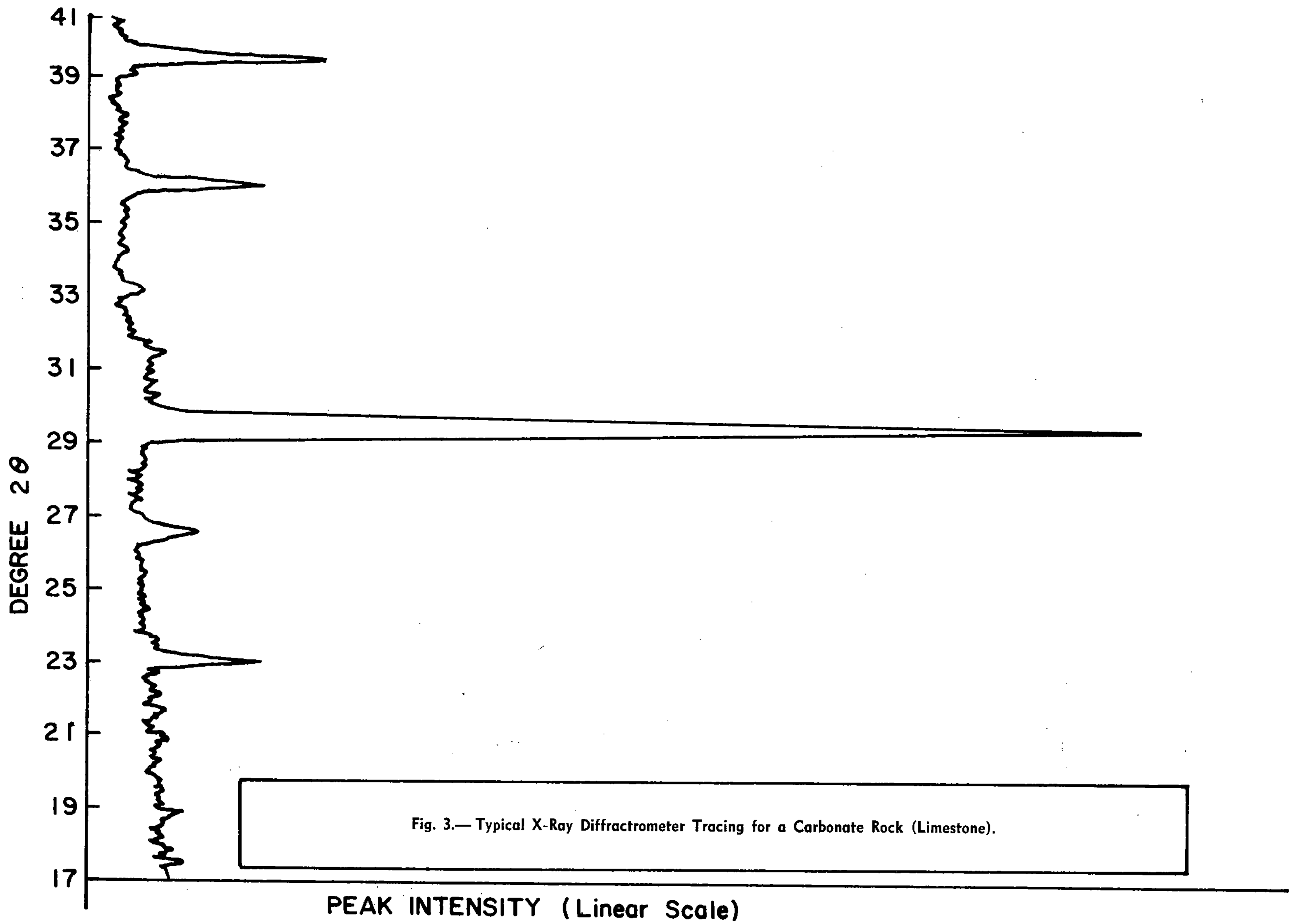

Fig. 3.— Typical X-Ray Diffractrometer Tracing for a Carbonate Rock (Limestone).

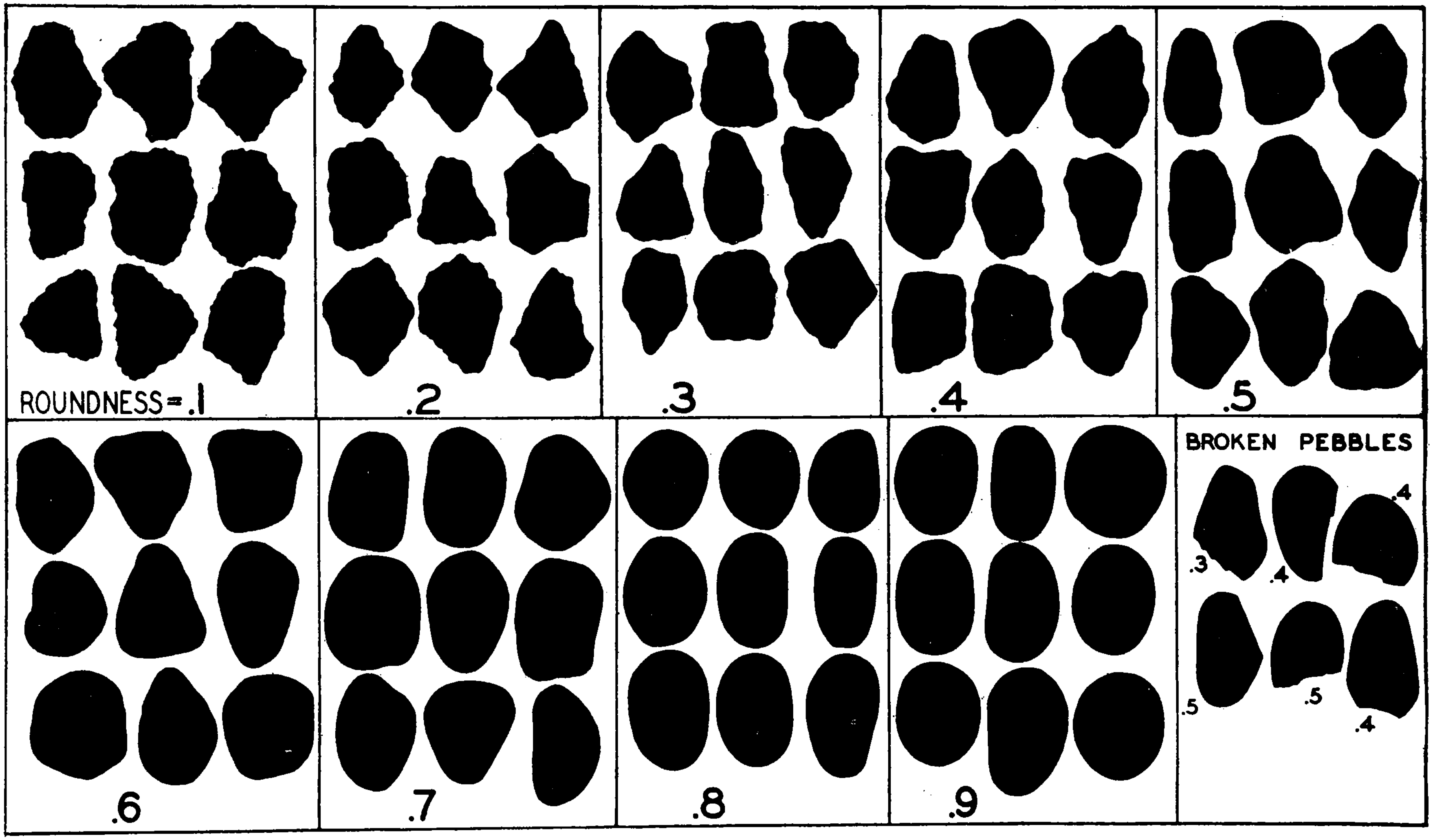

Fig. 4.— Pebble Roundness Chart.

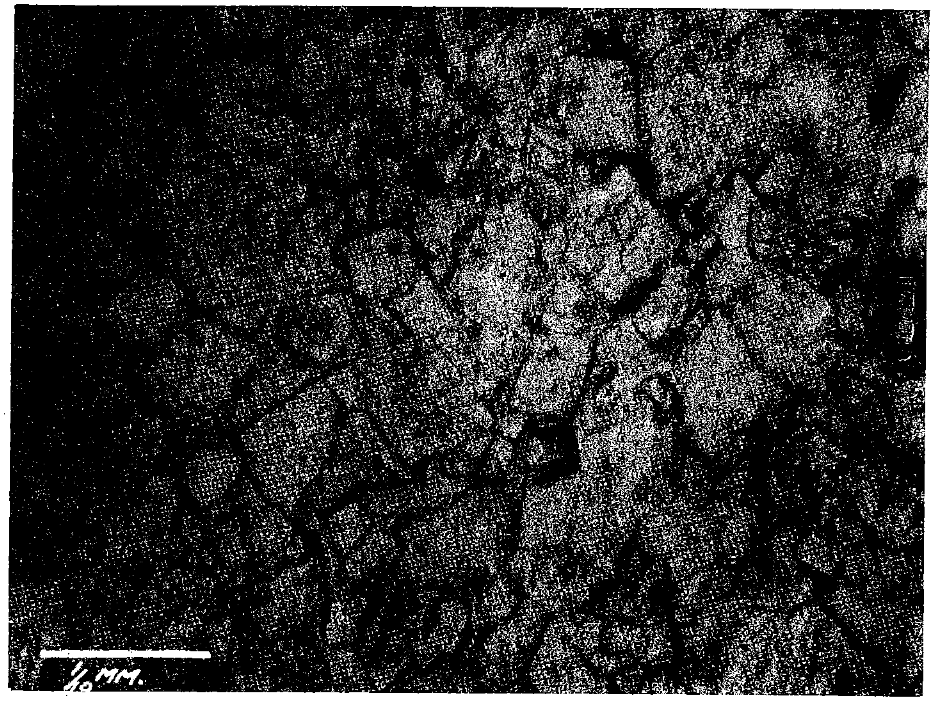

Fig. 5.— Photomicrograph Showing Straight Interlock.

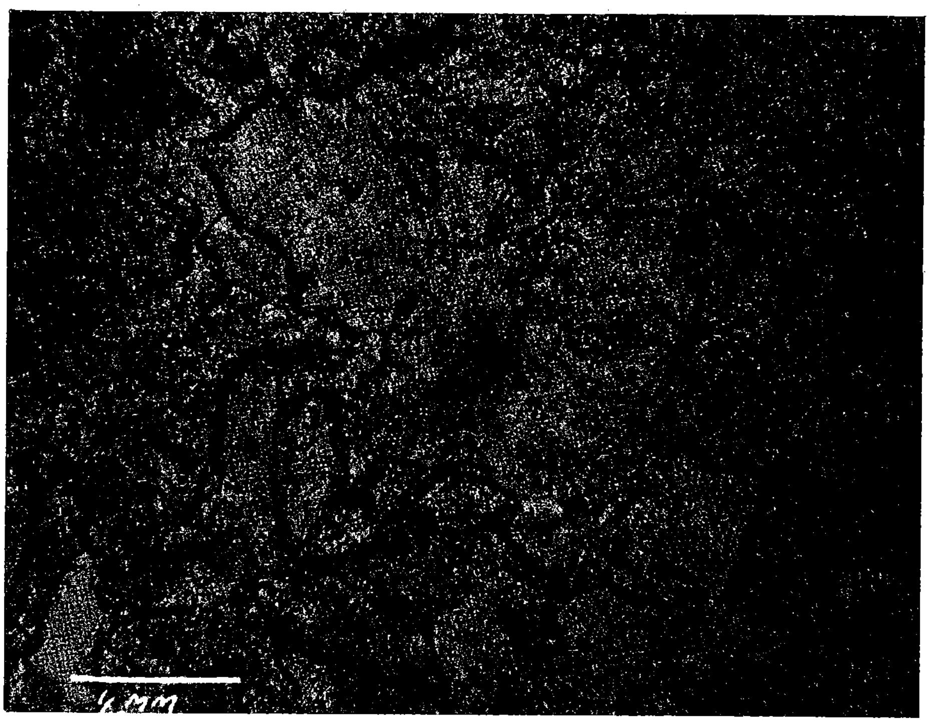

Fig. 6.— Photomicrograph Showing Concavo-convex to Straight Interlock.

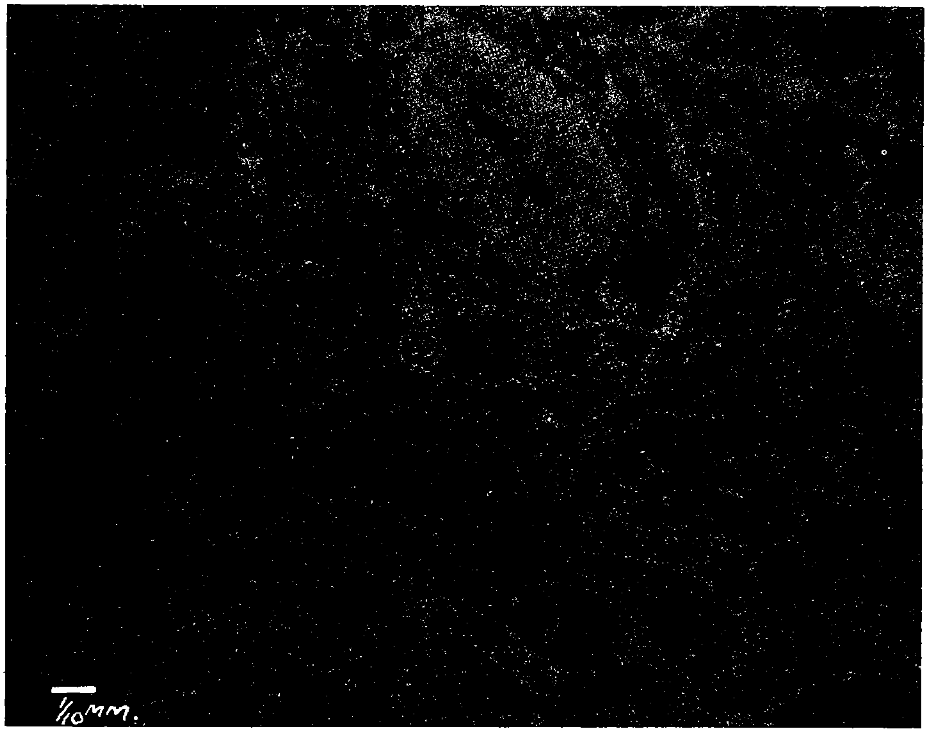

Fig. 7.— Photomicrograph of a Fossiliferous Limestone Aggregate.

THE ROLE OF AGGREGATE TYPE IN PAVEMENT SLIPPERINESS

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INTRODUCTION

Loss of life and property resulting from highway skidding accidents has become a matter of increasing concern to highway administrators in recent years. This concern has given rise to a considerable research effort, both in the United States and abroad, into the variables that might affect skidding. In a paper presented at the First International Skid Prevention Conference, Moyer (1959) recognized over 30 variables as affecting coefficients of friction in road surface tests. Fifteen of these were thought to be major factors. However, as the literature on skidding increases it is becoming apparent that pavement slipperiness, and more specifically the polish susceptibility of the included aggregates, is a major if not the major factor involved.

Studies in England (Road Research—1959) show that 27 percent of the highway accidents there involve skidding. Other work indicates that aggregates formerly considered polish resistant are beginning to show signs of becoming slick under the influence of constantly expanding traffic volume. This expanding traffic volume and increased vehicle speeds combine to accentuate the importance of maintaining skid resistant pavements.

In 1946, Virginia began an investigation of the skid resistance of highway pavements throughout the State. Results of this investigation were published by Shelburne and Sheppe in 1948. In 1956, Nichols et. al. published results of additional skid resistance measurements in the state along with data on skidding accidents. The distribution of skidding accidents showed a very interesting pattern. Figure 1 is an outline map of Virginia showing the eight highway districts. Figure 2 is a graph of skidding accidents per 100 million vehicle miles. Districts 1, 2 and 8 showed a frequency of skidding accidents almost double that of the remainder of the state. These are also the districts where limestone is the primary aggregate material.

SLIPPERINESS MEASUREMENTS

The suggestion that aggregate type might be important in the skid resistance of pavements in the early studies in Virginia gave rise to an intensive research effort to determine to what extent this might be true. Actually, Virginia turned out to be very well suited to a study of this type since about equal thirds of the state are located in three distinctly different physiographic provinces; namely, the Valley and Ridge, the Piedmont and the Coastal Plain (see Figure 1). This difference in physiography allowed a great variety of aggregates types to be tested and compared under a single system of design, similar climatic conditions, etc.

All standard testing prior to 1957 was performed by skidding an instrumented car on wet pavements at a speed of 40 mph. While this method was improved over several years to yield accurate results in most cases, the method was obviously time consuming and limited as to the type of sites that could be tested. In August 1957 the General Motors skid trailer was brought into the state for 11 days for an intensive statewide survey. Nearly 2400 sites, on predominantly bituminous pavements, were tested in this brief period (Nichols-1959). Table I from Nichols' data (1959) shows the results of this survey based on broad aggregate type in bituminous surfaces.

In earlier work with the skidding automobile, Virginia had set an arbitrary limit for a safe stopping distance on a wet pavement as 133 feet at 40 mph. This limit correlates roughly with a coefficient of friction value of .35 on the G. M. trailer. With this in mind, a check of Table I shows that virtually all the non-carbonate aggregates quarried in Virginia gave acceptable road surface friction. Most of the carbonate aggregates apparently did not. However, the wide divergence of coefficient of friction values within the carbonate group appeared to be an area worthy of further study.

Subsequent slipperiness measurements using the Virginia skid trailer, which was patterned after the General Motors vehicle, and the British portable tester yielded results not at variance with those of the original statewide survey.

Since 1946 the Virginia Department of Highways has devoted considerable effort to remedial measures aimed at eliminating slippery pavements within the state. Briefly, these efforts have resulted in two developments, (1) use of thin sand overlays on existing pavements and (2) use of non-polishing coarse aggregate in all new asphaltic concrete surface courses. These measures have proved very successful to date and their use is accepted as standard procedure.

AGGREGATE WEARING CHARACTERISTICS

Subsequent to the gathering of data on the gross friction characteristics of pavements containing different aggregate types, it appeared logical to investigate the wearing behavior of these aggregates under traffic. A photographic technique employing a single lens reflex 35 mm camera with a bellows attachment was developed. Figures 3 through 8 show some of the typical results obtained with this technique.

Figures 3 and 4 show two distinct carbonate aggregates—Figure 3 a high calcium limestone and Figure 4 an impure dolomitic limestone. In both, a high degree of polishing is evident. The aggregate in Figure 5 is a siltstone with fair to good skid resistance. Figure 6 shows a granite gneiss that has given very good skid resistance. Here the phenomenon of "surface renewal" (a concept first suggested to the author by Dr. James L. Eades) is quite evident. This phenomenon is believed to be caused by the presence of coarse grain size and large differences in grain hardness. These two factors appear to combine to cause differential wear and a plucking out or shearing of grains that result in a constantly renewed abrasive surface. Figure 7 shows a non-polishing (quartzite) coarse aggregate with limestone fines. This type of mix is now being used in some of the primary pavements in the limestone areas of Virginia. In Figure 7 a sharp contrast is evident between the surface roughness of the coarse quartzite and the polished limestone fines. Finally, Figure 8 illustrates a problem that is only indirectly caused by aggregate, that of flushing. In this case an altered basalt which has fair to good surface texture but high specific gravity has been applied in a plant mix and flushing has taken place. This flushing has allowed the asphalt binder to migrate upward and form a smooth planar surface with the aggregate. These pavements were found to be quite slick through no fault of the aggregate wearing characteristics.

THE CASE OF CARBONATE ROCKS

Based on the coefficient of friction measurements shown in Tables I and II further studies of carbonate aggregates were undertaken. Outstanding questions to be considered were: (1)Why the significantly lower coefficient for carbonates vs. non-carbonates, and (2) Why the large range of coefficients within the carbonate group? Techniques used in this phase of the study included thin sections, insoluble residue analyses, and a laboratory wearing wheel developed by Dr. J. L. Eades. Aggregates from several quarries were studied but the points to be made here can be shown by considering only three. These are Rockydale of Lynchburg (Arch Marble), Perry (Conococheague Formation) and Dominion, (New Market Formation) chosen to represent high, intermediate and low coefficients of friction respectively.

Figure 9 shows photomicrographs of typical thin sections from the rocks from these sources. Section 9a from Rockydale closely resembles a granite gneiss or schist. The coarse grained texture and presence of considerable mica and quartz along with calcite would appear to give good skid resistance. Section 9c from Dominion shows the typical dense high calcium limestone from the same source as the aggregate in Figure 3. This very fine grained texture coupled with the lack of particles of differing hardness apparently is largely responsible for the high polish this material will quickly attain under traffic. Section 9b from Perry shows an intermediate partially banded texture and composition which would easily account for its intermediate behavior in road surfaces. Insoluble residue analyses of the samples a, b and c from these three sources yielded averages of 35.04 per cent, 17.55 per cent and 1.96 per cent, respectively. The grain size of the insols also differed significantly, with Rockydale having predominantly a silt to sand size particle, Perry yielding some silt and much clay, and Dominion virtually all clay.

Field wear of aggregates from the three quarries under consideration was simulated by mounting asphalt mixtures on a specially designed wheel and turning this wheel by contact with a motor driven rubber tired wheel. Water and sand abrasive were constantly fed at the point of contact. Figures 10 a, b and c are close up photos of the three aggregates under discussion taken after approximately 96 hours of wear. The Rockydale material (Figure 10a) maintained a rough, harsh surface throughout the test. Wearing characteristics were very similar to those of the granite gneiss shown in Figure 6.

Aggregate from the Perry quarry is shown in Figure 10b. The composition bands evident in thin section are reflected in the worn surfaces. Bands containing concentrations of silt size insolubles wore to a sandpaper-like finish. The remainder of the rock polished. This presence of coarse grained insoluble material was concluded to be of fundamental importance. This conclusion is also borne out in the work of Gray and Renninger (1960, 1965) of the National Crushed Stone Association.

The high calcium Dominion stone (Figure 10c) as would be expected polished readily and attained a surface texture devoid of any irregularities which might contribute to skid resistance. The similarity with the fine grained, high calcium aggregate in Figure 3 is obvious.

RESUME AND CONCLUSIONS

With the rapid increases in traffic volume and vehicle speed on modern highway pavements the need for maintaining adequate skid resistance is becoming more acute. Subsequent to World War II statistics were compiled in Virginia which indicated that not only was the rate of skidding accidents high but in some areas of the state the rate of this type of accident was nearly double that in other areas. This gave rise to an intensive research effort into pavement slipperiness which has resulted in the following conclusions:

1. In bituminous pavements the most important factor influencing the coefficient of friction is the type of coarse aggregate used.

2. In virtually every case, non-carbonate aggregates (granite, diabase, sandstone, quartzite, altered basalt, quartz gravel, etc.) have given adequate skid resistance based on a minimum coefficient of friction of .4 on wet pavements.

3. The average coefficient of friction value for pavements containing carbonate aggregates was significantly below all others tested and was below the minimum required in Virginia.

4. A large variation in coefficient of friction exists among carbonate aggregates.

5. Studies of selected limestones and dolomites which show a wide range in skid resistance behavior have yielded the following:

a. Composition of the carbonate minerals is relatively unimportant.

b. The gross amount of insoluble residue contained in the rock is of only minor importance.

c. Texture and particularly the presence or absence of silt to sand sizes in the insoluble residue is thought to be of primary significance.

6. Finally, the concept of "surface renewal" is offered to explain the high skid resistance provided over extended periods by medium to coarse grained, polymineralic rocks containing constituents of widely varying hardness.

TABLE 1

SUMMARY OF COEFFICIENT OF FRICTION DATA OF VIRGINIA PAVEMENTS BASED ON MAJOR AGGREGATE TYPES

Aggregate Type	No. Readings	No. Sections	Range	Average
Limestones & Dolomites	1298	101	.16 — .62	.369
Granites	114	10	.43 — .68	.525
Diabase	115	7	.35 — .59	.476
Gravels	34	4	.43 — .52	.474
Sands (F-1)	270	23	.46 — .72	.572
Sands (Deslicking)	106	11	.53 — .85	.674

TABLE 2

SUMMARY OF COEFFICIENT OF FRICTION DATA FOR LIMESTONE AND DOLOMITE PAVEMENTS IN VIRGINIA

Source	No. Readings	No. Sections	Range	Average
Rockydale & Blue Ridge (Lynchburg)	47	3	.46 - .62	.538
Blue Ridge (Roanoke)	164	12	.27 - .59	.447
Betts	26	2	.35 - .59	.439
Elkton	57	4	.26 - .53	.433
Acme	19	1	.36 - .48	.417
M. J. Grove (Md.)	33	2	.34 - .52	.416
Belmont	10	1	.31 - .50	.392
Pendleton	66	4	.30 - .52	.392
Holston River	27	3	.32 - .44	.388
Riverton	43	4	.24 - .45	.370
Perry	128	10	.26 - .43	.369
Augusta	20	1	.23 - .45	.360
Rockydale (Roanoke)	93	6	.24 - .50	.359
Liberty	41	4	.26 - .43	.356
Mundy #2	27	2	.27 - .40	.350
Virginian	47	4	.27 - .45	.346
Pounding Mill	83	7	.21 - .45	.342
Barger	99	8	.24 - .43	.342
Frazier	27	2	.26 - .41	.317
Mundy #1	14	1	.27 - .35	.312
Radford	100	8	.21 - .39	.308
Sword's Creek	25	1	.18 - .40	.296
Pope	9	2	.21 - .32	.288
Pembroke	12	1	.25 - .30	.277
Dominion	60	7	.16 - .36	.241
Lambert	21	1	.19 - .30	.238
Totals	1298	101	.16 - .62	.369

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LANDSLIDE RESEARCH

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Landslides in the Turnagain Heights section of Anchorage and in other parts of Alaska caused by Anchorage and in other parts of Alaska caused by the Good Friday earthquake on March 27, 1964 (1)[1], and the landslide at the Vaiont Reservoir in Italy on October 9, 1963 (2) have focussed the attention of the public on the great damage to life and property that can be caused by landslides. Such catastrophes also result in a public demand that the cause be detemined, and that appropriate measures be taken to prevent their recurrence.

INTRODUCTION

Highway engineers have always been concerned about the cause and prevention of landslides, as well as the highway location, design and maintenance methods to minimize the detrimental effects of such movements. Several Highway Geology Symposium papers have been presented on various aspects of the subject. Eckel gave a summary of landslide studies at the 1957 Symposium (3). He stated that "we need to know more about when and where the next slide is going to occur and we need to know more exactly what treatment to apply to a given slide, potential or actual." He recommended that more studies of actual slides, and component materials and conditions, be made and that engineers and geologists report more fully the investigation of structures that have failed due to faulty engineering design against landslide action.

It is the aim of this paper to present some of the results of landslide studies recently completed as well as current research on the subject, particularly as it applies to the location, design, construction and maintenance of highways. One definition of landslide (4) is the "downward and outward movement of slope-forming materials—natural rock, soils, artificial fills, or combinations of these materials." This definition permits the discussion of a great variety of studies of detection and movement of earth or rock materials. The paper is primarily concerned with landslide research by (a) State highway departments utilizing Federal highway planning and research funds and (b) the Bureau of Public Roads. However, some space is also devoted to engineering work not generally considered to be research, as well as investigations by other agencies.

[1]Number in parenthesis refers to reference at end of paper.

REGIONAL LANDSLIDE STUDIES

Highway location and design engineers have learned that a study of available geologic maps and reports, aerial photographs and similar office sources of information, supplemented by extensive ground investigations, is needed for selection of the best highway route in a slide-prone region. A knowledge of geology and other environmental consitions is essential. Cultural and economic factors may dictate that a specific highway be located in an area that has landslides or is susceptible to sliding. In such areas, the highway route should avoid the slides; if that is not possible, considerable field and laboratory investigation may be necessary for proper design.

Some regions, e.g., portions of the St. Lawrence valley having thixotropic marine clays, and the western portion of the Alleghenies from Kentucky to Pennsylvania, have been extensively studied. An interesting historical point is that Ladd (5) reported on the regional landslide problem in West Virginia in 1927, while Baker (6) reported on a rather extensive landslide investigation in such materials in a paper at the 1956 Geology Symposium. Ladd stated that the following conditions contribute to this regional landslide problem: "the nature and composition of the two principal rock materials; the vertical joints in the sandstone layers; the massive and impervious character of most of the shales; the dip of the strata (even though very slight); the characteristic topography of the region; and the abundant rainfall, averaging 43 inches a year." He then identified four principal types of landslides in the region, one of which was in detrital masses lubricated by wet clay, as reported by Baker.

At the 1963 Symposium, Leith and Gupton (7) reported on regional landslide investigations in central and western North Carolina. The distribution of landslides in specific parent rock types and soil series was determined, and further research was to be done to relate such information to quantitative analysis of conditions on specific highway routes. It is now planned that the research be completed in 1966.

A recently completed study in Arkansas (8) had the objectives of developing information that would assist highway design and maintenance engineers to recognize slide-prone areas and to remedy actual slides.

A current South Dakota study aims to identify the geological units that are alide prone and to prepare a manual concerning (a) the recognition of potential landslide areas and (b) design methods to prevent detrimental sliding.

MOVEMENT

It is sometimes important to detect initial movement of the land mass or the increased rate of movement in existing slides. Such knowledge may be useful in maintenance or design of highways, or may permit remedial measures to be taken before damage to the highway, other property, or life occurs.

While the Bureau of Mines has used seismic apparatus for several years to measure small disturbances in rock masses, the value of such apparatus in highway landslide investigations has only recently been utilized by research engineers, notably in California (9) and North Carolina (10). In the California work, these small, low-frequency disturbances or rock noises were given the designation, SARN (sub audible rock noises), while in North Carolina and in previous work by the Bureau of Mines they were called microseismims.

Essentially, the appratur consists of a geophone, amplifier, and a means of monitoring the signals received.

In California, apparatus developed by earlier investigators was modified and improved for use in field studies of slides in 16 highway cuts involving a variety of rock formations. Further modification of the apparatus was made as the field work progressed. Their report concluded that: (a) the apparatus was capable of detecting and recording noises in the ground caused by active landslides, but that true rock noises must be identified and isolated from extraneous sources of noise, (b) while it is theoretically possible to determine the focus of noises, their use of a 4-channel instrument (with probe dectors at 4 different locations in the ground) was not very successful in determining the exact focus of the noise; and (c) although further work will probably permit determining the focus of noises in landslides in uniform, hard-rock formations, determining the focus of noises in heterogeneous clayey slide debris seems hopeless. Until apparatus for such focus locus location can be perfected, the researchers recommended for highway use a 2-channel apparatus for identifying rock noises in landslides. The probe dectors should be placed below the water table in drilled holes, and in shallow measurements the holes should be covered to exclude air noises.

The North Carolina researchers used apparatus loaned by the Bureau of Mines. They first made laboratory tests to determine that the rock noises are associated with shearing forces in the rock, then determined the best method of making proper contact of the geophone probe to the soil or rock mass, and determined the ground distance through which the noises could be heard. Investigations were subsequently made in landslide areas. The overall conclusion was that the difference in count rate between unstable and stable materials in the same landslide area indicates that the microseismic rate can be related to slope stability.

A slope indicator, involving an electronically actuated pendulum fitted into grooved plastic casting in a borehole, has been used successfully in measuring horizontal deflections or movements of the ground (11). In practice, the insturment is mounted on wheels that ride in grooves in the casing, with one pair of grooves being referenced to the direction that measurements are desired, and the other at right angles to it; dial readings are taken at frequent depth intervals; dial readings are converted mathematically to deflections, and these are referenced to some fixed point, so actual movements may be established.

An automatic time-lapse motion-picture camera has been used by the U. S. Geological Survey (12, 13) to measure the movement at a specific location in a landslide.

DETAILED STUDY OF LANDSLIDE

A detailed study of the slide is usually necessary, in order to determine the extent and cause and to plan corrective measures. A long-time study of the slide area may be useful in order to properly understand the mechanics of the ground movement. Geologists have been making observations on the Slumgullion slide near Lake City, Colorado, for about 100 years (13). It is estimated that the slide is 700 years old. Estimates of recent ground displacements have been made by reference to aerial photographs taken in 1939 and ground stakes placed in 1958, and by use of an automatic time-lapse motion picture camera in 1960.

Knowledge regarding the pattern of ground water flow is needed for determining the best method of draining the slide area or intercepting the flow of water. Landslide research in Kentucky and Idaho includes the detection of the source or location of the ground water causing the sliding. Fluorescent (and perhaps other) dyes and radio-active trace elements, such as tritium, are planned or being used in both studies.

The Bureau of Public Roads has reported (14) the successful use of fluorescent dyes to determine the source of seepage water in highway construction in the Great Smoky Mountains National Park. Several colors of dye may be required on a single project, in order that the dye may be injected into several possible water sources and observed at the point of seepage. Introduction of the dye in daylight and subsequent survey of the seepage area at night by means of a portable ultra-violet lamp has been recommended.

Tritium (Hydrogen 3) has been the suggested radioactive trace element because it has only slight toxicity (low beta energy), its biological transfer is very high or rapid, and it is exempted from licensing by the Atomic Energy Commission.

At the 12th Symposium, Moore (15) described the use of electrical resistivity apparatus by the Bureau of Public Roads in landslide studies, and Idaho, Montana, Ohio and South Dakota are using resistivity apparatus in current landslide research studies. Excess water at a slip surface often produces a downtrend in the cumulative resistance curve at the depth corresponding to the thickness of the moving material. Thus, in Fig. 1, the slip surface in inferred to be at a depth of 34.6 feet. This resistivity information should be used to supplement boring data or to indicate where borings should be made, inasmuch as resistivity tests can be made much more rapidly and economically than borings.

Moore also reported at the 12th Symposium some "stray potential" research being done by Public Roads in conjunction with other resistivity surveys. Supporting data are still being collected in landslide investigations. The hypothesis is that excess water at the slip surface (or at the interface between two differing geologic layers) produces measurable natural potentials. The resistivity apparatus used is the milliammeter-potentiometer type, making use of direct current, which is alternated once normally when obtaining current and potential readings.

PRESPLITTING AND CONTROLLED BLASTING

As stated earlier, the definition of landslide permits the discussion of a variety of rock mass movements. On many highways, there is a considerable accumulation of debris at the back of backslopes. This may be due partly to weathering of the rocks, stratification and natural fractures or joints. However, much of the accumulation of rock debris subsequent to highway construction is due to the specific construction method. Detonation of large quantities of explosive near the proposed cut face may result in (a) excavation behind the proposed cut surface, (b) shattered rock in the exposed backslope that causes unnecessary maintenance and endangers the highway user, and (c) a ragged, unesthetic rock slope. (Figure 2).

Considerable attention is being given by highway engineers and explosive manufacturers to the use of controlled blasting or pre-splitting methods that will result in clean, smooth backslopes, in un-shattered rock.

Pre-splitting involves the drilling of small-diameter, closely spaced holes in the plane of the proposed rock cut slope, and firing the optimum amount of explosive to produce a clean break at the proposed cut face. Blasting of the rock in the principal portion of the proposed rock cut is then accomplished by normal means. A good knowledge of the nature of the rock, together with some experience in this blasting method, are needed in order to determine the proper size and spacing of the holes and the optimum explosive charge. A normal specification requires that the drill holes be 2½ in. diameter, spaced 2 ft. center to center. (Figure 3).

The following quotations from a Kentucky "Special Provision for Presplitting" may be of interest

"Excavation of rock by use of explosives shall be done in such a manner as will result in a minimum of breakage outside the neat lines of the typical cross-section as staked by the Engineer. Faces of cut slopes through rock shall be formed by pre-splitting. Pre-splitting is defined as the establishment of a free surface or shear plane in rock by the controlled usage of explosives and blasting accessories in appropriately aligned and spaced drill holes. Drilling and blasting for pre-splitting shall be kept well in advance of normal blasting operations.

"Drill holes for pre-splitting shall be made along the slope stake lines established by the Engineer, and the Contractor shall exercise sufficient care to insure that the holes conform to the slope as established.

"The pre-split face shall not deviate more than 6 inches from the front of the line of drill holes, nor more than 1 foot from the back; except where the character of the rock being pre-split (badly broken rock, vertical seams, etc.) will unavoidably result in irregularities."

Other controlled blasting methods, such as line drilling, pre-shearing, cushion blasting, perimeter blasting (in tunneling), are also used. Their principal purpose is to outline the proposed cut face and prevent the main blasting from damaging that face.

While the overall cost of blasting may be increased by use of the pre-split and other controlled blasting methods, the contractor's cost may be offset by having the excavation conform to the design slope, rather than having to remove excessive material due to overbreakage. In many types of rock cuts, presplitting will result in reduction of slope maintenance costs, may greatly improve the estheties of the slopes and result in greater safety to the traveling public.

The Bureau of Public Roads is making a survey of pre-splitting and other controlled blasting practices being used by the State highway departments, Federal agencies, and others. Information on geologic investigations, specifications, blasting techniques, costs, and other items of interest is being obtained. It is anticipated that a report will be prepared in 1965.

SUMMARY

Soils engineers and geologists with adequate experience are capable of making appropriate field investigations of landslides or slide-prone areas, and such information will normally be adequate for proper design to prevent detrimental sliding in the highway slope subsequent to construction. However, maximum advantage should be taken of apparatus and methods that will result in an adequate field investigation, but at minimum time and cost. Construction methods should be controlled so the slopes are constructed in accordance with the design, and thus prevent excessive maintenance of slopes subsequent to construction.

Further research is needed in (a) the detection of ground water movements by means of dyes and radioactive tracers, (b) rapid but adequate means of delineating the slip surface, and (c) detection of landslide movement that requires immediate corrective action or action to prevent injury to the highway user.

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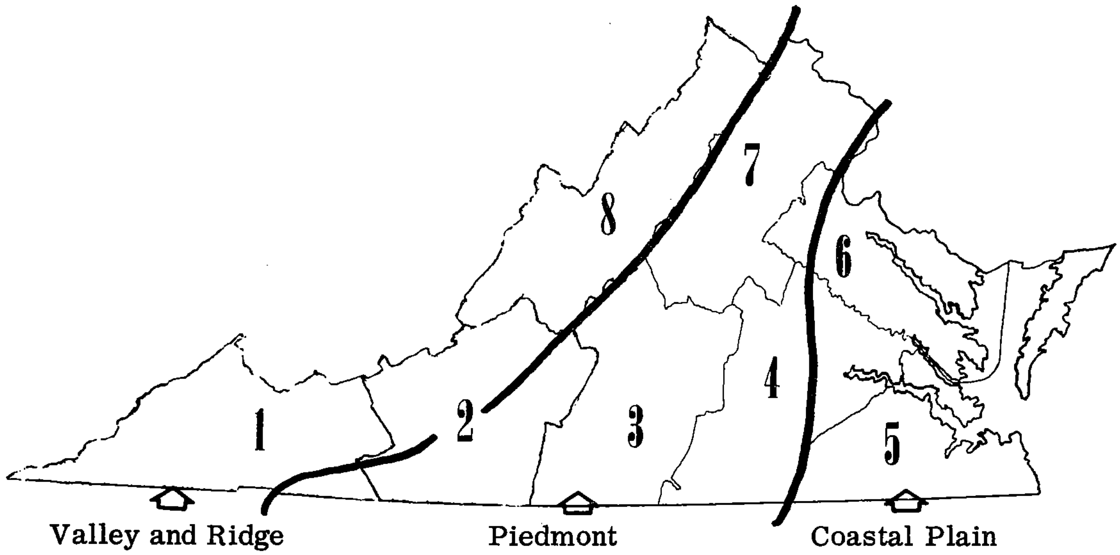


Fig. 1.—Outline Map of Virginia Showing Highway Districts of Physiographic Provinces.

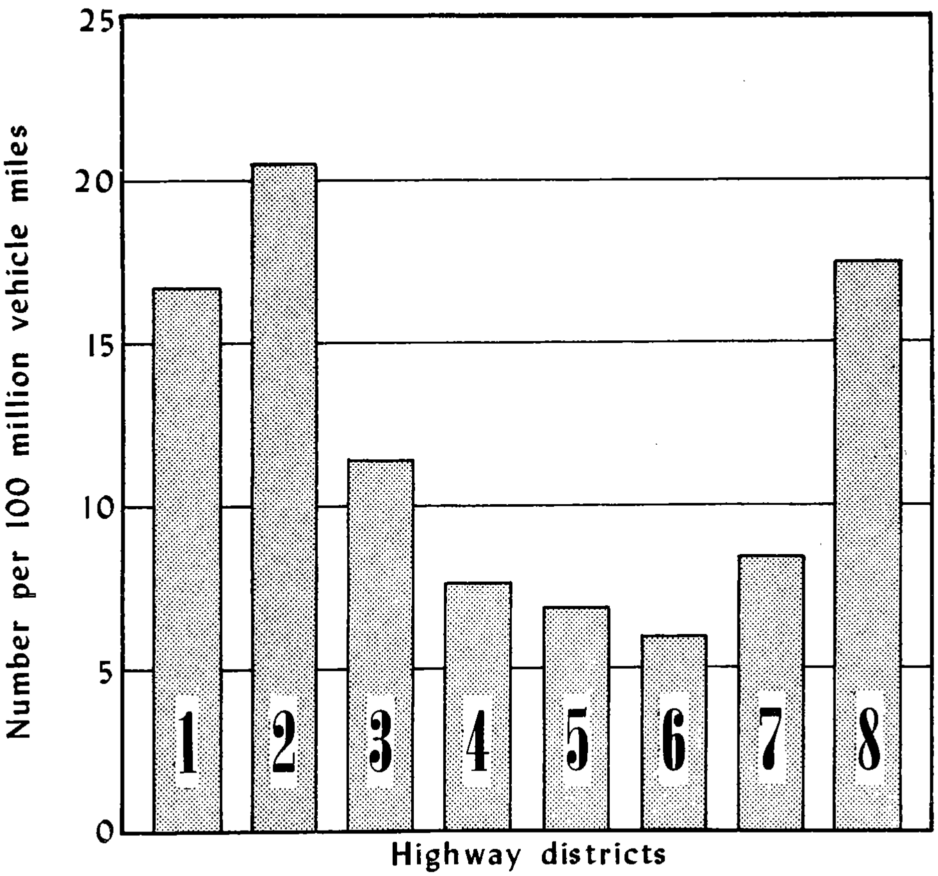


Fig. 2.—Skidding Accident Rates in Virginia.

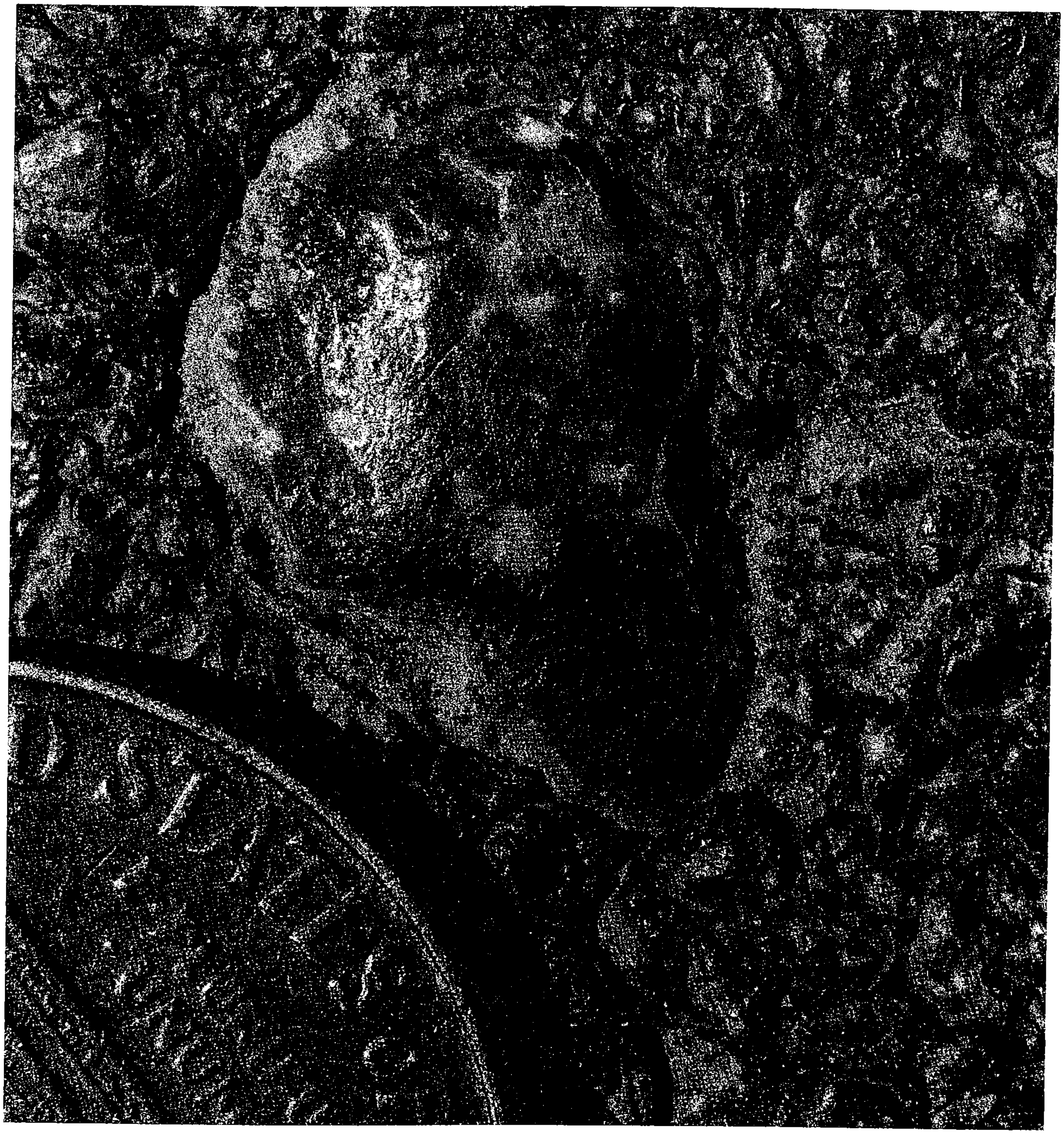


Fig. 3.—Fine Grained, High Calcium Limestone Aggregate. Note Penny for Scale.

Fig. 4.— Impure Dolomitic Limestone Aggregate.

Fig. 5.— Triassic Siltstone Aggregate.

Fig. 6.— Micaeous Granite Gneiss Aggregate. Note Paper Clip for Scale.

Fig. 7.— Non-polishing Quartzite Coarse Aggregate with Limestone Fines.

Fig. 8.— Altered Basalt Aggregate in a Pavement Where Flushing of the Asphalt Binder Has Occurred.

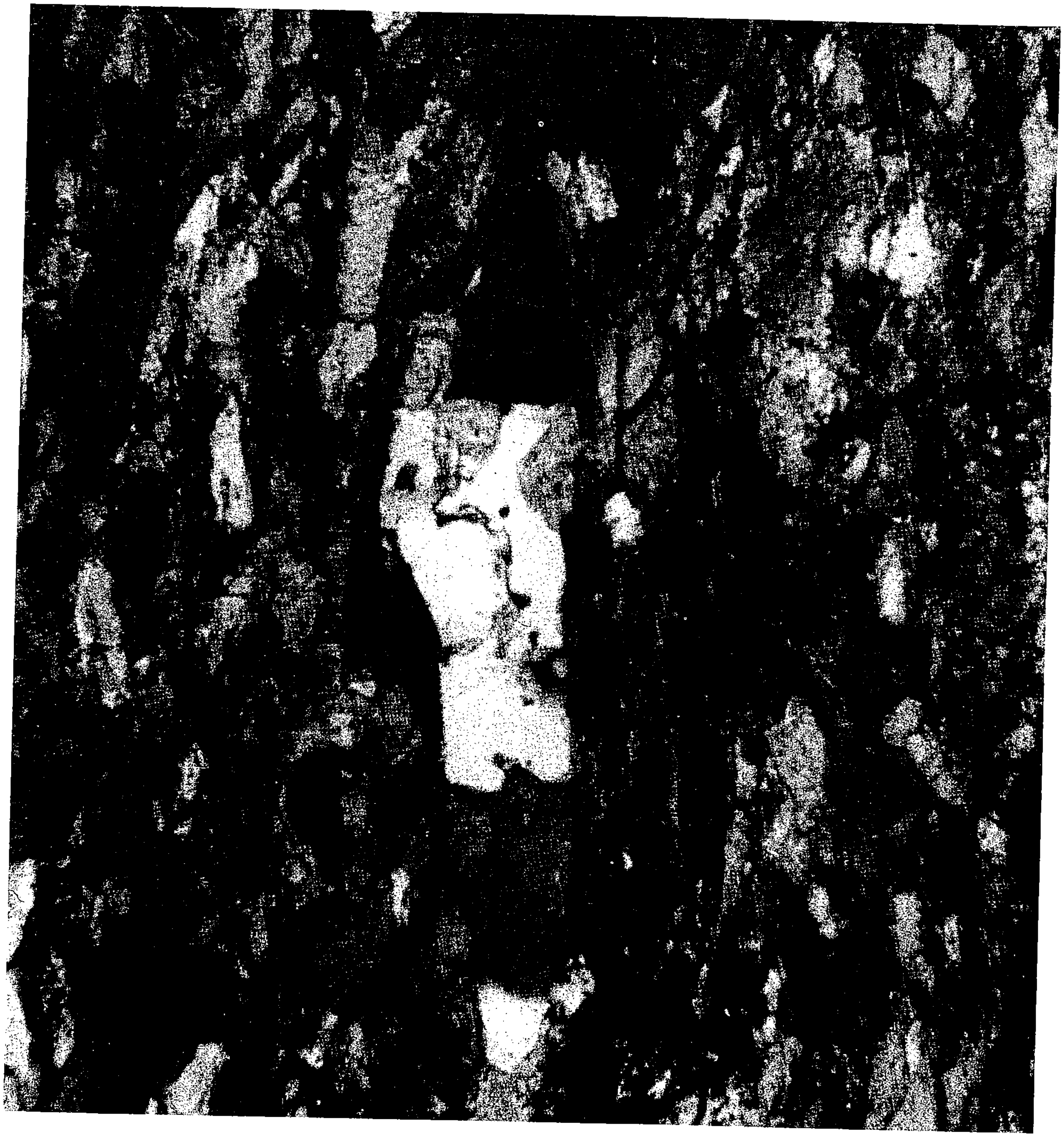


Fig. 9.— Photomicrographs of Carbonate Aggregate Types.

(a) Coarse Grained, Impure Marble.

(b) Medium Grained, Impure, Dolomitic Limestone.

(c) Fine Grained, High Calcium Limestone.

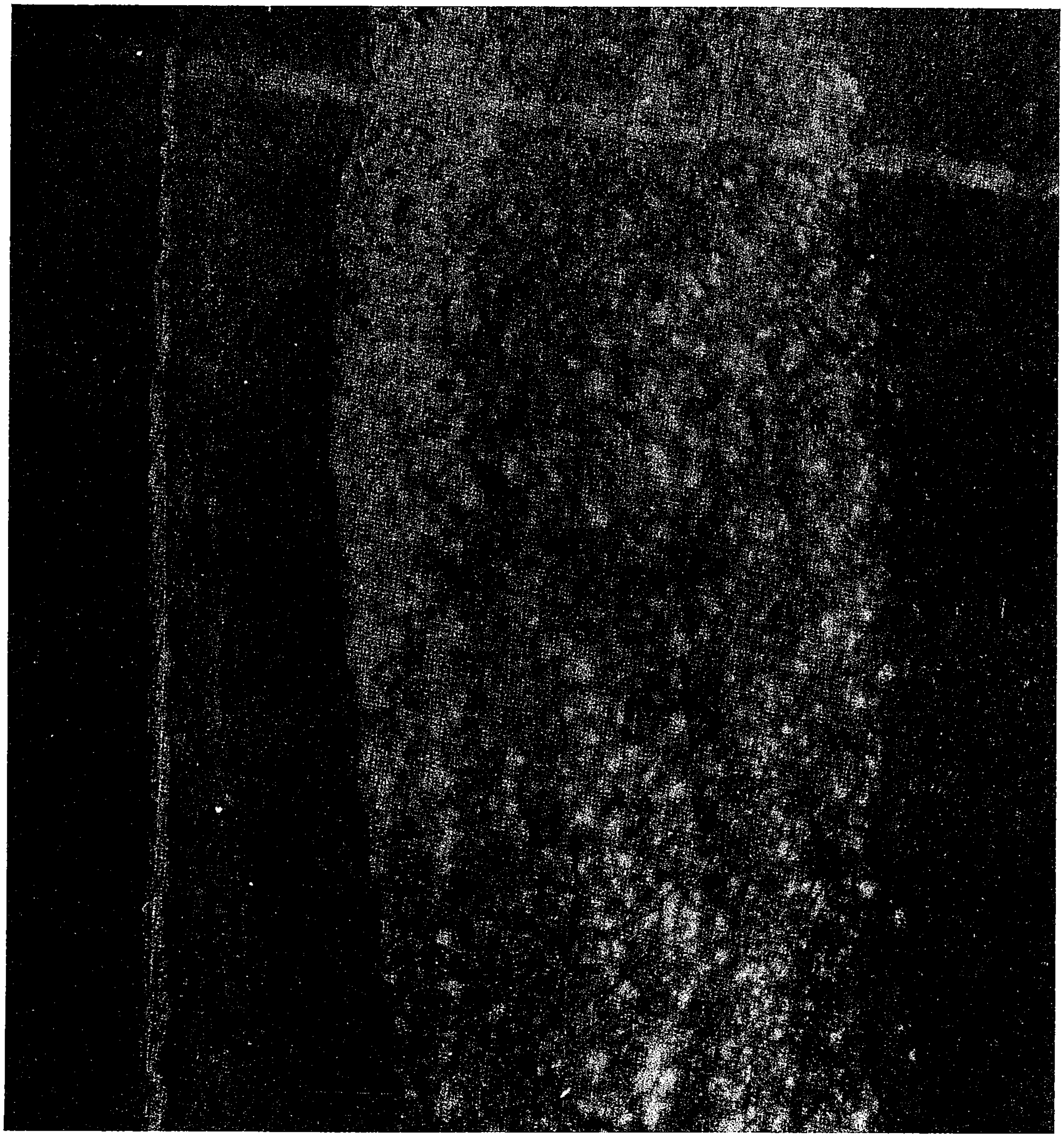


FIGURE 9-B

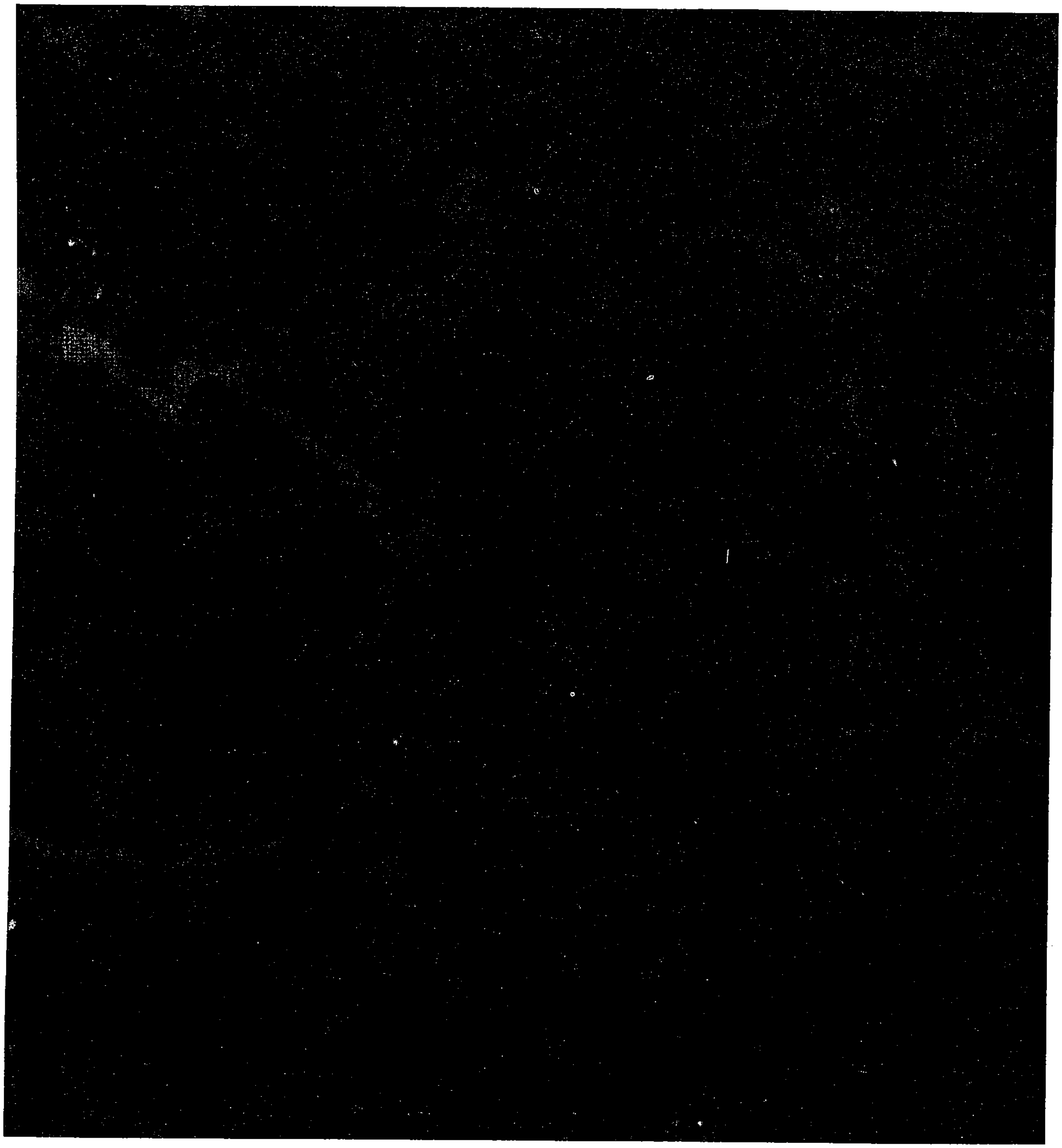


FIGURE 9-C

Fig. 10.— Laboratory Simulated Field Wear of Carbinate Aggregates Shown in Figure 9.

(a) Coarse Grained, Impure Marble.

(b) Medium Grained, Impure, Dolomitic Limestone.

(c) Fine Grained, High Calcium Limestone.

FIGURE 10-B

FIGURE 10-C

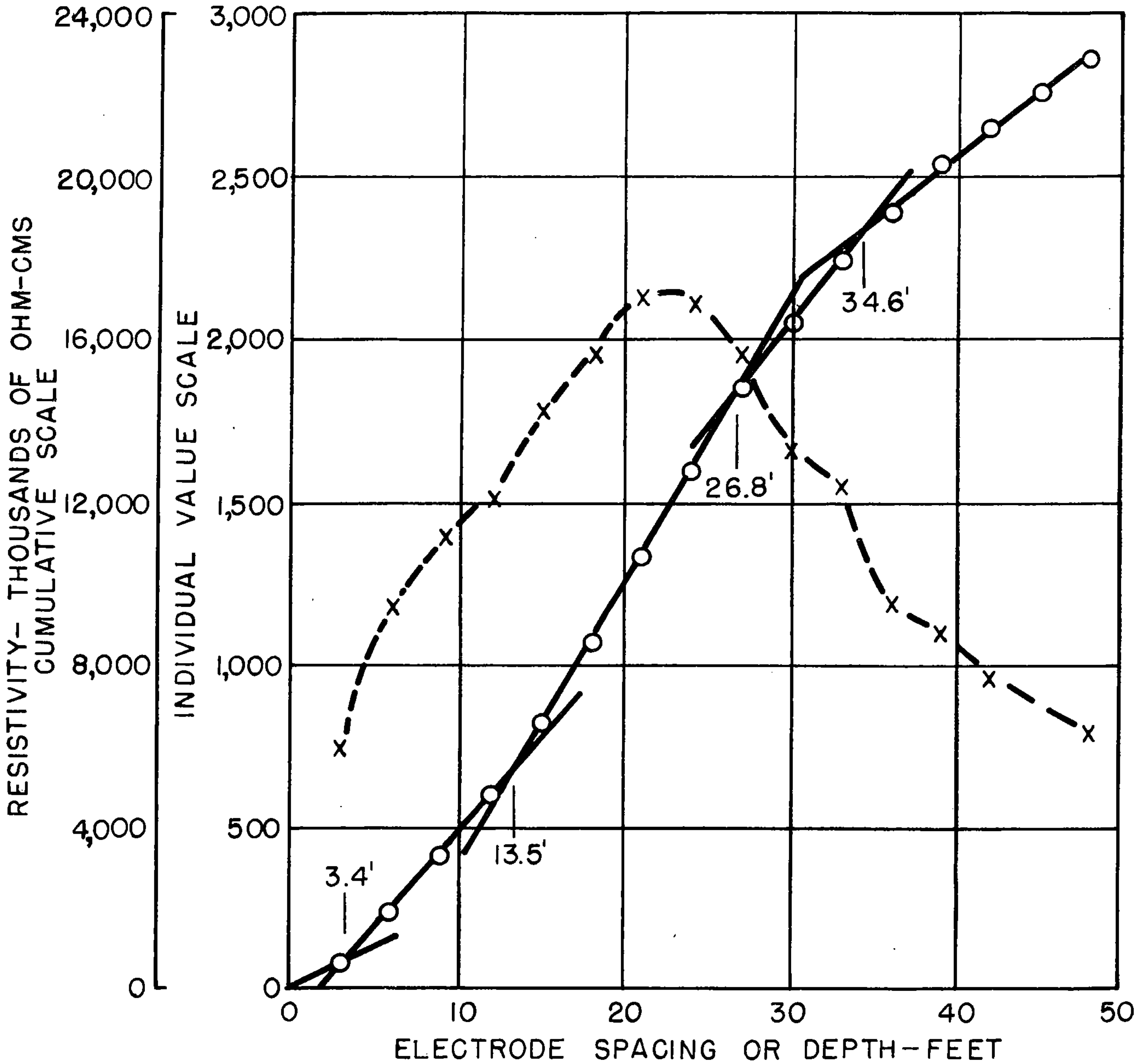


Fig. 1.— Resistivity Curves, Blue Ridge Parkway Project. Downward Trend in Cumulative Resistivity at Slip Surface. (15)

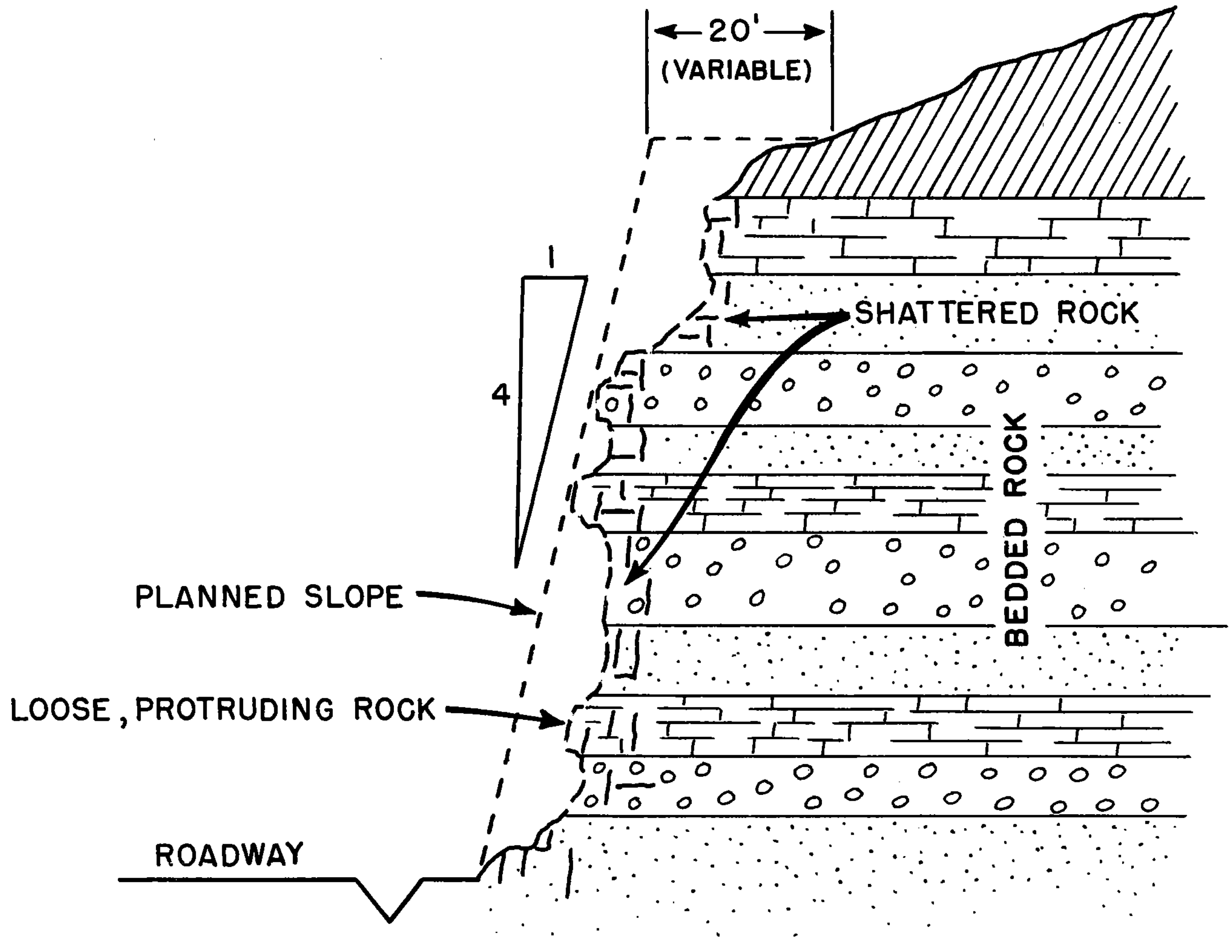
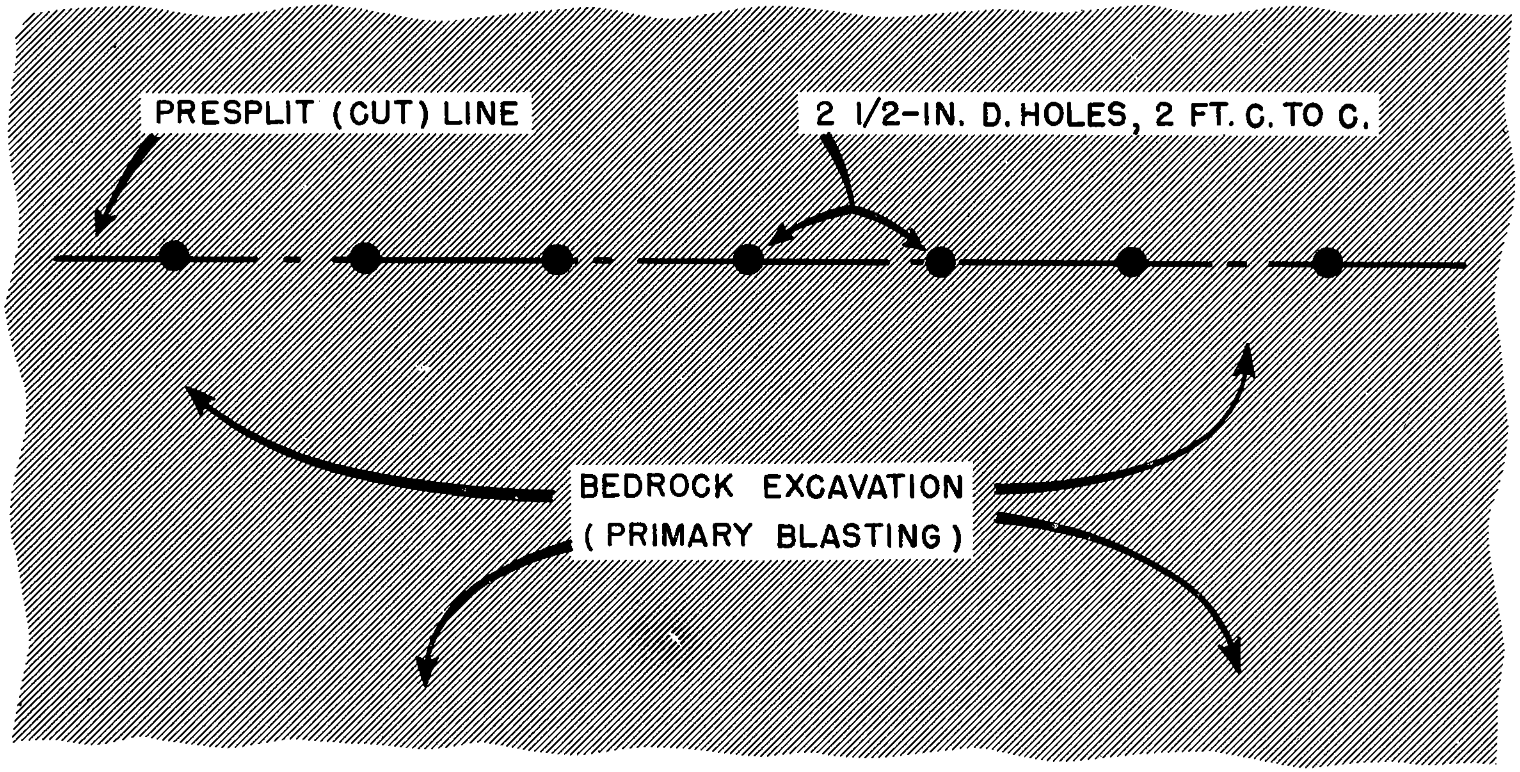


Fig. 2.— Typical Rock Slope where Presplitting Was Not Used.



Note: Presplit holes are drilled to conform to the slope line for rock excavation. If benching of slope is required, parallel lines of presplit drill holes are necessary.

Fig. 3.— Drill-hole Pattern for Presplit Blasing.

SHALLOW SUBSURFACE EXPLORATION UTILIZING AIRPHOTO INTERPRETATION AND GEOPHYSICAL TECHNIQUES

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For many years airphoto interpretation has been utilized by highway engineers as a tool for procuring soil and bedrock information useful in preliminary highway design. The tool has not had complete acceptance because of the limitation of the method to provide detailed shallow subsurface layering, the depth to bedrock, the bedrock composition and stratigraphy, and the structural and surface attitude of the bedrock.

In *Highway Research Bulletin,* vol. 316 (1962), the results of a recent questionnaire on subsurface exploration methods sent to all Highway Departments in the United States are tabulated. In answer to the question regarding the use of aerial photographs as a method of subsurface exploration for soil and ground information, of 47 departments responding, 21 answered in the affirmative, 26 in the negative, and one gave no answer. This recent questionnaire emphasizes the current partial acceptance of the airphoto method for subsurface exploration by Highway Departments throughout the United States.

The limitations of airphoto interpretation in shallow subsurface investigations can be overcome to a large degree by the utilization of geologic interpretation and geophysical techniques. Geologic interpretation is not to be confused with the excerpts from general geologic and pedologic reports which so often accompany preliminary engineering reports and airphoto interpretations. Geological interpretation is the analysis of available geologic data in an area or the projection of geologic information from areas of detailed geologic control to areas where geological information is limited or even non-existent. Most highway engineering projects generally fall within areas of limited or non-existent geologic control, so geologic interpretation is a necessary part of the subsurface investigation. Geologic interpretation requires the interpreter to be familiar with the principles and processes of stratigraphy and sedimentation as well as equipped with the essential background of structural and physical geology, geomorphology and pedology. A great deal of surface and subsurface information may be inferred if the interpreter is familiar with the processes associated with the transportation and deposition of sediments. What appears, in some cases, to be an area of complex stratigraphy may prove to be a regular sequence of lateral facies changes associated with an ancient shore line. For this reason an interpreter should have a good understanding of the regional paleogeography of his area of interest in order to fit the local environmental facies relationships into the regional stratigraphic picture. Knowledge of depositional strike is important because litholgic continuity generally occurs parallel to the depositional strike, while rapid facies changes occur in the direction of the depositional basin. This type of geologic knowledge makes it possible to project the bedrock geology along depositional strike from an area of dense well control or surface outcrops to areas where control is limited. The interpreter should be familiar with nonmarine depositional environments as well as marine. He should be familiar with the geometry and textural distribution of the sediments in the straight and meandering courses of the transporting streams and with the textural limits possible for localized stream deposits. Knowledge of the textural limits and the textural distribution of sediments in stream channels makes it possible to infer the subsurface stratification, distribution, and the textural limits of the sediments occurring on the floodplains of mature and old age streams. Knowledge and recognition of landforms, erosional or depositional, and their origin, permits inference as to the general texture, composition, stratigraphy and depth of the associated engineering soils.

If the photo interpreter would use geologic interpretation, rather than merely collect the available geologic data for a particular area of interest, the quality of the photo interpretation would improve.

The recommended proceedures for subsurface investigations by photo interpretation are first to collect all geologic data for the project area. If sufficient data is available for the area the data should be analyzed and a geological interpretation made. The geologic interpretation should include analysis of all topographic, geologic, and pedologic maps available, analysis of all pertinent geological and pedological reports, and a study of water well logs available in the project area. When little or no information is available for the project area, geologic or pedologic investigations of adjacent areas should be analyzed to determine, if through geologic interpretation, geologic and pedologic information can be projected into the area of interest. All geologic data gathered through collection or interpretation should be recorded prior to the airphoto analysis.

The airphoto analysis should consist of a study of small scale photography in mosaic and stereoscopic form. The small scale (1:20000) mosaic study provides an areal or regional concept of the terrain, and through analysis of the photo patterns, permits the general delineation of the major landform subdivisions. The stereoscopic study of the small scale photography permits the identification and possible origin of the landforms, the finer definition of the landform boundaries, and serves as a base for the construction of a landform and drainage map. Through landform identification the general texture, distribution, composition and stratigraphy of the soil can be inferred and through analysis of the surface drainage pattern, the structural attitude of the bedrock may be inferred if the overburden is thin. The large scale (1:9600 or 1:2400) photo study, through analysis of the gully characteristics, photo tones, land use, vegetation, erosional features and bedding, provides more detailed information as to the composition and texture of the soil and rock and the structural attitude of the bedrock. It also provides delineation of exposed soft subsoil deposits, the delineation of existing landslides, and identification of landslide susceptible terrain.

At this stage of the subsurface investigation the interpreter possesses general information relating to the aerial distribution, texture, and stratigraphy of the soil and bedrock, but lacks specific information as to the depth of the soil and rock stratigraphic disontinuities, and an accurate depth to bedrock. To overcome the subsurface limitation of the airphoto methods, geophysical methods can be utilized to verify the photo interpretation inferences and to compliment the photo interpretation by adding detailed information related to the depth of soil and bedrock discontinuities.

In the electrical resistivity method a current is introduced into the ground through two current electrodes and the potential drop between two potential electrodes is measured. The most common electrode configuration is to place the four equally spaced electrodes in a straight line, the outer electrodes being the current electrodes and the inner the potential electrodes. The electrical resistivity of hemispheres of soil and/or rock is measured, the size of the hemisphere being determined by the spacing of the electrodes on the surface of the ground. The depth of investigation is increased as the electrode spacing and size of the hemispheres increases. As successively larger and larger electrode spacings are utilized at one station, it is possible to interpret depths to discontinuities between earth materials which possess different electrical properties. The primary factors affecting the electrical properties of a soil are the amount of moisture and the amount of desolved salts in the water.

Through utilization of geologic interpretation and a few necessary calibration borings it is possible to determine the depths of soil and rock materials with contrasting electrical properties and draw inferences as to the texture, composition, and distribution of the subsurface soil and rock materials.

The seismic refraction method is also a valuable tool for shallow subsurface investigation. In seismic refraction surveys in the travel-time of artifically produced seismic waves is measured between the shot hole and seismic detectors placed along the earths surface. Since the distance between the shot hole and the detectors can be measured, it is possible to compute the velocity of the seismic waves transmitted by the earth materials. The seismic velocity of earth materials depends upon the elastic moduli and the density of the materials.

Seismic measurements allow the calculation of the depths to interface between materials having different seismic velocities and since the velocity is dependent upon the elastic properties and density of materials, it is possible to infer the texture and distribution of the subsurface materials.

The combination of airphoto interpretation techniques and geophysical methods, over a section of a proposed relocation of USR 40 in Guernsey County, Ohio, will illustrate the usefulness and merits of the combination in preliminary highway design. The alignment segment is one of several areas studied under the Ohio State University, Engineering Experiment Station Project, 196 with which I have been associated for the past three years. The research is being jointly sponsored by the Ohio Department of Highways and the Bureau of Public Roads. A portion of the data presented here appeared in the Annual Report of EES-196 entitled *Terrain Investigation Techniques for Highway Engineers* (Mintzer, 1962).

Geographically the test area is located in Wills Township, Guernsey County, Ohio between the villages of Old Washington and Middlebourne. Physiographically the test area is located in the Allegheny Plateau province and is predominantly in slope except for the floodplain development of Salt Fork Creek and its tributaries.

The geologic interpretation of the test area was obtained by evaluating and analyzing data collected from water wells, geologic and topographic maps, the geologic and pedologic literature pertinent to the area, and by field reconnaissance. The major landform subdivisions, upland residual and floodplain, were easily delineated on the topographic maps and verified by field reconnaissance. The upland residual terrain in the study area extended from alignment stations 289 to station 303. Floodplain alluvium terrain extended from stations 264 to 289 and from station 303 to station 310.

Horizontal or nearly horizontal shale bedrock with thin stringers of sandstone and limestone was established in the upland residual terrain through field reconnaissance and water wells. The upland residual soils were inferred to consist of silts and clays developing from the underlying Conemaugh shale. The soil throughout the upland residual landform was inferred to be less than 10 feet thick but to vary in thickness with changes in slope. Soil depth along the crest of the ridges was estimated to be greater than 5 feet but less than 10 feet with a silty textured "A" horizon and clay textured "B" horizon. Slope-phase soils were estimated to be less than 5 feet thick and consist of mainly clay textures. Colluvial soils, occurring on the lower slopes, were anticipated to be greater than 5 feet in thickness and composed primarily of silt textures eroded from the crests and upper slopes of the hills.

The floodplain sediments were inferred to be thinly interbedded, discontinuous layers of clay, silt and sand with some local accumulations of rock fragments. The textural inference is based upon the knowledge that the Salt Fork Creek drainage area is restricted to rocks of the Conemaugh and Monongahela formations in which the clastics do not exceed sand size textures. The thinly interbedded, discontinuous nature of the floodplain sediments is characteristic of alluvial deposits. A gradation in floodplain textures from coarser, near the center of the valley, to finer, towards the valley margins, was also anticipated through geologic interpretation.

Bedrock beneath the floodplain sediments was inferred as shale but the depth could not be established.

A photo interpretation of the study area was conducted following the proceedures already outlined. Stereoscopic analysis of the large and small scale photography delineated the two major landform boundaries and generally delineated the distribution of the colluvial deposits at the base of the upland residual slopes between alignment stations 283 and 289 and between stations 303 and 305. The analysis of the gully characteristics verified the geological interpreted textures of the upland and floodplain soils. A photo interpretive profile was developed utilizing all information gathered by geologic interpretation and photo pattern analysis (Figure 1.). This profile indicates the limitation of the photo interpretation technique. Only general information regarding the composition and the distribution of the soil and rock are shown. The lateral changes in texture, inferred from the geologic interpretation, are indicated at stations 277 and 305 but the boundary is questioned and the nature of the lateral discontinuity is not know. The boundary between the colluvial and floodplain sediments is also in question and nature of the boundary is again not known. The soil-bedrock interface is questioned throughout the profile.

Geophysical surveys, electrical resistivity and seismic refraction, were conducted over the study area to obtain detailed soil layering and rock information. One boring in each landform subdivision was selected for calibration station in order to determine what stratigraphic discontinuities were reflected by contrasts in resistivity values and contrasts in seismic velocities. The resistivity profile, Figure 2, was developed from information obtained from eight resistivity stations along the test segment. The interpretation was based upon the relative magnitude of the apparent resistivity values. High apparent values, above 500 ohm-ft., were interpreted as dry soil or bedrock in the upland terrain or as coarse textured sands on the floodplain terrain. Intermediate values, 200 to 500 ohm-ft., in the floodplain deposits were interpreted as representing predominantly silts. The upper values between 300 and 500 ohm-ft. were interpreted as silts but with some sandy textures and the lower values of 200-300 ohm-ft. were interpreted to be silts with some clay materials. Lower values, less than 200 ohm-ft., were interpreted as clay textures or shale bedrock. At stations where an insulating layer was at the surface, as at stations 267, 281 plus 50, 297 and 308, it was kept in mind that all deeper resistivity values would be affected by the high surface value resistivities. In cases where a conductive layer was on the surface, the magnitude of the deeper apparent resistivity values was lower due to the influence of the low resistivity surface layer. At station 270 the upper soil layer with a resistivity value of 310 ohm-ft. was interpreted as silt and sand and layer two, with a value of 200 ohm-ft., was interpreted as silt and clay. At station 267 the high resistivity surface layer value of 625 ohm-ft. was interpreted as silt, sand and gravel and the second layer with a resistivity value of 425 ohm-ft. was interpreted as silt and clay. The 425 ohm-ft. value was interpreted as silt and clay because the effect of the high surface value would increase the magnitude of the apparent resistivities of all the lower conductive layers. This reasoning was applied throughout the electrical resistivity interpretation.

The seismic refraction profile was based upon data collected at 5 refraction stations taken in the study area (Figure 3). Seismic velocities between 6,000 ft./sec. and 9,000 ft./sec. were interpreted as shale bedrock. Intermediate values, 3000 to 4500 ft./sec., were interpreted as alluvial deposits on the floodplain and as weathered shale in the upland residual terrain. Low velocities, 1200 ft./sec. or less, were interpreted as the weathered zone above the ground water table. The 7100 ft./sec. velocity at station 308 was interpreted as alluvium with rock fragments but the velocity seems to be much more in the range of a shale velocity and a boring would be recommended at this location to verify the interpretation.

The combined technique profile, Figure 4, is a composit of the other three profiles. The depth to bedrock and the bedrock stratigraphy are based upon information supplied by the seismic refraction profile. The soil stratigraphy is supplied primarily by the electrical resistivity profile and the composition and texture of the soil are supplied by the photo interpretation and electrical profile.

The above discussion outlines the procedures in developing a soil profile utilizing airphoto and geophysical techniques but also serves to illustrate the depth limitations of the airphoto interpretation method and how these limitations can be overcome by use of geophysical techniques with little additional cost.

The combined techniques profile corresponds well with the soil profile developed by the Highway Department, Figure 5, and shows that shallow subsurface investigations, utilizing airphoto and geophysical methods, will furnish soil and bedrock data which is sufficiently reliable for preliminary highway design.

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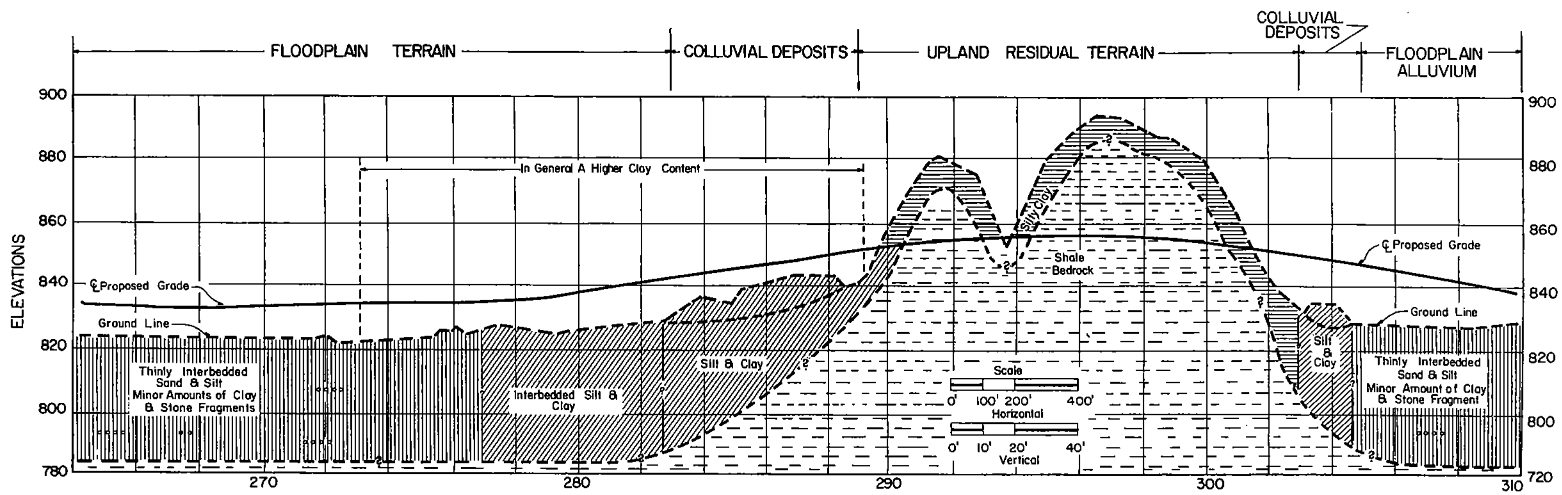

Fig. 1.— Photointerpretation Profile Guernsey, 40-18. 13.

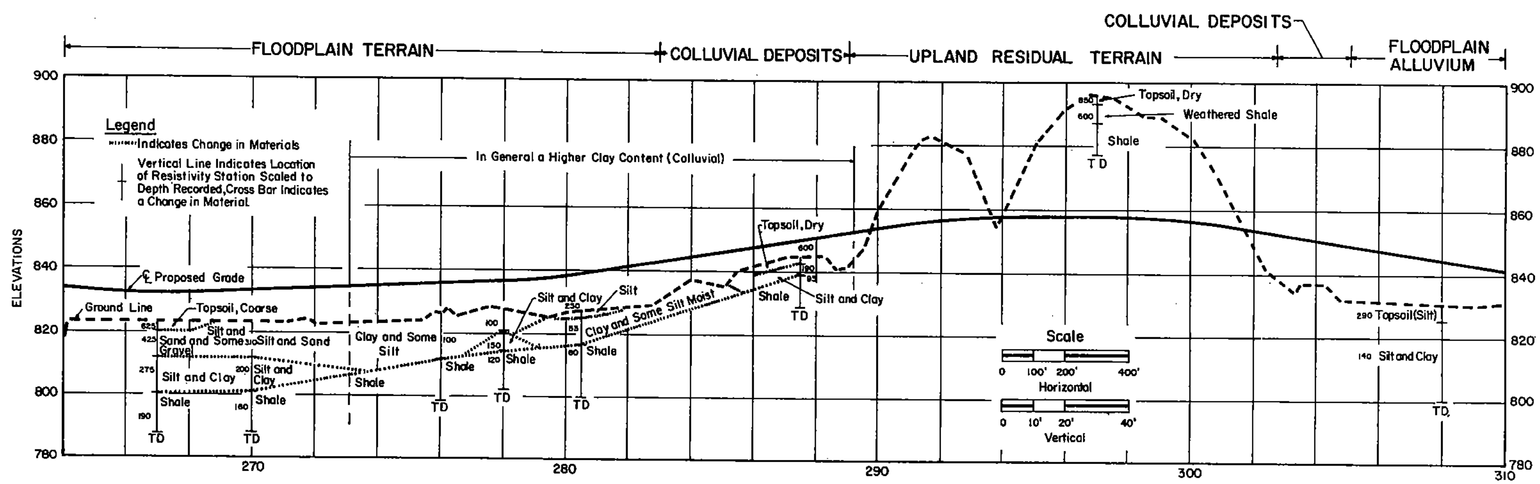

Fig. 2.— Electrical Resistivity Profile, Guernsey 40-18. 13.

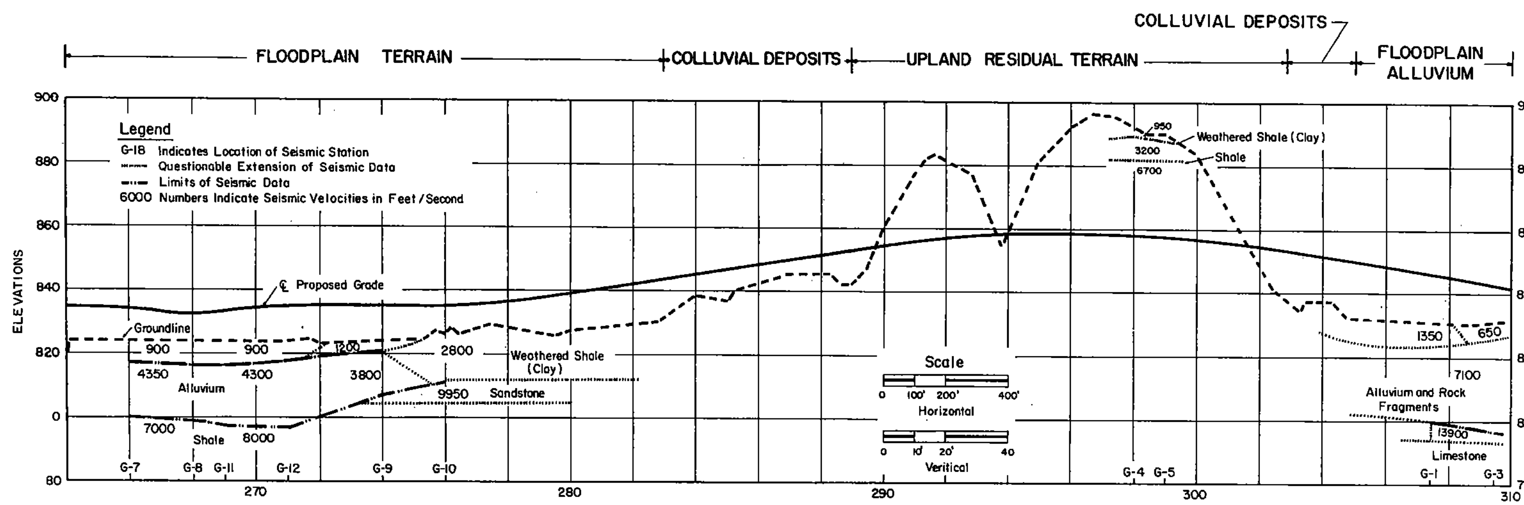

Fig. 3.— Seismic Refraction Profile, Guernsey 40-18. 13.

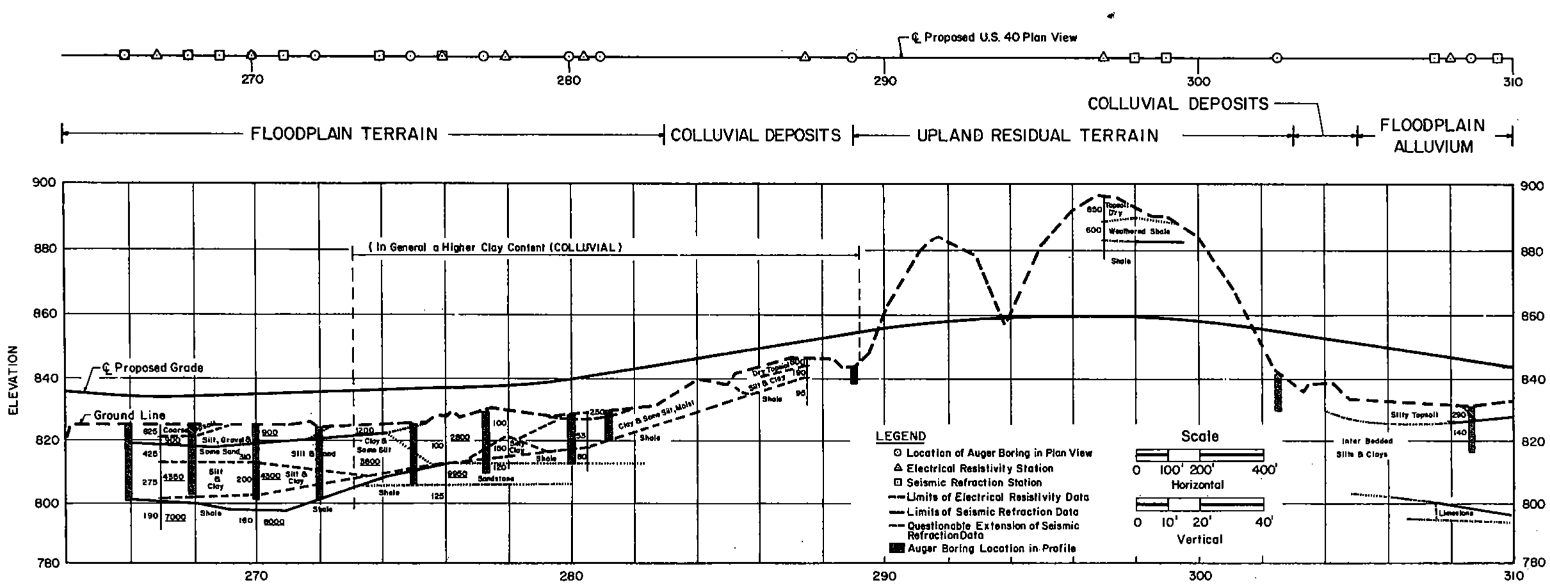

Fig. 4.— Combined Techniques Profile, Guernsey, 40-18. 13.

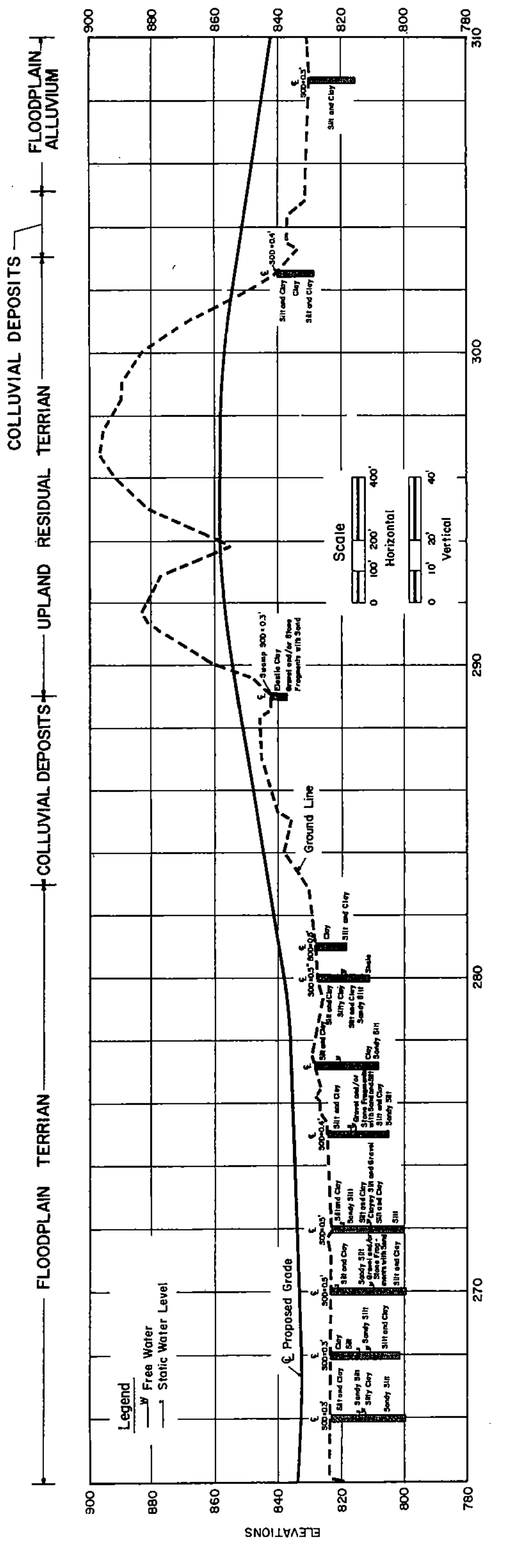

Fig. 5.— Soil Exploration Profile, Guernsey, 40-18. 13.

THE HIGHWAY RESEARCH BOARD AND ITS COMMITTEE ON ENGINEERING GEOLOGY

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The Highway Research Board operates within the Division of Engineering and Industrial Research of the National Academy of Sciences-National Research Council. The Board collects, organizes and disseminates technical information on highway engineering and provides a forum for exchange of ideas. The Department of Soils, Geology and Foundations of the Highway Research Board established a Committee on Engineering Geology in 1964. The Committee is concerned with research and investigative studies as they relate to applying principles of engineering geology to location, design, construction and maintenance of highways. The major problem facing this committee is to identify areas of research that need investigating and might be rewarding.

The purposes of this paper are to describe some of the functions and organization of the Highway Research Board (HRB), to discuss current aims and activities of the HRB Committee on Engineering Geology which was established in 1964, and to solicit additional suggestions on research needed in highway geology. To provide a background for the position of the newly formed Committee on Engineering Geology in the scheme of the HRB, the following is quoted from the National Academy of Sciences-National Research Council pamphlet (1964b) on Organization, Aims and Activities.

"The Highway Research Board (HRB), established in 1920 to encourage research in highway administration, transport and technology, operates within the Division of Engineering and Industrial Research of the National Academy of Sciences-National Research Council.

"The Highway Research Board participates within the broad field of highway transportation. Within this field the purposes of the Board are: (1) to encourage research and the utilization of research findings; (2) to provide a national correlation service for highway research activities; (3) to provide a forum for presentation and discussion of research papers and reports; (4) to form committees qualified to consider research needs, plan research work, advise on the conduct of research work, correlate and evaluate the results and form recommendations as to the utilization of research findings; (5) to publish and disseminate reports of research activities and findings and other information useful to highway administrators and technologists; (6) to administer and direct specific research projects when requested to do so by highway officials or others, when funds are provided by such sponsors, and when approval for such direct research is given by the National Academy of Sciences-National Research Council.

"The Highway Research Board is a cooperative organization of highway engineers, administrators and educators. It is supported by the several State highway departments, the Bureau of Public Roads, industry, and many other organizations interested in the development of highway technology and transportation. * * * * * * * * * *

"A major part of the technical work of the Board in stimulating and correlating research activities and in the dissemination of research findings is done by committees organized under eight major departments.

"Perhaps the most important single function of the Highway Research Board is its annual meeting. In January of each year engineers, administrators and scientists meet in Washington, D. C., to present and to hear papers and reports dealing with recent research activities. These are working meetings—no large banquets and receptions interfere with the attention of the delegates, who attend committee meetings and open sessions in the morning, afternoon, and evening for an entire week. These meetings attract over 2,500 persons, all involved in highway administration and research, from the United States and several other countries.

"A highly important activity of the Board is the dissemination of information through its publication program."

The Board issues the following major publications:

Highway Research Record
Yearbook
Highway Research Abstracts
Highway Research News
Highway Research Review

Bibliographies
Special Reports

The above quotation is but a general sketch illustrating some of the many ramifications of the HRB. Within this broad area of applied inquiry the earth sciences play a significant role, and geology has been given special recognition in that a committee on engineering geology was recently established.

The Committee on Engineering Geology is one of 18 committees in the Department of Soils, Geology and Foundations. In this department the committees are grouped into three Divisions designated A, B, and C. Descriptive titles are not used in grouping the committees because, in many cases, the activities of a committee extend beyond the activity of a particular division. Division A contains committees whose activities relate chiefly to compaction and stabilization; Division B activities relate chiefly to features studied with the aid of soil mechanics; Division C, which includes the Committee on Engineering Geology, relates to activities of a general and/or basic nature.

The purposes of the committees are:

1. To encourage research.
2. To assist the Board in providing a clearing house for research information.
3. To assist the Board in organizing and conducting forums, symposia and open discussions presenting research information and reports.
4. To suggest and plan needed research.
5. To correlate and evaluate research results.

Though geology has long been recognized in the name of the Department of Soils, Geology and Foundations, it was not until 1964 that a Committee on Engineering Geology was established. There are a number of reasons for this late recognition of geology in highway engineering. When roads were but trails in comparison with modern highways and their associated deep cuts and high fills, there waslittle need for geology. Also, geologists of an earlier era were more interested in mineral resources than in engineering applications.

The Committee on Landslides, established in the early 1950's and disbanded in 1964, was composed mainly of geologists. This committee made a most favorable impression with publication in 1959 of its special report "Landslides in Engineering Practice." The writing of a book by a committee is in itself an exceptional accomplishment.

Even before 1950, however, a few geologists participated on some of the HRB committees when the total number of staff geologists on highway departments was small. A canvass in 1949 showed that 16 States employed 43 geologists; in 1951, 23 States employed 89 geologists (Horner and Dobrovolny, 1952, p. 1). By 1963 approximately 300 geologists were employed by 34 States (Sherman, *in* Spangler, 1964, p.112). In the last canvass, made in 1964, a total of 350 geologists were employed by State Highway Departments (Spangler, 1964, p. 112). With that many geologists in the highway profession there is justification for an active Committee on Engineering Geology within the framework of HRB.

Currently the defined scope of the Committee on Engineering Geology includes support of research or investigative studies as related to principles of engineering geology applied to location, design, construction and maintenance of highways. This scope encompasses virtually all aspects of geology and therefore is so broad that it is difficult to come to grips with the problem of identifying areas of research that need investigating and that, at the same time, might be rewarding. The last readily available record of a systematic canvass of ideas solicited from highway departments was published in 1952 (Horner and Dobrovolny, 1952, p. 2). Respondents indicated that the following were the most important fields of research: (1) sub-drainage, (2) foundations, (3) clay minerals and soil performance, (4) analysis of aggregates, (5) materials survey techniques, and (6) landslides.

An informal canvass in 1964 of members of the Committee and interested individuals indicated several changes. Research on landslides as a geologic process was not mentioned. Modification and adaptation of geophysical methods for a variety of uses was mentioned by several. Where geophysical methods are used in New England, 85 percent of the cases yield accurate results, but 15 percent lead to erroneous conclusions; therefore, the suggestion was made that research be concentrated on geological situations which cause erroneous predictions. In other respects, the broad areas of research needs seem to be the same as for 1951.

There are two aspects to achieving the major objective of the Committee on Engineering Geology. One is to select a few topics from the many possible facets upon which to concentrate committee activity within its framework of operation. The second is to list research needs, organized in project form, that require a sustained effort by a research agency or academic group for solution or partial solution of geologic problems encountered in highway engineering. The main reason for presenting this paper is to solicit ideas on both aspects of the problem. A presentation of thoughts on needed research would be most helpful to the Committee.

A well thought-out listing of research needs in highway geology can be a rewarding exercise. Work on some of the research needs can be accomplished through the National Cooperative Highway Research

Program. This program, started in 1962 and supported by the American Association of State Highway Officials (AASHO), is administered by the National Academy of Sciences—National Research Council through the Highway Research Board.

In the context here used, research areas refer to problems considered to be of national importance. "Each year, specific areas of research needs to be included in the program are proposed to the Academy and HRB by AASHO. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Official recognition by AASHO of new research areas will be made only for those areas suggested by committees of AASHO or chief administrators of highway departments." (National Academy of Sciences—National Research Council, 1964a, p. 2.) These suggestions are in part selected from the listings of research needs as compiled by committees of HRB.

Research areas embrace a wide range of inquiry which may include more than one discipline. They are broken down into bite-size projects for which the problems and objectives can be defined. As an example, the translation of AASHO road-test findings to local conditions was considered as a broad problem area and submitted to the National Cooperative Highway Research Program. Projects in this area are intended to extend the AASHO road-test findings to provide for variations in soil, climate and materials that were not included in the road test (Burggraf, 1963, p. 13-15). Specific projects in the first program were: (1) The development of guidelines for studies of pavement performance over a wider varity of conditions than existed at the road test; (2) further development and correlation of methods for evaluating pavement conditions, particularly as these relate to the determination of present serviceability; (3) identification and ranking of all factors that have a significant effect on pavement performance, special attention being paid to factors that have geological or regional characteristics; and (4) utilization of fundamental principles of engineering mechanics in relating pavement parameters directly to pavement performance.

A list of research needs in engineering geology was prepared by members of the Committee on Landslides shortly before that committee was disbanded (E. B. Eckel, written communication, 1963). The list is presented here to place it on record and to serve as a guide for additional suggestions.

1. Research Area: Relatively rapid physical and chemical changes in earth materials.

Research Objective: To determine what processes may change the properties of earth materials used in construction, within the period of useful life of the structure, so as to significantly alter their engineering behavior. Concentrate on two types of material: (1) volcanic rocks, which may or may not alter to clayey minerals, and (2) shale and clayey rock, which may or may not change in physical and chemical properties under the thermal, chemical, or stress environment in a structure that is different from the natural state.

2. Research Area: Performance record of rock cuts.

Research Objective: To determine what are the significant factors, geologic and otherwise, that affect the long-term performance of large rock cuts. By suitable choice of field study areas, attempt to isolate and quantify the variables. Use repreated ground photography to record details. Use paired cuts to extimate effect of isolation on raveling. Evaluate effect of benching. Use maintenance record.

3. Research Area: Analysis of stability of large rock slopes.

Research Objective: To determine the factors that control the stability of large artificial or natural slopes of rock; to derive means of measuring these factors, quantitatively if possible; and to relate these factors to some quantitative measure of stability. This large field should be limited, initially, to study of homogeneous geologic material with emphasis on the relation of joint systems and topographic form of stability of large masses. This may include laboratory study of the mode of failure of jointed bodies and field geologic and geophysical study of the distribution and orientation of fractures.

4. Research Area: Formation of disintegrated granite.Research Objective: To determine the geologic factors that control the disintegration of granite. Disintegrated granite often occurs irregularly within the exposed parts of large granitic masses. Its presence means easy excavation, and it is much used as granular construction material. Yet it may lie adjacent to unaltered hard rock requiring blasting. A knowledge of the factors that control its distribution would aid in both selecting road alinements and locating sources of granular materials.

5. Research Area: Treatment of highways over old mine workings.

Research Objectives: To determine amount and thickness of cover needed between a highway subgrade and an old mine. Study other methods of treatment, as removal of all naturally subsided material and backfilling.

6. Research Area: Slope design for differentially weathered materials and other mixtures of earth and rock.

Research Objective: Development of design criteria that would permit field personnel to design stable, yet economical, back slopes in mixtures of earth and rock. Design of slopes in earth materials is well known; design in uniformly solid rock seldom presents difficult problems. On the other hand, design of slopes in differentially materials, or where rock pinnacles exist, poses highly complex problems.

Some of these defined research needs might be considered at project level rather than as a problem area and should be tied to a larger roadway problem. For example, the suggestion dealing with relatively rapid physical and chemical changes in earth materials could be part of a larger investigation of pavement and roadways performance on alinements that traverse shale and chayey rocks.

These suggestions are only a few of the areas in highway geology that deserve additional or more concentrated research effort. Calling attention to them is the problem. The Committee welcomes additional ideas from the readers. Suggestions may be directed to Chairman, Committee on Engineering Geology, Highway Research Board, Department of Soils, Geology and Foundations, 2101 Constitution Avenue, Washington 25, D. C.

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