Freeze-and-Thaw Phenomena in Concretes and Aggregates

James H. Havens
Kentucky Department of Highways

This paper is posted at UKnowledge.
https://uknowledge.uky.edu/ktc_researchreports/983
MEMORANDUM TO:  A. O. Neiser, State Highway Engineer  
Chairman, Research Committee

SUBJECT:  Research Report; “Freeze-and-Thaw Phenomena in Concretes and Aggregates”; KYHPR-64-6; HPR-1(5), Part II.

The enclosed report summarizes and finalizes many years of progressive study concerning the mechanisms causing damage to concrete during freezing. Strength, absorptivity, and severity of temperatures have been demonstrated to be controlling factors. Quality criteria for aggregates and concrete, in relationship to resistance to freezing and thawing, are implied. Other mechanisms may cause deterioration and be equally devasting. Expansive aggregates are recognizable by other tests.

Aggregate particles which absorb 4% water (by weight) should be considered to be deleterious. The permissive percentage of particles exceeding 3% absorption should be severely restricted in quality specifications. Aggregate particles having absorptions ranging from 1% to 4% may be potentially deleterious -- the likelihood or probability of damage to concrete increasing as the absorption approaches 4%. Existing requirements limiting the percentages of deleterious particles may be realistic; however, absorption seems to offer a purer basis for quality control than specific gravity (cf Sections of 612.2.4 and 612.5.2, Standard Specifications...). Forced saturation would become an essential part of the test procedure.

Respectfully submitted,

Jas. H. Havens  
Director of Research

JHH/cel

Attachment  
cc's: Research Committee

Assistant State Highway Engineer, Research and Development  
Assistant State Highway Engineer, Planning and Programming  
Assistant State Highway Engineer, Pre-Construction
Mr. A. O. Neiser

Assistant State Highway Engineer, Construction
Assistant State Highway Engineer, Operations
Assistant State Highway Engineer, Staff Services
Assistant Pre-Construction Engineer
Assistant Operations Engineer
Executive Director, Office of Computer Services
Executive Director, Office of Equipment Properties
Director, Division of Bridges
Director, Division of Construction
Director, Division of Design
Director, Division of Maintenance
Director, Division of Materials
Director, Division of Photogrammetry
Director, Division of Planning
Director, Division of Right of Way
Director, Division of Roadside Development
Director, Division of Rural Roads
Director, Division of Traffic
Division Engineer, Bureau of Public Roads
Chairman, Department of Civil Engineering, University of Kentucky
Assistant Dean for Continuing Education, College of Engineering, University of Kentucky
All District Engineers
RESEARCH REPORT

FREEZE-AND-THAW PHENOMENA IN CONCRETES AND AGGREGATES

KYHPR-646; HPR-1(5), Part II
Final

by

J. H. Havens
Director of Research

Contributors
Knox R. Burchett (1957-1958)
John W. Scott (1962-1968)
George R. Laughlin (1964-1965)
Ronald D. Hughes (1969)
Robert C. Deen (1969)

Division of Research
Department of Highways
Commonwealth of Kentucky

In cooperation with the
U.S. Department of Transportation
Federal Highway Administration
Bureau of Public Roads

The opinions, findings, and conclusions in this report are not necessarily those of the Department of Highways or the Bureau of Public Roads.

February 1970
ABSTRACT

FREEZE-AND-THAW PHENOMENA
IN CONCRETE AND AGGREGATES

This report summarizes several years of research relating to damage to concrete and aggregates undergoing freezing and thawing. Basic principles involving freezing and attendant pressures are considered. Application of these principles to the evaluation of concrete was accomplished in experiments on concrete having low and high air contents. The effects of air entrainment upon freeze-thaw resistance were demonstrated.

Freeze-thaw characteristics of saturated aggregates relative to their physical properties such as porosity, absorption and bulk specific gravity were studied by submerging individual particles in pre-chilled mercury. Pressures associated with popouts in concrete were monitored and are presented along with accompanying theoretical considerations.
INTRODUCTION

During the past 50 years, exposed concrete has been used extensively in structures and pavements. Bridges, roads, sidewalks, parking decks, and modernistic roofs are but a few general examples. While there are many instances in which such concrete has been exposed to the “elements” for many years without sustaining serious damage, there are other instances in which the effects of time and weather have been ruinous. It is widely recognized that one of the most destructive elements or mechanisms in the weathering and deterioration of concrete is the absorption and freezing of water within the pores of the mortar and aggregate. Laboratory tests have shown repeatedly that damage by freeze-thaw is related to porosity of the concrete, rate of water absorption, degree of saturation and total water content. In many respects, the mechanics of freeze-thaw in concrete are similar to that of a water pipe that freezes and bursts; and, when such damage occurs in concrete, it is attributable to overstressing of the mortar structure or of the aggregate as a result of expansion as water changes to ice.

The endurance of concrete depends upon the severity of exposure as well as characteristics of the concrete itself. It is known that some types and sources of aggregate perform consistently well in concrete while others do not. Those which are highly absorptive have generally given poor performance; although, some aggregates which are not highly absorptive have given consistently poor performance also--probably due to other mechanisms in addition to that of freeze-thaw. Mortar in concrete is no less immune to damage if it is porous, easily saturated, or is not otherwise protected. A cement-rich, dense, air-entrained, mortar phase affords protection to porous aggregate as well as to the concrete as a whole.

Voids occur in concrete through the entrapment and entrainment of air, the occlusion of excess mix water, differences in the specific volumes of reactants and hydration products, leaching of hydration products (CaO), and the use of porous aggregates. Those voids which are easily saturated affect durability unfavorably while those which are less permeable are highly favorable to durability. Considerable water is occluded in concrete in the form of excess mix-water and water absorbed by the aggregate. That water may remain more or less sealed permanently therein unless exposed to severe drying conditions. Concrete which has not been allowed to dry following curing and prior to the onset of freezing does not give as good performance, in laboratory tests, as concrete which has been dried but which is otherwise similar. This indicates that a somewhat irreversible fixation or tightening of the mortar structure attends drying and that the concrete becomes less susceptible to resaturation. In other words, the process of drying may close some of the pores. The use of aggregate which is merely moistened, as opposed to soaked or saturated, in the concrete favorably affects durability--at least in laboratory freeze-thaw tests. Either or both of these effects may merely be the result of delayed saturation. Significantly enough, all such effects are somehow related to water absorption and saturation, yet none of them is especially revealing as to the critical moisture content, degree of saturation, or pressures which actually produce damage.

Dry concrete is unaffected by the freeze-thaw mechanism. It appears, in fact, that serious damage is sustained only when a high degree of saturation exists. If concrete is relatively impermeable, even though it contains a large percentage of voids, saturation is unlikely to occur unless exposure to water is sustained for a long period of time. In natural weather, periods of wetting occur alternately with periods of drying; and the durations of drying periods tend to exceed those of wetting periods. In the random courses of nature, a concrete structure such as a bridge would tend to dry; whereas, long interim periods of wetness followed by freezing may be extremely damaging.

In conducting laboratory freeze-thaw tests on concrete, it is customary to monitor weight, natural frequency, and temperature. Heretofore, temperature records have simply provided assurance that the concrete has been alternately frozen to 0°F and thawed to 40°F. These temperature records are usually obtained by embedding thermocouples in companion specimens of concrete under test. Automatic recording instruments, of course, have proved desirable. Using such equipment in a study on the freeze-thaw durability of concrete made with a highly porous aggregate, a rather wide isothermal step in the
time-temperature curves was detected (25) -- which, except for a slight depression of the freezing point, corresponds to the normal phase transition from water to ice. While the effects, there, were magnified by the large quantity of absorbed water in the porous aggregate, it became obvious that they also portrayed certain fundamental physical principles involved in the disruptive action of freeze-thaw on ordinary concrete. The isothermal steps were truly manifestations of the latent heat of fusion of water absorbed in the concrete, and the freezing-point depressions were apparently manifestations of pressure. These findings suggested that time-temperature records, if sufficiently accurate, could provide heretofore unavailable information concerning the amount of freezable water in concrete undergoing freeze-and-thaw, the progress of saturation, permeability, the internal pressures induced by freezing at various levels of saturation, and the progress of damage to the concrete. The purpose of the present study, therefore, was to examine these possibilities further and to offer a critical and informative analysis of related theories and the extent of their practical application.

**BASIC THERMAL ANALYSIS**

The density of water at 0°C is 0.9998 gms per cu cm; whereas the density of ice at 0°C is 0.917 gms per cu cm. There is an increase in volume of approximately 9% when water freezes. If a closed vessel filled with water at 0°C is sealed, and if the water is then frozen, the vessel must yield or dilate as much as 9% by volume or else it will rupture. However, if the vessel were filled with water to 91.7% of its volume, the volume of ice produced would just equal the volume of the vessel, and no dilation of the vessel would occur. Likewise, if water completely fills the pores or voids in concrete and is frozen therein, the concrete must dilate or expand proportionally. Thus, 10% water by volume of concrete would induce 0.9% dilation in the volume of the concrete -- neglecting compressibilities of ice and concrete.

Therefore, 91.7% saturation of all voids is termed "critical saturation". The term simply alludes to the degree of saturation beyond which the freezing of water overfills the voids with ice and creates internal expansive pressures. While its theoretical meaning is quite clear, its practical meaning as applied to concrete is more in the sense of a statistical average which can be interpreted in two ways: 1) 91.7% of the voids being completely filled, or 2) all voids being filled to 91.7% of their capacity. The first possible condition could be extremely damaging to concrete undergoing freezing; whereas, if the second possibility accurately described the condition of the water in the concrete, there could be no damage at all from freezing. Obviously, it is not possible to measure the degree of saturation within each individual pore or void in concrete, but it is logical to assume that damage may occur at moisture contents considerably below 91.7% saturation and is more likely to occur at or above this value.

**Analogous Phenomena**

In order to demonstrate some basic principles involved, preliminary experiments were made on rather idealized models. First, a quart volume of water, contained in an open vessel, was frozen in air, and a time-temperature record was made from a thermocouple positioned near the center of volume. The resulting thermograph, Figure 1, gave a broad step at 32°F, as expected, without any noticeable depression of freezing point. Since the water had been more or less unrestrained in the open vessel, a closed vessel or "steel bomb" was improvised from a 3"-diameter pipe nipple and two cast-iron caps. A valve and thermocouple-well were tapped into the nipple. The bomb was filled with water and frozen in air as before, and the resulting thermograph is shown in Figure 2. It is apparent that freezing began at 32 F, that the freezing point gradually decreased to 25 F, and then suddenly reverted back to 32 F. This pressure is in close agreement with the estimated ultimate strength of the cast iron caps, 7000 psi.

Continuing with these experiments, the ruptured cap was replaced and the bomb filled with water to 96% of its capacity. Upon freezing, as before, the thermograph shown as Figure 3 was obtained. Here again, the water began freezing at 32 F but there was no immediate depression of the freezing point and
Figure 1. Thermograph Obtained from Freezing Water in Open Vessel

Figure 2. Thermograph Obtained from Freezing Steel Bomb Filled with Water
Figure 3. Relationship between Pressure and Freezing Temperature of Water

Figure 4. Thermograph Obtained from Freezing Steel Bomb Containing Water to 96% of its Capacity
consequently no immediate build-up of pressure. In this case, development of pressure was delayed until most of the water had frozen. There was only 6-degree depression of the final freezing point, equivalent to 6200 psi; and, while there was no explosive rupture, the caps yielded considerably under that pressure.

Verification of pressures corresponding to freezing-point depressions, when attributable entirely to pressure, is provided in Dorsey's *Properties of Ordinary Water Substance* (7) and in the International Critical Tables. Both of these sources are based on the work of P. W. Bridgman, published in 1912. Figure 4 shows the relationship between temperature and pressure.

**Experimental Phenomena**

Figure 5 illustrates a typical time-temperature record obtained from an oven-dried specimen of concrete undergoing one cycle of freeze-thaw in automatic freeze-thaw equipment. Since the freezing was done in air, the specimen temperature is seen to lag behind the air temperature; and there are no sharp breaks or steps in the cooling curve. Upon flooding the freezing chamber with water at 40°F, the specimen temperature rose rapidly; but, there are no sharp steps or breaks in the curve.

In contrast, Figure 6 illustrates a typical thermograph obtained from highly porous, highly absorptive, highly saturated concrete undergoing a single cycle of freeze-thaw. The steps in the freezing-and-thawing curves occur at approximately the same temperature, near but not exactly at the normal freezing temperature of water (32°F). The portions of the curves above and below that step have characteristically different curvatures, and the steps are not merely off-sets in otherwise normal cooling and heating curves. By graphically projecting these curves through and beyond the steps, according to their characteristic curvatures, the time-durations of the steps are more clearly outlined. The slopes of either or both curves as they approach the step may be described as \( \frac{dT}{dt} \) or the incremental change in temperature per increment of time. This factor, when multiplied by the product of the specific heat and weight of the concrete specimen would give the rate at which heat was being removed from the specimen. The product of the time-lapse and rate of heat removal should correspond approximately with an isothermal change in heat content \( \Delta H \) accompanying freezing of water in the specimen; and:

\[
\text{Weight of Water} = \Delta H/\text{latent heat of fusion of water}.
\]

From Figure 6, it is also apparent that the step itself is sloping slightly downward from 32°F, signifying that water began freezing under a normal pressure of 1 atm (14.7 psia) but finished freezing at 28°F, which corresponds to the freezing point of pure water under a pressure of approximately 4500 psi. Since there is no apparent depression of the initial freezing point, no super-cooling or other interfering mechanism is evident. This point requires much more detailed examination.

A large portion of the water absorbed into concrete at saturation may be regarded as being "free" water which is freezable and evaporable in much the same sense that pure water is; and it is this portion of water to which analysis of freezing point depression and pressure applies most directly. However, a portion of the water, in addition to the water of hydration and crystallization, exists in an infinitely more complex state -- due, of course, to internal surface absorption and capillary tensions which abound within the concrete. This portion of water is not expected to exhibit a normal freezing point, and it may be of little or no significance here unless its effects appear as an interfering mechanism distorting the prototype thermograph.

The only other serious possibility of an interfering mechanism is the influence of dissolved electrolytes upon the freezing characteristics of the so-called "free" water. The most abundant solute in hardened concrete is \( \text{Ca(OH)}_2 \); but its solubility in water, at 32°F, is in the order of 1.85 gms per liter of water. Assuming complete saturation and 90% dissociation, this would depress the initial freezing point approximately 0.2 degrees. Since the solution is saturated, it becomes a eutectic or constant-freezing mixture which should otherwise respond to pressure according to the prototype thermograph. The possible
Figure 5. Typical Time-Temperature Record Obtained from Oven-Dried Concrete Undergoing a Single Cycle of Freeze-Thaw.

Figure 6. Typical Time-Temperature Record Obtained from Highly Absorptive, Highly Saturated Concrete Undergoing a Single Cycle of Freeze-Thaw.
effects of alkali available from cement may be similarly estimated by assuming 0.6% alkali by weight of cement and the maximum free water to be approximately equal to the excess mix-water. This quantity of alkali, if available as solute, would depress the initial freezing point a maximum of 2.5 degrees F. Since no such initial depressions are yet apparent, the concentration of alkali must be rather small, at least while the greater portion of water is freezing. Alkalies such as NaOH and KOH are more or less infinitely soluble in comparison to Ca(OH)$_2$. In dilute solutions of this type, the water freezes into ice which is pure or essentially free of solute -- thereby concentrating the solute in the remaining water. For instance, when one-half the water is frozen, the concentration of solute in the remaining solution is twice as great as it was initially, and the freezing point then is depressed twice as much as it was initially. The effects of this kind of interfering mechanism, if present to a significant degree, would cause the freezing point step to curve downward at an increasing rate and there would be no discrete terminus to the step.

A general, but cursory, conception of the effects of pressure and different concentrations of highly soluble electrolytes is illustrated in Figure 7 and are superimposed there upon a normal freezing curve for pure, unrestrained water. Figure 7 serves to identify the nature of any interfering mechanism should any appear upon a thermograph and may serve also as a basis from which to judge the probable degree of any such interference.

Application of these principles to the evaluation of concrete depends considerably upon the precision and accuracy of the time-temperature records and on some mathematical problems in heat transfer. While numerous variations in instrumentation and methods of analysis are foreseeable, the primary objective, at present, is to offer a preliminary insight into the theoretical aspects of thermal analysis and to demonstrate the type of information to be derived therefrom.

In a subsequent experiment, two sets of 6-in cubes were cast using a 6-bag mix, 3-in slump, and dense limestone aggregate. In one set, 13% air was obtained; and 0.7% air was obtained in the other set. By normal mixing, without air-entraining agents, 1.5% to 2.0% air was obtained; and in order to dispel a good portion of this air, the concrete was vibrated until 0.7% was obtained. This was a deliberate attempt to establish a true, non-air-entrained reference standard for comparison with the set of specimens containing 13% air. The two sets of specimens represent forced extremes in the range of air contents -- as a deliberate attempt to magnify the effects of air content on the thermographs. Three thermocouples were cast into these specimens in order to further magnify the thermographs.

One specimen from each of the two sets was moist cured for 5 weeks prior to beginning of freeze-thaw, and these specimens failed to exhibit any indication of a freezing-point step even after 30 cycles. The companion specimens for each set were cured 14 days and immersed in water for 24 hours prior to beginning freeze-thaw. These specimens exhibited a progressive broadening of the step and a progressive differentiation between the air-entrained and non-air-entrained concretes. Freeze-thaw was continued to 77 cycles, at which time the non-air-entrained specimens were in virtual ruin. The appearance of the specimens after 77 cycles is shown in Figure 8.

Thermographs obtained from the 1st, 40th and 77th cycles are shown in Figure 9. It is apparent that the non-air-entrained concrete developed a much broader thermal step than the air-entrained specimen, indicating that the rate of absorption of water by the non-air-entrained concrete was also much greater.

Failure of the two concrete specimens, containing 0.7% and 13% air, which were cured 5 weeks, to exhibit a significant thermal step even after 30 cycles of freeze-thaw offers an example of highly retarded or delayed saturation. It is recognized that continued curing and hydration tends to desiccate smaller interstices of the cement gel and to reduce the “free” water content of the concrete. Continued curing would likewise densify and strengthen the mortar against subsequent ingresses of water. Thus, the mere number of rapid or slow freeze-thaw cycles which a concrete specimen is able to withstand before saturation becomes critical is probably not as significant as the time-duration of conditions causing
Figure 7. Probable Effects of Solute upon the Time-Cooling Curve
Figure 8. Concrete Specimens, 6-in Cubes, after 77 Cycles of Freeze-Thaw; Cured 14 Days, Soaked 24 Hours; Right Specimen Had 0.7% Air; Left Specimen Had 13% Air
Figure 9. Time-Temperature Records from Concrete Containing 0.7 and 13% Air; Moist-Cured 14 Days; Soaked 24 Hours
absorption of water and eventual saturation. It is suggested, therefore, that the number of regular cycles or
the time preceding development of a significant thermal step and freezing-point depression provides a
significant basis for evaluating concrete durability in addition to that of determining the number of cycles
producing complete or impending destruction of the concrete.

It seems necessary to justify the fact that pressures attending freezing of water in the concrete, as
deduced from freezing-point depressions, were in the same order of magnitude as the compressive strength
of the concrete - or, at least, greatly exceeded the tensile strength. Keeping in mind that the thermocouples
were positioned near the center of the specimens, the primary zone of influence was the core of the
specimen which was restrained by the thick shell of surrounding concrete. The core of the specimen became
a center of compression from which the radial compressive stresses diminish rapidly. Both compressive
stresses and tensile stresses are at a maximum at the inner boundary. Therefore, if the wall thickness is large
in comparison to the radius of the compression center, failure would necessarily emanate from the core and
appear there as a yielding or crushing of the concrete. This explanation does not apply to the outer
boundary of the concrete. There the reactance to pressure may not greatly exceed the tensile strength or
shear resistance of the concrete; and it is for this reason that failure appears to proceed inwardly from the
outer boundary. This aspect of damage and mode of failure are discussed further in following sections.

FREEZE-THAW RESISTANCE OF AGGREGATES

Present test methods for aggregates employ composite samples and results of such tests provide
average values. For example, the average value of absorption obtained from a composite sample may exhibit
a low value - generally 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 important. 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. Maximum saturation definitely establishes the ultimate susceptibility of
aggregate to damage from freezing and thawing.

Phenomena

In order to obtain objective data pertaining to the freeze-thaw characteristics of aggregate - that is, to
establish more definitive relationships between 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 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
immiscible with water; and it has a low freezing point. The test consisted of submerging the aggregate
particle in pre-chilled mercury. From preliminary testing, it was found that if a particle did not show visual
distress at the end of 4 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. A sample was secured from a glacial outwash
deposit and kept inundated in order to meet those conditions.

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 provided an assortment of particles needed for
the study.

The sample was graded into the following sieve sizes:
Passing 1-1/2-in and retained on 1-in,
Passing 1-in and retained on 3/4-in,
Passing 3/4-in and retained on 1/2-in,
Passing 1/2-in and retained on 3/8-in, and
Passing 3/8-in 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 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,
$\gamma_M$ = density of mercury at temperature of test,
$W_M$ = weight of mercury displaced by particle, and
$\gamma_W$ = 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 5 minutes. The freezing apparatus is shown in Figure 10. The mercury was contained in a glass cylinder enclosed in an insulated container of dry ice (solid CO$_2$). A low-temperature thermometer was used to observe 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 cycles was recorded.

For obvious reasons, absorption 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:

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

where $w$ = absorption expressed as a percent,
$W_T$ = 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:

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

where $\eta$ = porosity expressed as a percent,
$w$ = absorption expressed as a percent, and
$G_{SSD}$ = bulk specific gravity (saturated surface dry).
Figure 10. Quick-Freezing Apparatus
It should be emphasized that the above equation applies only for saturated aggregate.

After all other testing were 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.

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 11, and results of the tests are tabulated in the Appendix. The slope of the curve indicates the increase in 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% 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 particles at the end of the fourth cycle of freeze-and-thaw testing was used for correlation. Typical fractures are shown in Figures 12 and 13.

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 14. The largest percentage of fractured particles occurred in the 1/2-in size; whereas, the smallest percentage of fractured particles occurred in the 1-in and No. 4 sizes. The average percentage of fractured particles in all sizes combined is indicated by the dashed line, at 53.6%. The 1-in 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-in particles consisted of porous cherts and porous dolomites. The bulk of the No. 4 size particles was mostly quartz.

The relationship between soundness of the test particles and their adjusted porosity values is shown in Figure 15. 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% fractured; whereas, less than 25% of the particles having a porosity of less than 2% fractured.

A graph depicting the relationship between percentage of fractured particles and absorption values of test particles is shown in Figure 16. All particles having absorptions of 4% or greater failed when subjected to freezing and thawing. Very few specimens having absorptions of less than 1% failed. Between these extremes, percentage of particles failing increased as 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 17. A definite trend is established, indicating that aggregate in the lower specific gravity ranges has lower resistance to freezing and thawing than aggregate in the higher specific gravity ranges. 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 the following table. Sedimentary rock particles had a much larger percentage of fractures than igneous or 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.
Figure 11. Accumulative Percentage of Fractured Particles for Each Cycle of Freeze-Thaw
Figure 12. Distress in Form of Opened Fractures

Figure 13. Specimen Fractured into Three Pieces
Figure 14. Relationship between Particle Size and Percentage of Fractured Particles after Exposure to 4 Cycles of Freeze-Thaw
Figure 15. Relationship between Porosity (Adjusted Value) and Percentage of Fractured Particles after Exposure to 4 Freeze-Thaw Cycles.
Figure 16. Relationship between Absorption and Percentage of Fractured Particles after Exposure to 4 Freeze-Thaw Cycles
Figure 17. Relationship between Bulk Specific Gravity (SSD) and Percentage of Fractured Particles after Exposure to 4 Freeze-Thaw Cycles

DURABILITY OF AGGREGATE ACCORDING TO CLASSIFICATION

<table>
<thead>
<tr>
<th>Classification</th>
<th>Percentage of Fractured Specimens at End of 4 Cycles of Freeze-and-Thaw</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sedimentary</td>
<td>66</td>
</tr>
<tr>
<td>Dolomite</td>
<td>77</td>
</tr>
<tr>
<td>Chert</td>
<td>59</td>
</tr>
<tr>
<td>Limestone</td>
<td>41</td>
</tr>
<tr>
<td>Sandstone &amp; Siltstone</td>
<td>69</td>
</tr>
<tr>
<td>Igneous and Metamorphic</td>
<td>23</td>
</tr>
</tbody>
</table>
Figure 15 and Figure 16, 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 is related directly to absorption. With a given porosity and assuming that all voids are permeable, the corresponding saturated absorption value may be computed by assuming a value for apparent specific gravity. Figure 18 shows such a relationship. These absorption values were calculated by taking the specific gravity of the solid components to be 1.65 and 2.94.

If critical porosity is taken to be 2%, 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 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 having higher gravities may be just as deleterious. This is well illustrated by the dolomites shown in Figure 17. 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.

Absorptive rates of aggregate were studied by obtaining samples of stream-saturated gravel from a commercial dredge on the Ohio River, and upon removal from the river bottom, immersing them in water. A number of specimens of varying sizes were selected and the saturated surface-dry weight of each was obtained. Each specimen was allowed to air dry to constant weight, re-soaked in water to constant weight, oven dried, and re-soaked again in water. During the soaking and drying cycles, the weight of each particle was determined daily. The rate of water loss and absorption for each cycle is shown graphically in Figure 19.

The larger and porous rock particles which were air dried required approximately 21 days of soaking to approach 100% saturation. After 39 days of re-soaking, the denser, smaller particles which were air dried never regained their original moisture content and, in fact, reached a degree of saturation just slightly greater than critical. When oven dried, the larger particles regained 95% of the original moisture after soaking 24 days, whereas the smaller particles regained only 85% after soaking for the same length of time.

The data indicate that porous aggregate particles are more receptive to water; and this, of course, greatly reduces the durability of such aggregate when used in concrete. Figure 19 also shows the benefits which might be gained by drying aggregates before they are used in concrete.

Theory

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 (33) explanation of Lame's solution for the radial and tangential normal stresses in a thick-walled, spherical container under internal and external pressures (Figure 20). According to Timoshenko, the radial normal stress is given by the expression:

$$\sigma_r = \frac{P_0 b^3 (R^3 - a^3)}{R^3 (a^3 - b^3)} + \frac{P_1 a^3 (b^3 - R^3)}{R^3 (a^3 - b^3)}$$

and the tangential normal stress is obtained by the equation:
Figure 18. Theoretical Relationship between Porosity and Absorption When All Pores Are Permeable and Saturated
Figure 19. Rates of Water Loss and Absorption
Figure 20. Thick-Walled Sphere under Internal and External Pressures (cf. Reference 33)
\[
\sigma_T = \frac{P_0 b^3 (2R^3 + a^3)}{2R^3 (a^3 - b^3)} - \frac{P_i a^3 (2R^3 + b^3)}{2R^3 (a^3 - b^3)}
\]

where \( a \) = inner radius of the sphere, 
\( \phi \) = outer radius of the sphere, 
\( P_i \) = internal pressure, 
\( P_0 \) = external pressure, and 
\( R \) = radial distance.

If the exterior confining pressure \( (P_0) \) is zero, the equations for the normal stresses at the extreme outer fiber are as follows:

\[
\sigma_T = 0
\]

\[
\sigma_T = \frac{3P_i a^3}{2(a^3 - b^3)},
\]

and by substituting

\[
V_v = \frac{4}{3} \pi a^3 = \eta V_t \quad \text{and} \quad V_t = \frac{4}{3} \pi b^3,
\]

where \( V_v \) = volume of voids, 
\( V_t \) = volume of sphere, and 
\( \eta \) = porosity = \( V_v/V_t \),

the equation reduces to

\[
\sigma_T = \frac{3}{2} \frac{P_i}{1 - \eta}
\]

The radial normal stress, \( \sigma_r \), along the outer edge of the spherical container is zero and the tangential normal stress, \( \sigma_T \), 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, \( \sigma_u \), is inserted in the above equation for \( \sigma_T \), the equation becomes

\[
P_u = \frac{2}{3} \sigma_u \frac{1 \cdot \eta}{\eta}
\]

where \( P_u \) = maximum allowable internal pressure and 
\( \sigma_u \) = tensile strength.

Tensile strengths of the gravel particles were not determined, but similar materials have tensile strengths ranging from 100 to 1000 psi. 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 21.

The determination of the internal pressure accompanying freezing of water within a hollow sphere
Figure 21. Suggested Theoretical Relationship between Allowable Internal Pressure, $P_u$, and Available Internal Pressure, $P_a$, For Varying Porosities and Tensile Strengths
may, likewise, be approached from 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:

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

where \( V_i \) = volume of ice,
\( S \) = degree of saturation,
\( \beta \) = 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:

\[ \varepsilon_T = \frac{\delta}{R} = \frac{\sigma_T}{E} - \mu \frac{\sigma_r}{E} - \mu \frac{\sigma_r}{E} \]

where \( \varepsilon_T \) = tangential strain,
\( \delta \) = radial displacement,
\( E \) = Young's modulus of elasticity,
\( \mu \) = Poisson's ratio, and
\( R \) = radial distance.

Assuming \( P_0 = 0 \), the increase in the radius of the internal cavity or void due to an internal pressure, \( P_a \), is

\[ \delta_a = \frac{(4\mu - 2) a^3 - (1 + \mu) b^3}{2E(a^3 - b^3)} a P_a \]

and by substituting

\[ V_v = \frac{4}{3} \pi a^3 = \eta V_t \] and
\[ V_t = \frac{4}{3} \pi b^3 \],

the equation reduces to

\[ \delta_a = \frac{1 + \mu + 2\eta - 4\mu\eta}{2E(1 - \eta)} a P_a \]

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

\[ V_c = \frac{4}{3} \pi (a + \delta_a)^3 \]

or

\[ V_c = \frac{4}{3} \pi a^3 (C^3P_a^3 + 3C^2P_a^2 + 3CP_a + 1) \]

where \( C = \frac{1 + \mu + 2\eta - 4\mu\eta}{2E(1 - \eta)} \).

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:

\[ \frac{4}{3} \pi a^3 S (1 + \beta) (1 - \frac{P_a}{K}) = \frac{4}{3} \pi a^3 (C^3P_a^3 + 3C^2P_a^2 + 3CP_a + 1) \]
or
\[ P_a^3 + \frac{3}{C} P_a^2 + \frac{3CK + S + S\beta}{C^2K} P_a + \frac{1 - S - S\beta}{C^3} = 0. \]

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 ratio, and modulus of elasticity of the sphere.

The available internal pressure, \( P_a \), was calculated for various porosity values by using the following values:

- \( U = 0.365 \)
- \( E = 1.0 \times 10^6 \) psi,
- \( K = 1.2 \times 10^6 \) psi,
- \( S = 1.00 \), and
- \( \beta = 0.0917 \).

This plot is superimposed on the maximum allowable internal pressure curves in Figure 21.

The intercepts between the available-pressure curves are maximum safe porosity values. At porosities greater than this, the maximum available pressure is not needed to cause rupture. Pressure is dependent upon temperature; and so, as porosity increases, less pressure -- and consequently less severe temperatures -- will cause failure. Figure 22 shows the same theoretical relationship between disruptive temperature, porosity, and tensile strength. For a given tensile strength, the disruptive temperature increases as porosity increases, and it may be noted that rupture would occur at a temperature slightly below freezing if the porosity is high.

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 depth 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.

**PRESSURES ASSOCIATED WITH POP-OUTS**

Pop-outs and deep-seated deterioration are generally associated with high absorptivity and high saturation. Aggregate particles of all sizes are capable of inducing damaging pressure. Particles which are near the surface may produce pop-outs even though the dilating pressures are relatively low -- the surrounding shell of concrete offers only minor restraint.
Figure 22. Suggested Theoretical Relationship between Temperature, Porosity and Tensile Strengths
Phenomena

In order to gain a better understanding of the mechanisms associated with these signs of distress, failures were induced by subjecting voids of varying diameters and depths below the concrete surface to hydrostatic pressure. Concrete specimens, 10 x 10 x 4 in, were cast from Class A (6-bag mixture) concrete containing limestone coarse (1-1/2 in to No. 8) and fine aggregate. The specimens were moist cured at 77 F for 28 days prior to testing. Each specimen was then drilled with a flat faced masonary bit to form a cylindrical hole in the center of a 10-x 10-in surface. Hole depths for the various specimens ranged from 2-1/2 in to 3-7/8 in so as to provide depths of burial ranging from 1-1/2 in to 1/8 in. The diameters of bits used for drilling the various specimens were 1/4, 3/8, 1/2, 3/4, 1 and 1-3/8 in. A 1/4-in, high-pressure tube and rubber gasket assembly was placed in each specimen to within 3/4 in of the bottom of the void. The tubing and gasket were held in place with high-strength epoxy cement. Figure 23 is a sketch of the void and tubing arrangement. In all cases, the void depth was 3/4 in, and only the void diameter and depth of burial were varied.

A hand-operated pump (Figure 24) capable of producing a maximum pressure of 5000 psi was connected to the pressure tubing, and glycerin was pumped into the artificial void at such a rate that the pressure zone was limited essentially to the pressure cavity. The time required to attain the final pressure was approximately 10 seconds. Characteristics of resultant pop-outs are illustrated in Figure 25 and depicted in Figure 26. All pop-outs were predominantly conical, and thinning was more pronounced along the outer surface perimeter of the larger pop-outs. Failure widths for the 3/4-in and 1-in cavities at varying depths of burial are shown in Figure 27. Failure angles were computed from the fitted curves and are plotted in Figure 28. The failure angle is significantly related to cavity diameter and depth of burial.

Pressures required to produce pop-outs in the various specimens are shown by the series of curves in Figure 26. These curves present data obtained from tests on 150 specimens. Results obtained had a remarkably high degree of duplication. For similar depths of burial, greater pressures were required to produce pop-outs in the small diameter cavities than required for the larger cavities.

Figure 26 indicates a protective cover of one inch would be necessary for protection against failure from pressures expected within a 1/4-in cavity, and a cover of approximately 1-1/4 in would be needed for 1-3/8 in cavity.

Specimens containing cavities which were more deeply buried split into two sections as depicted in the sketch inserted on Figure 29. The pressures required to produce this type failure varied with cavity diameter but were independent of depth of burial - that is, above the limit of pop-out failure. Pressures to induce failures of this type should not vary appreciably with depth of burial since the area of resistance to failure was constant (4 x 10 in) for all specimens. The limiting depth of burial for a given cavity diameter at which the mode of failure changes is naturally governed by the specimen dimensions, and pop-outs would occur at greater depths of burial in specimens of larger sizes than those tested in the series reported herein.

Splitting-type failures are represented on the curves in Figure 26 in the zones of zero slope. Failures of this type may be considered as deep-seated deterioration commonly associated with deeply embedded and highly absorptive aggregate particles. High dilating pressures are necessary for these failures since the particles are more restrained. Pressures necessary for splitting-type failures within the various cavity diameters are shown in Figure 29.

Additional tests were conducted by casting water-filled, spherical capsules in concrete specimens of the same size as previously tested. A copper-constantan thermocouple was placed in the center of each sphere of water, and voltages were amplified by means of a Dymec Amplifier (Model Dy-24-61 A-M2) and recorded on a Sanborn DC Amplifier Recorder (Model 326). Temperature measurements within 1/4 degree C were possible with this arrangement. Pressures accompanying confined freezing of water within the
Figure 23. Test Specimen Details

Figure 24. Hand-Operated, High-Pressure Pump Used to Induce Pop-Outs in Concrete Specimens
Figure 25. Photograph of Test Specimen Showing Pop-Out
Figure 26. Relationship between Depth of Burial, Void Diameter, and Pop-Out Width
Figure 27. Relationship between Failure Angle and Depth
Figure 28. Relationship between Pop-Out Pressure, Void Diameter, and Depth of Burial

Figure 29. Pressure Necessary to Produce Splitting Failure in Deeply Buried Voids
spheres (Figure 30) were then computed from the temperature data.

A typical thermogram is shown in Figure 31. The temperature at failure was -4.0°C with an accompanying pressure of 7200 psi. Following rupture, the pressure subsided and the freezing point returned to normal. Freezing thereafter was isothermal. The events apparent in Figure 31 were displayed over a series of successive cycles rather than within a single temperature excursion. This was interpreted as a manifestation of progressive damage.

Theory

Within a semi-infinite mass, there is a free-field stress distribution. When the continuity of an "assumed" homogeneous material such as concrete is interrupted by the presence of a porous and saturated aggregate particle undergoing freezing, the stresses in the region near the particle are no longer equal to the free-field stresses since a discontinuity of strains or deformations occur at the aggregate-matrix interface. Because of this discontinuity, the induced differential pressures alter the at-rest stress conditions that existed prior to freezing. Shear stresses are therefore mobilized within the concrete matrix; and, if the shearing strength of the concrete is exceeded, the material may fail in the form of a pop-out.

When a freezing aggregate particle is contained in a concrete mass, a modulus mismatch between the particle and the surrounding matrix exists. Therefore, relative displacements between the particle and the surrounding matrix will occur. As a result, shearing stresses will be mobilized along those planes which experience shearing distortions. Because of the extremely high pressures which sometimes are developed in freezing aggregate particles, the modulus of the porous particle may be assumed to be greater than that of the surrounding concrete matrix. The condition which results may be, in a general way, assumed to be analogous to the phenomenon of stress transference called "passive arching" in the soil mechanics literature.

The application of theoretical solutions implies that the material behaves elastically. It is known that such behavior may depart from ideal elasticity under the high pressures obtained. However, elastic solutions are good approximations when small displacements are involved. At large displacements, the stresses in certain regions reach the yield stress, and the stress distribution that finally results may be quite different from the stress distribution determined by considering only elastic equilibrium.

Immediately above the CD, the movement of the concrete matrix is denoted in Figure 32 by the arrows E and F. The material just above CD tends to move upward as a result of the high pressures induced by freezing and expanding water within the aggregate. The material to the sides of the aggregate particle thus tend to move laterally, creating a condition analogous to the passive pressure state. If deformations proceed far enough and the depth of burial is sufficiently small, the failure surfaces may propagate to the surface of the concrete mass and result in a pop-out failure. The lateral stresses which occur at the limiting stage of lateral compression are called passive pressures. Rankine's theory gives the passive pressure as:

\[ P_p = \gamma y \tan^2 \left( \frac{45^\circ + \phi}{2} \right) + 2c \tan \left( \frac{45^\circ + \phi}{2} \right), \]

where:
- \( P_p \) = passive pressure,
- \( \gamma \) = unit weight,
- \( y \) = depth of burial,
- \( c \) = cohesion, and
- \( \phi \) = angle of friction.
Figure 30. Concrete Pop-Out Resulting from the Freezing of an Internal Sphere of Water

Figure 31. Typical Thermogram for a Sphere of Water Encased in Concrete Undergoing Freezing
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>4800</td>
<td>1600</td>
<td>2040</td>
<td>2150</td>
</tr>
<tr>
<td>1/2</td>
<td>1</td>
<td>2400</td>
<td>800</td>
<td>1020</td>
<td>1300</td>
</tr>
<tr>
<td>1</td>
<td>1 3/8</td>
<td>6600</td>
<td>2200</td>
<td>1465</td>
<td>1800</td>
</tr>
<tr>
<td>1/2</td>
<td>1 3/8</td>
<td>3300</td>
<td>1100</td>
<td>732</td>
<td>950</td>
</tr>
<tr>
<td>1</td>
<td>1/4</td>
<td>1200</td>
<td>400</td>
<td>7850</td>
<td>4700</td>
</tr>
<tr>
<td>1/2</td>
<td>1/4</td>
<td>600</td>
<td>200</td>
<td>3925</td>
<td>4050</td>
</tr>
</tbody>
</table>

Figure 32. Passive Pressures and Pop-Outs
Using typical values of 150 lbs per cu ft for the unit weight of concrete, cohesion of 950 psi and a friction angle of \(47^\circ\), the passive pressures calculated from the above equation are in the order of 4800 psi. It is interesting to note, since the depth of burial is small, that the term in the above equation involving the depth is extremely insignificant. It is also interesting to note that the passive pressures obtained from the above equation are on the same order of magnitude as the compressive strength of the concrete.

In Figure 32, assuming that the passive pressures act over a rectangular surface with dimensions equal to the depth of burial and the void diameter (perpendicular to the plane of the figure), the passive forces can be calculated. Assuming a coefficient of passive pressure of 3, the forces acting at the top interface between the void and the concrete mass can be determined. Knowing this vertical force and the diameter of the void, a calculated value of the void pressure can be obtained. The table in Figure 32 summarizes such calculations for the range of the depth of burial and the void diameter sizes investigated in this study. It is interesting to note the remarkable agreement between the calculated void pressures and the void pressures observed to cause pop-out failure. The comparison of the calculated and observed values is extremely satisfactory in all except one case, the 1-in depth of burial for a void diameter of 1/4 in.

On the basis of the few calculations presented in the table in Figure 32, it seems reasonable to consider the passive pressure concept as being adequate in explaining the mode of failure for pop-outs. In the case of the 1/4-in diameter void buried 1 inch beneath the surface, the mode of failure may be different and thus would account for the discrepancy between the calculated and observed void pressures. Apparently, pressures originating at depths of 3 or 4 or more times the cavity size cannot significantly exceed the compressive strength of the concrete.

**SUMMARY**

It has been demonstrated that aggregates having an absorption greater than 4% are likely to rupture upon freezing; whereas, aggregates having an absorption less than 1% are generally unaffected by freeze-thaw. A composite sample of aggregate particles may, collectively, absorb less than 4% water by weight and still contain individual particles having absorptions significantly greater than 4%. Absorption tests based solely on composite samples are not adequate in this respect. Degradation of composite samples subjected to saturation and quick freezing cycles could provide a measure of the proportions of sound and unsound particles. Alternatively, each particle within a composite sample may be tested for absorption; and the proportions of deleterious material may then be resolved statistically.

Concretes may contain as much as 20% voids. Theoretically, the critical volume of water in a vessel is 91.7% of saturation (capacity). It is intuitively apparent that the system of voids within a mass of concrete is infinitely complex. It would be possible, therefore, for certain cavities or pores to be critically saturated while others may be relatively unsaturated. Here again, gross absorption alone fails to define the state of saturation - that is, unless saturation is complete in all respects. Forced saturation renders concrete and (or) aggregates susceptible to damage through freezing. The entrapment of air, evidently, causes concrete to be infinitely more difficult to saturate - probably due to the back-pressure of air in minute spherical voids. Aggregate particles are usually not "immunized" in this way and tend to saturate preferentially. Nevertheless, it seems reasonable to conclude that any mechanism whereby saturation is delayed or rendered improbable enhances the welfare or durability of concrete exposed to freezing weather.

The phenomenological aspects of the freeze-thaw mechanism have been demonstrated in three ways: 1) pressure depression of freezing temperatures, 2) rapid freezing of saturated aggregate particles, and 3) by synthesizing pop-outs in concrete. A high degree of repeatability was obtained by all methods. These phenomena have been approximated in some degree by rational theories.


14. Ledbetter, W. B., *Correlation Studies of Fundamental Aggregate Properties with Freeze-Thaw Durability of Structural Lightweight Concrete*, Texas Transportation Institute, August 1965.


APPENDIX
### TABLE 1

RESULTS OF TESTS PERFORMED ON 1-INCH AGGREGATE SPECIMENS

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>Bulk Sp.Gr. (SSD)</th>
<th>Absorption (%)</th>
<th>Porosity (%)</th>
<th>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</th>
<th>Petrographic Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.34</td>
<td>8.65</td>
<td>18.63</td>
<td>D - - - -</td>
<td>White, porous chert</td>
</tr>
<tr>
<td>2</td>
<td>2.43</td>
<td>5.93</td>
<td>13.62</td>
<td>C C C D</td>
<td>Brown, porous chert</td>
</tr>
<tr>
<td>3</td>
<td>2.66</td>
<td>4.18</td>
<td>10.67</td>
<td>D - - - -</td>
<td>Brown, porous, limonitic quartz siltstone</td>
</tr>
<tr>
<td>4</td>
<td>2.69</td>
<td>0.62</td>
<td>1.65</td>
<td>A A A A</td>
<td>Gray, dense, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>5</td>
<td>2.61</td>
<td>5.54</td>
<td>13.71</td>
<td>C D - -</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>6</td>
<td>2.63</td>
<td>0.76</td>
<td>1.99</td>
<td>A A A A</td>
<td>Brown, dense, limonitic quartz siltstone</td>
</tr>
<tr>
<td>7</td>
<td>2.55</td>
<td>1.71</td>
<td>4.28</td>
<td>B B C C</td>
<td>White, porous, fine-grained quartzite</td>
</tr>
<tr>
<td>8</td>
<td>2.63</td>
<td>0.69</td>
<td>1.79</td>
<td>A A A A</td>
<td>Brown, dense, medium-grained quartz monzonite</td>
</tr>
<tr>
<td>9</td>
<td>2.55</td>
<td>3.17</td>
<td>7.83</td>
<td>B C D -</td>
<td>Tan, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>10</td>
<td>2.71</td>
<td>0.36</td>
<td>0.96</td>
<td>A A A A</td>
<td>Gray, dense, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp. Cr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>----------</td>
<td>------------------</td>
<td>---------------</td>
<td>-------------</td>
<td>---------------------------------------------------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>11</td>
<td>2.22</td>
<td>12.52</td>
<td>24.70</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>12</td>
<td>2.65</td>
<td>0.21</td>
<td>0.55</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>13</td>
<td>2.65</td>
<td>0.15</td>
<td>0.39</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>14</td>
<td>2.61</td>
<td>1.04</td>
<td>2.69</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>15</td>
<td>2.54</td>
<td>2.66</td>
<td>6.58</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>16</td>
<td>2.66</td>
<td>0.68</td>
<td>1.80</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>17</td>
<td>2.62</td>
<td>0.65</td>
<td>1.68</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>18</td>
<td>2.64</td>
<td>0.44</td>
<td>1.16</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>19</td>
<td>2.71</td>
<td>3.05</td>
<td>8.02</td>
<td>D</td>
<td>-</td>
</tr>
<tr>
<td>20</td>
<td>2.61</td>
<td>1.26</td>
<td>3.25</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>21</td>
<td>2.66</td>
<td>1.23</td>
<td>3.23</td>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>
TABLE 1 - Continued

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>Bulk Sp.Gr. (SSD)</th>
<th>Absorption (%)</th>
<th>Poreosity (%)</th>
<th>Condition of Particle at End of Freeze-and-\Thaw Cycle Indicated</th>
<th>Petrographic Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>2.80</td>
<td>0.75</td>
<td>2.07</td>
<td>A A A A</td>
<td>Gray,dense, aphanitic dolomite</td>
</tr>
<tr>
<td>23</td>
<td>2.61</td>
<td>1.02</td>
<td>2.52</td>
<td>A A A A</td>
<td>Brown,dense,fine-grained quartz monzonite</td>
</tr>
<tr>
<td>24</td>
<td>2.52</td>
<td>3.58</td>
<td>8.72</td>
<td>B C C D</td>
<td>Brown,porous,fine-grained, limonitic quartz sandstone</td>
</tr>
<tr>
<td>25</td>
<td>2.82</td>
<td>0.43</td>
<td>1.21</td>
<td>A A A A</td>
<td>Gray,dense andesite</td>
</tr>
</tbody>
</table>
TABLE 2

RESULTS OF TESTS PERFORMED ON 3/4-INCH AGGREGATE SPECIMENS

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>Bulk Sp.Gr. (SSD)</th>
<th>Absorption (%)</th>
<th>Porosity (%)</th>
<th>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</th>
<th>Petrographic Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1  2  3  4</td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>2.65</td>
<td>0.66</td>
<td>1.75</td>
<td>A  A  A  A</td>
<td>Brown, dense, fine-grained granite</td>
</tr>
<tr>
<td>27</td>
<td>2.70</td>
<td>2.64</td>
<td>6.96</td>
<td>D  -  -  -</td>
<td>Gray, vugular, fine-grained dolomite</td>
</tr>
<tr>
<td>28</td>
<td>2.68</td>
<td>3.61</td>
<td>9.33</td>
<td>D  -  -  -</td>
<td>Gray, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>29</td>
<td>2.45</td>
<td>3.06</td>
<td>7.27</td>
<td>B  C  D  -</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>30</td>
<td>2.70</td>
<td>1.11</td>
<td>2.95</td>
<td>A  A  B  B</td>
<td>Tan, dense, medium-grained, crystalline limestone</td>
</tr>
<tr>
<td>31</td>
<td>2.62</td>
<td>0.90</td>
<td>2.33</td>
<td>A  A  A  A</td>
<td>Brown, dense, fine-grained monzonite</td>
</tr>
<tr>
<td>32</td>
<td>2.47</td>
<td>4.60</td>
<td>10.86</td>
<td>B  C  C  D</td>
<td>Brown, porous, fine-grained limonitic quartz sandstone</td>
</tr>
<tr>
<td>33</td>
<td>2.64</td>
<td>4.50</td>
<td>11.39</td>
<td>A  A  B  B</td>
<td>Brown, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>34</td>
<td>2.60</td>
<td>4.12</td>
<td>10.28</td>
<td>C  D  -  -</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>35</td>
<td>2.59</td>
<td>1.35</td>
<td>3.45</td>
<td>A  A  A  A</td>
<td>Gray, dense, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp.Gr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>----------</td>
<td>------------------</td>
<td>----------------</td>
<td>--------------</td>
<td>---------------------------------------------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>36</td>
<td>2.61</td>
<td>5.85</td>
<td>14.43</td>
<td>B C C D</td>
<td>Brown, porous, coarse-grained, weathered granite</td>
</tr>
<tr>
<td>37</td>
<td>2.42</td>
<td>5.39</td>
<td>12.36</td>
<td>C C C D</td>
<td>White, porous chert</td>
</tr>
<tr>
<td>38</td>
<td>2.51</td>
<td>6.98</td>
<td>16.36</td>
<td>D - - -</td>
<td>Brown, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>39</td>
<td>2.72</td>
<td>1.43</td>
<td>3.83</td>
<td>A A A A</td>
<td>Gray, dense, calcareous siltstone</td>
</tr>
<tr>
<td>40</td>
<td>2.72</td>
<td>1.62</td>
<td>4.34</td>
<td>A A A A</td>
<td>Gray, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>41</td>
<td>2.72</td>
<td>2.31</td>
<td>6.12</td>
<td>A A A A</td>
<td>Tan, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>42</td>
<td>2.60</td>
<td>0.61</td>
<td>1.58</td>
<td>A A A A</td>
<td>Gray, dense, aphanitic, crystalline limestone</td>
</tr>
<tr>
<td>43</td>
<td>2.69</td>
<td>0.64</td>
<td>1.72</td>
<td>A A A A</td>
<td>Tan, dense, medium-grained, crystalline limestone</td>
</tr>
<tr>
<td>44</td>
<td>2.67</td>
<td>3.51</td>
<td>9.04</td>
<td>C D - -</td>
<td>Gray, porous, medium-grained dolomite</td>
</tr>
<tr>
<td>45</td>
<td>2.68</td>
<td>0.27</td>
<td>0.73</td>
<td>B B B B</td>
<td>Gray, dense, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>46</td>
<td>2.71</td>
<td>2.72</td>
<td>7.18</td>
<td>B B C C</td>
<td>Gray, aphanitic dolomite</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp.Gr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-\nThaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>47</td>
<td>2.63</td>
<td>4.24</td>
<td>10.71</td>
<td>C D - -</td>
<td>Brown, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>48</td>
<td>2.62</td>
<td>0.45</td>
<td>1.18</td>
<td>A A A A</td>
<td>Gray, dense, fine-grained nepheline syenite</td>
</tr>
<tr>
<td>49</td>
<td>2.54</td>
<td>2.18</td>
<td>5.42</td>
<td>A A A A</td>
<td>Tan, porous, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>50</td>
<td>2.58</td>
<td>0.70</td>
<td>1.80</td>
<td>A A A A</td>
<td>Gray, dense, aphanitic, dolomitic limestone</td>
</tr>
</tbody>
</table>
TABLE 3
RESULTS OF TESTS PERFORMED ON 1/2-INCH AGGREGATE SPECIMENS

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>Bulk Sp. Gr. (SSD)</th>
<th>Absorption (%)</th>
<th>Porosity (%)</th>
<th>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</th>
<th>Petrographic Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>51</td>
<td>2.65</td>
<td>2.23</td>
<td>5.78</td>
<td>1 2 3 4</td>
<td>Tan, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>52</td>
<td>2.81</td>
<td>0.15</td>
<td>0.41</td>
<td>1 2 3 4</td>
<td>Tan, dense, medium-grained dolomite</td>
</tr>
<tr>
<td>53</td>
<td>2.72</td>
<td>2.39</td>
<td>6.35</td>
<td>1 2 3 4</td>
<td>Gray, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>54</td>
<td>2.69</td>
<td>2.68</td>
<td>7.00</td>
<td>1 2 3 4</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>55</td>
<td>2.73</td>
<td>2.04</td>
<td>5.45</td>
<td>1 2 3 4</td>
<td>Tan, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>56</td>
<td>2.71</td>
<td>2.23</td>
<td>5.93</td>
<td>1 2 3 4</td>
<td>Tan, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>57</td>
<td>2.56</td>
<td>6.20</td>
<td>14.92</td>
<td>1 2 3 4</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>58</td>
<td>2.40</td>
<td>5.84</td>
<td>13.22</td>
<td>1 2 3 4</td>
<td>Gray, porous chert</td>
</tr>
<tr>
<td>59</td>
<td>2.50</td>
<td>3.59</td>
<td>8.64</td>
<td>1 2 3 4</td>
<td>Gray, porous, calcareous siltstone</td>
</tr>
<tr>
<td>60</td>
<td>2.62</td>
<td>0.10</td>
<td>0.25</td>
<td>1 2 3 4</td>
<td>Black, dense, aphanitic basalt</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp.Gr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>----------</td>
<td>------------------</td>
<td>----------------</td>
<td>--------------</td>
<td>---------------------------------------------------------------</td>
<td>-------------------------</td>
</tr>
<tr>
<td>61</td>
<td>2.39</td>
<td>6.76</td>
<td>15.14</td>
<td>D - - -</td>
<td>Brown, porous chert</td>
</tr>
<tr>
<td>62</td>
<td>2.62</td>
<td>3.65</td>
<td>9.22</td>
<td>C D - -</td>
<td>Tan, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>63</td>
<td>2.68</td>
<td>3.05</td>
<td>7.95</td>
<td>D - - -</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>64</td>
<td>2.69</td>
<td>2.01</td>
<td>5.31</td>
<td>A A A A</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>65</td>
<td>2.84</td>
<td>2.23</td>
<td>6.20</td>
<td>A A A C</td>
<td>Brown, porous, medium-grained, weathered granite</td>
</tr>
<tr>
<td>66</td>
<td>2.52</td>
<td>1.75</td>
<td>4.33</td>
<td>A A B B</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>67</td>
<td>2.60</td>
<td>0.86</td>
<td>2.21</td>
<td>A A A A</td>
<td>Gray, dense, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>68</td>
<td>2.62</td>
<td>1.08</td>
<td>2.80</td>
<td>A A A A</td>
<td>Gray, dense, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>69</td>
<td>2.31</td>
<td>8.38</td>
<td>17.88</td>
<td>A A B B</td>
<td>White, porous chert</td>
</tr>
<tr>
<td>70</td>
<td>2.69</td>
<td>1.55</td>
<td>4.11</td>
<td>A A B B</td>
<td>Gray, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>71</td>
<td>2.58</td>
<td>4.12</td>
<td>10.22</td>
<td>B D - -</td>
<td>Gray, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp.Gr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------</td>
<td>----------------</td>
<td>--------------</td>
<td>-----------------------------------------------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>72</td>
<td>2.73</td>
<td>1.56</td>
<td>4.18</td>
<td>B B B C</td>
<td>Gray, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>73</td>
<td>2.56</td>
<td>1.16</td>
<td>2.94</td>
<td>A A A A</td>
<td>Gray, dense chert</td>
</tr>
<tr>
<td>74</td>
<td>2.61</td>
<td>0.82</td>
<td>2.12</td>
<td>A A A A</td>
<td>Gray, dense, aphanitic, crystalline limestone</td>
</tr>
<tr>
<td>75</td>
<td>2.61</td>
<td>0.22</td>
<td>0.57</td>
<td>A A A A</td>
<td>Tan, dense, fine-grained quartzite</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp.Gr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>----------</td>
<td>------------------</td>
<td>----------------</td>
<td>--------------</td>
<td>---------------------------------------------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>76</td>
<td>2.78</td>
<td>0.35</td>
<td>0.96</td>
<td>1 2 3 4</td>
<td>Gray, dense, fine-grained dolomite</td>
</tr>
<tr>
<td>77</td>
<td>2.53</td>
<td>3.08</td>
<td>7.56</td>
<td>B  C  D -</td>
<td>Tan, porous siltstone</td>
</tr>
<tr>
<td>78</td>
<td>2.59</td>
<td>1.79</td>
<td>4.57</td>
<td>A  A  A A</td>
<td>Tan, porous, aphanitic, crystalline limestone</td>
</tr>
<tr>
<td>79</td>
<td>2.53</td>
<td>4.18</td>
<td>10.14</td>
<td>B  B  C  C</td>
<td>Gray, porous, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>80</td>
<td>2.77</td>
<td>1.82</td>
<td>4.95</td>
<td>A  A  B  B</td>
<td>Gray, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>81</td>
<td>2.74</td>
<td>0.51</td>
<td>1.39</td>
<td>A  A  A  A</td>
<td>Pink, dense, fine-grained granite</td>
</tr>
<tr>
<td>82</td>
<td>2.53</td>
<td>2.82</td>
<td>6.94</td>
<td>A  B  C  C</td>
<td>Tan, porous chert</td>
</tr>
<tr>
<td>83</td>
<td>2.62</td>
<td>1.52</td>
<td>3.92</td>
<td>A  B  C  D</td>
<td>Tan, porous, fine-grained quartzite</td>
</tr>
<tr>
<td>84</td>
<td>2.65</td>
<td>4.18</td>
<td>10.60</td>
<td>C  C  C  D</td>
<td>Tan, porous, calcareous claystone</td>
</tr>
<tr>
<td>85</td>
<td>2.39</td>
<td>6.94</td>
<td>15.49</td>
<td>B  B  D -</td>
<td>Brown, porous chert</td>
</tr>
<tr>
<td>86</td>
<td>2.76</td>
<td>1.97</td>
<td>5.33</td>
<td>B  B  C  D</td>
<td>Tan, porous, fine-grained, dolomitic limestone</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp. Gr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>-----------</td>
<td>-------------------</td>
<td>----------------</td>
<td>--------------</td>
<td>---------------------------------------------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>87</td>
<td>2.36</td>
<td>7.05</td>
<td>15.54</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>88</td>
<td>2.62</td>
<td>0.26</td>
<td>0.68</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>89</td>
<td>2.62</td>
<td>1.67</td>
<td>4.29</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>90</td>
<td>2.65</td>
<td>2.72</td>
<td>7.01</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>91</td>
<td>2.69</td>
<td>0.08</td>
<td>0.22</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>92</td>
<td>2.79</td>
<td>0.86</td>
<td>2.37</td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>93</td>
<td>2.79</td>
<td>0.91</td>
<td>2.53</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>94</td>
<td>2.68</td>
<td>1.02</td>
<td>2.71</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>95</td>
<td>2.67</td>
<td>0.28</td>
<td>0.74</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>96</td>
<td>2.60</td>
<td>0.28</td>
<td>0.73</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>97</td>
<td>2.77</td>
<td>0.95</td>
<td>2.61</td>
<td>B</td>
<td>D</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp.Gr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>----------</td>
<td>------------------</td>
<td>----------------</td>
<td>--------------</td>
<td>---------------------------------------------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td>98</td>
<td>2.62</td>
<td>0.27</td>
<td>0.70</td>
<td>A A A A</td>
<td>White, dense, coarse-grained vein quartz</td>
</tr>
<tr>
<td>99</td>
<td>2.30</td>
<td>7.25</td>
<td>15.55</td>
<td>A B B D</td>
<td>Tan, porous chert</td>
</tr>
<tr>
<td>100</td>
<td>2.64</td>
<td>0.49</td>
<td>1.29</td>
<td>A A A A</td>
<td>Gray, dense, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>Spec. No.</td>
<td>Bulk Sp.Gr. (SSD)</td>
<td>Absorption (%)</td>
<td>Porosity (%)</td>
<td>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</td>
<td>Petrographic Description</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------</td>
<td>----------------</td>
<td>--------------</td>
<td>---------------------------------------------------------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>111</td>
<td>2.71</td>
<td>0.13</td>
<td>0.34</td>
<td>A A A A</td>
<td>Black, dense, aphanitic basalt</td>
</tr>
<tr>
<td>112</td>
<td>2.65</td>
<td>3.71</td>
<td>9.46</td>
<td>A B B C</td>
<td>Tan, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>113</td>
<td>2.36</td>
<td>9.31</td>
<td>20.10</td>
<td>B B B C</td>
<td>Gray, porous, fine-grained syenite</td>
</tr>
<tr>
<td>114</td>
<td>2.47</td>
<td>0.47</td>
<td>1.16</td>
<td>A A A A</td>
<td>Black, dense, fine-grained diorite</td>
</tr>
<tr>
<td>115</td>
<td>2.62</td>
<td>0.84</td>
<td>2.18</td>
<td>A A A A</td>
<td>Black, dense, fine-grained diorite</td>
</tr>
<tr>
<td>116</td>
<td>2.64</td>
<td>1.12</td>
<td>2.94</td>
<td>A A A A</td>
<td>Pink, dense, medium-grained monzonite</td>
</tr>
<tr>
<td>117</td>
<td>2.41</td>
<td>3.22</td>
<td>7.52</td>
<td>A A A A</td>
<td>Brown, porous, fine-grained, limonitic quartz sandstone</td>
</tr>
<tr>
<td>118</td>
<td>2.63</td>
<td>0.47</td>
<td>1.23</td>
<td>A A A A</td>
<td>Brown, dense, fine-grained syenite</td>
</tr>
<tr>
<td>119</td>
<td>2.40</td>
<td>6.90</td>
<td>15.50</td>
<td>B B C D</td>
<td>Tan, porous, aphanitic, crystalline limestone</td>
</tr>
<tr>
<td>120</td>
<td>2.58</td>
<td>0.13</td>
<td>0.34</td>
<td>A A A A</td>
<td>Gray, dense, fine-grained granite</td>
</tr>
<tr>
<td>121</td>
<td>2.59</td>
<td>0.29</td>
<td>0.74</td>
<td>A A A A</td>
<td>Pink, dense, medium-grained syenite</td>
</tr>
</tbody>
</table>
### TABLE 5

RESULTS OF TESTS PERFORMED ON NO. 4 SIEVE SIZE AGGREGATE SPECIMENS

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>Bulk Sp.Gr. (SSD)</th>
<th>Absorption (%)</th>
<th>Porosity (%)</th>
<th>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</th>
<th>Petrographic Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1  2  3  4</td>
<td></td>
</tr>
<tr>
<td>101</td>
<td>2.49</td>
<td>7.02</td>
<td>16.34</td>
<td>A  B  B  B</td>
<td>Brown, porous, fine-grained sandstone</td>
</tr>
<tr>
<td>102</td>
<td>2.76</td>
<td>0.04</td>
<td>0.10</td>
<td>A  A  A  A</td>
<td>Gray, dense, aphanitic andesite</td>
</tr>
<tr>
<td>103</td>
<td>2.70</td>
<td>2.56</td>
<td>6.72</td>
<td>B  C  C  C</td>
<td>Tan, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>104</td>
<td>2.72</td>
<td>1.52</td>
<td>4.07</td>
<td>A  B  C  D</td>
<td>Gray, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>105</td>
<td>2.47</td>
<td>3.57</td>
<td>8.52</td>
<td>A  A  A  A</td>
<td>Brown, porous chert</td>
</tr>
<tr>
<td>106</td>
<td>2.77</td>
<td>0.12</td>
<td>0.32</td>
<td>A  A  A  A</td>
<td>Gray, dense, aphanitic dolomite</td>
</tr>
<tr>
<td>107</td>
<td>2.42</td>
<td>4.37</td>
<td>10.13</td>
<td>A  A  B  C</td>
<td>Gray, porous, aphanitic dolomite</td>
</tr>
<tr>
<td>108</td>
<td>2.45</td>
<td>3.11</td>
<td>7.38</td>
<td>A  A  A  B</td>
<td>Tan, porous, fine-grained dolomite</td>
</tr>
<tr>
<td>109</td>
<td>2.31</td>
<td>8.33</td>
<td>17.77</td>
<td>B  C  C  D</td>
<td>Gray, porous siltstone</td>
</tr>
<tr>
<td>110</td>
<td>2.51</td>
<td>2.53</td>
<td>6.18</td>
<td>A  A  A  A</td>
<td>Brown, porous, limonitic chert</td>
</tr>
</tbody>
</table>
TABLE 5 - Continued

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>Bulk Sp.Gr. (SSD)</th>
<th>Absorption (%)</th>
<th>Porosity (%)</th>
<th>Condition of Particle at End of Freeze-and-Thaw Cycle Indicated</th>
<th>Petrographic Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>122</td>
<td>2.64</td>
<td>0.42</td>
<td>1.11</td>
<td>1 2 3 4</td>
<td>Gray, dense, fine-grained, crystalline limestone</td>
</tr>
<tr>
<td>123</td>
<td>2.73</td>
<td>0.11</td>
<td>0.29</td>
<td>A A A A</td>
<td>Gray, dense, aphanitic, calcic dolomite</td>
</tr>
<tr>
<td>124</td>
<td>2.60</td>
<td>0.20</td>
<td>0.52</td>
<td>A A A A</td>
<td>Pink, dense, fine-grained monzonite</td>
</tr>
<tr>
<td>125</td>
<td>2.57</td>
<td>3.53</td>
<td>8.78</td>
<td>C D -</td>
<td>Brown, porous, fine-grained, weathered granite</td>
</tr>
</tbody>
</table>