NONBLOATED SYNTHETIC AGGREGATE CONCRETE

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PREFACE

The primary objective of the synthetic aggregate research being conducted by the Texas Transportation Institute is to develop a recommended acceptance criterion for synthetic aggregates for use in all phases of highway construction.

This is the fourteenth report issued under Research Study 2-8-65-81, one of the synthetic aggregate research studies being conducted at the Texas Transportation Institute in the cooperative research program with the Texas Highway Department and the Federal Highway Administration. The first thirteen reports are:

"Correlation Studies of Fundamental Aggregate Properties with Freeze-Thaw Durability of Structural Lightweight Concrete," by W. B. Ledbetter, *Research Report 81-1,* Texas Transportation Institute, August, 1965.

"Effect of Degree of Synthetic Lightweight Aggregate Pre-Wetting on the Freeze-Thaw Durability of Lightweight Concrete," by C. N. Kanabar and W. B. Ledbetter, *Research Report 81-2,* Texas Transportation Institute, December, 1966.

"Aggregate Absorption Factor as an Indicator of the Freeze-Thaw Durability of Structural Lightweight Concrete," by W. B. Ledbetter and Eugene Buth, *Research Report 81-3,* Texas Transportation Institute, February, 1967.

"Flexural Fatigue Durability of Selected Unreinforced Structural Lightweight Concretes," by J. C. Chakrabarti and W. B. Ledbetter, *Research Report 81-4,* Texas Transportation Institute, July, 1967.

"Suitability of Synthetic Aggregates Made from Clay-Type Soils for Use in Flexible Base," by W. M. Moore, Richard S. Van Pelt, F. H. Scrivner, and George W. Kunze, *Research Report 81-5,* Texas Transportation Institute, February, 1968.

"Performance Studies of Synthetic Aggregate Concrete," by C. E. Buth, H. R. Blank, and R. G. McKeen, *Research Report 81-6,* Texas Transportation Institute, March, 1969.

"Fundamental Factors Involved in the Use of Synthetic Aggregate Portland Cement Concrete," by W. B. Ledbetter, C. E. Sandstedt, and A. H. Meyer, *Research Report 81-7,* Texas Transportation Institute, October, 1969.

"A Sandblast Abrasion Test for Synthetic Aggregate Evaluation," by James T. Houston and W. B. Ledbetter, *Research Report 81-8,* Texas Transportation Institute, October, 1969.

"Studies of the Thermal Transformation of Synthetic Aggregates Produced in a Rotary Kiln," by James T. Houston, H. R. Blank and George W. Kunze, *Research Report 81-9,* Texas Transportation Institute, November, 1969.

"Effect of Synthetic Aggregate Thermal Transformation on Performance of Concrete," by James T. Houston and W. B. Ledbetter, *Research Report 81-10,* Texas Transportation Institute, October, 1969.

"Evaluation of Shrinkage-Cracking Characteristics of Structural Lightweight Concrete," by R. G. McKeen and W. B. Ledbetter, *Research Report 81-11,* Texas Transportation Institute, October, 1969.

"Fired-Clay Aggregates for Use in Flexible Bases," by W. M. Moore, *Research Report 81-12,* Texas Transportation Institute, November, 1969.

"Shrinkage-Cracking Characteristics of Structural Lightweight Concrete," by W. B. Ledbetter and Gisela Nichols, *Research Report 81-13,* Texas Transportation Institute, August, 1970.

In addition, a special report has been published under this research study. The report is:

"A Recommended Synthetic Coarse Aggregate Classification System (Revised August 1969)," by W. B. Ledbetter, B. M. Gallaway, W. M. Moore, and Eugene Buth, *Special Report,* Texas Transportation Institute, August, 1969.

The authors wish to thank all members of the Institute who assisted in this research.

The authors wish to acknowledge the guidance and assistance given by the advisory committee for this study. The members are as follows: (a) Texas Highway Department personnel—Mr. Kenneth D. Hankins, Study Contact Representative and Research Area Representative; Mr. H. A. Sandberg, Jr., Materials and Tests Division Representative; and Mr. Clarence R. Rea, Bridge Division Representative; (b) Federal Highway Administration personnel—Mr. Edward V. Kristaponis, Division Representative; and Mr. W. J. Lindsey, Regional Representative.

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Federal Highway Administration.

ABSTRACT

Thirty-five batches of concrete made from 17 different aggregates made from 5 different clays were investigated. Aggregates were tested for chemical, physical, and mechanical durability by a variety of tests. Relations between clay raw material, aggregate and processing parameters were determined. Concretes were tested for strength, chemical and physical durability. Results indicate that (a) nonbloated synthetic aggregate is potentially useful in portland cement concrete *base,* (b) drying of concrete greatly improves its freeze-thaw durability, (c) aggregate quality varies significantly with processing parameters, and (d) improvements in aggregate quality control test procedures are required.

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CHAPTER 1
Introduction and Summary

1.1 Purpose

The purpose of this phase of the study was to determine the feasibility of using nonbloated synthetic aggregate for making portland cement concrete, especially concrete that is strong and chemically and physically durable in pavements and other highway structures. This is part of an overall objective to develop a recommended synthetic aggregate classification system and performance standards for synthetic aggregate portland cement concrete.

1.2 Scope

Thirty-five batches of concrete were made from 17 different aggregates made from five different clays. Controlled processing variables involved in the production of the aggregates included firing temperature, retention time and raw material. An investigation of each material—clay, aggregate, concrete—was made.

1.3 Conclusions

The following conclusions are drawn from the findings of this investigation, within the range of aggregate and concrete variables considered:

Aggregates

1. Process variables (firing temperature, retention time, and clay raw material) influenced aggregate physical properties and durability as follows:

a. Significant variations with increased kiln temperature were: decreased unit weight, specific gravity, absorption and saturation; improved freeze-thaw resistance; and a trend from innocuous to deleterious classification in the aggregate potential reactivity test.

b. Increased retention time resulted in increased rate and degree of saturation.

c. The type of clay raw material significantly influenced: aggregate potential reactivity, absorption and saturation, and aggregate freeze-thaw resistance.

d. Less significant trends observed as firing temperature increased were: slightly lower mechanical durability (Texas sandblast test) and a very small reduction in pressure slaking loss and 5N NaOH test loss.

e. Retention time had no measurable effect on: unit weight, specific gravity, aggregate freeze-thaw resistance, potential reactivity or mechanical durability.

f. The type of clay charge had no observable effect on: absolute specific gravity.

2. Intercorrelation among results from a) aggregate freeze-thaw tests, b) 100-minute saturation tests, and c) saturation coefficient tests indicates that, for some purposes insofar as synthetic aggregates made from clay are concerned), only the simplest of these tests need be conducted in synthetic aggregate evaluation.

3. Mechanical durability (Texas sandblast test) decreases significantly with decreased aggregate porosity.

Concrete

4. Strong, durable, chemically inert concrete can be made from synthetic nonbloated aggregate.

5. For the same strength more cement will be required in concrete made with synthetic nonbloated aggregate than that made with natural dense aggregate.

6. Cement factors in excess of 7 sks per cu yd appear to be required to obtain the 650 psi modulus of rupture (7-day, center point) specified by the Texas Highway Department for concrete pavement.

7. Prospects for use of synthetic nonbloated aggregate in concrete base appear favorable.

8. Drying of concrete greatly improves its freeze-thaw durability.

9. The splitting tensile strength values exhibit a lower coefficient of variation than the flexural strength values.

Concrete-Aggregate Relations

10. The aggregate potential reactivity test is not a reliable indication of deleterious expansion for concretes made with synthetic aggregates.

11. The aggregate freeze-thaw test can indicate synthetic aggregate which will be potentially unsafe with regard to concrete freeze-thaw durability, but may also reject materials which will perform satisfactorily.

1.4 Recommendations

1. The significant variations shown in the properties of synthetic aggregate with changes in raw materials and process parameters indicate that close control of aggregate quality by application of appropriate test procedures should be maintained to ensure adequate quality control of concrete made from such aggregates.

2. Improvements should be made in test procedures and specifications applied to quality control of nonbloated synthetic aggregates used in concrete. The research required to develop an adequate set of such specifications should emphasize the objective of achieving a close relation between test results and concrete performance parameters. Such tests should include a more applicable potential reactivity test and a test (or tests) having a more definitive relation to concrete strength.

3. Further research should be directed to determining why the results from the aggregate potential reactivity test fail to relate to concrete expansion in the autoclave expansion test with the ultimate objective of developing a more definitive aggregate laboratory test. Until such a laboratory test is developed, aggregate potential reactivity test should be dropped from the recommended coarse aggregate classification system, and the concrete autoclave expansion test substituted for it.

4. Some consideration should be given to substitution of a requirement for the splitting tensile strength in place of the flexural strength in the specifications from some grades of concrete made from synthetic aggregates.

1.5 Implementation Statement

Based on the results of this investigation, it is suggested that the Texas Highway Department permit the use of synthetic nonbloated aggregates in a portland cement concrete *base* on an experimental basis.

The above statement represents the combined opinions of the study contact representative and the authors and should not be construed as departmental policy.

CHAPTER 2

Background

A review of the technical literature reveals the almost universal use of synthetic aggregate: bridges and buildings have been built in England, Germany, France, Russia, Japan, and Australia. Its widespread and increasing use has stimulated research into its manufacture and application. Design assumptions are based on knowledge of the properties of engineering materials, and continuing study of the various kinds of synthetic aggregate will no doubt lead to even greater use.

One type of synthetic aggregate is made in a rotary kiln from clay or shale. Some of the clays characteristically expand, or bloat, in the manufacturing process and some do not. The latter are referred to as "nonbloated." These nonbloated synthetic aggregates are the subject of this investigation and henceforth "aggregate" will mean the nonbloated type unless preceded by a qualifying adjective.

Clays expand, or bloat, when subjected to high temperatures because some part of the clay is transformed into a gas while the clay is in a pyroplastic state. Bubbles are trapped in the clay producing a vesicular, lightweight material. In the nonbloating clay, the gas is either not produced, or escapes, and no expansion takes place; but the clay does become dehydrated and hard and usually somewhat denser than the bloated type. No completely satisfactory explanation of this phenomenon has yet been put forth, although a number have been proposed. Hill and Crook (1) * discount the theory (2) that breakdown of carbonates to produce CO_2 is the cause of bloating. They also reject the theory (3) that the dehydroxylation of micaceous minerals is the source of the gas on the ground that the dehydroxylation temperature is considerably below the bloating temperature. This appears reasonable for if the hydroxyl water is driven off before the clay reaches a pyroplastic condition, it is not likely to entrap any of the gas. Hill and Crook contend that the main cause of bloating is the reduction of ferric iron to the ferrous state and that, for good bloating, the raw material should contain 5 to 15 percent iron oxide. As will be seen, none of the clays used in this investigation refute this, for all had an iron content below 2 percent. One anomaly did appear in the Hill and Crook data: one sample containing 9.78 percent Fe_2O_3 did not bloat which could indicate that iron is not the sole factor controlling bloating. They add, ". . . no one reaction can be put forward as the sole cause of the evolution of gas . . . but the results of this study show that in the majority of cases, it is not necessary to postulate any source of gas other than that provided by the reduction of ferric iron." On the other hand, a plant in Japan claims to produce lightweight aggregate from shale with an Fe_2O_3 content of 1.96 percent (4).

In the manufacture of brick and tile it is imperative that the product be dimensionally uniform. It is not expansion or shrinkage per se, for this can be allowed for in the "green ware," but it is the differential expansion that is the bane of the brick maker. Fortunately, dimensional stability is not a requisite of synthetic aggregate. In an analysis (5) of the clays used to manufacture clay tile several showed percentages of Fe_2O_3 in the range that Hill and Crook (1) say are required for good bloating. But, either these clays were not heated to their bloating temperature or, somehow, bloating was prevented. The rate of firing may be a factor, for the retention times used in the manufacture of brick and tile are much longer and therefore if the temperature at which gas is produced is somewhat different from that at which the clay softens, then all of the gas may escape and no bloating take place. In the rotary kiln the gas may not escape so quickly at the high rate of heating and some will still be available for entrapment by the pyroplastic clay. According to Grim, something like this happens in the case of organic material in the clay. The oxidation of organic material begins at temperatures on the order of 200° to 300°C, but "it takes time to eliminate the organic material in firing a clay body, and these components must be eliminated before the ware is vitrified; otherwise, the gases produced may expand and disrupt the body" (6).

Analysis of both raw material and the product is very difficult. The clay varies with geography and geology of the source and the product varies with the raw materials and manner of processing. According to Grim, ". . . by about 900°C . . . all of the clay minerals are completely dehydrated . . . in the presence of a large quantity (more than about 5 percent) of iron, alkalies, and alkaline earths (fluxes) . . . there may be little development of distinct high temperature phases" (6).

Moffat, et al., say "The phases which are present in a ceramic and the scale on which they are distributed depend on how the ceramic is made, that is, on its processing" (7). To quote a manufacturer, ". . . the materials themselves were the most erratic because they were a product of nature, they contained many impurities, they determined the character of the processed material . . . specific gravity, variation in absorption and rate of absorption. Good quality control is essential" (8).

Synthetic aggregates can be divided then, into two categories, bloated and nonbloated. The main advantage of bloated aggregate is its light weight. Since the dead load of a concrete structure often constitutes a major portion of the load it must sustain, use of synthetic lightweight aggregate results in smaller beams and columns, shallower foundations, lighter formwork, and less handling costs. It has the additional merit of being a better insulator than normal concrete. Thus a great deal of literature exists concerning this type of aggregate and there are several associations actively promoting its use. Nonbloated aggregate has these same advantages but to a lesser degree; it is intermediate between lightweight and naturally dense aggregate in these respects as well. In contrast with lightweight aggregate, a search of the literature produced very little information specifically pertaining to the nonbloated variety.

*Numbers in parentheses refer to references contained in Section 5.2.

The question of which aggregate to use in a given structure is finally resolved by economic considerations. Synthetic aggregate will be used if it results in a net savings even though the unit cost of the aggregate may exceed that of natural rock. If all other variables are constant, the lighter the aggregate the greater the savings, and bloated aggregate has the advantage over nonbloated. However, there are situations where weight is not such an important factor, as in highway construction, and design is governed primarily by wheel loads rather than weight of the pavement. There are also areas where natural aggregate sources have been depleted or never existed. Where these two conditions exist, synthetic aggregates have found ready application. If bloating clays are not readily available, then nonbloated aggregates may become economically acceptable.

A few commercial plants now produce nonbloated synthetic aggregates and they are being used as base material and in asphaltic concrete. No instances of its use in portland cement concrete could be found, although it seems reasonable to expect that good quality concrete could be made with such aggregates.

CHAPTER 3

Experimental Program

3.1 The Clays

Raw materials were obtained from five different sites including two commercial plant sites. The details of these raw materials and their respective sites are given in the following paragraphs.

1. B—Raw material used by a commercial aggregate plant, located in southern Texas. It is red-colored, silty clay containing abundant white concretions of calcium carbonate. This red clay is an alluvial deposit from a former channel of the Colorado River or one of its tributaries. Five aggregates were produced out of this raw material.

2. T—Raw clay used by another commercial aggregate plant in southern Texas. It is a yellow clay. Some greenish-gray and less oxidized clay is mixed with it. It also contains some extremely fine silt and numerous hard, white concretions of calcium carbonate up to ¾ in. in diameter. The pit from which this material was obtained was broad and shallow and appeared to be entirely in the Beaumont clay formation.

3. RGH—Raw material for RGH aggregates came from the site adjacent to the Gifford-Hill aggregate plant near Bryan, Texas. This is red-colored clay which apparently came from one of the Brazos River terrace deposits and not from the underlying Eocene formation. It is a silty clay, containing some fine sand, white specks, and small pebbles. Calcium carbonate was found throughout the mass of the raw material. Traces of sulfate were also present.

4. GEW—This raw material came from near the Easterwood Airport at College Station, Texas. This is a clay and probably belongs to the Easterwood shale member of the Yegua formation. It is a gray silty clay, containing very fine sand. It also contains a very few small white and brown spots. The $CaCO_3$ test produced negative results. Sulfate was present in this raw clay.

5. FAC—The raw material for FAC aggregates came from a source in northwest Texas. It is a red-colored clay. This clay was not added to the study until after the clay analysis phase, therefore it is not discussed further here.

The general location of the source of all clays is shown in Figure 3-1.

Precision in the analysis of clays has not developed to the degree where conclusions can be drawn with desired certainty. According to Kelley (9) almost all soils contain two or more clay minerals and this compounds the problem of identification. He says, "Unless a given kind of clay comprises a considerable percentage of the total, none of these methods gives absolute identification or permits accurate determination of the amounts present, but by combining two or more methods, it is usually possible to determine what type of clay predominates, and in some cases, at least, to make an approximate estimate of the relative amounts of the different types that are present."

Grim (10) has pointed out that components of the soil other than clay minerals may play a dominant role in firing characteristics. It is well known that oxides of iron and the alkali earth metals act as fluxes to lower the fusion point of clays. The temperature at which the swelling potential of montmorillonite is permanently lost is a function of the absorbed cation, being low (105 to 225°C) for lithium and high (390 to 490°C) for sodium. Cation exchange is purposely affected in preparation of specimens for X-ray diffraction and results for these prepared specimens may not be the same as for the original, unaltered soil. Furthermore, removal of cementitious iron compounds may result in the dis-

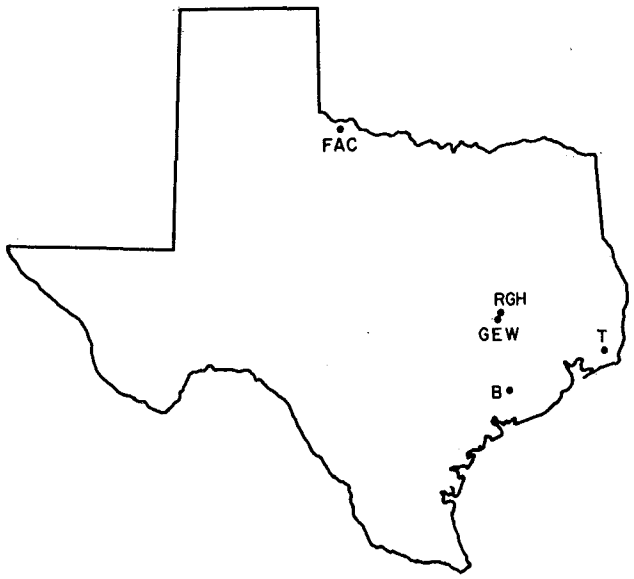

Figure 3-1. Geographic location of clays.

integration of aggregate particles whose properties could be a factor in the original soil.

Based on X-ray diffraction, the clays examined are predominately montmorillonite with lesser amounts of kaolinite and illite.

3.2 The Aggregates

3.2.1 General

Fourteen aggregates were made in the research rotary kiln at the Research Annex of Texas A&M University. In addition, three commercially produced aggregates were obtained; two were from the same plant but were produced at different firing temperatures and retention times. These seventeen aggregates were used in this investigation.

The aggregate designation is a combination of the clay designation and a numerical and/or literal symbol usually keyed to the kiln operating parameters used when the aggregate was produced. Those aggregates that were commercially produced will have the suffix -CO. Table 3-1 provides a cross reference to the aggregate designation and the kiln operating parameters. It should be kept in mind that the temperatures given in the table were read from an optical pyrometer pointed directly at the area of contact between burner flame and aggregate, and therefore represent the highest temperature to which the clays were subjected.

Two considerations were of primary concern in selecting test methods to be applied in this investigation for laboratory evaluation of the aggregates:

1. The test results should be useful in physically describing the material and/or should be related to some aggregate performance characteristic which might influence concrete performance.

2. The test results should be useful in synthetic aggregate quality control. That is, the data should be sensitive to changes in one or more of the process variables (firing temperature, retention time, or raw material).

TABLE 3-1. AGGREGATE PROCESSING PARAMETERS

Aggregate	Max. Temp. °F	Retention Time min.
TCO	~1500	
T1A	2110	29
T1B	2110	45
T1C	1900	30
T1D	1700	30
FAC	2050	37
BCO-S2	1960	45
BCO-S3	2010	39
B1A	1960	19
B1B	1960	45
B1C	1560	46
B1D	1560	17
B1E	1760	18
GEW-14A	1400	26
GEW-18A	1800	26
RGH-14A	1400	26
RGH-18A	1800	26

The tests applied to the synthetic aggregates in this investigation were selected on the basis of previous studies in this program as reported and recommended by Das and Ledbetter (19, 28) and Buth, Blank, and McKeen (23). The tests used are classified in accordance with these recommendations and are:

Test Category	Aggregate Test Method
General Physical Description	Sieve Analysis (Tex-401-A) Unit Weight (Tex-404-A) Dry Bulk Specific Gravity (Tex-433-A) Absolute Specific Gravity (Tex-433-A)
Physical Durability	Aggregate Absorption and Saturation (Tex-433-A) Saturation Coefficient (ASTM C67) Aggregate Freeze-Thaw Loss (Tex-432-A)
Chemical Durability	Pressure Slaking Loss (Tex-431-A) 5N Sodium Hydroxide Test (11) Potential Reactivity (ASTM C289)
Mechanical Durability	Los Angeles Abrasion Loss (Tex-410-A) Texas Sandblast Abrasion Loss (12)

The test methods, results, and relation of results to process variables are discussed in the following sections, under each of the categories named above.

3.2.2 General Physical Description

Unit Weight. This is THD Test Method Tex-404-A (19) and consists simply of determining the weight of a cubic foot of dry loose aggregate by weighing a known volume. The unit weight will vary to some extent with the gradation and packing. Values tabulated in Table 5-2 are for the aggregate as it comes from the kiln (Column I) and for the fraction passing a ½ in. and retained on a No. 4 sieve (Column II).

Sieve Analysis. A sieve analysis was run on the aggregate just as it came from the kiln in accordance with Tex-401-A. Results are tabulated in Tables 5-6, 5-7, and 5-8.

Specific Gravity. Because synthetic aggregates absorb water, their volume cannot be determined directly by measurement of water displaced. A method, known as the Bryant Method (13) is used (Tex-433-A). The specific gravity calculated from this is called "dry bulk" because the volume used includes the voids in the aggregates.

The absolute specific gravity is determined by submerging the aggregate in water and applying a pressure of 1200 psi and measuring the volume displaced (Tex-433-A). This volume is assumed to equal that of all solid material in the aggregate since all voids, or at least all fillable voids, are assumed to be filled with water under these conditions. Thus the specific gravity obtained by this method is called "absolute." If there are voids which are not filled, then the specific gravity is not absolute in the usual sense; however, for purposes of concrete mix design it may be considered so. The results of both specific gravity tests are summarized in Table 5-1.

Effect of Process Variables. The dry bulk specific gravities of the aggregates generally decreased with increase in burning temperature, although not linearly

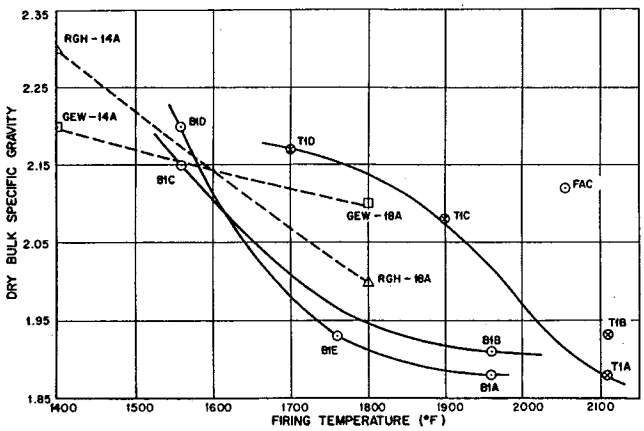

Figure 3-2. Relationship between firing temperature and dry bulk specific gravity.

and not in the same manner for different clays (Figure 3-2). Unit weight will tend to vary in the same manner. However, note in Figure 3-3 that while the trend of absolute specific gravity with kiln temperature is the same as noted for bulk specific gravity, the influence of clay charge type is no longer as evident. The trend observed for absolute specific gravity is consistent with data reported in Grim (10). At the same time, there may be a small amount of bloating from these so-called nonbloating clays resulting from an increase in volume from escaping gasses. This effect would tend to vary somewhat with the clay charged to the kiln, thus indicating why different clays produced varying dry bulk specific gravity, but did not result in varying absolute specific gravity at a given firing temperature.

In the firing of these aggregates it was found that there was very little attrition and thus little change between the sieve analysis of the charge and the burned aggregate. Accordingly, gradation control to meet a given specification might be accomplished by gradation control of the clay charge and thus eliminate crushing and/or screening after firing.

3.2.3 Physical Durability

Absorption and Saturation. The Bryant Method (13) also yields data for determining absorption and

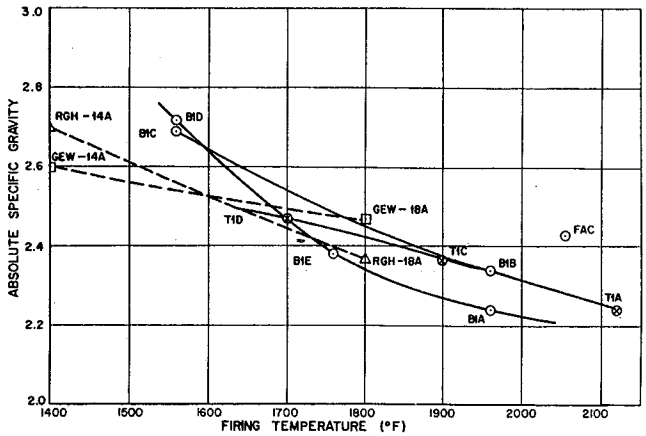

Figure 3-3. Relation between firing temperature and absolute specific gravity.

saturation characteristics and porosity of synthetic aggregates. Absorption and saturation are functions of time; to be meaningful the time at which absorption is measured must be determined. The Texas Highway Department (THD) uses 100 minutes as a reference time (Tex-433-A) (14). The range of 100-minute absorption was large for these aggregates, the largest (TCO) being fifty times the smallest (FAC). Figure 3-4 shows the absorption-time curves for these two aggregates as well as one intermediate curve. Curves for all the other aggregates fell between these two extremes. Besides the variation in amounts absorbed, the curves show the variation in rates of absorption. The theoretical maximum possible absorptions of TCO, B1B, and FAC are 10, 9, and 4 percent, respectively. In 100 minutes of soaking these aggregates become 69, 29, and 3 percent saturated by volume. These absorption and saturation characteristics exert a great influence on concrete mix design and on concrete freeze-thaw durability. The FAC aggregate behaves almost like natural dense stone but the TCO aggregate resembles a collection of small sponges with B1B somewhere between. Figure 3-5 shows graphically the 100-minute absorption and saturation values for each aggregate. The numerical data are tabulated in Table 5-1.

Aggregate Freeze-Thaw Test. This test, developed by Gallaway (21) measures the amount of the sample passing a given size sieve, on which it has previously been retained, after 50 cycles of freezing and thawing while partially submerged in water (Tex-432-A). Test values are shown in Table 5-3. A maximum loss of 7 percent has been recommended for aggregate used in concrete pavement and 15 percent for base material (14). However, as pointed out by Verbeck and Landgren (29) interpretation of aggregate freeze-thaw data in terms of concrete frost resistance is not a simple question since the behavior of both the aggregate and surrounding paste or mortar are involved. For example, they point out that permeability of the paste will govern water migration, and that this permeability depends significantly upon the water-cement ratio and the degree of hydration of the cement. Additionally, saturated aggregates of low porosity may accommodate pore water freezing by simple elastic expansion. Failure with aggregates of moderate to high porosity may be due to internal failure of the aggregate or may be the result of failure in the paste immediately adjacent to the aggregate particle due to aggregate pore water displacement. The magnitude of the hydraulic pressures developed depends on aggregate particle size and the permeability and air content of the surrounding paste.

Saturation Coefficient. The 5-hr boiling test and the resulting saturation coefficient (ASTM C67) have been found to provide a means of predicting the resistance of most types of brick to freezing and thawing. Thus it was hypothesized that this test might also be useful in predicting synthetic aggregate freeze-thaw resistance.

To apply this test to synthetic aggregates it was necessary to make some modifications of the test procedure. In order to make a comparison of the freeze-thaw loss and saturation coefficient for each aggregate, and in view of the effect of particle size on freeze-thaw loss, it was decided to use the same particle size in the same ratio for both tests. Hence, for the 5-hr boiling

test the particle size and the number of aggregates of each size were:

−⅝ in. to +½ in.	50 pieces
−½ in. to +⅝ in.	100 pieces
−⅜ in. to + No. 4	150 pieces

Three samples corresponding to the above grading were completely dried. The oven-dried samples were then subjected to 24-hr absorption tests in distilled water utilizing the Bryant method (13) (Tex-433-A).

Immediately after measuring the 24-hr absorption, the samples were transferred to 1000 mil. beakers and subjected to boiling in distilled water. Water was added to the beakers at frequent intervals in order to keep the samples submerged. After five hours the boiling was stopped and the samples were allowed to cool for not less than 16 or more than 18 hours. The aggregates were then transferred to the pycnometers to measure the absorption of water.

The saturation coefficient of each sample was calculated as follows:

$$\text{Saturation Coefficient } S_c = \frac{W_1}{W_2} (100)$$

where:

W_1 = wt of absorbed water at 24-hr absorption

W_2 = wt of absorbed water after 5-hr boiling

The saturation coefficients of 15 aggregates were determined. In order to make a comparative study of the boiling test results with the freeze-thaw losses and the 100-minute saturation values, these results are summarized in Table 3-2.

Relations Among Physical Durability Test Results. Figure 3-6 shows the general relationship that with increase in the saturation coefficient, the freeze-thaw loss increased. Some variations to this general relationship may be observed in the cases of aggregates having low values of freeze-thaw losses. For example, aggregate T1B had higher saturation coefficient than T1A but it sustained lower freeze-thaw loss than the latter. But this variation was small compared to the resistance to freeze-thaw losses of these aggregates.

TABLE 3-2. FREEZE-THAW LOSSES, 100-MINUTE SATURATION, AND SATURATION COEFFICIENT[a] OF AGGREGATES

Aggregate	100-Minute S_{100} Saturation (percent)	Freeze-Thaw Loss (percent)	Saturation, Coefficient, S_c (percent)
BCO-S2	28.8	48	61
BCO-S3	22.6	9	[b]
B1A	14.2	21	45
B1B	41.6	51	67
B1C	72.0	98	83
B1D	81.0	97	86
B1E	38.9	78	65
TCO	77.3	63	85
T1A	19.8	7	50
T1B	24.9	4	58
T1C	33.8	12	64
T1D	61.7	38	79
RGH-14A	75.9	88	84
RGH-18A	28.3	18	57
GEW-14A	68.1	52	80
GEW-18A	29.4	15	58
FAC	22.4	1	[b]

[a]ASTM C67.
[b]No test was performed.

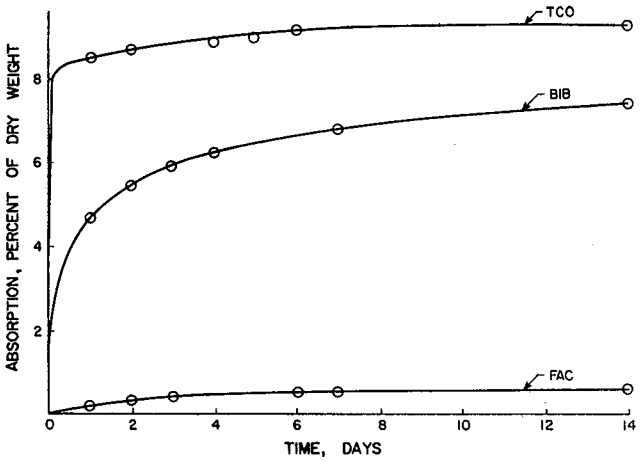

Figure 3-4. Typical absorption-time curves.

Figure 3-7 exhibits the relationship between saturation coefficient and 100-minute saturation. However, such a close correlation could not be established between freeze-thaw loss and the 100-minute saturation of the aggregates. Nevertheless, all of these data lead to the conclusion that the saturation coefficient might well be an effective means of predicting the freeze-thaw durability of synthetic aggregates. The advantage of the boiling test over the aggregate freeze-thaw test is that only about three days are required to perform it, whereas several weeks are generally required to perform the aggregate freeze-thaw test. At this stage of the study, no upper limit to the saturation coefficient is recommended for a performance criteria of synthetic aggregates. This can be done only after more study and comparative review of the saturation coefficient, freeze-thaw loss and 100-minute saturation values.

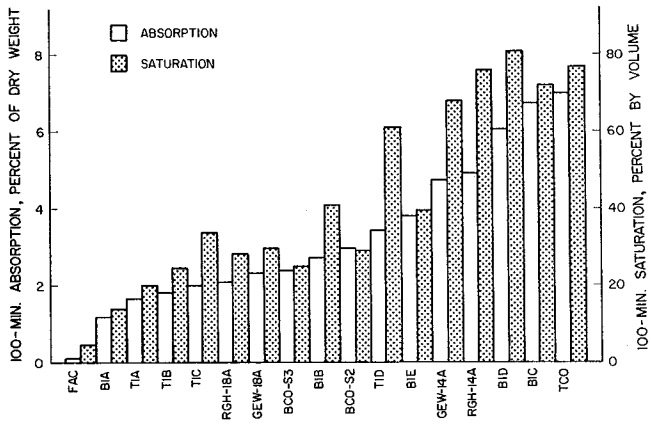

Figure 3-5. Comparison of absorption and saturation.

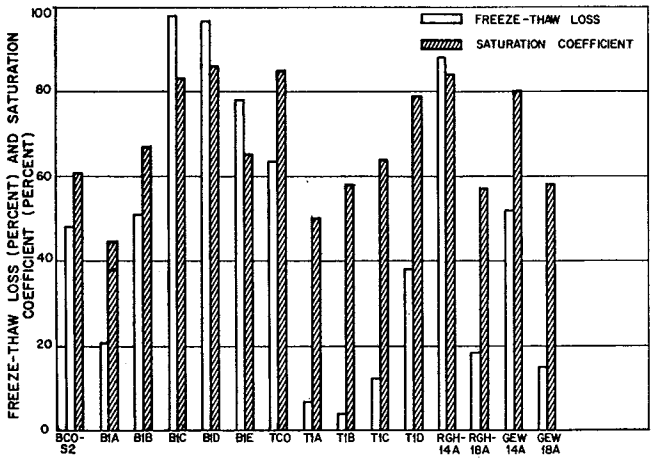

Figure 3-6. Relationship between freeze-thaw loss and saturation coefficient.

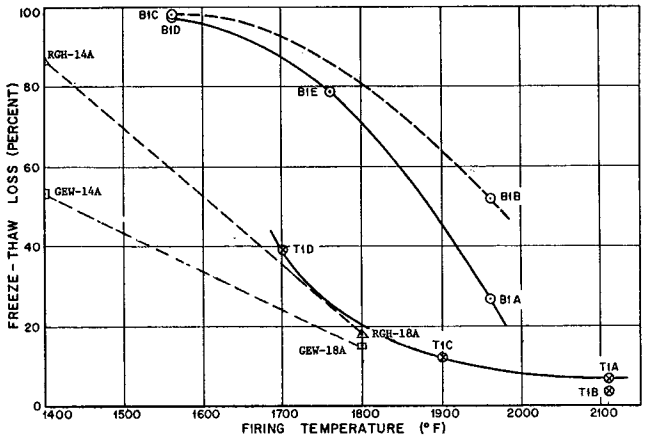

Figure 3-8. Relationship between firing temperature and total freeze-thaw loss.

Effect of Process Variables. Aggregate freeze-thaw resistance improved markedly with increased kiln temperature (Figure 3-8). However, it appears that while some clays are processed in the kiln so that the aggregate will reach an acceptable freeze-thaw loss at 1800°F, others will require temperatures over 2000°F.

Retention time appears to have very little effect on aggregate freeze-thaw loss.

Absorption and saturation both decreased significantly with increased kiln temperature. The effect of retention time was smaller but still significant: increased retention time tended to increase rate as well as the degree of saturation. Compare T1A with T1B, B1A and B1D with B1C in Figure 3-5. Each of these three comparisons are for short versus long retention times.

3.2.4 *Chemical Durability*

Pressure Slaking Test. With the exception of allophane, clays are crystalline in structure and one component of that structure is the OH^- ion. General agreement on what happens to the crystalline structure as the temperature is raised is lacking, but it is fairly well established that the hydroxyl "water" is eliminated between 400 and 700°C. This necessitates a rearrangement of the crystal lattice and there is apparently a period of transition between the initial destruction of the clay structure and the formation of different crystal structures at higher temperatures. The product is a function of the highest temperature to which the clay is raised and the rate of change of temperature. These changes are referred to collectively as "thermal transformation."

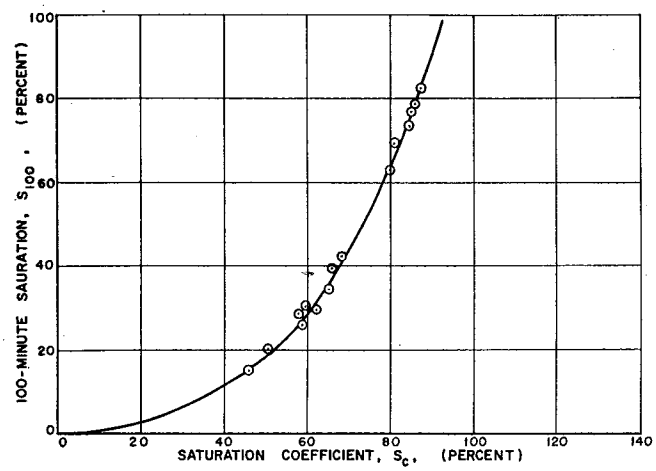

Figure 3-7. Relationship between saturation coefficient and 100-minute saturation.

A test has been proposed (20) as a method of determining the degree of transformation of the clays in the kiln (Tex-431-A). If a clay is not completely transformed, it may rehydrate, becoming soft and useless as aggregate. In this test the aggregate is "pressure cooked" in distilled water and then after a mechanical shaking, the amount of the material that passes a No. 40 sieve is measured. A proposed pressure slaking loss of 6 percent or less for concrete paving aggregates and 10 percent or less for base materials has been recommended (14). Test values are tabulated in Table 5-2.

5 Normal Sodium Hydroxide Test (24). This test is similar to the pressure slaking test except that the aggregate is "pressure cooked" in a 5*N* NaOH solution instead of distilled water. The minus 40 material is measured and reported as in the pressure slaking method (11). Additionally, reaction of the aggregate with the NaOH can be evaluated by determining reduction in alkalinity as in the potential reactivity test discussed below. The usefulness of the test awaits further study, but it shows promise at this time. Test results are shown in Table 5-3.

Potential Reactivity. This is a chemical test to determine the *potential* reactivity of the aggregate with the alkalies in portland cement and is described in ASTM C289. The products of the reaction imbibe water and can swell causing disruption of the concrete. This test can indicate that certain types of aggregate can be reactive but there is no certainty that a disruptive reaction will take place. A possible alternative is the Autoclave Expansion Test, ASTM C151, which is a concrete test and is discussed in Section 4. The service record of the aggregate, if it exists, should be relied on.

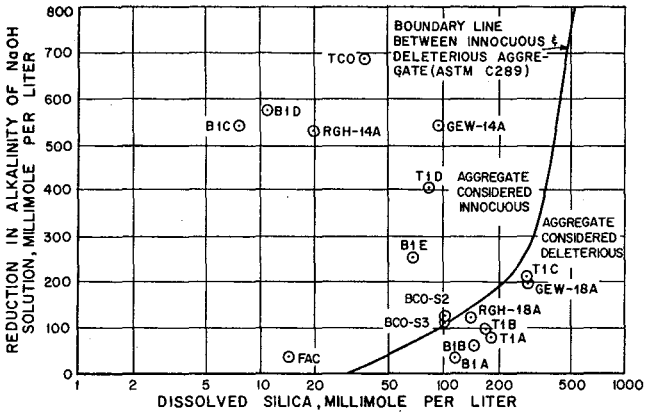

Figure 3-9. Results of potential reactivity test (ASTM C289).

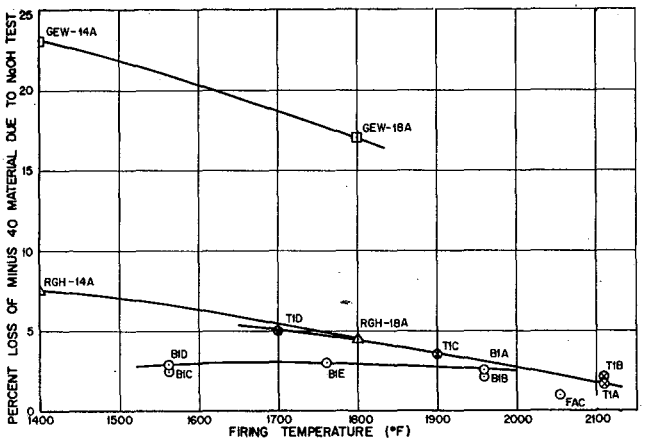

Figure 3-11. Relationship between firing temperature and loss of minus 40 material due to NaOH test.

The test determines the reactivity of a sample of aggregate with a one normal solution of NaOH under standard test conditions. Results are measured in terms of reduction in alkalinity and production of dissolved silica during the test. Generally, small reduction in alkalinity and high dissolved silica indicate potentially reactive aggregates. Figure 3-9 shows a standard plot of results of the ASTM C289 tests. The solid line is the line of separation between reactive and nonreactive aggregates, as proposed by several researchers (15-18), and finally recommended by ASTM C289. Numerical data are summarized in Table 5-2.

Effect of Process Variables. A small (barely significant) reduction in pressure slaking loss and NaOH test loss with firing temperature is indicated (Figure 3-10 and Figure 3-11). With the clay materials used in this study, since all pressure slaking losses were less than 10, it appears that practically complete and irreversible dehydroxylation occurs at 1400°F and above (see Reference 11 for a complete discussion of this test). Figure 3-12 also indicates a marked decrease in reactivity with increased kiln temperature in the 5N NaOH. This appears to relate to a similar trend noted in the potential reactivity test (ASTM C289). Retention time appears to have little effect on the slaking test or NaOH test results, at the kiln temperatures employed in this investigation. It is believed that the high slaking loss observed with the GEW aggregates is an indication of lower wet mechanical abrasion resistance rather than lower chemical durability.

In general, each sample showed an increase in dissolved silica content with firing temperature which might be attributed to the formation of an increased glassy phase in the higher temperature silicate structures, such as spinel, mullite, and crystobalite. The reaction of these silicates with alkalies in the cement are largely unexplored, but they may cause a deleterious effect in concrete (19).

The reduction in alkalinity in each aggregate was found to decrease with the firing temperature. Kiln retention time during the firing of aggregates exhibited insignificant effects both on dissolved silica and reduction in alkalinity. For example, aggregates B1C and B1D, with the same burning temperature but different retention times, fell very close to each other in Figure 3-4. Similar behavior was exhibited by B1A and B1B and by T1A and T1B. Aggregates B1C, B1D, TCO, RGH-14A, and GEW-14A exhibited very high reduction in alkalinity and low amount of dissolved silica. Such behavior of these aggregates might be attributed to

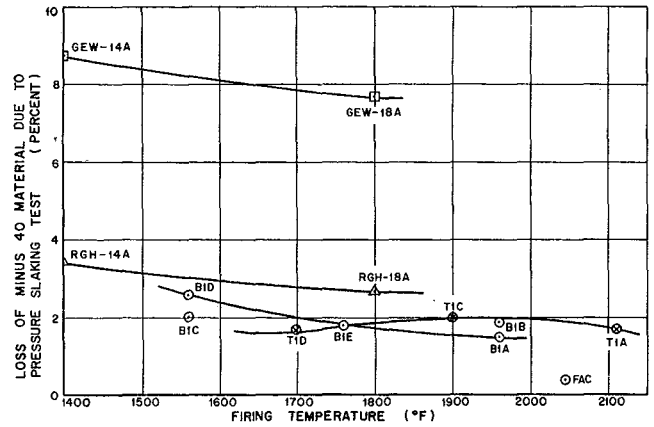

Figure 3-10. Relationship between firing temperature and loss of minus 40 material (pressure slaking test).

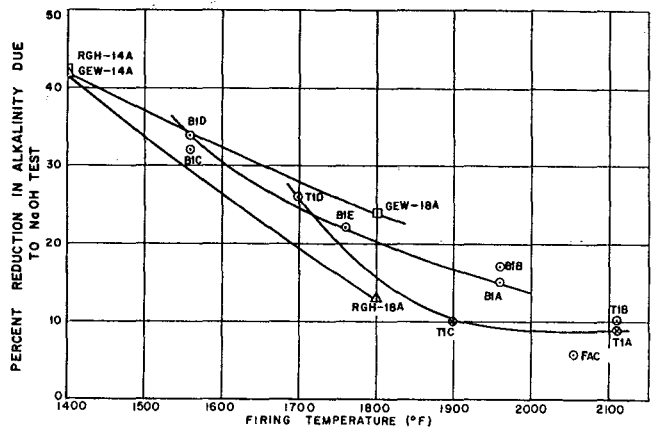

Figure 3-12. Relationship between firing temperature and reduction in alkalinity due to NaOH test.

their low firing temperature compared to other aggregates. The low firing temperature might have caused incomplete conversion of clay minerals into various silicates. Aggregates BCO-S2, BCO-S3, T1C, GEW-18A, and RGH-18A fell very close to the line of separation between the reactive and nonreactive aggregates.

In summary, increase in firing temperature tends (in Figure 3-9) to move an aggregate from the innocuous region toward the deleterious region as defined by ASTM C289 test procedure; retention time has little effect. However, aggregates fired with the lowest temperatures had abnormally large reductions in alkalinity which would also make them suspect. Clay charge type is a critical factor in producing an aggregate to pass this test. For example, one of the best aggregates fired at a very high temperature (2055°F) easily passed the C289 test, while others (in the T-group) were in trouble at any reasonable kiln temperature.

Finally, it should be emphasized that these results must be compared with results in concrete before any conclusion can be drawn.

3.2.5 Mechanical Durability

Los Angeles Abrasion Loss. This is a long used, standard test for abrasion (Tex-410-A). A sample of aggregate is subjected to a specified amount of impact and abrasion in a "ball mill" after which the amount of the sample that passes a No. 12 sieve is weighed and reported as a percentage of the original sample. Values are tabulated in Table 5-3.

Texas Sandblast Abrasion Test. This test has been proposed by Houston and Ledbetter (12) as a better means of differentiating between the quality of lightweight aggregates than the Los Angeles abrasion test. Serious questions have been raised as to the value of the latter test. For example, Rushing (22) states, "The Los Angeles abrasion test does not make any provision for the density of aggregates in that it requires a certain sample size, by weight, regardless of the weight-volume relationship of the material. Consequently, when the aggregate is lighter, the resulting volume charged into the drum is larger.

Furthermore, all aggregates, during the abrasion test, will break into smaller sizes. This breakage does not necessarily increase the percentage of material passing a No. 12 sieve."

In any case, the Los Angeles abrasion test was designed for natural, dense aggregate (30) and its adaptability to lightweight aggregate is open to question. Some modifications of the test have been proposed (22) to nullify the criticisms quoted above.

The Texas sandblast abrasion test definitely gives more pronounced differences in value for lightweight aggregates than does the Los Angeles abrasion test (12). The test consists of subjecting a selected number of certain size aggregate particles to a measured amount of sandblasting. The amount eroded away by this treatment is reported as a percent of the initial weight. Values are shown in Table 5-3.

Effect of Process Variables. While the Texas sandblast abrasion test is more sensitive than the Los Angeles abrasion test, even considering the greater range of data

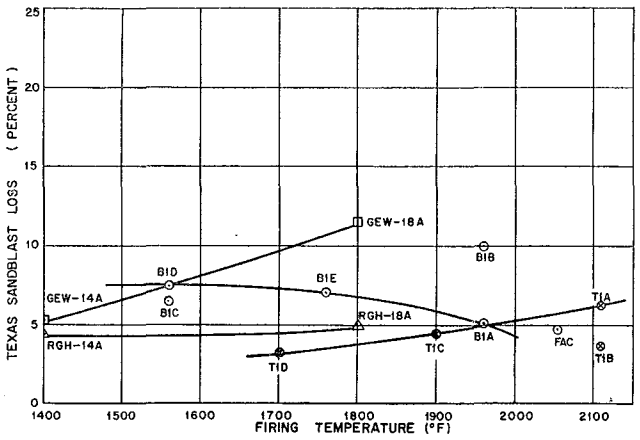

Figure 3-13. Relationship between firing temperature and Texas sandblast loss.

from the former test, only a slight trend of mechanical durability with kiln temperature is indicated. That is, higher temperatures tend to result in somewhat less durable aggregates, depending somewhat on the type of clay charged to the kiln (Figure 3-13). A consistent trend with retention time is not evident. However, the Texas sandblast abrasion test loss does appear to be related to porosity as indicated by the curve in Figure 3-14. Thus, any changes in process variables which tend to reduce porosity should improve the mechanical durability of a synthetic aggregate.

3.2.6 Interpretation of Aggregate Test Data

The aggregate test data vary over a wide range. Even if the aggregates made from one type of clay are considered, the test data vary considerably. This seems to indicate that processing parameters are at least as important as the raw material in determining the resultant product.

Based on the aggregate evaluations *alone*, if any one of the aggregates can be said to be the best it is the FAC aggregate. Test results for this one are all very

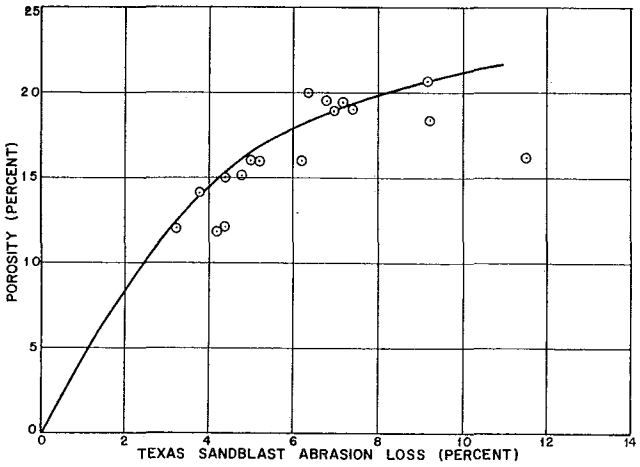

Figure 3-14. Relationship between Texas sandblast abrasion loss and porosity.

good. The TCO aggregate is probably the worst. Note the comparison.

	FAC	TCO
100-min. absorption, percent	0.14	7.00
Dry bulk specific gravity	2.20	2.08
Absolute specific gravity	2.43	2.63
Porosity	0.09	0.21
Unit weight, pcf	61	68
Potential reactivity:		
Reduction in alkalinity, mmol/L	36	684
Dissolved silica, mmol/L	14	36
Pressure slaking loss, percent	0.4	4.5
5N NaOH Test:		
Reduction in alkalinity, mmol/L	5.8	44
—40 loss, percent	0.9	5.1
Aggregate freeze-thaw loss, percent	1.2	63
Texas sandblast abrasion, percent	4.2	7.2
Los Angeles abrasion, percent	28	41

However, before final judgments were made, concrete evaluations were performed. They are reported in the following section and discussed in Chapter 4.

3.3 The Concretes

3.3.1 General

To answer the question of whether strong, durable, chemically inert concrete can be made with synthetic nonbloated aggregates, 35 batches were made from the 17 aggregates. After suitable periods of curing the specimens molded from the concrete were subjected to tests intended to determine the degree which the concrete met certain specifications.

Three standard strengths tests were used: compression (ASTM C39), flexure (ASTM C78), and splitting tensile (ASTM C496).

The physical durability determination was based on alternate freezing and thawing the concrete in water (ASTM C290). The fundamental transverse frequency and loss of weight were measured periodically.

The chemical durability determination was based on the expansion of concrete prisms after enclosure in an autoclave at elevated temperature in a stream atmosphere (ASTM C151, modified). The expansion was taken as an indicator of chemical reactivity.

3.3.2 Mixing Procedures

Prior to batching, all aggregates were presoaked until the rate of absorption had diminished to a point where very little additional absorption would be expected during the time from mixing to setting. The aggregate was drained 15 minutes prior to mixing. The total moisture content of the aggregate was determined by the method of ASTM C566. The moisture absorbed, from the absorption-time curve, was subtracted from the total moisture and the difference was assumed to be "free" water, available for mixing.

An air-entraining agent was mixed with about ⅔ of the mix water before introducing it into the mixer. The nonbloated aggregate and this water were put into the mixer first and mixed until the contents had a foamy appearance. The cement and sand were added and mixing began. While the mix was still fairly stiff, the slump was checked and, if inadequate, some water added with further mixing. The process was repeated until the desired slump was obtained.

Mix proportions and grading of aggregates are shown in Table 5-4, 5-5, 5-6, 5-7, 5-8, and 5-9.

Because of the great difference in the grading of the GEW and RGH aggregates and that of the others, these were separated into various size fractions and recombined into the following proportions:

Sieve Size	Percent
−1 + ¾ in.	5
−¾ + ⅜ in.	38
−⅜ + No. 4	47
−No. 4 + No. 10	10

The concretes were all made with the same brand of Type I cement, natural sand, the same source of water, and the same air-entraining agent. Although the quantities of these necessarily varied, the only material type varied in each batch was the coarse aggregate. All batches were mixed in 3 cu ft capacity mechanical mixer. Thirty-three batches were 2.5 cu ft in volume, and the other 2 were 1 cu ft. Slump, unit weight, and percent of entrained air was determined for each batch. For the fresh concrete:

	Range	Average
Unit weight (ASTM C138)	123.6-134.4 pcf	128.4 pcf
Slump (ASTM C143)	1-6 in.	2¼ in.
Entrained air (ASTM C173)	3-6.5 percent	4.5 percent

The test specimens made from the 2.5 cu ft batches were:

No.	Type	Size, in.
1 to 3	Autoclave Prisms	2 × 2 × 11¼
2	Compression Cylinders	6 × 12
2	Splitting Tensile Cylinders	6 × 12
1	Beam	6 × 6 × 36
6	Freeze-Thaw Prisms	3 × 3 × 16

3.3.3 Compression Tests

At least two cylinders from each batch were tested for compressive strength after 7 days of moist curing in accordance with ASTM C39. Compressive strengths ranged from 1420 psi to 4460 psi. Test results are shown in Table 5-10.

3.3.4 Splitting Tensile Strength Test

Two 6 × 12 in. cylinders from each 2.5 cu ft batch were tested for tensile strength after 7 days of moist curing in accordance with ASTM C496 (9). The splitting tensile strength varied from 205 psi to 485 psi. Test results are shown in Table 5-10.

3.3.5 Flexural Strength Test

One 6 × 6 × 36 in. beam was tested for flexural strength after 7 days of moist curing in accordance with ASTM C78. This method calls for third-point loading. The beam was broken twice using an 18 in. span each time. The flexural strength, also called modulus of rupture, ranged from 255 psi to 695 psi. Test results are shown in Table 5-10.

3.3.6 Freeze-Thaw Durability

Six 3 × 3 × 16 in. specimens were prepared from each 2.5 cu ft batch. These were subjected to a freezing and thawing test as described in ASTM C290. The fundamental transverse frequency was measured initially and at regular periodic intervals according to ASTM C215. The numerical data are shown in Table 5-11.

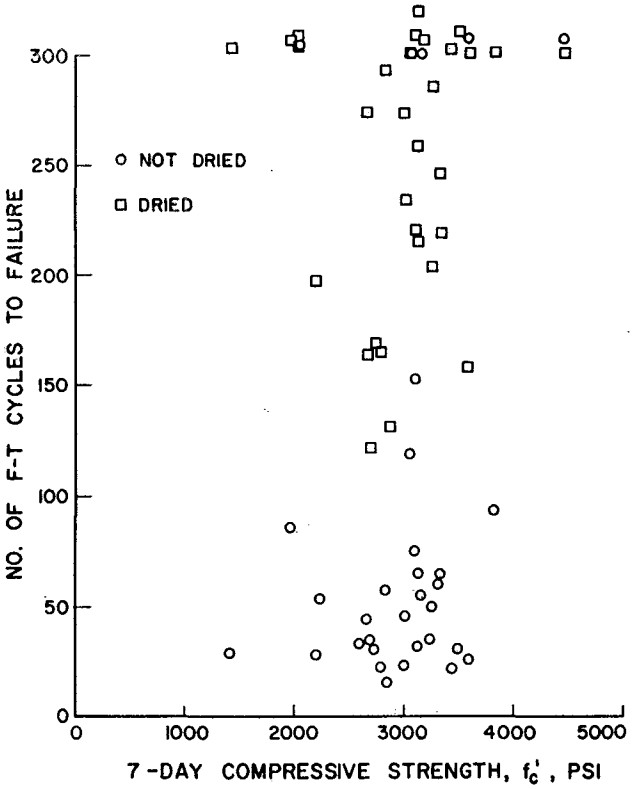

Figure 3-15. Physical durability of dried and nondried concretes.

Because of the known destructive effect due to concrete freeze-thaw (23) of high saturation of the aggregates, all aggregates were soaked to 50 percent, or above, with three exceptions. The object was to create the severest possible conditions for the test. It was also desired to ascertain the effect of drying the specimen before subjecting it to the freeze-thaw environment, therefore three of the specimens went to the freezers after 14 days of moist curing and the other three were stored 14 additional days in an environment of 70° ± 5°F and 50 ± 10 percent relative humidity before going to the freezer. Even a casual examination of the data for the freeze-thaw tests reveals that without a single exception the drying greatly enhanced the physical durability of the concrete. Figure 3-15 is a plot of the average number of cycles to failure, or to 300 cycles if failure does not occur, compared with compressive strength. The probability that any correlation exists between these two quantities seems remote from the scatter of the points. However, the plot gives a good graphical picture of how the freeze-thaw resistance is improved by drying the concrete prior to testing.

3.3.7 Autoclave Expansion

This test is essentially the same as ASTM C151 except that concrete is used instead of neat cement paste to make each test specimen. This test was proposed as a method of predicting the potential of concrete for alkali-aggregate reactions and aggregate rehydration (24). One suggested limit of expansion for acceptability was an expansive strain of 1500×10^{-6} in./in. Table 5-10 shows that all the concretes tested meet this criterion except the ones made with TCO aggregates. The two batches made with this aggregate had expansions of 4900 and 4800 microinches, more than three times the next highest figure. The specimens had visible cracks when removed from the autoclave and some appearance of lateral swelling. It has also been suggested that visible cracking be considered a criterion of failure. The test was not run on every batch since once with each aggregate was considered sufficient. A few replications were made to confirm the results of the first test.

CHAPTER 4

Discussion of Results

4.1 Comparison Between Types of Results

Figure 4-1 shows a plot of compressive strength versus water-cement ratio. Although scatter is great, the trend is the usual one of decreasing strength with increasing water-cement ratio.

A comparison of compressive and splitting tensile strength is shown in Figure 4-2 which indicates a linear relation between these two quantities, even though there is considerable scatter of the data. The straight line shown was fit by application of the Texas Transportation Institute multiple error regression program (31) which gives a correlation coefficient for these data of 0.84. This means that 70.7 percent of the variation in the splitting tensile strength is explained by differences in the compressive strength.

Figure 4-3 shows a similar comparison of the flexural strength and compressive strength. The scatter is

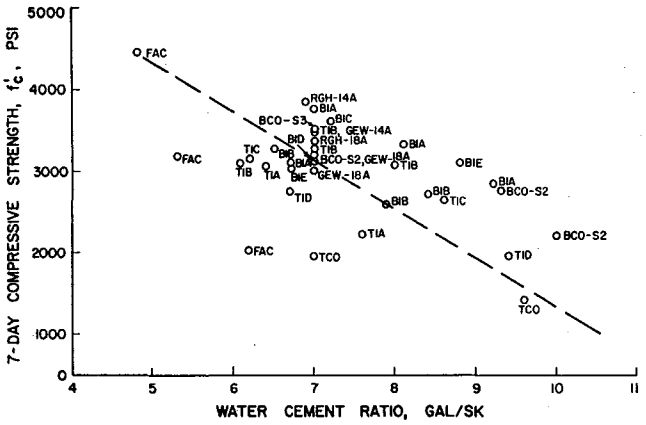

Figure 4-1. Effect of water cement ratio on compressive strength.

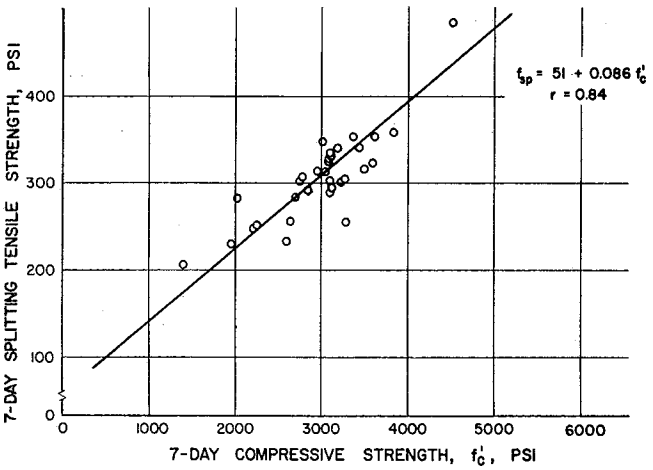

Figure 4-2. Comparison of splitting tensile and compressive strength.

more evident here and this observation is confirmed by the lower coefficient of correlation of 0.78. These data indicate that the splitting tensile strength method of determining tensile strength is preferable to the flexure method since it correlates better with compressive strength.

4.2 Strength as an Indicator of Performance

Figure 4-4 shows a plot of the compressive strength and the relative dynamic modulus of elasticity, P_c,*

*The relative dynamic modulus of elasticity is defined as:

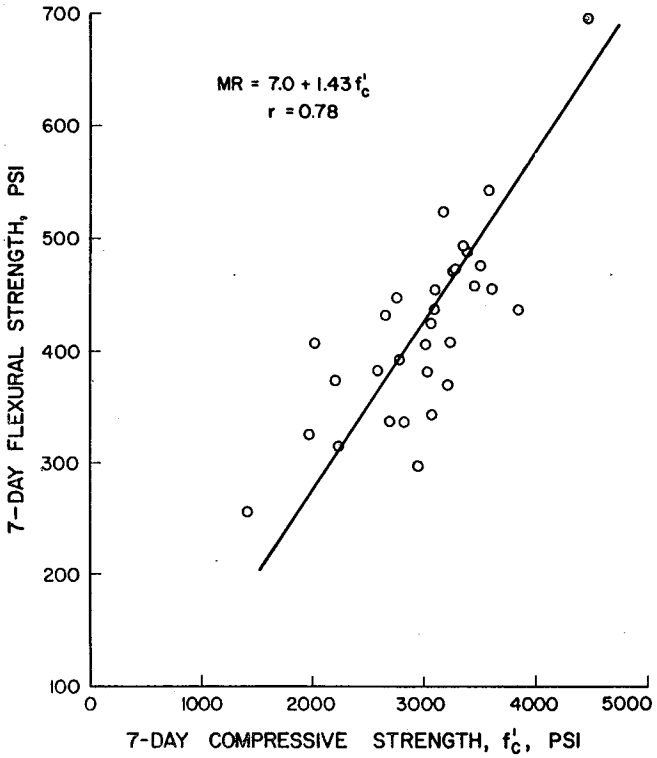

Figure 4-3. Comparison of flexural and compressive strength.

versus the cement content for concrete made with FAC aggregate. Table 5-4 shows the water cement ratio decreases as the cement content increases. Both of these changes generally result in improved strength as the graph shows. Each of these three concretes lasted the full 300 cycles of freezing and thawing, but a comparison of values of P_c clearly indicates improvement with increasing cement. *For this aggregate,* freeze-thaw resistance improves with strength. Figure 3-15 shows that this is not generally true. The freeze-thaw resistance of concrete made with natural dense aggregates generally improves with increasing strength because it is the cement paste matrix that fails (25). The FAC aggregate behaves very much like natural aggregate as Figure 4-4 shows. Figure 3-15 suggests that in the other concretes it is the aggregate that is failing since freeze-thaw durability does not improve with strength.

$$P_c = \frac{n_1^2}{n^2} \times 100$$

where P_c = relative dynamic modulus of elasticity, percent, after c cycles of freezing and thawing.

n = fundamental transverse frequency at 0 cycles of freezing and thawing.

n_1 = fundamental transverse frequency after c cycles of freezing and thawing.

A specimen is considered to have failed if $P_c \leq 60$ when $c < 300$.

P_c is an indication of the relative deterioration of a specimen and should not be taken as an indication of compressive or flexural strength.

Autoclave expansion is plotted against compressive strength in Figure 4-5. No relation between these two quantities appears. The two points labeled 4800 and 4900 should not be interpreted to mean that concrete with a compressive strength less than 2000 psi will have excessive expansion. Those two points represent the TCO aggregate and its anomalous behavior suggests it be ignored in seeking general relationships.

4.3 Aggregate-Concrete Strength Relations

There are several variables that affect the strength of concrete such as water-cement ratio, cement content, length and method of curing, degree of consolidation, and quality and quantity of ingredients. The type of coarse aggregate is also among the variables that affect the strength.

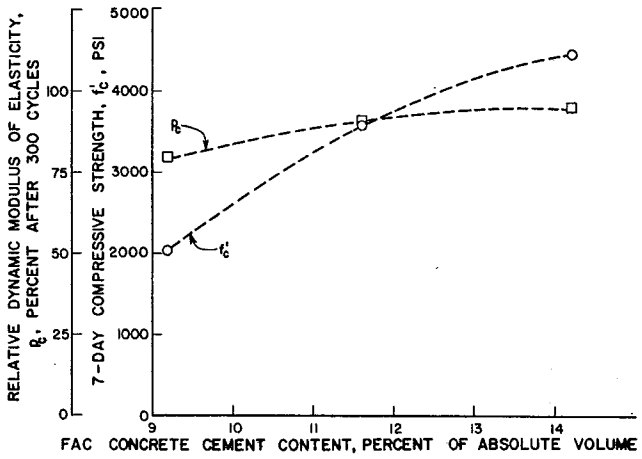

Figure 4-4. Effect of cement content on P_c and f'_c.

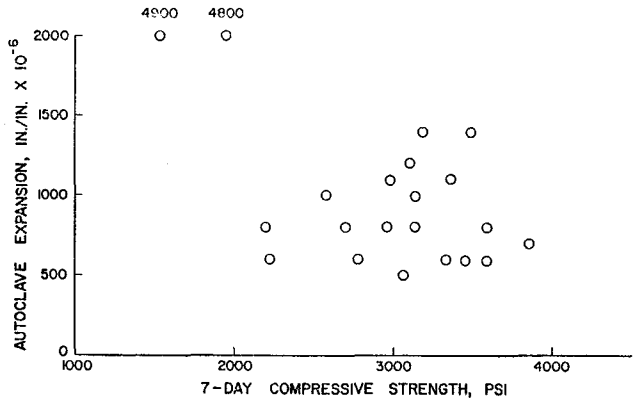

Figure 4-5. Comparison of expansion and strength.

If the compressive strength of the concretes with nearly equal cement contents are compared with 100-minute absorptions, porosity, unit weight, pressure slaking loss and Los Angeles abrasion loss of the aggregates, no correlation with any one of these properties is apparent. Still, the highest compressive strength concrete was made with FAC aggregate whose values for all aggregate tests are considered good. (See Section 3.3.) The lowest strength concrete was made with TCO aggregate which failed some of the aggregate tests and had values undesirably close to the limit of acceptability in others. This seems to support the existence of a relation between strength and the collective aggregate properties. Some future research might well be directed toward this area.

4.4 Alkali-Aggregate Reaction

The autoclave expansion of the concretes is shown in Figure 4-6. In the case of the T1A and T1B aggregates, there is a decrease in expansion with an increase in retention time, for the same clay burned at the same temperature. For the B1A and B1B, the reverse is true. A decrease in expansion is seen for the GEW aggregates with an increase in temperature and the reverse is seen with the RGH aggregates. If any correlation exists, it must be a complex one between autoclave expansion, kind of clay, temperature, and retention time.

A comparison of expansion with values of dissolved silica and reduction in alkalinity fails to show any correlation. If SiO_2 and NaOH were the only reagents, then there should be an inverse proportion relationship between these two. The results can be influenced by the presence of carbonates, magnesium, and ferrous iron in the aggregates (17). The B clay contained small white particles which may be calcium carbonate, and iron was present in all of the clays which no doubt left a residual amount in the aggregate. This may account for the lack of agreement between the potential reactivity test and the autoclave expansion test for the TCO aggregate. Aggregate TCO is an interesting anomaly; indicated as innocuous by the potential reactivity test, it produced the greatest expansion in the modified autoclave expansion test (24). There are other instances of this phenomenon although they seem to be few in number. It indicates that the test is less than 100 percent reliable, that some reaction is taking place whose potential is not detected by the test. Conversely, the test showed T1A, T1B, T1C, BCO, and B1B to be potentially deleterious but none of these expanded excessively in the autoclave expansion test.

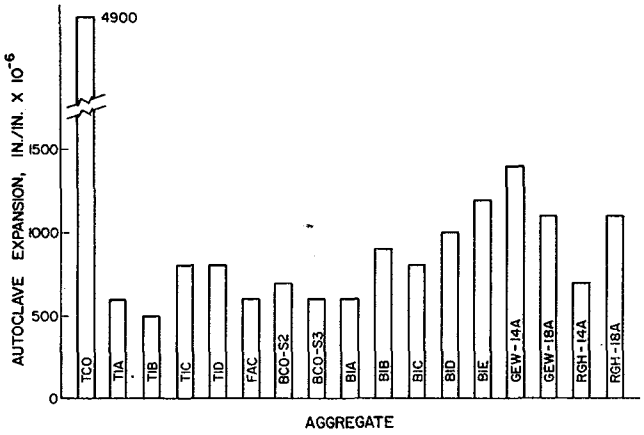

Figure 4-6. Autoclave expansion for concrete made with nonbloated aggregate.

ASTM C289 was designed to test natural aggregates and its applicability to synthetic aggregates is moot. On the basis of data acquired in this study, the conclusion is that the potential reactivity test is of little use in predicting alkali-aggregate reactions in synthetic nonbloated aggregate concrete.

4.5 Aggregate Freeze-Thaw Durability Relations

If the number of concrete freeze-thaw cycles to failure of the *nondried* specimens are plotted against the aggregate freeze-thaw loss, although there is considerable scatter, some semblance of a pattern appears (Figure 4-7). The five concretes that lasted through 300 cycles

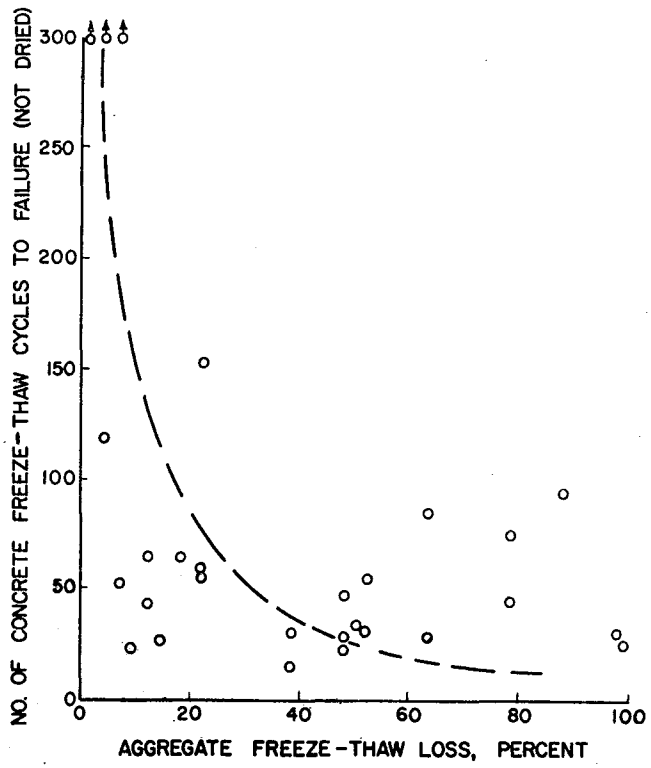

Figure 4-7. Comparison of concrete and aggregate freeze thaw durability (not dried).

are all made with aggregates of low freeze-thaw loss. The lowest number of concrete freeze-thaw cycles to failure does not occur with the highest aggregate freeze-thaw loss, but it is possible to visualize a curve (equilateral hyperbola?) as shown.

If the number of concrete freeze-thaw cycles to failure of the *dried* specimens is plotted in the same manner as in Figure 4-8, a failure envelope can be drawn. The failure envelope separates the graph into two areas. An aggregate with a given aggregate freeze-thaw loss can be expected to produce a concrete that will endure a number of concrete freeze-thaw cycles at least equal to the ordinate to the failure envelope. Additionally, the envelope indicates that aggregates with a freeze-thaw loss of less than around 10 percent can be relied on to produce concrete that would endure 300 cycles without failing if the concrete is allowed to dry before freezing. Based on the supposition that 300-cycle endurance is sufficient, this would indicate such aggregates are safe for use in concrete. It should be remembered, however, that there are other factors, such as cement content, which affect freeze-thaw durability.

Note that physically durable concrete can be made from some of these aggregates even though their strength may be less than desired.

4.6 Comparison Between Concrete Properties and Texas Highway Department Requirements

4.6.1 General

A comparison of the engineering properties of the materials with the existing THD specifications is a natural first approach; however, there are no contemporaneous THD specifications for nonbloated synthetic aggregate concrete and it should be kept in mind that serious obstacles can be encountered in trying to make one material fit specifications written for another.

4.6.2 Concrete Pavement

The THD specifications for Concrete Pavement (Item 360) calls for:

1. A particular grading.
2. 7-day flexural strength of 650 psi (center point loading).
3. Minimum cement factor of 5 sks per cu yd.
4. Maximum water-cement ratio of 6.5 gal per sk.
5. Slump of 1 to 3 in.
6. Los Angeles abrasion loss $\leq$ 45 percent.

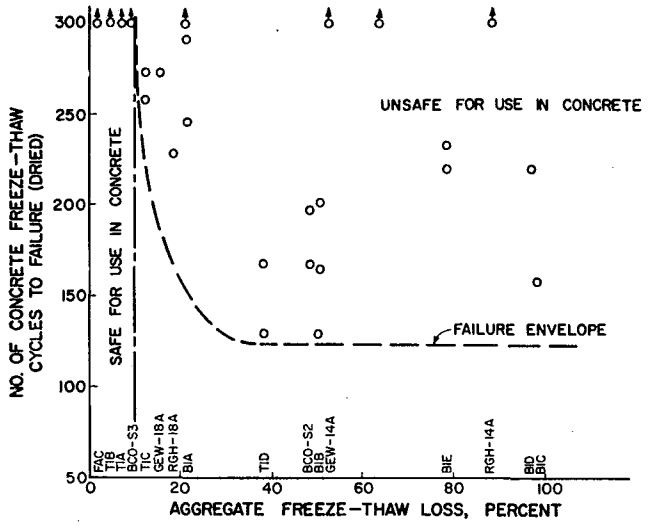

Figure 4-8. Comparison of concrete and aggregate freeze-thaw durability (dried).

Although the aggregates of this study do not fit the grading requirements, this is not a serious shortcoming. Grading of nonbloated aggregate is easily adjusted by sizing of the raw material.

The strength requirement is based on the flexural test (ASTM C293) with center point loading. All of the flexure tests of this investigation were performed using third point loading (ASTM C78) and this gives approximately 20 percent lower values, on the average, than the center point loading (25). Taking this difference in method of testing into account, only two batches of concrete were found to attain the necessary strength; these were batches 2 and 3 made with the FAC aggregate. Since the most economical mix uses the minimum amount of cement, the initial batches were designed for a nominal cement factor of 5 sks per cu yd which is equivalent to 8.85 percent of the absolute volume of the concrete. When it became apparent that this cement factor produced relatively low levels of strength, it was increased to 5½ sks which is equal to 9.74 percent of the absolute volume. This change improved the strength somewhat but not enough to meet the 650 psi flexural strength requirement. The factor for the two batches that did meet requirements were 6.6 and 8.0 sks per cu yd.

The amount of cement is of course not the only factor governing the strength of concrete. The water-cement ratio is also very important but with such absorbent aggregates as these it is next to impossible to control, especially in the field. As a consequence, mix designs using synthetic lightweight aggregate are usually based on the cement factor and a given slump (26). In practice water is added to the mix until the desired slump is attained and this sometimes leads to undesirable water-cement ratios. The THD specification states that the water-cement ratio shall not exceed 6.5 gal per sk and slump shall be within the range of 1 to 3 in. All but 6 batches had a slump within the specified range (Table 5-4) but as a consequence of using the mixing technique for synthetic lightweight aggregates, that is, adding water until the desired slump is obtained, the water-cement ratios were excessive. It emphasizes the difficulty of control to note that in the case of one batch —FAC, batch No. 2—the slump was excessive while the water-cement ratio was well below the limit.

None of the aggregates had a Los Angeles abrasion loss over 45 percent.

The writers conclude that these concretes cannot be made to fit the standard THD specifications for concrete pavement. This is not to say that good concrete pavement cannot be made with these aggregates for some of them show very promising prospects, particularly the FAC aggregate. The reader is reminded that in this section the properties of one material have been compared with specifications written for another and it

should not be too surprising to find that they are not completely compatible.

4.6.3 Concrete Bases

Item 290 of the THD specifications is entitled "Shell Concrete Base." This type of base is used where oyster shell can be economically obtained, that is, in coastal areas. If the nonbloated aggregates of this study could be used in concrete base, then those inland areas where use of shell is not available could benefit. A comparison with the existing specifications will be made.

The requirements are:

1. A particular grading.
2. 7-day flexural strength of 400 psi.
3. Minimum cement factor of 4½ sks per cu yd.
4. Maximum water-cement ratio of 10 gal per sk.
5. Slump of 1 to 3 in.

Again, none of the aggregates met the grading requirements for shell because of an excess of smaller sizes; however, this is not believed to be significant. No abrasion loss is specified for shell so there is no comparison to be made; however, there should probably be an abrasion loss specified for nonbloated synthetic aggregate if it were adapted for use in concrete base. Since all aggregates met the abrasion loss requirement for concrete pavement, they would meet one for concrete base.

The flexural strength requirement is 400 psi in 7 days. If the 20 percent factor previously mentioned for the difference in loading is permitted, then all but 5 of the concretes can meet this specification. Two of these are the anomalous TCO concretes and should not be used even if the strength were satisfactory. The other three have cement factors of less than 5 sks per cu yd and can probably be improved to an acceptable level by a small additional amount of cement. These aggregates should be suitable for a concrete base, since concretes of the specified strength were made from 16 of the 17 aggregates studied. All water-cement ratios are equal to or less than the maximum allowable of 10 gal per sk. The slump limits are the same as for concrete pavement and so the same 6 batches would be eliminated on this basis as in the preceding section. However, the slump of those particular concretes could easily be adjusted to put them in the acceptable range with the added benefit of some improvement in strength due to a lowered water-cement ratio.

Here again it must be said that these concretes do not rigorously meet all the specifications for concrete base but the prospects for making acceptable concrete base are very good.

CHAPTER 5

Appendix

5.1 Tables

This appendix contains tabulations of all data analyzed in this study.

TABLE 5-1. AGGREGATE TEST DATA

Aggregate	100-min. Absorption percent[a]	Dry Bulk Sp. Gr.[a]	Absolute Sp. Gr.[b]	Porosity
TCO	7.00	2.08	2.63	0.21
T1A	1.70	1.88	2.25	0.16
T1B	1.83	1.92	2.26	0.15
T1C	1.98	2.07	2.37	0.13
T1D	3.47	2.14	2.47	0.13
FAC	0.14	2.20	2.43	0.09
BCO-S2	2.93	1.92	2.40	0.20
BCO-S3	2.35	2.00	2.46	0.19
B1A	1.21	1.88	2.24	0.16
B1B	2.70	1.91	2.34	0.18
B1C	6.70	2.09	2.69	0.22
B1D	6.06	2.16	2.71	0.20
B1E	3.81	1.92	2.39	0.20
GEW-14A	4.77	2.07	2.47	0.16
GEW-18A	2.30	2.17	2.58	0.16
RGH-14A	4.95	2.01	2.37	0.15
RGH-18A	2.14	2.22	2.68	0.17

[a]Tex-433-A.
[b]By Rainhart Pressure Pycnometer (27).

TABLE 5-2. AGGREGATE TEST DATA

Aggregate	Unit Wt.,[a] pcf		Potential Reactivity[b]		Pressure Slaking[c] Loss, percent —40 Material
	I	II	Reduction in Alkalinity mmol/L	Dissolved Silica mmol/L	
TCO	68	63	684	36	3.7
T1A	62	60	79	172	1.7
T1B	64	60	99	171	1.4
T1C	68	65	209	282	2.0
T1D	70	65	405	83	1.7
FAC	61	65	36	14	0.4
BCO-S2	61	55	109	105	1.9
BCO-S3	60	56	124	102	1.2
B1A	59	56	38	117	1.5
B1B	63	57	61	149	1.9
B1C	68	62	560	7.4	2.0
B1D	68	64	574	10.5	2.6
B1E	61	60	249	57	1.8
GEW-14A	67	60	541	91	8.7
GEW-18A	63	56	201	281	7.7
RGH-14A	68	61	528	19.2	3.4
RGH-18A	64	59	102	130	2.7

[a]Tex-404-A.
[b]ASTM C289.
[c]Tex-431-A.

TABLE 5-3. AGGREGATE TEST DATA

Aggregate	5N NaOH Test[a]		Aggregate F-T Loss 50 Cycles[b] Percent	Texas Sandblast Loss[c] Percent	Los Angeles Abrasion Loss[d] Percent
	Percent Reduction in Alkalinity mmol/L	Loss of −40 Material Percent			
TCO	44	5.1	63	7.2	41
T1A	9.1	1.7	7.3	6.2	38
T1B	10	2.1	3.8	3.8	38
T1C	10	3.6	12	4.4	44
T1D	26	5.1	38	3.2	38
FAC	5.8	0.9	1.2	4.2	28
BCO-S2	29	1.8	48	6.8	36
BCO-S3	17	14	8.7	9.2	42
B1A	15	2.6	21	5.0	36
B1B	17	2.2	51	9.2	37
B1C	32	2.7	98	6.4	37
B1D	34	2.8	97	7.4	41
B1E	22	2.9	78	7.0	41
GEW-14A	42	23	52	5.2	45
GEW-18A	24	17	15	11.5	42
RGH-14A	42	7.4	88	4.4	34
RGH-18A	13	4.0	18	4.8	38

[a]See Reference No. 11.
[b]Tex-432-A.
[c]See Reference No. 12.
[d]Tex-410-A.

TABLE 5-4. CONCRETE MIX DATA

Aggregate	Batch	Cement Factor[a]	Water-Cement Ratio[b]	Slump[c]	Percent Air[d]	Unit Wt. pcf
TCO	1	4.5	9.6	3¼	3.5	133.4
	2	5.7	7.0	1½	5.0	131.6
TIA	1	4.9	7.6	1¼	4.3	123.6
	2	5.8	6.4	2	5.0	125.6
T1B	1	4.8	8.0	2	5.0	125.6
	2	5.4	7.0	6		125.6
	2A	5.0	7.0	2		123.6
	3	5.8	6.1	1	5.5	128.0
T1C	1	4.7	8.6	2¼	5.0	129.6
	2	5.8	6.2	1¾	5.5	129.6
T1D	1	4.7	9.4	2½	4.5	133.8
	2	5.6	6.7	3¼	6.5	130.0
FAC	1	5.2	6.2	1¼	6.5	127.6
	2	6.6	5.3	4	5.0	133.2
	3	8.0	4.8	3	3.5	134.4
BCO-S2	1	5.1	9.3	3¼	3.5	125.6
	2	4.5	10.0	1½	4.0	123.6
	3	5.7	7.0	1½	5.0	127.6
BCO-S3	1	6.0	7.0	2¾	4.8	129.6
B1A	1	5.2	8.1	3¼	3.5	123.6
	2	4.7	9.2	1¼	4.0	123.6
	3	5.8	7.0	3		
	4	5.8	6.7	2	4.5	126.4
B1B	1	5.2	7.9	5	5.0	125.6
	2	4.8	8.4	1¼	3.5	123.6
	3	5.8	6.5	2¼	6.0	126.4
B1C	1	5.8	7.2	2½	3.6	133.6
B1D	1	5.9	7.2	1	4.0	132.8
B1E	1	4.7	8.8	2	4.5	125.6
	2	5.7	6.7	1¼	5.8	127.6
GEW-14A	1	6.4	7.0	1¼	4.0	130.4
	2	6.2	7.2	1	4.0	129.6
GEW-18A	1	6.2	7.0	1¼	3.0	131.6
RGH-14A	1	6.0	6.9	2	3.0	128.8
RGH-18A	1	6.4	7.0	1¾	3.5	133.6

[a]sks per cu yd.
[b]gal per sk.
[c]ASTM C143.
[d]ASTM C173.

TABLE 5-5. CONCRETE MIX PROPORTIONS IN PERCENT OF ABSOLUTE VOLUME

Aggregate	Batch	Cement	Water	Fine Aggre.	Coarse Aggre.	Air
TCO	1	7.9	21.2	32.0	34.8	3.5
	2	10.1	19.7	32.6	32.6	5.0
T1A	1	8.7	18.4	32.7	36.2	4.3
	2	10.3	18.3	33.2	33.2	5.0
T1B	1	8.5	19.0	32.0	35.5	5.0
	2	9.6	18.8	35.6	29.9	6.0
	2A	8.9	17.4	40.4	27.7	5.6
	3	10.3	17.4	33.4	33.4	5.5
T1C	1	8.4	20.2	31.5	34.9	5.0
	2	10.3	17.7	33.3	33.3	5.5
T1D	1	8.3	21.8	31.1	34.5	4.5
	2	10.0	18.8	32.3	32.3	6.5
FAC	1	9.2	16.0	34.2	34.2	6.5
	2	11.6	17.3	33.0	33.0	5.0
	3	14.2	18.9	31.7	31.7	3.5
BCO-S2	1	9.1	23.5	29.4	35.6	3.5
	2	7.9	22.4	30.5	35.4	4.0
	3	10.1	19.7	32.6	32.6	5.0
BCO-S3	1	10.7	21.0	31.8	31.8	4.8
B1A	1	9.2	20.9	29.5	36.3	3.6
	2	8.3	21.4	32.0	34.5	4.0
	3	10.3	20.1	34.4	28.8	6.5
	4	10.2	19.2	33.0	33.0	4.5
B1B	1	9.2	20.2	32.8	32.8	5.0
	2	8.5	19.9	32.9	35.4	3.5
	3	10.2	18.1	32.9	32.9	6.0
B1C	1	10.3	20.8	35.5	29.9	3.6
B1D	1	10.4	21.0	35.1	29.5	4.0
B1E	1	8.4	20.6	31.5	35.0	4.5
	2	10.1	18.9	32.6	32.6	5.8
GEW-14A	1	11.4	22.2	34.0	28.5	4.0
	2	11.0	22.2	31.4	31.4	4.0
GEW-18A	1	11.0	21.4	35.1	29.5	3.0
RGH-14A	1	10.6	20.5	35.8	30.1	3.0
RGH-18A	1	11.3	22.0	34.3	28.8	3.5

TABLE 5-6. SIEVE ANALYSES FOR T AGGREGATES[a] PERCENT RETAINED

Sieve Size	TCO	TIA	TIB	TIC	TID
1 in.	1.2	0.0	0.20	0.20	0.0
¾ in.	4.1	4.6	3.0	7.4	4.8
½ in.	21.5	24.9	18.8	29.4	18.9
⅜ in.	20.3	12.7	17.1	16.5	15.9
No. 4	41.3	36.4	38.2	28.2	41.0
No. 10	10.0	12.3	12.8	13.1	13.3
No. 20	1.2	1.9	5.1	3.3	3.4
No. 40	0.1	0.6	2.8	1.0	1.6
No. 80	0.1	0.3	1.3	0.5	0.7
No. 200	0.1	0.2	0.2	0.1	0.1
—No. 200[b]	0.1	0.1	0.5	0.3	0.3

[a]ASTM C136.
[b]Percent passing.

TABLE 5-7. SIEVE ANALYSES FOR B AGGREGATES[a] PERCENT RETAINED

Sieve Size	BCO-S2	BCO-S3	B1A	B1B	B1C	B1D	B1E
1 in.	2.0	9.2	0.0	0.0	0.2	0.0	0.1
¾ in.	2.8	9.2	3.2	4.5	4.6	6.3	4.4
½ in.	12.0	15.0	19.2	21.3	20.9	22.6	23.1
⅜ in.	15.0	15.0	19.3	17.8	13.4	16.7	15.9
No. 4	48.0	38.7	35.9	39.2	48.8	41.8	43.1
No. 10	15.0	9.9	16.3	10.2	10.2	11.8	11.9
No. 20	3.7	1.4	3.2	2.9	1.1	0.5	1.0
No. 40	1.0	0.5	0.8	1.6	0.4	0.1	0.2
No. 80	0.3	0.6	0.5	0.8	0.2	0.1	0.1
No. 200	0.1	0.0	0.3	0.7	0.1	0.0	0.1
—No. 200[b]	0.1	0.5	1.3	1.0	0.1	0.1	0.1

[a]ASTM C136.
[b]Percent passing.

TABLE 5-8. SIEVE ANALYSES FOR FAC, GEW, AND RGH AGGREGATES[a]
PERCENT RETAINED

Sieve Size	FAC	GEW-14A	GEW-18A	RGH-14A	RGH-18A
1 in.		30.6	30.0	11.3	8.5
¾ in.	1.4	20.6	19.5	14.6	14.2
½ in.	20.8	17.7	21.2	21.9	24.5
⅜ in.	25.9	5.9	8.5	15.0	13.8
No. 4	40.1	9.0	12.8	27.5	24.5
No. 8	9.2				
No. 10		6.5	4.7	6.6	11.6
No. 16	1.7				
No. 20		3.7	1.7	1.9	2.1
No. 30	0.7				
No. 40		1.8	0.5	0.7	0.3
No. 80		1.6	0.4	0.3	0.2
No. 200		1.2	0.1	0.1	0.1
—No. 200[b]		1.4	0.6	0.1	0.2

[a]ASTM C136.
[b]Percent passing.

TABLE 5-9. SIEVE ANALYSIS FOR SILICEOUS SAND[a]

Sieve Size	Percent Retained
No. 4	0.2
No. 8	10.0
No. 16	9.0
No. 30	25.5
No. 50	41.6
No. 100	12.2
No. 200	1.3
Pan	0.2

[a]ASTM C136.

TABLE 5-10. CONCRETE TEST RESULTS

Aggregate	Batch	Compressive Strength[a] psi	Splitting Tensile[b] Str., psi	Flexural Strength[c] psi	Autoclave Expansion[d] 10^{-6} in./in.
TCO	1	1420	207	256	4900
	2	1950	233	326	4800
T1A	1	2230	254	313	600
	2	3070	328	426	
T1B	1	3060	325	343	500
	2	3460			
	2A	3240			
	3	3110	302	418	
T1C	1	2650	258	432	
	2	3130	335	456	800
T1D	1	2960	315	297	800
	2	2760	302	447	
FAC	1	2020	284	407	
	2	3590	324	543	600
	3	4460	485	695	
BCO-S2	1	2780	309	392	600
	2	2200	248	374	800
	3	3260	309	470	
BCO-S3	1	3450	343	458	600
B1A	1	3330	257	492	600
	2	2840	292	336	
	3	3770			
	4	3100	336	457	
B1B	1	2590	237	383	1000
	2	2700	285	318	800
	3	3260	307	466	
B1C	1	3590	357	457	800
B1D	1	3130	288	407	1000
B1E	1	3100	295	371	1200
	2	3020	314	382	
GEW-14A	1	3490	318	478	1400
	2	3180	341	524	1400
GEW-18A	1	2990	349	407	1100
RGH-14A	1	3850	360	438	700
RGH-18A	1	3370	356	491	1100

[a]ASTM C39.
[b]ASTM C496.
[c]ASTM C78.
[d]See Reference No. 24.

TABLE 5-11. FREEZE-THAW TEST RESULTS[a]

Aggregate	Batch	No. F-T Cycles to Failure or ≥300		Percent Initial Weight Loss		Not Dried Dynamic E[b] (psi × 10^6)		Dried Dynamic E[b] (psi × 10^6)	
		Not Dried	Dried	Not Dried	Dried	Initial	Final	Initial	Final
TCO	1	29	303	−6.1	−26.9	1.65	1.02	1.64	1.05
	2	86	307	−1.9	− 1.4	1.84	1.05	1.88	1.51
T1A	1	52	305	−3.1	− 5.8	2.37	1.43	2.24	1.90
	2	300	300	−4.0	0.0	2.61	2.12	2.38	2.49
T1B	1	119	300	−6.5	− 5.2	2.38	1.44	2.38	2.07
	3	300	319	−1.0	+ 1.2	2.66	2.44	2.46	2.45
T1C	1	44	273	−5.9	− 4.1	2.48	1.52	2.31	1.48
	2	65	259	−0.2	− 0.5	2.56	1.56	2.47	1.48
T1D	1	15	131	−2.1	− 0.7	2.28	1.38	2.28	1.36
	2	30	169	0.0	− 1.9	2.28	1.37	2.22	1.32
FAC	1	305	309	−2.0	+ 0.8	2.82	2.23	2.69	2.72
	2	307	300	−0.6	+ 1.3	2.99	2.73	2.83	2.97
	3	307	299	−1.2	+ 1.0	3.16	3.02	2.98	3.08
BCO-S2	1	22	168	+0.5	− 3.2	2.32	1.36	2.28	1.37
	2	28	198	−3.1	−10.3	2.23	1.33	2.07	1.22
	3	49	285	+1.1	− 1.1	2.30	1.48	2.29	1.75
BCO-S3	1	21	302	+0.6	+ 1.4	2.32	1.40	2.18	2.07
B1A	1	60	247	−1.3	− 5.0	2.36	1.42	2.38	1.46
	2	57	292	−2.9	− 8.2	2.28	1.37	2.21	1.38
	4	152	308	−0.6	− 0.2	2.50	1.50	2.41	1.53
B1B	1	32	164	−2.3	− 2.1	2.49	1.44	2.33	1.40
	2	34	121	−0.3	− 1.2	2.36	1.37	2.20	1.32
	3	34	203	+0.8	+ 1.4	2.45	1.52	2.27	1.35
B1C	1	25	159	+0.1	− 4.1	2.09	1.25	2.07	1.24
B1D	1	31	215	+0.2	− 4.5	2.02	1.19	2.07	1.30
B1E	1	75	220	−7.2	− 8.4	2.32	1.41	2.27	1.39
	2	45	233	0.0	− 0.9	2.42	1.45	2.31	1.41
GEW-14A	1	30	310	+0.6	+ 0.7	2.21	1.33	2.11	1.82
	2	54	306	+0.6	+ 1.8	2.18	1.35	2.01	1.95
GEW-18A	1	23	273	−0.5	− 2.3	1.98	1.18	1.75	1.05
RGH-14A	1	94	300	−3.6	− 3.4	2.49	1.48	2.39	2.04
RGH-18A	1	64	219	+0.1	− 0.5	2.19	1.32	2.00	1.22

[a]ASTM C290.
[b]ASTM C215.

TABLE 5-12. DYNAMIC MODULUS OF ELASTICITY[a]

Aggregate	Batch	Dynamic E (psi × 10^6) 28 days
TCO	1	2.61
	2	3.02
T1A	1	3.49
	2	3.80
T1B	1	3.74
	2	3.96
T1C	1	3.80
	2	4.13
T1D	1	3.79
	2	3.65
FAC	1	4.40
	2	4.75
	3	5.12
BCO-S2	1	3.54
	2	3.15
	3	3.59
BCO-S3	1	3.52
B1A	1	3.68
	2	3.41
	4	3.78
B1B	1	3.64
	2	3.38
	3	3.61
B1C	1	3.42
B1D	1	3.47
B1E	1	3.54
	2	3.72
GEW-14A	1	3.47
	2	3.23
GEW-18A	1	2.91
RGH-14A	1	4.02
RGH-18A	1	3.34

[a]ASTM C215.

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