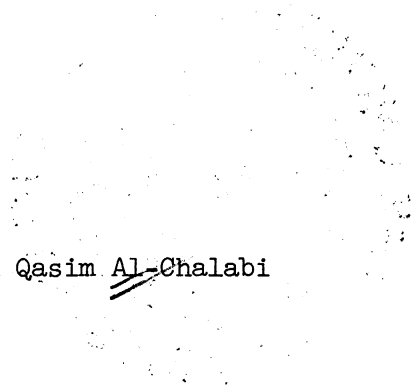
THE UNIVERSITY OF MICHIGAN

INDUSTRY PROGRAM OF THE COLLEGE OF ENGINEERING

LABORATORY PERFORMANCE STUDY OF COMMERCIALLY MANUFACTURED CONCRETE MASONRY UNITS MADE WITH LIGHTWEIGHT AGGREGATES



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ABSTRACT

The tremendous growth in the use of concrete masonry units made with lightweight aggregates has made desirable the accumulation of data on their performances in order that intelligent decisions can be made regarding their proper use in building. This study attempts to fulfill this need in the Michigan area for a cross section of the products presently available. Evaluation of the performance of the lightweight products is made by comparison with that of dense products, all under the same condition of test.

Materials were sampled from the regular production of ten commercial plants and included concrete units manufactured with several lightweight aggregates and three dense aggregates. The lightweight aggregates were: two cinders, one cinders mixed with fly ash, one cinders mixed with sand, one cinders mixed with expanded slag, three expanded slags including Waylite and Celocrete, one Waylite mixed with Beslite (expanded clay) and one expanded shale (Haydite). The dense aggregates were two natural sands and gravel and one air-cooled slag.

The tests conducted include: dimensional properties, unit weight, moisture content and absorption, compressive strength, flexural strength, elastic properties, freeze-thaw durability, drying shrinkage, thermal expansion and thermal conductivity. The tests were conducted on standard, hollow, 3-core, 8-inch block and on specimens cut from solid 4-inch slabs. Comparative studies were also

made of the physical properties of the aggregates themselves sampled at the plants involved.

Methods and results of tests are given in detail and overall averages are presented. The more important conclusions are:

(1) Adoption of the system of the modular dimensions by the industry is evident.

(2) Weights of 8-inch thick masonry walls containing the lightweight units are 29 to 36 psf and for dense units, 43 to 48 psf.

(3) Compressive strength of lightweight units varied between 58 and 88 per cent of the strength of dense units.

(4) Lightweight aggregate and air-cooled slag products are more absorbent than sand and gravel products.

(5) The moisture content of six of the products when sampled exceeded the 40 per cent maximum allowed by the ASTM specification. This requirement is considered of value in limiting shrinkage and subsequent cracking.

(6) There was no sharp line of demarcation between the dense and lightweight aggregate products with regard to the total shrinkage. Autoclaving resulted in lower shrinkage.

(7) Staining and popouts were shown by cinder products only.

(8) Flexural strength of dense aggregate products are 1.6 to 3 times higher than those of lightweight aggregate.

(9) Young's moduli for lightweight aggregate products range between 33 and 80 per cent of the lower value of the two sand and gravel aggregate products.

(10) Completely saturated lightweight aggregate products of highly cellular structure, unlike sand and gravel products, are vulnerable to a limited number of accelerated freeze-thaw cycles. However, this condition of full saturation is rarely met in service. At moisture contents of 40 per cent (maximum allowed by ASTM specifications) lightweight aggregate products withstood 500 cycles of freezing and thawing well.

(11) The coefficient of expansion of cinder products is lower than that of slag or sand and gravel products.

(12) The coefficient of thermal conductivity of slag and cinder products is approximately 1/3 to 1/2 that of the sand and gravel products. Good correlation exists between this thermal coefficient and unit dry weight.

It is concluded that lightweight aggregate masonry units possess characteristics that are desirable in construction materials for certain types of structures (residential buildings, etc.) such as their appealing surface texture, comparative lightness, which permits economy of construction, and lower thermal conductivity, which permits economy in heating and cooling. Methods have been devised and used with considerable success for reducing the shrinkage of certain lightweight aggregate products, such as autoclaving, etc. It is concluded that the continuing growth in the use of lightweight aggregate concrete masonry units for housing and similar types of construction is based on sound engineering judgment and is well justified.

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INTRODUCTION

This is a laboratory performance study of commercially manufactured precast concrete masonry units made with lightweight aggregate. Evaluation of the performance of these units is made by comparison with that of dense units (sand and gravel and air-cooled slag aggregate), all under the same conditions of test. Comparative studies were also made of the aggregates themselves sampled from the current production of the plants involved. This study will also offer comprehensive and useful data on the physical properties of these materials. The study is designed to provide a cross-section view of the products in the State of Michigan. For this purpose ten plants were chosen both for reason of their geographical location and the nature of their production. Four of these are located in Detroit and the rest are located in different representative areas in the state.

Materials were sampled from the regular production of these plants, including concrete units manufactured with ten different lightweight aggregates, and three dense aggregates. The lightweight aggregates are as follows: two cinders, one cinder mixed with expanded slag, one cinder mixed with sand, one cinder mixed with fly ash, three expanded slags including the commercial names Waylite and Celocrete, one expanded slag mixed with expanded clay (Waylite and Beslite) and one expanded shale (Haydite). The dense aggregates are two sand and gravel and one air-cooled blast-furnace slag.

Tests were performed on the most common size hollow blocks of the modular dimensions 7-5/8 x 7-5/8 x 15-5/8 inches with three

rectangular core openings, and on specimens cut from solid slabs of the modular dimensions 3-5/8 x 7-5/8 x 15-5/8 inch.

The study concerns in detail the following physical properties of aggregates and concrete masonry units.

A. Aggregates.

1. Grading.
2. Unit weight.
3. Specific gravity (bulk oven-dry basis, bulk saturated surface-dry basic, and apparent), and absorption.
4. Deleterious inclusions.
   a) Materials finer than #200 sieve.
   b) Organic impurities.
   c) Materials causing popouts of concrete.

B. Concrete Units.

1. Dimensional properties of the Modular Unit.
2. Unit weight, moisture content, and absorption.
3. Compressive strength.
4. Flexural strength.
5. Elastic moduli (static and dynamic).
6. Resistance to accelerated cycles of freezing and thawing.
7. Staining and popouts of concrete masonry units.
8. Drying shrinkage.
9. Thermal expansion.
10. Thermal conductivity.

Results of tests, presented and analyzed in the following chapters, indicate that concrete masonry units having the advantage of light weight, generally exhibited good performance when compared with denser units.

Each plant selected (except one) uses one or more lightweight aggregates in manufacturing their products. This is true for a great number of plants in the state. Indeed, the use of lightweight aggregate, because of its many advantages, has advanced so rapidly in recent years that it is now dominating the industry. This, coupled with the very large increase in the use of precast concrete units in construction, makes lightweight aggregate units highly important building materials.

CHAPTER I

HISTORY

The use of lightweight building material is known early in history. The Romans used natural lightweight stones, such as volcanic cinders, to build their great aqueducts to carry water to their cities. The use of hay to reinforce and reduce the weight of mud bricks was also known at that time. Until recently the use of lightweight building material was on a small scale. In this country, following the introduction of portland cement concrete, the emphasis was, for a long time, mainly on strength which dense aggregate concrete produced. It was not until recently by experience, research and modern technology that lightweight aggregate was fully recognized as an important building material..

A. Lightweight Aggregate.

Lightweight aggregate for concrete was first used not strictly because of its physical characteristics. In the western states, because of economy, natural stones such as volcanic cinders were processed and used. In the eastern states, factory by-products such as industrial cinders and blast furnace slag came into use. None of the concrete so produced was accepted as of structural quality.

The use of cinder concrete as a filler for roofs was common as early as the year 1896 in the city of New York[1]* but was viewed with suspicion. This suspicion was greatly lessened by the research of Columbia University and the Underwriter's Laboratory's full scale

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* Reference 1. See list of references.

fire test. Blast-furnace slag was considered a waste product at the beginning of this century, accumulating in huge quantities which became a major problem for the steel industry. Trying to find some use for this material, the city of Detroit, for example, used large quantities for surfacing unpaved streets or as aggregate for bituminous concrete[2]. Not until the year 1930 was it fully recognized as a concrete aggregate.

Stephen J. Hyde[3], a chemist working in his laboratory in Kansas City in the year 1917, perfected a method of making lightweight aggregate by heating a certain shale, slate and clay to incipient fusion, and it was found that this method resulted in lightweight aggregate of good structural strength and insulating value. Two years later a huge source of lightweight natural stone commercially known as Zonolite was discovered in California and used, after processing, as concrete aggregate. Soon the possibility of good concrete from lightweight aggregate became apparent to the engineers and builders who started developing methods and techniques for manufacturing and using these new aggregates. Expanded clay and shale, processed diatomaceous earth, expanded slag, expanded slate, sintered, and natural lightweight aggregate came into use.

Today it is estimated that more than 27 million cubic yards of this aggregate is manufactured annually[5]. This has a variety of uses. However, the bulk of the product is used for lightweight concrete and especially for precast units; - the latter is estimated to have used about 80 per cent of the total in the year 1954.

B. Precast Building Units.

The precast concrete industry was the main consumer and promoter of lightweight aggregate. The use of this aggregate resulted in lighter units with varied surface textures, better thermal insulation and acoustical absorption and less thermal expansion. It also resulted in a decrease of compressive and flexural strength and higher drying shrinkage. Yet the strength of properly made precast units is usually adequate for the type of structure for which it is used. Drying shrinkage is still a problem, but methods of reducing it are currently being investigated.

Precast concrete masonry units were originally frequently manufactured by the aggregate producers to dispose of an accumulation of certain sizes of aggregate. Concrete bricks were first manufactured followed by the two core block. Later the three core unit was adopted and became popular because of the ease of cutting the block symmetrically on the job.

Concrete precast units were at first made with sand and gravel. Later, industrial cinders and blast-furnace slag were used. The light weight and the availability of these aggregates resulted in the growth of their use.

By 1930, a number of specifications were written around these products. This gradually forced out the small manufacturers because of their inadequacy to meet the requirements of the specifications. This eliminated very low quality products from the market, permitting the growth of fewer but better established plants. This growth was continuous through the thirties and part of the forties but was greatly increased in the years following World War II.

This is apparent from Table 1. The unit price of the product did not change considerably during that time.

This fast rate of increase in production following the year 1946 continued through the fifties as will be seen from Table 2.

Another fact that the data does not reveal is the increase in the use of lightweight aggregate during that period of time. Available data estimated a consumption of about 20 million cubic yards of all aggregate in 1947, against 27 million cubic yards of lightweight aggregate alone in 1954. Approximately 80 per cent of the 1954 production of lightweight aggregate was used in manufacturing lightweight units. These units constitute about 54 per cent of all the precast units produced with the following percentage distribution:

43 per cent cinder

27 per cent expanded clay and shale

19 per cent expanded slag

7 per cent pumice

4 per cent scoria and other volcanic cinder.

TABLE 1. RATE OF GROWTH OF PRODUCTION OF CONCRETE MASONRY UNITS IN THE YEARS 1934-1947.[6]

Year	Number of Plants	Gross Selling Value in Millions of Dollars
1934	88	1.0
1936	97	1.7
1938	119	3.2
1940	128	5.3
1942	141	7.3
1944	149	6.6
1946	---	140.0

TABLE 2. RATE OF GROWTH OF PRODUCTION OF CONCRETE MASONRY UNITS IN THE LAST DECADE
(Data obtained from different sources)

Year	Number of 8-inch Equivalent Units (in millions)
1944	340
1947	1000
1952	1780
1953	1800
1954	1900

C. Advancement in Technology.

Currently, the precast concrete masonry industry manufactures about 2 billion 8-inch equivalent units annually of different sizes and shapes, more than half of which are made with lightweight aggregate of many varieties.

This growth was a result of better products, of advanced technology, and new methods and materials. The modern plant of today is far from the plant of two decades ago. It is larger, better equipped, highly efficient and easier to control by automation.

The improvement, extending to all plant operations[7], will be summarized as follows:

(1) Automation of modern plants results in an economy and facilitate quality control. Automatic screening of aggregate, batching, mixing, molding and curing (heat and humidity control) are becoming standard.

(2) Today, manufacturers, especially those using cinder or slag, frequently have their own aggregate processing equipment. They also stockpile large quantities of aggregate in storage yards

Figure 1. Cinder Aggregate Processing Equipment.

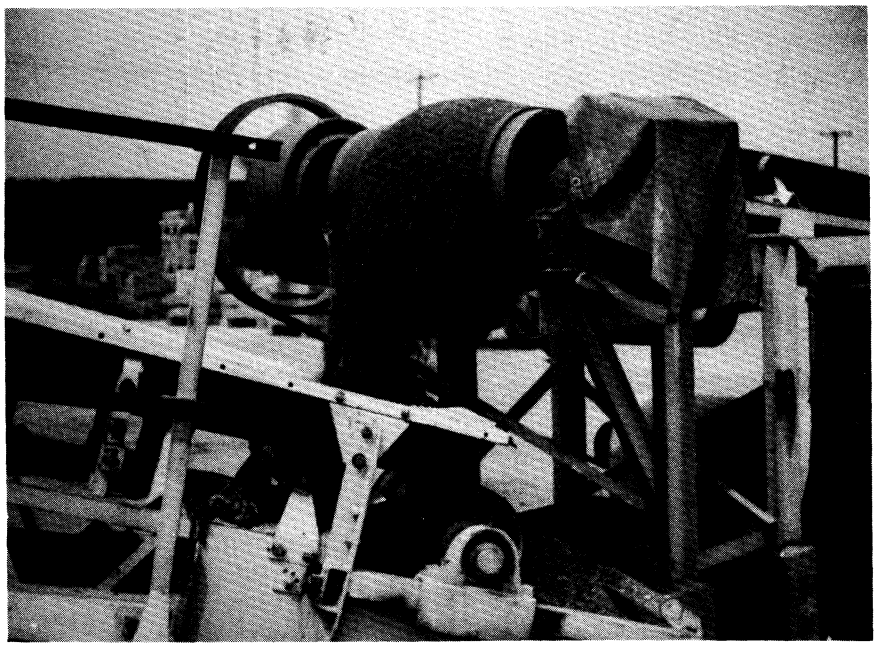

Figure 2. Magnet Pulley for Separating Metallic Particles from Aggregate.

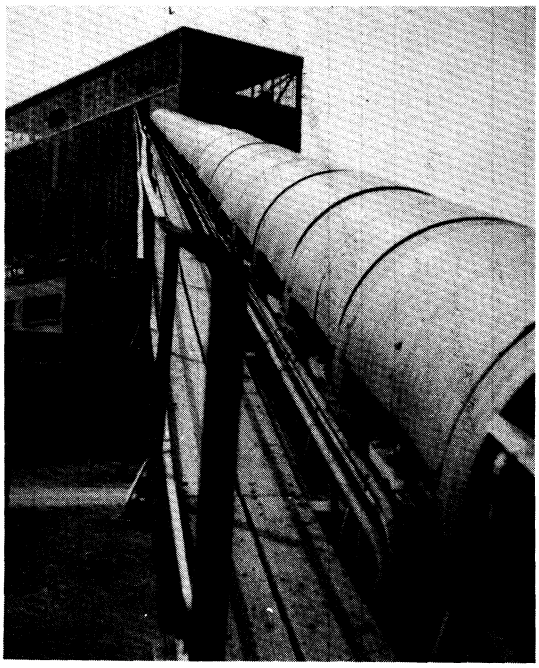

Figure 3. Pipe Conveyor Transporting Aggregates to Over-Head Bins for Distribution to Batchers.

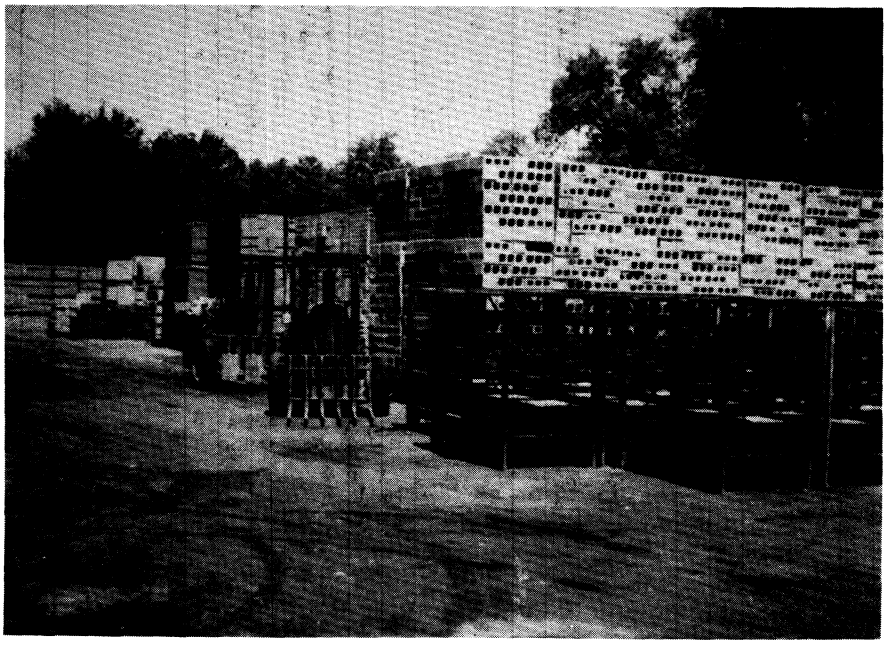

Figure 4. Open Yard Storage of Products. Also notice two fork-lift trucks at left and metal racks at right.

and over-head bins. Bulk cement is also stored in such bins and are all connected to the batcher.

(3) Quality control and the use of lightweight aggregate have made the once standard continuous mixer obsolete. In spite of its high capacity and its occupying a relatively small area, this open long drum horizontal mixer has been discarded in favor of the modern fifty cubic foot mixer which is usually equipped with an automatic batcher.

(4) Molding machines have undergone a great change. Before 1940, the tamping machine was the standard and is now displaced by the vibrating machine. The latter is fully automatic. Receiving the concrete from the mixer (usually located above the molding machine) it molds by both vibration and pressure. These molding machines are also superior to the old tamper type by reason of speed, economy and capability of handling drier mixes without crushing the softer lightweight aggregates. In addition to its ability to manufacture many sizes and shapes of units, only one operator is required. His duty is primarily to remove the units, automatically ejected, from the front of the machine to a rack by means of an air hoist.

(5) Moist curing, which was standard in earlier plants, has been supplanted by two more efficient curing methods, high-temperature (normal pressure) steam and high-pressure steam curing. The tendency of the modern plant is to have many relatively small curing chambers which are more efficient and easier to control. High-pressure steam curing, although many consider it superior to other methods and certainly less time consuming, is not as common as expected, possibly because of the high cost of equipment.

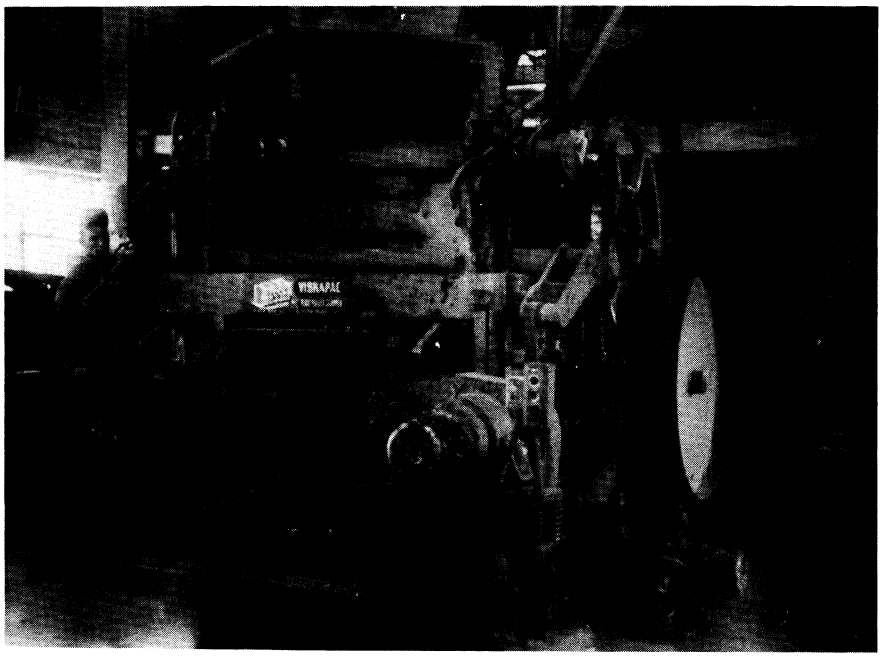

Figure 5. Automatic Molding Machine.

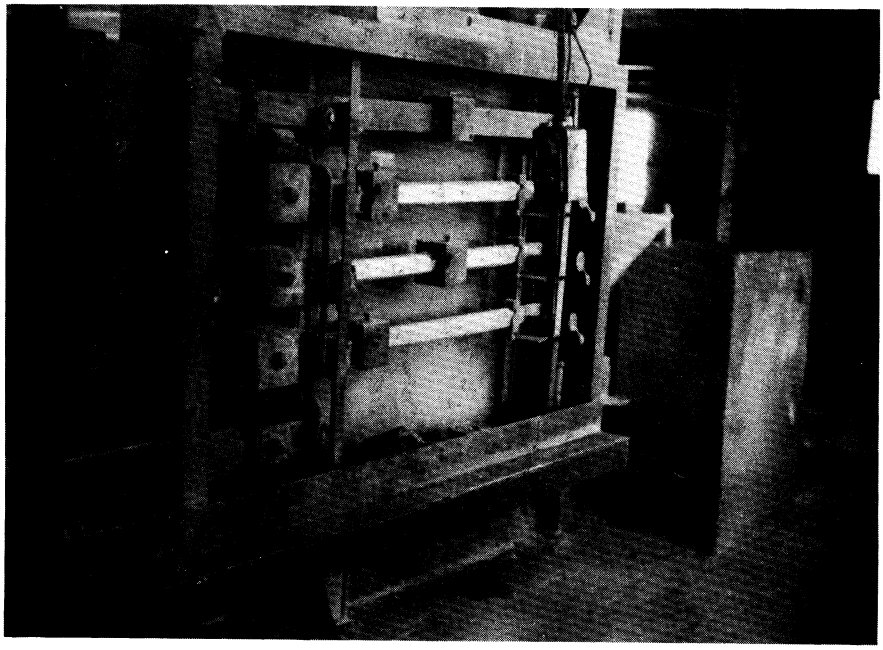

Figure 6. Scale of Automatic Batcher.

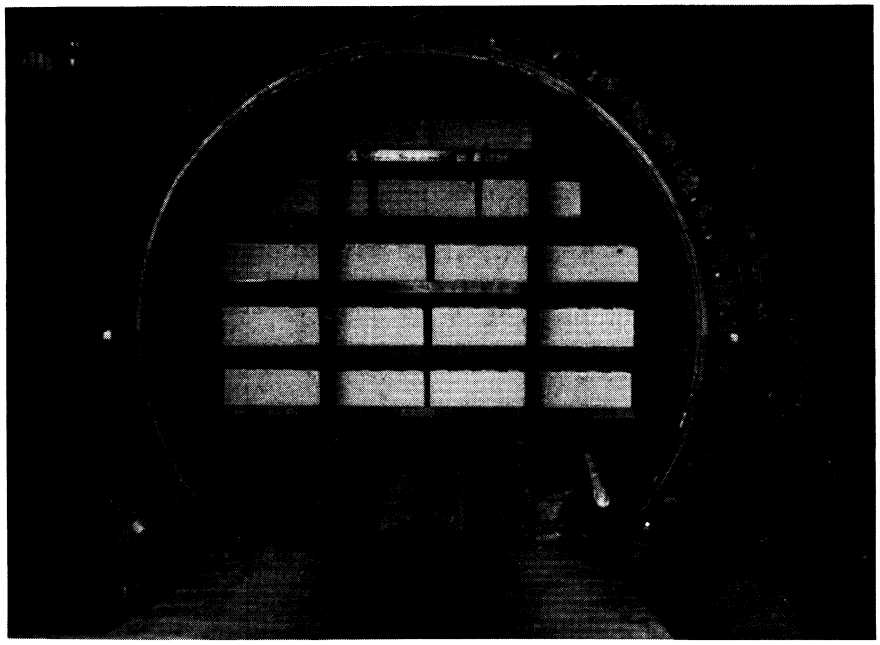

Figure 7. High-Pressure Steam Curing Kiln with "Cap" Removed.

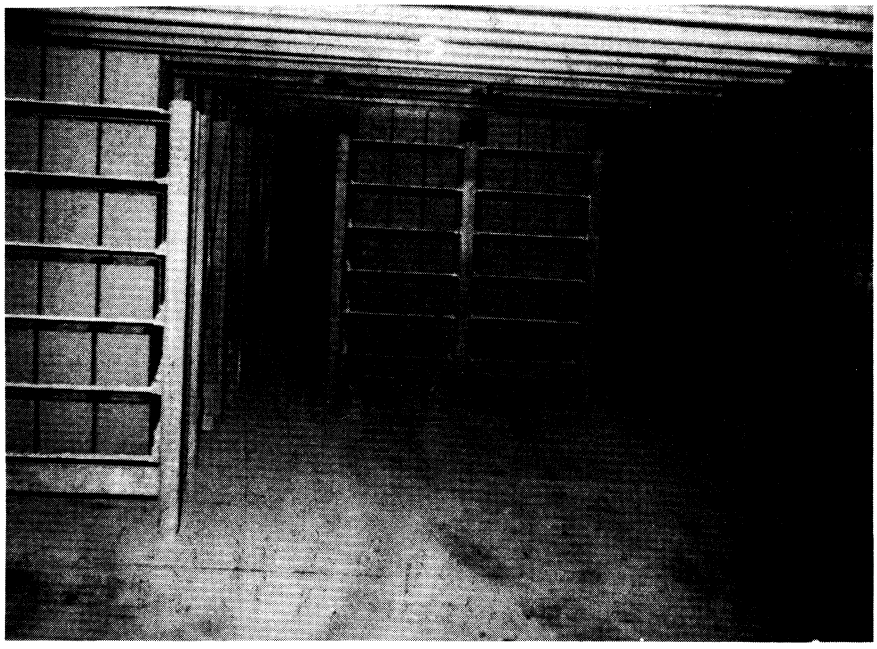

Figure 8. Low-Pressure Steam Curing Chamber.

Some modern plants are mechanically drying their products following the curing cycle.

(6) Product handling equipment advancement was essential to greater plant efficiency. Prior to 1938, products were moved by hand or by rail cars. About that time the spring supported platform truck was developed which is capable of handling green product racks and finished products. In the year 1944 the fork-lift truck was introduced, which is more efficient in handling finished products and often moves and loads large cubes of the product without necessity of a pallet.

(7) Better quality control has been established. Many plants have their own small laboratory for daily routine tests.

D. Research on Lightweight Concrete, Precast Masonry and Aggregates.

The foregoing advancement in materials and technique of manufacture was made possible by research of the industry and public organizations.

The history of the development of research in any industry is, usually, timed with the growth of that industry. Twenty-five years ago there was hardly any important research on lightweight aggregate and concrete. An early recommendation on precast masonry by the American Concrete Institute (A.C.I.), dated 1925, did not recognize lightweight aggregate for concrete; precast concrete masonry (blocks) were classified with so called concrete brick and with tile. However, a few years later it started to gain recognition.

The report of the A. C. I. committee, reviewing the above mentioned recommendations, published in 1930 is interesting[8].

Precast concrete masonry units were reported gaining importance as building materials with a growing industry of its own (not merely a device employed by the aggregate producers to dispose of an accumulation of a certain size of aggregate). Cinders and Haydite (burned shale) were recognized as concrete aggregate. Advancement of technique of manufacture of precast units, new methods in proportioning, mixing, curing, and molding were also reported and recommended.

In the following years, as the use of such products was increasing, research began developing on both old and new materials. This gained momentum during the last decade when the huge expansion in the use of lightweight concrete occurred.

Wide study of previous research on this subject would be out of place here; however, a brief presentation of the more important work is made.

It would not be significant to study these researches in sequence of their time of conduct. They may better be divided into two classes: one dealing with machine molded concrete, the second dealing with plastic concrete usually poured in forms.

1. Precast Concrete Masonry.

Although the use of industrial cinder as an aggregate for plastic concrete or concrete masonry was very common by 1930, there were very few scientific studies on the subject. The literature available at that time was not extensive. A. C. I. committee 203 was set up to bring the information on the subject up to date. Their comprehensive report, published in 1931[1], started by defining industrial cinders (anthracite and bituminous). Chemical composition and their effect on concrete

was also studied. Results of tests for physical properties of aggregate were presented and discussed. The making of masonry units was also presented. Specifications for aggregate, concrete and its products were offered along with the procedure of the necessary formal testing. This work helped a great deal in removing doubts about these materials.

One of the earliest researches on shrinkage of concrete masonry was published by W. D. Allen[9], in 1930, aimed at determining the rate and extent of shrinkage occuring in concrete walls as affected by the moisture content of the unit at the time the wall was layed. Twenty-four wall panels, each one block wide and eight blocks high, were constructed of units made with sand and gravel, Haydite or cinder aggregate and were two, seven or ninety days old when tested. Walls were saturated for forty-eight hours, then dried at a temperature varying from 69 to 80 F and relative humidity of 32 to 77 per cent. It was concluded that the use of air dry masonry units will reduce shrinkage 20 to 35 per cent. Recommendations were also offered to reduce drying shrinkage to a minimum.

An interesting early research on the structural performance of concrete masonry walls as related to the strength of the unit was published by the University of Illinois[10] in 1931. This relation is important since almost all the building codes put provision on the required strength of the unit. In this work, the walls were divided into two general classes, load-bearing and non-bearing. They were of hollow blocks of four sizes, with two or three, oval or rectangular, cores.

Seventy walls, 6 feet long and 9 feet 6 inches high and forty-four wallettes, two feet long and four feet high, all one block thick were constructed. These walls were built with units made of different aggregates, consisting of sand and gravel, Haydite, crushed limestone, cinders, and crushed air cooled slag. All units were commercially manufactured and moist cured for twenty-one days plus seven days storage in air. Walls built with two kinds of mortar, 1:1:4 1/2 cement lime mortar and 1:3 cement mortar containing ten per cent lime, were then stored at 70-75 F and 50-65 per cent relative humidity (R.H.) for 32 days before testing. They were tested to failure in compression and cross-bending. Loading was uniform or concentrated, axial or eccentric. Lateral and vertical deformations were measured by dial strain gages. Among the findings were:

(1) The compressive strength of a wall panel depends on the compressive strength of the unit with an average ratio of approximately 0.53. But the flexural strength depends on adhesion of mortar to the units with failure following along a horizontal mortar joint.

(2) Walls, eccentrically loaded at the edge of the middle third deformed uniformly and developed a strength equal to approximately 3/4 that of axially loaded walls.

(3) As expected, the deformations and working stresses of concrete masonry walls were less than that of reinforced concrete walls.

(4) There was a constant ratio between results obtained for walls and wallettes of the same materials and,

therefore, wallettes could be used for testing instead of the larger and more expensive assemblies.

An important research that promoted concrete masonry units as building material was the full scale fire endurance test conducted at the laboratory of the Portland Cement Association and published in the year 1932. The load-carrying capacity of concrete masonry walls, made of units of widely varying characteristics, both during and after exposure to standard fire test conditions were investigated[11]. Comparative data on similar walls not exposed to fire were included. These investigations consisted of tests on more than 200 walls 5-1/2 ft. wide, 6 ft. high and 4, 8, and 12 in. thick. All the blocks were manufactured in the laboratory, using a block machine (tamper) of standard make, with nine different aggregates including Haydite, air-cooled slag, cinder and coke breeze. Among the principle findings were:

(1) The comparative strength of walls both before and after exposure to fire was directly proportional to the strength of the unit.

(2) The strength of mortar, type of joints and characteristics of mortar bedding had a marked effect on strength both before and after the test.

(3) Substantial load-carrying capacity and safety were exhibited by walls during and after fire exposure.

The University of Wisconsin test (sponsored by the National Concrete Masonry Association) published in 1939, is held by many as being responsible for speeding the decision of

the industry to shift from tamping to molding by vibration. It was a comparative study of the physical properties of masonry units which were manufactured by the two methods from the same mixes[12]. A single grading of each of the seven following aggregates were used: cinders, Haydite, limestone, Pottsco (expanded slag), sand and gravel, Superrock (expanded slag), and Waylite. The cement was a blend of four brands. Six different batches were used for each aggregate. From each batch, 30 half units and 30 full units 8 x 8 x 16 inches were made. These units were tested for compressive strength, absorption, capillarity, specific weight, durability, volume change and thermal expansion coefficient. The findings were numerous and mostly in favor of molding by vibration.

Staining and popouts of concrete, especially in cinder masonry units, are common evils of bad quality aggregate. It has received much attention because of the increase in the practice of painting exterior walls. The Portland Cement Association, with the cooperation of a group of cinder block producers, conducted a comprehensive study of the cause and prevention of staining and popouts in cinder concrete. This report, published in 1948[13], divided popouts into two classes according to whether or not it is accompanied by staining. It also found that staining, whether or not accompanied by popouts, is usually caused by one or more compounds capable of decomposition in the presence of air and moisture, while free lime (or partially hydrated lime) is the main cause of popouts that are not accompanied by staining. Free magnesia and calcium sulphate are considered responsible

for very few cases of this type of popouts. The results are based on the chemical and microscopical analysis of several hundred cases of stains and popouts and analysis of aggregates collected from forty different sources. Procedure of determining the amount of this deleterious material is based on physical and chemical tests. Methods of treatment were offered for such impurities, ranging from simple magnetic separation (which is usually used for all kinds of cinder aggregate) to a steam or lime treatment for cinders containing many such impurities.

Attention was always focused on the important problem of drying shrinkage of concrete masonry units. Two important researches dealing with the subject were published recently.

The first is a progress report of A. C. I. committee 716, working on the physical properties of high-pressure steam cured concrete blocks[14], focusing the study on drying shrinkage of blocks. The research was conducted by four laboratories using three different procedures, called the "Reference Method", "British Method" and "Rapid Method". Shrinkage was measured by either the Whittemore strain gage, the "Ames" dial comparator or both. In the Reference Method, the blocks were immersed in water for 96 hours at 73 ± 3 F; then dried in cabinets at 73 ± 3 F over a saturated solution of sodium hydroxide (equivalent to 10 per cent R. H.). In the British Method the blocks were immersed in water for 96 hours at 73 ± 3 F; were then dried in cabinets over a saturated solution of calcium chloride (equivalent to 17 per cent R. H.) at 122 ± 3 F. In the Rapid Method the blocks were immersed in water for 24 hours and then dried at 220 to 230 F in the oven.

Tests were conducted on blocks made in four commercial plants with sand and gravel, cinder and expanded clay. Silica dust was also used with some of the mixes. Some of the main findings were:

(1) Twenty-four hours of immersion is adequate for total saturation.

(2) The quantity of the moisture lost was independent of the method of drying.

(3) The British Method and the Reference Method gave approximately the same total shrinkage; the Rapid Method gave approximately twice the value obtained by either of the other methods.

It was also suggested that if the foregoing provision (3, above) is made, the Rapid Method might be convenient to adopt.

The second research at the University of Toledo was aimed at the relation of shrinkage to moisture content in concrete masonry[15]. This was a broad program, ranging in scope from studies in the chemical make-up of the cementitious phase to dealing with the shrinkage cracking of restrained wall panels under controlled condition of moisture and humidity. The blocks, all 8 x 8 x 16 inches were manufactured under commercial conditions with sand and gravel, cinders, expanded shale, expanded slag, sintered shale and pumice. Curing was done by five methods: "moist-air curing", "low-temperature steam" (120 F), "high-temperature steam" (170 F) plus drying at 150 F and "high-pressure steam." The finished blocks were saturated under water for forty-eight hours; then stored in air at 25 ± 3 per cent R.H.

and 73 ± 3 F temperature with air circulation at 5-10 mph. Shrinkage measurements were taken until equilibrium was reached. The specimens were then resaturated for forty-eight hours and dried in air at 70 per cent R. H. and 73 ± 3 F until their shrinkage was again stabilized; then drying was continued at 25 per cent R. H. and 73 ± 3 F until the final shrinkage was also stabilized. Among the main findings were:

(1) High-pressure steam curing, except for pumice aggregate, reduces the drying shrinkage of the product by approximately one-half that which resulted when other methods of curing were used.

(2) A forty per cent moisture content (the usual specification requirement) is not accompanied by a marked reduction in shrinkage. Considering all factors, only about one-fifth to one-half the potential shrinkage has developed at this moisture content; the balance of shrinkage taking place when the shrinkage reached equilibrium at 25 per cent R. H.

2. Plastic Concrete (Made with Lightweight Aggregate).

Since cinders and Haydite were among the first used as lightweight aggregate, early studies of their properties are found. One of these, on cinders, was previously presented on page 9. Another study conducted at the University of Illinois in 1930, was on the physical properties of Haydite aggregate and concrete (plain and reinforced)[16]. Two types of Haydite concretes were considered, one with sand as fine aggregate, and the second with "all-Haydite" concrete. The construction and design features were also studied for the two types of concrete,

providing data and recommendations to facilitate the design and construction of this type.

Waylite was the subject of the same type of study at the University of Wisconsin[17], sponsored by the Waylite Company, Chicago, Illinois. The physical properties of Waylite aggregates and concrete (plain and reinforced) were investigated. Mix design and methods of mixing were also developed. Data on weight, strength, absorption, stiffness, and damage by freezing and thawing were also presented. Limited study was also done on reinforced Waylite concrete.

Burned shale and expanded slag aggregate in concrete were studied at the National Bureau of Standards. The physical properties of three grades of concrete from each aggregate were tested[18]. It was found that if the cement content was low (3 sacks per cubic yard) workability was very poor. This was corrected by entrained-air, up to 25 per cent. For reinforced concrete, bond and flexural stresses were also studied.

A comprehensive research on lightweight plastic concrete was sponsored by the Housing and Home Finance Agency and conducted, separately, by both the National Bureau of Standards and the Bureau of Reclamation. The first mentioned, investigated aggregates available in the eastern part of the United States, and the second, aggregates available in the west. The Bureau of Standards worked on 11 lightweight aggregate samples including exfoliated vermiculite, expanded perlite, three samples of expanded blast-furnace slag, sintered fly ash, pumice, expanded slate and clay[19, 20]. The Bureau of Reclamation

worked on 19 samples obtained from 17 different producers, including two samples of expanded shale, one scoria, three expanded slag, five pumice, five expanded perlite, two exfoliated vermiculite, and one diatomaceous earth[19, 21]. Both laboratories included sand and gravel aggregate in their tests for comparison with the lightweight aggregate. Both studied the physical properties of aggregate and concrete. In the making of concrete, the Bureau of Standards used the aggregate in the same grading as furnished by the producers, adding Vinsol resin, an air-entraining agent, to the mix to lend workability to the concrete while the Bureau of Reclamation tried to make workable concrete without the use of an air-entraining agent, when aggregates were of suitable grading, or by altering the grading, such as crushing or regrading; however, the addition of air-entraining agents was a necessary step for a number of aggregates.

Four grades of concrete per aggregate, approximating a cement content of three, five, seven, and nine sacks per cubic yard of concrete, were tested by both laboratories. Concrete was mixed by a drum mixer, molded by hand and cured by fog.

Both researches resulted in useful data on the following physical properties of aggregate and concrete:

(1) On aggregate: sieve analysis, unit weight, bulk specific gravity, absorption, and crushing strength.

(2) On concrete: unit weight, absorption, compressive strength, modulus of elasticity, modulus of rupture, thermal conductivity, resistance to freezing and thawing, drying shrinkage, sawability, and nailability.

Both laboratories analyzed their data, drawing conclusions and offering recommendations:

(1) The use of air-entraining improves workability to a large degree, especially in leaner mixes.

(2) Drying shrinkage is the most unfavorable physical property of lightweight aggregate concrete, because of higher absorption of aggregate and because higher cement contents for a given strength are required.

(3) Thermal conductivity of lightweight aggregate concrete is much lower than that of dense concrete.

(4) Certain lightweight aggregate concretes, containing suitable amounts of entrained air, have a high resistance to freezing and thawing, despite the fact that their densities and strength are relatively low.

Much early work was conducted on plastic concrete made with lightweight aggregate. In these early studies, concrete mixes were designed to suit the purpose of the study, i.e., concrete manufactured by laboratory methods and controls and poured in forms. The result of such investigations will not necessarily represent commercially produced concrete masonry.

Much research was also done on concrete masonry, some described in the foregoing pages, conducted on specimens which are either manufactured in the laboratory, using block making machines, or are obtained from commercial plants, which, in some cases manufactured the units especially for the tests. Moreover, the majority of this research is aimed at studying

one property of closely connected properties, such as drying shrinkage and absorption, or a comparative study of a certain method or technique of manufacture.

Therefore, a comprehensive, comparative study of the physical properties of commercially produced concrete masonry units made with lightweight aggregates and sampled from the regular production is desirable. After all, these are the products actually being used in building today.

In a study of this nature, it will be practical to conduct research on products selected to represent a certain locality. The present work, as said before, was designed to give a cross-section study of the products in the State of Michigan.

CHAPTER II

TEST SPECIMENS

Discussion will now be made of procuring the test specimens. Each of the ten participating plants was visited to obtain samples and to prepare a field report on materials and methods of manufacture. These field reports are summarized in Table 3, which will be discussed later.

A. Sampling.

Concrete units were sampled from the lots of the regular products available at the plant at the time. The procedure of sampling was according to the requirements of ASTM Designation C140-52 (Methods of Sampling and Testing Concrete Masonry Units)[22]. Aggregate samples were also obtained from the processed materials at the plant. The procedure followed in sampling the aggregate was according to the requirements of ASTM Designation D75-48 (Methods of Sampling Stone, Slag, Gravel, Sand, and Stone Block for Use as Highway Materials)[22].

A number was assigned to each producer participating in this study. Starting at number one, each number represents a single producer. After each number a letter(s) has been used to represent the aggregate(s). This procedure has been followed throughout this work (see Table 3, column 1).

Plants #2, #3, and #7 each submitted two types of products (made with two different aggregates) while the remaining plants submitted one type each, making a total of thirteen different products submitted by the participating plants.

(1) Twenty-five hollow three-core masonry units (blocks) of the modular dimension 7-5/8 x 7-5/8 x 15-5/8 inch were obtained.

(2) Also from each product except 2H and 7BW (see Table 3, column 1), ten solid masonry units (slabs), of the modular dimension 3-5/8 x 7-5/8 x 15-5/8 inch, were obtained. Slabs were not available from 2H and 7BW.

(3) From all the plants, except #2 and #9, aggregate samples were obtained. Aggregates were not available from these two plants at the time of sampling.

Upon arrival at the laboratory, the samples were marked for identification. Five specimens were selected from the hollow masonry units for immediate testing (ASTM Designation C140-52; see also Chapter V); the remainder of the specimens and the concrete aggregate samples were stored in the laboratory under normal temperatures and humidity conditions.

Full size hollow blocks were used for testing whenever possible, otherwise specimens of desirable sizes were cut from the slabs. Since products 2H and 7BW were not available in slabs, it was not possible to conduct tests on these in which such specimens were needed. Details of each specimen used for each test will be given when discussing that test.

B. Plant Operation.

Plant operation is meant to include materials, methods and equipment used in making the products, mix data, methods of mixing, molding, curing, and storage.

Plant operations were studied and data obtained from the producers to prepare the field report. These operations, as expected,

differed from one plant to another, but in general, all were found to have certain features of the modern plant described in Chapter I. The field reports are summarized in Table 3.. Column 1 of this table gives the identification number to aid interpretation of the letter symbols. The fine and the coarse aggregate from which the products were made are listed in columns 2 and 3. Column 4 indicates types of cement. All plants except #1 and #3 use high early cement (plant #3 uses high pressure steam curing). Column 5 covers admixtures and indicates that only plants #1, #5, and #7 use added air-entraining agents (plant #1 also uses a wetting agent); plant #10 adds fly ash to its mix. Mix data are given in columns 6 to 11. Methods of measuring the mix are given in column 6 from which it is seen that all but plants #1 and #4 measure by weight. Plants #1 and #4 measure by volume but, as will be seen later, they also use a different method of mixing. The mixing water reported in column 7, variable in most cases, does not permit precise calculation of the water cement ratio. This is because the moisture content of the aggregate, which stands in large piles in open yards, is variable, and the consistency of the mix is actually measured by that required for molding by pressure and vibration and permitting subsequent stripping of the unit without physical damage. Columns 8, 9 and 10 give the cement, fine and coarse aggregates used in making one batch. It is to be noticed that the maximum size of the coarse aggregate is usually 1/2 inch; some plants use combined fine and coarse aggregate due to the small top size. Column 11 gives the amount of admixture, if used, and is normally added to the mixing water. Methods of mixing are summarized in column 13 from which it is observed that except for

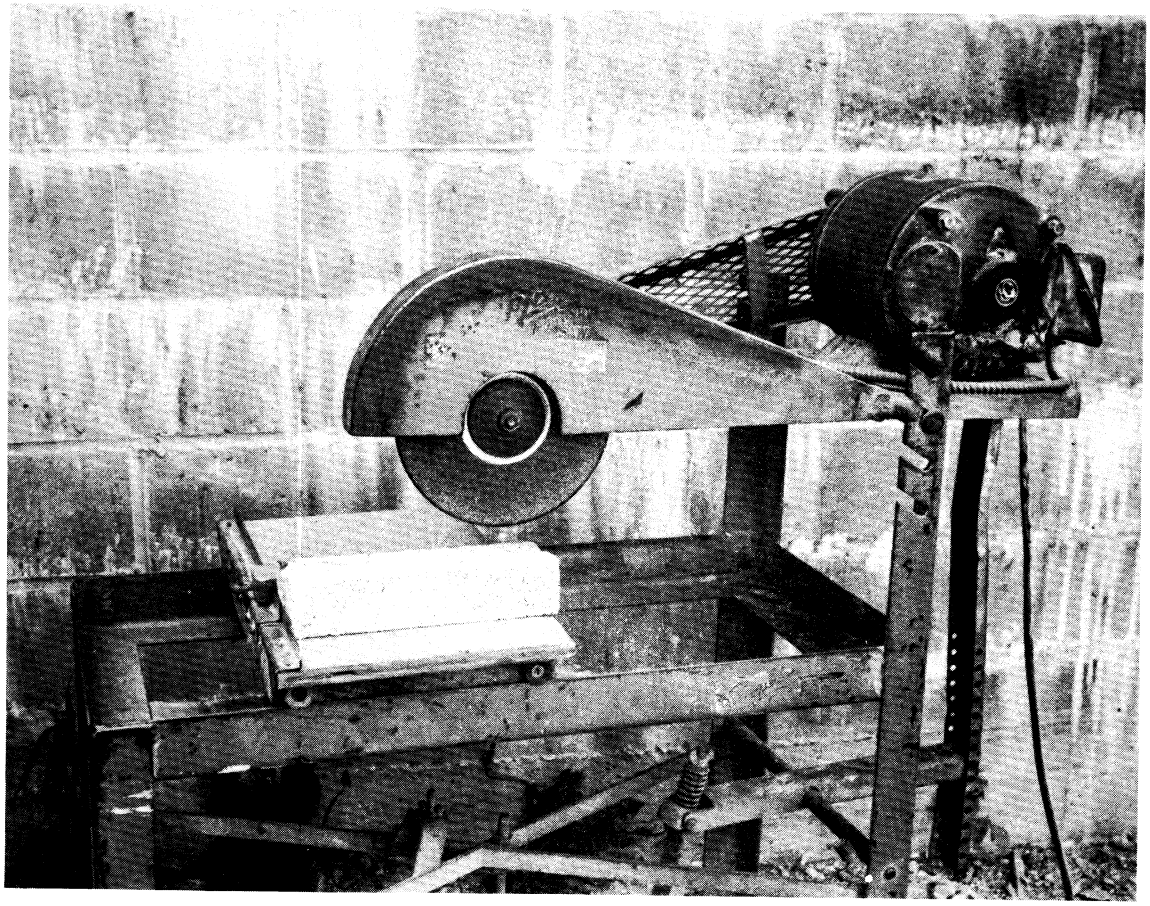

Figure 9. Saw Used for Cutting Specimens.

TABLE 6. PHYSICAL PROPERTIES OF AGGREGATES

Sample No.	Type of Aggregate	Material Passing (#200 Sieve % by Weight)	Specific Gravity			Absorption		Organic Impurities Colorimetric Test	Popouts
			B.S.G Oven-Dry Basis	B.S.G. Saturated Surface Dry Basis	A.S.G.	% of Dry Weight	% by Volume (Dry Compacted)		
1C9	Cinder Combined	7.3	1.71	1.85	1.99	8.40	7.87	No Change in Color	None
3C9	Cinder Combined	5.6	1.74	1.90	2.06	9.00	8.26	No Change in Color	None
3SG9	Sand and Gravel Combined	2.23	2.63	2.66	2.73	1.50	2.50	Slight Change in Color	None
4SG9	Sand and Gravel Combined	3.15	2.66	2.69	2.75	1.30	2.47	Very Slight Change in Color	None
5C9	Cinder Combined	3.67	1.68	1.82	1.95	8.20	7.73	No Change in Color	None
6ES19	Expanded Slag Coarse	0.14	1.47	1.57	1.64	6.75	5.16	No Change in Color	None
6ES29	Expanded Slag Fine	0.59	1.78	1.87	1.96	5.11	5.33	No Change in Color	None
7W19	Waylite Coarse	0.14	1.48	1.59	1.67	7.57	5.81	No Change in Color	None
7W29	Waylite Fine	6.72	1.80	1.89	1.98	5.12	4.66	No Change in Color	None
7B9	Beslite Combined	5.70	1.50	1.71	1.89	14.13	15.2	No Change in Color	None
8CEL19	Celocrete Coarse	0.93	0.97	1.21	1.27	24.3	11.05	No Change in Color	None
8CEL29	Celocrete Fine	8.73	1.49	1.71	1.90	14.41	10.68	No Change in Color	None
10CA9	Cinder Combined	10.7	1.71	1.85	1.99	8.41	8.13	No Change in Color	None

plants #1 and #4, batch mixers are used, usually of fifty foot capacity and with an automatic control for batching by weight. Mixing by this method is accurate and efficient, the mixer is located under the cement and aggregate bins and over the molding machine. Some are capable of traveling on rails to serve more than one molding machine. Plants #1 and #4 use continuous mixers and thus proportion their mixes by volume; the horizontal mixer has a capacity of about ten cubic feet. All plants, except #1 and #4, use approximately the same procedure of mixing, although they differ as to mixing time; the lightweight aggregate is first mixed with most of the water, then the cement and the rest of the water added and mixing continued. This mixing sequence is considered to add to the workability and strength of lightweight aggregate concrete. By mixing first with water the surface cells of the aggregate are filled, preventing cement paste from occupying these spaces and thus keeping the water available for workability and cement hydration. Plants #1 and #4 use the procedure of mixing cement and aggregate dry, then adding the water; the same method is used by plant #3 for sand and gravel products. Molding methods are summarized in column 14 from which it can be seen that all the plants use the vibration method, which is considered advantageous (see Chapter I). Most of the plants use a rather complex, fully automatic machine which molds by vibration and pressure. Usually located under the mixer, the molding machine receives concrete through a hopper and ejects the molded products to the front automatically. It is capable of molding many shapes and sizes of masonry at a speed of about 900 blocks, 8-inch equivalent, per hour. Curing is summarized in column 15 which indicates that all the

plants use low pressure steam (plant #3 uses, in addition, high-pressure steam for its cinder products). The procedures are similar although the cycles are different, depending on the size of the curing chamber and the equipment used by the plant. The green products on the racks are moved to the curing chamber until it is full, which usually allows the product more than an hour of setting. With the chamber closed, live steam is injected until the temperature reaches 170-180 F. The concrete is "soaked" for several hours before opening the chamber. Soaking means the storage of the product in the steam chamber after the steam is turned off, thus permitting gradual cooling in the atmosphere of residual moisture.

The length of the cycles given in the table varies from 12 to 24 hours. Plant #3 uses high-pressure steam curing for cinder products, and employs a cycle of 5 hours. The product is moved from the curing chambers to the storage yards where it normally remains from one to three weeks before it is distributed to the comsumer. From column 16 it is apparent that most of the plants use open storage yards; some cover their product during the winter.

CHAPTER III

AGGREGATES

Properties of the aggregates used by the various plants for producing precast concrete units are covered in this chapter. The study consists of a general discussion of the characteristics of commercially produced lightweight aggregate and discussion of the physical properties of the specific aggregates in this program.

Tests were conducted on samples obtained from the production lines of the plants concerned. The samples obtained and the procedure of sampling and marking were discussed in Chapter II. Upon arrival at the laboratory, these samples were marked and left to dry at room temperature until tested.

A. General Discussion of the Characteristics of Commercially Produced Lightweight Aggregates for Concrete.

Lightweight concrete products in this study are made of three general types of aggregate: cinders, expanded slag, and expanded clay and shale. The general characteristics of these lightweight aggregates and the common methods of their production will be discussed.

1. Cinders.

Cinders suitable for use as concrete aggregate are the inorganic residue from the high temperature combustion of coal or coke and are sometimes called "industrial cinders" as distinguished from the product of small domestic furnaces. The latter product is not usually considered desirable as concrete aggregate. In this study, the word "cinder" will be used to mean the so-called "industrial

TABLE 6. PHYSICAL PROPERTIES OF AGGREGATES

Sample No.	Type of Aggregate	Material Passing (#200 Sieve % by Weight)	Specific Gravity			Absorption		Organic Impurities Colorimetric Test	Popouts
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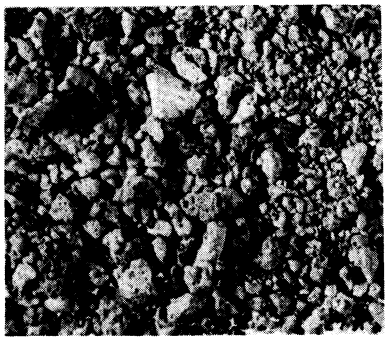
5C9	Cinder Combined	3.67	1.68	1.82	1.95	8.20	7.73	No Change in Color	None
6ES19	Expanded Slag Coarse	0.14	1.47	1.57	1.64	6.75	5.16	No Change in Color	None
6ES29	Expanded Slag Fine	0.59	1.78	1.87	1.96	5.11	5.33	No Change in Color	None
7W19	Waylite Coarse	0.14	1.48	1.59	1.67	7.57	5.81	No Change in Color	None
7W29	Waylite Fine	6.72	1.80	1.89	1.98	5.12	4.66	No Change in Color	None
7B9	Beslite Combined	5.70	1.50	1.71	1.89	14.13	15.2	No Change in Color	None
8CEL19	Celocrete Coarse	0.93	0.97	1.21	1.27	24.3	11.05	No Change in Color	None
8CEL29	Celocrete Fine	8.73	1.49	1.71	1.90	14.41	10.68	No Change in Color	None
10CA9	Cinder Combined	10.7	1.71	1.85	1.99	8.41	8.13	No Change in Color	None

7W19
Waylite, Coarse



7W29
Waylite, Fine

8CEL19
Celocrete, Coarse

8CEL29
Celocrete, Fine

7B9
Beslite, Combined

10CA9
Cinders, Combined

Figure 10. Samples of Aggregates, Continued

cinder" as defined above. Two types of cinders are produced by burning two coals: anthracite and bituminous. These are somewhat different in their properties but both have proven suitable, under favorable conditions, for use as a concrete aggregate.

Physically, cinder particles are porous with an angular shape and uneven faces. Their physical characteristics depend on both the type of coal and the temperature of burning.

Chemically, cinders are composed of silica, alumina, lime and iron with small quantities of magnesium, titanium and alkali compounds. Small amounts of sulphur are generally present in the form of sulphides and sulphates. Carbon from unburned coal or coke is practically always present in cinders due to incomplete combusion.

When used as concrete aggregates, cinders must be processed to a specified grading by crushing and screening. They must be cleaned of deleterious materials. Deleterious materials in cinders are excessive amounts of carbonaceous materials, sulphur compounds, iron compounds, and free lime.

(1) Carbonaceous materials: These are present as a result of incomplete combustion. Bituminous cinders contain carbonaceous materials in the form of coke, while anthracite cinders may contain unburned coal. No relationship has been established between the carbon content of cinder aggregate and the strength of concrete, probably because of the difficulty, in comparative tests, of controlling the consistency of the mix when varying the percentage of dense coal and porous coke. However, the Underwriter's Laboratories found by tests that the combustible content of cinder aggregate should not exceed 35 per cent by weight of dry mixed aggregate.

(2) Sulphur compounds: Unburned coal in cinders may contain pyrites and marcasite (both iron sulphide), or organic sulphur compound; both are deleterious. Sulphur compounds may also be present as sulphide or sulphate in ashes but these are not necessarily harmful if present in small quantities.

(3) Iron compounds: The presence of some iron oxide, such as (Fe_2O_3) or (Fe_3O_4) in cinders is not by itself harmful to concrete. Other iron compounds, such as iron sulphide (pyrite), may cause staining and popouts in concrete. Free metallic iron can, by oxidation, have the same deleterious action in concrete exposed to moisture, but it is usually separated out during the processing of the aggregate.

(4) Free lime: Free lime in cinder aggregate is very deleterious. It is usually formed by the decomposition at high temperature of the calcium carbonate or the mineral gypsum or anhydrite (calcium sulphate) which may be present in coals. Free lime in cinders occurs as small hard-burned particles which hydrate very slowly, accompanied by a large expansion, often causing popouts.

2. Slags.

Slags are formed by many metal producing processes including the iron blast furnace, the open kettle furnace, the Bessemer converter, copper or lead smelters and others. Slag used for concrete is the by-product of the iron blast furnace and the term "slag" is so meant in this study.

Slag (iron blast furnace) is defined by the American Society of Testing Materials as the "non-metallic product consisting of silicate and aluminosilicate of lime bases which developed simultaneously with iron in blast furnaces." The development of the slag is essential for the separation of the metal from the ore.

Cooling of the molten slag is done by three methods. The resulting products are called "air-cooled", "granulated", or "expanded slag" and are produced under many commercial names.

a) Air-cooled Slag. The bulk of the slag is produced by this method, in which molten slag is run to a large pit where it is left to cool slowly in air. Many kinds of pits are used but the most common are the "furnace pit" and the "modified pit". The furnace pit may be 40 ft. wide and 200 ft. long into which the molten slag runs directly from the furnace. Two such pits are usually used, one for receiving the molten material while the hardened slag is being excavated from the other. Modified pits are of several kinds but they are usually long and narrow. Molten slag is dumped on the side of the pit from large ladles moving on rails along the side. Hardened slag may be excavated from one end of the pit while molten slag is dumped in the other end. In both kinds of pits, hardened slag is formed in layers which can be dug out easily with a power shovel. It must be further processed by crushing and screening to the required specification sizes before it is used as concrete aggregate.

Air-cooled slag is characterized by sizable internal cavities, broken surfaces and rough texture. Air-cooled slag is normally not recommended as concrete aggregate if it has a unit weight, compacted dry, of less than 70 pounds per cubic ft. It is therefore, considered as dense aggregate even if it produces concrete lighter in unit weight than does sand and gravel.

b) Granulated Slag. Granulated slag is produced by cooling the molten slag suddenly in water, causing it to expand and form a glassy

lightweight material different in physical properties from the air-cooled slag. Three methods are used in producing granulated slag commercially:

(1) The pit method: Molten slag runs directly into a pool of water.

(2) The jet method: Molten slag is poured slowly on a strong jet of water, expanding suddenly in air and falling in the pool of water to quench further.

(3) The dry method: For this a special machine is used to break the stream of molten slag into small particles before it is introduced into the water. A lesser amount of water is used by this method, and the still hot, granulated slag usually dries rapidly.

c) Expanded Slag. This is also produced by cooling molten slag suddenly. It is different from the granulated slag in that only a limited and predetermined amount of water is used. It is commercially produced by two methods:

(1) The jet method: A high pressure jet of water impinges on a stream of molten slag which expands in air and falls into a dry pit with slight further expansion.

(2) The machine method: Molten slag is broken into particles which expand by coming into contact with a limited amount of water thus generating steam for quenching. Aggregate is produced practically dry by this method and may be processed directly.

Expanded slag is a friable foamed material with a cellular structure containing a large number of vesicular internal cavities. It is, therefore, light in weight and has a rough texture. It is produced under many commercial names, among which are Waylite and Celocrete, both of which are included in this study.

Mineralogically, iron blast furnace slag is crystalline. The formation of the crystals depends on the rate of cooling. Crystallation usually starts at 2640 F and continues until the mass is solidified. Crystals may range in size from submicroscopic to about 1/4 inch long.

Slag is relatively free of deleterious materials. Being formed at a very high temperature, this eliminates organic materials unless it is contaminated later during storage or handling. Under unusual furnace operations, small quantities of metallic iron, coke or fluxing stone might escape from the furnace into the slag. But the metallic iron is separated out if properly processed and the coke does not usually exceed one per cent. However, fluxing stone incompletely consumed by the furnace may be very deleterious and may cause popouts and spalling upon long exposure of concrete to moisture.

3. Expanded Clay, Shale and Slate.

Lightweight aggregate suitable for use in concrete is produced by heating clay, shale or slate by a special method to a temperature of incipient fusion when "bloating" occurs, forming, upon cooling, hard lightweight cellular particles. The basic mineral structure of the clay does not have a large effect on the bloating process. Common clay or shale (kaolinite and/or hydromica) may bloat successfully, but the montmorillonite types of clay may be equally good bloaters. The essential requirement is that they contain a small percentage of potential sources of expansive gases such as carbonates, sulphides, sulphates or carbonaceous materials. Commercially, there are two widely used methods of bloating: the rotary kiln method and the sintering method. They are different methods

but the essential process is the same: The raw material is heated to incipient fusion, between 1000 C and 2000 C, at which time the material should be in a pyrophastic condition with gases evolving throughout the mass. In this state, the gases cannot escape readily; some become entrapped, causing the bloating to occur.

a) Rotary Kiln Method. The raw clay is usually crushed to the size of 1-1/2 inch or larger and fed to slowly rotating kilns in which it is heated. The temperature is allowed to rise slowly until a determined high temperature is reached at which time the temperature is raised suddenly to that of incipient fusion where the bloating occurs. The bloated materials will then solidify by cooling. The hardened material must be processed to a suitable grading before it is used for concrete.

b) Sintering Method. Raw clay, crushed to the required sizes, between 3/4 inch and #16 mesh is blended with five to fifteen per cent coal, coke or charcoal that has been ground to pass #16 mesh. The blend is then fed to a pelletizing machine. Pelletizing is important because it has a considerable effect on the type of bloating experienced; each clay appears to have individual pelletizing characteristics. The sintering process is commercially done by two methods:

(1) The continuous method: The pellets are fed to traveling grates less than one foot deep traveling in a continuous cycle. At the beginning, the grates pass under the ignition hood which is fired by gas or oil to ignite the fuel in the pellets which will then burn by induced draft as the grates pass a series of wind boxes. By this method, the clay pellets are heated rapidly to the softening point

when the bloating occurs. The fuel in the pellets will soon be exhausted, and the bloated material starts to cool to form cakes of hard cellular materials which are ready for processing.

(2) The batch method: This process is essentially the same as that for the continuous method except that the sintering is done by the batch in stationary inclined grates. Each hearth is charged, ignited, bloated, cooled and discharged individually.

Expanded clay, shale or slate is light friable aggregate with visicular structure. When well graded, these aggregates frequently provide strong concrete. It has been found possible to make concrete having compressive strength as high as 7000 psi with some expanded shales. These aggregates are produced commercially under several names, among them are "Haydite", which is produced by the rotary method, and "Beslite", which is produced by the sintering method. Both aggregates are included in this study.

B. Physical Properties of Aggregates.

The physical properties of lightweight aggregate are difficult to test using normal techniques because of their cellular structure, low specific gravity, rough surface and high absorption. Modification of the standard methods of test are sometimes required, and the reproducibility of the results is more difficult than for dense aggregate. Probably because of this, the literature is brief and results are not comparable, due to differences in method or in conditions of test. The ASTM methods of tests for dense, and those suitable for lightweight aggregate, were followed in this work whenever possible. The physical properties studied were the following:

1. Grading

2. Unit weight

3. Specific gravity and absorption

4. Deleterious inclusions

   a) Materials passing #200 sieve
   b) Organic impurities
   c) Popouts

1. Grading.

Grading, maximum size and surface texture of aggregate, including those of lightweight, have important effects on concrete. The workability and consistency of plastic concrete, the amount of cement required to produce a certain strength and surface texture of hardened concrete are all influenced by these properties of the aggregate.

Sieve analysis is a measurement of the grading of the aggregate which in turn is essential for the designing of concrete mixes.

Fineness modulus (F.M.) is a factor used for "identifying" the grading. Defined by the ASTM as an "emperical factor obtained by adding the total percentage of a sample of aggregate retained on each of a specific series of sieves and dividing the sum by 100." The fineness modulus alone does not supply the complete information--the grading of aggregate must be known because two aggregates having the same fineness modulus may differ sharply in their grading.

a) Procedure of Test. Samples of aggregate of suitable size were obtained from the original by using a sample splitter and were dried to constant weight prior to the test. The procedure used in conducting this test is as that described by the ASTM Designation

C136-46 (Method of Test for Sieve Analysis for Fine and Coarse Aggregates).[22] Standard woven wire sieves, conforming to the requirements of the ASTM Specifications were used. The following sizes were included: 3/8 inch, Nos. 4, 8, 16, 30, 50 and 100 sieve. Sieving was done with a mechanical shaker. Shaking time of approximately five minutes for lightweight aggregate was found to result in thorough sieving without breaking the aggregate particles.

b) Discussion of Test Results. Results of the grading test are given in Table 4. Aggregate samples are identified in the first column of the table in accordance with the procedure of marking discussed in the second chapter, while the second column gives the type of aggregate and grade (fine, coarse or combined) as sampled. The ASTM Designation C136-46 (Method of Test for Sieve Analysis for Fine and Coarse Aggregates) divides aggregates into find and coarse on #4 sieve. In the second column, the aggregates are defined as they were used in the plant mix (see also Table 3). Such a division was attempted because it is more useful for a comparative study of the aggregates in the condition used in the plant mixes.

(1) Sieve analysis: Results of the analysis are given in column 3 of Table 4 and are reported as both passing and retained percentages on each of the sieves used. Comparative study of these values is difficult, because the aggregates are of different materials or sources. Suitable grading of a certain aggregate is determined by physical properties and those required for the concrete. Almost all the plants included in this study have their own processing equipment (see Chapter I) and can therefore choose the most suitable grading for their purpose.

TABLE 4. GRADING AND UNIT WEIGHT OF AGGREGATES

1	2		3							4	5	6
			Total Percentage Retained or Passing the Sieve Indicated									
Sample No.	Type of Aggregate		3/8	#4	#8	#16	#30	#50	#100	F.M.	Maximum Size Inch or Sieve No.	Unit Weight Pounds per cu.ft.
1C9	Cinder Combined	Retained	3.9	23.0	56.0	73.3	83.0	88.2	91.5	4.19	3/8"	58.6
		Passing	96.1	77.0	44.0	26.7	17.0	11.8	8.5			
3C9	Cinder Combined	Retained	1.2	24.2	49.9	65.2	75.1	82.3	87.6	3.86	3/8"	57.3
		Passing	98.8	75.8	50.1	34.8	24.9	17.7	12.4			
3SG9	Sand and Gravel Combined	Retained	0.0	15.0	33.9	53.3	74.5	92.3	98.2	3.67	3/8"	111.8
		Passing	100.0	85.0	66.1	46.7	25.5	7.7	1.8			
4SG9	Sand and Gravel Combined	Retained	0.0	14.0	25.8	49.1	65.1	81.3	96.3	3.32	#4 sieve	118.0
		Passing	100.0	86.0	74.2	50.9	34.9	18.7	3.7			
5C9	Cinder Combined	Retained	12.2	46.1	61.0	74.9	81.8	86.7	90.8	4.54	3/8"	58.8
		Passing	87.8	53.9	39.0	25.1	18.2	13.3	9.2			
6ES19	Expanded Slag Coarse	Retained	30.4	97.5	100.0	100.0	100.0	100.0	100.0	6.28	1/2"	47.5
		Passing	69.6	2.5	0.0	0.0	0.0	0.0	0.0			
6ES29	Expanded Slag Fine	Retained	6.9	42.8	56.3	73.1	83.3	91.2	96.5	4.50	3/8"	66.1
		Passing	93.1	57.2	43.7	26.9	16.7	8.8	3.5			
7W19	Waylite Coarse	Retained	25.0	88.5	96.2	97.1	97.1	97.6	98.1	6.00	1/2"	47.9
		Passing	75.0	11.5	3.8	2.9	2.9	2.4	1.9			
7W29	Waylite Fine	Retained	0.0	1.5	15.4	35.3	60.5	79.5	88.7	2.81	3/8"	56.9
		Passing	100.0	98.5	84.6	64.7	39.5	20.5	11.3			
7B9	Beslite Combined	Retained	4.2	34.1	67.4	79.9	85.1	88.2	91.7	4.51	1/2"	66.9
		Passing	95.8	65.9	32.6	20.1	14.9	11.8	8.3			
8CEL19	Celocrete Coarse	Retained	0.0	83.4	90.0	90.7	90.7	91.4	92.1	5.38	3/8"	28.5
		Passing	100.0	16.6	10.0	9.3	9.3	8.6	7.9			
8CEL29	Celocrete Fine	Retained	0.0	4.7	22.5	44.5	60.7	73.8	87.5	2.94	3/8"	46.3
		Passing	100.0	95.3	77.5	55.5	39.3	26.2	12.5			
10CA9	Cinder Combined	Retained	1.7	31.2	53.7	66.8	77.2	84.3	89.3	4.04	3/8"	60.3
		Passing	98.3	68.8	46.3	33.2	22.8	15.7	10.7			

Workability of plastic concrete is an important factor influenced by the grading of aggregate. Research has generally indicated that to obtain a given workability, lightweight aggregate requires a larger percentage of the material passing #4 sieve than dense aggregate. This is probably because the lightweight aggregate contains a higher percentage of voids. From column 3, it can be seen that such is not the case for the aggregates under test. Some plants (see Table 3) used air entraining admixtures or add a certain percentage of sand to their mix. It should also be noted that the workability is not such an important problem when molding by vibration and pressure; the addition of fine material to the mix may, therefore, be to improve the surface texture of the concrete rather than to provide workability.

TABLE 5.* UNIT WEIGHT REQUIREMENTS OF LIGHTWEIGHT AGGREGATE FOR CONCRETE MASONRY UNITS

Size Designation	Dry loose weight, Max. lb. per cubic ft.
Fine aggregate	70
Coarse aggregate	55
Combined fine and coarse aggregate	65

Limitations on the grading of lightweight aggregate have been tentatively established by the ASTM Designation C331-53T (Specifications for Lightweight Aggregate for Concrete Masonry Units). The grading of the aggregates in Table 4, except for 6ES29 (Expanded

* ASTM Desig. (C331-53T - Tentative Specification for Lightweight Aggregates for Concrete Masonry Units). [22]

Slag, fine) and 8CEL19 (Celocrete, coarse) are found to be in agreement with these limits. Fine expanded slag is found to be coarser and coarse Celocrete found to be finer than these limits; but it should be noted that the same designation permits waiving the grading requirements to acquire special characteristics for the concrete, such as texture or weight, etc.

(2) Maximum size: The maximum size of the aggregates are given in column 5 of Table 4 which were calculated by permitting a tolerance of 15 per cent; thus if 15 per cent or less of the aggregate is retained on a certain sieve, the sieve opening will be considered as the maximum size for the aggregate, but if more than 15 per cent of the aggregate is retained, the next largest size will be considered as the maximum size of the aggregate. The maximum size of the aggregate as given in column 5 are 3/8 inch for fine and 1/2 inch for coarse aggregate. Coarse sizes would be unsuitable for producing hollow precast concrete units which have thin sections.

(3) Fineness Modulus (F.M.): The F.M. of the aggregates differ considerably as can be seen from column 4 of the table. Because of the maximum sizes, the F.M. of the coarse aggregates are comparatively low. Combined aggregates - lightweight - have a higher F.M. than dense aggregates, with sample 5C9 having the highest value, which might explain the addition of the natural sand to the plant mix.

2. Unit Weight.

Expressed in pounds per cubic foot, the unit weight of the aggregate provides useful information. It is used in calculating the percentage of voids in the aggregate which enters into the calculations of concrete mixes or in predicting the weight of the concrete. For

air-cooled slag and lightweight aggregates, it is used to indicate the quality of the aggregate, which is covered by specifications.

a) Procedure of Test. The unit weights of dry compacted aggregates were determined by following the procedure described by the ASTM Designation C29-42 (Method of Test for Unit Weight of Aggregate).[22] Samples were dried to constant weight and compacted by the jigging method for lightweight aggregate and the rodding method for dense aggregate in a steel container of 0.10 cubic ft. capacity.

b) Discussion of Unit Weight Test Results. Results of the test are given in Table 4 which represents the average of five determinations.

The unit weight of the dry aggregate depends on the bulk specific gravity and the volume of external voids it contains. The volume of external voids depends on the grading of the aggregate and the shape and size of the particles. It is observed from Table 4 that Beslite (7B9, combined) is the heaviest of the lightweight aggregates, having a unit weight of 66.9 pcf compared to 118.0 pcf for sand and gravel (4SG9, combined). Celocrete is the lightest, having a unit weight of 28.5 pcf for coarse aggregate (8CEL19) and 46.3 pcf for fine aggregate (8CEL29). Unit weights of all these aggregates are sufficiently low so as to conform to the requirements of ASTM Designation C331-53T (Tentative Specifications for Lightweight Aggregates for Concrete Masonry Units).

3. Specific Gravity and Absorption.

Knowledge of the specific gravity of the aggregate is important for the calculation of its solid volume contribution in concrete mixes. It is sometimes used in the comparative study of the weight of lightweight aggregate independently of the gradation. For

some aggregates it is also used for judging their quality. The "bulk specific gravity" (B.S.G.) and the "apparent specific gravity" (A.S.G.) are most commonly used in these calculations. There is some confusion in the literature as to the meaning of the different terms used for the specific gravity, but in this study they bear the meaning contained in the following definitions of the ASTM Designation E 12-27 (Definition of Terms Relating to Specific Gravity):[22]

> "Apparent Specific Gravity (of solids). The ratio of the weight in air of a given volume of the impermeable portion of a permeable material (that is, the solid matter including its impermeable pores or voids) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature...."

> "Bulk Specific Gravity (of solids). The ratio of the weight in air of a given volume of a permeable material (including both permeable and impermeable voids normal to the materials) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature...."

Absorption of the aggregate and natural moisture content aid determination of the amount of water to be added to the mix to obtain the required water-cement ratio. Certain physical characteristics of the concrete were studied in terms of the absorption of the aggregate. It has been found that the absorption of concrete and, therefore, its shrinkage is affected by the absorption of the aggregate. An attempt has also been made to correlate the durability of the concrete with the absorption, porosity characteristic and the specific gravity of the aggregate.

Different methods of tests were used for the fine and coarse aggregate in determining the specific gravity and absorption.

a) Coarse Aggregate. Four different coarse aggregates, all lightweight, were tested by the standard procedure of the ASTM

Designation C127-42 (Method of Test for Specific Gravity and Absorption of Coarse Aggregate),[22] with slight necessary modifications. Because of the relative lightness of the aggregate tested, a smaller sample (by weight) was used, and since the maximum size of the aggregate is 1/2 inch, it was decided to reject the part of the sample passing #4 sieve instead of 3/8 inch.

Results of the tests are given in Table 6 and will be discussed later (in part c). Detailed data and equations used in the calculations of the results are presented in the appendix.

b) Fine Aggregate. Six combined and three different fine aggregates also were tested by this method. Combined aggregates were considered as fine aggregates because they consist of mostly fine materials (passing #4 sieve).

In preparing test samples for the dense aggregates, the procedure specified by the ASTM Designation C128-42 (Methods of Test for Specific Gravity and Absorption of Fine Aggregate)[22] was followed. However, modification of this procedure was necessary to determine the saturated surface dry condition of the test samples of the lightweight aggregates. This will be discussed in part c. A procedure different than that specified above was used in conducting the test.

The Le Chatelier Flask was used for determining specific gravity. It has a bulb in the bottom which has a capacity of approximately 250 cc and is equipped with a long graduated neck of 24 cc capacity. A clean flask was filled with distilled water to a point reading between 0.0 to 1.0 cc, stoppered and left standing in a bath of water at 20 C while preparing the aggregate sample. Upon reaching the saturated surface dry condition, the sample was mixed

thoroughly and divided into two portions, the small portion (35 to 50 gms.) weighed and immediately introduced inside the flask while the rest of the sample, used for determining the absorption, was weighed and placed inside an oven to dry to constant weight at 210 F. Just before introducing the sample to the flask, the reading of the water was taken and recorded as the "initial reading". Rolling the stoppered flask on its side was found to help remove the air bubbles from the water; after this, the flask was placed again in the water bath which was constantly maintained at 20 C for one hour before taking the "final reading". The volume of the aggregate sample inside the flask is, of course, equal to the difference between the initial reading and the final reading. When the aggregate sample (of the absorption test) inside the oven reached a constant weight, it was taken outside to cool to room temperature and weighed. The difference in weight between the saturated surface dry condition and the oven dry condition of the sample is, of course, the weight of the water of absorption.

Results of the test are given in Table 6 and will be discussed later (in part c).

Detailed data and the equations used in the calculation of the results are presented in the appendix.

c) Discussion of Test Results for Coarse and Fine Aggregates. It is difficult to determine the saturated surface dry condition of light-weight aggregate. At this condition the aggregate must contain no free surface moisture and must absorb no water; but because of the rough surface texture and the large number of open surface voids, the drying of these surfaces without drawing out water of absorption, is

a careful and time consuming process, especially for fine aggregates. The "cone method", which was specified by the ASTM Designation C128-42 (Method of Test for Specific Gravity and Absorption of Fine Aggregate) for determining the saturated surface dry condition of fine dense aggregate was found to give inconsistent results when applied to lightweight aggregate. This is, probably, because the rough and angular shapes cause them to interlock when compacted inside the cone by this method, providing a mechanical bond causing the cone of the aggregate to retain its shape in absence of surface moisture. Several other methods were therefore tried and it was decided to establish a procedure actually including three methods simultaneously, as a criterion of the surface dry condition: (1) using a modified cone method in which the aggregate was tamped inside the cone with a wooden rod 15 times only, (2) determining the point at which the aggregate will run loose through the fingers and (3) by noticing the change in color of the aggregate when it loses its surface sheen when dried.

Results given in Table 6 for the specific gravity and absorption, which are the average of two determinations, will be discussed separately. Calculations are given in Tables 31, 32 and 33 in the appendix.

(1) Specific Gravity: No great difficulty was met in determining the specific gravity and the absorption of saturated surface dry samples of coarse aggregates. The Le Chatelier method proved difficult to apply for some lightweight aggregates of low specific gravity. These aggregates have a few particles that have a specific gravity close to 1.0 which will float. The number of

these particles is so small in comparison with the test sample that it should not affect the results but does make the final reading difficult.

Results are given for the bulk specific gravity and the apparent specific gravity of the aggregates; the bulk specific gravity is calculated on both the saturated surface dry basis and the oven dry basis. Any one set of these results, such as those of the bulk specific gravity oven dry basis, should be adequate for comparison. Thus from Table 6 it is observed that the latter gravity for cinders, except for one sample, range from only 1.71 to 1.74, while expanded slag, Waylite and Beslite have close results ranging from 1.47 to 1.5 for coarse aggregate and from 1.78 to 1.80 for fine aggregate. Celocrete has the lowest specific gravity, 0.97 for coarse and 1.49 for fine. Dense aggregate gave relatively high results of 2.63 and 2.66 for combined aggregates. It is also to be noticed that, generally, fine aggregates have a higher specific gravity than coarse aggregates which may be explained by the fact that coarse aggregates contain a higher percentage of internal voids.

(2) Absorption: No difficulty was experienced in determining the absorption of lightweight aggregate after establishing a criterion for the saturated surface dry condition of the sample. It should be noted that the results given in Table 6 represent partial saturation due to 24 hours immersion in water at room temperature. The result of such a test is used in concrete mix calculations because only partial saturation is considered to occur inside the mixer.

The absorption is reported in Table 6 in both percentages of dry weight and dry compacted volume. Absorption as percentage of

TABLE 6. PHYSICAL PROPERTIES OF AGGREGATES

Sample No.	Type of Aggregate	Material Passing (#200 Sieve % by Weight)	Specific Gravity			Absorption		Organic Impurities Colorimetric Test	Popouts
			B.S.G Oven-Dry Basis	B.S.G. Saturated Surface Dry Basis	A.S.G.	% of Dry Weight	% by Volume (Dry Compacted)		
1C9	Cinder Combined	7.3	1.71	1.85	1.99	8.40	7.87	No Change in Color	None
3C9	Cinder Combined	5.6	1.74	1.90	2.06	9.00	8.26	No Change in Color	None
3SG9	Sand and Gravel Combined	2.23	2.63	2.66	2.73	1.50	2.50	Slight Change in Color	None
4SG9	Sand and Gravel Combined	3.15	2.66	2.69	2.75	1.30	2.47	Very Slight Change in Color	None
5C9	Cinder Combined	3.67	1.68	1.82	1.95	8.20	7.73	No Change in Color	None
6ES19	Expanded Slag Coarse	0.14	1.47	1.57	1.64	6.75	5.16	No Change in Color	None
6ES29	Expanded Slag Fine	0.59	1.78	1.87	1.96	5.11	5.33	No Change in Color	None
7W19	Waylite Coarse	0.14	1.48	1.59	1.67	7.57	5.81	No Change in Color	None
7W29	Waylite Fine	6.72	1.80	1.89	1.98	5.12	4.66	No Change in Color	None
7B9	Beslite Combined	5.70	1.50	1.71	1.89	14.13	15.2	No Change in Color	None
8CEL19	Celocrete Coarse	0.93	0.97	1.21	1.27	24.3	11.05	No Change in Color	None
8CEL29	Celocrete Fine	8.73	1.49	1.71	1.90	14.41	10.68	No Change in Color	None
10CA9	Cinder Combined	10.7	1.71	1.85	1.99	8.41	8.13	No Change in Color	None

dry weight is the most commonly used in the calculation but it is not significant in a comparative study of aggregates with widely varied unit weights as in the case of lightweight aggregate. Considering, therefore, values based on dry compacted volume, it is observed in Table 6 that lightweight aggregates, as expected, have a higher absorption than dense aggregates, and the lightweight coarse particles absorb more water than the fine particles. Waylite, fine (7W29), has the lowest absorption for lightweight aggregate of 4.66 per cent, while dense aggregate has the value of 2.50 per cent and 2.47 per cent for (3SG9) and (4SG9) respectively. Cinders have a high absorption value, ranging between 7.73 per cent and 8.26 per cent; Waylite and Expanded Slag have lower values while Celocrete and Beslite have a higher value than cinders. In fact, Celocrete has the highest absorption, approximately six times that of dense aggregates. The number of tests conducted are too limited to draw any conclusions as to the relationship between the specific gravity and absorption; however, others found that absorption is not necessarily related to the specific gravity nor to the total pore space; it has been found possible to produce lightweight aggregate of low specific gravity and absorption due either to an impervious shell or to a visicular structure with a discontinuous pore space.

4. Deleterious Inclusions.

Although lightweight aggregates are relatively free of deleterious materials such as organic impurities, they may contain other materials that are injurious to concrete, such as potential staining or popout materials. Three types of extraneous inclusions, all potentially deleterious, will be studied separately: (a) materials

finer than #200 sieve, (b) organic impurities, and (c) materials causing popouts of concrete.

a) Materials Finer Than #200 Sieve. Excessive amounts of this very fine material are deleterious to concrete. Two types of such material may occur, both present in the form of surface coating to aggregate particles. These are:

(1) That resulting from dust accumulation which might be produced by crushing the aggregate. This does not adhere firmly to the surface of the particles and can be removed by washing. Inside the mixer this will increase the amount of fines and require more water for workability. An excessive amount of this material is therefore deleterious.

(2) Materials that are clayey in nature. These materials are very deleterious because when wet they will adhere to the surface of the aggregate particles up to the time they are imbedded in concrete, thus preventing proper bond with the cement.

The purpose of this test is, therefore, to determine the amount of such materials in the aggregate. The procedure used for this test is the same as that described by the ASTM Designation C117-49 (Method of Test for Amount of Material Finer than No. 200 Sieve in Aggregates).[22] Results are reported in Table 6 as percentage (by weight) of the total aggregate; each result is the average of three determinations. Care was employed to prevent the decantation of coarse particles of some lightweight aggregates that have low specific gravity. In general the results indicate that aggregates are free of excessive amounts of such materials. Dense aggregates showed less than the maximum allowed by the ASTM. There are presently no

limitations specified by the ASTM for the amount of these materials in lightweight aggregates. In this test, the coarse aggregate showed a small percentage of materials finer than #200 sieve, but combined and fine aggregate showed a higher percentage than coarse aggregate alone. Cinders have a widely varying result ranging from 3.7 per cent for (5C9) to 10.7 per cent for (10CA9), with the latter showing the highest value obtained. Of the fine aggregates, Celocrete showed a high result of 8.7 per cent.

b) Organic Impurities. This test provides a quick method for determining the presence of deleterious organic materials in aggregates. The amount of such materials and effect on concrete are to be determined by further tests. (See ASTM Designation C87-52 - Method of Test for Measuring Mortar-Making Properties of Fine Aggregate, and Designation C123-44 - Method of Test for Coal and Lignite in Sand).

Slag and expanded clay are relatively free of organic impurities because of the very high temperature at which slag is formed or at which expanded clay is bloated. Cinders are usually free of such materials unless they contain coal or lignite due to poor firing of the furnace. But it should also be kept in mind that aggregate originally free of such materials may be contaminated during storage or handling. Therefore, such a test is necessary.

The procedure described by the ASTM Designation C40-48 (Method of Test for Organic Impurities in Sands for Concrete)[22] was used on aggregate samples weighing less than one pound and which was obtained from the original samples by a sample splitter.

Table 6 contains the results of this test. These results indicate that none of the aggregates have important amounts of

organic impurities. Lightweight aggregate caused no change in color in the sodium hydroxide solution after 24 hours of soaking; one dense aggregate sample (4SG9) caused a slight change in color while the second one sample (3CSG9) showed a little darker but much lighter than the reference color which was also prepared to the requirements of the ASTM Designation mentioned above.

These results indicate that further testing for organic impurities in the aggregate and the effect in concrete is not necessary.

c) Materials Causing Popouts of Concrete. Popouts in concrete masonry units are a very undesirable characteristic which is the result of a disruptive expansion of considerable magnitude usually generated beneath the surface. It is caused by the presence of certain deleterious materials in the concrete. In cinders, popouts are usually caused by iron compounds, freelime or organic sulphur; in slag they are caused by incompletely fused fragments of flux stones which may be discharged with the slag during unusual operation of the furnace; in expanded clay, raw clay lumps might cause popouts of concrete.

The test was conducted on concrete specimens made in the laboratory with the aggregates under test and a constant slump and cement-aggregate ratio. This was in accordance with the ASTM Designation C331-53T (Tentative Specifications for Lightweight Aggregate for Concrete Masonry Units).[22] Specimens were then autoclaved for three hours using the procedure ASTM Designation C151-52 (Method of Test for Autoclave Expansion of Portland Cement).[22]

Two concrete specimens, 2 x 2 x 11 inches were cast from each aggregate sample. Data on concrete mixes are shown in Table 34

in the appendix. Proportioning was done by dry volume, but all measurements were done by dry weight. A constant proportion of 1:6 was used for all mixes, that is, one part of cement to six parts of combined aggregates by volume. Aggregates were combined in the laboratory in the same proportions as used by the producers in their plant mix unless they were sampled from materials that had already been combined by the producers. Regular cement was used without an air-entraining agent.

Mixing was done by hand which was more convenient for the small concrete mixes. Trial batches were made to determine the amount of water to be used with the mix to produce a constant slump of approximately 2.7 to 2.9 inch. Workability was poor for lightweight aggregate at such consistency and without the use of an air-entraining agent, but it was found possible to improve the workability by mixing the aggregate and approximately one-half of the mixing water first, then adding the cement and the rest of the water and continuing mixing.

Moist curing was employed at normal pressure and a temperature of 80 F for 24 hours after which the specimens were taken outside the chamber, weighed and autoclaved immediately.

Table 6 contains the results of this test which indicates that none of the concrete tested showed any popouts (or staining) on the surface. This fact was determined by visual inspection of the specimen and by reweighing to determine any loss of weight which might result from autoclaving. It may, therefore, be concluded that the aggregates under test are free from deleterious material capable of causing popouts in concrete. See Chapter X, however, for results of staining tests on full size commercially manufactured units.

Figure 11. Concrete Specimens After Staining and Popouts Test.

CHAPTER IV

DIMENSIONS OF THE MODULAR UNIT

The term "modular unit" is used here to mean a hollow precast concrete masonry unit having the nominal dimensions of 8 x 8 x 16 inches. The actual overall dimensions of this unit are specified as 7-5/8 x 7-5/8 x 15-5/8 inches, thus allowing 3/8 inch for a standard mortar joint.

Although a large number of different shapes and sizes of precast concrete units are available, the bulk of the product is presently being made from these modular units. Substantial economy is the reason for this modular coordination; it requires the dimensional standardization of building materials and equipment on the basis of a certain module and the fitting of these materials into a building planned and dimensioned on this modular basis. The "American Standard Module" is equal to 4 inches; nominal dimension of the modular unit is a multiple of this. Flexibility in building dimensions equivalent to this module is accomplished by using supplementary units or cutting the modular unit.

In this chapter study will be made of the dimensional properties of the modular unit including its actual overall dimensions, gross cross-sectional area, net volume and their interrelation.

A. Definitions.

(1) Gross cross-sectional area: ASTM Designation C140-52[22] defines the gross cross-sectional area of the modular unit as the "total area of a section perpendicular to the direction of the load, including areas within cells and within re-entrant spaces unless these

spaces are to be occupied in the masonry by portions of the adjacent masonry."

(2) Gross volume: The gross volume of the modular unit is the total volume included within its overall dimension.

(3) Net cross-sectional area: The net cross-sectional area of the modular unit is the average of the areas of concrete present in sections perpendicular to the direction of load.

(4) Net volume: The net volume of the modular unit is the total volume of concrete in the unit.

(5) Hollow masonry unit: The A. S. A. defines the hollow masonry unit as "a masonry unit whose net cross-sectional area in any plane parallel to the bearing surface is less than 75 per cent of its gross cross-sectional area measured in the same plane."[29]

B. Procedure of Test.

The overall dimensions of the unit were measured directly by a caliper and a scale which reads to 0.01 inch. Five specimens were tested for each type of product; the length, depth and thickness were separately measured at the two faces of the specimen and averages recorded.

A different procedure (using the same specimens) was used for determining the net volume (ASTM Designation C140-52 - Methods of Sampling and Testing Concrete Masonry Units[22]). Each specimen was saturated with water for 24 hours, then weighed suspended in water, removed from the water and left to drain for one minute over a 3/4 inch wire mesh; visible surface moisture was then removed and the specimen immediately weighed in air. The net volume of the specimen is, in the metric system, numerically equal to the difference between the

weight of the saturated specimen in air and its weight suspended in water. Further details on this procedure will be presented in Chapter V.

C. Discussion of Test Results.

Dimensional properties of the modular units are reported in Table 7. Only average results (of the 5 specimens) are given in this table, while detailed data and calculations are presented in Tables 35, 36 and 37 in Appendix II. The industry's adoption of the system of modular coordination of dimension is evident in these tables. In columns 3, 4, and 5 of Table 7, the average overall dimensions (height, thickness and length) of the modular units are presented. These results correspond to standard dimensions of 7.625 x 7.625 x 15.625 inches. Specifications (see references 28 and 29) permits a maximum deviation in the dimension of the individual unit of $\pm$ 0.125 inch; only three types of the products tested (see Table 36 in Appendix II) showed units with deviations exceeding this. These products are 1Cl, 3Cl, and 8CEL1; of these, 8CEL1 has the largest deviation, +0.225 inch. It is also noticed that no single specimen of a given product showed deviation from the average exceeding that permitted.

Columns 6 and 8 of Table 7 present the average gross volume and gross area of the units: both are useful in estimating, for example, total requirements for a building. Column 10 presents the average net areas as percentage of the gross areas: these values fall between 50 to 60 per cent which indicates all are within the definition of hollow masonry units (see definition on page 63); data for calculations of these percentages are given in Table 37 in the appendix.

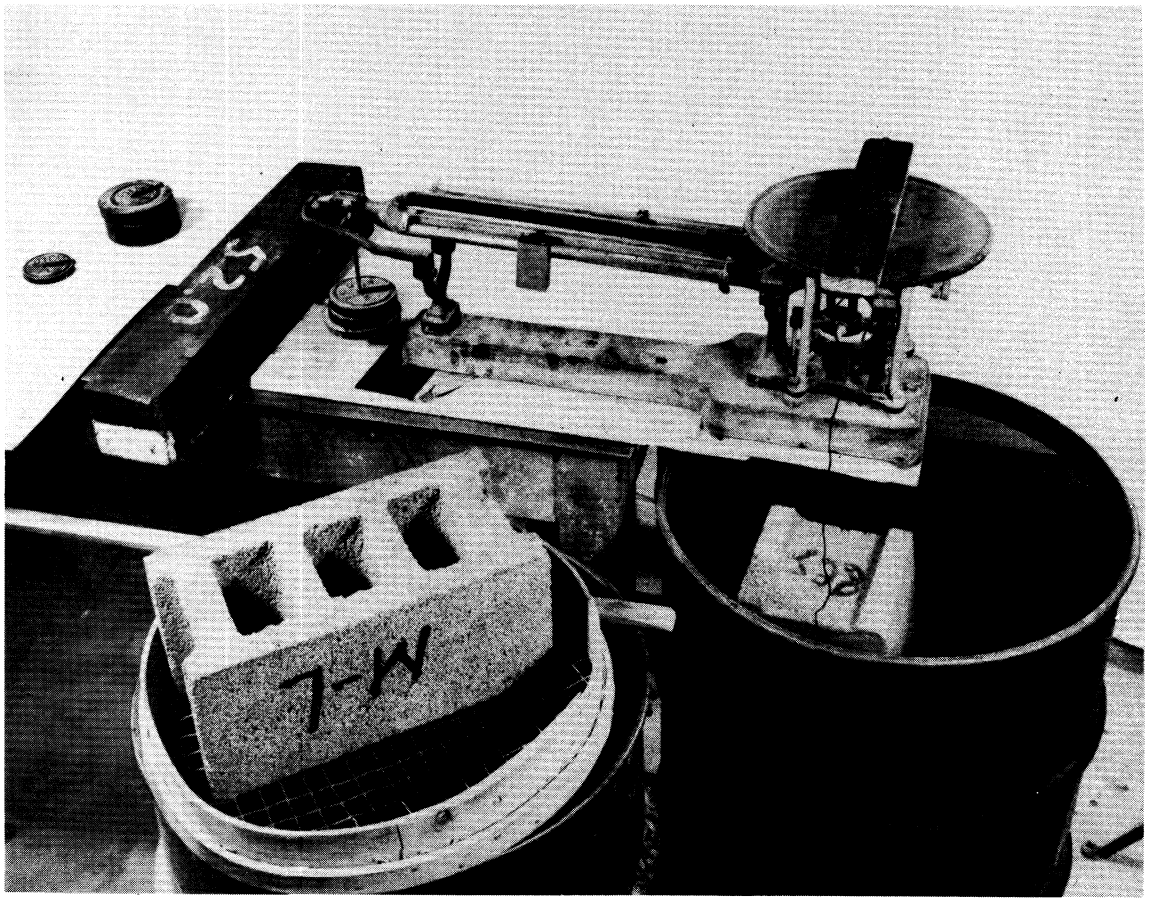


Figure 12. Setup for the Determination of Net Volume of the Unit.

TABLE 7.* DIMENSIONAL PROPERTIES OF THE MODULAR UNITS (AVERAGE VALUES)

1	2	3	4	5	6	7	8	9	10
		Gross Dimensions							
Sample No.	Type of Aggregate	Thick-ness t in.	Length L in.	Height h in.	Gross Volume (t·L·h) in^3	Net Volume in^3	Gross Area (t·L) in^2	Net Area in^2	Net Area as % of Gross Area
1C1	Cinders	7.70	15.68	7.74	935.6	477.2	120.7	61.7	51
2CES1	Cinders & Expanded Slag	7.67	15.63	7.67	918.4	517.3	119.8	67.5	56
2H1	Haydite	7.68	15.65	7.67	922.5	515.2	120.3	67.2	55
3C1	Cinders	7.69	15.74	7.63	924.1	554.3	121.1	72.6	60
3SG1	Sand & Gravel	7.67	15.64	7.62	914.5	502.3	120.0	65.9	55
4SG1	Sand & Gravel	7.72	15.74	7.71	937.6	469.4	121.6	60.9	50
5CS1	Cinders & Sand	7.70	15.61	7.67	921.7	464.5	120.2	60.4	50
6ES1	Expanded Slag	7.70	15.65	7.70	927.7	459.0	120.4	59.5	50
7W1	Waylite	7.68	15.63	7.67	921.3	474.4	120.1	61.8	51
7BW1	Beslite & Waylite	7.63	15.65	7.65	913.9	470.6	119.5	61.5	51
8CEL1	Celocrete	7.77	15.72	7.73	944.4	507.0	122.1	65.5	54
9S1	Air-Cooled Slag	7.64	15.61	7.62	908.3	512.6	119.3	67.3	56
10CA1	Cinders & Fly Ash	7.71	15.67	7.67	925.7	503.1	120.7	65.6	54

* Notes:

1. Values are average of 5 specimens.
2. Net Volume - see Table 35 in the appendix.
3. Net Area = $\frac{\text{Net Vol.}}{h}$ (See Tables 35 and 37 in Appendix II.)
4. Net Area as per cent of Gross Area = $\frac{\text{Net Area}}{\text{Gross Area}} \times 100$ (Table 37).

CHAPTER V

UNIT WEIGHT, MOISTURE CONTENT AND ABSORPTION

The weight of the masonry units, as sampled, and in the wet and dry conditions, moisture contents and absorption are considered in this chapter.

Sampled weight of the masonry unit is useful for calculating its moisture content at the time of sampling and for estimating its dead load on structural members. Wet weight is the weight in air of the units after 24 hours of submersion in water, used in calculating the absorption. Dry weight is the weight of the unit after drying in an oven to an essentially constant weight.

Unit weight of the concrete (in pounds per cubic foot) was determined under three conditions (1) as sampled, (2) wet, and (3) dry. These unit weights were calculated using the measured weight of the unit under these conditions and the net volumes of the individual units (presented in Chapter IV). Unit weight (dry) is probably the most convenient basis for the correlation of the physical properties of the various concretes.

The moisture content of the unit when sampled (sometimes called "natural moisture content") is usually expressed as percentage of 24 hour absorption. Specifications frequently limit the permissible natural moisture content to prevent excessive shrinkage of concrete masonry walls.

Drying shrinkage of concrete is also found to be correlated with absorption of the aggregates with which it is made.[19] It is more pronounced in those having high absorption. Resistance of concrete

to the action of freezing and thawing and the attack of silage acids is found to be correlated with the rate of absorption.[12]

A. Procedure for Determining Unit Weight, Moisture Content and Absorption.

Five full size units were used for each product. They were selected at random upon bringing the samples to the laboratory, marked and weighed in air. This weight, called here the "sampled weight", was determined for the specimen within less than 12 hours after sampling at the plant.

In determining the moisture content and absorption of the units, the procedure of the ASTM Designation C140-52 (Methods of Sampling and Testing of Concrete Masonry Units)[22] was followed: Specimens were submerged in water at room temperature (60 to 80 F) for 24 hours. They were then weighed suspended in water, taken from the water, allowed to drain for one minute and weighed in air (see Chapter IV, part B). This latter weight is called "wet weight". Specimens were then dried in a forced ventilation oven at 210 F-230 F for more than 24 hours. Drying was discontinued when two successive weighings at 2 hour intervals showed a loss in weight less than 2 per cent of the last previously determined weight of the specimen. This last weight is called the "dry weight".

B. Discussion of Test Results.

1. Unit Weight.

Unit weight of the specimens (as sampled, wet and dry) are given in Table 8. These are calculated from the weights of the units in the three foregoing conditions (also given in Table 8) and the unit volume as first reported in Chapter IV. Each value in Table 8

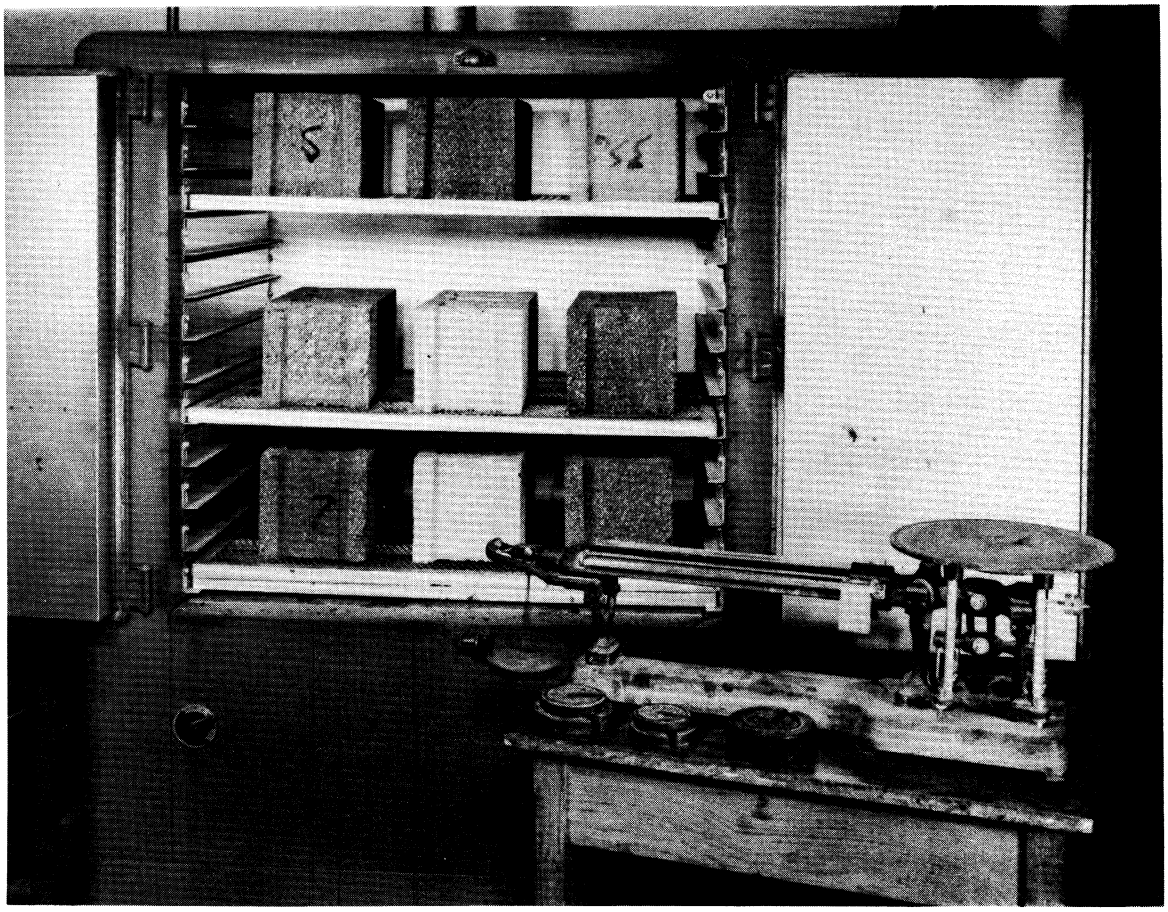


Figure 13. Electrical Automatic Oven Used for Drying Masonry Units.

TABLE 8. WEIGHTS OF THE MASONRY UNITS AND THE UNIT WEIGHTS

1	2	3	4	5	6	7	8
	Weight of the Masonry Unit (Pounds)			Net Volume	Unit Weight (based on net volume) (pcf)		
Sample No.	As Sampled	Wet	Dry	of the Unit cu.ft.	As Sampled	Wet	Dry
1C1	26.8	29.3	25.1	.276	97.1	106.1	90.9
2CES1	29.7	32.1	27.9	.299	99.1	106.7	93.1
2H1	27.8	29.4	25.1	.298	93.1	98.7	84.2
3C1	27.7	32.1	26.7	.321	88.5	100.1	83.4
3SG1	39.6	41.1	38.2	.291	136.3	141.2	132.8
4SG1	36.4	38.3	35.5	.272	134.1	140.8	129.2
5CS1	27.3	29.3	26.1	.269	101.5	108.9	97.1
6ES1	28.8	30.9	26.9	.266	108.4	116.3	101.3
7W1	27.3	29.7	26.7	.275	99.3	108.3	97.3
7BW1	28.7	31.4	27.8	.272	105.3	114.1	102.2
8CEL1	24.6	26.8	22.6	.296	83.0	90.3	76.2
9S1	38.3	40.8	37.5	.297	129.2	137.7	126.3
10CA1	26.4	28.9	24.8	.291	90.7	99.0	85.2

is the average of the results of the five specimens tested from each product. Detailed data and calculations of the results are given in Tables 38 and 39 in Appendix III.

The unit weight in dry condition is the most convenient form for a comparative study. Returning to column 8 of Table 8, it can be seen that the weights of the products vary, ranging between a minimum of 76.2 pcf (pounds per cubic ft.) for 8CEL1 (Celocrete) to a maximum of 132.8 pcf for 3SG1 (sand and gravel). The other sand and gravel product (4SG1) had a similar weight of 129.3 pcf. Air-cooled slag (9S1) averaged 126.3 pcf. Concrete made with such aggregate is usually classified as dense. The maximum weight of the products made with strictly lightweight aggregate is 97.3 pcf for 7W1 (Waylite).

Rough correlation exists between the unit weight of dry aggregate and concrete. This can be seen by comparing the values in Tables 4 (of Chapter III) and 8. Good correlation is not to be expected because of the difference in plant mixes and the fact that some plants add air-entraining agents to their mixes.

2. Moisture Contents and Absorption.

Moisture contents and absorption of the masonry units are presented in Table 9. Each value is the average of the results of five specimens tested for each product. Detailed data and calculations are given in Appendix III (Tables 40 and 41).

a) Absorption. Absorption is presented in two forms: as pounds of water absorbed per cubic foot of concrete (pcf) and as percentage of dry weight of the unit. The latter form, although it is a common and convenient method of expressing the absorption, is not very useful for a comparative study of aggregates of widely varying

specific gravity, as in these tests. The prior method of reporting the absorption (as pcf) is based on the net volume of the unit.

Concrete made with lightweight aggregate generally absorbs more water than does that made with dense aggregate (see column 4 of Table 9). All-cinder products gave high absorptions with 3Cl producing the maximum of 16.8 pcf followed closely by 1Cl, also cinder, averaging 15.1 pcf. Cinders mixed with sand (5CSl) or fly ash (10CAl) produced lower results of 12.3 and 13.9 pcf, respectively. Other lightweight aggregates showed varied results, ranging between 14.9 pcf for 6ESl (expanded slag) and 11.0 pcf for 7Wl (Waylite). Dense aggregate gave absorptions of 8.5 pcf for 3SGl (minimum value) and 10.3 pcf for 4SGl (both are sand and gravel), while air-cooled slag had a higher absorption of 12.7 pcf.

The ASTM Designation C90-52 (Specifications for Hollow Load-Bearing Concrete Masonry Units)[22] specifies a maximum permissible absorption of 15 pcf. Only two of the products under test had absorptions exceeding this maximum, both are cinder products, namely 1Cl and 3Cl.

There is very slight correlation, if any, between the concrete absorption and the unit dry weight or between the absorption of aggregates and concretes. Research has indicated that type of aggregate, mix proportion, use of admixtures and, probably, the method of curing all affect the absorption of the units.

b) Moisture content. Moisture contents, as sampled, of the masonry units are presented in Table 9, column 7, as percentages of the absorption. Results varied, ranging between a low 19.4 per cent for 3Cl (cinders) and a high 62.6 for 2Hl (Haydite). The ASTM

TABLE 9. MOISTURE CONTENT AND ABSORPTION

1	2	3	4	5	6	7	8
Sample No.	Type of Aggregate	Water Absorbed gms.	Absorption pcf	Absorption % of Dry-Weight	Moisture Content As Sampled gms.	Moisture Content As Sampled % of Absorption	Month of Sampling
1C1	Cinders	1896	15.1	16.6	795	41.9	May
2CES1	Cinders & Expanded Slag	1900	14.0	15.0	844	43.7	May
2H1	Haydite	1972	14.1	17.3	1236	62.6	May
3C1	Cinders	2441	16.8	20.2	476	19.4	May
3SG1	Sand & Gravel	1110	8.5	6.4	503	45.4	May
4SG1	Sand & Gravel	1264	10.3	7.9	467	37.5	May
5CS1	Cinders & Sand	1503	12.3	12.7	557	37.2	May
6ES1	Expanded Slag	1797	14.9	14.7	874	48.6	May
7W1	Waylite	1369	11.0	11.3	266	19.7	June
7BW1	Waylite & Beslite	1602	13.0	12.7	408	25.5	June
8CEL1	Celocrete	1895	14.0	18.8	934	49.7	June
9S1	Air-Cooled Slag	1529	12.7	9.0	424	27.7	June
10CA1	Cinders & Fly Ash	1833	13.9	16.3	727	39.7	October

Designation C90-52 specifies a maximum moisture content of 40 per cent. Table 9 shows six different products having moisture contents exceeding that value. It will also be noted that in Table 3 (Chapter II) that

almost all the products are normally stored in open yards and, therefore, their moisture content varies considerably from time to time, being higher in damp seasons. Column 8 indicates that most of the sampling was done during the normally moist months, May and June.

It may also be repeated here (see Chapter I, page 22) that some investigators actually consider the usual specification requirement of the maximum moisture content (40 per cent) as being too lenient in view of anticipated later shrinkage. But it may also be pointed out that these investigations are based on gradual drying in air rather than the rapid drying in oven which was used in this test. Other research also indicated that rapid drying in the oven at over 210 F results in an abnormally high value for shrinkage or absorption. They concluded that such drying "causes not only the withdrawal of the mechanically suspended water which might be absorbed or withdrawn at ordinary temperatures, but also a certain amount of absorbed or colloidal water more tenaciously held in the cement gel. Hence the absorptions are too large and do not reflect the condition or behavior of the concrete in its usual temperature-humidity environment."[25]

CHAPTER VI

COMPRESSIVE STRENGTH

Compressive strength is perhaps the most thoroughly studied physical property of concrete. It is variously employed in mix design studies and in testing concrete-making qualities of cement and aggregates. Many of the other physical properties of concrete are studied in terms of, or correlated with, the compressive strength. It was found that the elasticity of concrete, its tensile and flexural strength, wear, permeability and fire resistance are affected (besides the affect of the physical characteristics of the aggregates) by the compressive strength.[25] What makes this physical property of concrete even more important is that the usual methods of design are based on allowing concrete to carry compressive stresses only. This makes compressive strength, in this case, the sole criterion of the strength of the concrete.

This wide use of the compressive strength of concrete is (among other reasons) because of its economy and simplicity of test. However, the evaluation of the results is, indeed, not as simple as the test itself. It involves many factors affecting the actual strength of concrete and its "indicated strength" (strength indicated by tests which is not necessarily equal to the actual strength). Among factors affecting the actual strength of concrete is the type and quality of the ingredients and their proportions in the mix, water-cement ratio, methods of mixing and curing.... Other factors further affecting the indicated compressive strength are the age of

the specimens, shape and size, capping, speed of testing and even temperature during the test. Some of the factors affecting both types of strengths will be discussed in more detail later. It may also be noted that the strength of concrete structures is not necessarily proportional to either the actual or indicated strength. In plastic concrete the test specimens must be made (mixed, cured...etc.) in the same manner and from the same materials used for the concrete in the structure. In concrete masonry, workmanship is found to affect the strength of the walls. This will also be discussed later.

A. Procedure of Test.

The procedure of the ASTM Designation C140-52 (Methods of Sampling and Testing Concrete Masonry Units)[22] was used for this test.

Five full size units were selected at random from samples of each product, capped and tested not later than 60 hours from the time of their arrival at the laboratory. Test load was applied in a direction the same as that for the unit when built in the wall.

The two bearing surfaces of the specimen were capped with a paste made of one part (by volume) of cement and one part of calcined gypsum (plaster of paris) mixed with water so that it was plastic and spread evenly without excessive flowing. A cap not thicker than 1/8 inch was placed in the same manner described by the ASTM Designation mentioned above. Allowing the first cap to harden for six hours, the specimen was turned over for capping the opposite surface. Capped specimens were then aged for 24 hours before testing. Imperfect, cracked or damaged caps were completely removed and replaced by new caps.

The 300,000 pound test machine was equipped with a circular, spherically seated upper bearing block which has the same diameter (10 inches) as the lower bearing block. The blocks did not, therefore, cover the bearing area of the specimen, and two bearing plates having machine polished plane surfaces, were placed at top and bottom, between the bearing blocks and the specimen. These plates were 16 inches long, 12 inches wide and 1-3/4 inches thick, thus covering the bearing area of the specimen. During test they were placed so that a vertical line passed through their centers and the centers of the specimens and the bearing blocks of the machine. Placed thus, the distance between the edge of the bearing blocks and the corner of the plate is 5 inches which is less than 3 times the thickness of the plate as required by ASTM Designation C140-52.

Loading was applied at the rate of 40,000 pounds per minute which conforms with the ASTM requirement. Most of the specimens were broken within 2 to 3 minutes.

B. Discussion of Test Results.

Results of the compression test are summarized in Table 10, which contains the averages of the results of five specimens tested from each product. Detailed data and the calculation of the results of all the specimens are given in Table 43 in Appendix IV.

In Tables 10 and 43 the compressive strength (psi) is reported on the basis of the gross cross-sectional area of the unit (defined in Chapter IV). These areas were found by the actual measurements of the overall dimensions of the units using the procedure discussed in Chapter IV.

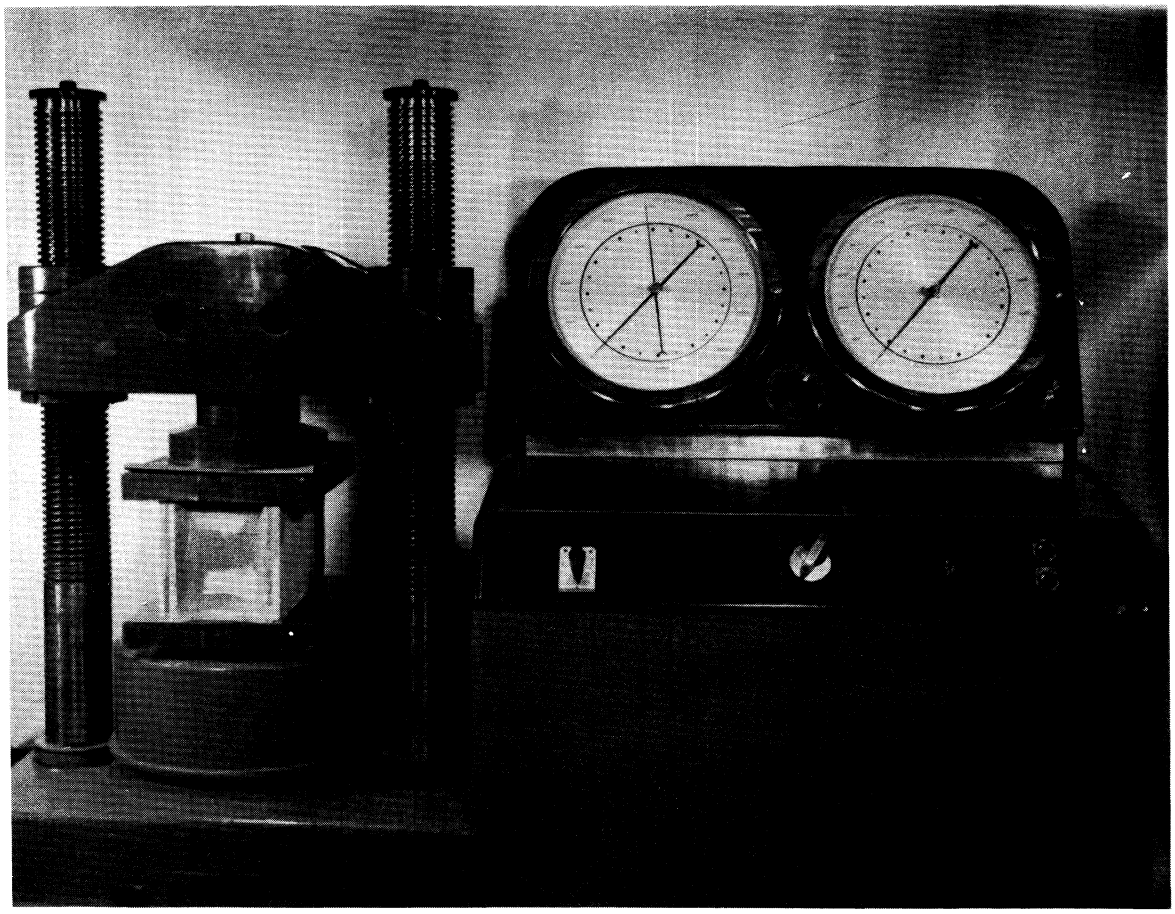


Figure 14. Compressive Strength Test.

In Table 10, the compressive strengths are also given on the basis of the net cross-sectional area (defined in Chapter IV) of the units (column 7). These areas are also reported in column 5. They are, unlike the gross areas, not found by measurements of the same specific test specimens, but are the average results (also five specimens per product) of measurements on companion units. These latter units were also used for the dimensional properties studied in Chapter IV and which are reported in Table 7, column 9. The reason for this is that it was considered that the compression test specimens should be tested in the sampled condition and without being exposed to any preliminary conditioning. Net sectional areas of the unit are calculated from their net volume. Testing for net volume (see Chapter IV, part B) involves submerging the specimen in water for 24 hours. The effect of this saturation on the strength of units made with those various dense and lightweight aggregates is not accurately known, and it was further found detrimental to successful capping. Drying of the specimen is not desirable because drying by heat (after soaking) is considered by some to adversely affect the strength of the concrete. Drying in air at room temperature is not practical because it requires considerable time, while the ASTM Designation C140-52 (Standard Methods of Sampling and Testing Concrete Masonry Units)[22] requires that compression tests should be performed within 72 hours after sampling. The results reported in Table 10 of the compressive strength (net cross-sectional area basis) should not deviate much from the results that would be obtained by using the actual net areas of the tested specimen. This can be concluded by comparing the gross cross-sectional areas of the units used for the compression test and those

used for the dimensional analysis (of Chapter IV and which are also used in these calculations) which are presented in Tables 42 (Appendix IV) and 36 (Appendix II) respectively. It is obvious, from this comparison, that the gross areas of a certain product in both tables are very similar and the corresponding net areas may, therefore, be expected to be also similar.

Referring again to Table 10, study of the compressive strength based on the gross area of the unit reveals that:

(1) The highest strength of concrete made with any of the aggregates (dense and lightweight, 1332 psi) is given by 2H2 (Haydite), followed by 9S2 (air-cooled slag) with 1243 psi.

(2) Products made with sand and gravel show a lower strength of 1208 psi for 3SG2 and 994 psi for 4SG2.

(3) Products made with all-cinder aggregates show a lower strength than those made with cinders mixed with other materials. For example, 1C2 and 3C2, both made with all-cinder aggregates, give strengths of 698 psi and 754 psi respectively, while 2CES2 (cinders mixed with expanded clay) gives 1004 psi. Cinders mixed with sand (5CS2) or fly ash (10CA2) give 877 psi and 906 psi respectively.

(4) Products made with expanded slag generally gave lower results of 695 psi by 6ES2, 777 psi by 7W2 (Waylite) and 647 psi by 8CEL2 (Celocrete). Those made with expanded slag mixed with expanded clay (Waylite and Beslite) 7BW2, have higher strength of 989 psi.

In the beginning of this chapter it was noted that the actual compressive strength depends on numerous factors, among which are the types of cement and aggregates, proportioning, water-cement ratio, use of admixtures, method of mixing and curing. Further affecting the

TABLE 10. COMPRESSIVE STRENGTH OF THE MASONRY UNITS

1	2	3	4	5	6	7
					Compressive Strength	
Sample No.	Type of Aggregate	Compressive Force lbs.	Gross Area in^2	Net Area in^2	Based on Gross Area psi	Based on Net Area psi
1C2	Cinders	83,900	120.0	61.7	698	1357
2CES2	Cinders & Expanded Slag	120,000	119.6	67.5	1004	1778
2H2	Haydite	158,620	119.1	67.2	1332	2361
3C2	Cinders	90,900	120.5	72.6	754	1252
3SG2	Sand & Gravel	145,770	120.8	65.9	1208	2212
4SG2	Sand & Gravel	120,700	121.4	60.9	995	1983
5CS2	Cinders & Sand	104,400	119.0	60.4	877	1728
6ES2	Expanded Slag	83,000	119.9	59.5	695	1395
7W2	Waylite	92,900	119.4	61.9	777	1506
7BW2	Beslite & Waylite	118,300	119.6	61.5	989	1923
8CEL2	Celocrete	78,600	121.7	65.6	647	1199
9S2	Air-Cooled Slag	147,800	119.0	67.3	1243	2195
10CA2	Cinders & Fly Ash	109,120	120.5	65.6	906	1663

indicated compressive strength are the size, shape and age of the specimens. Some of these factors will be discussed in the following.

(1) Type of cement: In Table 3 (Chapter II) it is reported that all but plants 1 and 3 use high-early strength cement. Plant 1

(product 1C2) and Plant 3 (product 3C2 and 3SG2) use regular cement. Since the age of samples is not reported, the effect of the type of cement on strength cannot be evaluated.

(2) Crushing strength of aggregate: The effect of the crushing strength of the aggregates on the strength of the units cannot be evaluated because such tests were not conducted. However, it is believed that it does not have a marked effect at such comparatively low strength as exhibited by the units tested. It has been pointed out previously that some of the lightweight aggregate products have a higher strength than those made with dense aggregates. The lightweight aggregates tested are recognized to have lower crushing strength than dense aggregates. It was found possible by some researchers[20] to make concrete from expanded shale having a strength (based on standard test cylinders) of 7000 psi. Compressive strength as high as 4000 psi (much higher than the present test values) is not uncommon with concrete made with lightweight aggregates.

(3) Proportioning: Accurate study of the cement content of the plant mixes (Table 3) is not possible. All mixes (except for products 1C2, 3C2 and 4SG2) are reported by weight and since the specific gravity of the aggregates varies widely, a comprehensive study on weight basis cannot be attained. However, rough analysis was employed by converting some of the mixes to volume using the available unit weights of the aggregates as determined by tests and reported in Table 4 (Chapter III). The conclusion on the basis of this rough analysis is that there is no correlation existing between the cement content and the strength of the product.

(4) Water-cement ratio: An accurate determination of the water-cement ratio of the plant mix is not possible because in most cases the amounts of water added vary with the moisture contents of the aggregates at the time of mixing. Many plants find it necessary to adjust the amount of water several times during the day. In many cases the consistency of the mix is measured by that required for molding by pressure and vibration and permitting subsequent stripping of the units without physical damage (see Chapter II). It may therefore be expected that the consistency varies slightly from one mix to another in the same plant and that the variations are even larger in the mixes of different plants. The effect of the water-cement ratio on strength cannot be over-stressed and is believed responsible for the observed variations in the strength of some units in the same product (see Table 43, Appendix IV).

(5) Age of the specimen: The effect of the age of the specimen on the indicated strength is important. Most of the plants, however, do not specifically classify their products according to age and its effect is, therefore, undetermined in this study.

The compressive strength of the units based on their net cross-sectional areas (Table 10, column 7) gives a better view of the strength variation of concrete in the units. However these strengths are not equivalent to those in design codes (or literature in general) which are usually determined by tests on standard 6 x 12 inch cylinders. Research indicates that the size and shape of specimens have an important effect on the indicated compressive strength.

Correlation between the compressive strengths and the unit dry weight of the products is attempted but such relations were found

not to be existing. Correlation between the compressive strength and the flexural strength or the modulus of elasticity of concrete will be discussed later.

The ASTM Designation C90-52 (Specifications for Hollow Load-Bearing Concrete Masonry Units)[22] specifies for the compressive strength at the time of delivery to the site of the work to be as in Table 11.

The compressive strengths in Table 11 are based on gross cross-sectional area of the unit. By comparing the average results of tests in Table 10 in terms of the values specified in Table 11, the following may be concluded:

(1) Only four products have average strengths higher than 1000 psi (column 6). These are 2CES2, 2H2, 3SG2 and 9S2. The product 3SG2 has one individual unit showing a strength below 800 psi (see Table 43).

(2) Three products display a relatively high result; these are 4SG2 (995 psi), 7BW2 (989 psi) and 10CA2 (906 psi). None of these showed a unit to have a strength lower than 700 psi.

(3) The remaining products (cinder or expanded slag aggregates) have average results ranging between 877 psi (5CS2) and 647 psi (8CEL2). It may also be noticed that except for the latter, all of the remaining products averaged higher or very close to 700 psi. From Table 43 it may further be seen that of all these products investigated, only two showed individual units with strengths below 600 psi. These are 6ES2 (averaged 695 psi) with only one such unit and 8CEL2 (averaged 647 psi) with 3 such units.

TABLE 11. PHYSICAL REQUIREMENTS*

Minimum Face Shell Thickness in.	Compressive Strength, min., psi (Average Gross Area)	
	Average of 5 Units	Individual Units
1.1/4 or over:		
Grade A[a]	1000	800
Grade B[b]	700	600
Under 1.1/4 and over 3/4	1000	800

a. For use in exterior walls below grade and for protected exterior walls above grades that may be exposed to frost action.

b. For general use above grades in walls not subjected to frost action or where protected from the weather with two coats of portland cement paint or other satisfactory water-proofing treatments approved by the purchaser.

* Table 11 and its footnotes are from Table 1 of the ASTM Designation C90-52 (reference 22).

It is of significance here to note that the actual bearing strength of a concrete masonry wall is lower than the compressive strength of its units. This was proven by the University of Illinois tests on the structural performance of concrete masonry walls as related to the strength of the unit and which was discussed previously in Chapter I. It was there concluded that the compressive strength of a wall panel depends on the compressive strength of the unit with an average ratio of approximately 0.53. The American Standard Building Code Requirements for Masonry (1954)[29] specifies the allowable stresses of concrete masonry of hollow units to be 85, 75 or 70 psi (based on gross area) according to the type of mortar used. This permits a safety factor (considering also the University of Illinois

tests) of approximately 4.5 to 9.0. It is also indicated (see page 28 of reference 29) that the "compressive strength of masonry of block or tile depends importantly upon the completeness with which the units are bedded. For end-construction masonry, the bedding has ranged from covering the two face shells only, to complete coverage of the ends of all shells and webs...."

CHAPTER VII

FLEXURAL STRENGTH

Concrete is much weaker in tension than it is in compression. The flexural strength of plain regular concrete beams is found to be approximately equal to 0.10 of its compressive strength. However, concrete made with some lightweight aggregates has produced flexural strength as high as 0.25 of its compressive strength.[21] Yet the flexural strength of concrete is always much lower than its compressive strength. Consequently, failure of plain concrete beams in cross-bending inevitably occurs in tension at the outer fibers. This is why, as was mentioned in Chapter VI, most methods of design for concrete members (slabs, beams, columns....) depend on concrete to carry only compressive stresses. However, in some concrete structures such as plain footing and especially roads and air field slabs, high flexural strength is considered imperative. Tensile stresses can develop also in masonry concrete walls by bending under transverse loads such as from wind or earth pressures. Masonry units in these walls will, therefore, be under such stresses. As will be discussed later, the allowable stresses for tension in the extreme fibers of masonry units has not yet been proposed by the common codes and specifications dealing with such materials. The resistance of unreinforced masonry walls to such stresses is usually neglected. Research has also proved that the strength of mortar is a very important factor in this resistance; this will also be discussed later.

The flexural strength test of the masonry unit is important because it does give an indication of the tensile strength of concrete

and thus its resistance to cracking. It is also used as an additional measurement of the effect of repeated freezing and thawing on concrete.

As in the compression test, the indicated flexural strength depends among other things on the type and size of the specimen and method of test; these will be discussed later.

A. Test Specimen.

Test beams of the approximate dimensions 2-1/2 x 3-5/8 x 15-5/8 inches were sawed off solid masonry units (slabs) of the modular dimensions 3-5/8 x 8-5/8 x 15-5/8 inches (see Chapter II). Three beams were obtained for this test from each product except 2H and 7BW. The products 2H (Haydite) and 7BW (Beslite and Waylite) were not available in slabs.

B. Apparatus.

The test apparatus is shown in Figures 15 and 17. It was constructed in the University of Michigan shops. It consists of a base and two support blocks. The base is made of an American standard channel, size 8 x 2-1/4 inches, 40 inches long. Mounted on the face of the channel are the two supporting blocks, each consisting of a base plate and edge support. The base plate has the dimensions 1 x 4 x 8 inches and is bolted to the face of the base channel. Additional bolt holes are provided in the face of the channel so that the span between the two supports is adjustable. Mounted on the base plate are the two edge supports which have the shape shown in Figures 15 and 17, thus providing a line support to the specimen. One of these edge supports is seated on a steel rod while the second is seated on a steel ball. This arrangement permits a free bending

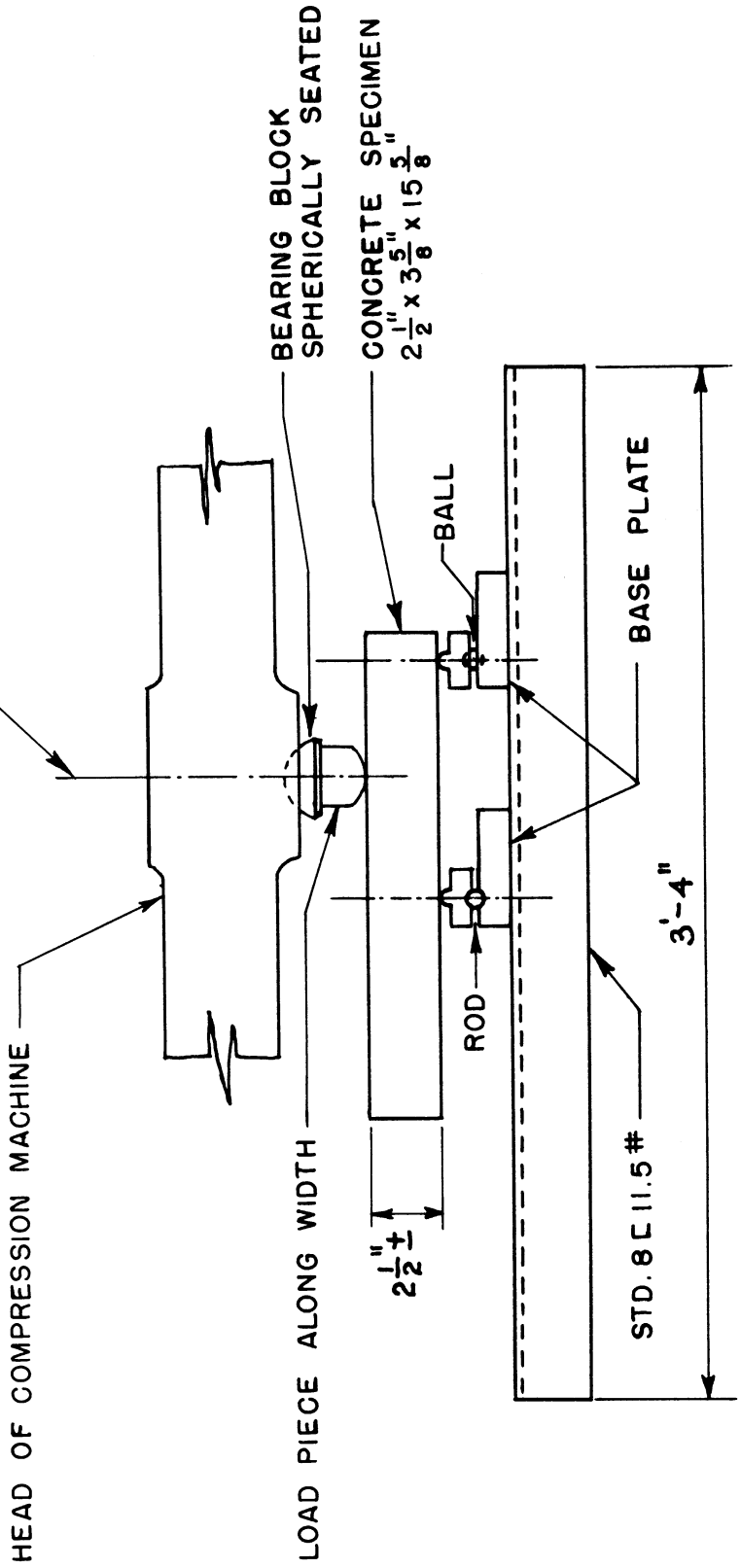


FIG. 15 - SCHEMATIC DIAGRAM OF APPARATUS FOR FLEXURAL TEST

TABLE 6. PHYSICAL PROPERTIES OF AGGREGATES

Sample No.	Type of Aggregate	Material Passing (#200 Sieve % by Weight)	Specific Gravity			Absorption		Organic Impurities Colorimetric Test	Popouts
			B.S.G Oven-Dry Basis	B.S.G. Saturated Surface Dry Basis	A.S.G.	% of Dry Weight	% by Volume (Dry Compacted)		
1C9	Cinder Combined	7.3	1.71	1.85	1.99	8.40	7.87	No Change in Color	None
3C9	Cinder Combined	5.6	1.74	1.90	2.06	9.00	8.26	No Change in Color	None
3SG9	Sand and Gravel Combined	2.23	2.63	2.66	2.73	1.50	2.50	Slight Change in Color	None
4SG9	Sand and Gravel Combined	3.15	2.66	2.69	2.75	1.30	2.47	Very Slight Change in Color	None
5C9	Cinder Combined	3.67	1.68	1.82	1.95	8.20	7.73	No Change in Color	None
6ES19	Expanded Slag Coarse	0.14	1.47	1.57	1.64	6.75	5.16	No Change in Color	None
6ES29	Expanded Slag Fine	0.59	1.78	1.87	1.96	5.11	5.33	No Change in Color	None
7W19	Waylite Coarse	0.14	1.48	1.59	1.67	7.57	5.81	No Change in Color	None
7W29	Waylite Fine	6.72	1.80	1.89	1.98	5.12	4.66	No Change in Color	None
7B9	Beslite Combined	5.70	1.50	1.71	1.89	14.13	15.2	No Change in Color	None
8CEL19	Celocrete Coarse	0.93	0.97	1.21	1.27	24.3	11.05	No Change in Color	None
8CEL29	Celocrete Fine	8.73	1.49	1.71	1.90	14.41	10.68	No Change in Color	None
10CA9	Cinder Combined	10.7	1.71	1.85	1.99	8.41	8.13	No Change in Color	None

of the specimen while keeping the supports parallel to the direction of application of the load.

The loading block at the head of the machine is spherically seated. The load is applied through a steel half cylinder along the entire width of the specimen, thus providing a line bearing.

C. Procedure of Test.

Using the apparatus described above, the specimens were tested as simply supported beams. The mid-point loading method is used because it is more convenient for specimens of the size used for this test. The procedure of the ASTM Designation C293-54T (Tentative Method of Test for Flexural Strength of Concrete - Using Simple Beam with Center-Point Loading)[22] was used.

By using a span length of 8-1/2 inches, it was possible to break the specimen at two sections thus obtaining two test values for each specimen. This span was held constant throughout the program. The specimen was centered in the supporting blocks. The steel half cylinder was placed at midspan along the width of the specimen. The load applying block was then carefully brought to contact. The load was applied at the rate of 200 pounds per minute. This speed corresponds to an increase in the extreme fiber stresses of less than 150 psi per minute. The breaking force was recorded, then the second marked section of the specimen was centered and broken using the same procedure as above. The dimensions of the two breaking sections of the specimen were then measured and the average recorded.

D. Calculations.

Modulus of rupture is used as a convenient means of expressing the flexural strength of beams. It is the calculated tensile

stresses at the extreme fibers of the beam caused by the breaking load indicated by the machine.

For simply supported beams broken by concentrated load at the center, the modulus of rupture can be approximated by equation (2) which is derived from the pure bending formula of equation (1).

$$S = \frac{M\,c}{I} \qquad (1)$$

$$R = \frac{3\,P\,L}{2\,b\,d^2} \qquad (2)$$

Where,

R = modulus of rupture (psi)

P = the breaking load (pounds)

L = span length in inch (equal to 8-1/2 inches, constant)

b = width of the specimen at the breaking section (inch)

d = depth of the specimen at the breaking section (inch).

Equation (2) is derived by ignoring the weight of the specimen which is negligible. The assumption used and the derivation of equation (2) is presented in Appendix V.

E. Discussion of Test Results.

Calculations of the results are given in Table 12. The modulus of rupture of the two breaking sections of each specimen are calculated separately and reported in columns 8 and 14. Their average (the average result of each specimen) is given in column 15 of the table. While the average of the results of the three specimens tested from each product is also reported in column 16 of the table.

The modulus of rupture, like the compressive strength, varies widely from one product to another and sometimes substantial

TABLE 12. MODULUS OF RUPTURE (R) CALCULATIONS

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
		Breaking Section No. 1						Breaking Section No. 2							
Sample No.	Type of Aggregate	Breaking Force P lbs.	Dimensions of the Section Width b in.	Depth d in.	3 P L (L=8.5 inch constant)	$2bd^2$	R_1 psi	Breaking Force P lbs.	Dimensions of the Section Width b in.	Depth d in.	3 P L (L=8.5 inch constant)	$2bd^2$	R_2 psi	Average R for the Specimen psi	Average Modulus of Rupture for the Product psi
1C41	Cinders	460	3.66	2.45	11,730	43.9	267	480	3.68	2.45	12,240	44.2	277	272	272
1C42		515	3.62	2.52	13,132	46.0	285	500	3.65	2.52	12,750	46.4	275	280	
1C43		485	3.65	2.60	12,368	49.3	251	520	3.66	2.56	13,260	47.9	279	265	
2CES41	Cinders &	610	3.63	2.50	15,555	45.4	343	250	3.63	2.50	6,375	45.4	140	242	224
2CES42	Expanded	410	3.64	2.30	10,455	38.5	271	180	3.63	2.25	4,590	36.7	125	198	
2CES43	Slag	610	3.65	2.55	15,555	47.5	328	245	3.61	2.55	6,248	46.9	133	231	
3C41	Cinders	720	3.67	2.56	18,360	48.1	382	775	3.65	2.60	19,762	49.4	400	391	354
3C42		580	3.67	2.40	14,790	42.3	350	605	3.65	2.40	15,427	42.0	367	358	
3C43		565	3.62	2.45	14,407	43.4	332	505	3.63	2.45	12,877	43.6	295	314	
3SG41	Sand &	1070	3.67	2.50	27,285	45.9	594	780	3.68	2.50	19,890	46.0	432	513	510
3SG42	Gravel	1075	3.65	2.35	27,412	40.1	685	730	3.62	2.30	18,615	38.3	434	56	
3SG43		1065	3.62	2.62	27,158	49.7	546	705	3.62	2.60	17,978	48.9	368	457	
4SG41	Sand &	1155	3.75	2.58	29,452	49.9	590	1195	3.76	2.62	30,472	51.6	591	591	635
4SG42	Gravel	1405	3.75	2.51	35,828	47.3	757	1270	3.75	2.46	32,385	45.4	713	735	
4SG43		1010	3.78	2.38	25,755	42.8	602	955	3.78	2.41	24,353	43.9	555	579	
5CS41	Cinders	620	3.65	2.41	15,810	42.3	374	500	3.68	2.40	12,750	42.4	300	337	344
5CS42	& Sand	655	3.65	2.53	16,702	46.7	357	590	3.65	2.50	15,045	45.6	330	344	
5CS43		660	3.65	2.51	16,830	46.0	366	600	3.65	2.50	15,300	45.6	336	351	
6ES41	Expanded	430	3.58	2.40	10,965	41.2	266	470	3.57	2.40	11,985	41.1	292	279	338
6ES42	Slag	705	3.65	2.65	17,977	51.3	350	795	3.62	2.62	20,272	49.7	408	379	
6ES43		670	3.67	2.45	17,085	44.0	388	580	3.65	2.50	14,790	45.6	324	356	
7W41	Waylite	530	3.65	2.50	13,515	45.6	296	630	3.65	2.52	16,065	46.4	346	321	334
7W42		540	3.66	2.38	13,770	41.4	333	425	3.65	2.32	10,837	39.1	377	355	
7W43		670	3.65	2.60	17,085	49.4	346	595	3.65	2.60	15,172	49.3	307	327	
8CEL41	Celocrete	310	3.78	2.45	7,905	45.4	174	350	3.78	2.45	8,925	45.4	197	186	193
8CEL42		470	3.76	2.48	11,985	46.2	259	320	3.75	2.56	8,160	49.1	166	212	
8CEL43		280	3.75	2.35	7,140	41.3	173	325	3.75	2.40	8,287	43.2	192	183	
9S41	Air-Cooled	1140	3.65	2.62	29,070	50.1	580	980	3.68	2.59	24,990	49.3	507	544	526
9S42	Slag	890	3.68	2.48	22,695	45.3	500	800	3.68	2.48	20,400	45.3	451	476	
9S43		940	3.56	2.48	23,970	43.8	547	980	3.65	2.45	24,990	43.8	571	559	
10CA41	Cinders &	460	3.65	2.41	11,730	42.3	277	470	3.65	2.45	11,985	43.8	274	276	277
10CA42	Fly Ash	285	3.65	2.25	9,818	36.9	266	430	3.65	2.26	9,650	37.2	259	263	
10CA43		550	3.63	2.61	14,025	49.4	284	600	3.65	2.65	15,300	51.3	298	291	

variations exist between specimens of the same product. This latter variation can probably be explained as in the compressive strength. However, flexural strength is recognized as being more sensitive to the character of the aggregates than is compressive strength. Further study of Table 12 reveals that:

(1) Some variations exist in the results of the two breaking sections of the specimen. However, the products 2CES4 (cinders and expanded slag) and 3SG4 (sand and gravel) show larger variations. This is believed to be caused by variation in concrete quality and not by method or procedure of test, otherwise it should appear in a greater number of specimens.

(2) Products made with dense aggregates have higher moduli of rupture than those made with lightweight aggregates. Average results for the products made with sand and gravel are 510 psi and 635 psi for 3SG4 and 4SG4 respectively, while those made with air-cooled slag (9S4) averaged 526 psi. The maximum result for lightweight aggregate products is 354 psi by 3C4 (cinders).

(3) The modulus of rupture of the products made with lightweight aggregates averaged between 354 psi for 3C4 (cinders) and 193 psi for 8CEL4 (Celocrete). However, four of these products have higher results than 330 psi. These are 3C4, 5CS4, 6ES4 and 7W4. The rest are lower than 300 psi.

Very poor correlation exists between the flexural strength and the compressive strength of the products, as can be seen in Table 13. The compressive strengths are the average results of tests on hollow masonry units based on net areas and were originally reported in Table 10. Moduli of rupture are taken from column 16 of Table 12.

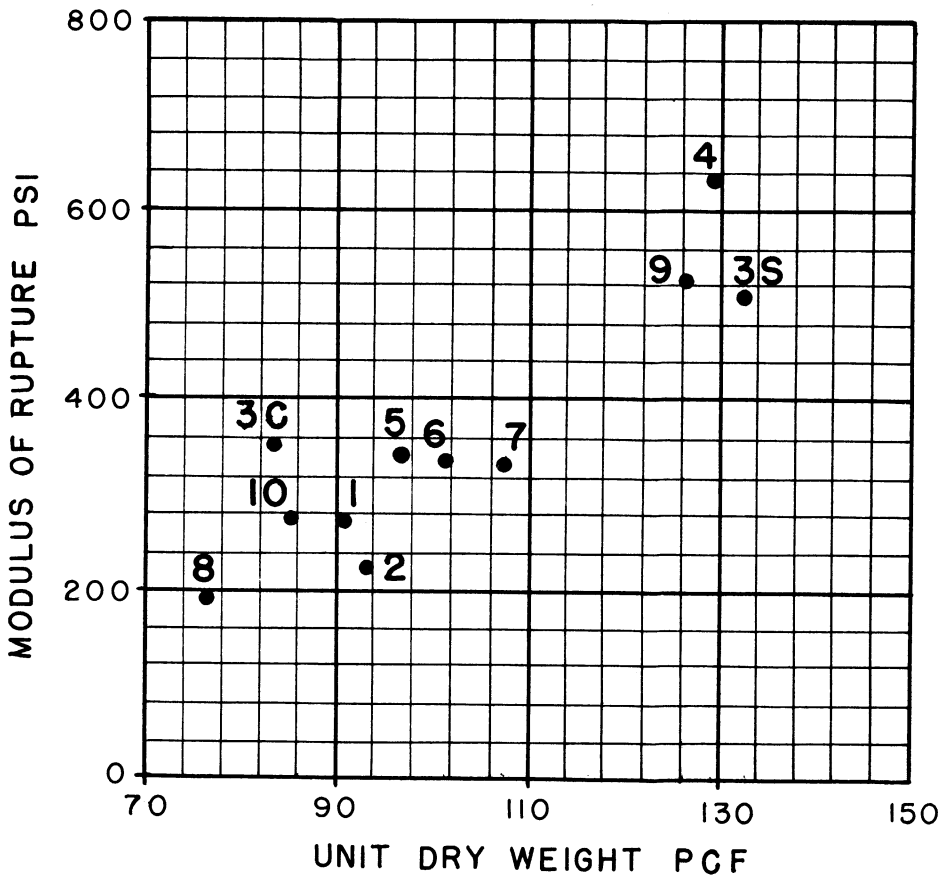
This lack of correlation can be seen also from column 5 of Table 13 where the ratio of the modulus of rupture and the compressive strength for each product is given.

TABLE 13. FLEXURAL STRENGTH OF CONCRETE - ANALYSIS OF THE RESULT

1	2	3	4	5
Sample No.	Dry Weight p.c.f.	Compressive Strength psi	Modulus of Rupture psi	Mod. of Rupture -Comp. Strength Ratio
1C4	90.9	1357	272	.20
2CES4	93.1	1778	224	.13
3C4	83.4	1252	354	.38
3SG4	132.8	2212	510	.23
4SG4	129.2	1983	635	.32
5CS4	97.1	1728	344	.20
6ES4	101.3	1395	338	.24
7W4	97.3	1506	334	.22
8CEL4	76.2	1199	193	.16
9S4	126.3	2195	526	.24
10CA4	85.2	1663	277	.17

However, rough correlation exists between the moduli of rupture and the unit dry weights of the products. This can be seen from Table 13 or Figure 18. The unit dry weights are the average results of tests conducted on hollow masonry units and originally reported in Table 8.

Comparison of the results with those reported by other investigators is difficult because of the wide variation in the results



LEGEND

1	= 1C4	6	= 6ES4
2	= 2CES4	7	= 7W
3C	= 3C4	8	= 8CEL4
3S	= 3SG4	9	= 9S4
4	= 4SG4	10	= 10CA4
5	= 5CS4		

FIG. 18 – MODULUS OF RUPTURE VS. UNIT DRY WEIGHT

and their significance. This is probably because the results are easily affected by the method and rate of application of load, dimensions of the specimen, its temperature and moisture condition. Many theories were advanced to explain and account for the effect of these and other factors on the indicated flexural strength of concrete; but there is no general agreement as to which of these theories is the most valid. The most important of these are the works of Weibull and later of Tucker on the "weakest link" theory, the "strength summation" theory of Tucker[30] and the works of Reagel and Willis[33], Wright and Garwood[31] and most recently, Nielson.[32]

The weakest link theory assumes that the concrete beam is formed of intersecting links (sections) along the length of the beam and are probably of various strengths. The failure does not therefore necessarily occur in the section of highest stress but might occur in the weakest section, although the stress might be lower in this section. The strength summation theory assumes that the strength of a specimen is equal to the sum of the strengths contributed by the component parts or elements. This theory has a different application as we shall see later. Some of the factors affecting the results of test will be discussed in the following:

(1) Method and rate of application of the load: Research proved that mid-point loading gives higher results than third-point loading (ASTM Designation C78-49: Method of Test for Flexural Strength of Concrete - Using Simple Beam with Third-Point Loading), but the result of the latter method is more uniform. This was explained statistically by the weakest link theory as being due to the fact that in the third-point loading larger portions of the beam (the entire

middle third) is under maximum stress while in center-point loading, only the middle section is under maximum stress.

The modulus of rupture has been found to vary directly with the rate of applying the load. In conducting the present tests, the maximum rate of application of the load was about 200 pounds per minute corresponding to an increase in stress at the extreme fiber of about 150 psi per minute which is in accordance with the ASTM method. Wright suggested 100 psi per minute for standardization in Britain.[31]

(2) Size of the specimen: It was found by Tucker, Wright, Garwood and others that the modulus of rupture decreases with the increasing length or depth of the specimen. This was also explained by the weakest link theory by assuming that the beam is formed of intersecting layers or fibers and that the failure occurs in the weakest fiber and not necessarily in the outer fiber as it is usually assumed.

Tucker also showed that the increase in the width of the specimen does not affect the modulus of rupture numerically. However, the dispersion of the results decreases with increasing width. This was explained by Tucker's strength summation theory.

(3) Moisture condition of the specimen: The moisture condition of the specimen affects the result of test. When the outer layers of the specimen dry out, tensile stresses set up in these layers reduce the indicated modulus of rupture. This effect varies with the depth of the specimen. Nielson[32] concludes that this might explain the decrease in modulus of rupture with increasing depth and not the weakest link theory discussed before.

Concrete design codes, and specifications in general, contain no proposal for the allowable tensile stresses in extreme fibers for

concrete made with lightweight aggregates. However, most codes propose values for regular concrete, usually equal to three per cent (0.03) of the compressive strength. Table 13 shows this ratio to be between .13 and .38 with most of the products showing over 0.2. Notwithstanding the many factors affecting both types of tests, the results imply a factor of safety of from 4 to 9 on the usual 0.03 ratio.

Concrete masonry codes, as previously stated, do not propose values for allowable stresses for tension in the extreme fibers of the masonry unit because agreement has not yet been reached as to the appropriate values under various conditions of loading.[29] It is also repeated that the flexural strength of unreinforced masonry walls depends largely on the strength of the mortar. This was shown by the University of Illinois test[10] (see also Chapter I) on full size wall panels which were supported laterally at top and bottom, providing a 9 foot span and loaded laterally above a horizontal center line. Almost all of these walls failed in a mortar joint near mid-height of the wall, usually along a horizontal joint with most of the failure in the adhesion of the mortar to the unit. Attention is invited to the fact that many specifications now require the use of reinforcement in the mortar joint to alleviate this deficiency.

CHAPTER VIII

ELASTIC PROPERTIES

Concrete is considered elastic within certain limits; that is, when subjected to the action of external forces it will suffer a deformation which will disappear upon the removal of these forces. It is also considered, by the designer, as elastically homogeneous and isotropic. Actually, the above are only approximations. Since concrete is a coalescence of materials, its elastic properties depend on the elastic properties of the individual components and their proportions. The elastic properties also depend on the age of concrete, its moisture and temperature conditions and type of loading. Nevertheless, tests have justified these approximate assumptions thus permitting the application of the theory of elasticity in concrete design.

Elastic properties of concrete are measured by its Young's modulus of elasticity (E), its modulus of rigidity in shear (G) and its Poisson's ratio (μ).

Young's modulus of elasticity is based on Hooke's law which states that stress is proportional to strain within the elastic limit. The modulus E is this proportionality factor and therefore is defined as the ratio of unit stress to corresponding unit strain.

The modulus of elasticity in shear, G (sometimes called the "modulus of rigidity"), is defined as the ratio of unit shear stress to corresponding unit shear strain of a body subjected to pure shear stresses.

Poisson's ratio (μ) of isotropic materials is the ratio of lateral strain to the corresponding axial strain.

For concrete these elastic constants can be determined experimentally. Two general methods are used: (1) the static method, and (2) the dynamic method. The results of the two differ somewhat for reasons that will be discussed later.

In the static method, Young's modulus of elasticity (E) is usually determined on prismatical or cylindrical specimens which are subjected to known compressive, tensile or flexural stresses; the corresponding strains are then measured by suitable mechanical or electrical strain gages. The modulus of elasticity in shear (G) can be determined statically by applying a known torque to a specimen of suitable dimensions (usually cylindrical) thus subjecting the specimen to "pure" shearing stresses; the corresponding shear strain is then measured as the angle of twist developed over a certain gage length. Poisson's ratio can be determined statically by measuring the axial and lateral strains caused by a certain compressive or tensile stress. Sensitive gages are needed for measuring the lateral strain because of its small order of magnitude. A suitable mechanical-optical system or electrical variable resistance strain gage usually possesses sufficient accuracy.

In the last decade a special nondestructive dynamic method of determining the elastic constant was developed and used extensively. In this so-called "sonic" method (sometimes called the "resonant" method) the specimen (cylinder or prism of suitable dimensions) is vibrated in free-free fundamental mode; transverse (flexural) vibration is used for determining E, while torsional vibration is used for

determining G. Poisson's ratio (μ) can then be calculated from the interrelation of E and G.

Accurate determination of the elastic constants for concrete, although they are affected by many factors as has been previously mentioned, is desirable: Young's modulus of elasticity (E) is necessary for designing plain and reinforced concrete and for predicting the deformations of concrete members under certain loadings, or for the calculation of stresses caused by volume change of concrete by variation in temperature and moisture. The modulus of elasticity in shear (G) is important for the design of concrete members subjected to torsion such as those of some statically indeterminate structures. Poisson's ratio (μ) also enters into the design of some structures (domes, skew-arches....) or for interpreting strain data of concrete.

As will be seen later, dynamic methods might not be suitable for the determination of absolute values of the elastic constants used for calculation of static cases such as discussed above. However, since the dynamic test is not destructive, it serves as a check on the values of the static tests and is excellent for detecting the rate of change of concrete quality as affected by certain other treatments (freezing and thawing, wetting and drying....).

This chapter will present the elastic constants E, G and μ determined by both the static and dynamic methods.

A. Test Specimen.

Since the dynamic (sonic) test is not destructive and to facilitate the analysis of the results, it was decided to use the same specimen for both static and dynamic tests. However, in order to obtain specimens of suitable dimensions for each method, the following procedure was used.

The dynamic test was conducted first on specimens of the approximate dimensions of 2-1/2 x 3-5/8 x 15-5/8 inches which were cut from solid masonry units (slabs) of the modular dimensions of 3-5/8 x 7-5/8 x 15-5/8 inches. (See Chapter II.) Two specimens were tested from each product. Upon the completion of the dynamic test, the specimens were cut in half and one-half of each specimen (approximate dimensions of 2-1/2 x 3-5/8 x 7-1/4 inches) was selected for the static test. Some of the remaining halves were also used as inactive specimens (dummies) in this latter test as will be discussed later.

B. Dynamic (Sonic) Method.

As has been said previously, E and G can be determined dynamically by determining the resonant frequencies of, respectively, transverse (flexural) and torsional vibrations of a specimen vibrating in free-free fundamental mode.

Theory leading to this method will be discussed briefly in the following:

1. Theoretical Discussion.

a) Young's Modulus of Elasticity. (E) When a prismatic bar is vibrating in free-free transverse (flexural) mode of vibration, the elementary form of the differential equation of vibration (references 35, 36) is as follows:

$$\frac{Er^2}{\rho}\frac{\partial^4 v}{\partial x^4} + \frac{\partial^2 v}{\partial t^2} = 0 \qquad (3)$$

Where: x is a member of mutally perpendicular coordinates (x, y, z) at the center of the cross section of the bar; y is in the direction of the vibration while x is in the direction of the length; v is the deflection (displacement) of the bar in the

direction of y axis; r is the radius of gyration of the cross-section with respect to z axis; t = time; ρ = mass per unit volume.

Equation (3) does not include the effect of rotary or lateral inertias or of shear. More accurate equations have been presented by Rayleigh, Love and Mason in which corrections were made for the effect of rotary or lateral inertia. However, Timoshenko's equation accounts for the effect of rotary moment of inertia and shear (see reference 35). This equation is as follows:[36]

$$\frac{Er^2}{\rho}\frac{\partial^4 v}{\partial x^4} + \frac{\partial^2 v}{\partial t^2} - r^2\left(1 + \frac{E}{KG}\right)\frac{\partial^4 v}{\partial x^2 \partial t^2} + \frac{r^2\rho}{KG}\frac{\partial^4 v}{\partial t^4} = 0 \quad (4)$$

For which, in addition to the notations given before:

K = constant introduced by Timoshenko to correct for the effect of shear on the elastic line, defined as the ratio of average unit shear across a section to the unit shear at the neutral axis.

Equation (4) does not correct for the lateral inertia which was proven to be small. The general solution of this differential equation for the problem under consideration, substituting the boundary conditions (both ends are free), can be reduced to the following form:[36]

$$\frac{M}{\tan h \frac{\alpha}{2}} + \frac{N}{\tan \frac{\beta}{2}} = 0. \quad (5)$$

Equation (5) called frequency equation, applies to the first, third, fifth, etc., modes of vibration. Goens presented

solutions in which α and β are determined by satisfying the differential equation (4) above. The values α, β, M, and N are given in terms of $\frac{r}{L}$, μ, K and k.

Where: L is the length of the specimen; μ is Poisson's ratio; K is the shear constant defined above; k is as follows:

$$k = \left(\frac{4 \pi^2 \rho L^4 n^2}{r^2 E} \right)^{1/4} \tag{6}$$

or

$$E = \frac{4 \pi^2 \rho L^4 n^2}{r^2 k^4} \tag{6a}$$

or

$$E = \frac{4 \pi^2 L^3 W n^2}{I k^4 g} \tag{6b}$$

or

$$\underline{\underline{E = C W n^2}} \tag{7}$$

in which

$$C = \frac{4 \pi^2 L^3}{I k^4 g} \tag{8}$$

Where in addition to the notations previously given:

W = weight of the specimen

n = resonant frequency, fundamental mode of transverse vibration

g = acceleration of gravity (386.4 inches per second squared)

I = moment of inertia of the cross-section with respect to the z axis.

Equation (7) will be used for calculating E after determining (n) experimentally. The coefficient C is a dimensional factor which can be calculated, for any specimen of any $\frac{r}{L}$ value, from equation (8) by finding the value of k that satisfies equation (5). Approximate values of the Poisson's ratio (μ) and the shear constant (K) can be assumed for this calculation. For concrete, $\mu = \frac{1}{6}$ is usually assumed and, as it will be shown later, changes in the value of μ do not affect C materially. The shear constant of a rectangular section is assumed equal to $\frac{2}{3}$ by Timoshenko and $\frac{5}{6}$ by Goens. Pickett[36], who solved the frequency equation (5) by satisfying the boundary conditions in the mathematical equations of elasticity, found accurate values for K by selecting the empirical values that give results most nearly in accord when solved by Goens' equation or the mathematical theory of elasticity. He concluded that $K = \frac{8}{9}$ is the best value to use for a prism when $\mu = \frac{1}{6}$.

Goens' solution for C is as follows: Suppose at the limit as $\frac{r}{L}$ approaches zero, k approaches m, and $\alpha = \beta = k$, and if both ends are free, M = N. For this case, equations (5) and (8) approach equations (5a) and (8a):

$$\tan \frac{m}{2} + \tan h \frac{m}{2} = 0 \qquad (5a)$$

$$C' = \frac{4 \pi^2 L^3}{g I m^4} \qquad (8a)$$

Where C' is the limiting value of C and m is the limiting value of k.

Now setting

$$C = C' T \tag{9}$$

Where C is defined by equation (8) then

$$T = \frac{k}{m} \tag{10}$$

T is called the correction factor and was introduced by Goens.

Equation (5a) is the solution (frequency equation) of the elementary equation (3). The value of m, computed from equation (5a) by satisfying equation (3) is equal to 4.730 for the first mode. Substituting this value of m in equation (8a), the value of C can be found to equal the following for any value of $\frac{r}{L}$:

$$C = 0.0002044 \frac{L^3 T}{I} \qquad \text{(for a prism).}$$

However, the coefficient C should be corrected by multiplying by the factor T (equation 9) to satisfy the (adopted) differential equation (4).

The values of the correction factor T should, therefore, be obtained for each value of $\frac{r}{L}$. Goens arrived at an approximate equation for T in terms of $\frac{r}{L}$, and K by expanding the frequency equation obtained by his solution in a Taylor series and retaining only the first few terms. Using $K = \frac{8}{9}$ and $\mu = \frac{1}{6}$, Goens' equation for the first mode (reference 36) becomes:

$$T = 1 + 81.79 \left(\frac{r}{L}\right)^2 - \left[\frac{1314 \left(\frac{r}{L}\right)^4}{1 + 81.09\left(\frac{r}{L}\right)^2}\right] \tag{11}$$

Equation (11) is used largely for the calculation of the correction factor because it involves less computation than other

methods. Pickett[36] found that values of T calculated by equation (11) agree closely with those obtained from the original frequency equation from which the Taylor's expansion was obtained. This agreement was almost exact for values of $\frac{r}{L}$ not higher than 0.3. Since, as is apparent from Table 45 (Appendix VI), the specimens used for this test have $\frac{r}{L}$ ratios much lower than 0.3, it was decided to use Goens' equation. However, the actual calculations of T, C' and C were not necessary. Pickett has calculated the coefficient C using Goens' equation for prismatic and cylindrical specimens of different sizes. He also plotted the results versus the ratio $\frac{y}{L}$ of the specimen (y here being the depth of the specimen in the driven direction). Pickett's curve was, therefore, used for the calculation of C. This will also be discussed with details later.

b) Modulus of Elasticity in Shear (G). Equations for the calculation of (G) from the resonant frequency of the free-free fundamental mode of torsional vibration are as follows:[36,22]

$$G = B\ W\ (n')^2 \qquad (12)$$

Where:

G and W are as defined before

n' = torsional frequency

$$B = \frac{4\ L\ R}{g\ A} \qquad (13)$$

Where:

L and g are as defined before

A = gross sectional area of the specimen

R = ratio of the polar moment of inertia to the shape factor of the specimen. This ratio is unity for cylinders and equal to the following for a specimen with rectangular cross section.

$$R = \frac{\frac{a}{b} + \frac{b}{a}}{4\frac{a}{b} - 2.52\left(\frac{a}{b}\right)^2 + 0.21\left(\frac{a}{b}\right)^6} \qquad (14)$$

Where:

a and b are the dimensions of the cross section, with $a < b$.

Equations (12) and (13) for shear modulus of elasticity are analogous to equations (7) and (8) for Youngs's modulus of elasticity. However, the nature of the factor (R) will be indicated in the following: When a prismatical bar is twisted, its section does not remain plane but warps and radial lines through the center do not remain straight. The distribution of the shear stress is therefore not linear. The ratio (R) was introduced to correct for the effect of warping.

c) Poisson's Ratio (μ). Poisson's ratio of homogeneous and isotropic materials can be calculated from the following equation:

$$\mu = \frac{E}{2G} - 1 \qquad (15)$$

where E and G are determined by the dynamic method as discussed previously. However, it should be kept in mind that concrete is never perfectly elastic, homogeneous or isotropic, and equation (15) will only give approximate results when applied to concrete. The significance of this will be discussed later.

2. Test Apparatus.

The apparatus used for determining the resonant frequency of the specimen in transverse and torsional vibrations is shown in Figures 19 and 20. It consists mainly of three parts: the driver circuit, the pickup circuit and the specimen support. These components, except for the oscillograph, were assembled by "Electro Products Laboratories". The oscillograph is manufactured by Allen B. Du Mont Laboratories.

a) The Driver Circuit. The driver circuit consists mainly of (1) variable frequency audio oscillator, (2) amplifier, and (3) driver unit.

The audio oscillator, power and pickup amplifier and the indicator are housed in the "sonometer" cabinet (Figure 19 at left). The frequency of the oscillator can be varied between 20 cycles to 20 kilocycles per second with approximately 2 per cent accuracy of operation. The oscillator is coupled directly to the power amplifier which in turn is coupled to the driving unit. The amplifier can deliver up to 18 watts of controlled power (controlled by variable resistor) to the driving unit. This power is used to actuate the driver hammer which vibrates the specimen. The hammer is small in mass when compared to the specimen so that its resonant frequency is different than that of the specimen.

b) The Pickup Circuit: The pickup circuit consists of (1) pickup unit, (2) amplifier, (3) resonance indicator, and (4) oscillograph.

Motion generates voltage in the pickup unit which is carried to the pickup amplifier and then to the resonance indicator

(both housed in the sonometer cabinet). Similarly, the pickup needle has much smaller mass in comparison with that of the specimen. Output of the amplifier is also controlled by a variable resistor before it is delivered to the resonance indicator and the oscillograph. The resonance indicator is a milliameter with a range from 0.0 to 1.0.

The cathode-ray oscillograph[37] is used as an auxiliary instrument to facilitate the identification of the modes of vibration and resonant frequency. This instrument indicates visually the essential characteristics of the signal received from the amplifier. This is done in Lissajou's figures that are traced on a screen by feeding the known sinusoidal signals of the driver into the horizontal channel of the instrument and the unknown signals of the pickup into the vertical channel. The ratio of the unknown frequency to the known frequency is the number of loops (of the Lissajou's figure) cutting the vertical axis to those cutting the horizontal axis of the screen. This ratio also indicates the mode of vibration. The ratio 1:1, appearing on the screen as a circle or ellipse, indicates a fundamental vibration which is used in this test. The type of vibration (flexural, torsional....) can also be checked by determining the phase difference of the driver and the pickup signal and the nodal points of vibration. This will be discussed later.

c) Specimen Support. The specimen support consists of a heavy concrete block base and thick rubber pads. The concrete base supports, beside the specimen, the driving and pickup units. Its mass is much larger than any of the specimens tested. The three rubber pads are put between the specimen and the concrete base. This

thick rubber support permitted the specimen to vibrate in a free-free condition without restrictions.

3. Procedure of Test.

Before test, dimensions of the specimens were determined at several sections and their averages were recorded. Also the apparatus was turned on to warm up for 30 minutes before tests were started. While the apparatus warmed, the specimen was weighed and placed on the rubber pads so that it was perpendicular to and almost touching the driver at one end. When the apparatus was ready the right end of the vertical face of the specimen was brought into contact with the driver at a point located at about one inch from the end and thus away from the nodal points. The pickup needle was then brought into contact with the left end of the center line of the upper face of the specimen.

The specimen was then forced to vibrate transversely at a slowly increasing frequency delivered to the driver which was at first pulsating at low frequency. Meanwhile the motion was being studied by observing the resonance indicator and the oscillograph screen which were both brought to a varying "operating level" by the signal received continually from the oscillator and pickup units. When the (fundamental) resonant frequency was reached the resonance indicator responded to the highest reading and the Lissajou's figure on the oscillograph screen formed an ellipse with a well defined peak. The maximum indications occur at resonance frequency because at this frequency the signal received from the pickup unit is maximum; this is because the electrical output of the pickup is proportional to its mechanical input which is determined by the motion of the specimen; at the resonance frequency the amplitude of vibration is,

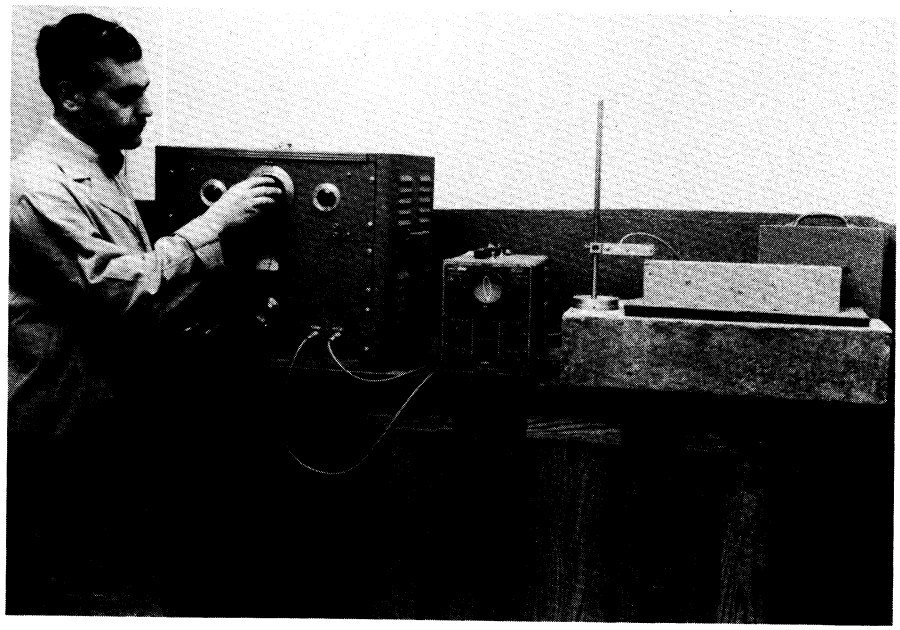


Figure 19. Dynamic (Sonic) Test Apparatus for Determining the Elastic Constants.

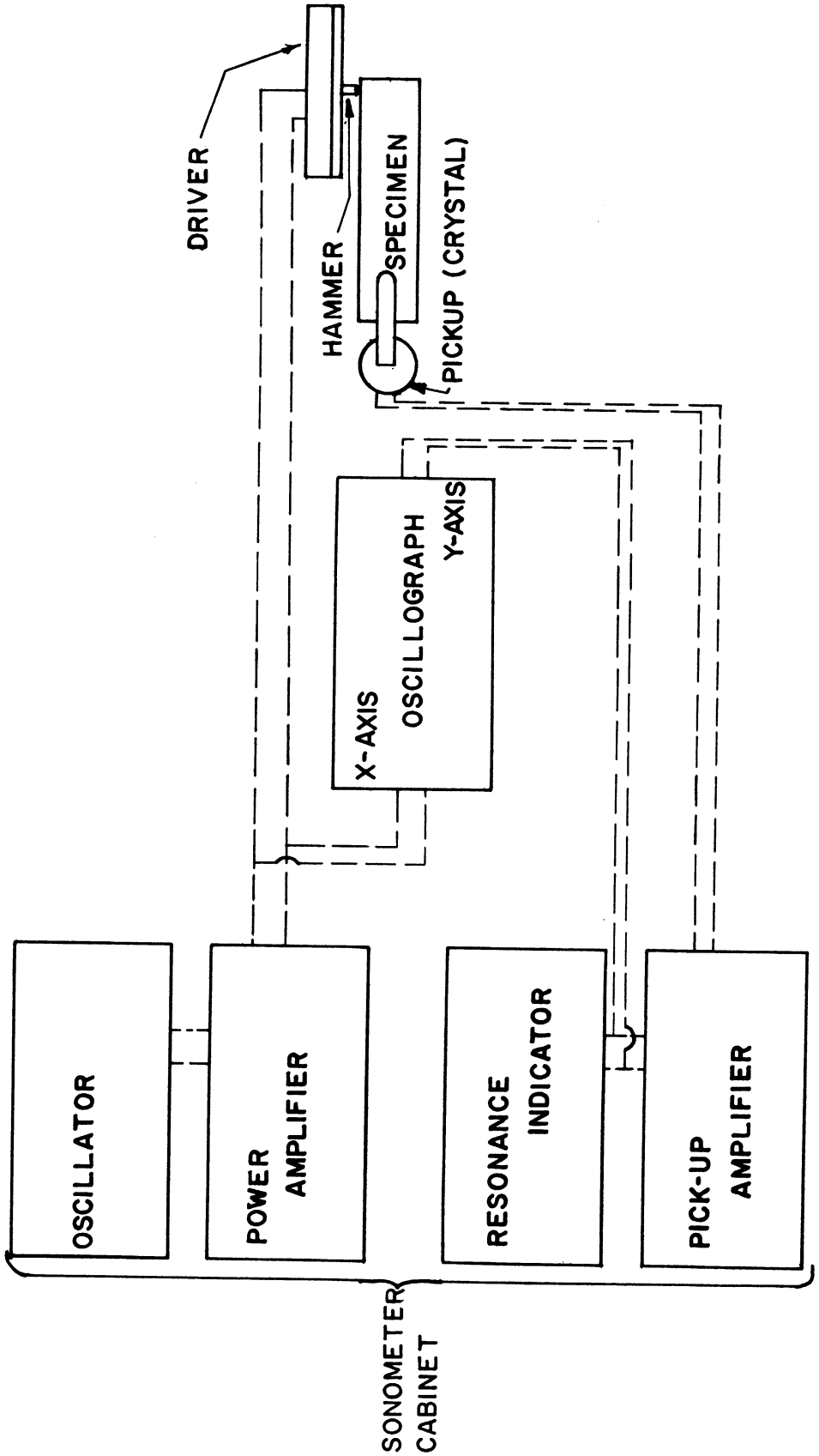


FIG. 20 SCHEMATIC DIAGRAM OF THE SONIC APPARATUS

of course, maximum. The driving force was held constant after determining the resonant frequency, while the phase difference of the driver and the pickup frequencies and the nodal points were being checked to insure that the motion was a fundamental flexural vibration. The nodal points and the phase difference were determined by bringing the pickup to contact with the key points along the entire length of the specimen (detailed procedure is given in Appendix VI). The resonant frequency was then recorded directly from the frequency dial of the apparatus which is marked in cycles per second.

Returning the pickup unit to its first position (at the left end of the specimen) the frequency of the driver was again increased slowly, this time for determining the resonant frequency of the torsional vibration of the specimen. The procedure is generally the same as described above except this time the specimen was forced to vibrate in torsion. When the resonant frequency was determined the motion was also checked in the same manner used for flexural vibration, by bringing the pickup to contact with the key points of the fundamental torsional vibration along the entire length of the specimen. More discussion of the operation is in Appendix VI.

The specimen was first tested by driving in the direction of the larger side of its section. After several determinations were made for flexural and torsional frequencies with the speciman in this position, it was turned 90° so that the driving was this time in the direction of the shorter side of the section and tests were repeated. By this procedure it was possible to check results of the test and arrive at a more accurate average. For flexural vibration the two resonant frequencies of the specimen obtained by

the two tests described above were different because the flexural stiffness of a bar is proportional with the depth. Young's modulus of elasticity calculated from the two frequencies, however, are theoretically equal and the experimental values must be close (see Table 14). For the torsional vibration a quick check was possible because the resonant frequencies obtained from the two tests are theoretically equal as were the experimental values, as may be seen in Table 47 in Appendix VI.

4. Calculations.

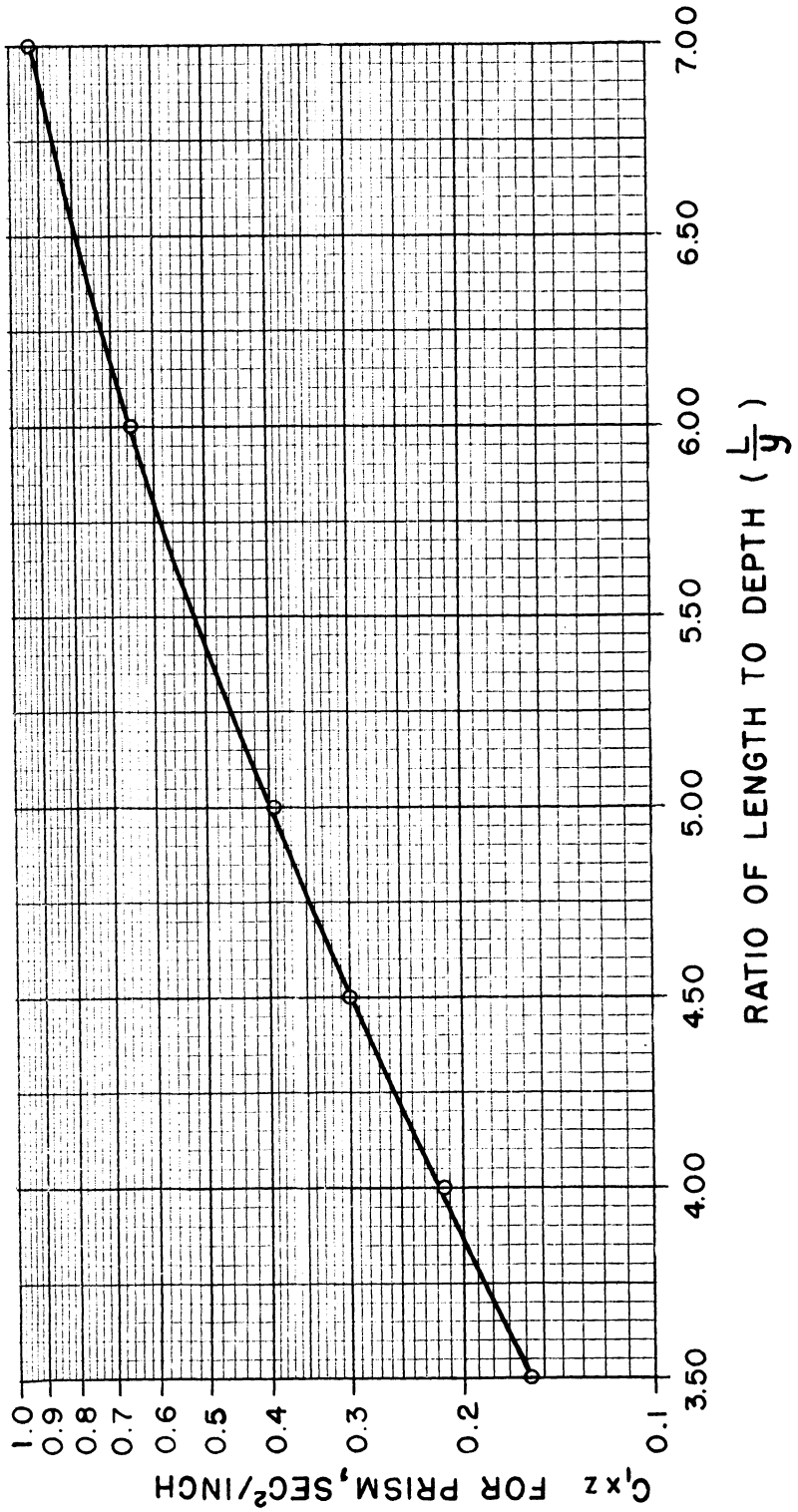
Calculations of Young's moduli of elasticity and the moduli of elasticity in shear for the specimens are given in Tables 14 and 15 (text) and Tables 44 to 49 (Appendix VI). These calculations will be discussed in the following:

a) Calculation of Young's Modulus of Elasticity. Young's moduli of elasticity of the specimens were calculated from the previously given equation (7) which is again as follows:

$$E = C\ W\ (n)^2 \qquad \text{(psi)}$$

Where the dimensional factor C (sec^2/in^2) has been defined by equation (8), W is the weight of the specimen in pounds and n is the resonant frequency of the flexural vibration (cycles per second).

As was previously discussed, values of C for the specimen in this test were taken from Pickett's curve. The portion of the curve that was used in these calculations was replotted in Figure 21. In this figure, values of C x z are plotted versus the corresponding values of $\frac{L}{y}$ where z is the width of the specimen and y is the depth of the specimen during vibration (i.e., y is in the direction of the



y = DEPTH OF BEAM IN DIRECTION OF DRIVING
z = WIDTH OF BEAM

FIG. 21 — CURVE FOR THE COEFFICIENT C_1, CALCULATED BY PICKETT FOR $\mu = \frac{1}{6}$

drive). It must be noted that Pickett's curve is based on Poisson's ratio (μ) equal to $\frac{1}{6}$ which is used commonly for concrete. This value of μ differs from those obtained for some of the specimens: however, actual calculations showed that these variations in the values of μ have little effect in the values of C; it was therefore decided to use Pickett's curve without correcting for the variation in μ.

Dimensions of the specimens (a, b, and L) and the ratio $\frac{L}{a}$ and $\frac{L}{b}$ are given in Table 44. Values of C x z corresponding to these ratios are given in Table 46 (Appendix VI). Two sets of E calculations were made for each specimen corresponding to vibrations in b and a directions respectively.

Resonant frequencies (n) of the specimen and their squared values for both directions of the vibration are given in Table 47 while weights (W) of the specimens are given in Table 45 (both tables are in Appendix VI).

Final calculations for equation (7) were made in Table 14. Young's modulus of elasticity, when the specimen is driven in b direction, is called E_b, and E_a when the specimen is driven in the direction of a. Their average E (column 9 of Table 14) is considered as representing Young's modulus (dynamic) for the specimen tested.

b) Calculation for Modulus of Elasticity in Shear. Equations for the calculation of the dynamic modulus of elasticity in shear were previously discussed on pages 106 and 107. These are equations (12), (13), and (14) which will be repeated here for convenience:

$$G = B\ W\ (n')^2 \qquad (12)$$

$$B = \frac{4\ L\ R}{g\ A} \qquad (13)$$

and

$$R = \frac{\frac{a}{b} + \frac{b}{a}}{4\frac{a}{b} - 2.52\left(\frac{a}{b}\right)^2 + 0.21\left(\frac{a}{b}\right)^6} \qquad (14)$$

and as previously given: W = weight in pounds, n' is the torsional resonant frequency in cycles per second, g is acceleration of gravity (386.4 in. per sec^2), a and b are sectional dimensions in inches (b larger than a), A = a x b, in^2 and L is the length of the specimen in inches.

Calculations of R are started in Table 44 where a, b, and the ratios $\frac{a}{b}$ and $\frac{b}{a}$ are given and are completed in Table 48. In these calculations the last term of the denominator in equation (14) is omitted because its effect is negligible. Coefficients B for all the specimens were calculated, by equation (13), in Table 49.

Resonant frequencies (n') are given in Table 47, columns 6 and 7 for the two directions of drive, a and b. It may be noticed that the values in columns 6 and 7 of each specimen are equal (see page 255).

Final calculations of (G) by equation (12) are given in Table 15.

c) Calculations of Poisson's Ratio. Poisson's ratios are calculated from previously given equation (15) which is as follows:

$$\mu = \frac{E}{2G} - 1$$

These calculations are given in Table 15. Values of the modulus E for these calculations are taken from Table 14, column 9.

TABLE 14. CALCULATIONS FOR YOUNG'S MODULUS OF ELASTICITY (DYNAMIC)

1	2	3	4	5	6	7	8	9
		Drive in b direction			Drive in a direction			
Specimen No.	Weight W Pounds	n^2 $\left(\frac{\text{Cycle}}{\text{Sec}}\right)^2$	C $\frac{\text{Sec}^2}{\text{in}^2}$	E_b ksi	n^2 $\left(\frac{\text{Cycle}}{\text{Sec}}\right)^2$	C $\frac{\text{Sec}^2}{\text{in}^2}$	E_a ksi	Average E ksi
1C51	6.84	1,404,000	.106	1014	714,000	.187	912	963
1C52	6.70	1,440,000	.107	1033	810,000	.193	1046	1040
2CES51	7.39	1,626,000	.105	1263	865,000	.186	1191	1227
2CES52	6.92	1,690,000	.113	1323	828,000	.225	1288	1305
3C51	6.24	1,346,000	.115	966	706,000	.226	995	981
3C52	6.31	1,428,000	.113	1016	706,000	.215	957	986
3SG51	10.63	2,592,000	.108	2962	1,440,000	.190	2910	2936
3SG52	9.76	2,723,000	.114	3034	1,276,000	.223	2778	2906
4SG51	11.03	3,610,000	.102	4077	1,742,000	.203	3896	3987
4SG52	10.85	3,276,000	.100	3568	1,513,000	.192	3152	3360
5CS51	7.83	1,769,000	.107	1479	960,000	.199	1498	1489
5CS52	7.98	2,074,000	.104	1726	1,188,000	.187	1771	1748
6ES51	8.38	2,045,000	.107	1830	1,103,000	.198	1827	1829
6ES52	8.21	1,960,000	.107	1727	1,082,000	.201	1780	1754
7W51	8.64	2,624,000	.105	2374	1,513,000	.182	2376	2375
7W52	7.74	2,592,000	.115	2315	1,416,000	.236	2581	2448
8CEL51	6.95	1,960,000	.102	1389	1,000,000	.197	1371	1380
8CEL52	6.37	1,823,000	.108	1255	846,000	.229	1237	1246
9S51	9.53	2,465,000	.109	2558	1,254,000	.201	2404	2481
9S52	9.26	2,657,000	.110	2711	1,346,000	.207	2574	2643
10CA51	6.92	1,513,000	.112	1173	764,000	.218	1150	1161
10CA52	6.62	1,346,000	.114	1015	723,000	.227	1087	1051

TABLE 15. CALCULATIONS FOR MODULUS OF ELASTICITY IN SHEAR, G, AND POISSON'S RATIO, μ, (DYNAMIC)

1	2	3	4	5	6	7	8
Specimen No.	Weight W Pounds	$\left(\frac{(n')^2}{\text{Cycle}}{\text{Sec.}}\right)^2$	$\left(\frac{B}{\text{Sec}}{\text{in}}\right)^2$	G ksi	$E_{average}$ ksi	$\frac{E}{2G}$	$\frac{E}{2G} - 1 = \mu$
1C51	6.84	2,560,000	.0236	413	963	1.16	.16
1C52	6.70	2,756,000	.0242	447	1040	1.16	.16
2CES51	7.39	2,890,000	.0237	506	1227	1.21	.21
2CES52	6.92	2,890,000	.0267	534	1305	1.22	.22
3C51	6.24	2,372,000	.0266	394	981	1.25	.25
3C52	6.31	2,496,000	.0257	405	986	1.22	.22
3SG51	10.63	4,928,000	.0238	1247	2936	1.18	.18
3SG52	9.76	4,840,000	.0263	1273	2906	1.14	.14
4SG51	11.03	5,808,000	.0249	1595	3987	1.25	.25
4SG52	10.85	5,290,000	.0241	1383	3360	1.22	.22
5CS51	7.83	3,312,000	.0246	638	1489	1.17	.17
5CS52	7.98	3,960,000	.0237	749	1748	1.17	.17
6ES51	8.38	3,842,000	.0246	792	1829	1.16	.16
6ES52	8.21	3,725,000	.0248	758	1754	1.16	.16
7W51	8.64	4,973,000	.0233	1001	2375	1.19	.19
7W52	7.74	4,601,000	.0273	972	2449	1.26	.26
8CEL51	6.95	3,534,000	.0246	604	1380	1.14	.14
8CEL52	6.37	2,958,000	.0272	513	1246	1.21	.21
9S51	9.53	4,494,000	.0247	1058	2481	1.17	.17
9S52	9.26	4,731,000	.0253	1108	2643	1.19	.19
10CA51	6.92	2,789,000	.0262	506	1161	1.15	.15
10CA52	6.62	2,560,000	.0267	452	1051	1.16	.16

C. Static Method.

Elastic constants were also determined statically by subjecting the specimen to known compressive stresses and measuring the corresponding axial (and for some specimens also lateral) strains by resistance wire strain gages. The elastic constants were calculated from these measurements.

1. Test Apparatus.

In addition to the compression machine, the test apparatus consisted of the gages, strain indicator and a switch box. The test machine can be operated to hold the load constant while the strain is determined.

a) Strain Gages. The variable resistance wire strain gage known as Baldwin SR-4 Strain Gage was used for this test. These gages are made of different patterns, sizes and materials, but type A-1 was used because of suitable size and adequate sensitivity. It is flat grid of thin wire mounted on paper base and covered with felt. This gage has a nominal gage length (length of grid) equal to $\frac{13}{16}$ inch and grid width of $\frac{9}{64}$ inch. The gage length was adequate since (see Table 4, Chapter III) maximum sizes of aggregate were usually $\frac{3}{8}$ inch. The grid wire is constantan (copper-nickel alloy) which has a gage factor (ratio of unit resistance change, ohms per ohm, to unit strain change inch per inch in the wire) of 2.07 and a resistance of 120 ohms.

The operation of the SR-4 gage was as follows: The gage was cemented firmly to the specimen (methods will be discussed later), direct electric current was passed through the gage, known compressive force was then applied, causing the specimen and the gage to

undergo a certain strain. Change of the electrical resistance of the wire is proportional to the strain and was measured by a specially designed Wheatstone bridge.

During the test a specimen may suffer an additional strain caused by temperature or moisture changes. To compensate for these unknown additional strains a similar gage attached to a duplicate specimen was added to the circuit. This so called "dummy" specimen was then placed as close as possible to the tested "active" specimen so it would be under the same atmospheric conditions but not subjected to statical stresses.

b) Strain Indicator. A specially designed Wheatstone bridge (Baldwin portable strain indicator, type MB, Figure 22 at middle) was used for this test. The instrument is calibrated to read directly in micro-inches per inch, with a measuring range of 2000 mic in per in. A ten-step switch extends the range another 1000 mic in. per in. for each step. A simplified Wheatstone bridge circuit is shown in Figure 24.

c) Switch Box. A switch box was used as auxiliary equipment to facilitate the selection of gages during the test. The available instrument was Baldwin's SR-4 twenty point switching unit which was used although no more than two active gages were recorded during the test. This instrument (Figure 22, at left) has 20 poles for active gages and the same number of compensating gages. It is switched manually and provides no means of initially balancing the individual gages.

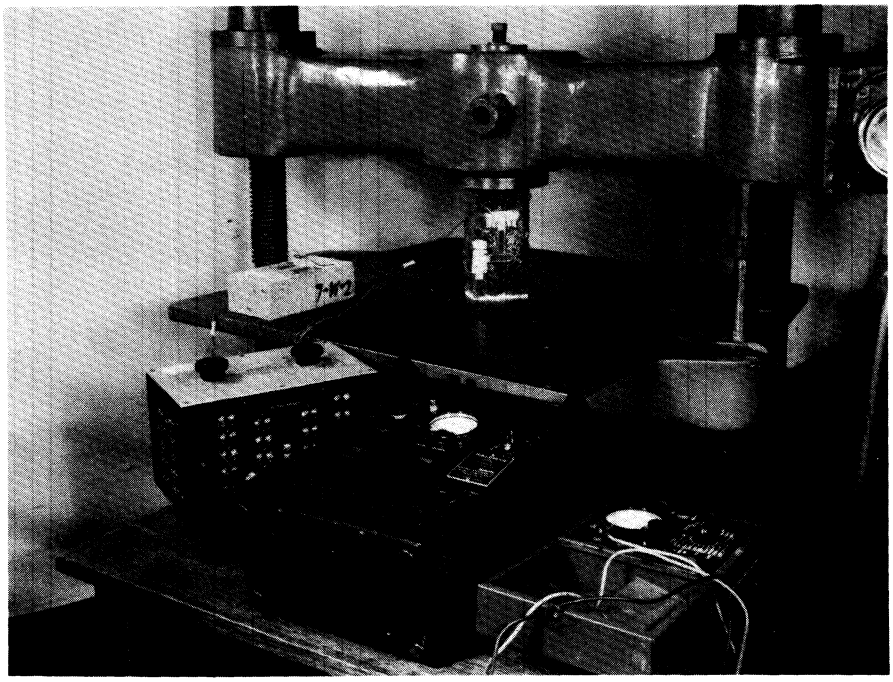


Figure 22. Static Test Apparatus for Determining the Elastic Constants (Close-up).

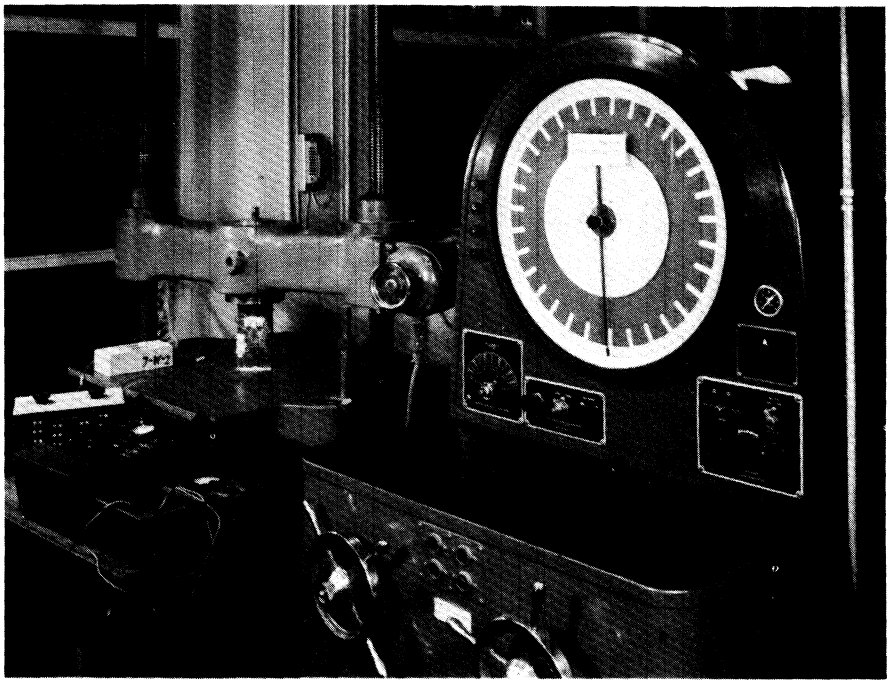
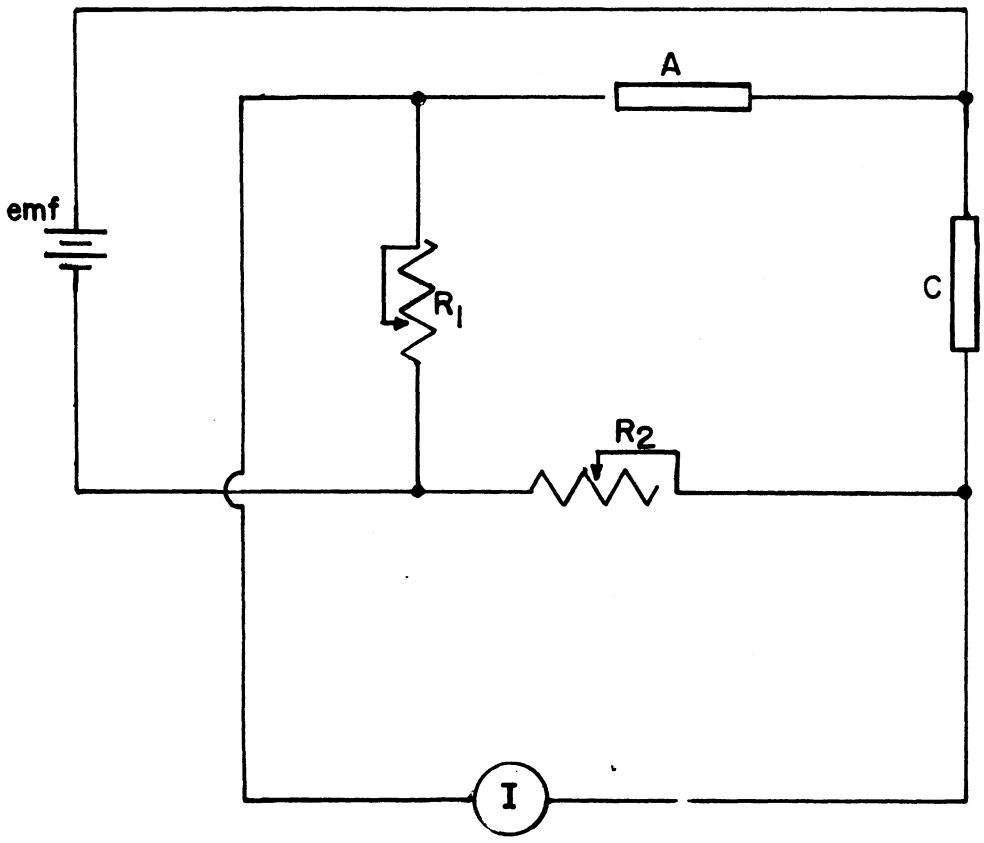


Figure 23. Static Test Apparatus (General View).



A= ACTIVE GAGE

C= COMPENSATING GAGE

I= STRAIN INDICATOR

R_1 & R_2 =RESISTORS

emf= BATTERY (D.C.)

FIG. 24 SCHEMATIC DIAGRAM OF SIMPLIFIED WHEATSTONE BRIDGE CIRCUIT

2. Preliminary Investigations.

Because of the cellular characteristic of lightweight aggregate products, further preparation of the surfaces of the specimen before cementing the gages was necessary. Preliminary investigations were conducted on companion specimens to those used for the final tests to develop a suitable method. Specimens were taken from three different products: sand and gravel (3SG5), air-cooled slag (9S5), and cinders plus expanded slag (2CES5). Three methods of surface treatment were investigated, filling the surface cavities with (1) Duco cement, (2) plaster of Paris and (3) neat portland cement paste or sand-cement mortar.

In the first method surface cavities were filled by applying several layers of thinned Duco cement. Each layer consists of applying a liberal amount of this cement and leaving it to dry before the succeeding layer is added. For specimen 2CES5 as many as eight layers were used with this method.

In the second method a small quantity of slightly thinned paste of plaster of Paris was pressed into the cavities and the excess paste removed. The specimen was aged for 24 hours before grinding the surface carefully so that all the plaster was removed except that which was in the cavities.

In the third method the cavities were filled with neat portland cement paste on one face of the specimen and with sand-cement mortar on the second face. This was done by the same procedure used in the second method, including the curing for 24 hours before grinding the surface.

Early in this investigation, the third method was proven unsatisfactory because of excessive shrinkage and difficulty of grinding away the excess material. The first method was found time consuming. The second method was adopted after a comparative study was made of the performance of gages cemented to faces of specimen that had been prepared by this method and the first method. This latter study was conducted on specimens taken from the product 3SG5, 9S5 and 2CES5. Two opposite faces were prepared by the first method while the two remaining faces were prepared by the second method. Gages of the two opposite faces were then connected in a series and the specimen was tested in compression. The results (presented in detail in Appendix VI) of the two methods of preparing the surface were very similar.

3. Procedure of Test.

The procedure that was finally used for preparing the surfaces of lightweight aggregate test specimen was similar to the second method described above, except that the surface of the specimen to which the gages were to be cemented had first been smoothed by a commercial grinding machine. Plaster of Paris was then applied to the surface as in the second method, this time the pre-smoothed surface was easily cleaned of the excess paste. The specimen was then aged for 24 hours before grinding with a grinding belt. This second grinding was a necessary step to roughen the surface for better gage binding and to further clean plaster from the surface outside the cavities.

Surface preparation by the method above was found unnecessary for specimens made with sand and gravel (products 3SG5 and 4SG5) since their surfaces are free of large cavities.

The following procedure was used for cementing the gages to the test specimen: the surface was cleaned thoroughly with acetone, a thin layer of Baldwin SR-4 precoat was applied and left to dry for about 1/2 hour, finally Duco cement was applied and the gages placed on their predetermined location (vertical gage at center of the face of the specimen, horizontal gage directly under and perpendicular to the vertical gage, see Figure 25) and pressed evenly for 15 minutes after which a weight of approximately one pound was applied through a rubber pad and held constant for 12 hours. Gages on the opposite face of the specimen were applied by the same procedure.

About half of the specimens had vertical and horizontal gages (in both faces) while the remaining specimens had only vertical gages. Each gage was connected with its companion gage on the opposite face of the specimen in series, so that errors due to possible buckling of the specimen under load were eliminated.

During the test an active specimen was centered under the load applying block at the head of the machine (see Figure 22) while the dummy was placed on the bed of the machine close to the active specimen. Wire leads of the gages were then connected directly to the strain indicator when the specimen was bearing vertical gages only, otherwise connections were made through the switch box where the vertical and horizontal gages were connected to the active points 1 and 2 respectively while dummy gages were connected to both points 1 and 2 at dummy poles of the switch. Initial balances of the bridge were then accomplished for the gage. Load was applied at uniform speed and held constant at 1000 pound intervals until the indicator was balanced again for the gage (s). Tests were stopped at 8000

pounds for specimens made with lightweight aggregates and 14,000 pounds for specimens made with dense aggregates.

4. Calculations.

Detailed data and calculations of the results are given in Tables 53 to 73 and Figures 51 to 71 (in Appendix VI) along with the stress for vertical and horizontal (if present) gages for all specimens tested except 1C52 which were presented here (Table 16 and Figure 26) for purpose of discussion.

TABLE 16. STRESS-STRAIN DATA (STATIC) FOR 1C52 (CINDERS)

1	2	3	4	5	6
Load Pounds	Stress psi	Vertical Gage		Horizontal Gage	
		Reading mic-in	Strain mic-in	Reading mic-in	Strain mic-in
0	0	[5]1305	0	[6]1370	0
1000	123	1180	125	1380	-10
2000	246	1050	255	1400	-30
3000	369	920	385	1420	-50
4000	491	[4]1790	515	1440	-70
5000	614	1650	655	1460	-90
6000	737	1515	790	1485	-115
7000	860	1380	925	1510	-140
8000	983	1245	1060	1536	-166

In Table 16 compressive stresses are given in column 2 while the corresponding strains are listed in columns 4 and 6 for vertical and horizontal gages respectively. From these, information stress-strain curves for the vertical and horizontal gages were

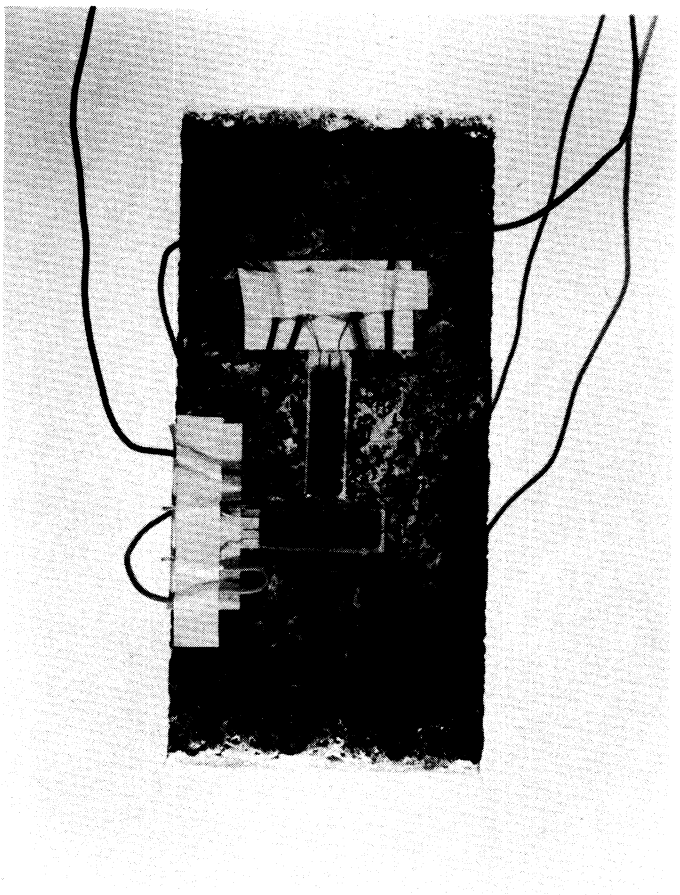


Figure 25. Static Test Specimen with SR-4 Gages.

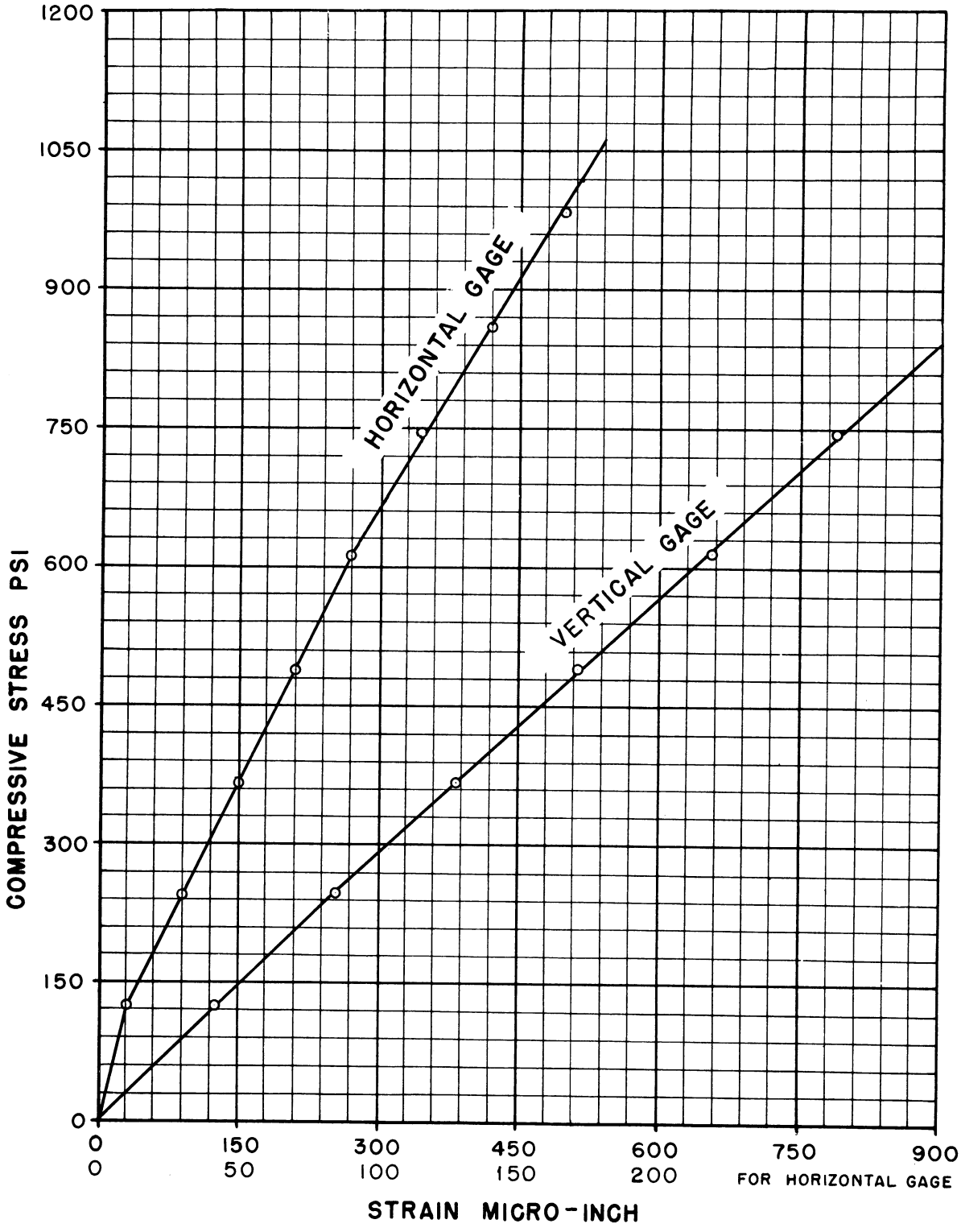


FIG. 26 - STRESS-STRAIN CURVES FOR IC52

plotted in Figure 26. Young's modulus was then calculated as the slope of the initial tangent (straight portion) of the stress-strain curve of the vertical gage. Poisson's ratio (μ) was calculated as the average ratio of the horizontal and vertical unit strains corresponding to the straight portions of the curves. Values near zero stress are not reliable and were excluded from the calculations. (See also discussion in Appendix VI). Modulus of elasticity in shear was then calculated from the previously given equation (15). Results of all specimens were calculated in the same manner and summarized in Table 17.

D. Discussion of the Results of Elastic Constants as Determined by Dynamic and Static Methods.

Average results of the static and dynamic tests are summarized in Table 18. Each value is the average result of two specimens except values of μ and G of the static test which are mostly the results of one specimen (see Appendix VI).

The most apparent feature in studying these results is that the values of the static tests are lower (some are appreciably so) than those of the dynamic tests. This has been observed by almost all other investigators (see reference 19, 39, 40) and it was concluded (reference 40) that the same properties of concrete are not measured by the two tests. It is also true that in a material like concrete, with less homogeneity and isotropy, that the theoretical equations used in the calculation of the results of the dynamic test and some of the static test are only approximately applicable. This, coupled with some possible creep in the static test, makes a difference in the results of both tests expected. This is why it was suggested by some investigators[40], as was discussed at the beginning of this

TABLE 17. ELASTIC CONSTANTS (STATIC TEST)

1	2	3	4	5
Specimen No.	Young's Modulus E ksi	Poisson's Ratio μ	$2(1+\mu)$	Modulus of Elasticity in Shear $G = \frac{E}{2(1+\mu)}$ ksi
1C51	908	.161	2.32	391
1C52	920	.150	2.30	400
2CES51	1013	.188	2.38	426
2CES52	1020			
3C51	784	.167	2.334	336
3C52	880	.150	2.300	383
3SG51	2333			
3SG52	2133			
4SG51	3357	.240	2.48	1354
4SG52	2765			
5CS51	1056	.171	2.34	451
5CS52	1593			
6ES51	1705	.133	2.266	752
6ES52	1687			
7W51	2200	.154	2.308	953
7W52	2061			
8CEL51	1388			
8CEL52	1109			
9S51	2100	.130	2.26	929
9S52	2416			
10CA51	797	.126	2.252	354
10CA52	984			

chapter, that the dynamic test may not be suitable for determination of constants to be used in the design of static structures.

Any of the two sets of the elastic constants are suitable for a comparative study of the products tested; the results of the

TABLE 18. AVERAGE RESULTS OF ELASTIC CONSTANT FROM THE STATIC AND DYNAMIC TESTS

1	2	3	4	5	6	7	8
		Dynamic Test			Static Test		
Specimen No.	Aggregate	Young's Modulus E ksi	Modulus of Elasticity in Shear G ksi	Poisson's Ratio μ	E ksi	G ksi	μ
1C5	Cinders	1002	430	.16	914	396	.16
2CES5	Cinders & Expanded Slag	1266	520	.22	1017	426	.19
3C5	Cinders	984	400	.24	832	360	.16
3SG5	Sand & Gravel	2921	1260	.16	2233	-	-
4SG5	Sand & Gravel	3674	1489	.24	3061	1354	.24
5CS5	Cinders & Sand	1619	694	.17	1325	451	.17
6ES5	Expanded Slag	1792	775	.16	1696	752	.13
7W5	Waylite	2412	987	.23	2131	953	.15
8CEL5	Celocrete	1313	559	.18	1249	-	-
9S5	Air-Cooled Slag	2562	1083	.18	2258	929	.13
10CA5	Cinders & Fly Ash	1106	479	.16	891	354	.13

dynamic test will be used, however, because they are the average of two specimens. Returning to Table 18, it may be observed that products made with dense aggregates have higher results than those made with lightweight aggregates. Of the former, 4SG5 (sand and gravel) has the maximum results of E = 3674 ksi (kips per sq. in.) and G = 1489 ksi.

Other dense aggregates, 3SG5 (sand and gravel) and 9S5 (air-cooled slag), have the results of E = 2921 ksi and G = 1260 ksi and E = 2562 ksi, G = 1083 ksi respectively. Lightweight aggregates 7W5 (Waylite) has the highest results of E = 2412 ksi while 3C5 (all cinders) has the lowest value of E = 984 ksi.

Other studies include correlation of Young's modulus with dry weights, compressive and flexural strengths. For these, values determined by the static tests were used because the specimen in these tests were also subjected to high stress. Data presented in Table 19, columns 3, 4, 5, and 6 were taken from Tables 18, 8, 10, and 12 respectively. It can be observed in Table 19 that (1) rough correlation exists between compressive strength and Young's modulus. This correlation is better between products of similar aggregates (cinders, expanded slags....), (2) correlation also exists between Young's moduli and unit dry weights of the products. This is also obvious in Figure 27, (3) rough correlation also exists between modulus of rupture (R) and Young's modulus (E) as may also be seen in Figure 28.

Another observation pertains to Poisson's ratio as related with the intensity of the stresses. As may be seen from the stress-strain curves of the horizontal gages, Poisson's ratio is higher at relatively high compressive stresses. This was also observed by other investigators.[41]

TABLE 19. COMPARATIVE STUDY OF YOUNG'S MODULUS (STATIC) WITH OTHER CONCRETE PROPERTIES

1	2	3	4	5	6
Specimen No.	Aggregate	Young's Modulus (Static) E ksi	Dry Wt. pcf.	Compressive Strength psi	Modulus of Rupture psi
1C5	Cinders	914	90.9	1357	272
2CES5	Cinders & Expanded Slag	1017	93.1	1778	224
3C5	Cinders	832	83.4	1252	354
3SG5	Sand & Gravel	2233	132.8	2212	510
4SG5	Sand & Gravel	3061	129.2	1983	635
5CS5	Cinders & Sand	1325	97.1	1728	344
6ES5	Expanded Slag	1696	101.3	1395	338
7W5	Waylite	2131	97.3	1506	334
8CEL5	Celocrete	1249	76.2	1199	193
9S5	Air-Cooled Slag	2258	126.3	2195	526
10CA5	Cinders & Fly Ash	891	85.2	1663	277

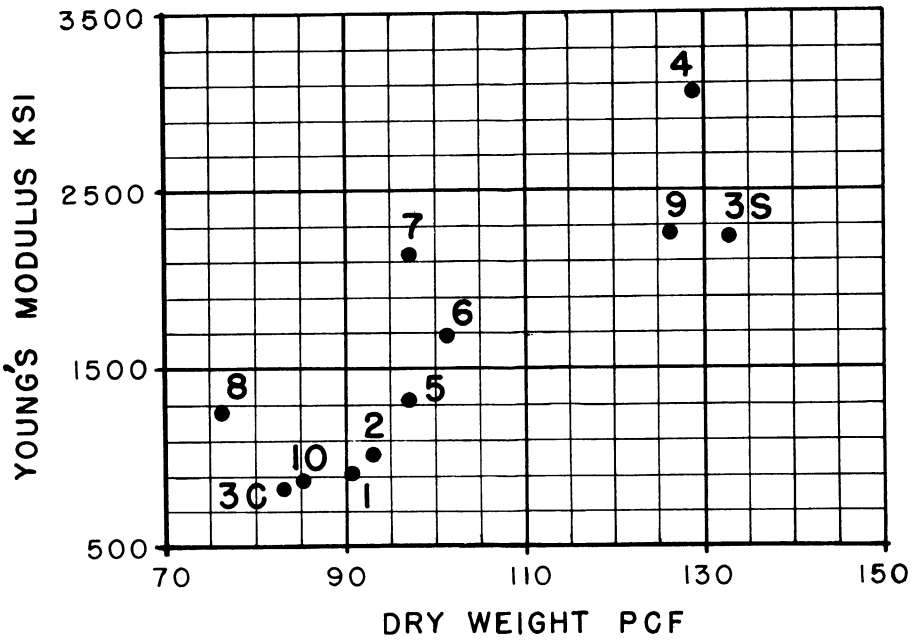


FIG. 27 – YOUNG'S MODULUS (STATIC) VS. UNIT DRY WEIGHT

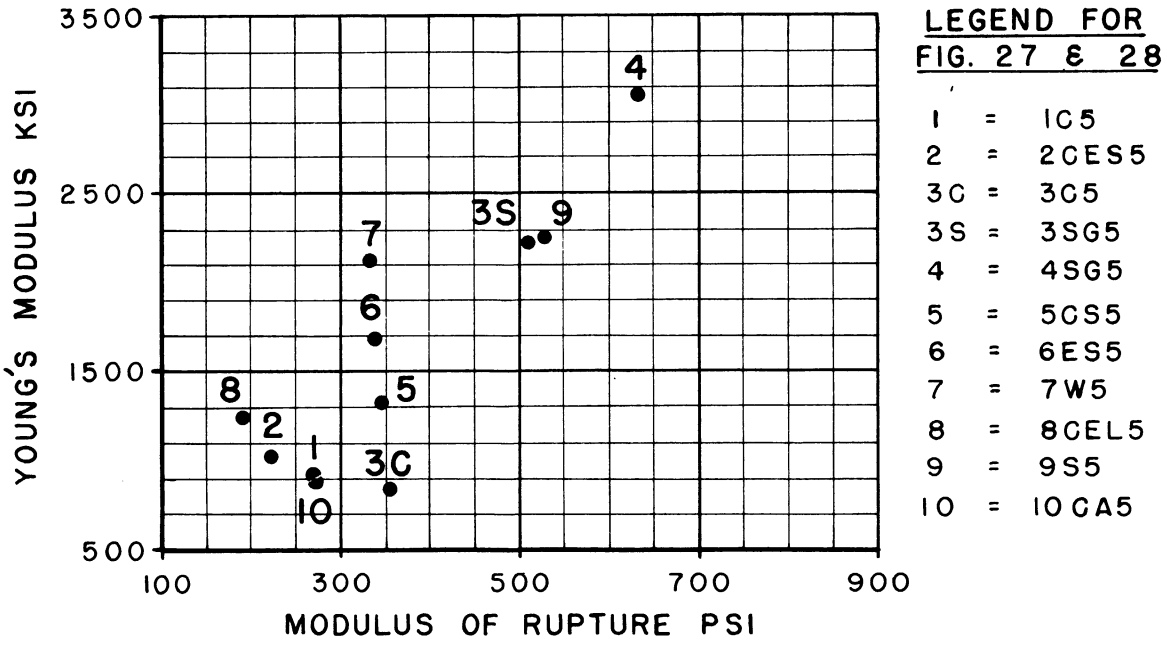


FIG. 28 – YOUNG'S MODULUS (STATIC) VS. MODULUS OF RUPTURE

CHAPTER IX

FREEZING AND THAWING DURABILITY

Repeated freezing and thawing is one of the most destructive agencies to which concrete is exposed. In some climates, concrete masonry may be exposed to many cycles of freezing and thawing annually. The severity of the exposure depends on the seasonal temperature changes and the location of masonry units in the structure. Units, for instance, near moisture at the ground level or in parapet walls may suffer a severe exposure.

Investigators, realizing the extent of damage caused by this agency, have attempted to solve the problem of predicting the freeze-thaw durability of concrete. Most of their progress was accomplished in the last decade by extensive research, generally aimed at either investigating the mechanism of the forces of deterioration and conditions of their action such as the work of Powers[43] ..., or at developing various methods and apparatus for laboratory accelerated freezing and thawing and conducting tests to determine the comparative resistance of various concretes[44,45] ..., or at determining the resistance by exposing concrete to long-time weathering at different climates and in different conditions. Attempts were also made to correlate the results of accelerated tests with those of natural weathering.[46,47]

These investigations have indicated that damage, originating in the paste or the aggregate, or both, is mainly from stresses developed by rise of the internal pressure during freezing of the retained water. It may also be caused by thermal stresses due to sharp rise

and fall in temperature; or by expansive action of deleterious aggregate particles.

The mechanism of frost action in the paste is adequately treated by Powers[43] in his theory of hydraulic pressure and water diffusion. Cement paste is capable of holding large quantities of freezable water in its capillary structure. This water produces hydraulic pressure upon freezing, and is caused by the resistance of this fine capillary structure to the flow of water displaced by the advancing ice front. Pressure may also be caused by the growth of the ice body in the capillary structure by diffusion of gel water at lower temperatures than at which the ice bodies were originally formed. Both kinds of pressure can produce destructive expansion of concrete although the latter acts more slowly. Presence of air voids relieves these pressures provided they are closely and uniformly distributed so that the displaced water can readily move to them. This explains the role of air-entrainment in improving the resistance of concrete to freezing and thawing.

For concrete masonry units made with lightweight aggregates another factor may affect their resistance decisively. It was noticed that all the products tested in this study contain a large percentage of open air voids aside from those contained by the aggregates. This is apparently because the mixes are designed to result in highly cellular concrete, thus promoting light weight. This condition however, permits the concrete to retain a large quantity of freezable water in some cases (as may be seen from Tables 6 and 9, Chapters III and V respectively), thus making it very vulnerable to severe cycles of freezing and thawing such as those employed by the accelerated test.

Coarse aggregate was also shown to play an important part in the overall freeze-thaw durability of concrete.[48] Concrete made with aggregate that is structurally weak, possessing high rate of absorption or containing deleterious particles may exhibit poor performance. Rapid drop and rise of temperature in accelerated freeze-thaw test might develop destructive thermal stresses, especially when the thermal incompatibility of the paste and the aggregate is large.

A short time before the present tests were conducted, the Michigan State Highway Department Laboratory at Ann Arbor finished building a new accelerated freeze-thaw apparatus and have since been carrying on a rather large program of tests to evaluate the durability of concrete made with the different aggregates used in Michigan. The author was fortunate enough to secure space in the chamber for the specimens of his tests. A brief description of this apparatus will follow later. The cycle employed consists of 3 hours of freezing in air at 0 F, followed by 1 hour of thawing in water at 40 F. This procedure was designed to expose the aggregates in concrete to the more severe conditions of the test[52] and proved unsuitable for the lightweight concretes. Concrete specimens made with most of these deteriorated after a few cycles. This is believed to be caused mainly by the absorption of the highly cellular concrete containing the lightweight aggregates and their capacity of retaining large quantities of freezable water as was discussed previously.

Since it has been intended to study only the comparative performance of the products made with dense and lightweight aggregates, it was decided to modify the preceding conditions by keeping the moisture content of the specimen constant during the test. This was accomplished by sealing the specimens in tight copper jackets.

Question arose as to the proper moisture content to be employed. The actual moisture content of concrete masonry in normal

service is presently indeterminate, and, of course, variable. However, the average of the natural moisture contents of all the products, as sampled from lots stored in open yards, was about 37 per cent of the 24 hour saturation (see Table 9, Chapter V). Sampling was done in the relatively wet season. Thus it was decided to choose a moisture content equal to 40 per cent of the 24 hour saturation of the particular specimen. The chosen moisture content is also equal to the maximum allowed by the ASTM specifications for units delivered to the construction site.

A. Specimen.

Test specimens were beams of approximate dimensions of 2.2 x 3.2 x 15.6 inches, cut from solid masonry units of the modular dimensions of 3-5/8 x 7-5/8 x 15-5/8 inches (see Chapter II).

Absorption of each specimen after 24 hours submersion was determined by the same procedure that has been described in Chapter V. The dried specimens were wrapped tightly in a thin (No. 30 gage) copper foil which was then sealed by solder except at one end where a quantity of water, equal to 40 per cent of the 24 hour saturation absorption, was injected just before this end was also sealed. The jacket was tested by submerging the specimen in water for 24 hours. Increase in weight indicated leakage in which case the jacket was removed, the specimen dried and equipped with a new jacket.

B. Apparatus.[52]

Test apparatus consists of freezing equipment, thawing equipment and the specimen chamber. As is shown in the simplified diagram of the apparatus (Figure 29) the specimen chamber is located inside the cold room. The well insulated room is about 12 x 10 feet in size and continuously kept at a temperature of 0 F to -5 F. The specimen

chamber is open at the top and is placed under the cooling fans where it is subjected directly to the cold air circulation. Thawing water is stored in a tank, located just outside the cold room at a lower level, at a temperature of approximately 42 F. During the thawing cycle the water is circulated to the specimen chamber by a centrifugal pump. The loss in temperature of the thawing water is small because the capacity of the tank is much larger than that of the specimen chamber and is corrected in the thawing tank by five immersion heaters. The apparatus is completely automatic. The freezing equipment circuit is separate from that for thawing. The apparatus performs within the requirements of ASTM Designation C291-52T (Tentative Method of Test for Resistance of Concrete Specimen to Rapid Freezing in Air and Thawing in Water).[22]

C. Procedure of Test.

The specimen was first cooled to 40 F and then weighed; its fundamental resonant frequency of transverse vibration was determined using the sonic apparatus and method described in Chapter VIII. However, preliminary investigation had indicated that accurate determination of the sonic modulus is difficult when the drive and pickup were made through the copper jacket. This difficulty was overcome by equipping the specimen with brass plugs at these points. Plugs were cemented in holes drilled in the specimen. The specimen was then moist cured for 24 hours and air cured for one week (in normal room conditions) before drying in the oven prior to wrapping in copper foil. The plugs extended about 1/8 inch beyond the surface of the specimen, through the copper jacket (see Figure 31) and were suitable for transmitting the drive and pickup. Test showed that the presence

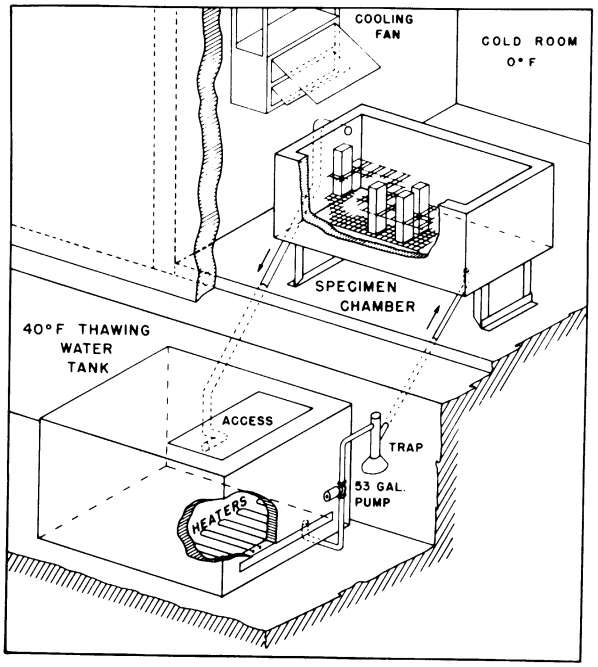


Figure 29. Schematic Diagram of Freezing and Thawing Apparatus. Courtesy of the Michigan State Highway Department.

Figure 30. Specimens Chamber of the Freezing and Thawing Apparatus.

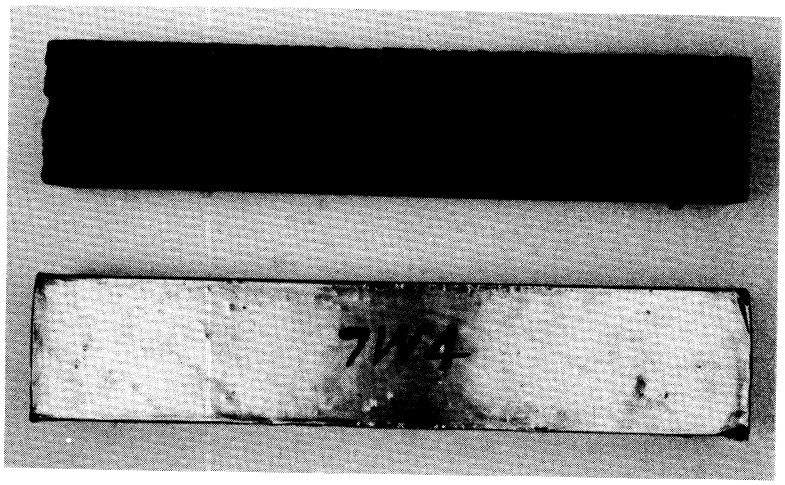


Figure 31. Freezing and Thawing Durability Test Specimen Before and After Copper Jacket is Added.

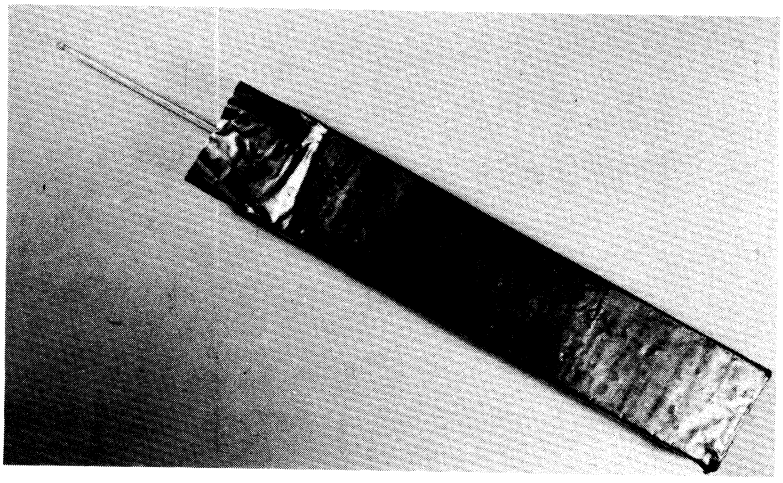


Figure 32. Control Specimen for the Freezing and Thawing Test.

of the copper jacket and the brass plugs affected the frequency reading but slightly because their masses are small compared with that of the specimen. The resonant frequencies of the specimen thus determined at 40 F was considered as a measure of its initial Young's modulus of elasticity.

After the initial resonant frequencies of the test specimen were determined it was placed in the specimen chamber at the beginning of the thawing cycle. Each specimen was placed vertically in a grid compartment so that it was surrounded by uniform air space (see Figure 30).

As was previously indicated, the apparatus is automatic and the freeze-thaw cycles were continued without interruption (except when sonic readings were made), until the termination of the test at the end of 500 cycles. Resonant frequencies and weight of the specimen were measured at the end of 6, 12, 18, 30 and every additional 24 cycles thereafter. Specimens showing signs of deterioration were measured more frequently. In all measurements, the specimens were taken and returned to the chamber during a thawing cycle.

D. Discussion of Test Results.

Results of the test are given in Table 21; additional data in Table 20. In Table 21 the durability of the specimen is measured by the effect of the freezing test on Young's modulus and the modulus of rupture of the specimen. These results are based on the alternate freezing to 0 F and thawing at 40 F maintaining a constant moisture content (40 per cent of 24 hours saturation) in the specimen. Two control specimens, one of which was made with sand and gravel and the other with expanded slag, were used to measure the change in

temperature at their centers during the cycle. The temperature of the lightweight aggregate specimen dropped from 40 F at the start of the freezing cycle to 32 F in 20 minutes and 0 F in 140 minutes. It then remained at 0 F to -2 F for the last 40 minutes of the freezing cycle. The temperature climbed rather rapidly at the beginning of the thawing cycle, reaching 32 F in 35 minutes and to 40 F in about 50 minutes from the start of the thawing cycle. The time-temperature relation of the dense aggregate specimen followed the same pattern except it reached the indicated temperatures in slightly shorter time.

Detailed data of the sonic and weight measurements which were made regularly during the test, as described in part C, need not be presented here. Weight measurements were made to insure against the leakage of the copper jacket; initial weights, given in column 8 of Table 20, remained constant during the test.

Effect of a number of cycles of test on the specimen is evaluated by its Young's modulus at the end of these cycles (E_c) which was determined as a percentage of the initial modulus (E_o). Such values (of $\frac{E_c}{E_o} \times 100$) are given for the test specimens in columns 3 and 4 of Table 21 for the number of cycles indicated in column 2. Actual calculations of the values of Young's modulus are not necessary for these calculations since it is directly proportional to the n^2 value (n being the resonant frequency of transverse vibration. See equation (7) in Chapter VIII). The initial values of n (copper jacketed specimen) are given in columns 2 and 3 of Table 20 for drive in the directions of a and b respectively (a and b are dimensions of the section of the specimen, in the same manner used in

TABLE 20. FREEZING AND THAWING DURABILITY - INITIAL YOUNG'S MODULUS AND WEIGHT MEASUREMENTS

1	2	3	4	5	6	7	8
	Initial Resonant Frequency-Copper Jacketed Specimen		Initial Modulus of Elasticity (E_o) of Concrete		Weight of Specimen		
Specimen No.	n_a Cycles per sec.	n_b Cycles per sec.	Individual ksi	Average ksi	Dry Weight gms.	Wet Weight gms.	Test Weight gms.
1C81	775	1100	1271	1308	2378	2726	2688
1C82	790	1100	1344		2256	2634	2790
2CES81	790	1100	1438	1453	2618	2944	2982
2CES82	810	1110	1467		2647	2974	2964
3C81	840	1100	1109	1116	2940	3528	3498
3C82	840	1110	1123		2950	3555	3510
3SG81	1100	1550	3772	3656	2633	3875	3989
3SG82	1050	1550	3539		2664	3963	4056
4SG81	1100	1550	3875	4285	3697	3948	4101
4SG82	1180	1700	4695		3853	4057	4223
5CS81	825	1150	1603	1697	2676	2954	3047
5CS82	840	1200	1791		2802	3088	3207
6ES81	985	1330	2517	2511	3088	3412	3461
6ES82	1010	1340	2504		2969	3280	3330
7W81	1050	1450	2918	2761	2940	3170	3455
7W82	1010	1410	2603		2925	3190	3297
8CEL81	800	1180	1448	1560	2426	2733	2767
8CEL82	900	1250	1672		2339	2642	2682
9S81	995	1390	2824	3000	3312	2586	3684
9S82	1030	1400	3175		3445	3736	3969
10CA81	810	1120	1462	1356	2531	2889	2907
10CA82	775	1050	1249		2367	2737	2764

TABLE 21. FREEZING AND THAWING DURABILITY - EFFECT OF TEST ON YOUNG'S MODULUS AND MODULUS OF RUPTURE

1	2	3	4	5	6	7
		Per Cent of Initial Modulus of Elasticity = $\frac{E_c}{E_o}$ x 100		Modulus of Rupture "R"		
Specimen No.	Number of Cycles	After 500 Freeze-Thaw Cycles				"Original" Tests (No Freezing and Thawing)
		Individual per cent	Average per cent	Individual psi	Average psi	psi
1C81	500	100	98	389	423	272
1C82	"	96		456		
2CES81	"	100	100	529	523	224
2CES82	"	100		517		
3C81	"	100	99	570	567	354
3C82	"	98		564		
3SG81	"	100	100	698	694	510
3SG82	"	100		689		
4SG81	"	100	100	897	836	635
4SG82	"	99		774		
5CS81	"	94	97	419	425	344
5CS82	"	100		431		
6ES81	"	98	97	558	550	338
6ES82	"	96		542		
7W81	"	100	100	574	584	334
7W82	"	99		594		
8CEL81	"	83	88	305	272	193
8CEL82	"	92		239		
9S81	102	70	62	443	357	526
9S82	13	53		271		
10CA81	500	93	93	506	442	277
10CA82	"	93		378		

Chapter VIII). Sonic measurements were always made for both directions and the average results presented.

Examination of the results of Table 21 reveals that, generally, all specimens except those made with air-cooled slag (9S8) withstood well the exposure to the test conditions. Considering for the moment the reduction in Young's modulus as the measure of durability, it can be seen from column 4 of the table that Young's moduli at the end of 500 freeze-thaw cycles (E_c) are equal to their initial values (E_o) for products made with sand and gravel (3SG8 and 4SG8) and two of the lightweight aggregates, namely cinders plus expanded slag (2CES8) and Waylite (7W8). The rest of the lightweight aggregate products, except for Celocrete, did almost as well, producing values of E between 99 and 93 per cent of original E. Celocrete (8CEL8) averaged a lower ratio, 88 per cent. Of the dense aggregates, air-cooled slag (9S8) produced a much lower result. Young's modulus of one specimen of this product was lowered to 70 per cent of its initial value after 102 cycles while the ratio of the second specimen was 53 per cent after only 13 cycles. The significance of these results will be discussed later.

Modulus of rupture (R) is another measure of freeze-thaw durability of concrete. All specimens were broken in flexure at the end of the test. Values of R are given in Table 21. All products except air-cooled slag, showed larger values of R than those obtained earlier ("original" test in Table 21) from similar specimens, as may be seen from columns 6 and 7 of Table 21. Values of column 7 were taken from Table 12, Chapter VII. This increase is believed to be due to gain in strength during the period separating the two tests

(one year) in which the specimens were stored in normal laboratory conditions. Nevertheless, the results conform with the indications offered by Young's modulus. Air-cooled slag showed a substantial reduction (instead of increase) in its R value. The increase in strength during the storing period is also apparent in the values of Young's modulus. This can be seen by comparing the values reported in Tables 20 (column 5) and 18 (column 3, Chapter VIII). Values reported in Table 20 are taken directly after the specimens were taken from storage (they were not computed from n_a and n_b of columns 2 and 3 of the table which are the initial values for copper jacketed specimens), those in Table 18 are for similar specimens that had been tested earlier.

The causes of the relatively poor performances of the air-cooled slag and Celocrete products are not apparent. The performance of the Celocrete may partially be assigned to its relatively high absorption and inherent structural weakness. But this is not true for air-cooled slag which exhibited the poorest durability performance. Neither can the amount of absorption be the reason at constant moisture content of 40 per cent. Another reason considered is the thermal incompatibility of aggregate and mortar which is in some cases held responsible for some durability failures.[49] As will be shown later in Chapter XII, the thermal coefficient of expansion of this product is not far from that of sand and gravel products. It can also be seen in Chapter XII that the thermal coefficient of expansion is affected by the moisture condition of the specimen which complicates matters. It may also be noticed that cinder products in general have lower thermal coefficients which may indicate greater incompatibility; this, however, showed no effect on their freeze-thaw durability. The

probable reason for the poor performance of the air-cooled slag is deleterious particles in the coarse aggregate which can inflict considerable damage by expansion during freezing and thawing. Fine cracks generating radially from a point, discovered upon removal of the copper jacket of specimen 9S82, support this conclusion.

Poor durability of concrete cannot normally be assigned to a single cause. More accurate determination of the probable reasons of the performances of these products requires a larger program of tests than was possible. It was also not possible to isolate the effect of one element on the results, such as coarse aggregate, mortar, air entrainment....because, as was discussed in the opening of this chapter, the overall durability of concrete depends on the rest of its physical properties such as the amount of absorption, presence of effective air voids, elastic properties, and presence of deleterious materials....The preceding properties depend, besides the individual properties of the constituents, on the water-cement ratio and other proportioning of mix, mixing, curing....These as have been shown before (Table 3, Chapter II) and varied from one product to another.

Consideration was also given to the proper point of termination of test. It was decided, after reviewing the literature, that because of the nature of the study no important information could be obtained that warranted extending the test beyond the 500 cycles employed. It should also be noted that the results obtained by this test do not necessarily indicate the actual performance of the products under natural weathering. It is desirable that correlation with natural weathering include at least one aggregate of known

history of field performance in the accelerated freeze-thaw test. However, the comparative study of the durability of dense and lightweight aggregate products as affected by accelerated freezing and thawing permitted drawing certain broad conclusions. These conclusions are summarized in the following:

(1) These lightweight aggregate and air-cooled slag concretes of a highly cellular nature, unlike sand and gravel products, are vulnerable to a very limited number of cycles of accelerated freezing and thawing when fully saturated. This is probably because of their high absorption and capability of retaining a large amount of freezable water in their highly cellular structure as was discussed at the beginning of this chapter. When the moisture content is lower, about 40 per cent of the 24 hour saturation which is the maximum allowed by ASTM specifications for units going into masonry, the resistance of lightweight aggregate products is generally equivalent to sand and gravel.

(2) Indications are that thermal incompatibility of coarse aggregate and mortar did not affect the freezing and thawing durability materially. This conclusion is offered with reservation. It is based on the observation that products which were believed to have larger thermal incompatibility such as those made with cinders (see Chapter XII) did not exhibit poor performance. Researchers differ on the role of thermal incompatibility in durability. For instance, the Bureau of Reclamation studies[51] of concrete with large thermal incompatibility reached a conclusion comparable to that offered above, while the Waterway Experiments Station studies on concrete with similar characteristics reached an opposite conclusion.[49] It is

believed that the picture is clouded with the fact that the moisture content of the specimen affects the thermal coefficient of expansion of the mortar. A more detailed discussion on thermal incompatibility is presented in Chapter XII.

(3) Poor freeze-thaw durability performance of air-cooled slag products is probably caused by deleterious particles in the coarse aggregate. Additional tests are necessary before reaching a positive conclusion, however.

CHAPTER X

STAINING AND POPOUTS OF CONCRETE MASONRY UNITS

Staining and popouts are unwanted characteristics of concrete masonry, producing a disagreeable appearance when prevalent in a wall. Furthermore, popouts, especially when accompanied by cracking, weakens concrete and makes it more vulnerable to the destructive action of weathering.

Lightweight aggregates, as has been discussed in Chapter III, may contain certain deleterious materials that are capable of producing staining and popouts in concrete.[13,23] Stains may be produced by metallic (tramp) iron or iron compounds such as pyrite (iron sulphide) which may exist in partially burned pieces of coal in slag and cinders. Both are capable of causing popouts. However, pieces of free lime which are formed by the high temperature decomposition of calcium carbonate are the main source of popouts that are not accompanied by staining.

The presence of air and moisture is necessary for the process of oxidation of these deleterious materials which result in the staining and popouts. By oxidation, iron sulphide (pyrite) is turned into soluble iron sulphate, which may hydrolyze to form iron hydroxide (the insoluble stain) and sulphuric acid may react with lime in the cement yielding the iron hydroxide and calcium sulphate. The hydration of a particle of free lime located at a point below the surface may result in an expansion of considerable force, causing the popout of concrete at the surface.

In this test favorable conditions were created for the process of staining and popout by storing the products in the presence of moisture and relatively high temperature for a period of 26 months. Performances of these products were then determined by visual examination and weight measurements.

The application of paint to the outside surface of concrete masonry has become a common practice. It offers some protection from water penetration by sealing the surface cavities, especially for products made with lightweight aggregates. A variety of such paints are commercially available. One of these, made with a cement base, was used for painting one face of the specimen before exposing to test conditions.

A. Procedure of Test.

Test specimens were hollow masonry units (blocks) of the modular dimensions 7-5/8 x 7-5/8 x 15-5/8 inches (see Chapter II). Two specimens were used per product.

Units were cleaned (of dust, etc.) and one face painted with a commercial cement base paint. Two layers of paint were applied with a stiff brush. In the first application, the paint was carefully worked inside the surface cavities of the specimen, which was then moist cured for 24 hours before the second layer was applied. Curing after the second application was 7 days in the moist room followed by 7 days in air at normal room conditions. At the end of this period the specimens had lost moisture during the air curing and were further dried in the oven at 210 F for 24 hours. Near the end of this drying, two successive weighings at 2 hour intervals showed a loss of weight not exceeding 2 per cent of the last previously determined weight of

the specimen which was the criterion used before for discontinuing the drying (see Chapter IV). Specimens were taken then from the oven and allowed to cool to room temperature before they were taken back to the moist room for the long time (26 months) storage.

Inside the moist room, temperature is approximately 75 F and relative humidity is 100 per cent. This condition is maintained by fine water jets and an automatic cooler.

At the end of the storage period, the specimens were taken outside the moist room and immediately weighed. They were then individually examined for possible stains and popouts. After examination they were taken to the oven for drying. This second drying followed the same procedure as the first drying although required more than 48 hours.

B. Discussion of Test Results.

It was previously stated that performance of each specimen was determined by visual examination and by dry weight measurements before and after the test. Visual examination indicated the following:

(1) None of the products tested showed evidence of staining or popouts except those made with cinders.

(2) Of the cinder products, 1C31 showed significant popouts. The resulting hole was conically shaped, about 3/4 inch in diameter at the surface and more than 1/2 inch deep. It was not accompanied by staining and examination revealed that it was caused by hydration of free lime.

(3) Stains were seen on the surfaces of the specimens 2CES31 (cinders and expanded slag), 3C31 (cinders) and 5CS32 (cinders

and sand). Each was a single spot of small area (except for 5CS32). All staining was caused by tramp iron observable by the naked eye.

(4) Popouts and stains appeared only on the unpainted surface of the specimens.

Dry weights before and after test are given in Table 22, columns 3 and 5 respectively. These did not indicate loss in weight during the test which might be expected. In fact, all of the specimens showed slight increase in dry weight after test. The increase is less than 3 per cent and believed to be caused by water still held in the capillary structure. As was previously noted (part A) both weights are based on minimum drying time of 24 hours after which drying was discontinued when the weight of the specimen reached a certain stability. Weight stability was reached for all specimens in the first drying time of 24 hours but not before 48 hours of the second drying (after storage). After this latter drying, although stability of weight was reached, the specimens, as said before, still held a certain amount of water in their capillary structure. Another interesting observation is that in previous tests of companion specimens for determining their capacity for water absorption by 24 hour submersion (see Chapter V), such a degree of weight stability was reached within 24 hours of drying. This suggests that a higher degree of saturation was obtained in the long time storing of this test than was possible by the 24 hour submersion. The former processes provide ample time for the rather slow operation of filling the finer capillary structure with water, a certain portion of which remained after 48 hours of drying. Further experimenting with some

TABLE 22. RESULTS OF STAINING AND POPOUTS OF TEST OF CONCRETE MASONRY UNITS

1	2	3	4	5	6
Specimen No.	Type of Aggregate	Dry Weight Before Storage gms	Wet Weight gms	Dry Weight After Storage gms	Staining and Popouts
1C31	Cinders	11,820	13,830	12,020	Popout
1C32		11,770	13,810	11,980	
2CES31	Cinders &	13,075	15,460	13,300	Stain
2CES32	Expanded Slag	13,030	15,500	13,330	
2H31	Haydite	11,560	14,410	11,700	
2H32		11,470	14,310	11,600	
3C31	Cinders	12,600	15,610	12,830	Stain
3C32		12,240	14,650	12,430	
3SG31	Sand &	17,260	18,540	17,460	
3SG32	Gravel	17,345	18,610	17,590	
4SG31	Sand &	16,340	17,690	16,560	
4SG32	Gravel	16,710	17,850	16,860	
5CS31	Cinders	11,750	13,600	11,930	Stain
5CS32	& Sand	12,310	13,970	12,510	
6ES31	Expanded	13,135	14,880	13,400	
6ES32	Slag	12,360	14,190	12,640	
7W31	Waylite	12,400	13,900	12,600	
7W32		12,250	13,770	12,470	
7BW31	Beslite &	11,735	13,650	11,960	
7BW32	Waylite	13,100	15,010	13,360	
8CEL31	Celocrete	8,810	11,720	9,110	
8CEL32		11,430	13,930	11,620	
9S31	Air-Cooled	16,960	18,530	17,160	
9S32	Slag	16,750	18,100	16,950	
10CA31	Cinders +	11,535	13,840	11,730	
10CA32	Fly Ash	11,460	13,700	11,650	

of the lightweight aggregate products determined that it was not possible to return the specimen to its original dry weight by as much as 60 hours of oven drying.

CHAPTER XI

DRYING SHRINKAGE

Drying shrinkage and thermal expansion are main causes of volume changes in concrete masonry. The latter will be presented in Chapter XII.

Concrete shrinks when losing moisture. The magnitude of this shrinkage depends on the physical and chemical properties of the constituents (cement and aggregate), mix proportions, mix consistency and method of manufacture, and especially on the method of curing (see references 53,54,55; see also Chapter I). Shrinkage may subject concrete to stresses exceeding its ultimate strength, and being relatively weaker in tension may result in cracking. Pickett[56] advanced a theory formulating shrinkage stresses on concrete by employing the laws of heat diffusion, i.e., by assuming "that the laws governing the development of shrinkage stresses during drying are analogous to those governing the development of thermal stresses in an ideal body during cooling", and then accounting for the effect of inelastic deformation and change in the moisture content. He showed that when a saturated specimen is dried at 50 per cent relative humidity (R.H.) the stress developed is high. In fact, he showed that it would be much greater than the strength of the concrete if it were not for the effect of the plastic flow. Plastic flow, occuring when concrete is exposed to sustained stress, such as that caused by shrinkage, serves to relieve this stress. The "relative immunity" to cracking of concrete masonry

was therefore found[54] to depend upon its modulus of elasticity under sustained stresses, its tensile strength, its tendency to shrink and its degree of restraint.

As will be seen later, saturated concrete may lose a great portion of its moisture before shrinkage stresses start to develop--- this is the water contained in the relatively large voids existing in both the aggregate and paste. However, the loss of moisture from fine pores like the capillaries, or the "desorption" of the surface adsorbed water of the cement gel, will cause rapid development of these stresses. Knowledge of the mechanism of water adsorption is therefore essential.

Powers and Brownyard[57] studied this mechanism in hardened cement paste which influences the shrinkage of concrete. They proposed a theory based on combining the Brunauer, Emment and Teller Theory (which is also known as the multimolecular-adsorption theory, or B-E-T theory for brevity) and the capillary condensation theory. Briefly, the B-E-T theory implies that a solid surface which is exposed to continuous bombardment of gas molecules attract physically and holds some of the gas molecules at least momentarily. Moreover, when the gas is a vapor, such as water, the molecules caught on the surface will be in a condensed state which may be considered as a separate phase. This condensation is accompanied by evolutions of heat energy. Some of the adsorbed molecules may escape the solid surface by acquiring sufficient kinetic energy; this causes a continuous interchange between the interior of the vapor phase and the solid surface but the average of molecular concentration at the latter surface remains

higher by the virtue of surface attraction. It was also recognized in the derivation of the mathematical expressions of this theory that a molecule may be condensed on a bare solid surface or on a layer of previously consensed molecules. The capillary condensation theory[57] is based on the fact that the surface of the liquid in the capillaries is "the seat of the available energy". When the capillary is partially emptied the water surface assumes a curvature and becomes subject to tension, consequently the vapor pressure of the water in the capillaries will be less than normal for the existing temperature. This pressure is inversely proportional with the curvature of the surface. Evaporation of the moisture from the surface or its condensation depends on whether the vapor pressure in contact with the surface is, respectively, lower or higher than that of the liquid. Since hardened cement paste is a porous solid with its capillary structure usually containing water, Powers and Brownyard proposed that condensation of moisture in its surface is influenced by the combined action of the forces considered in the B-E-T and the capillary condensation theories.

Another requirement of the B-E-T theory is that at any given pressure the amount of liquid adsorbed is directly proportional to the surface area of the solid. Powers and Brownyard were therefore able to measure the apparent surface area of the solid in the hardened cement paste by measuring the quantity of water adsorbed. Other investigators measured this surface by the amount of nitrogen adsorbed, with a large difference in the results of the two methods. This was observed by, among others, Kalousek[53], who, in his more recent work, also found that the amount of water adsorption of concrete masonry units (autoclaved and normally cured) was approximately twice that of

the nitrogen adsorption. On the other hand he showed that the amounts of water adsorbed by xonotlite (hydrous monocalcium silicate with large surface area) is equal to that of nitrogen. These observations are of importance because water and nitrogen molecules do not differ greatly in size and nitrogen adsorption is considered to provide correct measurement of the surface area. Moreover, xonotlite undergoes a small drying shrinkage. Kalousek concluded that the excess water adsorbed is accommodated in the structure of the paste which he called the "interlayer water". He further assumed that the movement of this water into and out of the crystal lattice is one of the causes of volume changes in concrete products. In view of the preceding and other findings, he then proposed a tentative theory for the mechanism of stresses causing shrinkage during the various stages of drying. With this theory he explained the moisture-shrinkage relations in a rather large program of tests on concrete masonry units cured with various methods including high-pressure steam and normal-pressure steam.[15] Briefly, the theory states that shrinkage is caused by two types of stress developments: first, by surface tension of the partially filled capillary water, and , second, by "spontaneous desorption" of the interlayer water. According to the theory, the first stress occurs during the first stages of drying and is released when the capillaries are dried in which case the specimen should re-expand. But re-expansion did not occur because simultaneously with the elastic re-expansion, another stress, due to desorption of interlayer water, is developed and shrinkage continued later. These stresses being of opposite sign may also balance each other at certain humidities and thus permit no change in length during this period.

A large number of investigations have been conducted in the past decade in an effort to contribute to the solution of the problem of drying shrinkage of concrete masonry. These researches are of two types: (1) basic, which are aimed at studying the mechanism of loss of moisture and shrinkage such as the works discussed previously, or at developing methods of tests for determining the shrinkage of concrete (see references 14, 55), and (2) comparative studies of the shrinkage and loss of moisture properties of concrete made with different mixes, with different aggregates or by various methods of curing (see references 54, 55, 57). Some of the findings in the latter researches will also be referred to later.

The present work is a comparative study of loss of moisture and shrinkage by drying of concrete masonry units made with the various dense and lightweight aggregates under study. It involves one cycle of drying of saturated concrete, to a constant moisture content in air at 73 F and 50 R.H. Tests were conducted on bars sawed from solid concrete masonry (slabs). Volume changes were measured, as the decrease in the linear dimension of the test specimen, by a comparator at certain time intervals.

A. Procedure of Test.

Test specimens were bars having a 2 x 2 inch cross section and approximately 11 inches long. They were cut from solid concrete masonry units of the modular dimensions 3-5/8 x 7-5/8 x 15-5/8 inches. (See Chapter II.) Two specimens were used per product. Reference points were provided at both ends of the specimen by standard gage plugs, which were installed as follows: Two holes (3/8 x 1/2 inch) were drilled at both ends along the longitudinal center line of the

specimen as shown in Figure 33, cleaned, moistened and partially filled with thick cement paste with 2 per cent (by weight) calcium chloride (ASTM Designation C341-54).[22] Plugs were then inserted to points providing a gage length (distance between the innermost ends) of 10 inches. Care was taken to keep the outside ends clean and free of cement paste. Specimens were then moist cured for 3 days, after which the reference points were tested and cleaned before their storage in water.

Test procedure consists of storing the specimens 14 days under water at 73 ± 2 F, followed by drying in air at the same temperature and 50 per cent R.H. At the end of the water storage each specimen was drained for one minute, the surface water was wiped off, and the specimen weighed prior to length measurements. These measurements were considered the initial values to which changes in weight and length of the specimen were referred. Weight measurements were made with a balance that reads to 1 gram, while length measurements were made by an extensometer with a dial micrometer reading to .0001 inch as shown in Figure 34. After determining the initial readings for all specimens they were stored in air, having the previously stated conditions, on steel racks with a space of not less than one inch around each specimen to permit the slow circulation of air. This storage continued for 133 days during which weights and lengths of practically all the specimens were stabilized. Weight and length measurements were made at the end of 1, 4, and 7 days and several 1 week, 2 week, and 3 week periods successively as may be seen from Table 23.

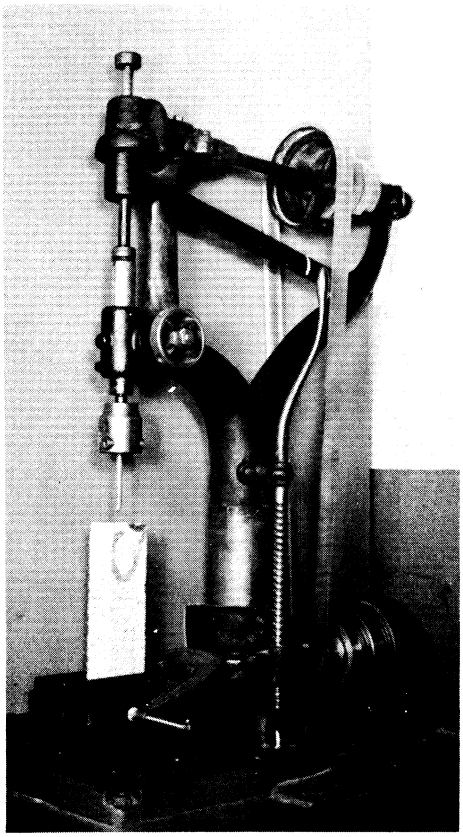


Figure 33. Drilling Hole in One End of Specimen for Cementing Steel Plug.

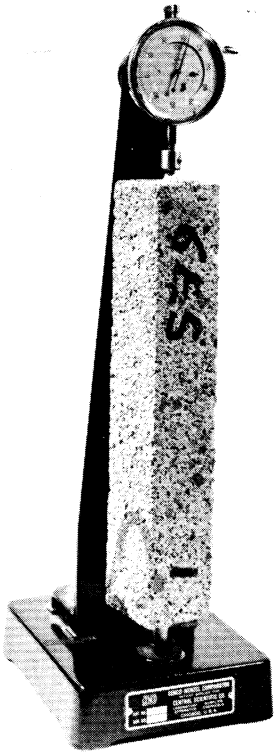


Figure 34. Extensometer with Dial Micrometer Used for Length Measurements.

B. Discussion of Test Results.

Results of the test are presented in terms of the accumulated loss of moisture and shrinkage at stated drying times (days) from the saturated condition. They are referred to the initial values of length and weight measurements at the end of the water storage as described in part A. Moisture loss is expressed in percentages of the total water of saturation. (Table 23.) The latter was determined by oven drying at 210 F to constant weight as is given in Table 26. Drying shrinkage is expressed in percentage of the gage length which is similar for all specimens (Table 24).

Results of Tables 23 and 24 are plotted in Figures 35 and 36 respectively. Each curve in these figures represents the average results of the two specimens tested for the indicated product. Figure 35, therefore, shows the rate of moisture loss during the drying, while Figure 36 shows the rate of shrinkage.

Examination of Figure 35 reveals that all moisture loss curves followed a general pattern consisting of large loss of moisture (20-51 per cent) after the first day of drying and continued at a relatively fast rate during the following three days to reach values of 59-83 per cent at the end of 4 days. They then continued to increase at successively lesser rates in the following periods reaching a comparative stability at the end of 21 to 35 days for most products. This stability continued for a long time, then was interrupted for some products which resumed slight moisture loss, at slow rate, to near the end of drying.

Likewise, examination of Figure 36 also indicates a similarity of pattern for shrinkage. The products underwent no shrinkage

TABLE 23. MOISTURE LOSS (% OF SATURATION) - DRYING TIME RELATION

Specimen	Drying Time (Days) 1	4	7	14	21	35	49	63	84	98	112	133
1C71	41	81	85	89	89	89	89	89	89	89	91	91
1C72	33	80	85	90	91	91	91	91	91	91	92	93
2CES71	24	75	80	88	90	91	91	91	93	93	93	93
2CES72	18	69	75	84	87	87	87	89	89	89	89	89
3C71	25	75	81	89	89	89	89	89	89	91	92	92
3C72	31	75	82	91	91	91	91	91	92	92	93	94
3SG71	43	75	78	83	84	84	84	84	85	87	88	89
3SG72	46	75	79	84	85	85	85	85	86	88	88	89
4SG71	49	69	75	80	82	82	82	82	82	83	85	85
4SG72	52	75	80	85	85	85	85	85	85	85	87	88
5CS71	52	76	79	85	86	87	87	88	89	89	91	91
5CS72	39	79	83	88	89	90	90	90	91	91	92	93
6ES71	30	62	69	79	82	83	83	83	85	86	87	88
6ES72	34	67	73	80	83	83	83	83	85	87	88	89
7W71	38	66	73	82	84	84	84	84	84	85	86	88
7W72	36	64	70	79	81	82	82	82	83	84	85	87
8CEL71	23	60	66	74	77	78	78	78	78	80	82	84
8CEL72	17	58	64	72	76	77	78	78	78	80	81	83
9S71	45	86	89	92	92	92	92	92	92	92	92	92
9S72	37	79	83	89	90	90	90	90	90	90	92	94
10CA71	27	81	84	90	91	91	91	91	93	93	94	94
10CA72	26	80	84	90	91	91	91	91	93	93	94	95

in the first day of drying, and only a small shrinkage occurred at the end of 4 days. However, shrinkage progressed at rapid rate afterward to the end of 21 days when, for most of the products, it stopped temporarily or continued at low rate in the period between 21 and 35 days of drying. The first 21 days will therefore be called the first stage

of shrinkage. During the second stage, after the 35th day of drying, shrinking resumed at a much slower rate than in the first stage but continued at approximately uniform rate to near the end of drying time when the lengths seemed stabilized for most of the products.

By considering the two sets of curves in Figures 35 and 36, it can be seen that large loss of moisture in the first four days of drying produced only small shrinkage (.001 - .007 per cent), which indicates that these losses were by evaporation from the comparatively large voids of the aggregates and paste. Afterwards, when drying progresses to the smaller pores and to the capillaries, moisture loss continued at successively decreasing rates while shrinkage was accumulating at a fast rate, which may indicate full development of the capillary stresses through the mechanism discussed at the beginning of this chapter. It can also be seen that moisture content of most of the products were nearly stabilized after 21 days of drying while rates of shrinkage decreased drastically during the 21 - 35 day period, which later increased and continued approximately uniformly to near the end of the drying time. This may indicate a partial release of the stresses in the 21 - 35 day period by complete drying of some of the larger capillaries. However, stresses, and consequently shrinkage, resumed and continued afterward, without any significant loss of moisture, possibly by partial drying of the smaller capillaries and desorption of the interlayer water. This is the mechanism of shrinkage proposed by Kalousek and which was discussed at the beginning of the chapter.

Close examination of Figure 35 will reveal the comparative losses of moisture from the various products during the various periods

TABLE 24. SHRINKAGE (% OF LENGTHS) - DRYING TIME RELATION

Specimen No.	Type of Aggregate	Drying Time (Days) 1	4	7	14	21	35	49	63	84	98	112	133
1C71	Cinders	.000	.006	.014	.022	.025	.026	.030	.032	.034	.039	.041	.043
1C72		.000	.004	.014	.025	.029	.031	.037	.039	.041	.047	.050	.052
2CES71	Cinders &	.000	.005	.011	.019	.026	.028	.031	.036	.041	.045	.048	.049
2CES72	Expanded Slag	.000	.005	.012	.020	.027	.029	.034	.038	.044	.049	.052	.054
3C71	Cinders	.000	.003	.008	.013	.016	.017	.018	.022	.026	.029	.030	.030
3C72		.000	.002	.007	.015	.017	.017	.021	.023	.027	.029	.030	.031
3SG71	Sand &	.000	.004	.012	.019	.021	.021	.023	.026	.029	.029	.030	.030
3SG72	Gravel	.000	.002	.009	.020	.021	.022	.026	.028	.032	.033	.033	.034
4SG71	Sand &	.000	.005	.013	.021	.025	.027	.033	.036	.041	.044	.044	.046
4SG72	Gravel	.000	.005	.014	.024	.029	.029	.032	.036	.040	.044	.045	.046
5CS71	Cinders	.000	.006	.013	.022	.026	.027	.030	.034	.040	.044	.046	.046
5CS72	& Sand	.000	.006	.015	.028	.031	.033	.035	.037	.042	.044	.046	.047
6ES71	Expanded	.000	.003	.009	.021	.027	.029	.032	.036	.041	.045	.047	.049
6ES72	Slag	.000	.001	.007	.016	.023	.026	.029	.034	.039	.043	.045	.046
7W71	Waylite	.000	.001	.007	.015	.020	.022	.027	.031	.037	.042	.045	.045
7W72		.000	.001	.005	.013	.018	.020	.022	.025	.029	.035	.037	.037
8CEL71	Celocrete	.000	.007	.018	.036	.046	.050	.057	.062	.068	.073	.076	.079
8CEL72		.000	.002	.012	.029	.042	.048	.052	.057	.064	.069	.072	.074
9S71	Air-Cooled	.000	.002	.007	.011	.012	.013	.015	.018	.022	.023	.024	.024
9S72	Slag	.000	.001	.005	.010	.011	.012	.013	.016	.020	.022	.023	.023
10CA71	Cinders +	.000	.013	.018	.026	.026	.028	.031	.035	.039	.041	.041	.042
10CA72	Fly Ash	.000	.002	.007	.013	.014	.016	.022	.024	.027	.028	.029	.029

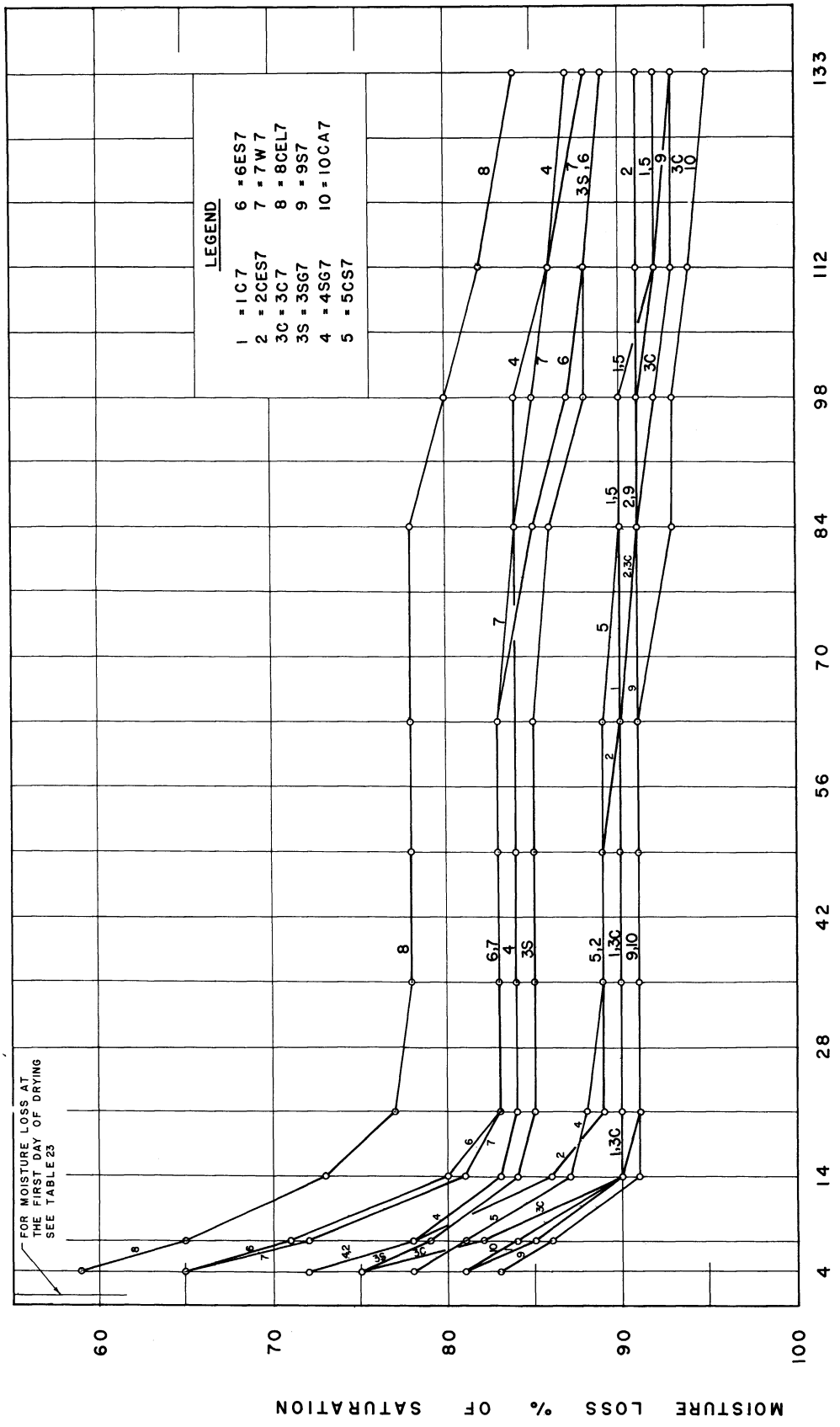


FIG. 35 · MOISTURE LOSS - DRYING TIME RELATION

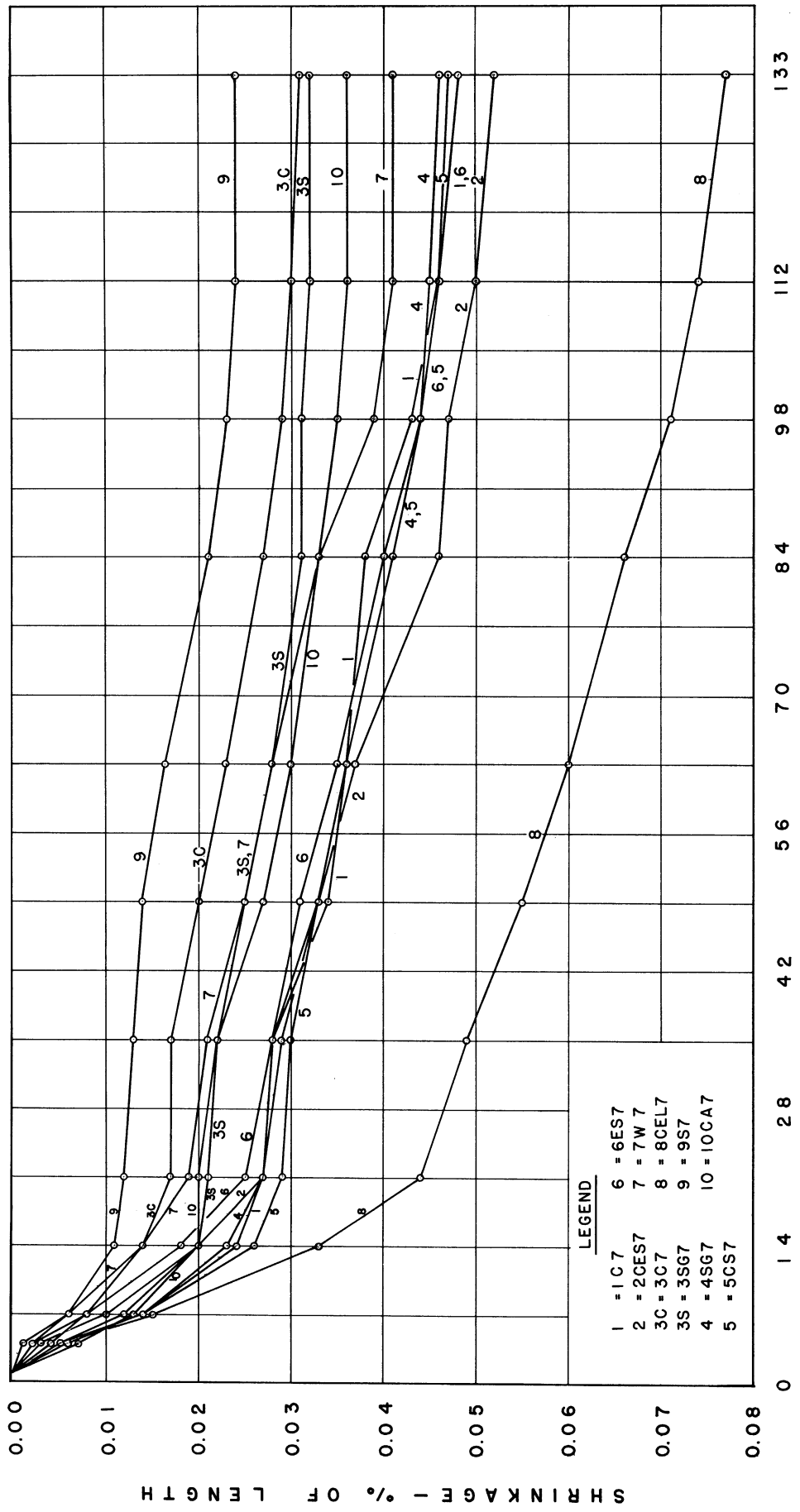


FIG. 36 SHRINKAGE — DRYING TIME RELATION

of drying. It can be seen that there is no significant division in the performances of dense and lightweight aggregate products. However, it may also be seen that expanded slags (6ES7, 7W7 Waylite, and 8CEL7 Celocrete) lost smaller percentages of moisture in the first stage of drying than cinders or dense aggregate products, but they suffered higher rates of loss in the later periods. Cinders lost moisture at higher rates than sand and gravel products which came next to the expanded slags during the early period of drying. Air-cooled slag showed the highest loss of moisture during this period. However, some of these proportions were changed at the later periods of drying which affected the total values as may be seen in Figure 35. Expanded slag products (6ES7, 7W7, and 8CEL7) losses of moisture at 133 days of drying were 89, 88 and 84 per cent respectively as compared with sand and gravel products 3SG7 and 4SG7 which lost 89 and 87 per cent, respectively. Cinder and air-cooled slag products showed a higher loss ranging between 91 and 95 per cent for the former and 93 per cent for the latter.

Similar examination of Figure 36 will reveal the relative shrinkage performances of the various products. Again there is no significant division between dense and lightweight aggregate products. However, it can readily be seen that air-cooled slag (9S7 - dense) suffered the least shrinkage, while cinder product (except 3C7) shrinkages progressed at much higher rates. The products 3C7 showed much lower shrinkage, coming next to air-cooled slag; the reason of this will be discussed later. Sand and gravel products performed

differently with 4SG7 showing higher rates of shrinkage, close to cinder products, while 3SG7 shrinkage was relatively lower. Expanded slag products also exhibited different performances with 7W7 (Waylite) showing the lower rate of shrinkage while 6ES7 and 8CEL (Celocrete) showing, successively, higher rates. In fact, Celocrete showed the highest rate of shrinkage throughout the entire period of drying.

The total shrinkage at the end of the drying, when the lengths were approximately stabilized, is of considerable significance. These values can be determined from Figure 36 at 133 days. It is apparent that Celocrete suffered the maximum shrinkage, .08 per cent compared with .024 per cent for air-cooled slag (9S7) which is the least. Cinder products (except 3C7 and 10CA7) came next to Celocrete but showing much lower values than the latter, ranging from .047 to .052 per cent. The product 3C7 showed a low value .031 per cent. Sand and gravel products had different values; a relative low .032 per cent for 3SG7 compared with a relative high .046 per cent for 4SG7. The remaining expanded slag products showed values close to those of cinder products with 7W7 (Waylite) having the lower shrinkage.

Comparison of the two sets of curves in Figures 35 and 36 showed no correlation between moisture loss and shrinkage. For instance, air-cooled slag lost a large percentage of its moisture at the early stage of drying and continued to have its curve at the bottom of Figure 35 throughout the drying time but appeared at the top of Figure 36 with minimum shrinkage. Exactly the opposite held true

for Celocrete. These two products represent the extreme cases; others lie in between without permitting this correlation. This is also apparent in Table 25 and Figure 37 which contains the moisture content-shrinkage relation for the products. These curves are not parallel. Correlation of the shrinkage with other physical properties were also attempted. These properties are the previously determined unit dry weight, 24 hour absorption, modulus of rupture and Young's modulus of elasticity. No correlation was observed between shrinkage and any of these physical properties. This may be explained by the findings of other investigators which were discussed at the beginning of the chapter, that drying shrinkage is determined by the combined effects of the physical and chemical properties of the cement and aggregate, mix proportions, methods of curing...which influence the physical properties of concrete by varying degrees. But by such general consideration one can, in this study, explain the fact that lightweight aggregates products produced higher shrinkage than those of dense aggregates, except 4SG7 whose performance cannot be explained by such consideration. By the same token the comparatively low shrinkage of 3C7 (cinders) might be explained as caused by high-pressure steam curing which has been found to reduce shrinkage. Incidentally, the rest of the products were cured by normal-pressure steam at temperatures varying between 170-180 F (see Table 3).

Another important observation pertaining to the drying shrinkage can be deduced from Figure 37 where it is apparent that at a moisture content of 40 per cent (the specification's limit) only a small portion of the potential shrinkage of the products had occurred,

TABLE 25. SHRINKAGE (% OF LENGTH) = MOISTURE CONTENT (% OF SATURATION) RELATIONS*

Specimen	Drying Time (Days) 1	4	7	14	21	35	49	63	84	98	112	133
1C7	.000	.005	.014	.024	.027	.028	.034	.036	.034	.043	.046	.048
	62	19	15	10	10	10	10	10	10	10	8	8
2CES7	.000	.005	.012	.020	.027	.029	.033	.037	.043	.047	.050	.052
	79	28	22	14	11	11	11	10	9	9	9	9
3C7	.000	.004	.008	.014	.017	.017	.020	.023	.027	.029	.030	.031
	72	25	18	10	10	10	10	10	9	8	7	7
3SG7	.000	.003	.010	.020	.021	.022	.025	.028	.031	.031	.032	.032
	55	25	21	16	15	15	15	15	14	13	12	11
4SG7	.000	.005	.014	.023	.027	.028	.033	.036	.041	.044	.045	.046
	49	28	22	17	16	16	16	16	16	16	14	13
5CS7	.000	.006	.014	.026	.029	.030	.033	.036	.041	.044	.046	.047
	54	22	19	13	12	11	11	11	10	10	9	8
6ES7	.000	.002	.008	.019	.025	.028	.031	.035	.040	.044	.046	.048
	68	35	29	20	17	17	17	17	15	13	12	11
7W7	.000	.001	.006	.014	.019	.021	.025	.028	.033	.039	.041	.041
	63	35	28	20	17	17	17	17	16	15	14	12
8CEL7	.000	.005	.015	.033	.044	.049	.055	.060	.066	.071	.074	.077
	80	41	35	27	24	23	22	22	22	20	18	16
9S7	.000	.002	.006	.011	.012	.013	.014	.017	.021	.023	.024	.024
	59	17	14	9	9	9	9	9	9	9	8	7
10CA7	.000	.007	.013	.020	.020	.022	.027	.030	.033	.035	.036	.036
	73	20	16	10	9	9	9	9	7	7	6	5

* Values are average results of two specimens. Shrinkage is on the first line, moisture content is on the second line.

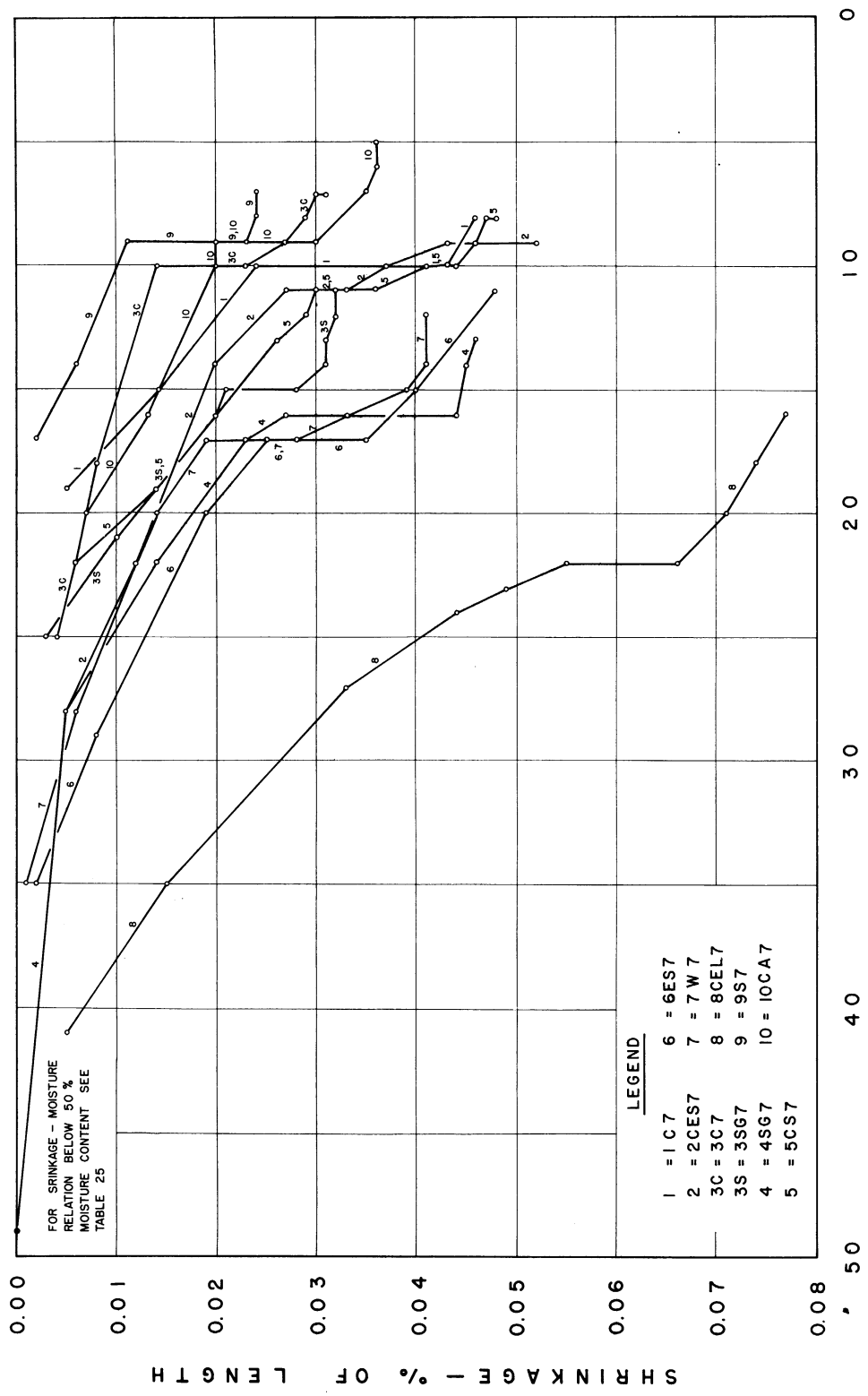


FIG. 37 SHRINKAGE - MOISTURE CONTENT RELATION

(Example: For 8CEL7 containing 35% moisture, expressed as % of total absorption, will suffer a shrinkage when dried to 20% moisture content of 0.071-0.015 = 0.056%)

TABLE 26. TOTAL SHRINKAGE AND MOISTURE CONTENT AT THE END OF DRYING. WET AND DRY WEIGHTS OF SPECIMEN.

Specimen No.	Type of Aggregate	Total Average Values at End of Drying		Weight of Specimen	
		Shrinkage % of Length	Moisture Loss % of Saturation	Saturated gms	Oven-Dried gms
1C71	Cinders	.048	92	1122	972
1C72				1176	1002
2CES71	Cinders &	.052	91	1281	1084
2CES72	Expanded Slag			1269	1090
3C71	Cinders	.031	93	1169	976
3C72				1120	941
3SG71	Sand &	.034	89	1597	1473
3SG72	Gravel			1699	1578
4SG71	Sand &	.046	87	1697	1590
4SG72	Gravel			1657	1540
5CS71	Cinders	.046	92	1296	1161
5CS72	& Sand			1298	1143
6ES71	Expanded	.048	89	1406	1262
6ES72	Slag			1420	1270
7W71	Waylite	.041	88	1355	1239
7W72				1387	1270
8CEL71	Celocrete	.077	84	1156	990
8CEL72				1169	1003
9S71	Air-Cooled	.024	93	1561	1428
9S72	Slag			1500	1410
10CA71	Cinders +	.036	95	1296	1092
10CA72	Fly Ash			1236	1045

the greater portion of shrinkage occurring at moisture contents below 20 per cent. This observation was also reported by other investigators[15] as was discussed previously in Chapters I and V. On the other hand, it should also be noted that the degree of drying obtained in this test is much higher than to be expected in the field. Menzel[58] states that the moisture content of the concrete masonry unit in the field at equilibrium depends upon the relative humidity of the surrounding air. He also indicated that for Michigan, along with most other parts of the United States, the average annual relative humidity is 65 to 75 per cent which is much higher than the 50 per cent employed in this test.

CHAPTER XII

THERMAL EXPANSION

It was stated in Chapter XI that temperature and moisture variations are main causes of volume changes in concrete masonry. The temperature-volume variation property of concrete is commonly measured by the coefficient of linear expansion, which is usually defined as the increment of length in a unit length contributed by a rise in temperature of one degree. The numerical value of this coefficient commonly[59] used for concrete design, 5.5×10^{-6} inch per inch per one degree F, does not agree with those determined for some lightweight aggregate concretes reported herein.

Thermal expansion, like drying shrinkage, could cause high stresses in concrete. Tensile stresses are of great significance because of the inherent low tensile strength of concrete. Such stresses may develop by contraction of restrained concrete, or by its uneven heating or cooling when the outside face is exposed to sudden rise or drop in temperature. Internal stresses may also be developed in concrete containing aggregate of low coefficient of expansion; or when the coefficients of coarse aggregate and mortar are different. The coefficient of expansion of hardened cement paste was determined by many investigators to be between 6 and 12 $\times 10^{-6}$, which is higher than those of most of the commercially used aggregates (see references 49, 60, 61, 64) themselves. Thermal incompatibility of paste and aggregate occurs when the latter has unusually low coefficient. It may also occur between the mortar and coarse aggregate when the coefficient

of the latter alone is comparatively low. Both incompatibilities were considered by some investigators as capable of causing destruction of concrete under certain conditions of exposure.

The problem of thermal incompatibility of aggregate and matrix was studied by many investigators; among them are Pickett[49] and later Glover.[50] Coarse aggregate in concrete was considered approximated by spherical bodies embedded in a matrix of much larger volume and having a different coefficient of expansion. These investigators, separately, produced mathematical expressions for radial and tangential stresses on the matrix as caused by differential strain at the sphere's boundary occurring by rise and drop in temperature and resulting from the difference of expansivities of the aggregate and the matrix. It was shown that for the case under discussion (coefficient of matrix is larger than that of the aggregate) and by considering the mutual restraints exerted by the aggregate and the matrix, a large rise in temperature will produce radial tensile stress and tangential compressive stress of considerable magnitudes. An equal drop in temperature will produce an equal amount of stress in the reverse direction. In discussing these stresses, Callan[49,50] showed that for certain aggregate combinations the tensile stress might be much higher than the average strength of most concretes. He also indicated that stresses are expected to be even higher by concentration in the angles of the irregularly shaped aggregate particles, a factor that was not considered in the approximate solution. However, the amount of thermal stresses in concrete depends on several factors; among them are the amount and rate of temperature change, the coefficient of expansion of the constituents, the moisture content and the

degree of mutual restraint of coarse aggregate and mortar which is determined by the degree of their interfacial bond and their moduli of elasticity. Robinson's[50] mathematical expressions for the radial and tangential stresses at the aggregate boundary imply that the larger these elastic moduli the higher the stresses caused by a certain temperature change.* Concrete made with lightweight aggregates has a comparatively low modulus of elasticity which is a favorable property to lower thermal stresses. If it were accepted that thermal incompatibility is the decisive factor in developing these internal stresses, consideration must then be given to the coefficients of expansion of aggregates when selecting a combination of two or more to be used with the mix, which is a common practice in the precast industry.

Investigators are, however, divided on evaluating the effect of thermal incompatibility on the durability of concrete. Callan[49] conducted a large series of tests on aggregates, mortars, and concretes made with aggregates of varied coefficient of expansion. He also exposed concrete specimens containing these aggregates to rather severe cycles of accelerated freezing and thawing. By employing statistical methods in analysis of the results, he arrived at the conclusion that where the difference between the coefficients of expansion of coarse aggregate and mortar is large, the durability of the concrete may be considerably lower than would be expected from the qualities of aggregates as determined by the usual tests. Pearson[62] arrived earlier at the same conclusion (considering, however, the thermal

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* For more discussion see the following references:
(1) K. Robinson, Journal of Applied Physics, Vol. 22, p. 1045, 1950.
(2) R. D. Mindlin and D. H. Cheng, Journal of Applied Physics, Vol. 21, p. 931, 1950.

incompatibility of aggregates and paste) by observing the unexpected failure after one winter of concrete in a certain structure which was made of aggregate of low coefficient of expansion, and later by conducting accelerated freeze-thaw tests on concrete made with the aggregate. Hornibrook[63] also observed the same kind of failure of another structural concrete which was also made with such aggregate and agreed with Pearson's conclusions.

Several other investigators, on the other hand, arrived at different or opposite conclusions to those described above. Bloem and Walker, also Willis and Reagel, in their discussion of the work of Callan[50] disagreed with some of the conclusions of the latter author, and although still recognizing the possible effect of thermal properties on the durability of concrete, they minimized the significance of the apparent correlation of the incompatibility and durability as based on the statistical analysis produced by Callan. They indicated that a similar analysis of test data also yielded a correlation between the coefficient of expansion of the mortar and the durability. Mitchell[60] described a large program of tests conducted by the U. S. Bureau of Reclamation for the purpose of studying the effect of the thermal characteristics of aggregate on the durability of concrete. Tests were performed on concrete made with numerous combinations of aggregates having widely varied coefficients of expansions. It was concluded that no direct relationship existed between the thermal incompatibility of the aggregates and accelerated freeze-thaw durability of concrete (see also reference 51). But it was observed that the coefficient of expansion and the durability are both affected by the ease with which concrete becomes saturated and the degree of actual

saturation. Walker, Bloem and Mullens[64] studies also indicated lack of correlation between thermal incompatibility and durability. They indicated that resistance of concrete to temperature changes are related to two separate factors. These are the rate of temperature variation and the coefficient of expansion of the concrete itself. The findings were that over a given range a fast rate of temperature change is much more destructive to concrete than a slow rate, and that concrete of high thermal coefficient failed more rapidly than that of low thermal coefficient. They also concluded that deterioration of concrete during temperature change appears to result from strains set up due to a difference in temperature within the mass rather than due to a difference in coefficient of expansion of the constituents.

As was discussed in Chapter IX, the present tests, which were conducted on companion specimens, showed no correlation between the thermal coefficients of concrete and its resistance to accelerated freeze-thaw cycles. Also no deleterious effect was observed on the durability of products made with two aggregates of different coefficients of expansion such as 2CES6 (cinders and expanded slag) or 5CS6 (cinders and sand).

Investigators agree on most of the other characteristics of the coefficient of thermal expansion. It was shown (see references 60, 61, 62) that the coefficient of a certain concrete is determined by the coefficients of its constituents and their proportions in the mix. Mullen thus suggested that the coefficient of expansion of concrete may be approximately calculated from the coefficients of its constituents. However, this is beset by the difficulty that the

coefficient of hardened paste is dependent on the moisture content. These investigators also agree that the coefficient of expansion of concrete in both the completely saturated and in the oven dried conditions are approximately equal but are considerably higher at intermediate degrees of moisture content. Mitchell[60] studied the variation of the coefficient of expansion of the hardened cement paste with moisture content. His results indicated that the coefficient of neat cement is minimum for vacuum saturated condition, slightly higher when oven dried, and maximum at some intermediate "optimum" moisture content which was estimated between 60 and 80 per cent of saturation. Results indicated that the coefficient increased by 100 per cent when the moisture content decreased from vacuum saturation by 10 to 20 per cent.

The results of the present work indicate that the coefficient of expansion of the concrete immersed is higher than for oven dried. This may partially be caused by the effect of the moisture content of the immersed specimen being lower than that of the vacuum saturated. Mitchell's results also indicated that moist cured concrete has higher coefficient than the vacuum saturated or the oven dried.

In the present study, coefficients of thermal expansion were determined for saturated (not vacuum saturated) and oven dried specimens by using two different methods of tests.

A. Methods of Test.

Two methods of test were used for measuring length changes: (1) the extensometer method, and (2) the variable resistance wire

strain gage (SR-4) method. The first method was used for saturated and oven dried specimens, while the second was used for oven dried specimens only.

1. Extensometer Method.

The same apparatus that was used for measuring shrinkage was used for measuring temperature length changes. The extensometer is equipped with a dial gage reading to a ten thousandth inch. It was described in Chapter XI and was shown in Figure 34. The specimens were also similar to those used for the shrinkage test: they were bars having they dimensions 2 x 2 x 11 inches, sawed off solid masonry units (slabs) of the modular dimensions 3-5/8 x 7-5/8 x 15-5/8 inches (see Chapter II) and were equipped with stainless steel plugs at both ends for reference points. The procedure used for cementing these plugs was also similar to that described in Chapter XI. Two specimens were used per product.

2. The SR-4 Gage Method.

Baldwin standard SR-4 gage type A-1 was used in this method. This is the same type of gage used for measuring the strain in the elastic properties tests. It is a flat wire gage about 13/16 inch long, mounted on a paper base. More detailed description of the gage is to be found in Chapter VIII. Specimens were cut from companion samples of the solid masonry units used for the first method of test, to the dimensions 3-1/2 x 2-1/2 x 7 inches. The two opposite 3-1/2 inch faces of each specimen were then cleaned and prepared for cementing the gages. A single gage was cemented longitudinally on each of the two faces. The two gages were later connected in series. The procedures of preparing the surfaces and cementing

the gages were the same as those described in Chapter IX. Two specimens were used per product; both were considered "active".

The method of test for determining the thermal expansion here reported utilizes a differential principal wherein the gage usually referred to as the "dummy" is mounted on a copper bar of accurately known coefficient of expansion. The copper bar and associated gage (actually two in series were used) is maintained at the same temperature as the gage on the specimen and should therefore compensate for different responses at different temperatures but undergoes known expansion and contraction.

A method was used for further compensation believed due to errors outside the gages themselves, and probably caused by the effect of heat variation on the electrical resistance of the lead wires or any such reasons. The method consists of providing the copper bar with another gage (also two in series were actually used), which is then considered as an active gage, i.e., connected to the circuit in the same manner used for the specimens. The reading of this active gage (which is obtained by connecting the active and the compensating gages of the copper bar) is theoretically constant at all temperatures. Any difference between two successive readings during the test is believed to indicate the error discussed above. Since the indicated error is outside the gages themselves, it is assumed to affect the results of all the specimens under test by the same amount. To compensate for this error, its amount should, therefore, be subtracted algebraically from the indicated unit expansions of all the specimens.

The copper bar, which is 1-1/2 x 1-1/2 x 14 inches, is shown in Figure 38. At one end of the bar is the compensating gage

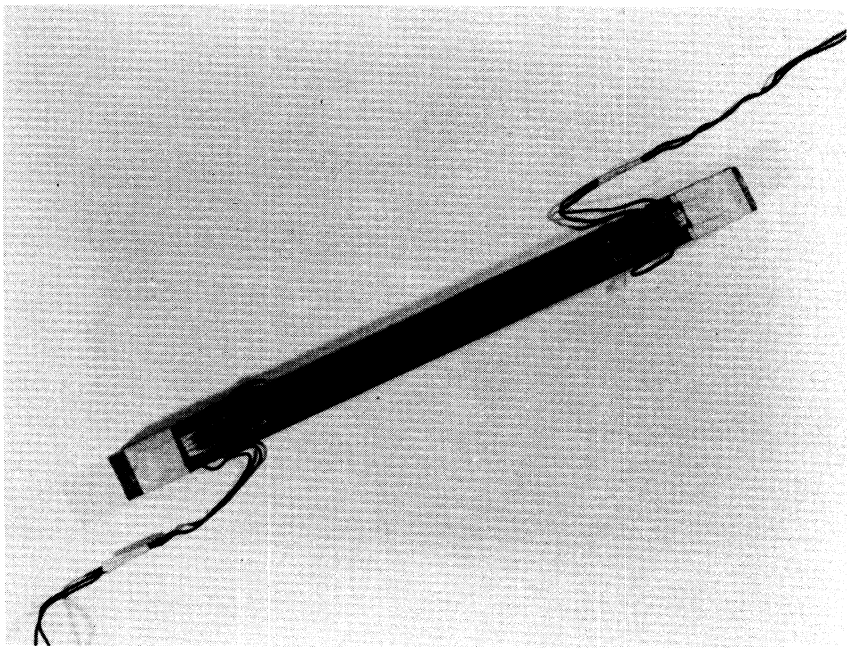


Figure 38. Copper Bar with Compensating and Active SR-4 Gages for the Thermal Expansion Test.

and at the other end is the active gage. Each of these gages is connected in series with its companion gage, which is mounted on the opposite face of the bar. During the test the compensating pair were directly connected to the designated points of the SR-4 strain indicator, while the active pair were connected to the switch box, similar to those of the concrete specimens.

The equation of the coefficient of expansion of the concrete is therefore as follows:[49]

$$e = \frac{-\Delta Rc - \Delta Rm + 9.8\ \Delta T}{\Delta T} \qquad (16)$$

Where:

e = linear coefficient of thermal expansion of concrete x 10^{-6} per degree F.

T = $T_2 - T_1$ deg. F where T_2 and T_1 are respectively the high and low temperatures of the cycle.

ΔRc = difference between successive readings of the strain indicator at temperatures T_2 and T_1, inch x 10^{-6} per inch (considered negative when due to temperature rise).

ΔRm = correction factor, equal to the difference between successive readings at temperatures T_2 and T_1, of the active gage of the copper bar, inch x 10^{-6} per inch.

The number (9.8) in the numerator is the coefficient of thermal expansion of the copper bar x 10^{-6} per degree F. Since ΔT = 60 F in this test, equation 16 then becomes:

$$e = \frac{\Delta Rc - \Delta Rm + 588}{60} \qquad (16a)$$

B. Procedure of Test.

As was previously stated, coefficients of expansion were determined for the saturated (not vacuum saturated) and oven dried concrete.

1. Saturated Specimen.

The first method of test was used for this condition. Concrete bars were placed in water baths of varied temperature, between a minimum of 35 F and a maximum of 140 F. Length measurements were made for each complete cycle at the following temperatures:

35, 73, 110, 140	73 and 35 degree F
heating	cooling

Coefficients were then calculated between these temperatures for each cycle (see Table 74, Appendix VII). Temperature was varied at a rather slow rate of 1/2 degree F per minute. The required temperature was held stabilized for one hour before lengths were measured. One cycle was completed every day to a total of 5 cycles.

2. Dry Specimen - Extensometer Method.

Upon the completion of the first test the specimens were oven dried to a constant weight. They were then cooled to room temperature, placed inside an automatically controlled oven and heated to 100 F. This temperature was held constant for two hours, after which lengths were measured by Method 1. The temperature was then raised slowly to 162 F and was also stabilized for two hours before length measurements were made at this temperature. The same procedure was used in cooling back to 100 F which designated a complete

cycle. Coefficients were then calculated for both heating and cooling for the five cycles of tests (see Table 75, Appendix VII).

3. Dry Specimen - SR-4 Gage Method.

Different size specimens were used for this test as was previously described in Method 2. The specimens which had already been dried before cementing the gages were placed in the oven, lead wires were placed outside the oven; those of the compensating gage were connected to the strain indicator directly, the rest were all active, including the active gage of the copper bar, and were all connected through the switch box selector (see Figures 39 and 40). The indicator, Baldwin's portable SR-4 strain indicator, type MB, is similar to that used for the elastic properties tests. More detailed description of the indicator is given in Chapter IX.

The temperature inside the oven was raised slowly to 100 F and left constant to stabilize for 2 hours before length measurements were made. The switch selector is manually operated. Each active gage was separately switched with the compensating gage (of the copper bar) and the indicator was balanced. This operation took a relatively short time since half of the specimens were tested at a time. Following the same procedure, the temperature was rotated between 100 F and 160 F until five complete cycles were obtained.

C. Discussion of Test Results.

The coefficients of thermal expansion of the saturated and oven dried concrete are given in Table 27. Each result is the average of two specimens and five heat-cool cycles of test. More detailed data of the saturated concrete are given in Table 74 (Appendix VII) where it may be seen that the coefficients were calculated for three

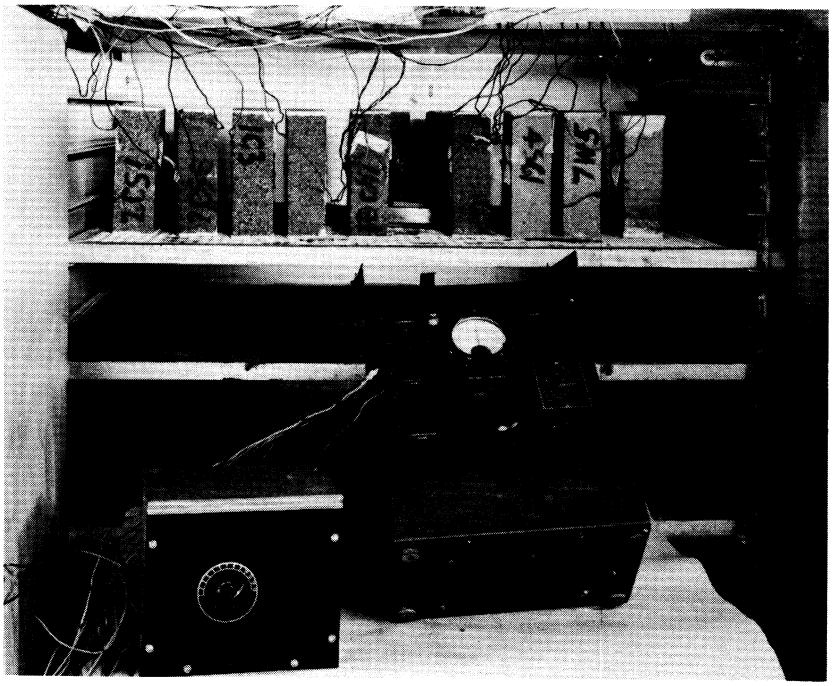


Figure 39. Apparatus for Thermal Expansion Test Using the SR-4 Gage Method (Close-up with Oven Open).

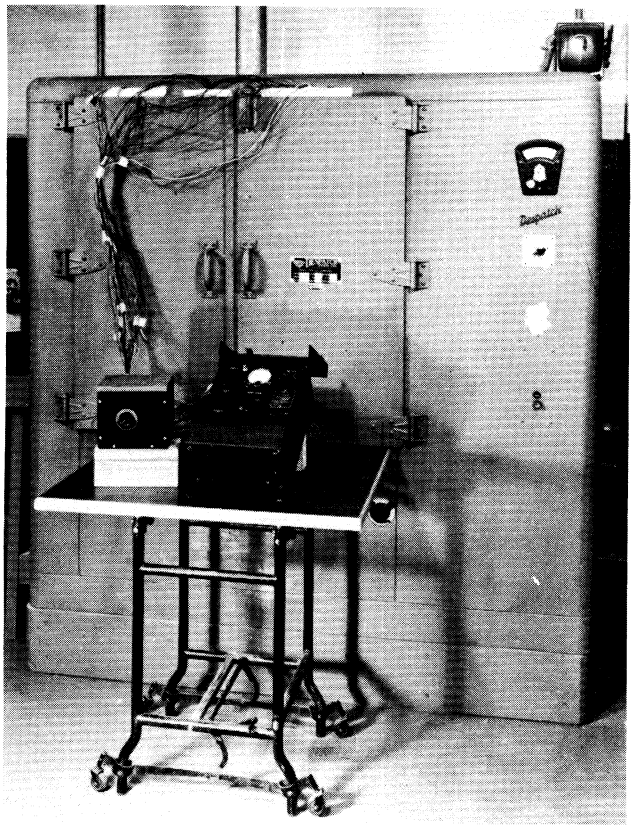


Figure 40. Apparatus for Thermal Expansion Test Using the SR-4 Gage Method (General View with Oven Closed).

stages of heating (35 to 73 F, 73 to 110 F and 110 to 140 F) and two stages of cooling (140 to 73 F and 73 to 35 F) for each cycle. Further discussion on Table 74 is given in the appendix. More detailed data is also given in Appendix VII for the dry concrete, Methods 1 and 2 of test, in Tables 75 and 76 respectively. Likewise, the coefficients were calculated for the heating and cooling stages of each cycle.

Generally, results of Table 27 indicate that cinder products have lower coefficients of expansion than sand and gravel products or the slag products. The latter two have values of similar magnitude. Close examination of any set of the results in Table 27 such as those of the saturated condition which represents a larger range of temperature, reveals that 10CA6 (cinders plus fly ash) has the lowest coefficient, 3.1×10^{-6}, while Celocrete has the highest, 5.7×10^{-6}. All-cinder products (1C6 and 3C6) have low and equal coefficients of 3.4×10^{-6} while those mixed with expanded slag or sand have higher values (2CES6 = 4.3×10^{-6} and 5CS6 = 3.9×10^{-6} respectively). Sand and gravel products have higher values (4.9 and 5.2×10^{-6}). These results are in agreement with the findings of other investigators, which were discussed at the beginning of this chapter, that the coefficient of expansion of the concrete depends on the coefficients of its constituents and their proportion in the mix.

Table 27 also indicates that the coefficients of the saturated (not vacuum saturated) concrete is somewhat higher than those of the dry concrete. The relation of the coefficient of expansion and moisture content was also discussed in some detail at the beginning of this chapter. The difference is believed to be contributed

TABLE 27.* COEFFICIENT OF THERMAL EXPANSION
(INCH PER INCH PER 1 F) X 10^6

Specimen No.	Type of Aggregate	Saturated Concrete Test Method #1	Oven-Dried Concrete Test Method #1	Oven-Dried Concrete Test Method #2
1C6	Cinders	3.4	2.8	2.9
2CES6	Cinders & Expanded Slag	4.3	3.7	3.9
3C6	Cinders	3.4	2.7	2.6
3SG6	Sand & Gravel	4.9	4.1	4.0
4SG6	Sand & Gravel	5.2	4.3	4.7
5CS6	Cinders & Sand	3.9	3.2	3.5
6ES6	Expanded Slag	5.4	4.5	5.1
7W6	Waylite	4.6	4.2	4.3
8CEL6	Celocrete	5.7	4.9	5.0
9S6	Air-Cooled Slag	4.8	3.8	4.6
10CA6	Cinders + Fly Ash	3.1	2.5	2.5

* For detailed data see Tables 74, 75, and 76 in Appendix VII.

mainly by the cement paste. It is sufficient to repeat here the findings of Mitchell, who observed that the coefficients of vacuum saturated and oven dried concrete are approximately equal and that it is higher at intermediate moisture contents reaching maximum at the "optimum" moisture content which is estimated between 80 and 60 per cent of vacuum saturation. Table 27 also indicates small differences

between the results of dry concrete as determined by the two methods of tests. This is believed to be caused from using different methods of test and from the fact that the specimens were taken outside the oven for length measurements in the first method which was not necessary in the second method of test.

CHAPTER XIII

THERMAL CONDUCTIVITY

A highly desirable characteristic of hollow concrete masonry, especially that made with lightweight aggregates, is its property of heat insulation. Concrete itself possesses a variable resistance to the flow of heat and this resistance is comparatively larger for that made with lightweight aggregates. Some of the latter concrete possesses sufficient insulation that it is now used in building construction specifically where such a property is considered desirable.

In solids, heat is transferred mainly by conduction, defined as the flow of heat from one part of the body to another part under the influence of temperature gradient without significant displacement of the particles. The term "steady state" is used when the flow of heat is such that the temperature at any point in the body does not vary with time.

The rate of flow of heat through a homogeneous and isotropic solid depends upon the temperature gradient and the thermal conductivity coefficient of the solid. The latter is a physical property of the material itself. It is defined as the time rate of heat flow, in a steady state, through a unit area of the homogeneous substance when acted upon by a unit temperature gradient in the direction perpendicular to the area. In engineering applications, the coefficient is most commonly measured as the number of British thermal units transferred per hour per square foot of area perpendicular to the direction

of heat flow per unit temperature gradient. The unit temperature gradient is expressed as the degrees Fahrenheit per inch of length of heat path. This will be abbreviated[65] as "Btu per (hr)(sq ft) (^{o}F per in.)" and is commonly denoted by "k".

For wall sections that are not homogeneous, i.e., consisting of two or more different materials or containing air cavities, the time rate of heat flow is usually measured for the entire thickness of the wall. This coefficient is termed the "thermal conductance" and is measured by the number of Btu transferred per hour per sq. ft. of area perpendicular to the direction of heat flow per unit temperature difference between the two surfaces of the wall. This is usually abbreviated as "Btu per (hr)(sq ft)(1 F) and denoted by "C".

When the rate of heat flow between air on the inside and air on the outside of a wall is considered, the term "overall coefficient of heat transmission" (sometimes called "thermal transmittance") is used. It is measured by the number of Btu transferred per hour per sq. ft. of area perpendicular to the direction of heat flow per unit temperature difference between air inside and outside of the wall and is usually denoted by "U".

The thermal conductivity of homogeneous materials is found to be affected mainly by density, moisture content, and the mean value of the temperature gradient, usually called the "mean temperature". Since concrete is actually a coalesence of different materials, its conductivity is further dependent upon the individual conductivities of the constituents; size distribution of the aggregates, mix proportion, water-cement ratio, etc. In concrete masonry construction,

the conductance is further affected by the geometrical design of the units, method of construction, property of the surface or surface finish.....

The relation of the thermal conductivity of concrete to the factors mentioned above are not yet formulated to the degree of accuracy known for many of concrete's other physical properties. This is partially because of the number of variables involved and also because accurate determination of the coefficient by the usual methods of test requires expensive apparatus and meticulous effort. However, various important studies have been conducted in the past for determining the currently used thermal coefficients. Some of these will be discussed later.

Although various methods of tests have been used, the principle involved in determining the thermal coefficient is the same. It consists of supplying a known amount of heat energy at one face of the specimen or built-up wall section while keeping the other face at lower temperature and then measuring the resulting temperature gradient between the two faces at steady conditions. The coefficient can then be calculated from the amount of heat supplied per unit time and the observed temperature gradient. The heat may be supplied by a specially designed electrically heated plate kept in contact with one face of the specimen, while the other face is cooled by being in contact with another plate with internally circulated water at lower temperature. The preceding method is commonly known as the "hot plate" method. The conductance of a wall section is usually determined by the "hot box" method. In the latter method, the wall section is built

between two chambers, one of which is heated by electrical heating elements while the second is kept at lower temperature by refrigeration coils or equivalent equipment. In some cases, the hot chamber is built entirely from the test materials. Other special methods have been used for determining the conductivity of water saturated concrete. However, all the methods follow the same principle described above. A brief review of the more important past research for determining the conductivity or the conductance coefficients of concrete under steady state of heat flow will follow. To facilitate comparison, all the coefficients presented will be in corresponding units as given at the beginning of the chapter.

Norton's research at the Massachusetts Institute of Technology[66] was among the earlier works on determining the thermal conductivity of concrete. Two aggregates were tested, cinders and stone. Three different methods were used: (1) The hot plate method, for low temperatures, consisting of concrete slabs which were applied to the faces and edges of thin electrically heated plates. The two outside faces of the concrete parallel to the hot plate were cooled by copper plates with internally circulated water. Since this apparatus is not insulated, it was necessary to run the apparatus several days before the steady state was attained. (2) For high temperature determinations, the specimens were cast in the form of a cylinder. The heat was supplied by passing a heavy current through a steel bar in the central axis of the cylinder. A calorimeter was then fitted closely over the concrete cylinder for measuring the heat flow. (3) For thicker sections, the hot box method was used. Cubical 36-inch concrete boxes of various wall thicknesses were made. During

the test, the box was heated with electrical heating elements sealed in the inside. The outside faces of the box were kept at room temperature. The conductivity coefficients were found to be 2.33 for cinder concrete and 6.25-8.42 for stone concrete, depending upon the mean temperature.

Willard and Litchty, at the University of Illinois,[67] employed the hot box method. The coefficients were determined for one concrete mix (1-2-4) at steady heat flow, and for still or moving air on the outside surface of the box. The speed and relative humidity of the air were varied. The box was shaped in a hollow column. For the moving air test, a hood was placed over the box and connected to an air duct at the top. A conductivity coefficient of 8.3 was assigned to the still air condition; other values resulted from the moving air tests. It was found that the conductance coefficient increased by an increase in speed of air or a large increase in the relative humidity.

Other early investigations at the University of Illinois were conducted by Carman and Nelson[68] on dry concrete of various mix proportion. A special method called the "cylinder method" was used. Concrete specimens were cast in forms of 7-1/2 x 24 inch cylinders with a 1-1/2 inch hole along the central axis. Heat was furnished by an electrical coil placed in this hole along the entire length of the cylinder. No cooling was provided on the outside surface of the specimen. No insulation was provided; the heat flow was assumed radial near the mid-point of the length. At steady state, temperatures were measured near the heating coil, near the surface and at a mid-point in between. The investigation also included the variation of the

conductivity with the mean temperature, age of the specimen, mix proportion and the water-cement ratio. The authors suggested a value for k equal to 10.84 to be used as a practical value for sand and gravel concrete.

Houghten and Gutberlet[69] studied the effect of age on the conductivity of concrete. A "double guard" hot plate apparatus was used. The plate was 2 feet square including the guard ring. The test specimen was inserted only on one side of the hot plate; the other side of the hot plate was kept in contact with an "auxiliary" plate which was kept at the same temperature as the hot plate thus acting as a double guard. A cold plate was placed on the opposite face of the specimen. Two specimens of different concrete mixes were tested at curing periods as much as 1300 days during which they were stored in the laboratory air. No allowance was made for the change in the moisture content of the specimen during the storage. The authors concluded that the conductivity coefficient of concrete decreased with time and changed most rapidly during the first 30 days. They also suggested a value for k equal to 12 for practical application.

The U. S. Bureau of Reclamation investigated the thermal properties of concrete made with aggregate similar to those used in the Hoover Dam construction. Concrete was tested in saturated condition to simulate service conditions. The method of test, as reported by Rippon and Snyder,[70] consisted of molding 8 x 16 inch cylindrical specimens with a hole, 1-1/2 inch diameter, along the central axis. An immersion heater was placed in this hole and the remaining space sealed with water. The cylinder, thus assembled, was then placed in a water bath of constant but lower temperature.

The current passing through the immersion heater and the temperatures at the inside and outside surfaces of the specimen were measured at steady state for calculating the coefficient. The results indicated that the conductivity coefficient varied with the aggregate and its mineral composition.

Probably the most comprehensive research on the conductivity of building materials are those done by Rowley and Algren at the University of Minnesota (see references 71, 72, 73). Reliable apparatus was developed by which the coefficients of thermal conductivity of the materials and the overall heat transmission coefficients of built-up walls were determined for a large variety of materials including plastic and "dry-tamped" concrete slabs and walls and concrete masonry made with various aggregates including sand and gravel, slags, cinders and Haydite. The study also included determining the coefficients of surface conductance in still and moving air, at various speeds and directions, and the conductance coefficients of various types of cavity construction. The effect of many factors, such as type of construction, moisture content and mean temperature on conductivity were also investigated. Both the guarded hot plate and the guarded hot box methods were used. The hot plate apparatus, used for homogeneous slabs having smooth surfaces, consisted mainly of a hot plate, two cold plates and the insulating frame. The electrically heated plate was one foot square including the central heater, 9 x 9 inches, and the guard ring, 1.5 inches wide including 1/8 inch air gap. The cold plates were kept at the required temperature by internal circulation of water. The guarded hot box, used for built-up wall sections, consisted of inner, or test, chamber and outer, or guard, chamber which

were built into one side of a cold storage room. This "double chamber" was built of insulating materials with one side omitted and was so constructed that the open face was in the same plane as the opening in the cold room wall. The wall under test was placed over the open side of the double chamber thus cutting off completely the passage of air between the test and guard chambers which were maintained at the same temperatures during the test. The findings offered were numerous and the coefficients determined were widely adopted and used by designers. Among the values given for the conductivity coefficients of concrete were the following: sand and gravel k = 12.4 to 13.0, cinders k = 4.45 to 5.75, and Haydite k = 3.73 to 4.03.

The most comprehensive research on the coefficient of thermal conductivity of concrete made with lightweight aggregates was probably that sponsored by the Housing and Home Finance Agency and conducted separately by the Bureau of Reclamation (BR) and the National Bureau of Standards (NBS) (references 19, 20, 21). This research included poured concrete of various mix proportions made with a large number of aggregates including sand and gravel, expanded clay and shale, expanded slag, pumice, etc. Concrete specimens were tested dry at mean temperatures of 27, 83 and 130 F by the BR and 119 F by the NBS. The guarded hot plate method was used. The BR employed a circular plate 9 inches in diameter while the NBS employed an 8 inch square plate; both sets of apparatus conformed with the requirements of the ASTM Designation C177-45 (Standard Method of Test for Thermal Conductivity of Materials by Means of the Guarded Hot Plate).[22] Results obtained by the two laboratories for concrete made with a given aggregate differ somewhat because of different sources

of materials, concrete mix proportions and method of manufacture, and because of different mean temperatures of the test. However, values of the following order of magnitude were found for the conductivity coefficients: sand and gravel k = 6.5, expanded slag k = 1.5 to 3.2, expanded clay and shale k = 2.9 to 4.0. It was also found that for lightweight aggregate concrete, good correlation exists between the unit weight and the thermal coefficient.

The present author's investigation constitutes a comparative study of the coefficients of thermal conductivity for products made with both dense and lightweight aggregates. Guarded hot plate apparatus, designed and constructed at the Department of Mechanical Engineering of the University of Michigan was used. The description of the apparatus, after undergoing certain improvements prior to the present tests, will follow later.

A. Brief Theoretical Analysis.

Fourier[74] has stated that the instantaneous rate of heat flow, $\frac{dQ}{dt}$, under the influence of a temperature gradient, is equal to the product of three factors: (1) the coefficient of thermal conductivity (k), (2) the temperature gradient ($-\frac{dT}{dx}$) which is the rate of change of temperature (T) with respect to the length of heat path (x) (negative in the direction of heat flow), and (3) the area of the section (A) measured normal to the direction of the flow. Thus the mathematical expression of Fourier's law is as follows:

$$\frac{dQ}{dt} = - k A \frac{dT}{dx} \qquad (17)$$

For steady state, the rate of heat flow is constant, thus:

$$\frac{dQ}{dt} = \frac{Q}{t}$$, usually denoted as q = rate of heat flow in Btu per hour.

For unidirectional heat flow through a homogeneous and isotropic body the temperature gradient $(-\frac{dT}{dx})$ is,

$$-\frac{dT}{dx} = \frac{T_2 - T_1}{L}$$

Where,

T_2 = temperature of the hot side of the specimen.

T_1 = temperature of the cold side of the specimen.

L = average thickness of the specimen in the direction of the heat flow.

For the conditions given above, equation (17) will therefore become as follows:

$$q = k\,A\,\left(\frac{T_2 - T_1}{L}\right) \qquad (18)$$

or by using the units given at the beginning of this chapter:

$$k = \frac{q}{A\left(\frac{T_2 - T_1}{L}\right)} \qquad (18a)$$

= Btu per (hour)(sq. ft.)(deg. Fahr. per inch of thickness).

or

$$k = \frac{qL}{A\,(T_2 - T_1)} \qquad (18b)$$

Equation (18b) above was used for calculating the coefficients of thermal conductivity in these tests. It is apparent that it is required to measure the rate of heat supplied by the central heater of the hot plate (q), the area of the central heater (test

area) (A), the thickness of the specimen (L), and the temperatures at the hot and cold sides of the specimens respectively, (T_2 and T_1). Since L and A are constants, it is required only to measure q, T_2 and T_1 instantaneously after realizing steady conditions. The temperatures can be measured by thermocouples, as will be seen later. The rate at which heat is supplied by the heater (q) can be calculated from the electrical power input. The latter is equal to the product of the voltage (V) and the current (I) which were measured instantaneously by a voltmeter and an ammeter respectively. Further discussion of the calculations of the results will follow.

B. Test Specimen.

Test specimens were cut from solid concrete masonry units (slabs) having the modular dimensions 3-5/8 x 7-5/8 x 15-5/8 inches (see Chapter II). Since the hot plate assembly is 12 inches square, it was necessary to use two pieces to form one specimen having this area. Each piece was cut slightly larger than 6 x 12 inches and then ground on a large commercial grinding machine to this dimension and to the same thickness so that the two pieces forming each test specimen were identical, and the two test faces of the resulting specimen were plane. The specimen thus prepared had its surfaces in complete contact with the hot and cold plates during the test.

After grinding, the pieces were cleaned thoroughly and dried in the oven at 210 F to a constant weight. Two test specimens thus prepared were used per product; each was placed on either side of the hot plate during the test.

C. Test Apparatus.

As was stated earlier, the guarded hot plate method was used for this test. The apparatus consists of the hot plate cabinet, the control board, the source of power and the electrical instruments for the necessary measurements. All are shown in Figures 41 and 44.

The hot plate consists of a central heater 9 inches square surrounded by a guard ring heater 1-7/16 inches wide. The inside edge dimensions of the ring are 9-1/4 x 9-1/4 inches which leaves an air gap of 1/8 inch around the central heater. The heater windings[75] of the two parts are also separate; each is insulated with a sheet of mica and protected with sheet copper. The copper sheath is clamped on each side over the mica sheets, forming a rigid plane unit, 12 inches square, which is called the "hot plate". Sectional plan of this plate and other details are also presented in Appendix VIII.

The two cold plates are identical. Each consists of a hollow box having outside dimensions of 12 x 12 x 3/4 inches, made with copper plates 1/4 inch thick. Cooling is accomplished by circulation of tap water through the plates.

The hot and cold plates are housed inside a wooden cabinet which is insulated on the outside by a 1-1/2 inch cork board with transite lining. The inside width and height of the cabinet are 12 inches. The hot plate is held permanently at the center vertical plane of the cabinet. The cold plates are placed parallel to the hot plate. The top of the cabinet and its sides parallel to the plates are removable to facilitate the loading of the specimens. The cabinet is also provided with a device for clamping the plate assembly

together, consisting of two threaded rods placed horizontally in both sides of the cabinet at about mid-height (see Figure 41).

The control panel is shown in Figure 41 and, separately, in Figure 43, where the circuit components and the instrument connections to binding posts are clearly labeled. Separate controls are provided for the central heater and the guard ring circuits, as may be seen in Figure 43. The central heater circuit control, across the top of the panel, consists mainly of three continuous rheostats, connected in series, having the following total resistances: No. 1 and 2 - 500 ohms each; No. 3 = 50 ohms. The guard heater circuit controls, placed on the lower line consist of the two continuous rheostats, also connected in series and having the total resistances of 250 ohms for No. 1 and 10 ohms for No. 2.

The current enters the control panel at the lower left binding post marked "input". It is then divided between the two circuits at the right, passing through the rheostats and the switch to the binding post (marked "output") at left, where the measuring instruments are connected as labeled. Separate connections were made to the central heater and to the guard ring heater (see Figure 41).

Measurements made include the voltage and the current of the central heater circuit, and the voltage of the guard circuit. The instruments are shown in Figures 41 and 42. They consist of two portable Weston D. C. voltmeters of 300 volts capacity, graduated to 1 volt, for measuring voltages of the two circuits. For measuring the current of the central heater, a Weston portable D. C. millivoltmeter, connected in series with a shunt was used; this instrument is

readable to 1/2 millivolt which provides measurement of the current to the nearest 0.01 amp.

Temperature measurements were made by thermocouples which were manufactured of thin (No. 30 A.W.G) copper-constantan wires fused together at the hot junctions. The hot junctions of the couples are placed on the surface of the specimen while the other ends are connected to a switch box (see Figure 73 in Appendix VIII and Figure 41) which is in turn connected to a temperature indicator. The latter is a portable Leeds and Northrup potentiometer graduated directly in degrees Fahrenheit. Cold junctions were not used because the instrument compensates internally for room temperature. This compensation and the potentiometer balancing are accomplished manually.

Source of power is a motor-generator set which is shown in Figure 44. The motor (at right in the figure) is three phase 15 horsepower, operating on alternating current of 220 volts, 60 cycles, which is obtained from the University line. The output of the generator is 250 volts D. C. which is reduced to 110 volts by a rheostat at the operating panel of the set (power panel), see Figure 44. The capacity of this set is much larger than that required; however, it provides a steady current, free of fluctuations that may cause unsteady heat flow which might cause erroneous results.

D. Procedure of Test.

The specimens were prepared as described in part B above. Before placing the thermocouples, the location of the test area (the 9-inch square opposite the central heater) and the points where the temperatures are to be measured were marked on the face of the specimen. The hot junction of a thermocouple was then placed on each of

Figure 41. General View of the Hot Plate Apparatus.

Figure 42. Close View of Measuring Instruments and Switch Box for the Hot Plate Apparatus.

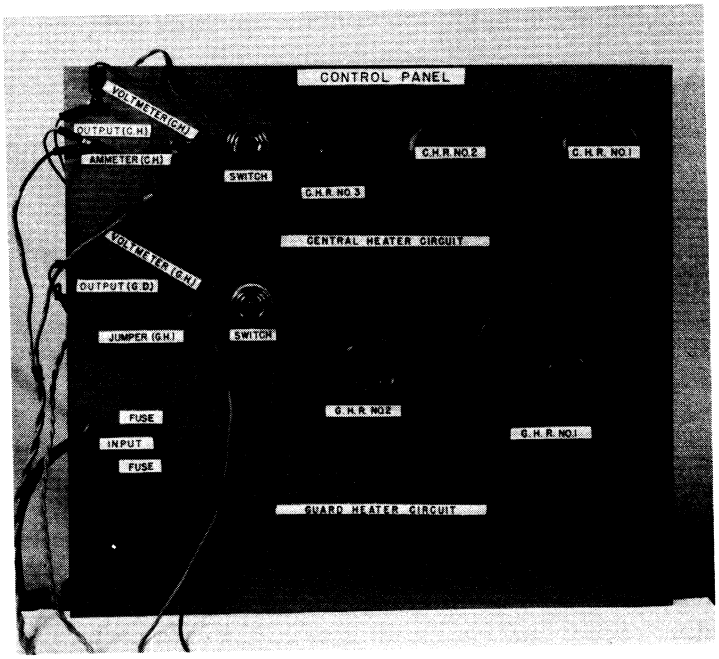


Figure 43. Control Panel for the Hot Plate Apparatus.

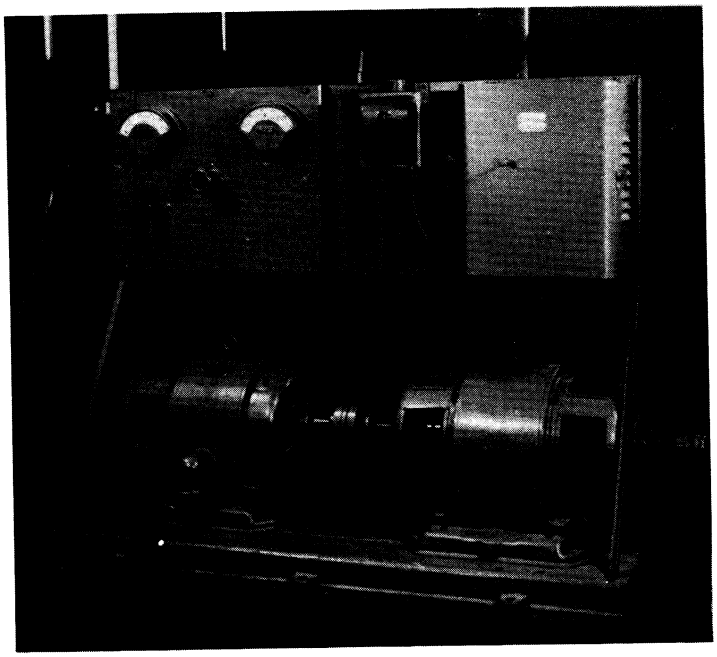


Figure 44. Source of Electrical Current for the Hot Plate, Motor-Generator Set.

these points and the wire was held in place with masking tape. Ten thermocouples were installed on each specimen; six of these were placed on the face adjacent to the hot plate, and the remaining four were placed on the face adjacent to the cold plate. An equal number of thermocouples were placed on the companion specimen at the opposite side of the hot plate. The total number of thermocouples used was therefore 20. They were located as shown in Figure 73 in Appendix VIII.

After placing the thermocouples, the two companion specimens were "loaded". This was accomplished by lifting the top panel of the apparatus, loosening the clamps and sliding one side of the apparatus including the cold plate away from the hot plate to provide room for the specimen. The specimen was then placed vertically in the cavity between the hot and cold plates; thermocouple leads were brought to the top; sheets of blotting paper, cut to face dimensions of the specimen, were slipped on the two sides of the specimen, and the side panels of the apparatus including the cold plates were pushed into contact with the specimen. The same procedure was used for loading the companion specimen on the other side of the apparatus. The assembly was then clamped tightly together as may be seen in Figure 45 .

The thermocouples were then connected and tested to see if any were damaged during the loading. The thermocouples of the central heater were connected and read separately and not differentially as has been sometimes done. Around the movable sides of the apparatus or any opening where heat loss was suspected, rock wool was carefully packed. A blanket of this material was placed on the top before inserting the top panel. Prior to starting the test the motor-generator

set was warmed up. The measuring instruments were then connected to the control panel as may be seen on Figures 41 and 42. The apparatus now was ready for the test, in which the following procedure was used. The general procedure is in agreement with the ASTM Designation C177-45 (Standard Method of Test for Thermal Conductivity of Materials by Means of the Guarded Hot Plate).[22]

Tap water was fed swiftly through the cold plates and was adjusted by two individual valves so that equal amounts ran through the two plates. For safety, the total resistance of the rheostats of each of the central heater and the guard heater circuits were inserted before the current was allowed in each circuit. The voltage was immediately adjusted to that required by removing some of the resistance from the circuit. The actual voltages required for the central heater and the guard to provide a quick and uniform heating differ according to the specimen tested; however, for most specimens 100 and 50 volts were used respectively. Smaller voltages were used for some of the lightweight products, but the 2:1 ratio above was found satisfactory.

During the heating period, temperature measurements were taken every fifteen minutes. When the required temperature of the hot plate was approached, but not quite reached, the voltages of the central heater and the guard were dropped to values which kept the temperature of the plates (heater and guard) at the level desired. The voltage required depends on the temperature and the type of concrete tested, and was determined by pilot tests. After setting the voltages at the "running" values, temperature measurements were made at 15-minute intervals for the first two hours, during which current adjustments, if necessary, were made in small increments to bring the

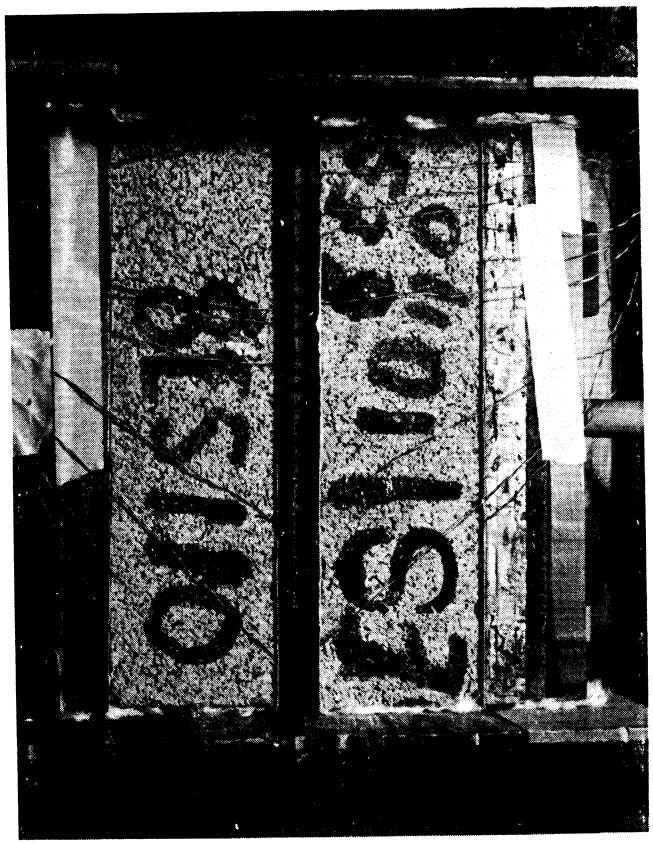


Figure 45. Looking Inside the Cabinet with Hot Plate, Cold Plate and Specimen in Between, Clamped Together.

temperatures of the central heater and the guard into equivalence. When this was accomplished, the apparatus was kept running while temperature measurements and the necessary adjustments were made at one-hour intervals to refine the temperature balance. After this the test continued for at least five hours, without current adjustment, before the final readings were taken. At the latter period the temperatures of the hot plate (heater and guard) and the cold plates were measured frequently to assure continued equilibrium.

The conductivity coefficient was calculated from the final readings which included careful determination of the central heater input (voltage and amperage) and the temperatures of the hot and cold plates. The total time required per run was 15 to 18 hours, differing somewhat according to the type of concrete. Two runs were conducted per product at different mean temperatures.

E. Calculations.

In part A, the coefficient of thermal conductivity (k) was defined in equation (18b) as follows:

$$k = \frac{qL}{A\,(T_2 - T_1)} \qquad (18b)$$

Where q is the rate of heat flow (Btu per hour)

But,

$$q = 3.412 \times P = 3.412 \times I \times V \qquad (19)$$

Where,

P = electrical power (watt)

I = rate of flow of the current (amp)

V = emf (volts)

L = length of heat path (inch)

Now, in equation (18b),

$$A = \frac{2\ (9 \times 9)}{144} = 1.125 \text{ sq. ft. total test area for the two companion specimens.}$$

Substituting above values of q and A in equation (18b):

$$k = \frac{3.412 \times I \times V \times L}{1.125\ (T_2 - T_1)} \quad \text{or}$$

$$k = \frac{3.033 \times I \times V \times L}{T_2 - T_1} \qquad (20)$$

Results were calculated by equation (20) for two runs per product, shown in Table 28. In this table, the current input of the central heater (columns 2 and 3) were taken from Table 77 (Appendix VIII) which will be discussed in part F. The temperatures T_2 and T_1 in Table 28 are respectively the average temperatures of the hot sides and the cold sides of specimen No. 1 (columns 5 and 6) and No. 2 (columns 7 and 8) as has been discussed in part A. T_2 is the average temperature indicated by four thermocouples located in the test area of the hot sides of each of the companion specimens; T_1 is the average temperature indicated by four thermocouples located in the cold side of each of the companion specimens. The individual readings of the thermocouples, for all the specimens, are given in Table 77. The temperature difference $(T_2 - T_1)$ in the calculations (column 9 of Table 28) is the average value of the two companion specimens.

F. Discussion of Test Results.

The final current and temperature determinations for the two runs of test of each product are given in Table 77 in Appendix VIII. These were determined (see part D) at the end of at least five hours of constant and steady state of heat flow in each test,

TABLE 28. CALCULATION OF THE COEFFICIENT OF THERMAL CONDUCTIVITY (k) BTU PER (HR)(SQ. FT.)(DEG. F/INCH)

1	2	3	4	5	6	7	8	9	10	11
Sample No.	Current Input Volt	Amp.	Average Thickness of Specimens #1 and #2 (L) inch.	Average Temperature Specimen #1 °F Hot Side T_2	Cold Side T_1	Average Temperature Specimen #2 °F Hot Side T_2	Cold Side T_1	Average Temp. Difference for Both Specimens °F ($T_2 - T_1$)	k	Average of Two Runs k
1C10	65	.39	3.55	123.8	56.1	124.5	57.0	67.6	4.04	4.04
	65	.39		124.4	56.5	125.6	57.3	67.6	4.04	
2CES10	65	.39	3.55	123.3	56.5	124.0	56.5	67.1	4.07	4.10
	70	.42		133.8	56.5	135.0	57.1	76.7	4.13	
3C10	65	.38	3.55	124.6	56.0	126.0	57.5	68.6	3.88	3.88
	70	.42		139.3	57.3	139.0	57.5	81.8	3.87	
3SG10	75	.43	3.55	109.8	64.3	110.6	64.5	45.8	7.49	7.69
	80	.48		117.0	64.5	117.6	65.1	52.5	7.88	
4SG10	80	.48	3.55	114.3	59.6	115.0	60.0	54.8	7.54	7.54
	80	.48		115.1	60.3	115.9	61.0	54.9	7.53	
5CS10	60	.36	3.51	115.5	85.5	116.3	58.5	57.4	4.01	4.22
	70	.42		130.4	60.5	131.1	59.3	70.9	4.42	
6ES10	55	.33	3.43	127.0	53.0	127.8	52.5	74.6	2.53	2.60
	60	.36		136.5	52.5	136.6	52.0	84.3	2.67	
7W10	50	.30	3.43	113.1	59.6	114.1	60.4	60.0	2.60	2.71
	60	.36		134.6	55.5	136.6	56.0	79.9	2.81	
8CEL10	50	.30	3.43	119.6	52.0	120.4	52.3	67.9	2.30	2.38
	60	.36		146.3	53.5	147.4	55.5	91.3	2.46	
9S10	60	.36	3.53	121.5	57.0	121.5	57.0	64.5	3.59	3.68
	70	.42		139.3	55.8	138.6	55.3	83.4	3.77	
10CA10	55	.33	3.51	104.8	57.8	105.3	58.5	46.9	4.12	4.18
	70	.42		134.0	60.0	134.3	60.5	73.9	4.24	

which was proved by frequent test observations during the period. Complete data, including these observations and other readings made earlier in the test for current adjustments, consists of over 400 determinations per run and are therefore not given since they are not necessary for the calculations of the results. Further discussion of Table 77 is given in Appendix VIII.

Calculations of k were made by equation (20) separately for each run and are presented in Table 28, which was discussed in part E. The results of each product are averaged in column 11 of the table.

The average results show that (1) sand and gravel products have k values equal to 7.69 and 7.54 for 3SG10 and 4SG10 respectively, which are much higher than those for lightweight aggregates; (2) cinder products have values of k averaging between 4.04 and 4.22 which varied in proportion with their unit dry weight (see Table 29); (3) expanded and air-cooled slag products have lower values of k, equal to 3.68 for air-cooled slag (9S10) and average between 2.38 and 2.71 for expanded slags. These also vary in proportion to their unit weight.

It should be noted that the average values of k discussed above do not correspond to the same average mean temperature although, as may be seen in Table 29, the difference between these temperatures is comparatively small. It may also be seen in Table 28 that, for most of the products, the average temperature (column 9) in the second run for the product is made higher than that for the first run in order to study the effect of increase in temperature on k. However, these temperatures are also not equal for all the products, although they are all in the same order of magnitude as that met by concrete

walls in normal service. The main reason is that it was not possible to keep the mean temperatures or the temperature gradients uniform for all the products is that the city water cooling the cold plates (see part D) was not maintained at a constant temperature. Nevertheless, it can be seen in Table 28 that values of k for a certain product are increased by increasing the mean temperature. This relation was also noticed by many investigators as was discussed at the beginning of this chapter.

The only correlation observed between the thermal conductivity and the remaining physical properties of the concrete products, as determined in previous tests, is that between the thermal conductivity and unit dry weight. For products of the same types of aggregates, k is proportional to the unit weight. This can be seen in Table 29 in which the unit weights were calculated from the weights of the specimen before test. The latter weights and the weights after test are given in Table 78 in Appendix VIII. Correlation between the thermal conductivities and unit weights of lightweight aggregates were also reported by many investigators (see references 19, 20, 21, 65 and 73); others (reference 76) have indicated that this correlation is peculiar to concrete made with lightweight aggregates and that it does not always exist for that made with dense aggregates.

As was mentioned in the brief historical notes, some investigators found that the coefficient increases with the age of concrete. This relation cannot be determined for the present products because the initial ages of the concrete are not known (see Chapter II). However, all the products were stored over two years in the normal laboratory air before being tested.

TABLE 29. COEFFICIENT OF THERMAL CONDUCTIVITY (k), AVERAGE MEAN TEMPERATURE, AND UNIT DRY WEIGHT (Waylite and Celocrete are the commercial names for expanded slag.)

Sample No.	Aggregate	Unit Dry Weight p.c.f.	Average Mean Temp. °F	Average k
1C10	Cinders	89.9	90.7	4.04
2CES10	Cinders & Expanded Slag	91.0	92.9	4.10
3C10	Cinders	88.1	94.7	3.88
3SG10	Sand & Gravel	129.1	89.0	7.69
4SG10	Sand & Gravel	125.4	87.6	7.54
5CS10	Cinders & Sand	93.2	91.2	4.22
6ES10	Expanded Slag	90.1	97.7	2.60
7W10	Waylite	100.0	91.2	2.71
8CEL10	Celocrete	82.8	93.4	2.38
9S10	Air-Cooled Slag	116.9	93.2	3.68
10CA10	Cinders & Fly Ash	91.5	89.4	4.18

It should also be borne in mind that the results given were determined for oven dry concrete and still air conditions. Presence of moisture in concrete or motion of air across the surface, such as may be met in service markedly increases the rate of heat flow. The amount of this increase depends on the moisture content of concrete and the speed of air. This was discussed briefly at the beginning of the chapter.

CHAPTER XIV

SUMMARY

The tremendous growth in the use of concrete masonry units made with lightweight aggregates in the last decade has made desirable the accumulation of data regarding the performance of these building materials in order that intelligent decisions can be made regarding their proper use. This study attempts to fulfill this need in the Michigan area for a cross section of the products contemporarily available. In this concluding chapter, effort is made to provide reference to the salient features of the test results on masonry units obtained from representative dense and lightweight aggregate products. The overall average results of most of these tests are presented in Table 30 at the end of this chapter, with a bar diagram (Figure 46) for comparative study. Important conclusions are here summarized:

(1) The adoption of the system of modular dimensions by the industry is evident in the results of the tests. Three products have units with deviations exceeding that permitted by the current ASTM standards. However, no single specimen of a given product showed a deviation from its own average exceeding the permitted value.

(2) Weights of 8 inch masonry walls containing the lightweight aggregate units range between 29 and 36 pounds per sq. ft., and for the dense aggregate between 43 and 48 psf.

(3) Compressive strength of the lightweight aggregate masonry units generally varied between 58 and 88 per cent of the strength of the dense units; no correlation was found between weight

and strength. Three lightweight products averaged just below the minimum ASTM requirement for load-bearing units (700 psi).

(4) Lightweight aggregate and air-cooled slag products are more absorbent than sand and gravel products. This is an adverse property for lightweight masonry exposed to the weather. Cinders have the highest absorption followed by the slag products. Some correlation exists between the compressive strength and the absorption. High strength usually accompanies low absorption.

(5) The moisture contents of the products when sampled varied between 19.4 and 62.6 per cent of the 24 hour absorption. Six products exceeded the 40 per cent maximum allowed by the ASTM specifications. The latter requirement is considered of value in limiting shrinkage and subsequent cracking which may result from using units of high moisture content. Most of the products are stored in open air and, therefore, their moisture content varies from time to time. Sampling was done during the relatively damp spring season.

(6) There was no sharp line of demarcation between the dense and lightweight aggregate products with regard to the total shrinkage or the shrinkage-moisture loss performance. For a given product, and between products, the shrinkage experienced is not predictable from the actual moisture content. Generally, the shrinkage tended to be greater for those lightweight aggregate products (not autoclaved) having higher absorption or for those requiring higher cement content to obtain the required strength. Cinders and expanded slag products (except Celocrete) undergo similar shrinkages and are much higher than those of the air-cooled slag product. No apparent correlation exists between shrinkage and the unit weight, strength

or modulus of elasticity. However, it is suggested that lightweight aggregate products, because of their lower modulus of elasticity and reported higher creep, have less tendency for cracking. High-pressure steam curing (autoclaving) resulted in lower shrinkage.

(7) Staining and popouts were determined by visual examination of the masonry units after 26 months of moist storage. One product (cinders) showed a popout believed to be caused by hydration of free lime. Three other cinder products showed stains that are apparently caused by oxidation of tramp iron particles. Stains and popouts did not appear on faces painted with a cement base paint indicating that the paint may have given at least temporary protection.

(8) Flexural strength of dense aggregate products are 1.6 to 3 times higher than those of lightweight aggregates. However, it has been suggested that the flexural strength of an unreinforced masonry wall depends largely on the strength of the mortar joints. Very poor correlation exists between the compressive strength and the modulus of rupture possibly because of the differences in the particle shapes and surface texture existing for the wide range of aggregate types included in the present study. It is now well recognized that the flexural test is much more sensitive to the characteristics of the coarse aggregate than is the compressive test.

(9) Elastic constants (Young's modulus of elasticity, E, modulus of rigidity, G, and Poisson's ratio, μ) as determined by the static method are lower than those determined by the dynamic (sonic) method. This conclusion has been confirmed by all other investigators. Products made with dense aggregates have higher constants (E and G) than those made with lightweight aggregates. Young's moduli for

lightweight aggregate products range between 33 and 80 per cent of the lower value of the two sand and gravel aggregate products in the study. Rough correlation exists between the static modulus of elasticity and the compressive strength and is still better between products having similar aggregates. Rough correlation exists between the elastic modulus and modulus of rupture. Better correlation exists between the elastic modulus and the unit dry weight.

(10) Lightweight aggregate products of highly cellular structure, unlike sand and gravel products, are vulnerable to a very limited number of accelerated freeze-thaw cycles when fully water saturated. This probably results from their high absorption and capability of retaining a large amount of freezable water in the open air voids in the porous concrete and distinct from the voids contained in the particles themselves. A special mix design was probably used to obtain this highly cellular concrete to promote light weight. However, the condition of full saturation is rarely met by masonry walls in normal service. A moisture content, of for instance, 40 per cent of the absorption (maximum allowed by the ASTM specifications) may therefore be established as a more appropriate limit. Tests on specimens sealed in copper jackets at the 40 per cent moisture content showed that lightweight products withstood 500 cycles of freezing and thawing well. Of the dense products, air-cooled slag exhibited inferior performance which was probably caused by deleterious particles in the coarse aggregate. However, this was not shown conclusively because poor performance of concrete cannot normally be assigned to a single cause, and a conclusive study would have required a larger program of tests than was possible. No correlation was observed between

durability and the other properties of the concrete, such as weight, strength, etc.

(11) Coefficient of thermal expansion was determined for water-immersed and oven-dried conditions. The coefficients for the oven-dried concrete were approximately 10 to 20 per cent lower than for the water-immersed condition. This difference is believed to be contributed mainly by the cement paste. In general, the results indicate that cinders have a lower coefficient than sand and gravel or slag products. The latter two have values of similar magnitude. No correlation was observed between the coefficient of expansion and any of the remaining physical properties of the concrete. However, the coefficient appears to be influenced by the coefficients of its constituents and their proportion in the mix. This is observable for products made from a blend of two different aggregates, where the coefficients of concrete containing the individual constituents were determined.

(12) Coefficient of thermal conductivity (k) was determined for oven-dried concrete by the hot plate method at mean temperatures between 88 and 98 F. The two sand and gravel products have (k) values of 7.7 and 7.5 Btu per (hr)(sq. ft.)(deg. F per inch), respectively, compared with (k) values for cinder products which averaged between 4 and 4.2 and the slags which averaged between 2.4 and 2.7. No correlation was observed between thermal conductivity and any of the remaining physical properties of the concrete except unit dry weight.

On the basis of the previous summary it may be concluded that: Lightweight aggregate masonry units possess certain characteristics that are desirable in construction materials for types of

structures such as residential buildings, etc., where high strength is not of primary importance and where severe weathering in the presence of high moisture content is not normally encountered. The varied and appealing surface texture of these materials have architectural advantages. Their comparative lightness permits economy of construction by reducing the dead loads on other structural members. Their lower thermal conductivity results in economy in heating and air conditioning, and provides a more comfortable housing in hot seasons where artificial cooling is lacking. It has also been suggested that lower thermal conductivity may indicate a somewhat longer fire-endurance period for the masonry wall. Much work has been done to reduce the potential excess shrinkage of certain lightweight aggregate products which may cause subsequent cracking of the masonry; certain methods have been suggested and used with considerable success such as high-pressure steam curing or drying of the units to a safe moisture content prior to use. It appears that the presently large and continuing growth in the use of lightweight aggregate concrete masonry units for housing and similar type construction is based on sound engineering and is well justified.

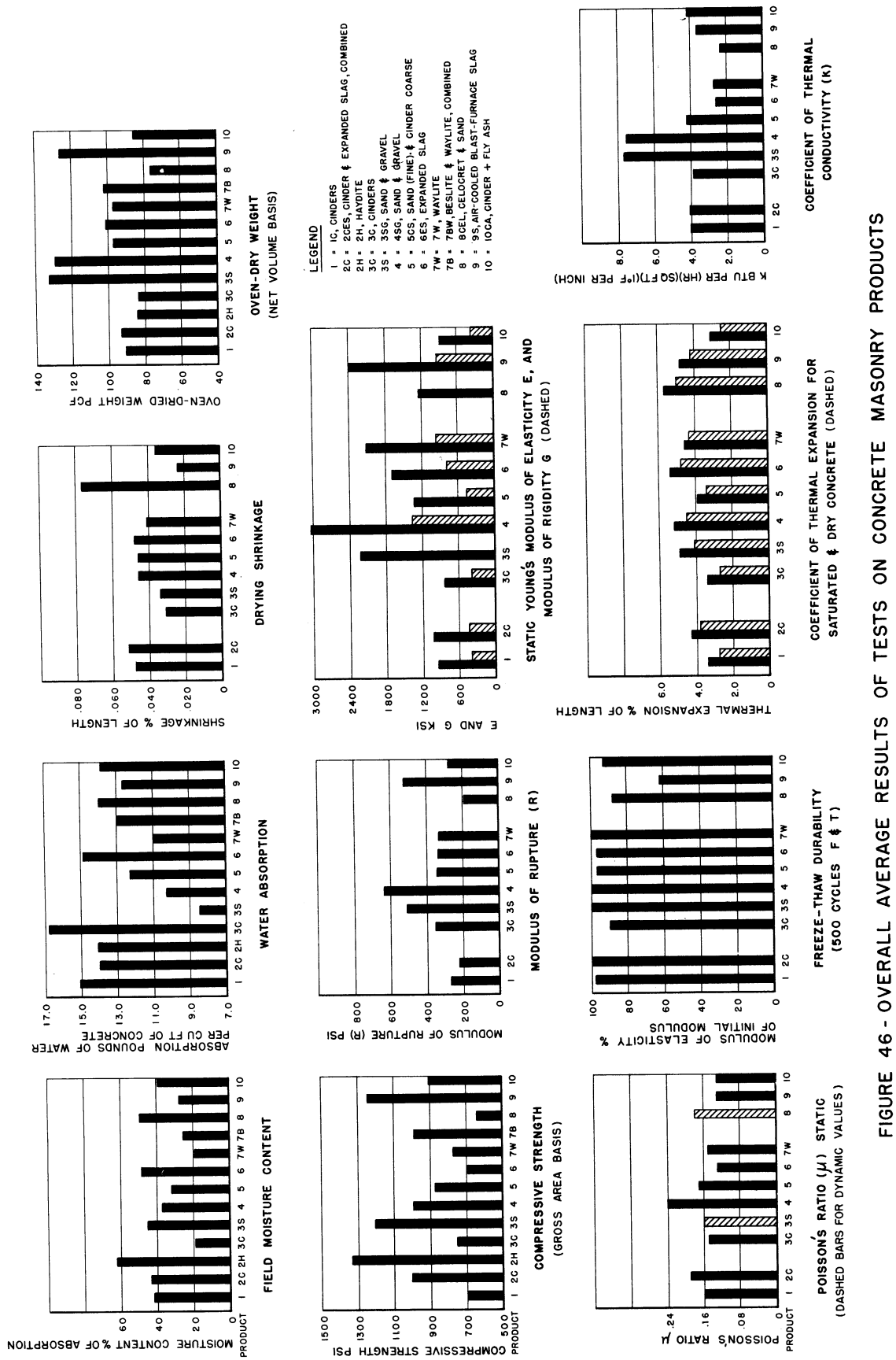


FIGURE 46 - OVERALL AVERAGE RESULTS OF TESTS ON CONCRETE MASONRY PRODUCTS

TABLE 30. SUMMARY OF THE OVERALL AVERAGE RESULTS OF TESTS ON CONCRETE PRODUCTS

Product No.	Type of Aggregate	Dimensional Properties: Gross Dimensions of Modular Units, Thickness t	Length L	Height h	Gross Volume t·L·h	Net Volume	Gross Area t·L	Net Area	Net Area Per Cent of Gross Area	Weight of Unit: As Sampled	Water Saturated	Oven-Dry	Unit Weight of Concrete: As Sampled	Water Saturated	Oven-Dry	Absorption 24 hr. Immersion: Pound of Water per cu.ft. of Conc.	Per Cent of Dry Weight	Moisture Content as Sampled-per cent of Absorp.	Compressive Strength: Based on Gross Area of Modular Unit	Based on Net Area of Modular Unit	Modulus of Rupture R	Elastic Properties: Dynamic (Sonic): Modulus of Elasticity E	Modulus of Rigidity G	Poisson's Ratio μ	Static: Modulus of Elasticity E	Modulus of Rigidity G	Poisson's Ratio μ	Freeze-Thaw Durability: Number of Cycles	Mod. of Elasticity After Test, % of Initial Modulus	Modulus of Rupture R After Test	Staining and Popouts	Total Drying Shrinkage & Moisture Loss: Shrinkage of Length	Moisture Loss per cent of Absorp.	Coefficient of Thermal Expansion: Water Saturated by Extensometer	Oven Dry by Extensometer	Oven Dry by Strain Gage	Thermal Conductivity: Unit Weight	Mean Temperature	Coeff. k Btu per (hr) (sq. ft.) (deg. F per inch)
		in.	in.	in.	in³	in³	in²	in²	%	pounds			pounds per cu. ft.			pcf	%	%	psi	psi	psi	ksi	ksi		ksi	ksi			%				%	%	inch per inch per 1°F			pcf	°F	
1C	Cinders	7.70	15.68	7.74	935.6	477.2	121.0	61.7	51	26.8	29.3	25.1	97.1	106.1	90.9	15.1	16.6	41.9	698	1357	272	1002	430	.16	914	396	.16	500	96	423	Popout	.048	92	3.4	2.8	2.9	99.9	90.7	4.04
2CES	Cinders & Expanded Slag	7.67	15.63	7.67	918.4	517.3	119.8	67.5	56	29.7	32.1	27.9	99.1	106.7	93.1	14.0	15.0	43.7	1004	1778	224	1266	520	.22	1017	426	.19	500	100	523	Stain	.052	91	4.3	3.7	3.9	91.0	92.9	4.10
2H	Haydite	7.68	15.65	7.67	922.5	515.2	120.3	67.2	55	27.8	29.4	25.1	93.1	98.7	84.2	14.1	17.3	62.6	1332	2316	--	--	--	--	--	--	--	--	--	--	None	--	--	--	--	--	--	--	--
3C	Cinders	7.69	15.74	7.63	924.2	554.3	121.1	72.6	60	27.7	32.1	26.7	88.5	100.1	83.4	16.8	20.2	19.4	754	1252	354	984	400	.24	832	360	.15	500	90	567	Stain	.031	93	3.4	2.7	2.6	[illegible]	94.7	3.85
3SG	Sand & Gravel	7.67	15.64	7.62	914.5	502.3	120.0	65.9	55	39.6	41.1	38.2	136.3	141.2	132.8	8.5	6.4	45.4	1208	2212	510	2921	1260	.16	2233	--	--	500	100	694	None	.034	89	4.9	4.1	4.0	139.1	89.0	7.69
4SG	Sand & Gravel	7.72	15.74	7.71	937.6	469.4	121.6	60.9	50	36.4	38.3	35.5	134.1	140.8	129.2	10.3	7.9	37.5	994	1983	635	3674	1489	.24	3061	1354	.24	500	100	836	None	.046	87	5.2	4.3	4.7	125.4	87.6	7.54
5CS	Sand-Fine, Cinders-Coarse	7.70	15.61	7.67	921.7	464.5	120.2	60.4	50	27.3	29.3	26.1	101.5	108.9	97.1	12.3	12.7	32.2	877	1728	344	1619	694	.17	1325	451	.17	500	97	425	Stain	.046	92	3.9	3.2	3.5	93.2	91.2	4.22
6BS	Expanded Slag	7.70	15.65	7.70	927.7	459.0	120.4	59.5	50	28.8	30.9	26.9	108.4	116.3	101.3	14.9	14.7	48.6	695	1395	338	1792	775	.16	1696	752	.13	500	97	550	None	.048	92	5.4	4.5	5.1	90.1	97.7	2.60
7W	Waylite	7.68	15.61	7.67	921.3	474.4	120.1	61.8	51	27.3	29.7	26.7	99.3	108.3	97.3	11.0	11.3	19.7	777	1506	334	2412	987	.23	2131	953	.15	500	100	584	None	.041	88	4.6	4.2	4.3	100.0	91.2	2.71
7BW	Beslite & Waylite, Combined	7.63	15.65	7.65	913.9	470.6	119.5	61.5	51	28.7	31.4	27.8	105.3	114.1	102.2	13.0	12.7	25.5	909	1923	--	--	--	--	--	--	--	--	--	--	None	--	--	--	--	--	--	--	--
8CEL	Celocrete	7.77	15.65	7.73	944.4	507.0	122.2	65.6	54	24.6	26.8	22.6	83.0	90.3	76.2	14.0	18.8	49.7	647	1199	193	1313	559	.18	1249	--	--	500	88	272	None	.077	84	5.7	4.9	5.0	82.8	93.4	2.38
9S	Air-Cooled Slag	7.64	15.61	7.62	908.3	512.6	119.3	67.3	56	38.3	40.8	37.5	129.2	137.7	126.3	12.7	9.0	27.7	1243	2195	526	2562	1083	.18	2258.	929	.13	57	62	357	None	.024	93	4.8	3.8	4.6	116.9	93.2	3.68
10CA	Cinders + Fly Ash	7.71	15.67	7.67	925.7	503.1	120.7	65.6	54	26.4	28.9	24.8	90.7	99.0	85.2	13.9	16.3	39.7	906	1663	277	1106	479	.16	891	354	.13	500	93	442	None	.036	95	3.1	2.5	2.5	91.5	89.4	4.18

APPENDIX I

AGGREGATES

A. Specific Gravity and Absorption.

Tests for determining the specific gravity and absorption of coarse, fine and combined aggregates were discussed in Chapter III. Results were given in Table 6. The equations used for calculating these results will be presented here. Detailed data are given in Tables 31, 32 and 33.

1. Coarse Aggregate.

The following formulas were used for calculations of results for coarse aggregate:

Let

A = weight in grams of oven-dried sample in air

B = weight in grams of saturated surface-dry sample in air

C = weight in grams of sample in water, and

w = unit weight of dry compacted aggregate (in grams per cc).

Then,

Bulk Specific Gravity (oven dry basis) $= \frac{A}{B-C}$ (21)

Bulk Specific Gravity (saturated surface-dry basis) $= \frac{B}{B-C}$ (22)

Apparent Specific Gravity $= \frac{A}{A-C}$ (23)

Absorption (per cent of dry weight) $= \frac{B-A}{A} \times 100$ (24)

Absorption (volume of water per unit volume of dry compacted aggregate, per cent) $= \frac{(B-A)w}{A} \times 100$ (25)

2. Fine Aggregate.

It may be repeated that unlike coarse aggregates (see Chapter III), specific gravity and absorption of the fine aggregates were determined from two different samples which were both obtained from an original sample in a saturated surface-dry condition. For the specific gravity sample, let:

a = weight in grams of oven dry sample in air, calculated from (b), (following), and the absorption.

b = weight in grams of saturated surface-dry samples in air.

v = bulk volume (cc) of sample in the flask, which is equal to the difference between the final reading and the initial reading of the flask.

For the absorption specimen, let A, B, and w be the same as for coarse aggregate.

Then,

$$a = \frac{100 \times b}{100 + \text{absorption (\% by weight)}} \qquad (26)$$

$$\text{Bulk Specific Gravity (oven dry basis)} = \frac{a}{v} \qquad (27)$$

$$\text{Bulk Specific Gravity (saturated surface-dry basis)} = \frac{b}{v} \qquad (28)$$

$$\text{Apparent Specific Gravity} = \frac{a}{v-(b-a)} \qquad (29)$$

$$\text{Absorption (per cent of weight)} = \frac{B-A}{A} \times 100$$

$$\text{Absorption (per cent of volume)} = \frac{(B-A)w}{A} \times 100$$

TABLE 31. CALCULATION OF SPECIFIC GRAVITY AND ABSORPTION OF COARSE AGGREGATE

Aggregate	6ES19		7W19		7B9		8CEL19		Re-
Sample	1	2	1	2	1	2	1	2	marks
A (gms)	730.0	892.0	1031.0	922.0	835.0	742.0	586.0	544.0	
B (gms)	779.5	952.0	1109.0	992.0	953.0	847.0	729.0	675.0	
C (gms)	284.0	347.0	412.0	370.0	392.0	352.0	120.0	104.0	
B-C (gms)	495.5	605.0	697.0	622.0	561.0	495.0	609.0	571.0	
A-C (gms)	446.0	545.0	619.0	552.0	443.0	390.0	466.0	440.0	
B-A (gms)	49.5	60.0	78.0	70.0	118.0	105.0	143.0	131.0	
B. S. G. Oven Dry Basis	1.47	1.47	1.48	1.48	1.49	1.50	0.96	0.98	Eq. 21
Average	1.47		1.48		1.50		0.97		
B. S. G. Saturated Surface Dry Basis	1.57	1.57	1.59	1.59	1.70	1.71	1.20	1.21	Eq. 22
Average	1.57		1.59		1.71		1.21		
Apparent S. G.	1.64	1.64	1.66	1.67	1.88	1.90	1.26	1.28	Eq. 23
Average	1.64		1.67		1.89		1.27		
Absorption % of Dry Weight	6.77	6.73	7.55	7.59	14.11	14.15	24.40	24.10	Eq. 24
Average	6.75		7.57		14.13		24.25		

B. Materials Causing Popouts of Concrete.

Tests for determining the presence of deleterious materials in aggregate that are capable of causing popouts of concrete were discussed in Chapter III. Here, the detailed data used in calculating the concrete mixes and determining the results will be presented.

TABLE 32. CALCULATION OF SPECIFIC GRAVITY AND ABSORPTION OF FINE AGGREGATE

Aggregate	1C9		3C9		3SG9		4SG9		5C9		6ES29		7W29		8CEI29		10CA9		Re-
Sample	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	1	2	marks
Initial Reading (cc)	0.10	0.20	0.20	1.10	0.30	0.60	0.40	0.70	0.10	0.30	0.50	0.70	0.50	0.80	0.20	0.70	0.20	1.10	
Final Reading (cc)	19.10	19.10	18.70	19.50	19.10	19.35	19.00	19.30	19.40	19.00	19.20	19.50	19.50	19.40	20.80	21.20	19.10	20.10	
v (cc)	19.00	18.90	18.50	18.40	18.80	18.75	18.60	18.60	19.30	19.20	18.70	18.80	18.50	18.60	20.60	20.50	18.90	19.00	
a (gms)	32.32	32.26	32.14	32.08	49.26	49.25	49.40	49.35	32.35	32.35	33.30	33.30	33.30	33.29	30.59	30.59	32.29	32.28	Eq.26
b (gms)	35.00	35.00	35.00	35.00	50.00	50.00	50.00	50.00	35.00	35.00	35.00	35.00	35.00	35.00	35.00	35.00	35.00	35.00	
B. S. G. Oven Dry Basis	1.70	1.71	1.74	1.74	2.62	2.63	2.66	2.65	1.68	1.68	1.78	1.77	1.80	1.79	1.49	1.49	1.71	1.70	Eq.27
Average	1.71		1.74		2.63		2.66		1.68		1.78		1.80		1.49		1.71		
B. S. G. (S. S. D.) Basis	1.84	1.85	1.89	1.90	2.65	2.67	2.69	2.69	1.81	1.82	1.87	1.86	1.89	1.88	1.70	1.71	1.85	1.84	Eq.28
Average	1.85		1.90		2.66		2.69		1.82		1.87		1.89		1.71		1.85		
Apparent S. G.	1.98	2.00	2.05	2.07	2.73	2.73	2.74	2.75	1.94	1.95	1.96	1.95	1.98	1.97	1.89	1.90	1.99	1.98	Eq.29
Average	1.99		2.06		2.73		2.75		1.95		1.96		1.98		1.90		1.99		
B (gms)	455.00	423.00	512.00	578.00	580.50	699.00	947.00	872.50	503.00	461.00	406.00	305.00	506.00	482.00	500.00	460.00	380.50	431.50	
A (gms)	420.00	390.00	470.00	530.00	572.00	690.00	935.00	861.00	465.00	426.00	387.00	290.00	481.50	458.50	437.00	402.00	351.00	398.00	
B-A (gms)	35.00	33.00	42.00	48.00	8.50	9.00	12.00	11.50	38.00	35.00	19.00	15.00	24.50	23.50	63.00	58.00	29.50	33.50	
Absorp. % by Weight	8.30	8.50	8.90	9.10	1.50	1.50	1.28	1.32	8.20	8.20	5.10	5.12	5.10	5.13	14.42	14.40	8.40	8.42	Eq.24
Average	8.40		9.00		1.50		1.30		8.20		5.11		5.12		14.41		8.41		

TABLE 27.* COEFFICIENT OF THERMAL EXPANSION
(INCH PER INCH PER 1 F) X 10^6

Specimen No.	Type of Aggregate	Saturated Concrete Test Method #1	Oven-Dried Concrete Test Method #1	Oven-Dried Concrete Test Method #2
1C6	Cinders	3.4	2.8	2.9
2CES6	Cinders & Expanded Slag	4.3	3.7	3.9
3C6	Cinders	3.4	2.7	2.6
3SG6	Sand & Gravel	4.9	4.1	4.0
4SG6	Sand & Gravel	5.2	4.3	4.7
5CS6	Cinders & Sand	3.9	3.2	3.5
6ES6	Expanded Slag	5.4	4.5	5.1
7W6	Waylite	4.6	4.2	4.3
8CEL6	Celocrete	5.7	4.9	5.0
9S6	Air-Cooled Slag	4.8	3.8	4.6
10CA6	Cinders + Fly Ash	3.1	2.5	2.5

* For detailed data see Tables 74, 75, and 76 in Appendix VII.

mainly by the cement paste. It is sufficient to repeat here the findings of Mitchell, who observed that the coefficients of vacuum saturated and oven dried concrete are approximately equal and that it is higher at intermediate moisture contents reaching maximum at the "optimum" moisture content which is estimated between 80 and 60 per cent of vacuum saturation. Table 27 also indicates small differences

Data for the concrete mixes are given in Table 34. It may be repeated that although proportioning is done by volume, measuring was done by weight as shown in column 5 of Table 34. These proportions correspond to a constant ratio of 1:6, one part cement to six parts of a combined aggregate, by volume. The amount of water was determined from trial batches to give the slump shown in Table 34. Aggregates were combined in the same proportions as that used in the plant mixes, shown in column 3 of Table 34.

The weight of the autoclave specimen before and after test is given in column 6 of Table 34. It is obvious from the results that no loss in weight, which might be caused by popouts or breakage of concrete, was suffered by the specimens; it was, therefore, concluded (see Chapter III) that the aggregates were free of deleterious popout materials.

TABLE 34. POPOUTS TEST - LABORATORY CONCRETE MIXES

1	2	3	4	5					6	
				Laboratory Mix					Weight of Concrete Spec. Saturated	
Specimen No.	Type of Aggregate	Aggregate Proportion for Plant Mix	F. M. of Combined Aggregate	w/C by Volume	Water gms	Cement gms	Combined Aggregate gms	Slump inch	Before Test gms	After Test gms
1C91	Cinders	Combined	4.19	1.07	1016	982	3668	2.70	1093	1102
1C92									1081	1090
3C91	Cinders	Combined	3.86	1.20	1116	982	3591	2.80	1121	1121
3C92									1124	1125
3SG91	Sand &	Combined	3.67	1.23	851	768	5493	2.90	1667	1667
3SG92	Gravel								1682	1680
4SG91	Sand &	Combined	3.32	1.31	703	725	5466	2.80	1698	1696
4SG92	Gravel								1700	1701
5CS91	Cinders	3100 lbs	4.41	1.15	961	896	3257	2.60	1117	1117
5CS92	& Sand	500 lbs					543		1144	1144
6ES19	Expanded	2500 lbs fine	4.78	1.04	811	896	3000	2.70	1316	1312
6ES29	Slag	450 lbs coarse					600		1343	1344
7W91	Waylite	1867 lbs fine	3.88	1.19	763	768	1764	2.75	1024	1027
7W92		933 lbs coarse					882		1026	1030
7BW91	Waylite &	1000 lbs	3.84	1.20	1098	896	1387	2.80	1199	1200
7BW92	Beslite	1600 lbs					2219		1211	1210
8CEL91	Celocrete	1280 lbs Cel. fine	4.59	1.06	1397	1067	2296	2.75	925	923
8CEL92	& Sand	220 lbs sand					397			
		320 lbs Cel.coarse					574		935	936
10CA91	Cinders	Combined	4.04	1.16	1024	939	3587	2.70	1068	1069
10CA92									1086	1086

APPENDIX II

DIMENSIONS OF THE MODULAR UNIT

Detailed data and calculations of the results are presented here. Definitions, procedure of tests, and discussion of test results are given in Chapter IV.

In Table 35 data is given for determining the net volume of each modular unit, and the average results of 5 specimens for each product.

Table 36 presents data for determining the dimensions (thickness, length and height), gross cross-sectional area and gross volume of units tested; average results for the 5 specimens used for each product are also given.

Table 37 presents net cross-sectional area as percentage of gross cross-sectional area of each modular unit; average results for the 5 specimens of each product are also given.

TABLE 35. CALCULATION OF NET VOLUME OF THE MODULAR UNITS

Specimen No.	Wet Weight gms	Suspended Weight gms	Wet Wt.- Suspended Weight= Net Vol. cc.	Net Volume in^3	Net Volume ft^3	Specimen No.	Wet Weight gms	Suspended Weight gms	Wet Wt.- Suspended Weight= Net Vol. cc.	Net Volume in^3	Net Volume ft^3
1C11	13,365	5,585	7,780	474.77	.275	6ES11	13,515	6,190	7,325	447.00	.258
1C12	13,325	5,470	7,855	479.34	.277	6ES12	13,725	6,255	7,470	456.00	.264
1C13	13,435	5,485	7,950	485.14	.281	6ES13	14,140	6,570	7,570	462.00	.267
1C14	13,335	5,475	7,860	479.65	.278	6ES14	14,050	6,475	7,575	462.30	.268
1C15	12,990	5,335	7,655	467.14	.270	6ES15	14,635	6,965	7,670	468.10	.271
Average			7,826	477.21	.276	Average			7,522	459.00	.266
2CES11	14,910	6,315	8,595	524.50	.304	7W11	13,520	5,735	7,785	475.10	.275
2CES12	14,320	5,895	8,425	514.13	.298	7W12	13,485	5,660	7,825	477.50	.276
2CES13	14,655	6,260	8,395	512.30	.297	7W13	13,735	5,940	7,795	475.70	.275
2CES14	13,580	5,525	8,055	491.55	.285	7W14	13,600	5,835	7,765	473.90	.274
2CES15	15,270	6,355	8,915	544.03	.315	7W15	13,135	5,435	7,700	469.90	.272
Average			8,477	517.30	.299	Average			7,774	474.40	.275
2H11	13,325	4,825	8,500	518.70	.300	7BW11	14,300	6,605	7,695	469.60	.272
2H12	13,445	5,005	8,440	515.04	.298	7BW12	14,380	6,610	7,770	474.20	.274
2H13	13,240	4,805	8,435	514.74	.298	7BW13	14,475	6,755	7,720	471.10	.273
2H14	13,330	4,895	8,405	512.91	.297	7BW14	14,435	6,705	7,730	471.70	.273
2H15	13,450	5,015	8,435	514.74	.298	7BW15	13,542	5,900	7,642	466.40	.270
Average			8,443	515.23	.298	Average			7,711	470.60	.272
3C11	14,770	5,555	9,215	562.34	.325	8CEL11	12,850	4,300	8,550	521.80	.302
3C12	14,895	5,885	9,010	549.80	.318	8CEL12	12,940	4,355	8,585	523.90	.318
3C13	14,450	5,375	9,075	553.80	.321	8CEL13	13,080	4,455	8,625	526.30	.305
3C14	14,250	5,235	9,015	550.10	.318	8CEL14	11,070	3,190	7,880	480.90	.278
3C15	14,470	5,390	9,080	554.10	.321	8CEL15	10,855	2,955	7,900	482.10	.279
Average			9,079	554.30	.321	Average			8,308	507.00	.296
3SG11	18,370	10,275	8,095	493.99	.286	9S11	18,270	9,975	8,295	506.20	.293
3SG12	18,230	10,050	8,180	499.18	.289	9S12	18,860	10,310	8,550	521.76	.302
3SG13	19,130	10,760	8,370	510.77	.296	9S13	18,740	10,280	8,460	516.26	.300
3SG14	18,670	10,450	8,220	501.61	.290	9S14	18,800	10,320	8,480	517.48	.300
3SG15	18,745	10,455	8,290	505.89	.293	9S15	17,965	9,750	8,215	501.31	.290
Average			8,231	502.29	.291	Average			8,400	512.60	.297
4SG11	17,395	9,630	7,765	473.85	.274	10CA11	13,165	4,960	8,205	500.70	.290
4SG12	17,150	9,510	7,640	466.20	.270	10CA12	13,060	4,825	8,235	502.50	.291
4SG13	17,775	10,045	7,730	471.70	.273	10CA13	13,010	4,810	8,200	500.40	.290
4SG14	17,565	9,830	7,735	472.00	.273	10CA14	13,070	4,810	8,260	504.06	.292
4SG15	16,880	9,270	7,610	463.40	.268	10CA15	13,140	4,820	8,320	507.72	.294
Average			7,696	469.40	.272	Average			8,244	503.08	.291
5CS11	13,465	5,870	7,595	463.50	.268						
5CS12	13,395	5,855	7,540	460.10	.266						
5CS13	13,190	5,615	7,575	462.30	.268						
5CS14	13,370	5,665	7,705	470.20	.272						
5CS15	13,300	5,655	7,645	466.50	.270						
Average			7,612	464.50	.269						

TABLE 36. CALCULATION OF GROSS AREA AND GROSS VOLUME OF THE MODULAR UNITS

Specimen No.	Thickness t in.	Length L in.	Height h in.	Gross Area t x L in^2	Gross Vol. t x L x h in^3	Gross Volume ft^3	Specimen No.	Thickness t in.	Length L in.	Height h in.	Gross Area t x L in^2	Gross Vol. t x L x h in^3	Gross Volume ft^3
1C11	7.69	15.60	7.70	119.96	923.69	.535	6ES11	7.66	15.72	7.68	120.42	924.79	.535
1C12	7.70	15.64	7.76	120.43	934.54	.541	6ES12	7.70	15.65	7.70	120.51	927.89	.537
1C13	7.76	15.75	7.76	122.22	948.27	.549	6ES13	7.70	15.61	7.70	120.20	925.52	.536
1C14	7.74	15.65	7.76	121.13	939.97	.544	6ES14	7.72	15.60	7.75	120.43	933.35	.540
1C15	7.70	15.72	7.70	121.04	932.01	.539	6ES15	7.70	15.65	7.69	120.51	926.68	.536
Average	7.70	15.68	7.74	120.69	935.60	.542	Average	7.70	15.65	7.70	120.41	927.65	.537
2CES11	7.66	15.60	7.65	119.50	914.18	.529	7W11	7.65	15.60	7.65	119.34	912.95	.528
2CES12	7.65	15.65	7.66	119.72	917.07	.531	7W12	7.71	15.70	7.70	121.05	932.06	.539
2CES13	7.62	15.62	7.70	119.02	916.48	.530	7W13	7.62	15.65	7.62	119.25	908.69	.526
2CES14	7.65	15.65	7.70	119.72	921.86	.534	7W14	7.70	15.60	7.70	120.51	924.90	.535
2CES15	7.75	15.62	7.62	121.06	922.43	.534	7W15	7.70	15.65	7.70	120.51	927.85	.537
Average	7.67	15.63	7.67	119.80	918.40	.532	Average	7.68	15.64	7.67	120.05	921.29	.533
2H11	7.65	15.65	7.65	119.72	915.87	.530	7BW11	7.62	15.65	7.61	119.25	907.52	.525
2H12	7.71	15.60	7.62	120.28	916.53	.530	7BW12	7.62	15.62	7.62	119.02	906.96	.525
2H13	7.66	15.56	7.68	119.96	921.29	.533	7BW13	7.65	15.70	7.68	120.10	922.41	.534
2H14	7.70	15.70	7.70	120.89	930.85	.539	7BW14	7.63	15.67	7.68	119.56	918.24	.531
2H15	7.70	15.65	7.70	120.51	927.89	.537	7BW15	7.65	15.62	7.65	119.49	914.12	.529
Average	7.68	15.65	7.67	120.27	922.49	.534	Average	7.63	15.65	7.65	119.48	913.85	.530
3C11	7.62	15.75	7.62	120.02	914.51	.529	8CEL11	7.79	15.75	7.70	122.69	944.71	.547
3C12	7.72	15.70	7.60	121.20	912.15	.533	8CEL12	7.79	15.72	7.71	122.46	944.15	.546
3C13	7.70	15.72	7.65	121.04	925.99	.536	8CEL13	7.80	15.70	7.85	122.46	961.31	.556
3C14	7.70	15.78	7.70	121.51	935.60	.541	8CEL14	7.66	15.70	7.70	120.26	926.02	.536
3C15	7.70	15.78	7.60	121.51	923.45	.534	8CEL15	7.80	15.75	7.70	122.58	945.95	.547
Average	7.69	15.74	7.63	121.06	924.14	.535	Average	7.77	15.72	7.73	122.14	944.43	.547
3SG11	7.65	15.60	7.60	119.34	906.98	.525	9S11	7.65	15.61	7.62	119.42	909.95	.527
3SG12	7.72	15.62	7.63	120.59	920.07	.532	9S12	7.67	15.62	7.60	119.81	910.52	.527
3SG13	7.68	15.70	7.67	120.58	924.82	.535	9S13	7.68	15.60	7.62	119.81	912.94	.528
3SG14	7.60	15.62	7.62	118.71	904.56	.524	9S14	7.57	15.62	7.62	118.24	901.01	.521
3SG15	7.70	15.65	7.60	120.51	915.84	.530	9S15	7.62	15.62	7.62	119.02	906.96	.525
Average	7.67	15.64	7.62	119.95	914.45	.529	Average	7.64	15.61	7.62	119.26	908.27	.526
4SG11	7.70	15.75	7.70	121.28	933.82	.540	10CA11	7.70	15.64	7.68	120.43	924.90	.535
4SG12	7.72	15.75	7.75	121.59	942.32	.545	10CA12	7.70	15.66	7.62	120.58	918.82	.532
4SG13	7.72	15.71	7.71	121.28	935.08	.541	10CA13	7.70	15.64	7.70	120.43	927.30	.537
4SG14	7.72	15.75	7.76	121.67	942.92	.546	10CA14	7.73	15.69	7.68	121.13	930.27	.538
4SG15	7.75	15.65	7.75	122.06	933.77	.540	10CA15	7.70	15.72	7.66	121.04	927.20	.537
Average	7.72	15.74	7.73	121.58	937.58	.543	Average	7.71	15.67	7.67	120.72	925.70	.535
5CS11	7.65	15.60	7.65	119.34	912.95	.528							
5CS12	7.75	15.61	7.65	120.98	925.47	.536							
5CS13	7.66	15.62	7.70	119.65	921.86	.534							
5CS14	7.75	15.62	7.71	121.06	933.33	.540							
5CS15	7.68	15.62	7.65	119.96	915.06	.530							
Average	7.70	15.61	7.67	120.20	921.73	.533							

TABLE 37. NET AREA AS PERCENTAGE OF GROSS AREA

Specimen No.	Net Area A_n in^2	Gross Area A_g in^2	Net Area as % of Gross Area (%)	Specimen No.	Net Area A_n in^2	Gross Area A_g in^2	Net Area as % of Gross Area (%)
1C11	61.66	119.96	51	6ES11	58.20	120.42	48
1C12	61.77	120.43	51	6ES12	59.20	120.51	49
1C13	62.52	122.22	51	6ES13	59.99	120.20	50
1C14	61.81	121.13	51	6ES14	59.64	120.43	50
1C15	60.67	121.04	50	6ES15	60.86	120.51	51
Average	61.69	120.96	51	Average	59.54	120.41	50
2CES11	68.56	119.50	57	7W11	62.10	119.34	52
2CES12	67.12	119.72	56	7W12	62.02	121.05	51
2CES13	66.53	119.02	56	7W13	62.43	119.25	52
2CES14	63.84	119.72	53	7W14	61.54	120.12	51
2CES15	71.40	121.06	59	7W15	61.02	120.51	51
Average	67.49	119.80	56	Average	61.82	120.05	51
2H11	67.80	119.72	57	7BW11	61.70	119.25	52
2H12	67.59	120.28	56	7BW12	62.23	119.02	52
2H13	67.02	119.96	56	7BW13	61.34	120.10	51
2H14	66.61	120.89	55	7BW14	61.42	119.56	51
2H15	66.85	120.51	55	7BW15	60.96	119.49	51
Average	67.19	120.27	56	Average	61.53	119.48	51
3C11	73.79	120.02	61	8CEL11	67.76	122.69	55
3C12	72.35	121.20	60	8CEL12	67.94	122.46	55
3C13	72.39	121.04	60	8CEL13	67.05	122.46	55
3C14	71.45	121.51	59	8CEL14	62.45	120.26	52
3C15	72.90	121.51	60	8CEL15	62.61	122.85	51
Average	72.58	121.06	60	Average	65.56	122.14	54
3SG11	64.99	119.34	54	9S11	66.42	119.42	56
3SG12	65.42	120.59	54	9S12	68.65	119.81	57
3SG13	66.59	120.58	55	9S13	67.75	119.81	57
3SG14	65.83	118.71	55	9S14	67.91	118.24	57
3SG15	66.56	120.51	55	9S15	65.79	119.02	55
Average	65.89	119.95	55	Average	67.32	119.26	56
4SG11	61.53	121.28	51	10CA11	65.19	120.43	54
4SG12	60.16	121.59	49	10CA12	65.94	120.58	55
4SG13	61.18	121.28	50	10CA13	64.99	120.43	54
4SG14	60.90	121.67	50	10CA14	65.63	121.13	54
4SG15	60.57	122.06	50	10CA15	66.28	121.04	55
Average	60.86	121.58	50	Average	65.60	120.72	54
5C11	60.58	119.34	51				
5C12	60.15	120.98	50				
5C13	60.03	119.65	50				
5C14	60.10	121.06	50				
5C15	60.98	119.96	51				
Average	60.42	120.20	50				

APPENDIX III

UNIT WEIGHT, ABSORPTION AND MOISTURE CONTENT

A. Unit Weight.

Average weight of the masonry units, as sampled, wet and dry conditions, are presented in Table 8 (Chapter V). Each value is the average of the results of 5 specimens tested for each product. Results of all specimens are given in Table 38 in grams (as tested) and in pounds.

Unit weights are calculated from the weight of the specimens (Table 38) using their net volume (Table 35). Results are given in Table 39 from which the average results of Table 8 are taken.

B. Absorption.

Procedure of test for determining the absorption of the unit was discussed in Chapter V. Average results of the test are presented in Table 9. Each value in the table is the result of 5 specimens used for each product tested. Results of all the specimens are presented in Table 40 in two forms: (1) in pounds per cubic foot (pcf), and (2) in percentage of dry weight of the unit. The following equations were used:

Let

W_w = wet weight of the unit, grams (see Chapter V)

W_d = dry weight of the unit, grams (see Chapter V)

v = net volume of the unit, cc (see Chapter V)

$$\text{Absorption (pcf)} = \frac{(W_w - W_d)\, 62.4}{v} \qquad (30)$$

$$\text{Absorption (per cent of dry weight)} = \frac{(W_w - W_d)\, 100}{W_d} \qquad (31)$$

TABLE 38. WEIGHTS OF MASONRY UNITS (SAMPLED, WET AND DRY CONDITIONS)

Sample No.	Sampled Weight		Wet Weight		Dry Weight		Sample No.	Sampled Weight		Wet Weight		Dry Weight	
	gms	lbs	gms	lbs	gms	lbs		gms	lbs	gms	lbs	gms	lbs
1C11	13,320	27.1	13,365	29.5	11,450	25.2	6ES11	12,575	27.7	13,515	29.8	11,655	25.7
1C12	12,204	26.9	13,325	29.4	11,455	25.3	6ES12	12,824	28.2	13,725	30.3	11,960	26.4
1C13	12,280	27.0	13,435	29.6	11,435	25.2	6ES13	13,345	29.4	14,140	31.2	12,330	27.2
1C14	12,170	26.8	13,335	29.4	11,430	25.2	6ES14	13,075	28.8	14,050	31.0	12,320	27.2
1C15	11,970	26.3	12,990	28.6	11,200	24.7	6ES15	13,635	30.0	14,635	32.3	12,810	28.2
Average	12,179	26.8		29.3	11,394	25.1	Average	13,091	28.8		30.9	12,215	26.9
2CES11	13,865	30.5	14,910	32.9	12,945	28.5	7W11	12,380	27.2	13,520	29.8	12,135	26.8
2CES12	13,320	29.3	14,320	31.6	12,555	27.7	7W12	12,350	27.2	13,485	29.7	12,030	26.5
2CES13	13,560	29.8	14,655	32.3	12,850	28.3	7W13	12,530	27.6	13,735	30.3	12,300	27.1
2CES14	12,330	27.1	13,580	29.9	11,800	26.0	7W14	12,420	27.3	13,600	30.0	12,200	26.9
2CES15	14,380	31.6	15,270	33.7	13,085	28.8	7W15	12,280	27.0	13,135	29.0	11,965	26.4
Average	13,491	29.7		32.1	12,647	27.9	Average	12,392	27.3		29.7	12,125	26.7
2H11	12,530	27.6	13,325	29.4	11,270	24.8	7BW11	13,150	28.9	14,300	31.5	12,710	28.0
2H12	12,820	28.2	13,445	29.6	11,420	25.2	7BW12	13,170	29.0	14,380	31.7	12,750	28.1
2H13	12,540	27.6	13,240	29.2	11,340	25.0	7BW13	13,280	29.2	14,475	31.9	12,905	28.4
2H14	12,540	27.6	13,330	29.4	11,400	25.1	7BW14	13,300	29.3	14,435	31.8	12,830	28.3
2H15	12,680	27.9	13,450	29.6	11,500	25.4	7BW15	12,260	27.0	13,542	29.9	11,925	26.3
Average	12,622	27.8		29.4	11,386	25.1	Average	13,032	28.7		31.4	12,625	27.8
3C11	12,793	28.1	14,770	32.6	12,250	27.0	8CEL11	11,920	26.2	12,850	28.3	11,040	24.3
3C12	12,880	28.3	14,895	32.8	12,600	27.8	8CEL12	12,160	26.8	12,940	28.5	11,150	24.6
3C13	12,508	27.5	14,450	31.9	11,960	26.4	8CEL13	12,315	27.1	13,080	28.8	11,270	24.8
3C14	12,130	26.7	14,250	31.4	11,760	26.0	8CEL14	10,130	22.3	11,070	24.4	9,100	20.1
3C15	12,697	27.9	14,470	31.9	12,060	26.6	8CEL15	9,465	20.8	10,855	23.9	8,760	19.3
Average	12,602	27.7		32.1	12,126	26.7	Average	11,198	24.6		26.8	10,264	22.6
3SG11	17,785	39.1	18,370	40.5	17,275	38.1	9S11	17,140	37.7	18,270	40.3	16,690	36.8
3SG12	17,560	38.6	18,230	40.2	17,050	37.6	9S12	17,765	39.1	18,860	41.6	17,300	38.1
3SG13	18,625	41.0	19,130	42.2	18,090	39.9	9S13	17,670	38.9	18,740	41.3	17,230	38.0
3SG14	17,920	39.4	18,670	41.2	17,450	38.5	9S14	17,635	38.8	18,800	41.4	17,290	38.1
3SG15	18,200	40.0	18,745	41.3	17,710	39.0	9S15	16,900	37.2	17,965	39.6	16,480	36.3
Average	18,018	39.6		41.1	17,515	38.2	Average	17,422	38.3		40.8	16,998	37.5
4SG11	16,570	36.5	17,395	38.3	16,100	35.5	10CA11	12,120	26.7	13,165	29.0	11,310	24.9
4SG12	16,208	35.7	17,150	37.8	15,745	34.7	10CA12	12,030	26.5	13,060	28.8	11,270	24.9
4SG13	17,230	37.9	17,775	39.2	16,735	36.9	10CA13	11,920	26.3	13,010	28.7	11,200	24.7
4SG14	16,840	37.1	17,565	38.7	16,405	36.2	10CA14	11,980	26.4	13,070	28.8	11,240	24.8
4SG15	15,930	35.1	16,880	37.2	15,460	34.0	10CA15	11,870	26.2	13,140	29.0	11,260	24.8
Average	16,556	36.4		38.3	16,089	35.5	Average	11,984	26.4		28.9	11,256	24.8
5CS11	12,610	27.7	13,465	29.7	12,100	26.7							
5CS12	12,580	27.7	13,395	29.5	12,030	26.5							
5CS13	12,280	27.0	13,190	29.1	11,700	25.8							
5CS14	12,240	26.9	13,370	29.5	11,695	25.8							
5CS15	12,280	27.0	13,000	28.7	11,680	25.7							
Average	12,398	27.3		29.3	11,841	26.1							

TABLE 39. CALCULATION OF WEIGHTS OF MASONRY UNITS (SAMPLED, WET AND DRY CONDITIONS) IN POUNDS PER CUBIC FOOT (BASED ON NET VOLUME)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Sample No.	Weight of the Unit (pound)			Net Volume of the Unit ft^3	Unit Weight pcf			Sample No.	Weight of the Unit (pound)			Net Volume of the Unit ft^3	Unit Weight pcf		
	Sampled	Wet	Dry		Sampled	Wet	Dry		Sampled	Wet	Dry		Sampled	Wet	Dry
1C11	27.1	29.5	25.2	.275	98.6	107.2	91.9	6ES11	27.7	29.8	25.7	.258	107.0	115.2	99.3
1C12	26.9	29.4	25.3	.277	96.8	105.9	91.0	6ES12	28.2	30.3	26.4	.264	106.9	114.7	99.9
1C13	27.0	29.6	25.2	.281	96.2	105.5	89.7	6ES13	29.4	31.2	27.2	.267	109.8	116.6	101.7
1C14	26.8	29.4	25.2	.278	96.4	105.9	90.7	6ES14	28.8	31.0	27.2	.268	107.6	115.8	101.5
1C15	26.3	28.6	24.7	.270	97.4	105.9	91.3	6ES15	30.0	32.3	28.2	.271	110.7	119.1	104.2
Average	26.8	29.3	25.1	.276	97.1	106.1	90.9	Average	28.8	30.9	26.9	.266	108.4	116.3	101.3
2CES11	30.5	32.9	28.5	.304	100.5	108.3	94.0	7W11	27.2	29.8	26.8	.275	99.1	108.4	97.3
2CES12	29.3	31.6	27.7	.298	98.5	106.1	93.0	7W12	27.2	29.7	26.5	.276	98.3	107.6	95.5
2CES13	29.8	32.3	28.3	.297	100.6	108.9	95.5	7W13	27.6	30.3	27.1	.275	100.2	110.0	98.5
2CES14	27.1	29.9	26.0	.285	95.4	105.2	91.3	7W14	27.3	30.0	26.9	.274	99.6	109.3	98.1
2CES15	31.6	33.7	28.8	.315	100.5	104.8	91.6	7W15	27.0	29.0	26.4	.272	99.4	106.5	97.0
Average	29.7	32.1	27.9	.299	99.1	106.7	93.1	Average	27.3	29.7	26.7	.275	99.3	108.3	97.3
2H11	27.6	29.4	24.8	.300	91.9	97.8	82.7	7BW11	28.9	31.5	28.0	.272	106.5	116.0	103.1
2H12	28.2	29.6	25.2	.298	94.6	99.4	84.4	7BW12	29.0	31.7	28.1	.274	105.6	115.5	102.4
2H13	27.6	29.2	25.0	.298	92.6	98.0	83.9	7BW13	29.2	31.9	28.4	.273	107.2	117.0	104.3
2H14	27.6	29.4	25.1	.297	93.0	99.0	84.7	7BW14	29.3	31.8	28.3	.273	107.2	116.5	103.6
2H15	27.9	29.6	25.4	.298	93.7	99.5	85.1	7BW15	27.0	29.9	26.3	.270	99.9	105.6	97.3
Average	27.8	29.4	25.1	.298	93.1	98.7	84.2	Average	28.7	31.4	27.8	.272	105.3	114.1	102.2
3C11	28.1	32.6	27.0	.325	86.5	100.0	83.0	8CEL11	26.2	28.3	24.3	.302	86.9	93.8	80.6
3C12	28.3	32.8	27.8	.318	98.1	103.2	87.3	8CEL12	26.8	28.5	24.6	.318	84.3	89.8	77.4
3C13	27.5	31.9	26.4	.321	85.9	99.4	82.3	8CEL13	27.1	28.8	24.8	.305	88.9	94.7	81.6
3C14	26.7	31.4	26.0	.318	84.8	98.7	81.4	8CEL14	22.3	24.4	20.1	.278	80.1	87.7	72.1
3C15	27.9	31.9	26.6	.321	87.1	99.4	82.9	8CEL15	20.8	23.9	19.3	.279	74.6	85.7	69.2
Average	27.7	32.1	26.7	.321	88.5	100.1	83.4	Average	24.6	26.8	22.6	.296	83.0	90.3	76.2
3SG11	39.1	40.5	38.1	.286	136.9	141.6	133.2	9S11	37.7	40.3	36.8	.293	128.8	137.5	125.6
3SG12	38.6	40.2	37.6	.289	133.7	139.1	130.1	9S12	39.1	41.6	38.1	.302	129.5	137.7	126.3
3SG13	41.0	42.2	39.9	.296	138.6	142.6	134.9	9S13	38.9	41.3	38.0	.300	130.1	138.2	127.1
3SG14	39.4	41.2	38.5	.290	135.8	141.8	132.5	9S14	38.8	41.4	38.1	.300	129.6	138.4	127.3
3SG15	40.0	41.3	39.0	.293	136.8	141.1	133.3	9S15	37.2	39.6	36.3	.290	128.2	136.5	125.2
Average	39.6	41.1	38.2	.291	136.3	141.2	132.8	Average	38.3	40.8	37.5	.297	129.2	137.7	126.3
4SG11	36.5	38.3	35.5	.274	132.9	139.8	129.4	10CA11	26.7	29.0	24.9	.290	92.1	100.1	86.0
4SG12	35.7	37.8	34.7	.270	132.2	140.1	128.6	10CA12	26.5	28.8	24.9	.291	91.2	98.9	85.4
4SG13	37.9	39.2	36.9	.273	138.9	143.5	135.1	10CA13	26.3	28.7	24.7	.290	90.6	98.9	85.1
4SG14	37.1	38.7	36.2	.273	135.6	141.7	132.4	10CA14	26.4	28.8	24.8	.292	90.5	98.7	84.9
4SG15	35.1	37.2	34.0	.268	131.7	138.7	127.0	10CA15	26.2	29.0	24.8	.294	89.0	98.5	84.4
Average	36.4	38.3	35.5	.272	134.1	140.8	129.2	Average	26.4	28.9	24.8	.291	90.7	99.0	85.2
5CS11	27.7	29.7	26.7	.268	103.4	110.7	99.4								
5CS12	27.7	29.5	26.5	.266	103.9	110.9	99.6								
5CS13	27.0	29.1	25.8	.268	101.0	108.7	96.4								
5CS14	26.9	29.5	25.8	.272	99.0	108.3	94.7								
5CS15	27.0	28.7	25.7	.270	100.1	106.1	95.3								
Average	27.3	29.3	26.1	.269	101.5	108.9	97.1								

TABLE 40. ABSORPTION OF THE MASONRY UNITS

1	2	3	4	5	6	7	8
Sample No.	Absorbed Water gms	Absorbed Water pcf	Absorbed Water per cent of Dry Weight %	Sample No.	Absorbed Water gms	Absorbed Water pcf	Absorbed Water per cent of Dry Weight %
1C11	1,915	15.4	16.7	6ES11	1,860	15.8	16.0
1C12	1,870	14.9	16.3	6ES12	1,765	14.7	14.8
1C13	2,000	15.7	17.5	6ES13	1,810	14.9	14.7
1C14	1,905	15.1	16.7	6ES14	1,727	14.2	14.0
1C15	1,790	14.6	16.0	6ES15	1,825	14.8	14.3
Average	1,896	15.1	16.6	Average	1,797	14.9	14.7
2CES11	1,965	14.3	15.2	7W11	1,385	11.1	11.4
2CES12	1,765	13.1	14.1	7W12	1,455	11.6	12.1
2CES13	1,805	13.4	14.1	7W13	1,435	11.5	11.7
2CES14	1,780	13.8	15.1	7W14	1,400	11.3	11.5
2CES15	2,185	15.3	16.7	7W15	1,170	9.5	9.8
Average	1,900	14.0	15.0	Average	1,369	11.0	11.3
2H11	2,055	15.1	18.2	7BW11	1,590	12.9	12.5
2H12	2,025	15.0	17.7	7BW12	1,630	13.1	12.8
2H13	1,900	14.1	16.8	7BW13	1,570	12.7	12.2
2H14	1,930	12.0	16.9	7BW14	1,605	13.0	12.5
2H15	1,950	14.4	17.0	7BW15	1,617	13.2	13.6
Average	1,972	14.1	17.3	Average	1,602	13.0	12.7
3C11	2,520	17.1	20.6	8CEL11	1,810	13.2	16.4
3C12	2,295	15.9	18.2	8CEL12	1,790	13.0	16.1
3C13	2,490	17.1	20.8	8CEL13	1,810	13.1	16.1
3C14	2,490	17.2	21.2	8CEL14	1,970	14.3	21.7
3C15	2,410	16.6	20.0	8CEL15	2,095	16.6	23.9
Average	2,441	16.8	20.2	Average	1,895	14.0	18.8
3SG11	1,095	8.4	6.3	9S11	1,580	11.9	9.5
3SG12	1,180	9.0	7.0	9S12	1,560	11.2	9.0
3SG13	1,040	7.8	5.8	9S13	1,510	11.2	8.8
3SG14	1,220	9.3	7.0	9S14	1,510	17.8	8.7
3SG15	1,035	7.8	5.8	9S15	1,485	11.3	9.0
Average	1,110	8.5	6.4	Average	1,529	12.7	9.0
4SG11	1,295	10.4	8.0	10CA11	1,855	14.1	16.4
4SG12	1,405	11.5	8.9	10CA12	1,790	13.6	15.9
4SG13	1,040	8.4	6.2	10CA13	1,810	13.8	16.2
4SG14	1,160	9.4	7.1	10CA14	1,830	13.8	16.3
4SG15	1,420	11.6	9.2	10CA15	1,880	14.1	16.7
Average	1,264	10.3	7.9	Average	1,833	13.9	16.3
5CS11	1,365	11.2	11.3				
5CS12	1,365	11.3	11.4				
5CS13	1,490	12.3	12.7				
5CS14	1,675	13.6	14.3				
5CS15	1,620	13.2	13.9				
Average	1,503	12.3	12.3				

C. Moisture Content.

Procedure for determining the moisture content of the units, as sampled, was discussed in Chapter V. Average results of the test are presented in Table 9. These averages are taken from Table 41 in which the results of tests for all specimens are calculated.

Moisture contents are presented as percentages of the absorption and were calculated by using the following equation:

Let

W_s = weight of the specimen as sampled

W_w and W_d as before

Thus,

$$\text{Moisture Content (per cent of absorption)} = \frac{(W_s - W_d)\ 100}{W_w - W_d} \qquad (32)$$

TABLE 41. CALCULATION OF MOISTURE CONTENT OF THE MASONRY UNITS AS SAMPLED

1	2	3	4	5	6	7	8
Sample No.	Sampled Moisture Content gms	Absorption gms	Moisture Content as per cent of Absorption	Sample No.	Sampled Moisture Content gms	Absorption gms	Moisture Content as per cent of Absorption
1C11	870	1,915	45.4	6ES11	920	1,860	49.5
1C12	749	1,870	40.0	6ES12	854	1,765	48.4
1C13	845	2,000	42.3	6ES13	1,015	1,810	56.1
1C14	740	1,905	38.8	6ES14	755	1,727	43.7
1C15	770	1,790	43.0	6ES15	825	1,825	45.2
Average	795	1,896	41.9	Average	874	1,797	48.6
2CES11	920	1,965	46.8	7W11	245	1,385	17.7
2CES12	765	1,765	43.3	7W12	320	1,455	22.0
2CES13	710	1,805	39.3	7W13	230	1,435	16.0
2CES14	530	1,780	29.8	7W14	220	1,400	15.7
2CES15	1,295	2,185	59.3	7W15	315	1,170	26.9
Average	844	1,900	43.7	Average	266	1,369	19.7
2H11	1,260	2,055	61.3	7BW11	440	1,590	27.7
2H12	1,400	2,025	69.1	7BW12	420	1,630	25.8
2H13	1,200	1,900	63.2	7BW13	375	1,570	23.9
2H14	1,140	1,930	59.1	7BW14	470	1,605	29.3
2H15	1,180	1,950	60.5	7BW15	335	1,617	20.7
Average	1,236	1,972	62.6	Average	408	1,602	25.5
3C11	543	2,520	21.5	8CEL11	880	1,810	48.6
3C12	280	2,295	12.2	8CEL12	1,010	1,790	56.4
3C13	548	2,490	22.0	8CEL13	1,045	1,810	57.7
3C14	370	2,490	14.9	8CEL14	1,030	1,970	52.3
3C15	637	2,410	26.4	8CEL15	705	2,095	33.7
Average	476	2,441	19.4	Average	934	1,895	49.7
3SG11	510	1,095	46.6	9S11	450	1,580	28.5
3SG12	510	1,180	43.2	9S12	465	1,560	29.8
3SG13	535	1,040	51.4	9S13	440	1,510	29.1
3SG14	470	1,220	38.5	9S14	345	1,510	22.8
3SG15	490	1,035	47.3	9S15	420	1,485	28.3
Average	503	1,110	45.4	Average	424	1,529	27.7
4SG11	470	1,295	36.3	10CA11	810	1,855	43.7
4SG12	463	1,405	33.0	10CA12	760	1,790	42.5
4SG13	495	1,040	47.6	10CA13	720	1,810	39.8
4SG14	435	1,160	37.5	10CA14	736	1,830	40.2
4SG15	470	1,420	33.1	10CA15	610	1,880	32.5
Average	467	1,264	37.5	Average	727	1,833	39.7
5CS11	510	1,365	37.4				
5CS12	550	1,365	40.3				
5CS13	580	1,490	38.9				
5CS14	545	1,675	32.5				
5CS15	600	1,620	37.0				
Average	557	1,503	37.2				

APPENDIX IV

COMPRESSIVE STRENGTH

Compressive strength of concrete masonry units is discussed in Chapter VI. Results are given in Table 10; each is the average of the results of test on 5 units. Detailed data is given in Table 43. Results are based on the gross cross-sectional area of the units which are determined by actual measurement of the overall dimensions of the unit. Results were also presented on the basis of the net cross-sectional areas of the units which are the average net areas of a similar specimen (five per product), as reported in Table 7 (Chapter IV).

Therefore:

Compressive strength (based on gross area) psi =

$$\frac{\text{compressive force (pound)}}{\text{gross area (in}^2\text{)}} \qquad (33)$$

Compressive strength (based on net area) psi =

$$\frac{\text{compressive force (pound)}}{\text{net area (in}^2\text{)}} \qquad (34)$$

Detailed data of the gross cross-sectional area is given in Table 42; average results are also given in Table 10.

TABLE 42. CALCULATION OF GROSS CROSS-SECTIONAL AREAS OF MASONRY UNITS, USED FOR COMPRESSIVE STRENGTH TEST

1	2	3	4	5	6	7	8
Specimen No.	Length L in.	Thickness t in.	Gross Cross-Sectional Area t x L in^2	Specimen No.	Length L in.	Thickness t in.	Gross Cross-Sectional Area t x L in^2
1C21	15.71	7.70	121.0	6ES21	15.65	7.68	120.2
1C22	15.63	7.69	120.2	6ES22	15.55	7.70	119.7
1C23	15.60	7.62	118.9	6ES23	15.58	7.71	120.1
1C24	15.63	7.68	120.0	6ES24	15.58	7.72	120.3
1C25	15.69	7.65	120.0	6ES25	15.63	7.61	118.9
Average	15.65	7.67	120.0	Average	15.60	7.68	119.9
2CES21	15.62	7.68	120.0	7W21	15.55	7.65	119.0
2CES22	15.60	7.60	118.6	7W22	15.62	7.62	119.0
2CES23	15.62	7.68	120.0	7W23	15.62	7.66	119.6
2CES24	15.65	7.62	119.3	7W24	15.66	7.66	119.6
2CES25	15.62	7.66	119.7	7W25	15.64	7.65	119.7
Average	15.62	7.65	119.6	Average	15.62	7.65	119.4
2H21	15.62	7.63	119.2	7BW21	15.59	7.72	120.4
2H22	15.63	7.63	119.3	7BW22	15.58	7.65	119.2
2H23	15.65	7.62	119.3	7BW23	15.60	7.65	119.3
2H24	15.60	7.62	118.9	7BW24	15.62	7.65	119.5
2H25	15.62	7.60	118.7	7BW25	15.62	7.66	119.7
Average	15.62	7.62	119.1	Average	15.60	7.66	119.6
3C21	15.70	7.72	121.2	8CEL21	15.71	7.75	121.8
3C22	15.70	7.68	120.6	8CEL22	15.70	7.76	121.8
3C23	15.66	7.70	120.6	8CEL23	15.75	7.72	121.6
3C24	15.67	7.68	120.4	8CEL24	15.73	7.76	122.1
3C25	15.65	7.67	120.0	8CEL25	15.75	7.69	121.1
Average	15.68	7.69	120.5	Average	15.73	7.74	121.7
3SG21	15.70	7.75	121.7	9S21	15.59	7.65	119.3
3SG22	15.70	7.68	120.6	9S22	15.60	9.62	118.9
3SG23	15.70	7.70	120.9	9S23	15.62	7.63	119.2
3SG24	15.70	7.68	120.6	9S24	15.56	7.62	118.6
3SG25	15.70	7.65	120.1	9S25	15.60	7.62	118.9
Average	15.70	7.69	120.8	Average	15.59	7.63	119.0
4SG21	15.65	7.71	120.7	10CA21	15.70	7.65	120.1
4SG22	15.78	7.72	121.8	10CA22	15.66	7.72	120.9
4SG23	15.75	7.71	121.4	10CA23	15.65	7.70	120.5
4SG24	15.72	7.70	121.0	10CA24	15.63	7.70	120.4
4SG25	15.74	7.75	122.1	10CA25	15.70	7.68	120.6
Average	15.73	7.72	121.4	Average	15.67	7.69	120.5
5CS21	15.55	7.65	119.0				
5CS22	15.63	7.63	119.3				
5CS23	15.51	7.65	118.7				
5CS24	15.61	7.63	119.1				
5CS25	15.62	7.62	119.0				
Average	15.58	7.64	119.0				

TABLE 43. CALCULATION OF COMPRESSIVE STRENGTH OF CONCRETE MASONRY UNITS (BASED ON GROSS CROSS-SECTIONAL AREA)

1	2	3	4	5	6	7	8
Specimen No.	Compressive Force lbs	Gross Cross-Sectional Area in^2	Compressive Strength psi	Specimen No.	Compressive Force lbs	Gross Cross-Sectional Area in^2	Compressive Strength psi
1C21	83,000	121.0	686	6ES21	78,000	120.2	649
1C22	89,000	120.2	740	6ES22	110,000	119.7	927
1C23	85,500	118.9	719	6ES23	80,000	120.1	666
1C24	73,000	120.0	606	6ES24	65,000	120.3	540
1C25	89,000	120.0	740	6ES25	82,000	118.9	694
Average	83,900	120.0	698	Average	83,000	119.9	695
2CES21	145,000	120.0	1,209	7W21	77,500	119.0	651
2CES22	107,500	118.6	907	7W22	81,500	119.0	681
2CES23	127,500	120.0	1,063	7W23	103,500	119.6	865
2CES24	118,000	119.3	990	7W24	111,500	119.6	933
2CES25	102,000	119.7	852	7W25	90,500	119.7	756
Average	120,000	119.6	1,004	Average	92,900	119.4	777
2H21	162,100	119.2	1,360	7BW21	126,000	120.4	1,047
2H22	172,700	119.3	1,448	7BW22	110,500	119.2	927
2H23	157,600	119.3	1,322	7BW23	131,000	119.3	1,098
2H24	152,600	118.9	1,284	7BW24	112,500	119.5	942
2H25	148,100	118.7	1,248	7BW25	111,500	119.7	932
Average	158,620	119.1	1,332	Average	118,300	119.6	989
3C21	92,500	121.2	763	8CEL21	66,000	121.8	542
3C22	95,500	120.6	792	8CEL22	118,000	121.8	969
3C23	79,000	120.6	655	8CEL23	65,000	121.6	535
3C24	102,500	120.4	852	8CEL24	85,000	122.1	696
3C25	85,000	120.0	708	8CEL25	59,500	121.1	491
Average	90,900	120.5	754	Average	78,600	121.7	647
3SG21	125,000	121.7	1,027	9S21	145,000	119.3	1,217
3SG22	177,000	120.6	1,468	9S22	162,100	118.9	1,364
3SG23	95,000	120.9	786	9S23	142,600	119.7	1,197
3SG24	171,500	120.6	1,422	9S24	130,100	118.6	1,097
3SG25	160,350	120.1	1,335	9S25	159,100	118.9	1,338
Average	145,770	120.8	1,208	Average	147,800	119.1	1,243
4SG21	124,000	120.7	1,028	10CA21	115,600	120.1	963
4SG22	113,500	121.8	932	10CA22	116,000	120.9	960
4SG23	140,000	121.4	1,153	10CA23	96,000	120.5	797
4SG24	135,000	121.0	1,115	10CA24	95,000	120.4	789
4SG25	91,000	122.1	746	10CA25	123,000	120.6	1,020
Average	120,700	121.4	995	Average	109,120	120.5	906
5CS21	113,000	119.0	950				
5CS22	113,000	119.3	948				
5CS23	89,000	118.7	750				
5CS24	102,000	119.1	856				
5CS25	105,000	119.0	882				
Average	104,400	119.0	877				

APPENDIX V

FLEXURAL STRENGTH

In calculating the modulus of rupture (R), the assumptions made were: (1) concrete is homogeneous elastic material and remains so near maximum stress, (2) the transverse section of the beam plane before bending remains so after bending, and (3) that the moduli of elasticities are equal in tension and compression, and the section stress diagram is triangular. Assuming also that the breaking section is in mid-span, the following formula can be concluded for the stresses at this section of a simply supported beam.[34]

Let

S = stresses at the extreme fiber of the beam (psi)

P = transverse concentrated load at mid-span (pounds)

M = bending moment caused by P, at mid-span (pounds)

L = span length (inch)

d = depth of the cross section (inch)

b = width of the cross section (inch)

$c = \frac{d}{2}$ = distance of the extreme fibers from the neutral axis (inch)

$I = \frac{b\ d^3}{12}$ = moment of inertia of the section along the neutral axis (in^4)

Then,

$$S = \frac{Mc}{I} \qquad (1)$$

but $M = \frac{PL}{4}$, $c = \frac{d}{2}$, and $I = \frac{b\ d^3}{12}$

Substitute in equation (1) and reduce to equation (2) below:

$$S = \frac{3\ P\ L}{2\ bd^2} \qquad (2)$$

When P is equal to the breaking force then, by definition, S is equal to the modulus of rupture (R).

It is realized that the assumptions made are not all applicable to the breaking load; however, the approximate value obtained from the formula is useful for practical applications.

APPENDIX VI

ELASTIC PROPERTIES

In Chapter VIII the elastic constants E, G, and μ were studied as determined by two methods of tests; namely, the dynamic (sonic) and the static. Detailed data and calculations will be presented here along with discussion of certain points in the tests.

A. Dynamic Method.

1. Identification of the Mode of Vibration.

Procedure of test for determining the resonant frequencies of the fundamental transverse and torsional vibrations was discussed in Chapter VIII. It was stated that each vibration is identified by the determination of its nodal points and phase difference between the driver and the pickup vibrations. This was accomplished by bringing the pickup needle in contact with the key point along the length of the specimen while observing the Lissajou's figure on the screen of the oscillograph. The identities of the two vibrations (transverse and torsional) are shown in Figure 47.

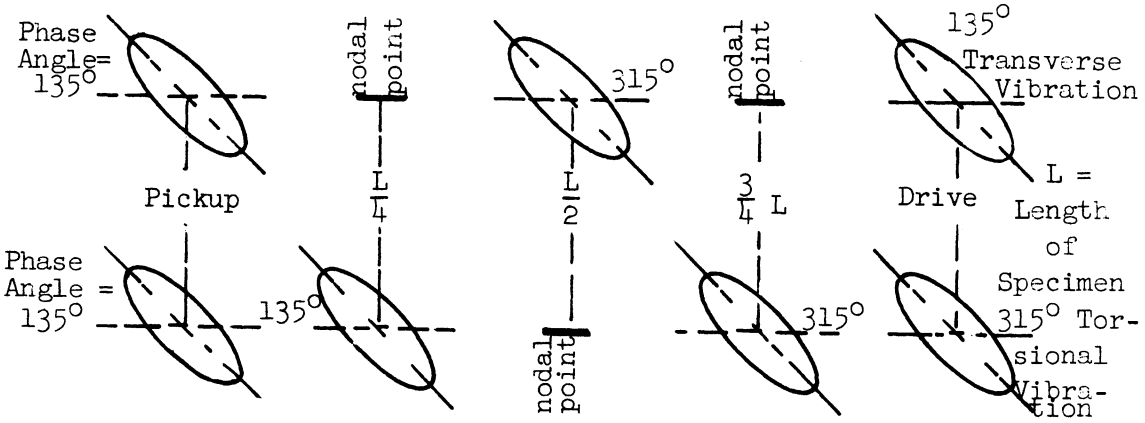


Figure 47. Identification of Flexural and Torsional Modes of Vibration

2. Calculations of the Elastic Constants.

Calculation of the elastic constants E, G, and μ have been discussed in Chapter VIII and summarized in Tables 14 and 15. Detailed data of the calculation are presented here in Tables 44 to 49 which were discussed in the previous presentation in Chapter VIII.

B. Static Method.

1. Preliminary Investigations.

Three methods of preparing the surfaces of the specimen before cementing SR-4 gages were discussed in Chapter VIII. It was stated that the second method (surface cavities filled with plaster of Paris) was adopted after a comparative study was made of the performances of gages cemented to faces of the specimen that have been prepared by this method and the first method (surface cavities filled with Duco cement). It was also stated that this study was performed on specimens taken from the products 2CES5, 3SG5, and 9S5. The procedure of this test was described in Chapter VIII while the results are presented in the following (Tables 50, 51, and 52, and Figures 48, 49, and 50) which indicate that gages cemented to surfaces that had been prepared by the first and second method gave almost identical results.

2. Calculations.

Calculations of the elastic constants were discussed in Chapter VIII and summarized in Table 17. Detailed stress-strain data for the vertical and horizontal (if present) gages of each specimen from which these results were obtained are presented here in Tables 53 to 73. These data were then plotted in Figures 51 to 71. Young's modulus of elasticity was then calculated for each

specimen from the slope of the straight portion of the stress-strain curve of the vertical gage; Poisson's ratio was calculated as the average ratio of the horizontal and vertical unit strain, and the modulus of elasticity in shear G was then calculated from E and by previously given (Chapter VIII) equation (15). Values near zero stress are considered unreliable and were excluded from the calculations.

TABLE 44. DIMENSIONAL ANALYSIS OF SPECIMENS FOR THE SONIC TEST

1	2	3	4	5	6	7	8	9
Specimen No.	Dimensions of the Section a in	b in	Length L in	Area of Section A=a x b in^2	$\frac{a}{b}$	$\frac{b}{a}$	$\frac{L}{a}$	$\frac{L}{b}$
1C51	2.56	3.63	15.66	9.29	.71	1.42	6.12	4.31
1C52	2.52	3.63	15.66	9.15	.69	1.44	6.21	4.31
2CES51	2.55	3.64	15.63	9.28	.70	1.43	6.12	4.29
2CES52	2.37	3.64	15.63	8.63	.65	1.54	6.59	4.29
3C51	2.39	3.63	15.76	8.68	.66	1.52	6.59	4.34
3C52	2.44	3.63	15.76	8.86	.67	1.49	6.46	4.34
3SG51	2.55	3.63	15.70	9.26	.70	1.42	6.16	4.33
3SG52	2.40	3.63	15.70	8.70	.66	1.51	6.54	4.33
4SG51	2.47	3.75	15.76	9.26	.66	1.52	6.38	4.20
4SG52	2.52	3.75	15.76	9.45	.67	1.49	6.25	4.20
5CS51	2.49	3.64	15.63	9.06	.68	1.46	6.28	4.29
5CS52	2.55	3.64	15.63	9.28	.70	1.43	6.13	4.29
6ES51	2.49	3.64	15.63	9.06	.68	1.46	6.28	4.29
6ES52	2.48	3.64	15.63	9.03	.68	1.47	6.30	4.29
7W51	2.58	3.63	15.63	9.37	.71	1.41	6.05	4.31
7W52	2.34	3.63	15.63	8.49	.65	1.55	6.68	4.31
8CEL51	2.49	3.75	15.75	9.34	.66	1.51	6.33	4.20
8CEL52	2.35	3.75	15.75	8.81	.63	1.60	6.70	4.20
9S51	2.48	3.63	15.63	9.00	.68	1.46	6.30	4.30
9S52	2.45	3.63	15.63	8.89	.68	1.48	6.37	4.30
10CA51	2.41	3.63	15.63	8.75	.66	1.51	6.48	4.31
10CA52	2.37	3.63	15.63	8.60	.65	1.53	6.59	4.31

TABLE 45. WEIGHTS AND RADIUS OF GYRATION - LENGTH OF SPECIMENS FOR THE SONIC TEST

1	2	3	4	5	6
Specimen No.	Radius of Gyration r in.	Length L in.	$\frac{r}{L}$	Weight W gms	Weight W pounds
1C51	1.049	15.66	.0670	3100	6.84
1C52	1.049	15.66	.0670	3040	6.70
2CES51	1.052	15.63	.0673	3350	7.39
2CES52	1.052	15.63	.0673	3140	6.92
3C51	1.049	15.76	.0667	2830	6.24
3C52	1.049	15.76	.0667	2860	6.31
3SG51	1.049	15.70	.0668	4820	10.63
3SG52	1.049	15.70	.0668	4425	9.76
4SG51	1.084	15.76	.0688	5000	11.03
4SG52	1.084	15.76	.0688	4920	10.85
5CS51	1.052	15.63	.0673	3550	7.83
5CS52	1.052	15.63	.0673	3620	7.98
6ES51	1.052	15.63	.0673	3800	8.38
6ES52	1.052	15.63	.0673	3725	8.21
7W51	1.049	15.63	.0671	3920	8.64
7W52	1.049	15.63	.0671	3510	7.74
8CEL51	1.084	15.75	.0688	3150	6.95
8CEL52	1.084	15.75	.0688	2890	6.37
9S51	1.049	15.63	.0671	4320	9.53
9S52	1.049	15.63	.0671	4200	9.26
10CA51	1.049	15.63	.0671	3140	6.92
10CA52	1.049	15.63	.0671	3000	6.62

TABLE 46. CALCULATION OF THE COEFFICIENT (C) FOR THE FLEXURAL VIBRATION

1	2	3	4	5	6	7
	Driving in b direction			Driving in a direction		
Specimen No.	$\frac{L}{b}$	C x a* sec^2/in	C sec^2/in^2	$\frac{L}{a}$	C x b* sec^2/in	C sec^2/in^2
1C51	4.31	.270	.106	6.12	.678	.187
1C52	4.31	.270	.107	6.21	.700	.193
2CES51	4.29	.268	.105	6.12	.678	.186
2CES52	4.29	.268	.113	6.59	.818	.225
3C51	4.34	.275	.115	6.59	.820	.226
3C52	4.34	.275	.113	6.46	.780	.215
3SG51	4.33	.274	.108	6.16	.690	.190
3SG52	4.33	.274	.114	6.54	.810	.223
4SG51	4.20	.253	.102	6.38	.760	.203
4SG52	4.20	.253	.100	6.25	.253	.192
5CS51	4.29	.266	.107	6.28	.725	.199
5CS52	4.29	.266	.104	6.13	.680	.187
6ES51	4.29	.266	.107	6.28	.720	.198
6ES52	4.29	.266	.107	6.30	.730	.201
7W51	4.31	.270	.105	6.05	.660	.182
7W52	4.31	.270	.115	6.68	.855	.236
8CEL51	4.20	.254	.102	6.33	.740	.197
8CEL52	4.20	.254	.108	6.70	.860	.229
9S51	4.30	.270	.109	6.30	.730	.201
9S52	4.30	.270	.110	6.37	.750	.207
10CA51	4.31	.270	.112	6.48	.790	.218
10CA52	4.31	.270	.114	6.59	.825	.227

* Values in columns 3 and 6 are selected from Pickett's curve (Figure 21).

TABLE 47. RESONANT FREQUENCIES FOR FLEXURAL AND TORSIONAL VIBRATION

1	2	3	4	5	6	7	8
	Flexural Vibration				Torsional Vibration		
	Drive in b direction		Drive in a direction		Drive in b direction	Drive in a direction	
Specimen No.	n cycle/sec	n^2 $(cy/sec)^2$	n cycle/sec	n^2 $(cy/sec)^2$	n' cycle/sec	n' cycle/sec	$(n')^2$ $(cy/sec)^2$
1C51	1,185	1,404,000	845	714,000	1,600	1,600	2,560,000
1C52	1,200	1,440,000	900	810,000	1,660	1,660	2,755,600
2CES51	1,275	1,626,000	930	865,000	1,700	1,700	2,890,000
2CES52	1,300	1,690,000	910	828,000	1,700	1,700	2,890,000
3C51	1,160	1,346,000	840	706,000	1,540	1,540	2,372,000
3C52	1,195	1,428,000	840	706,000	1,580	1,580	2,496,000
3SG51	1,610	2,592,100	1,200	1,440,000	2,220	2,220	4,928,000
3SG52	1,650	2,722,500	1,130	1,276,000	2,200	2,200	4,840,000
4SG51	1,900	3,610,000	1,320	1,742,000	2,410	2,410	5,808,000
4SG52	1,810	3,276,000	1,230	1,513,000	2,300	2,300	5,290,000
5CS51	1,330	1,769,000	980	960,000	1,820	1,820	3,312,000
5CS52	1,440	2,074,000	1,090	1,188,000	1,990	1,990	3,960,000
6ES51	1,430	2,045,000	1,050	1,103,000	1,960	1,960	3,842,000
6ES52	1,400	1,960,000	1,040	1,082,000	1,930	1,930	3,725,000
7W51	1,620	2,624,000	1,230	1,513,000	2,230	2,230	4,973,000
7W52	1,610	2,592,000	1,190	1,416,000	2,145	2,145	4,601,000
8CEL51	1,400	1,960,000	1,000	1,000,000	1,880	1,880	3,534,000
8CEL52	1,350	1,823,000	920	846,000	1,720	1,720	2,958,000
9S51	1,570	2,465,000	1,120	1,254,000	2,120	2,120	4,494,000
9S52	1,630	2,657,000	1,160	1,346,000	2,175	2,175	4,731,000
10CA51	1,230	1,513,000	874	764,000	1,670	1,670	2,789,000
10CA52	1,160	1,346,000	850	723,000	1,600	1,600	2,560,000

TABLE 48. CALCULATION OF THE SHAPE FACTOR (R)* FOR TORSIONAL VIBRATION

1	2	3	4	5	6	7	8	9
Specimen No.	$\frac{a}{b}$	$4\frac{a}{b}$	$(\frac{a}{b})^2$	$2.52(\frac{a}{b})^2$	$\frac{b}{a}$	$\frac{a}{b}+\frac{b}{a}$	$4\frac{a}{b}-2.52(\frac{a}{b})^2$	R
1C51	.705	2.820	.497	1.252	1.42	2.125	1.568	1.355
1C52	.694	2.776	.482	1.215	1.44	2.134	1.561	1.367
2CES51	.700	2.800	.490	1.235	1.43	2.130	1.565	1.361
2CES52	.651	2.604	.424	1.068	1.54	2.191	1.536	1.426
3C51	.658	2.632	.433	1.091	1.52	2.178	1.541	1.413
3C52	.672	2.688	.452	1.139	1.49	2.162	1.549	1.396
3SG51	.702	2.808	.493	1.242	1.42	2.122	1.566	1.355
3SG52	.661	2.644	.437	1.101	1.51	2.171	1.543	1.407
4SG51	.659	2.636	.434	1.094	1.52	2.179	1.542	1.413
4SG52	.672	2.688	.452	1.139	1.49	2.162	1.549	1.396
5CS51	.684	2.736	.468	1.179	1.46	2.144	1.557	1.377
5CS52	.700	2.800	.490	1.235	1.43	2.130	1.565	1.361
6ES51	.684	2.736	.468	1.179	1.46	2.144	1.557	1.377
6ES52	.681	2.724	.464	1.169	1.47	2.151	1.555	1.383
7W51	.710	2.840	.504	1.270	1.41	2.120	1.570	1.350
7W52	.645	2.580	.416	1.048	1.55	2.195	1.532	1.433
8CEL51	.664	2.656	.441	1.111	1.51	2.174	1.545	1.407
8CEL52	.626	2.504	.392	.988	1.60	2.226	1.516	1.468
9S51	.683	2.732	.466	1.174	1.46	2.143	1.558	1.375
9S52	.675	2.700	.456	1.149	1.48	2.155	1.551	1.389
10CA51	.664	2.656	.441	1.111	1.51	2.174	1.545	1.407
10CA52	.652	2.608	.425	1.071	1.53	2.182	1.537	1.420

* (R) is calculated by equation (14) following: (See Chapter VIII)

$$R = \frac{\frac{a}{b} + \frac{b}{a}}{4\frac{a}{b} - 2.52(\frac{a}{b})^2}$$

TABLE 49. CALCULATION OF THE COEFFICIENT (B) FOR TORSIONAL VIBRATION*

1	2	3	4	5	6	7
Specimen No.	R	L	4 L R	A = a x b	g x A	B
		in.	in.	in^2	$\frac{in^3}{sec^2}$	$\frac{sec^2}{in^2}$
1C51	1.355	15.66	84.88	9.29	3590	.0236
1C52	1.367	15.66	85.63	9.15	3536	.0242
2CES51	1.361	15.63	85.09	9.28	3586	.0237
2CES52	1.426	15.63	89.15	8.63	3335	.0267
3C51	1.413	15.76	89.08	8.68	3354	.0266
3C52	1.396	15.76	88.00	8.86	3424	.0257
3SG51	1.355	15.70	85.09	9.26	3578	.0238
3SG52	1.407	15.70	88.37	8.71	3366	.0263
4SG51	1.413	15.76	89.08	9.26	3578	.0249
4SG52	1.396	15.76	88.00	9.45	3651	.0241
5CS51	1.377	15.63	86.09	9.06	3501	.0246
5CS52	1.361	15.63	85.09	9.28	3586	.0237
6ES51	1.377	15.63	86.09	9.06	3501	.0246
6ES52	1.383	15.63	86.47	9.03	3489	.0248
7W51	1.350	15.63	84.42	9.37	3624	.0233
7W52	1.433	15.63	89.59	8.49	3281	.0273
8CEL51	1.407	15.75	88.64	9.34	3609	.0246
8CEL52	1.468	15.75	92.48	8.81	3404	.0272
9S51	1.375	15.63	85.97	9.00	3478	.0247
9S52	1.389	15.63	86.84	8.89	3435	.0253
10CA51	1.407	15.63	88.64	8.75	3381	.0262
10CA52	1.420	15.63	88.78	8.60	3323	.0267

* Calculated equation (13) follows:

$$B = \frac{4\ L\ R}{g\ A}, \quad \text{where } g = 386.4\ in/sec^2$$

TABLE 50. PRELIMINARY INVESTIGATION - STRESS-STRAIN DATA (STATIC) FOR 2CES5 (CINDERS AND EXPANDED SLAG)

Load	Stress	Cavities Filled with Duco Cement		Cavities Filled with Plaster of Paris	
		Gage Reading	Strain	Gage Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		6		4	
0	0	1470	0	1850	0
1000	109	1360	110	1725	125
2000	217	1255	215	1600	250
3000	326	1145	325	1475	375
4000	435	1030	440	1342	508
5000	543	920	550	1220	630
6000	652	790	680	1080	770
7000	761	655	815	935	915
8000	870	520	950	790	1060
		5		3	
9000	978	1350	1120	1600	1250

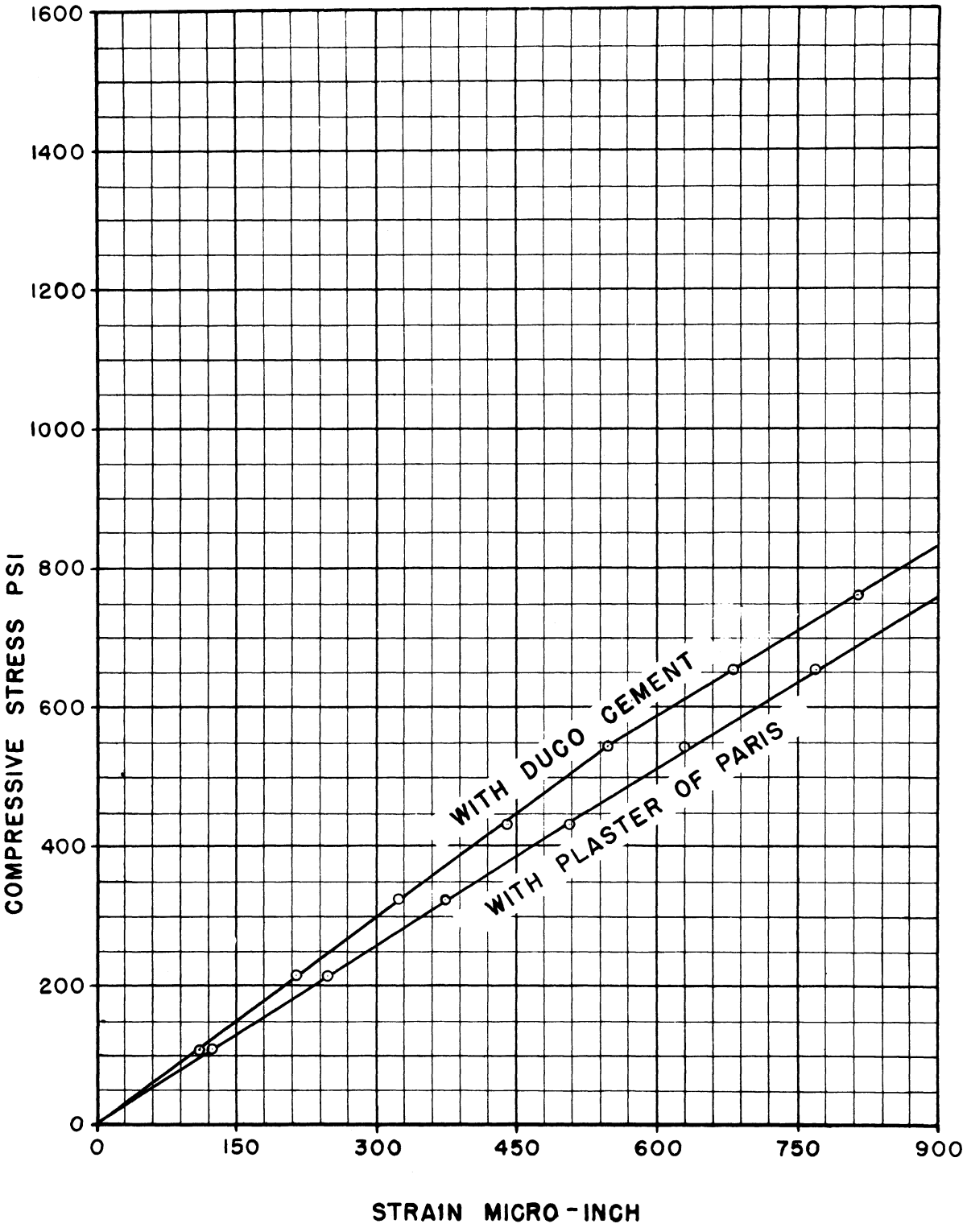


FIG. 48 - STRESS-STRAIN CURVES FOR 2CES5
PRELIMINARY INVESTIGATIONS

TABLE 51. PRELIMINARY INVESTIGATION - STRESS-STRAIN DATA (STATIC) FOR 3SG5 (SAND AND GRAVEL)

Load	Stress	Cavities Filled with Duco Cement		Cavities Filled with Plaster of Paris	
		Gage Reading	Strain	Gage Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
0	0	1630	0	1730	0
1000	117	1583	47	1680	50
2000	233	1535	95	1630	100
3000	350	1490	140	1581	149
4000	466	1440	190	1529	201
5000	583	1395	235	1576	248
6000	699	1350	280	1435	295
7000	816	1300	330	1387	343
8000	932	1250	380	1340	390
9000	1049	1200	430	1292	438
10000	1165	1150	480	1243	487
11000	1282	1098	533	1189	540
12000	1399	1045	585	1135	595
13000	1516	990	640	1078	653
14000	1632	935	695	1020	710

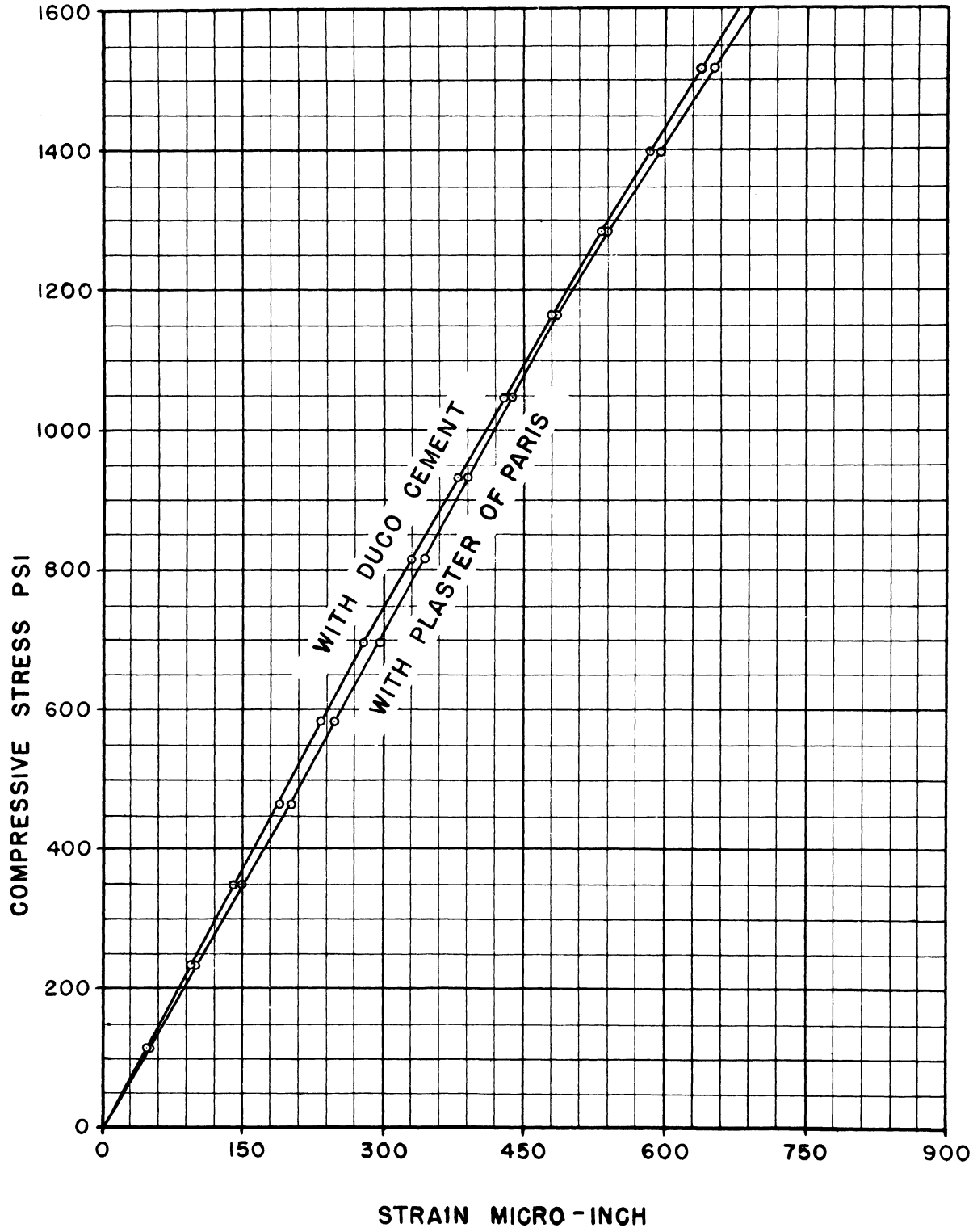


FIG. 49 - STRESS-STRAIN CURVES FOR 3SG5
PRELIMINARY INVESTIGATIONS

TABLE 52. PRELIMINARY INVESTIGATION - STRESS-STRAIN DATA (STATIC) FOR 9S5 (AIR-COOLED SLAG)

Load	Stress	Cavities Filled with Duco Cement		Cavities Filled with Plaster of Paris	
		Gage Reading	Strain	Gage Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
0	0	1040 [6]	0	1230 [6]	0
1000	118	990	50	1180	50
2000	236	945	95	1135	95
3000	354	900	140	1090	140
4000	472	850	190	1040	190
5000	590	805	235	993	237
6000	708	755	285	950	280
7000	826	705	335	898	332
8000	944	652	388	842	388
9000	1062	600	440	792	438
10000	1180	550	490	740	490
11000	1298	486	554	685	545
12000	1416	432	608	629	601
13000	1534	362	678	560	670
14000	1652	300	740	510	72

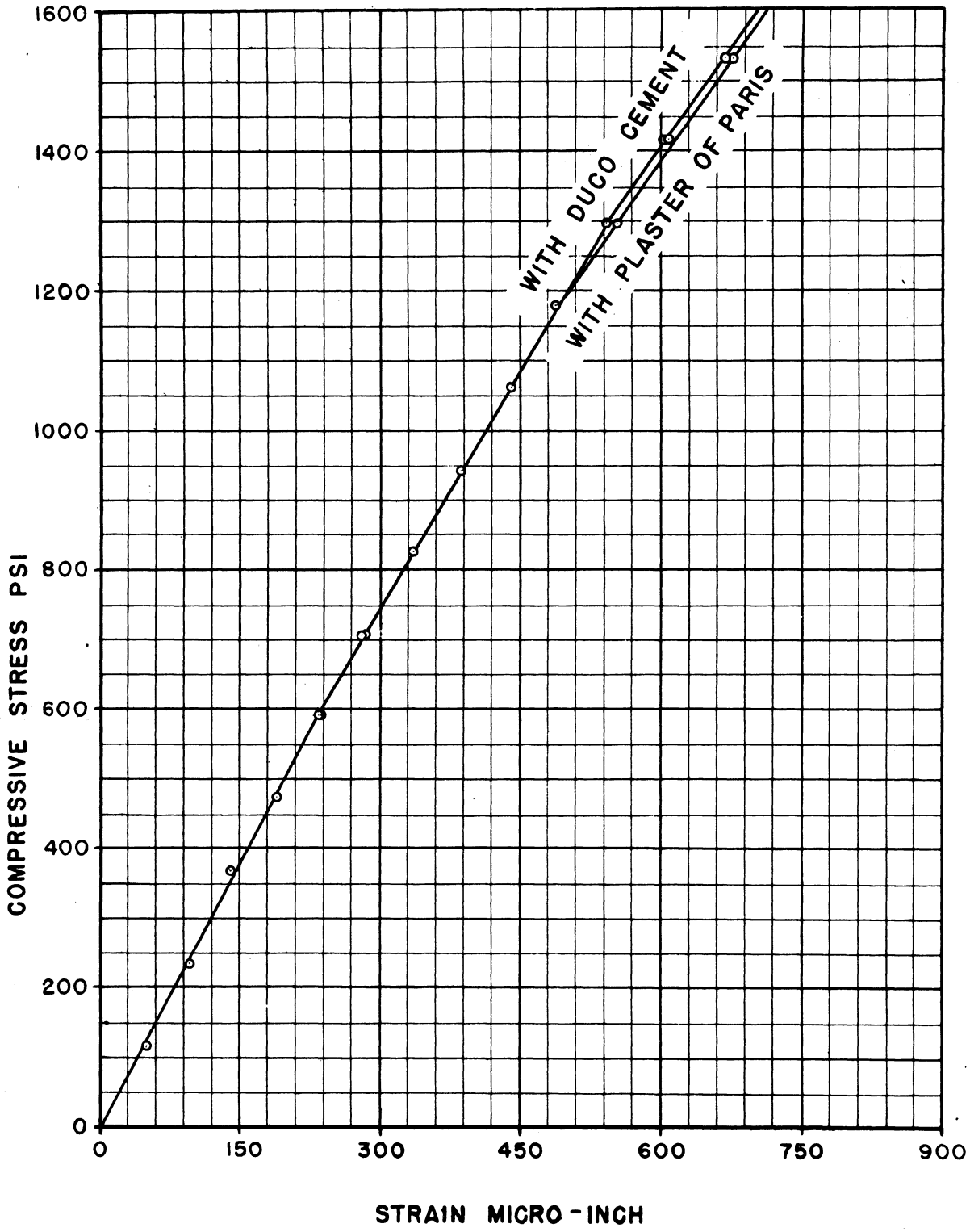


FIG. 50 — STRESS-STRAIN CURVES FOR 9S5
PRELIMINARY INVESTIGATIONS

TABLE 53. STRESS-STRAIN DATA (STATIC) FOR 1C51 (CINDERS)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		5		7	
0	0	1760	0	1280	0
1000	118	1650	110	1285	-5
2000	236	1530	230	1295	-15
3000	355	1410	350	1305	-25
4000	473	1290	470	1320	-40
5000	591	1165	595	1340	-60
6000	709	1030	730	1362	-82
7000	827	895	865	1382	-102
8000	945	760	1000	1403	-123

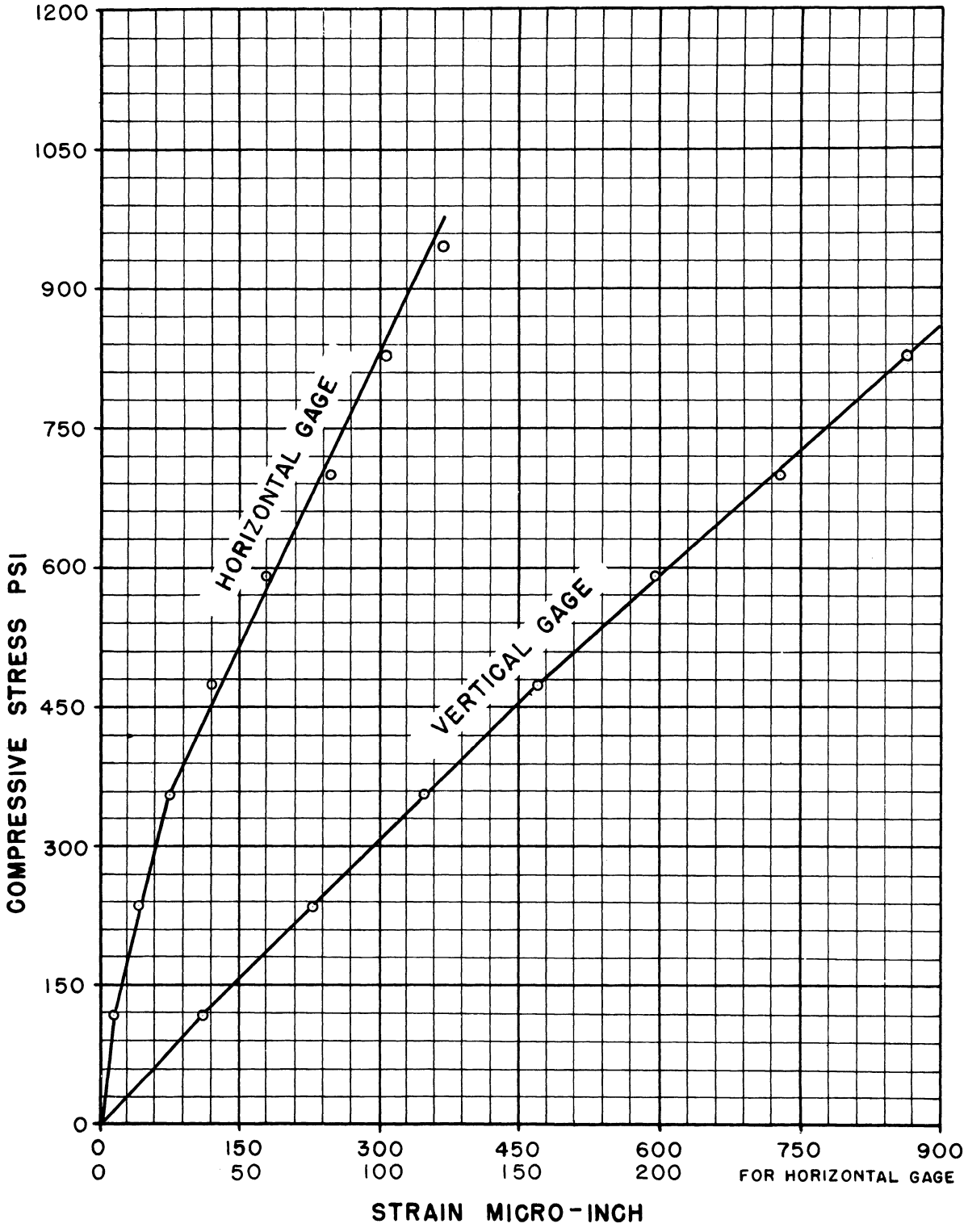


FIG. 51 - STRESS-STRAIN CURVES FOR 1C51

TABLE 54. STRESS-STRAIN DATA (STATIC) FOR 2CES51
(CINDERS AND EXPANDED SLAG)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		5		5	
0	0	1730	0	1040	0
1000	117	1620	110	1050	-10
2000	233	1510	220	1060	-20
3000	350	1400	330	1080	-40
4000	466	1280	450	1100	-60
5000	583	1170	560	1120	-80
6000	699	1050	680	1150	-110
7000	816	920	810	1170	-130
8000	933	790	940	1240	-160

TABLE 55. STRESS-STRAIN DATA (STATIC) FOR 2CES52
(CINDERS AND EXPANDED SLAG)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
		6	
0	0	1100	0
1000	123	990	110
2000	245	870	230
3000	368	750	350
4000	490	630	470
5000	613	500	600
6000	735	370	730
7000	858	230	860
8000	981	110	990

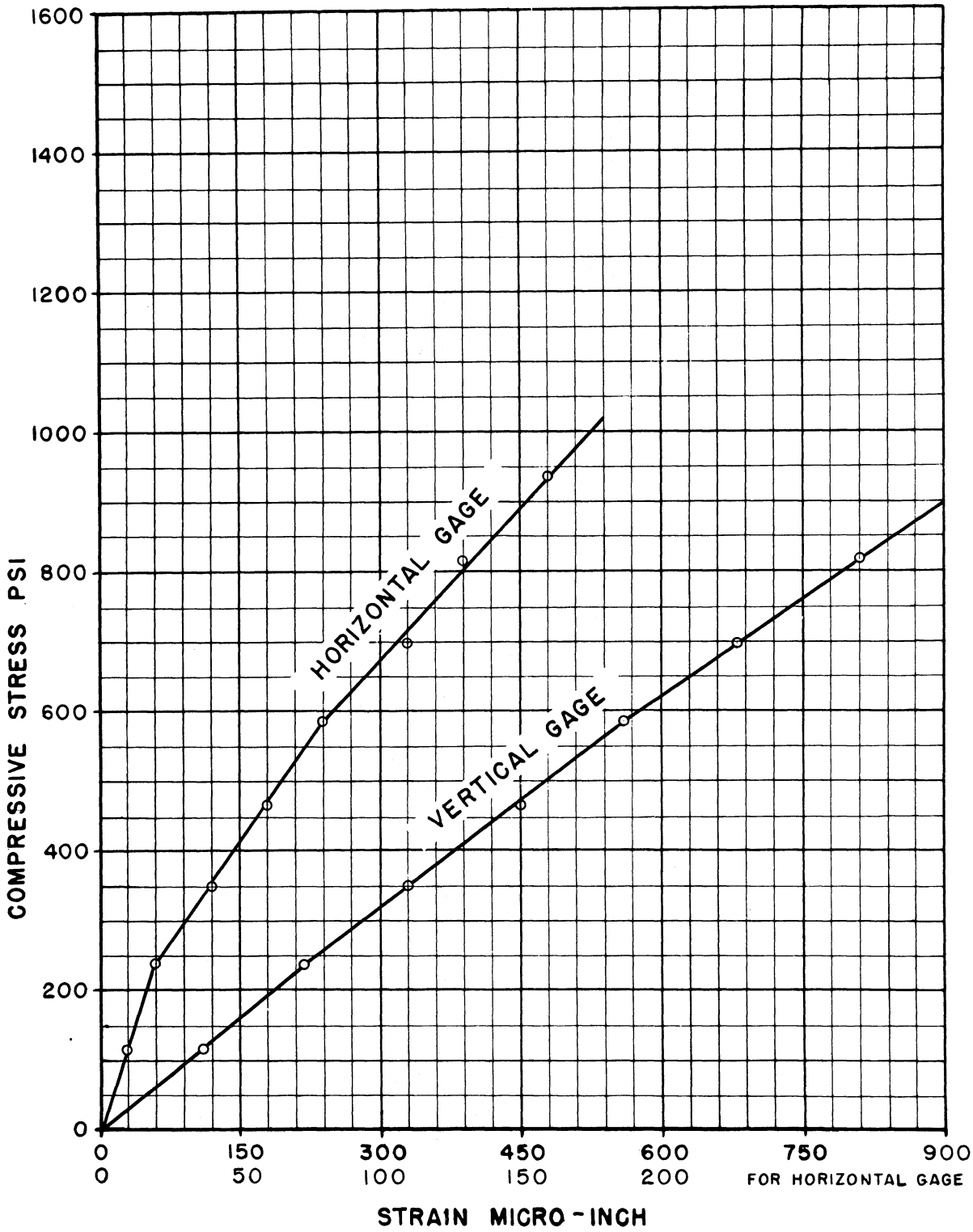


FIG. 52 – STRESS-STRAIN CURVES FOR 2CES51

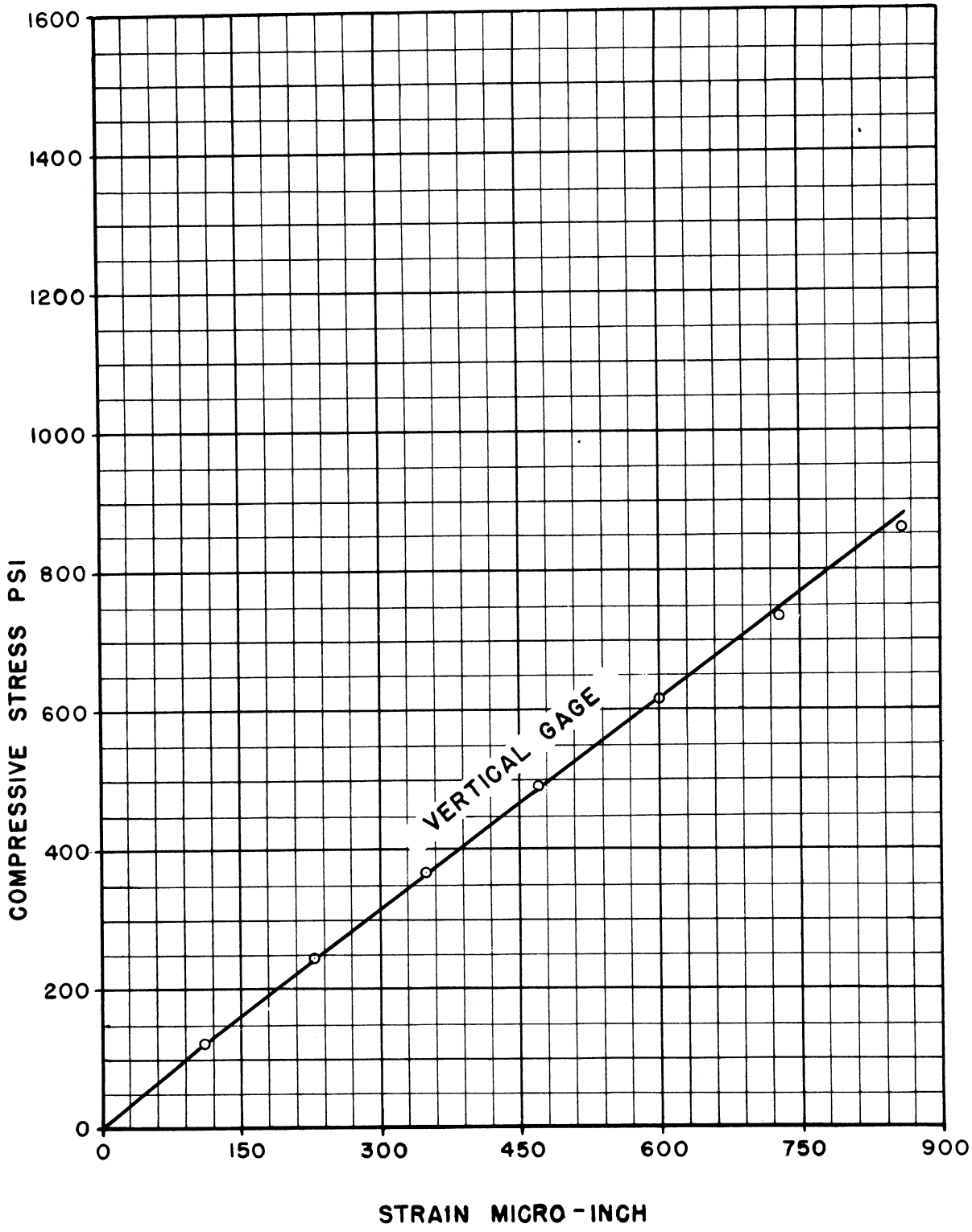


FIG. 53 — STRESS-STRAIN CURVE FOR 2CES52

TABLE 56. STRESS-STRAIN DATA (STATIC) FOR 3C51 (CINDERS)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		5		6	
0	0	1630	0	1480	0
1000	121	1475	155	1510	-30
2000	242	1330	300	1530	-50
3000	362	1180	450	1560	-80
4000	483	4 1015	615	1585	-105
5000	604	1850	780	1610	-130
6000	725	1685	945	655	-175
7000	845	1510	1120	1700	-220
8000	966	1330	1300	1750	-270

TABLE 57. STRESS-STRAIN DATA (STATIC) FOR 3C52 (CINDERS)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		6		6	
0	0	1420	0	1700	0
1000	121	1285	135	1710	-10
2000	242	1150	270	1730	-30
3000	362	5 1010	410	1750	-50
4000	483	1865	555	1780	-80
5000	604	1720	700	1810	-110
6000	725	1560	860	1840	-140
7000	846	1400	1020	1880	-180
8000	967	1240	1180	1920	-220

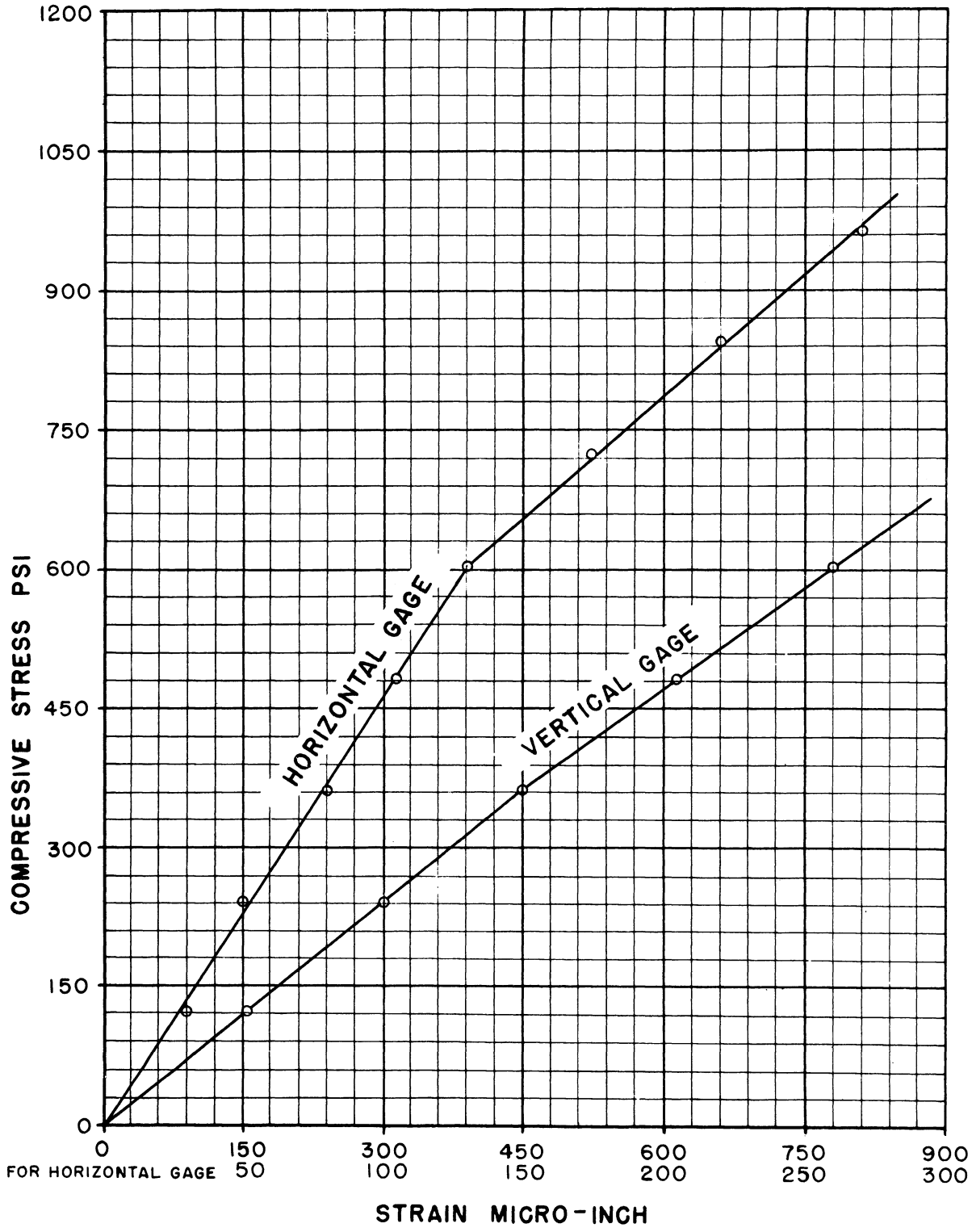


FIG. 54 - STRESS-STRAIN CURVES FOR 3C51

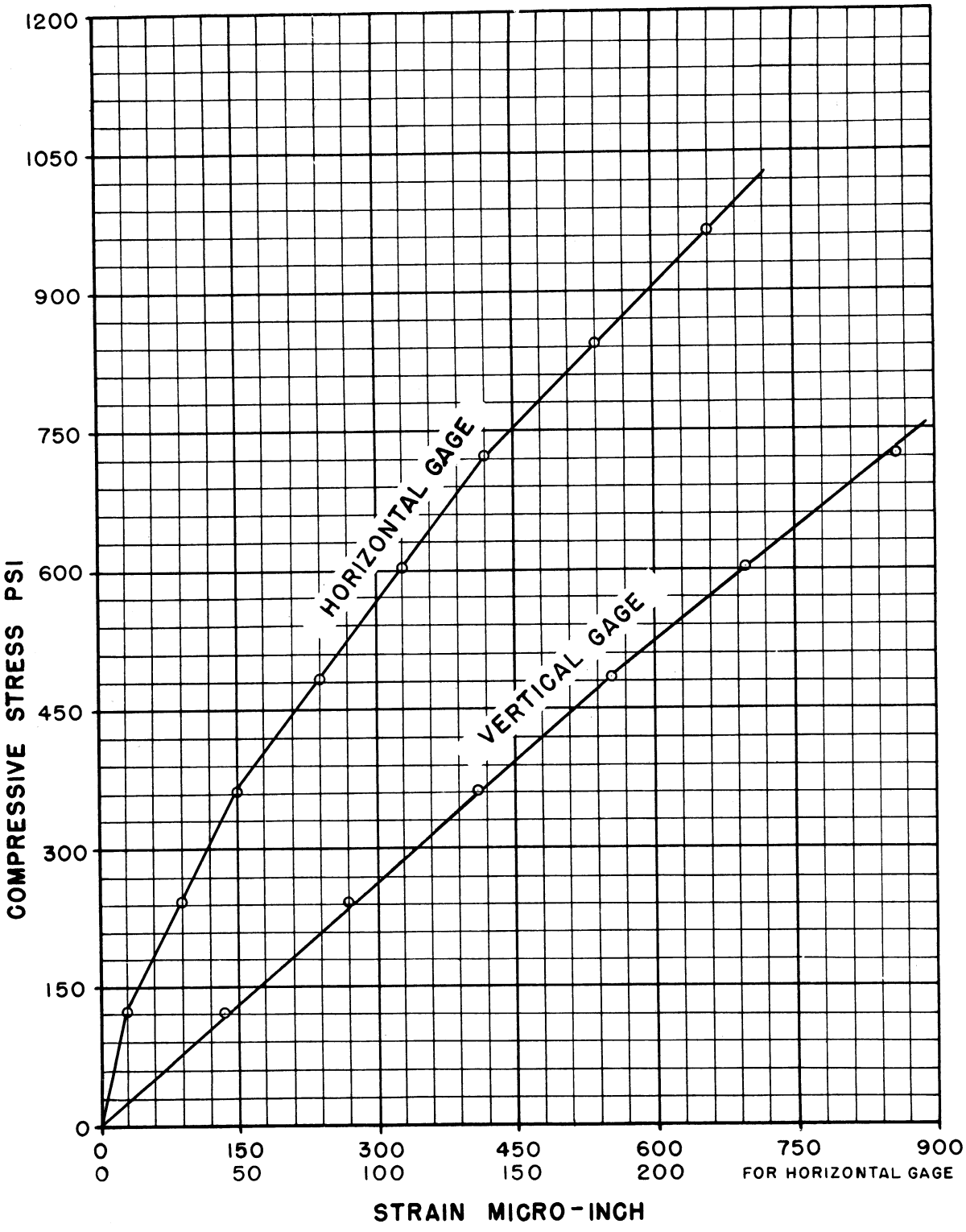


FIG. 55 - STRESS-STRAIN CURVES FOR 3C52

TABLE 58. STRESS-STRAIN DATA (STATIC) FOR 3SG51 (SAND AND GRAVEL)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
0	0	1140	0
1000	115	1095	45
2000	230	1050	90
3000	345	995	145
4000	460	950	190
5000	576	900	240
6000	691	850	290
7000	806	800	340
8000	921	750	390
9000	1036	695	445
10000	1151	640	500
11000	1266	585	555
12000	1381	525	615
13000	1496	468	672
14000	1612	410	730

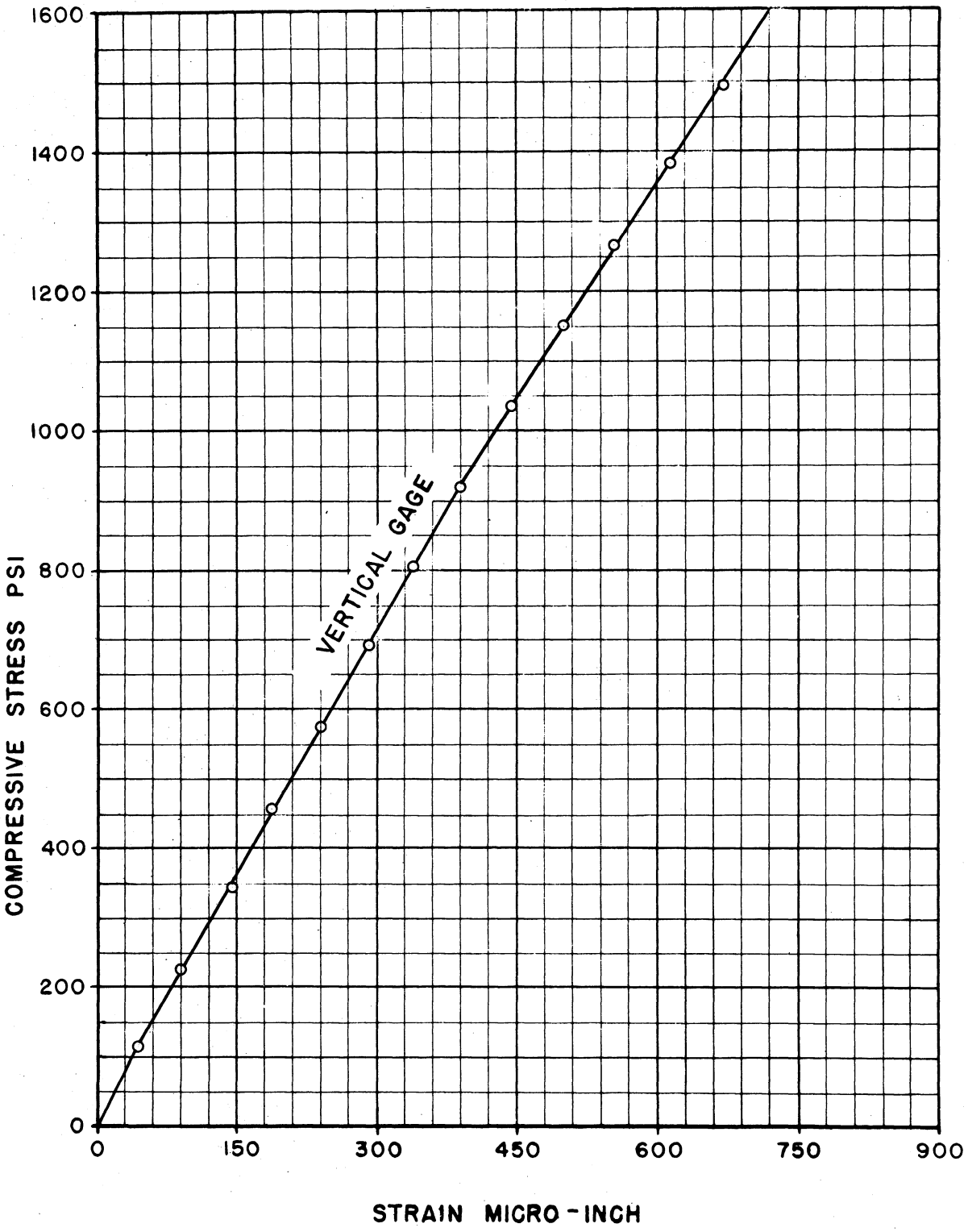


FIG. 56 - STRESS-STRAIN CURVE FOR 3SG51

TABLE 59. STRESS-STRAIN DATA (STATIC) FOR 3SG52 (SAND AND GRAVEL)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
0	0	1191	0
1000	123	1140	51
2000	246	1085	106
3000	369	1030	161
4000	493	970	221
5000	616	910	281
6000	739	855	336
7000	862	792	399
8000	985	730	461
9000	1108	667	524
10000	1232	600	591
11000	1355	535	656
12000	1479	478	713
13000	1602	410	781
14000	1724	343	848

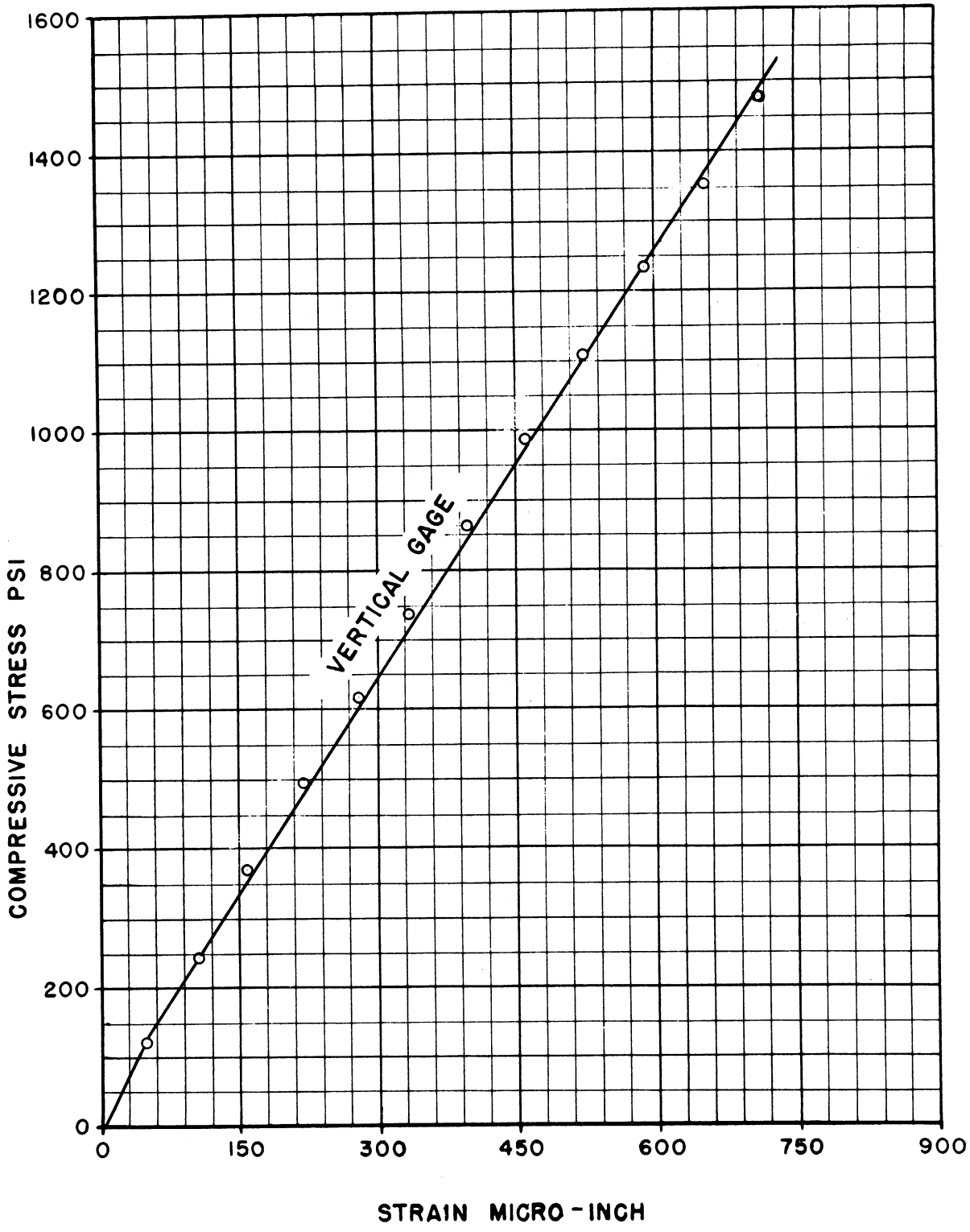


FIG. 57 — STRESS-STRAIN CURVE FOR 3SG52

TABLE 60. STRESS-STRAIN DATA (STATIC) FOR 4SG51 (SAND AND GRAVEL)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		6		6	
0	0	1670	0	1480	0
1000	118	1630	40	1485	-5
2000	235	1595	75	1490	-10
3000	353	1560	110	1500	-20
4000	470	1530	140	1510	-30
5000	588	1495	175	1515	-35
6000	705	1460	210	1523	-43
7000	823	1420	250	1532	-52
8000	940	1388	282	1540	-60
9000	1058	1354	316	1552	-72
10000	1175	1380	350	1565	-85
11000	1293	1285	380	1579	-99
12000	1410	1250	420	1590	-110
13000	1528	1215	455	1620	-140
14000	1645	1185	485	1650	-170

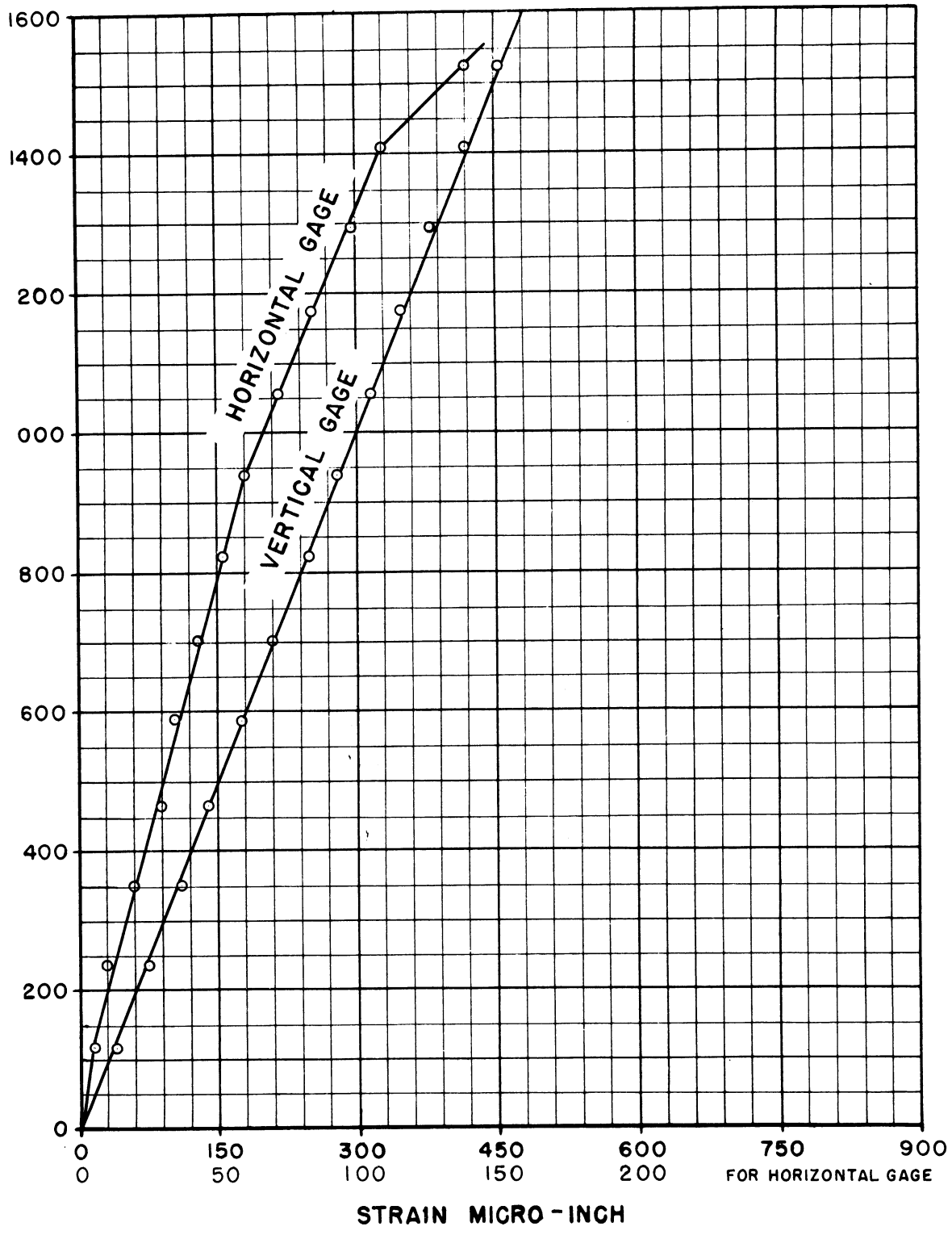


FIG. 58 - STRESS-STRAIN CURVES FOR 4SG51

TABLE 61. STRESS-STRAIN DATA (STATIC) FOR 4SG52 (SAND AND GRAVEL)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in/6	mic-in
0	0	1537	0
1000	118	1500	37
2000	235	1460	77
3000	353	1420	117
4000	470	1380	157
5000	588	1335	202
6000	705	1290	247
7000	823	1248	290
8000	940	1205	332
9000	1058	1160	377
10000	1175	1115	422
11000	1293	1070	467
12000	1410	1025	512
13000	1528	975	557
14000	1645	930	607

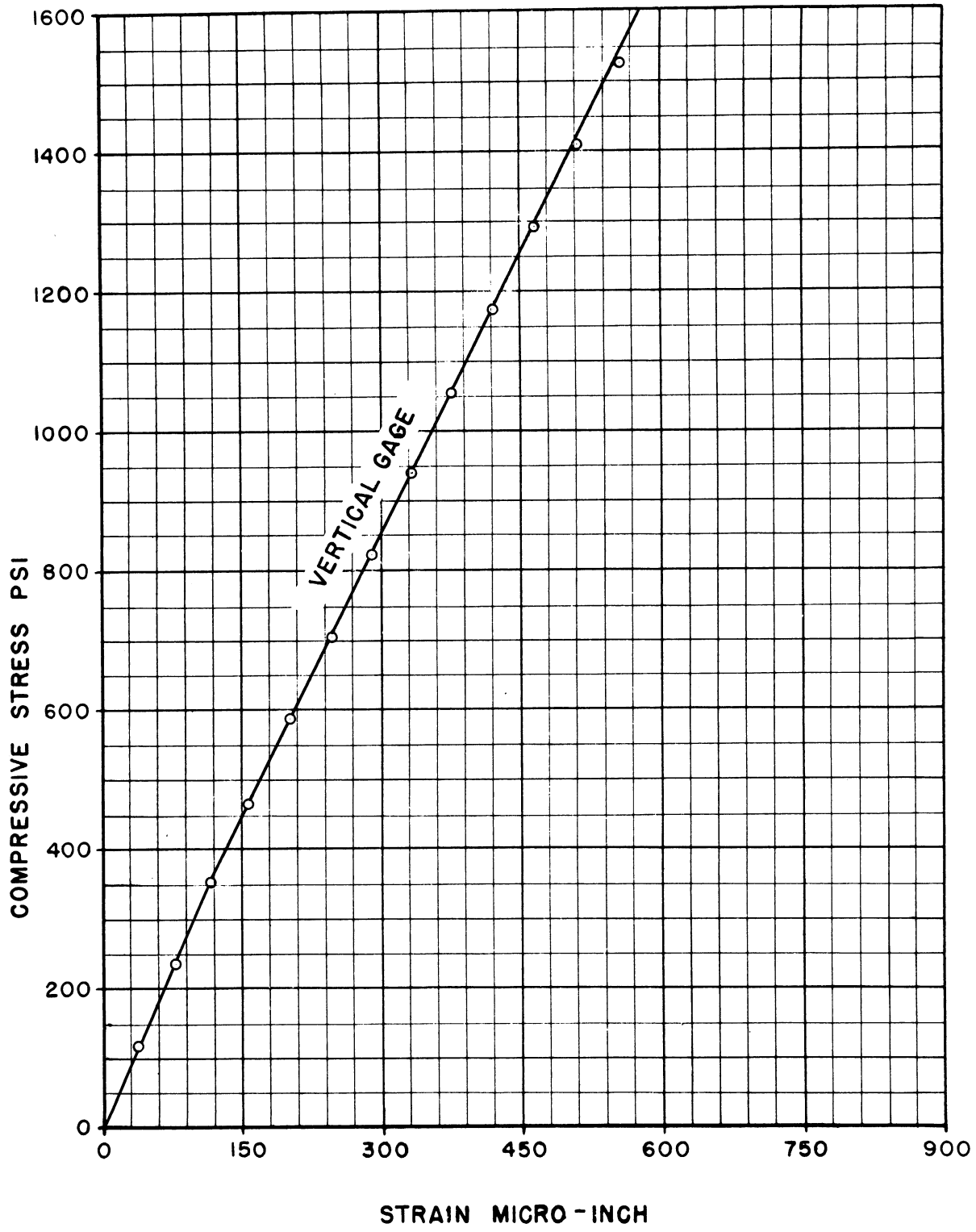


FIG. 59 — STRESS-STRAIN CURVE FOR 4SG52

TABLE 62. STRESS-STRAIN DATA (STATIC) FOR 5CS51 (CINDERS AND SAND)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		5			
0	0	1605	0	1120	0
1000	118	1505	100	1130	-10
2000	235	1400	205	1145	-25
3000	353	1290	315	1162	-42
4000	470	1180	425	1180	-60
5000	588	1070	535	1200	-80
6000	706	960	645	1220	-100
7000	824	850	755	1240	-120
8000	942	740	865	1260	-140

TABLE 63. STRESS-STRAIN DATA (STATIC) FOR 5CS52 (CINDERS AND SAND)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
		4	
0	0	1845	0
1000	116	1780	65
2000	231	1710	135
3000	347	1630	215
4000	463	1560	285
5000	578	1485	360
6000	694	1410	435
7000	809	1330	515
8000	925	1250	595

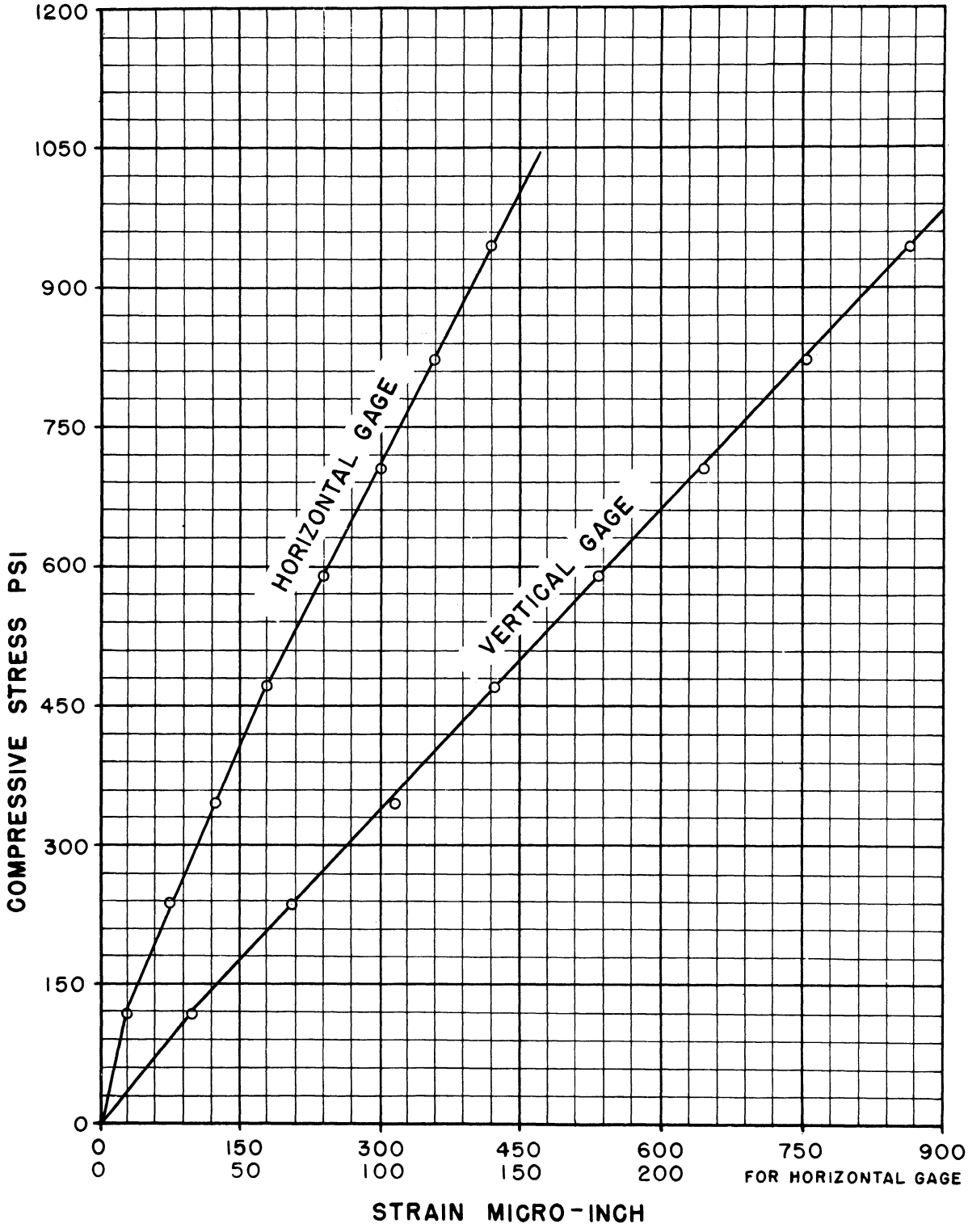


FIG. 60 - STRESS-STRAIN CURVES FOR 5CS51

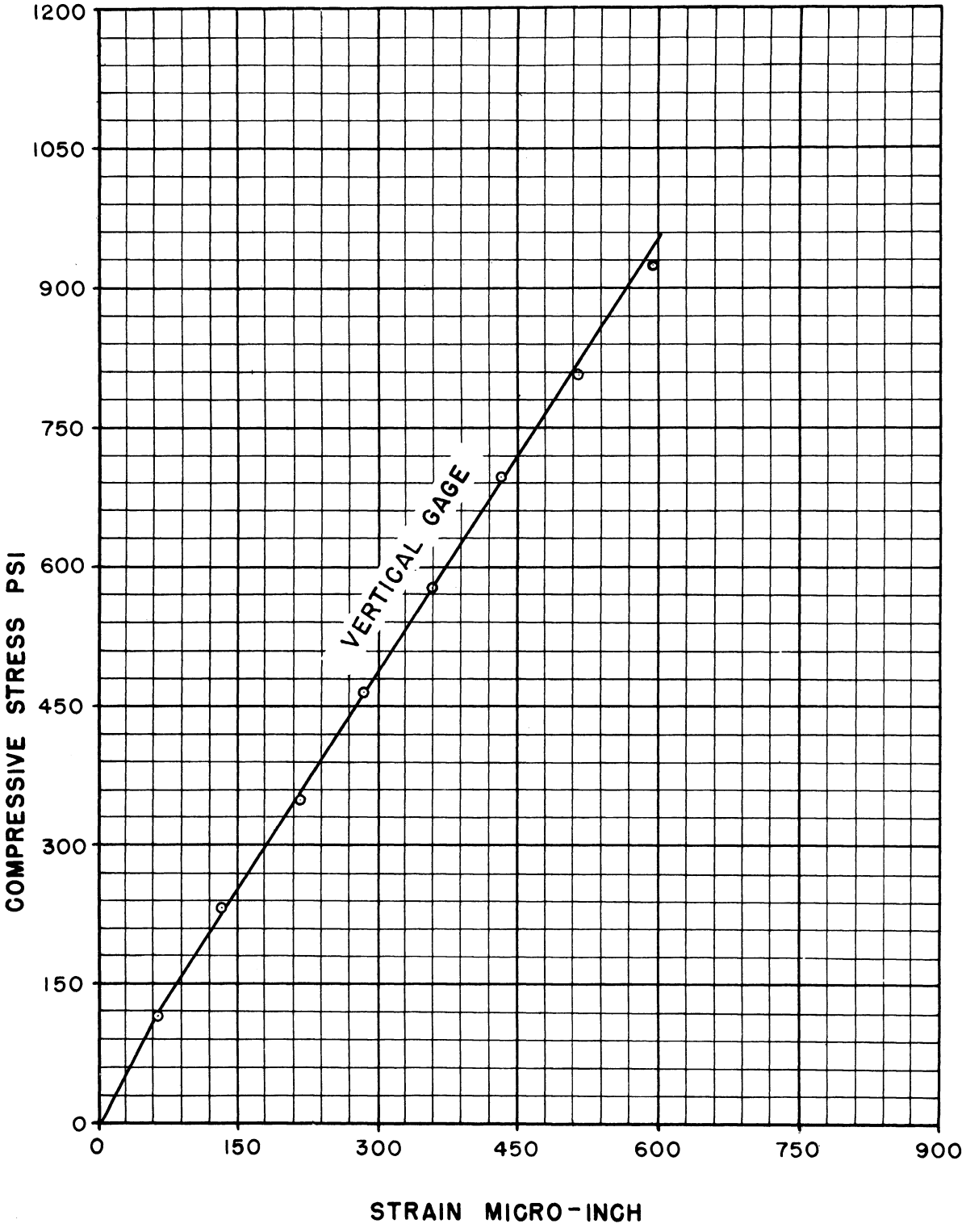


FIG. 61 – STRESS-STRAIN CURVE FOR 5CS52

TABLE 64. STRESS-STRAIN DATA (STATIC) FOR 6ES51 (EXPANDED SLAG)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		6		7	
0	0	1190	0	1115	0
1000	128	1100	90	1120	-5
2000	256	1020	170	1125	-10
3000	384	940	250	1135	-20
4000	512	865	325	1145	-30
5000	640	790	400	1155	-45
6000	768	720	470	1170	-55
7000	896	640	550	1185	-70
8000	1024	560	630	1195	-80

TABLE 65. STRESS-STRAIN DATA (STATIC) FOR 6ES52 (EXPANDED SLAG)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
		5	
0	0	1780	0
1000	126	1710	70
2000	253	1635	145
3000	379	1560	220
4000	505	1480	300
5000	632	1405	375
6000	758	1330	450
7000	884	1250	530
8000	1010	1170	610

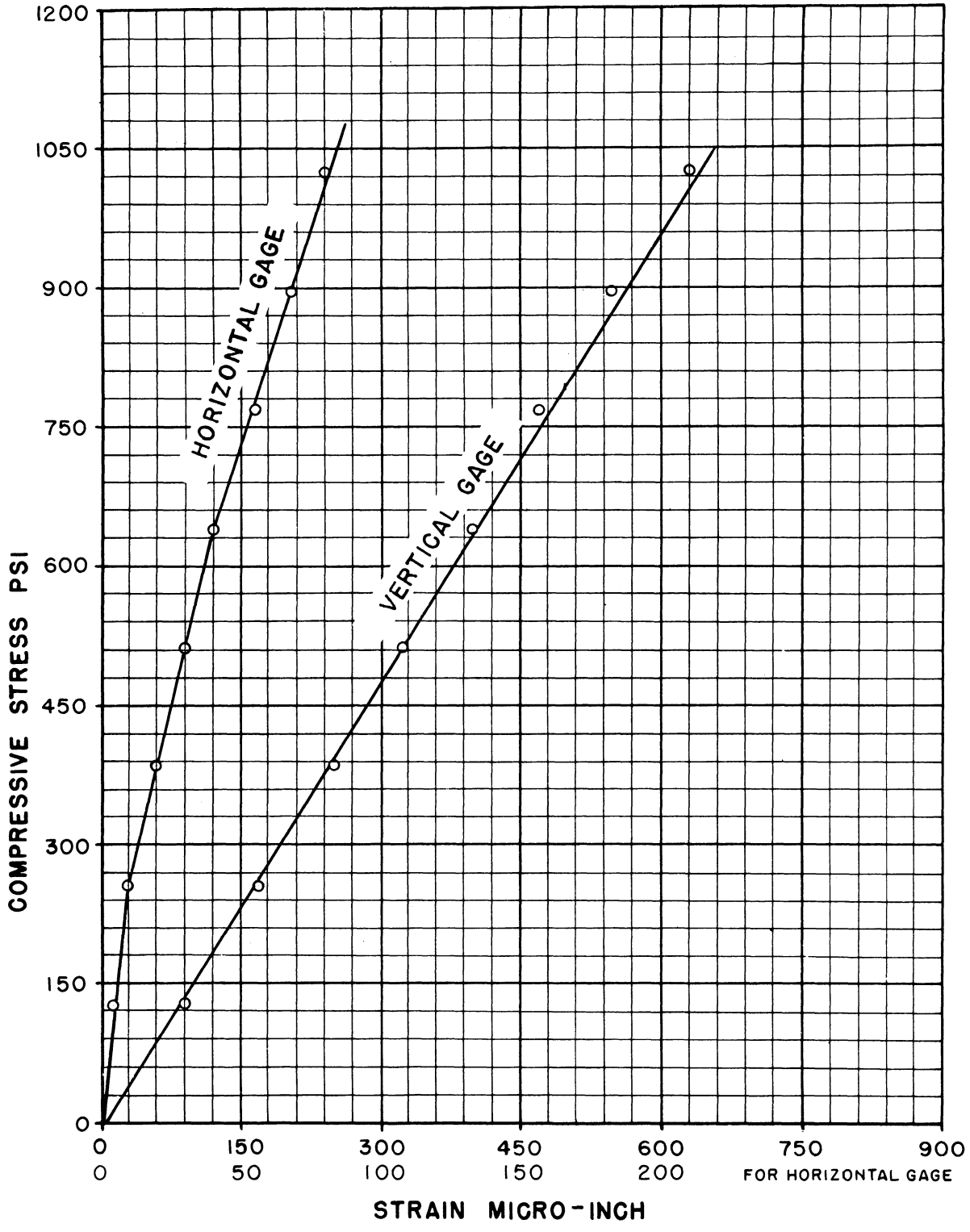


FIG. 62 – STRESS-STRAIN CURVES FOR 6ES51

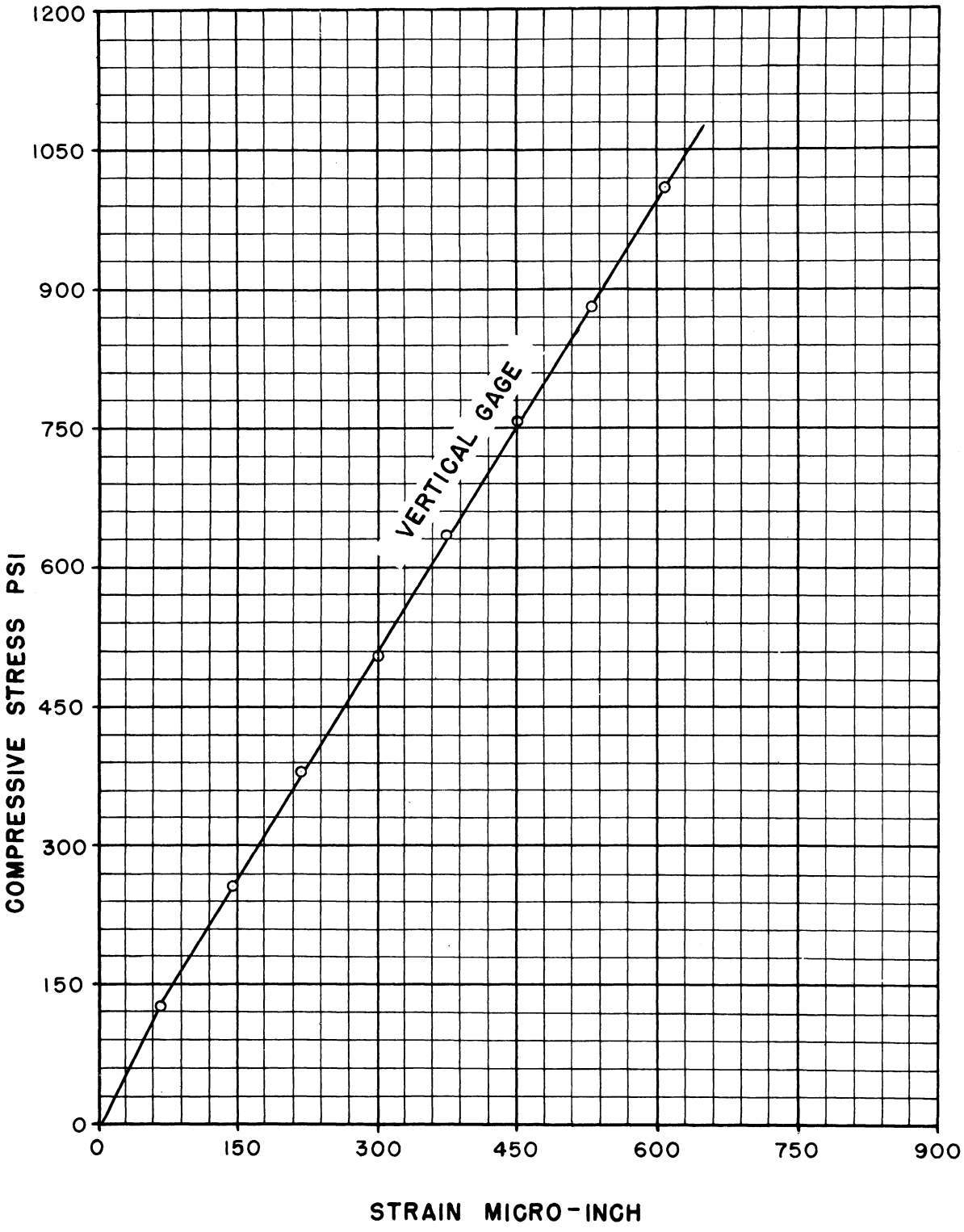


FIG. 63 – STRESS-STRAIN CURVE FOR 6ES52

TABLE 66. STRESS-STRAIN DATA (STATIC) FOR 7W51 (WAYLITE)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		6		7	
0	0	1100	0	950	0
1000	121	1040	60	960	-10
2000	242	980	120	965	-15
3000	363	930	170	975	-25
4000	483	885	215	980	-30
5000	604	820	280	990	-40
6000	725	770	330	1000	-50
7000	845	710	390	1010	-60
8000	966	660	440	1025	-75

TABLE 67. STRESS-STRAIN DATA (STATIC) FOR 7W52 (WAYLITE)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
		6	
0	0	1430	0
1000	126	1370	60
2000	253	1305	125
3000	379	1240	190
4000	505	1180	250
5000	632	1120	310
6000	758	1060	370
7000	884	1000	430
8000	1010	940	490

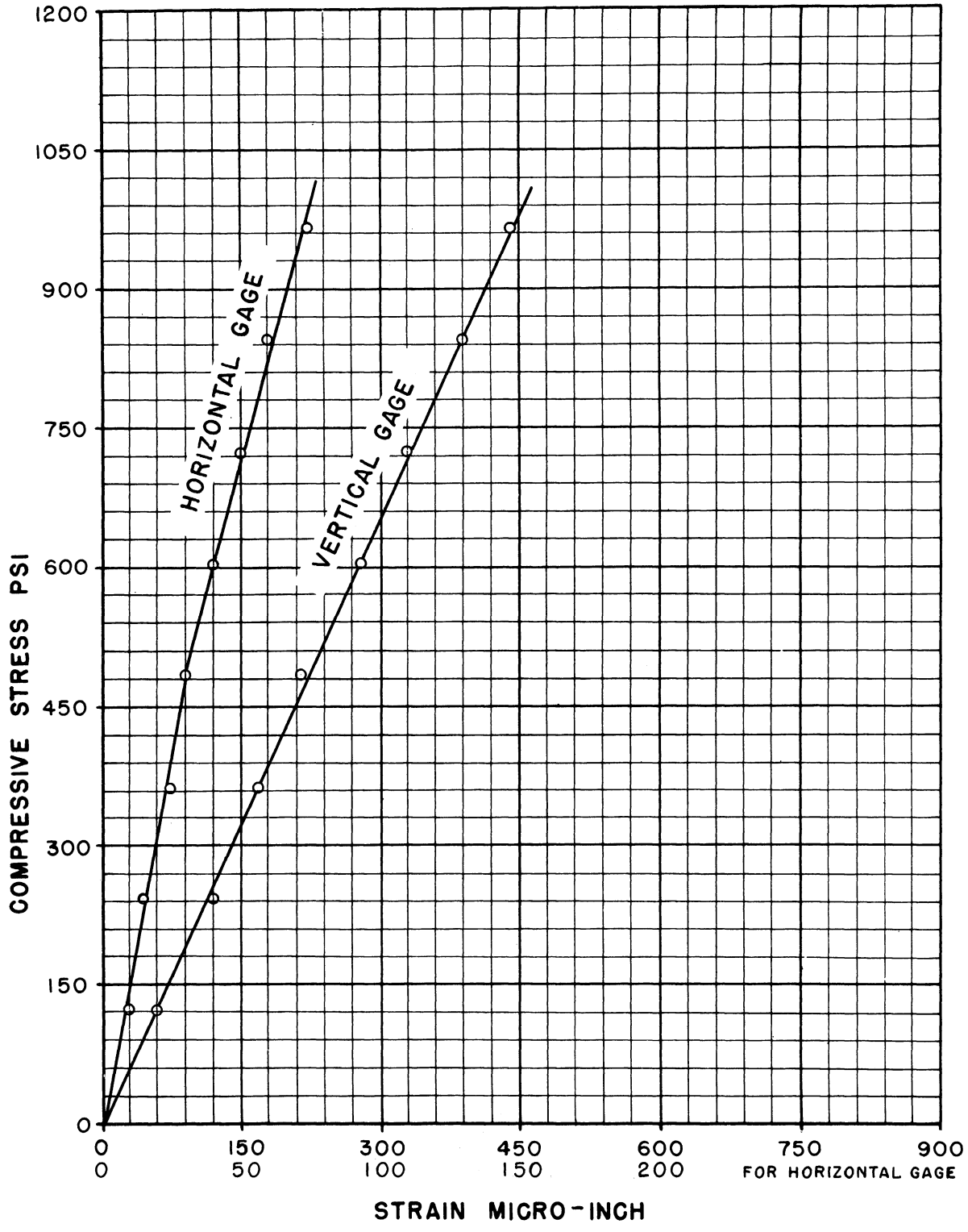


FIG. 64 - STRESS-STRAIN CURVES FOR 7W51

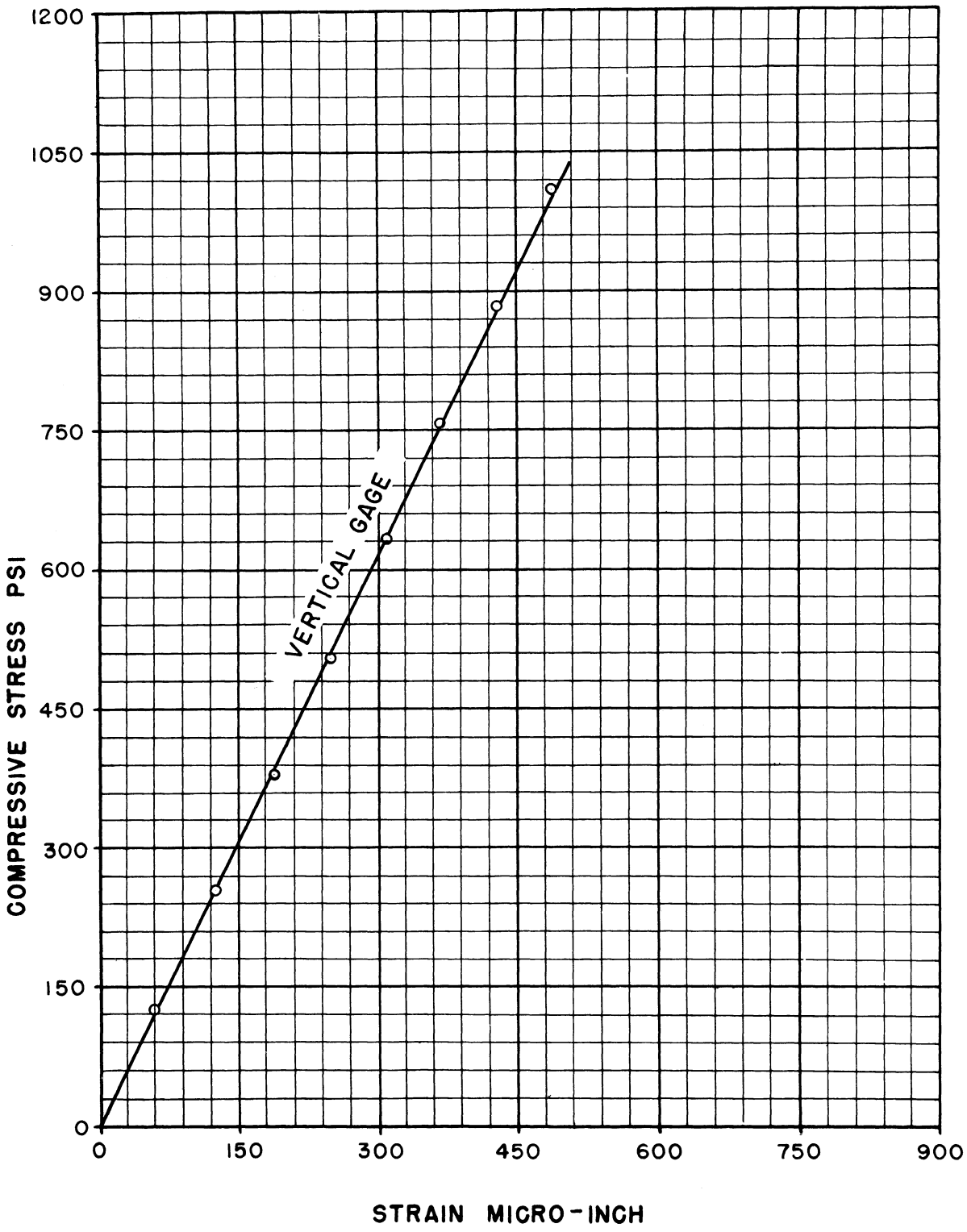


FIG. 65 – STRESS-STRAIN CURVE FOR 7W52

TABLE 68. STRESS-STRAIN DATA (STATIC) FOR 8CEL51 (CELOCRETE)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
		7	
0	0	1130	0
1000	125	1000	130
2000	249	890	240
3000	374	790	340
4000	498	700	430
5000	623	610	520
6000	749	520	610
7000	874	430	700
8000	999	340	790

TABLE 69. STRESS-STRAIN DATA (STATIC) FOR 8CEL52 (CELOCRETE)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
		5	
0	0	1130	0
1000	122	1020	110
2000	244	900	230
3000	367	800	330
4000	489	690	440
5000	611	4 580	550
6000	733	1470	660
7000	855	1370	760
8000	978	1250	880

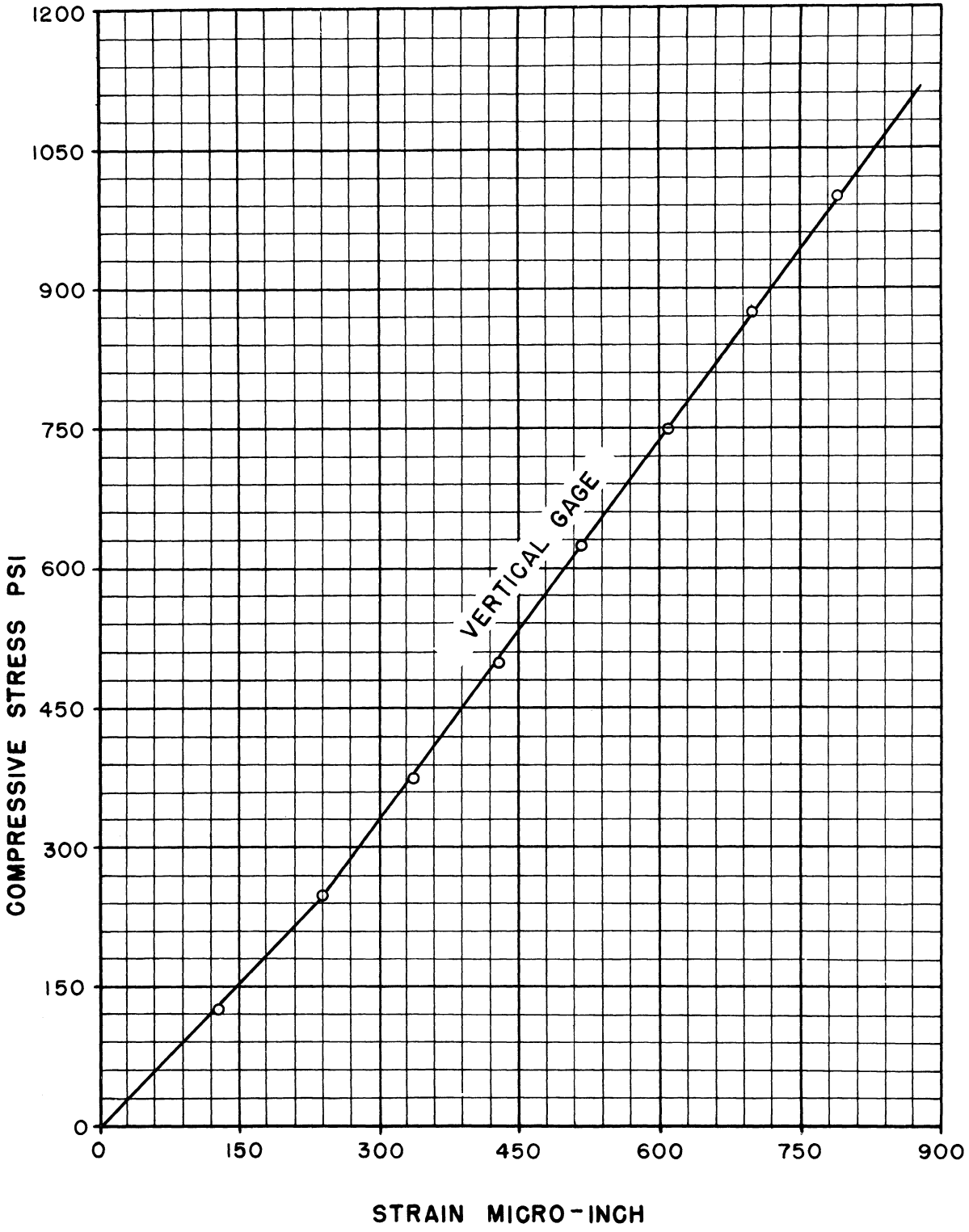


FIG. 66 – STRESS-STRAIN CURVE FOR 8CEL51

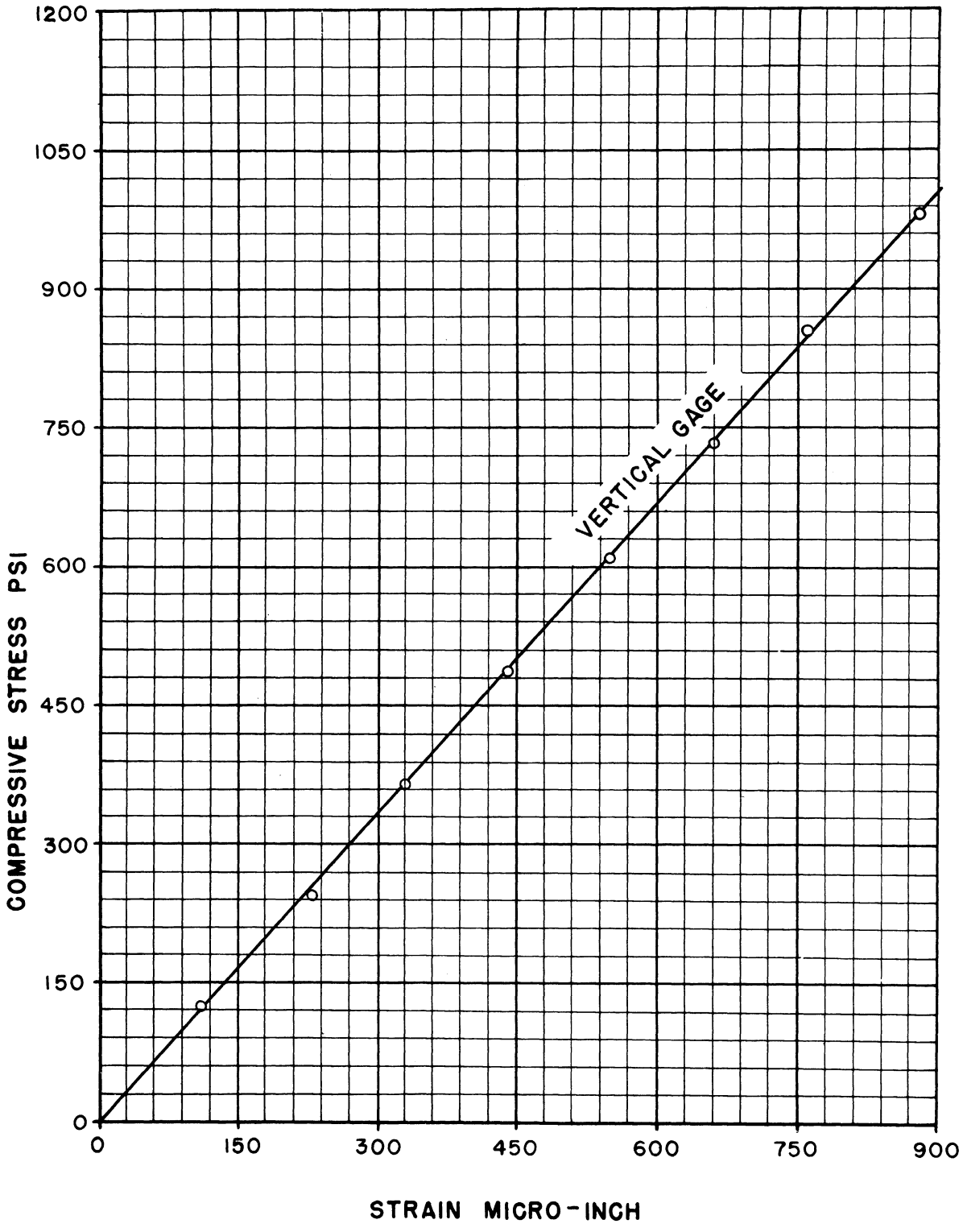


FIG. 67 STRESS-STRAIN CURVE FOR 8CEL52

TABLE 70. STRESS-STRAIN DATA (STATIC) FOR 9S51 (AIR-COOLED SLAG)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		2		5	
0	0	1890	0	1230	0
1000	121	1830	60	1235	-5
2000	242	1775	115	1240	-10
3000	362	1725	165	1244	-14
4000	483	1670	220	1250	-20
5000	604	1615	275	1260	-30
6000	725	1560	330	1265	-35
7000	845	1500	390	1270	-40
8000	966	1440	450	1280	-50
9000	1087	1375	515	1290	-60
10000	1208	1310	580	1300	-70
11000	1329	1240	650	1315	-85
12000	1449	1170	720	1330	-100
13000	1570	1100	790	1350	-120
14000	1691	1020	870	1370	-140

TABLE 71. STRESS-STRAIN DATA (STATIC) FOR 9S52 (AIR-COOLED SLAG)

Load Pounds	Stress psi	Gage Reading mic-in $\frac{6}{}$	Strain mic-in
0	0	1040	0
1000	118	990	50
2000	236	945	95
3000	354	900	140
4000	472	850	190
5000	590	805	235
6000	708	755	285
7000	826	705	335
8000	944	652	388
9000	1062	600	440
10000	1180	550	490
11000	1298	486	554
12000	1416	432	608
13000	1534	362	678
14000	1652	300	740

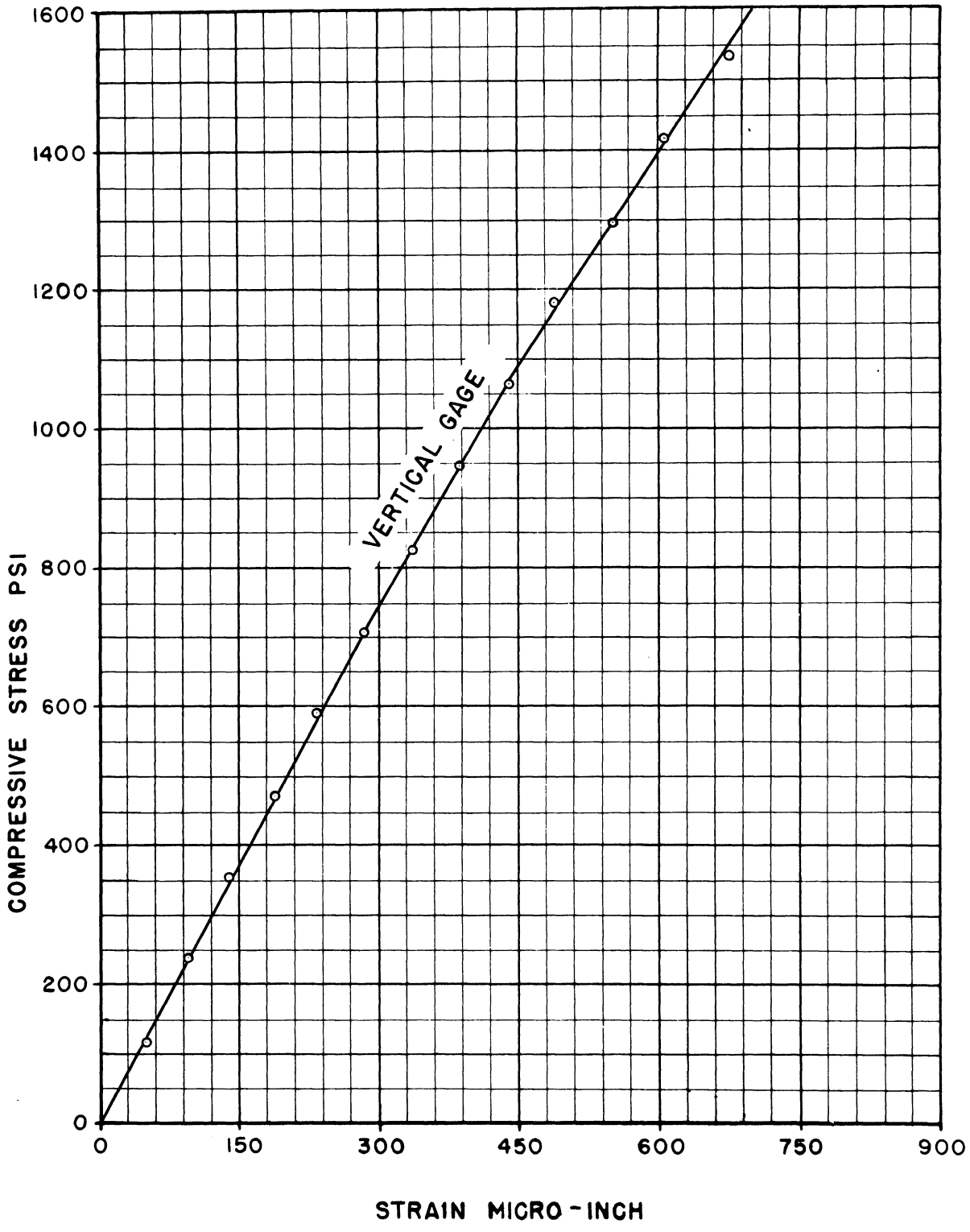


FIG. 69 - STRESS-STRAIN CURVE FOR 9S52

TABLE 72. STRESS-STRAIN DATA (STATIC) FOR 10CA51
(CINDERS PLUS FLY ASH)

Load	Stress	Vertical Gage		Horizontal Gage	
		Reading	Strain	Reading	Strain
pounds	psi	mic-in	mic-in	mic-in	mic-in
		6		5	
0	0	1280	0	545	0
1000	126	1150	130	550	-5
2000	253	1000	280	570	-25
3000	379	840	440	590	-45
4000	505	675	605	610	-65
5000	631	495	785	655	-110
6000	758	5 1310	970	700	-155
7000	884	1100	1180	755	-210
8000	1010	840	1440	840	-295

TABLE 73. STRESS-STRAIN DATA (STATIC) FOR 10CA52
(CINDERS PLUS FLY ASH)

Load	Stress	Vertical Gage	
		Reading	Strain
pounds	psi	mic-in	mic-in
		4	
0	0	1370	0
1000	126	1245	125
2000	253	1125	245
3000	379	990	380
4000	505	860	510
5000	632	720	650
6000	758	570	800
7000	884	400	970
8000	1010	3 1220	1150

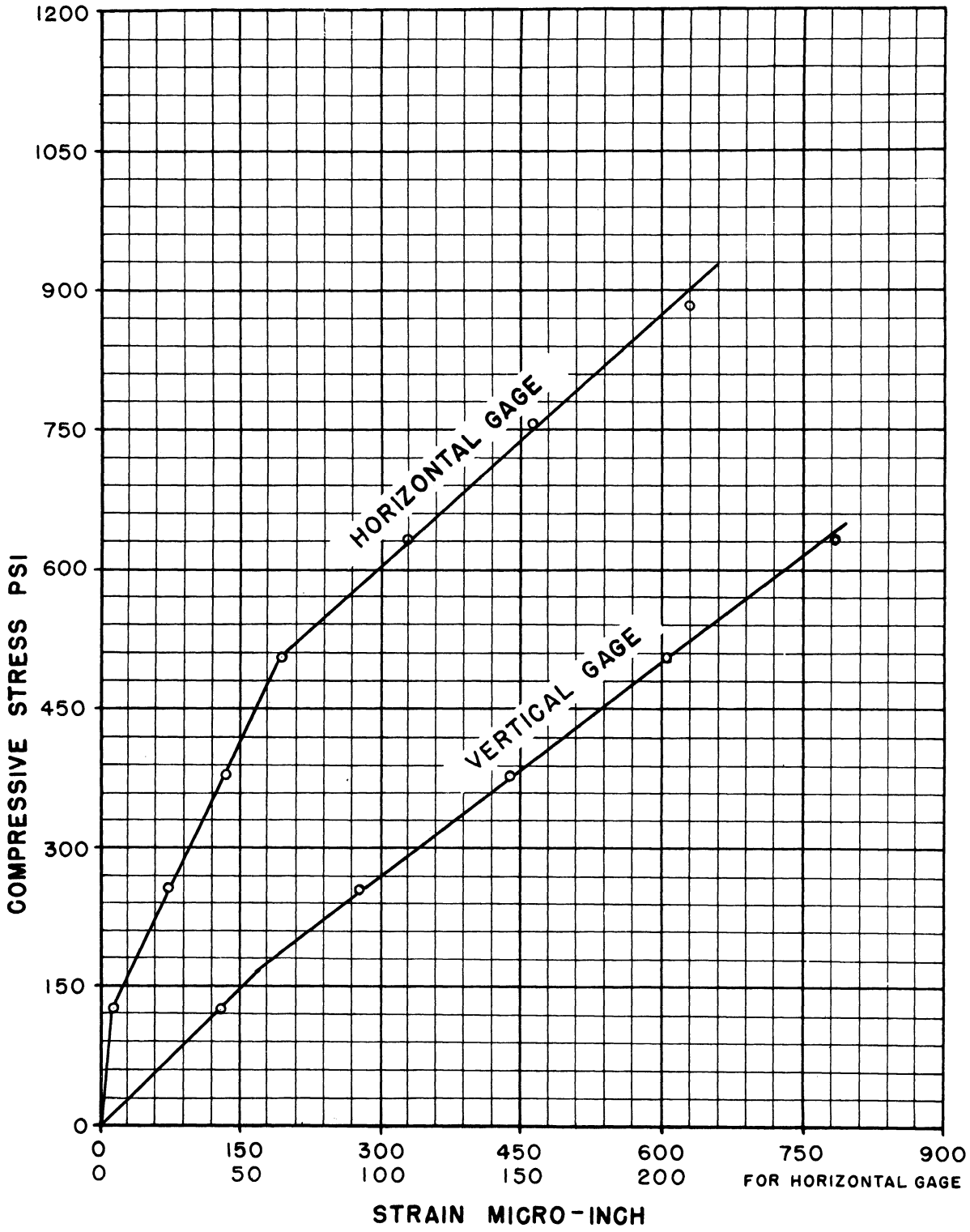


FIG. 70 – STRESS-STRAIN CURVES FOR 10 CA 51

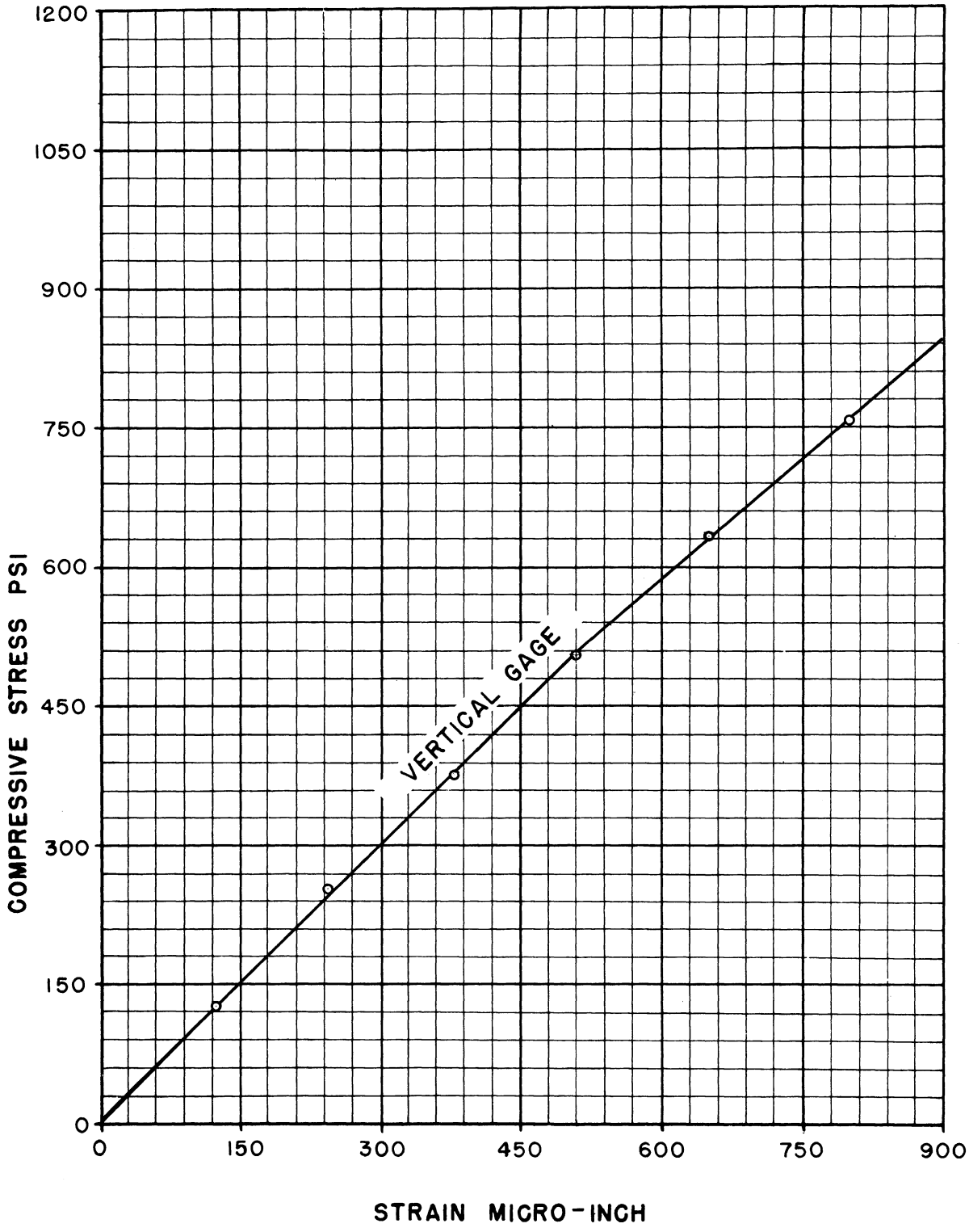


FIG. 71 – STRESS-STRAIN CURVE FOR 10CA52

APPENDIX VII

THERMAL EXPANSION

As was previously discussed in Chapter XII, the coefficients of thermal expansion were determined for saturated (not vacuum saturated) and oven dried specimens. Two methods of tests were used for measuring length change. These were the dial gage extensometer and the variable resistance wire gage (SR-4) which were both described in Chapter XII. Both methods of test were used for the dry specimen but the first method only was used for the wet specimen.

Table 74 gives the coefficients of the thermal expansion of saturated concrete for the various stages of heating and cooling. Each value represents the average of the 5 cycles of the test; their average is considered the thermal coefficient of the specimen. It may be seen from the table that the coefficients when heating are slightly higher than those when cooling. The actual extensometer readings are not given.

Table 75 gives the coefficient of thermal expansion for the dry specimens by the first method of test. The discussion given above holds for this table except that the cycle in this latter test consists of one stage of heating (100 to 162 F) and one stage of cooling (162 to 100 F).

Table 76 gives the coefficients of expansion of dried concrete by the second method of test. Results were calculated from equation (16a) which was presented in Chapter XII. The values ΔR_e

and ΔR_m in the table are the average of 5 complete heat-cool cycles. Again the actual readings of the strain indicator were not given.

TABLE 74. CALCULATION OF THE COEFFICIENT OF THERMAL EXPANSION OF WATER SATURATED SPECIMEN
(inch/inch/1 F) x 10^6

Specimen	Heating 35-73° F	Heating 73-110° F	Heating 110-140° F	Cooling 140-73° F	Cooling 73-35° F	Average Individual	Average Both Specimens
1C61	3.4	3.4	3.5	3.3	3.4	3.4	3.4
1C62	3.5	3.5	3.6	3.3	3.4	3.5	
2CES61	4.2	4.3	4.3	4.0	4.1	4.2	4.3
2CES62	4.4	4.4	4.6	4.3	4.3	4.4	
3C61	3.3	3.4	3.4	3.2	3.2	3.3	3.4
3C62	3.5	3.5	3.5	3.4	3.5	3.5	
3SG61	4.7	4.7	4.8	4.6	4.7	4.7	4.9
3SG62	5.0	5.0	5.2	4.9	4.9	5.0	
4SG61	5.0	5.1	5.1	4.9	5.0	5.0	5.2
4SG62	5.3	5.3	5.3	5.2	5.3	5.3	
5CS61	3.9	4.1	4.1	3.9	4.0	4.0	3.9
5CS62	3.8	3.7	3.8	3.6	3.7	3.7	
6ES61	5.3	5.3	5.3	5.3	5.3	5.3	5.4
6ES62	5.4	5.5	5.7	5.4	5.5	5.5	
7W61	4.5	4.6	4.6	4.4	4.4	4.5	4.6
7W62	4.6	4.7	4.7	4.6	4.6	4.6	
8CEL61	5.6	5.6	5.6	5.5	5.6	5.6	5.7
8CEL62	5.7	5.7	5.8	5.6	5.6	5.7	
9S61	4.7	4.7	4.8	4.6	4.6	4.7	4.8
9S62	4.8	4.9	4.9	4.8	4.9	4.9	
10CA61	3.1	3.2	3.2	3.0	3.0	3.1	3.1
10CA62	3.0	3.0	3.0	3.0	3.0	3.0	

TABLE 76. CALCULATION OF THE COEFFICIENT OF THERMAL EXPANSION OF DRY SPECIMEN
Test Method No. 2
(inch/inch/1 F) x 10^6

Sample	$-\Delta R_c$* x 10^6	ΔR_m* x 10^6	$-\Delta R_c + \Delta R_m + 588$ x 10^6	Coeff. of Thermal Expansion Individual**	Average of 2 Specimens
1C61	425	3	166	2.8	2.9
1C62	420	0	171	2.9	
2CES61	360	3	231	3.9	3.9
2CES62	360	0	231	3.9	
3C61	430	3	161	2.7	2.6
3C62	440	0	151	2.5	
3SG61	350	3	241	4.0	4.0
3SG62	355	0	236	3.9	
4SG61	305	3	286	4.8	4.7
4SG62	320	0	271	4.5	
5CS61	385	3	206	3.4	3.5
5CS62	380	0	211	3.5	
6ES61	295	3	296	4.9	5.1
6ES62	280	0	311	5.2	
7W61	350	3	241	4.0	4.3
7W62	320	0	271	4.5	
8CEL61	290	3	301	5.0	5.0
8CEL62	285	0	306	5.1	
9S61	320	3	271	4.5	4.6
9S62	315	0	276	4.6	
10CA61	440	3	151	2.5	2.5
10CA62	440	0	151	2.5	

*Values of R_c and R_m are averages of 5 complete cycles.

**See equation 16a (Chapter XII).

APPENDIX VIII

THERMAL CONDUCTIVITY

A. Hot Plate Apparatus.

The cut-away cross-sectional plan of the hot plate is shown in Figure 72. The windings of the central heater and of the guard ring are exposed for the right half of the plate, which is symmetrical with the left half. The heater wires are embedded in 1/16 inch grooves, cut in 1/4 inch transite board, spaced 3/8 inch apart. As has been discussed in Chapter XIII, the winding of the central heater is completely separate from that of the guard ring. The former[74] consists of 28.5 ft. of No. 33 nickel-chromium wire, which has a resistance of 5.739 ohms per foot; the latter consists of approximately 15 ft. of No. 25 Chromel-A wire. The windings are covered with sheet mica for insulation and the whole covered in turn with a copper plate 1/4 inch thick for protection and to minimize surface temperature differences. The copper plate consists of two parts: the central portion 9 inch square covering the central heater, and the 1-7/8 inch wide outside (guard) ring. They are separated with a 1/8 inch air gap. The opposite side of the hot plate is identical to the side shown in Figure 72. Other information is given in Chapter XIII.

B. Thermocouple Location.

In the present tests, temperatures were measured by thermocouples made of thin (No. 30 A.W.G.) copper-constantan wires. The

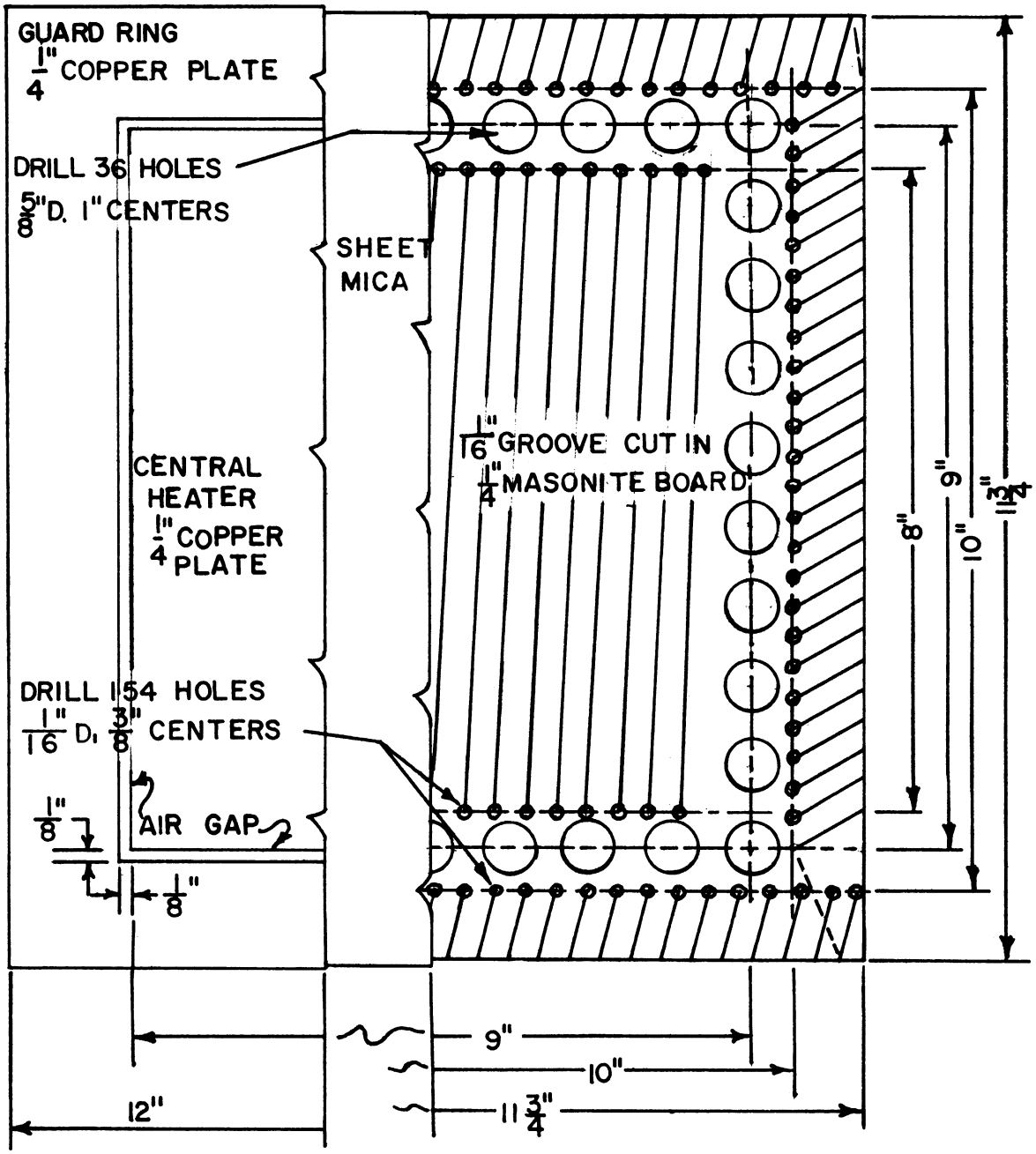


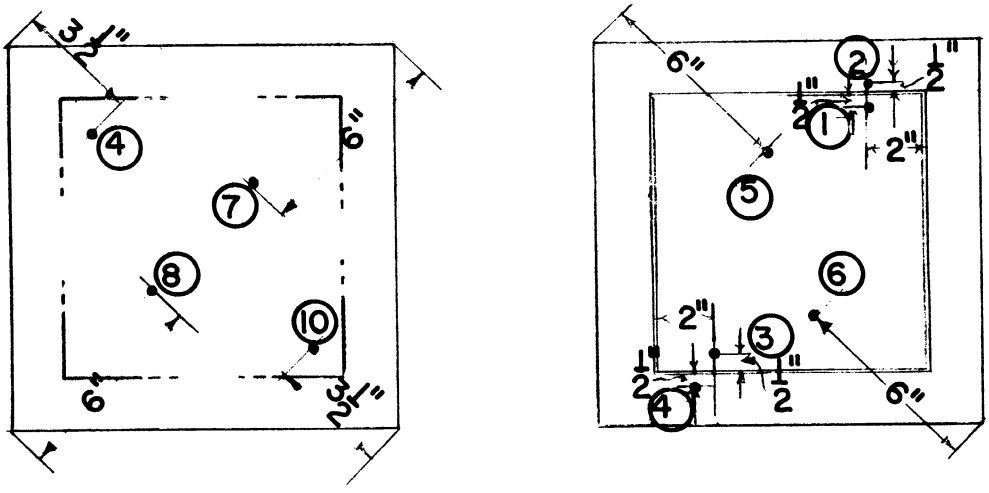
FIG. 72 CUT-AWAY PLAN FOR HOT PLATE

manufacture and the procedure of using these thermocouples were discussed in parts C and D of Chapter XIII. Ten thermocouples were used for each of the two companion specimens, tested simultaneously, for each product. The locations of these thermocouples are shown in Figure 73. In this figure, the companion specimens are denoted by numbers 1 and 2, and the thermocouples are numbered from 1 to 20 designating the pole numbers of the switch box to which each thermocouple was connected.

C. Discussion of Test Results.

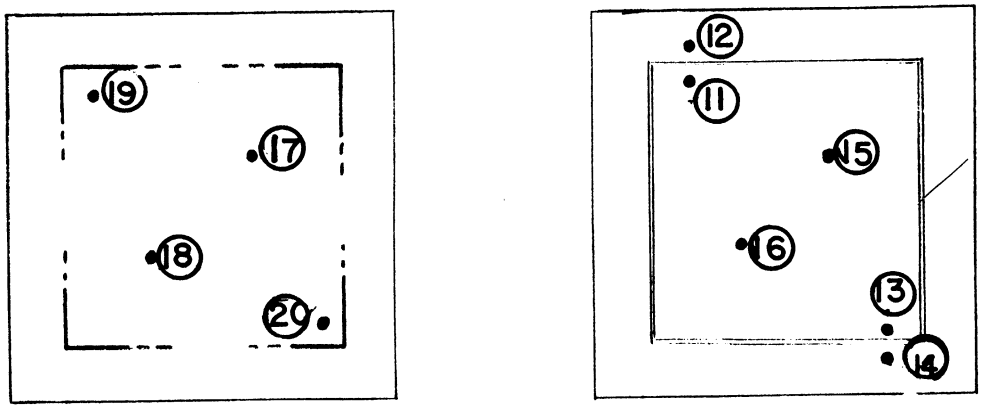
Results were calculated in Table 28 and summarized in Table 29; both tables are presented in Chapter XIII. Discussion of the results was also presented in part F of the chapter. However, the final individual current and temperature determinations are presented here, in Table 77. This table contains the measurements for two runs per product. In each case the values shown are those at the end of at least five hours of steady flow of heat through the specimens. The thermocouple's numbers in this table designate their locations, which may be seen in Figure 73.

It may be noted (see Table 77) that thermocouples No. 3 of the central heater and No. 4 of the guard ring consistently indicated lower temperatures than the remaining thermocouples. Pilot tests showed that this temperature variation is localized to a small area around the thermocouples, and is believed to be caused by some deficiency of the winding at this point. Because the area concerned is very small, the effect of this variation on the results should not be important. The actual readings of thermocouple No. 3 were included in the calculations of the results.



COLD PLATE HOT PLATE

SPECIMEN NO. 1



COLD PLATE HOT PLATE

SPECIMEN NO. 2

FIG.73 LOCATION OF THERMOCOUPLES FOR TWO COMPANION SPECIMENS

TABLE 77. FINAL CURRENT AND TEMPERATURE DETERMINATIONS FOR THERMAL CONDUCTIVITY TESTS
(Two Runs per Product)

Sample No.	Current Input			Temperature in Degrees Fahr.																				Room Temp. °F
	Central Heater		Guard Ring	Thermocouples of Specimens No. 1												Thermocouples of Specimens No. 2								
				Hot Plate						Cold Plate				Hot Plate				Cold Plate						
	volt	amp	volt	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	
1C10	65	.39	33	124	124	121	121	125	125	56.5	56	56.5	55.5	124	124	124	124	125	125	57	58	57	58	78
	65	.39	35	125.5	125	121	121	125.5	125.5	56	56	57	56	125.5	125.5	124	126.5	126.5	126.5	57.5	57	57.5	57	79
2CES10	65	.39	36	124	124.5	121	121	124	124	57	56	57	56	124	124.5	124	124.5	124	124	57	56	56	57	79
	70	.42	38	135	135.5	130	130	135	135	56.5	56.5	57	57	135	135	135	135	135	135	56.5	57.5	57.5	57	80
3C10	65	.38	32	126.5	126.5	120	120	126	126	56.5	55.5	56.5	55.5	126	126.5	126	126.5	126	126	58	57	57	58	78
	70	.42	36	140	140.5	137.5	138	139.5	140	57	56	57	59	139	139.5	139	139.5	139	139	58	57	58	57	79
3SG10	75	.43	39	111	111.5	106	107	111	111	64	64	65	64	111	111.5	111.5	111.5	111	109	64	65	65	64	78
	80	.48	41	119	120	110	110.5	119.5	118.5	65	64	65	64	119	119.5	119.5	120	119	113	65.5	65	65	65	80
4SG10	80	.48	40	114.5	115	113	114	114.5	115	60	59	60	59.5	114.5	115	115	115	115	115.5	60	59.5	60.5	60	79
	80	.48	40	115.5	116.5	113.5	114	116	115.5	61	60	61	60	116	117	115.5	116	116	116	61.5	60.5	62	60	78
5CES10	60	.36	32	115	116	116	116.5	115	116	59	58	59	58	114	115	117	117	117	117	59	58	59	58	79
	70	.42	38	130	131	130.5	131	131.5	130.5	61	60	61	60	128	129	132.5	133	132	132	59	59	60	59	79
6ES10	55	.33	32	127	127.5	127	127	127	127	53.5	52.5	53.5	52.5	128.5	130	126.5	127	128	128	53	52	53	52	81
	60	.36	32	136.5	136.5	137	136	136.5	136.5	52	53	53	52	137.5	137.5	136	135	136.5	136.5	52	52	52	58	79
7W10	50	.30	27	113	114	111.5	111	114	114	54	53	54	53	114	114	114	114	114.5	114	54	53.5	54	53.5	79
	60	.36	32	135.5	135.5	132	132	132.5	132.5	56	55	56	55	136.5	137	136.5	137	137	136.5	55.5	56.5	55.5	56.5	79
8CEL10	50	.30	25	120.5	120	117	118	120.5	120.5	52.5	51.5	52	52	120	120.5	120.5	121	120.5	120.5	53	52	52	52	79
	60	.36	34	147	147	143	144	147	148	54	53	54	53	146.5	147.5	147.5	148	147.5	148	56	55	56	55	79
9S10	60	.36	34	122	122.5	121	121	121	122	57.5	56.5	57	57	123	123	122	122	119	122	57	57	57.5	56.5	79
	70	.42	36	140	139	137	139	139.5	140.5	56	55.5	56	56.5	138.5	139	139.5	140	137	139.5	56	55	55	55	79
10CA10	55	.33	28	105.5	106	103	103	105.5	105	57.5	58	58	57.5	105	105	105.5	106	105	105.5	59	58	59	58	79
	70	.42	38	135	136	131	130	135	135	60	61	61	60	134	135	134	135	134.5	134.5	60	61	61	60	80

TABLE 78. THERMAL CONDUCTIVITY TEST
WEIGHTS OF SPECIMEN BEFORE AND AFTER TEST
(pounds)

Sample No.	Specimen No. 1		Specimen No. 2	
	Before Test	After Test	Before Test	After Test
1C10	25.12	25.16	28.14	28.16
2CES10	25.50	25.54	28.38	28.41
3C10	27.59	27.61	24.51	24.57
3SG10	40.20	40.22	36.16	36.20
4SG10	38.89	38.94	35.11	35.14
5CS10	28.40	28.43	26.11	26.10
6ES10	23.96	23.96	27.52	27.55
7W10	27.14	27.16	30.06	30.10
8CEL10	25.90	25.96	21.58	21.62
9S10	36.40	36.42	32.40	32.41
10CA10	24.76	24.79	28.96	28.99

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