Nondestructive Methods for Testing Concrete

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NONDESTRUCTIVE METHODS
FOR TESTING CONCRETE

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Department of Energy,
Mines and Resources, Ottawa

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PREFACE

Concrete industry has made rapid advances over the past 30 years and it is the Mines Branch policy to promote research and to disseminate information on subjects of importance to this industry. During this period of phenomenal growth there have been a number of attempts to develop quick, inexpensive and nondestructive methods for testing of concrete both in the laboratory and in the structures.

The Mines Branch has been in the forefront in the use of nondestructive methods for testing metals. During the past 12 years these methods have also been applied to the testing of concrete and as a result considerable data and experience have been accumulated in their use. Based upon this experience, this monograph critically examines the various nondestructive methods currently in use and discusses their advantages and limitations. It is hoped this monograph will be used by practising engineers, technologists and graduate students engaged in the testing of concrete and strength evaluation of concrete structures.

John Convey
Director

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PRÉFACE

L'industrie du béton a progressé rapidement au cours des 30 dernières années; la Direction des mines centre sa politique sur l'encouragement à la recherche et sur la diffusion de renseignements importants relatifs à cette industrie. Plusieurs tentatives de mise au point de méthodes rapides, peu coûteuses et non destructives d'essai du béton en laboratoire et dans les structures, ont marqué cette période de croissance phénoménale.

La Direction des mines demeure au premier plan dans la recherche de méthodes non destructives d'essai des métaux. Ces méthodes, appliquées au cours des 12 dernières années aux essais de béton, ont permis d'accumuler des données en quantité considérable et d'acquérir une riche expérience. La présente monographie, qui est le fruit de cette expérience, expose les diverses méthodes non destructives d'usage courant et passe en revue leurs avantages et leurs limitations. Espérons que les ingénieurs, les technologistes et les diplômés qui s'occupent des essais de béton et de l'évaluation de la résistance des structures de béton sauront mettre à profit cette monographie.

John Convey
Directeur

NONDESTRUCTIVE METHODS FOR TESTING CONCRETE

*V. M. Malhotra**

ABSTRACT

This monograph deals with nondestructive methods for testing concrete. The tests considered are based on (1) surface hardness (2) dynamic, and (3) radioactive methods.

Different methods of hardness testing are briefly mentioned and the Schmidt test hammer, based on the rebound principle, is described in detail. The calibrating procedure for the hammer is given and the test data published by various researchers are included.

Dynamic tests utilizing resonant frequency, mechanical sonic and ultrasonic pulse velocity are described in detail. The test equipment and the test procedures for these methods are outlined and their various applications and limitations are discussed.

A radioactive test method employing X-ray penetration tests is briefly mentioned. The principle of gamma radiography is given and some of its applications are described.

The monograph concludes with a list of 81 pertinent references.

RÉSUMÉ

La présente monographie traite des méthodes non destructives d'essai du béton. Les épreuves étudiées sont fondées sur (1) la dureté de surface, ainsi que sur (2) la méthode dynamique et (3) la méthode radio-active.

L'auteur mentionne brièvement différentes méthodes d'essais de dureté et décrit en détail le marteau d'essai Schmidt qui est une application du principe du rebondissement. Il explique aussi le procédé de calibrage du marteau et il rapporte les données sur les essais, publiées par divers chercheurs.

La monographie contient une description détaillée des essais dynamiques fondés sur la fréquence de résonance et sur la vitesse de propagation des ondes mécaniques sonores et ultrasonores. Elle traite brièvement de l'équipement et des procédés d'essai pour ces méthodes ainsi que de leurs diverses applications et limitations.

Enfin, l'auteur mentionne une méthode radio-active d'essai fondée sur la pénétration des rayons X, en plus de donner le principe de la gammagraphie et de décrire brièvement quelques-unes de ses applications.

La monographie se termine par une liste de 81 références appropriées.

This monograph was originally prepared for presentation as a basic text at the two-day course "Principles of Concrete Technology", sponsored by the Canadian Capital Chapter, American Concrete Institute, Ottawa, Canada, December 7 and 8, 1966.

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INTRODUCTION

At the present time, standard methods for testing hardened concrete consist of testing concrete specimens in tension, flexure, and compression. The main disadvantages of such methods are the delay in obtaining test results, the fact that the test specimens may not be truly representative of the concrete in the structure, the necessity of stressing the test specimens to failure, the lack of reproducibility in the test results, and the relatively high cost of testing. As a result, there have been a large number of attempts, over a period of about 30 years, to develop quick, inexpensive and nondestructive methods for testing concrete, both in the laboratory and in the structure.

Because the determination of strength implies that concrete specimens must be loaded to failure, it becomes abundantly clear that the nondestructive methods of testing cannot be expected to yield absolute values of strength. These methods, therefore, attempt to measure some other physical property of structural concrete from which an estimate of the strength and elastic parameters of concretes is obtained. Some such properties of concrete are its hardness, its resonant frequency, and its ability to absorb, scatter and transmit X-rays and gamma rays. Based upon these properties, various nondestructive methods of testing have been developed, which are classified as follows:

1. *Surface Hardness Tests*—These are used only for estimation of concrete strength.

2. *Dynamic Tests*—These include resonant frequency and mechanical sonic and ultrasonic pulse velocity methods. These are generally used to evaluate durability and uniformity of concrete, and to estimate its strength and elastic properties.

3. *Radioactive Methods*—These include the X-ray and gamma-ray penetration tests and have been used to measure the density and thickness of concrete.

This monograph discusses in some detail the surface hardness and dynamic tests; radioactive methods are of limited application and are only briefly described.

SURFACE HARDNESS TESTS

INDENTATION METHODS

Indentation methods are based upon the well-known nondestructive methods which use the Brinell indentation principle in testing metals. They consist essentially of hitting the surface of concrete in a standard manner and measuring the size of the indentation. The three known methods are:

(i) The Williams testing pistol

(ii) The Frank spring hammer

(iii) The Einbeck pendulum hammer.

The indentation tests have won only limited recognition because of difficulties in measuring accurately the diameter and the depth of the indentation.

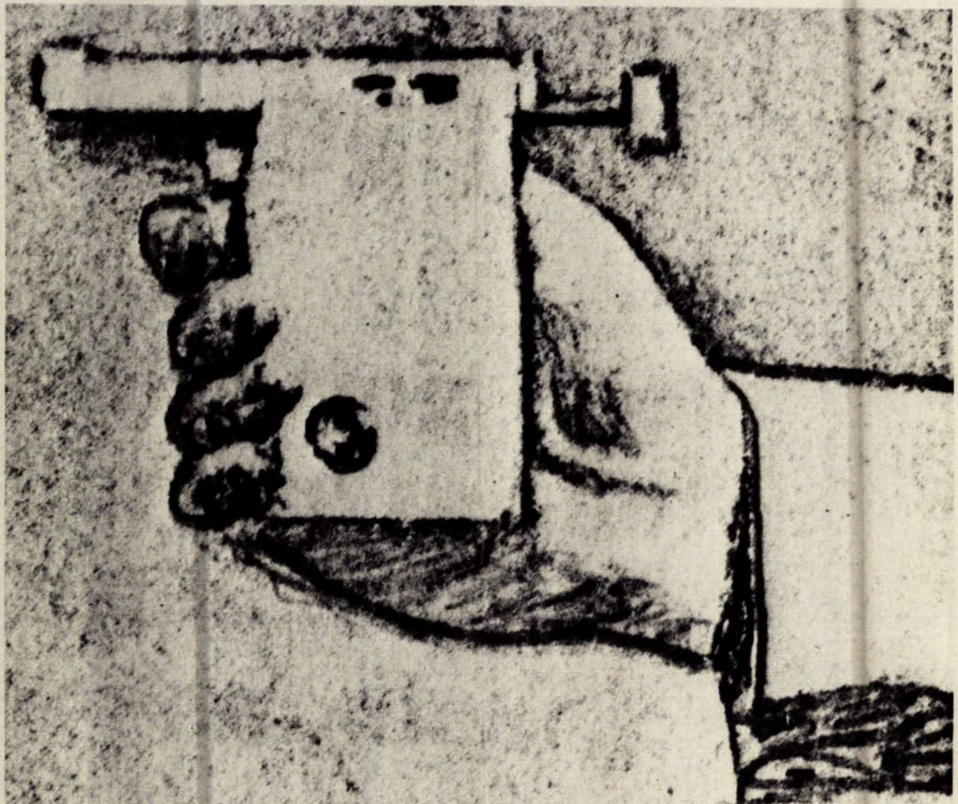

FIGURE 1. Testing pistol. After Williams (1).

Williams Testing Pistol

In 1936, Williams (1)* developed a "testing pistol" which uses a ball as an indenter as in the Brinell Test. Instead of being acted upon by a sustained load, however, the ball is projected with a suitable force by means of a specially designed pistol. The diameter of the impression made by the ball is measured by a magnifying scale or other means.

The testing pistol (Figure 1) measures approximately 6 x 5 x 1½ inches and weighs about 2 pounds.

The utility of the method, according to Williams, depends on the approximate relationship which has been found to exist between the compressive strength of concrete and the resistance of its surface to impact. On the basis of some 200 tests, Williams established the following relationship:

$$f_c \text{ is proportional to } 1/Z,$$

where f_c is the compressive strength, and Z is the curved surface area of indentation.

Frank Spring Hammer

This hammer was developed in Germany (2)** and consists of a spring-controlled mechanism housed in a tubular frame. A line diagram of the hammer is shown in Figure 2. The tip of the hammer can be fitted with different diameters of ball, and impact is achieved by placing the hammer up against the surface under test and manipulating the spring mechanism. The diameter or depth of indentation is measured and this in turn is correlated with the compressive strength of concrete.

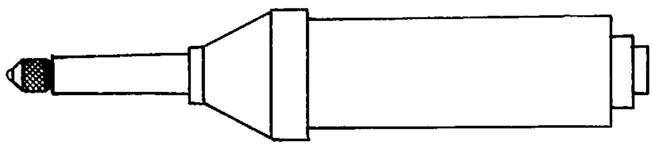

FIGURE 2. Frank spring hammer. After Gaede (2).

Einbeck Pendulum Hammer

This hammer was developed in Germany (2, 3) by Einbeck. A line diagram of the hammer is given in Figure 3. The hammer consists of a horizontal leg at the end of which is pivoted an arm with a pendulum head weighing about 5 pounds. The indentation is made by holding the horizontal leg against the concrete surface under test and allowing the pendulum head to strike the concrete. The diameter

*The numbers in parentheses refer to the list of references appended to this monograph.

**Manufactured by Karl Frank G.M.b.H. Prufmaschinenbau, 17a Weinheim-Birkenau (Odenwald), West Germany.

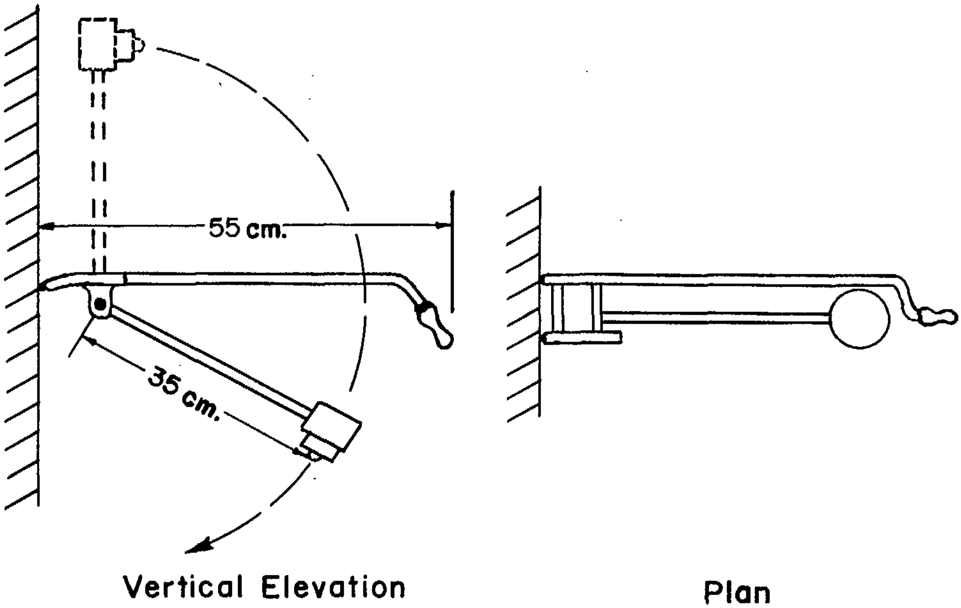

FIGURE 3. Einbeck pendulum hammer. After Gaede (4).

and depth of indentation are measured and these are then correlated with the compressive strength of concrete. The height of fall of the pendulum head can be varied from full impact (180 degrees) to half impact (90 degrees). Figure 4 shows the hammer in use.

The biggest drawback of this instrument is that it can only be used on vertical surfaces.

The main features of the Frank spring hammer and the Einbeck pendulum hammer are compared in Appendix I.

REBOUND METHOD

This method is based upon the rebound principle as postulated by Shore(4). The Shore scleroscope method depends on the height of rebound of a hardened steel hammer when it is dropped on the metal under test. The only known instrument using this principle for concrete testing is the test hammer by Schmidt.

Schmidt Test Hammer

In 1948 a Swiss engineer, Ernst Schmidt, developed a test hammer for measuring the surface hardness of concrete by the rebound principle. Results of his work were presented to the Swiss Federal Materials Testing and Experimental Institute of Zurich (2, 5, 6, 7), where the hammer was constructed and extensively tested. Since then this nondestructive method has gained world-wide recognition in laboratories, at construction sites, and in the precast concrete industry.

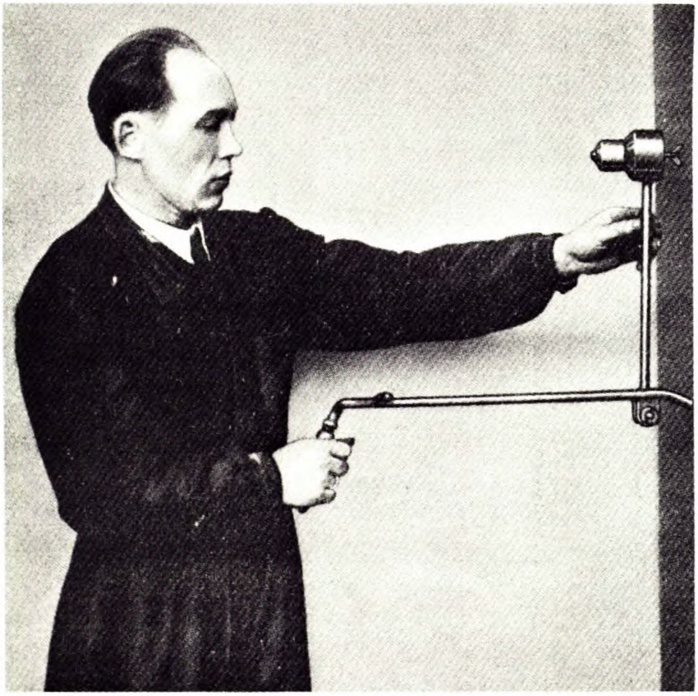

FIGURE 4. Einbeck pendulum hammer in use. After Gaede (2).

Principle

The Schmidt rebound hammer is principally a surface hardness tester with little apparent theoretical relationship between the strength of concrete and the rebound number of the hammer. However, within limits, empirical correlations have been established between strength properties and the rebound number. Further, Kolek (8) has attempted to establish a correlation between the hammer rebound number and the hardness as measured by the Brinell method.

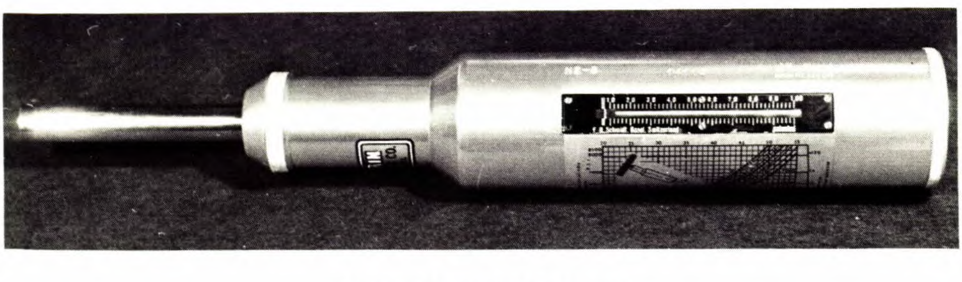

FIGURE 5. Schmidt test hammer.

Description

The Schmidt rebound hammer type N 2 (others are type M and type L) is shown in Figure 5. The hammer weighs about 4 pounds and is suitable for use both in the laboratory and in the field.

A cut-away view of the hammer is shown in Figure 6. It consists of a spring-controlled weight E, which slides on plunger A, housed within the tubular frame B. When the spring H is fully extended, an automatic trigger J is engaged, thus causing the weight to strike the plunger. As the weight E rebounds from the plunger, it moves the sliding index C along the scale D. By pushing button F, the sliding index can be held in position to allow readings to be taken.

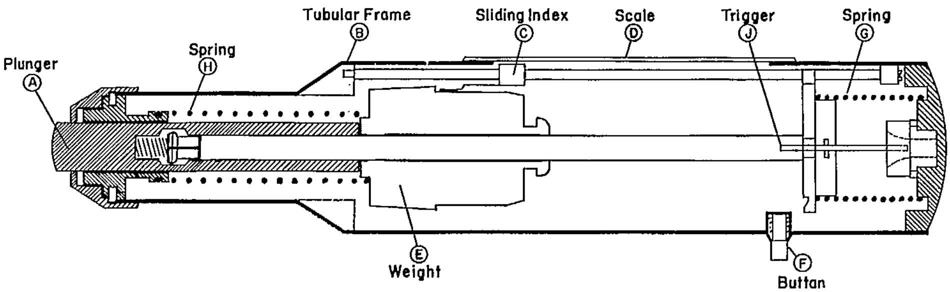

FIGURE 6. A cut-away view of the Schmidt test hammer. After Kolek (8).

Method of Testing

The determination of the hammer rebound number is a very simple procedure and is outlined in the manual supplied by the manufacturer. Briefly, it consists of releasing the plunger from the locked position by pressing it gently against a hard surface. The hammer is then ready for use. To carry out the test, the plunger is pressed strongly against the concrete surface under test. This releases the spring-loaded weight from its locked position, thus causing an impact. While the hammer is still in its testing position, the sliding index is read to the nearest whole number. This reading is designated as the hammer rebound number. The number of the readings to be taken per test is the same as for calibrating the hammer, as described in the section on calibration procedure.

Figures 7 and 8 show the hammer being used to test a 6 x 12-inch concrete cylinder and the concrete wing wall of a roadway bridge in Ottawa, Canada (9).

The test can be conducted horizontally, vertically upwards or downwards, or at any intermediate angle. At each angle the rebound number will be different for the same concrete, and will require separate calibration or correction charts. Zoldners (10) has shown that 5 points have to be added to the readings in the downward direction to translate these readings to the values for horizontal testing.

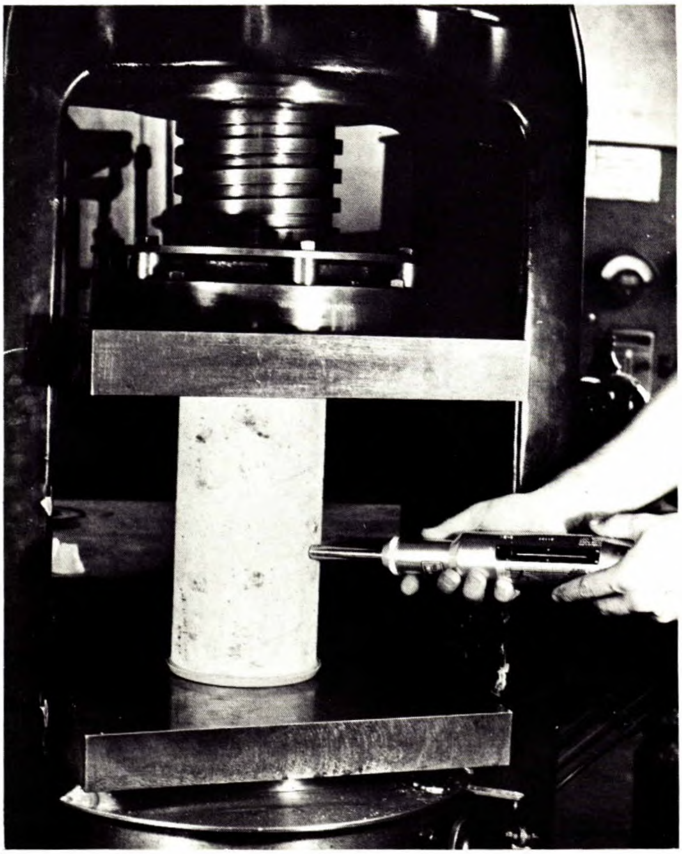

FIGURE 7
Schmidt test hammer in use to test a 6 x 12-inch concrete cylinder. After Malhotra (9). Note that the cylinder has been restrained in a compression testing machine.

Calibration Procedure

Each hammer is furnished with a calibration chart supplied by the manufacturer. This calibration chart can be used only when material and testing conditions are similar to those in effect when the calibration of the instrument was carried out. Each hammer varies slightly in performance, and needs calibration for use on concrete made with aggregates produced from a specific source. A practical procedure for calibrating the hammer is outlined below:

(*a*) Prepare a number of 6 x 12-inch cylinders (or 6-inch cubes) covering the strength range which is to be encountered on the job site. Use the same cement and aggregates as are to be used on the job. Cure the cylinders under standard moist curing room conditions (temperature 73 $\pm$ 2°F and relative humidity 100 per cent), keeping the curing period the same as the specified control age in the field.

(*b*) After capping, place the cylinders in a compression testing machine, under an initial load of approximately 15 per cent of the ultimate load to restrain the specimen (Figure 7). Ensure that cylinders are in a saturated surface-dry condition.

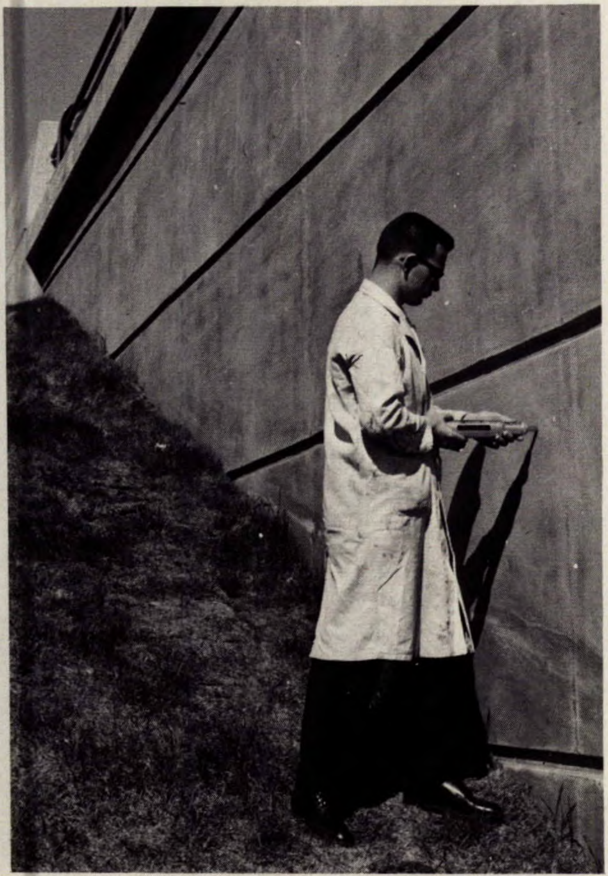

FIGURE 8
Concrete wing wall of a roadway bridge under test using the Schmidt test hammer. After Malhotra (9).

(*c*) Make 15 hammer rebound readings, five on each of three vertical lines, 120 degrees apart, against the side surface in the middle two-thirds of each cylinder. Avoid hitting the same spot twice. For cubes, select the central area of the moulded face and take 15 readings without hitting the same spot twice.

(*d*) Discard those five readings that are either too high or too low. These may be due to hitting stone particles or air voids behind apparently sound surface areas.

(*e*) Average the readings and call this the rebound number for the cylinder (or cube) under test.

(*f*) Repeat this procedure for all the cylinders (or cubes).

(*g*) Test the cylinders (or cubes) to failure in compression, and plot the rebound numbers against the compression strengths in psi on a graph.

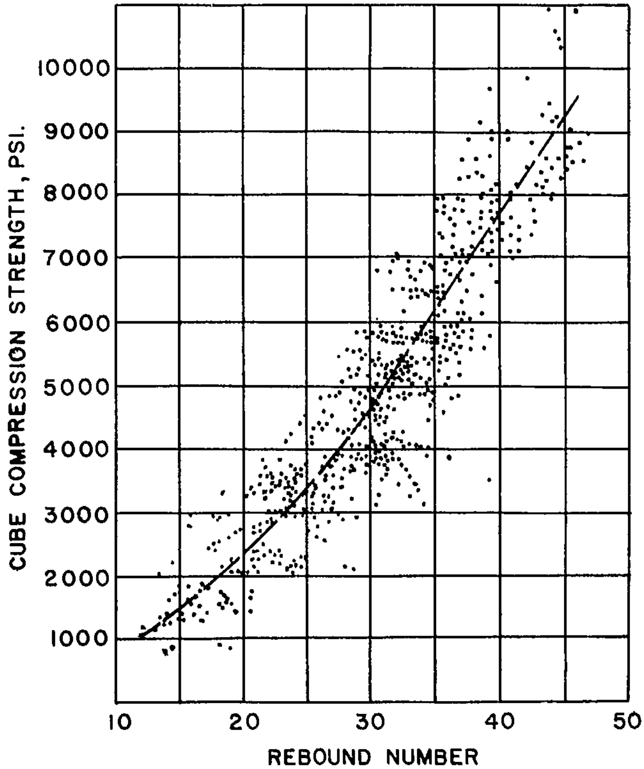

FIGURE 9. Relationship between cube compressive strength and Schmidt rebound number determined by Swiss Federal Testing Laboratory for hard-aggregate concretes. After Greene (11).

Fit a curve or a line by the method of least squares and draw the respective 95 per cent confidence limits.

A typical curve established by the Swiss Federal Testing Laboratory (11) for hard aggregate concrete is shown in Figure 9.

Figure 10 shows four calibration curves obtained by research workers in five different countries. It is important to note that some of the curves deviate considerably from the curve which is supplied with the hammer.

Limitations of Schmidt Test Hammer

Although the test hammer provides a quick, inexpensive means of checking concrete quality, it has serious limitations and these have to be recognized. The results of the Schmidt test hammer are affected by:

(*a*) Smoothness of the surface under test;

(*b*) Size, shape and rigidity of the specimen;

(*c*) Age of the specimen;

(*d*) Surface and internal moisture condition of the concrete;

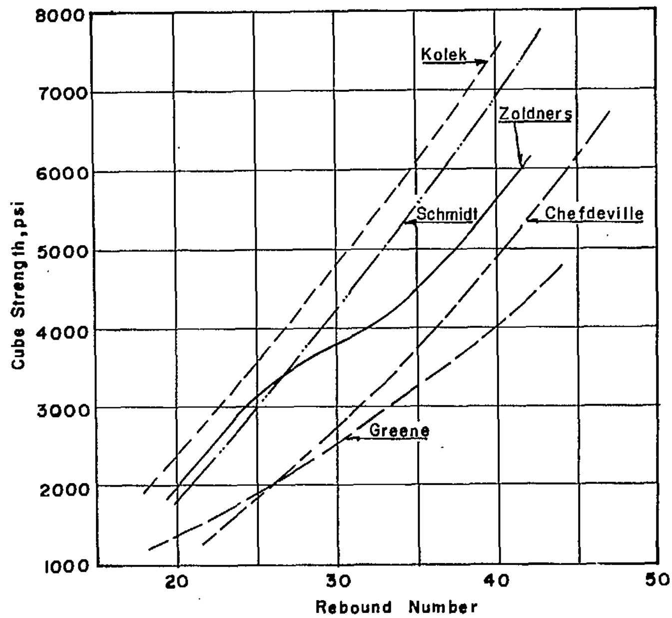

FIGURE 10
Calibration curves obtained by different investigators with a Schmidt type No. 2 test hammer. Curve by Greene was obtained with type N. After Kolek (8) and Zoldners (10).

(*e*) Type of coarse aggregate;

(*f*) Type of mould.

These limitations are discussed in the foregoing order.

(*a*) *Smoothness of surface under test*—This has an important effect on the accuracy of the test results. The Swiss Federal Materials Testing and Experimental Institute recommends that the hammer be used only on surfaces where the concrete was cast against forms (11). Whenever the formed surface is rough, more accurate results can be obtained by grinding it to *uniform smoothness* with a carborundum stone. It has been shown (8, 11) that trowelled surfaces or surfaces made against metal forms yield rebound numbers from 5 to 25 per cent higher than do surfaces made against wooden forms. This implies that if such surfaces are to be used, a special calibration curve or a correction chart must be obtained. Further, trowelled surfaces will give a higher scatter of individual results and, therefore, a low confidence for such a correlation.

(*b*) *Size, shape and rigidity of test specimens*—If the concrete section or a specimen is small, such as a thin beam, wall, 6-inch cube, or 6 x 12-inch cylinder, any movement under the impact will lower the rebound readings. In such cases the member has to be fixed or backed up by a heavy mass.

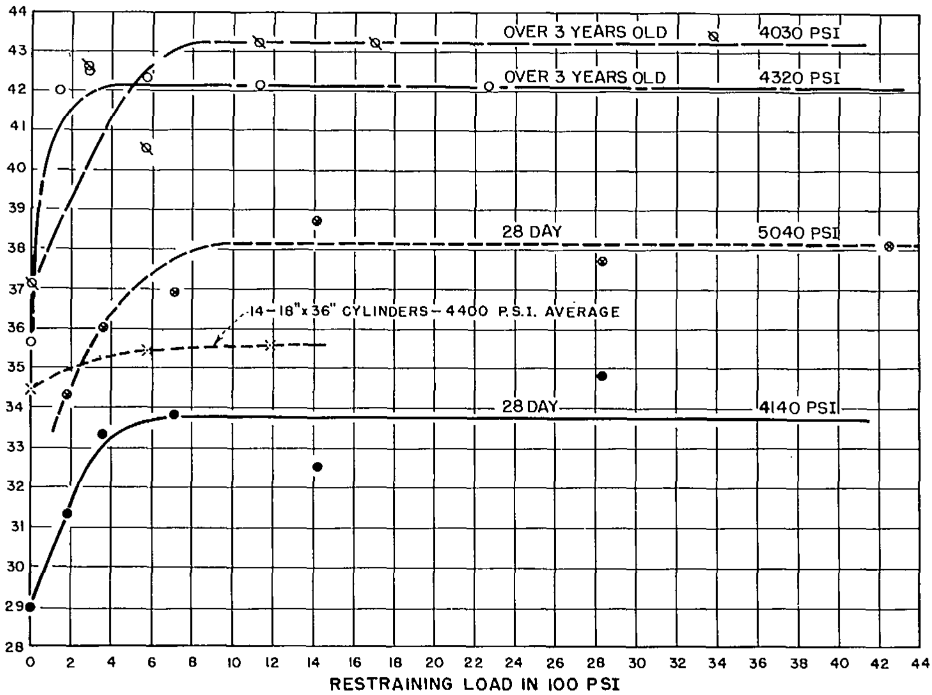

FIGURE 11. Restraining load vs rebound readings for 6 x 12-inch cylinders. After Mitchell and Hoagland (12).

If small test specimens, for example, 6 x 12-inch cylinders, are the only ones available, it is best to grip the specimen in the testing machine as outlined in the calibrating procedure. This eliminates movement and increases the effective mass of the cube by that of the machine. It is, however, suggested that smaller test pieces should preferably be avoided, because they give consistently lower rebound numbers and a higher scatter of results.

It has been shown (12) that the restraining load at which the rebound number remains constant appears to vary with the individual specimen; however, the effective restraining load for consistent results appears to be about 15 per cent of the ultimate strength of the specimen (Figure 11). Zoldners (10), Greene (11), Grieb (13), have indicated effective loads of 150, 250 and 300 psi respectively and these are considerably lower than the 15 per cent value obtained by Mitchell and Hoagland (12).

(*c*) *Age of test specimens*—It is claimed (10) that for equal strength, higher rebound values are obtained on 7-day cylinders than on 28-day cylinders. The explanation is that at early ages the outside surface of concrete probably hardens faster than the internal strength increases. It is suggested that when old concrete is to be tested, direct correlation should be established between the rebound numbers and the compressive strength of cores taken from the structure.

The use of the Schmidt hammer for testing low-strength concrete at early ages, or where concrete strength is less than 1,000 psi, is not recommended (12), because rebound numbers are too low for accurate reading and the test hammer badly damages the concrete surface. Figure 12 shows blemishes on surfaces of 8-hour-and 3-day-old concrete cylinders, caused by hammer impact.

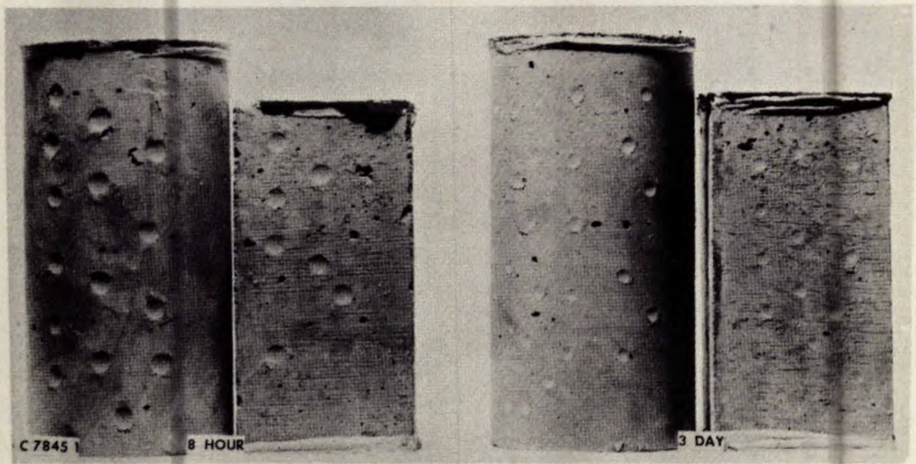

FIGURE 12. Eight-hour-old (left) and three-day-old (right) specimens showing surface blemishes after hammer impact. After Mitchell and Hoagland (12).

(*d*) *Surface and internal moisture condition of the concrete*—The degree of saturation of the concrete and the presence of surface moisture have a decisive effect on the evaluation of test hammer results (10, 14). It has been demonstrated (10) that well cured, air-dry specimens, when soaked in water and tested in the saturated surface-dry condition, show rebound readings 5 points lower than when tested dry. When the same specimens were left in a room at 70°F and air dried, they recovered 3 points in 3 days and 5 points in 7 days. Klieger (15) has shown that differences up to 10 to 12 points in rebound numbers exist in a case of 3-year-old concrete (made with type III cement and a sand and gravel from Texas) between specimens stored in a wet condition and laboratory-dry samples. This difference in rebound numbers represents approximately 2,000 psi compressive strength.

It is suggested that, whenever the actual moisture condition of the field concrete or specimen is not known, it would be desirable to presaturate the surface several hours prior to testing and use the correlation for the saturated surface-dry condition.

(*e*) *Type of coarse aggregate*—It is generally agreed that the rebound number is affected by the type of aggregate used. According to Klieger (15), for equal compressive strengths, concretes made with crushed limestone coarse aggregate show rebound numbers approximately 7 points lower than those for concretes

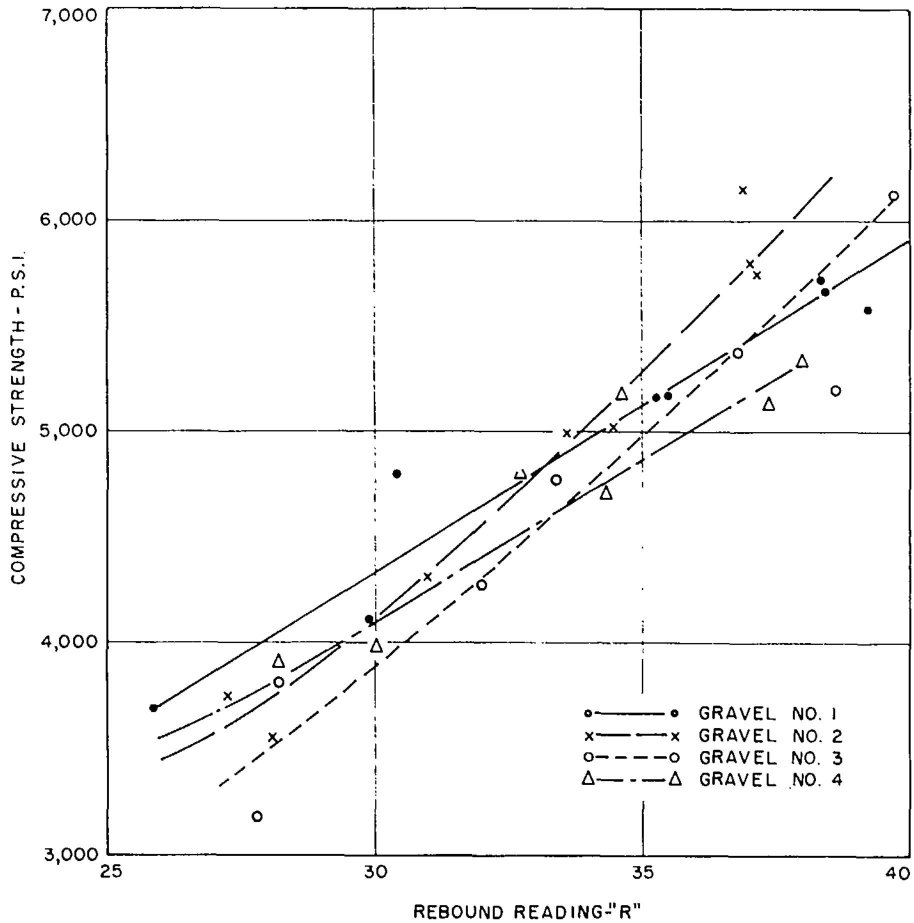

FIGURE 13. Effect of gravel from different sources on rebound readings of 6 x 12-inch concrete cylinders. After Grieb (13).

made with gravel coarse aggregate, representing approximately 1,000 psi difference in compressive strength.

Grieb (13) has shown that, even though the type of coarse aggregate used is the same, if it is obtained from different sources different calibration curves would be needed. Figure 13 shows results of one such study where four different gravels were used in making the concrete cylinders tested. The spread in compressive strength among the curves representing the concrete prepared with the four gravel coarse aggregates varied from 250 to 600 psi.

Greene(11), in his applications of the Schmidt hammer, found that the use of the test hammer on specimens and structures made of lightweight concrete showed widely differing results. For example, lightweight concrete made with expanded shale aggregate yielded different rebound numbers than concrete made with pumice aggregate, at equal compressive strengths. But for any given type of lightweight aggregate concrete, the rebound numbers proved to be proportional to the compressive strength.

(*f*) *Type of Mould*—Mitchell and Hoagland (12) have carried out studies to determine the effect of the type of concrete mould on the rebound number. Companion cylinders cast in steel, tin can, and paper carton moulds showed no significant difference in the rebound readings between those cast in steel moulds and tin can moulds, but the paper carton-moulded specimens gave higher rebound numbers. This is probably due to the fact that the paper mould withdraws moisture from the plastic concrete, thus lowering the water-cement ratio at the surface and resulting in a higher strength in this area. Since the hammer is a surface hardness tester, it is possible for the hammer to indicate nonexistent high strength from a hardened surface. It is therefore suggested that if paper carton moulds are being used in the field, the hammer should be calibrated against the strength results obtained from test cylinders cast in such moulds.

Rebound Number and Compressive Strength

It is generally agreed (16, 17, 18) that there is a correlation between compressive strength of concrete and the hammer rebound number. However, there is a wide degree of disagreement among various researchers concerning the accuracy of the estimation of strength from the rebound readings. Coefficients of variation for compressive strength for a wide variety of specimens average 18.8 per cent, and exceed 30 per cent for some groups of specimens (12). Figure 14 shows one typical histogram for rebound number 20 (18). These large deviations in strength can be narrowed down considerably by proper calibration of the hammer which allows for various variables discussed earlier. By consensus, the accuracy of estimation by a properly calibrated hammer lies between $\pm$ 15 and $\pm$ 20 per cent.

Recently, Boundy and Hondros (19) have suggested the use of the rebound hammer in conjunction with some method of accelerated curing to provide a rapid and convenient method for estimating the strength and quality of concrete.

Rebound Number and Flexural Strength

Various investigators (11, 15) have established correlations between the flexural strength of concrete and the hammer rebound number. They have found that the relationships are similar to those obtained for compressive strength, except that the scatter of the results is greater. Further, they found that the results of tests conducted on the top or finished surface of a beam were 5 to 15 per cent lower than those conducted on the sides of the same beam.

The effects of moisture condition and aggregate type on the flexural strength are similar to those found in the compressive strength.

Rebound Number and Modulus of Elasticity

Mitchell and Hoagland (12) have attempted to correlate hammer rebound number with the modulus of elasticity of the concrete specimens. They concluded that no valid correlation could be made directly between the rebound number

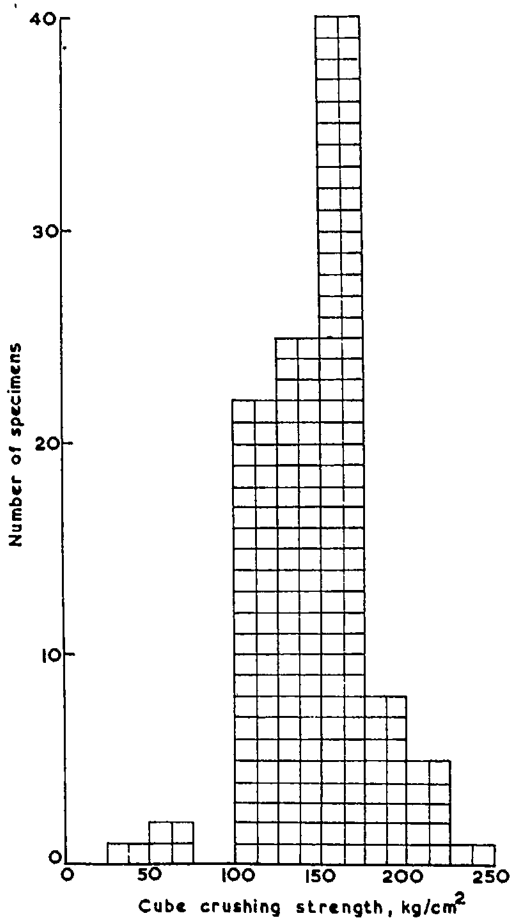

FIGURE 14
Typical histogram for rebound number 20. After Victor (18).

and modulus of elasticity; however, a satisfactory relationship between the two might be possible if the hammer were to be calibrated for each individual mix tested.

Petersen and Stoll (16) and Klieger (17) have established an empirical relationship between dynamic* modulus of elasticity and rebound number. They have shown that the relationships are affected by both moisture condition and aggregate type in the same manner as for compressive and flexural strengths.

LIMITS AND USEFULNESS OF SURFACE HARDNESS TESTS

On the basis of the literature reviewed and the work carried out at the Mines Branch, it is concluded that:

*Modulus of elasticity obtained by flexural vibration of cylindrical or prismoidal specimen.

1. Concrete hardness test methods based on the indentation principle have not found wide acceptance.
2. The Schmidt test hammer, based on the rebound principle, provides a simple and quick method for nondestructive testing of concrete in the laboratory, in the precast concrete industry, and in the field.
3. The limitations of the Schmidt hammer should be recognized. It is emphasized that this instrument must not be regarded as a substitute for standard compression tests but as a method of providing very limited field control at a negligible cost. Prediction of strength of concrete by the rebound hammer may be possible only for specimens cast, cured and tested under identical conditions as those from which the calibration curves are established.

DYNAMIC METHODS

Since 1938 dynamic testing techniques have been used for testing concrete, both in the laboratory and in the field. In addition to measuring the fundamental properties of concrete in the laboratory, these methods have been used to check the quality of concrete in bridge piers, road pavements, and concrete hydraulic structures of up to 50 feet in thickness.

The fundamental principles on which these methods are based were given by Rayleigh as early as 1877, when he reported the mathematical relationship existing between resonant frequency of vibration of a specimen, the velocity of sound through the material, and its modulus of elasticity (20).

Dynamic testing techniques can be divided into three principal methods. These are:

(*a*) *Resonant Frequency Method*—This method is based upon determination of the fundamental resonant frequency of vibration of a specimen, the continuous vibrations being generated electronically. This method has been standardized by the American Society for Testing and Materials (ASTM).

(*b*) *Mechanical Sonic Pulse Velocity Method*—This method involves measurement of the time of travel of longitudinal or compressional waves generated by a single impact hammer blow or repetitive blows. It has found relatively little acceptance.

(*c*) *Ultrasonic Pulse Velocity Method*—This method involves measurement of the time of travel of electronically generated pulses through concrete, the time interval being measured by a cathode ray oscilloscope. This method has gained considerable popularity and also has been standardized by the ASTM.

Resonant Frequency Method

This method was first developed by Powers (21) in the United States in 1938. He determined the resonant frequency by matching the musical tone given by the specimen, when tapped by a hammer, with that given by one of a set of orchestra bells calibrated according to frequency.

A year later, in 1939, Hornibrook (22) refined the method by using electronic equipment to measure the resonance. Other early investigations on the development of this method included those by Thompson (23) in 1940, Obert and Duvall (24) in 1941, and Stanton (25) in 1949. In all the tests that followed the work of Hornibrook, the specimens were driven vertically and the resonant frequency read accurately from the graduated scale of a variable audio oscillator. The equipment is usually known as a Sonometer. The ASTM published this method as a tentative standard in 1947 and since then has revised it four times (26). In the United Kingdom this method was incorporated in British Standards (27) in 1952.

Resonant frequency methods are used almost exclusively within the laboratory and involve the determination of frequencies of vibration of concrete prisms and cylinders for the purpose of calculating Young's moduli of elasticity and rigidity, for determining the Poisson's ratio, and for durability studies. The modulus measured by this method is commonly referred to as the resonance or dynamic modulus instead of the pulse or sonic modulus (28) in order to avoid confusion with the Soniscope, which is the name of the instrument used for determining pulse velocity and which will be described later.

Salient Features of a Typical Testing Apparatus

The testing apparatus required by ASTM Standard C 215-60 is shown schematically in Figure 15. Equipment meeting the ASTM specifications has been

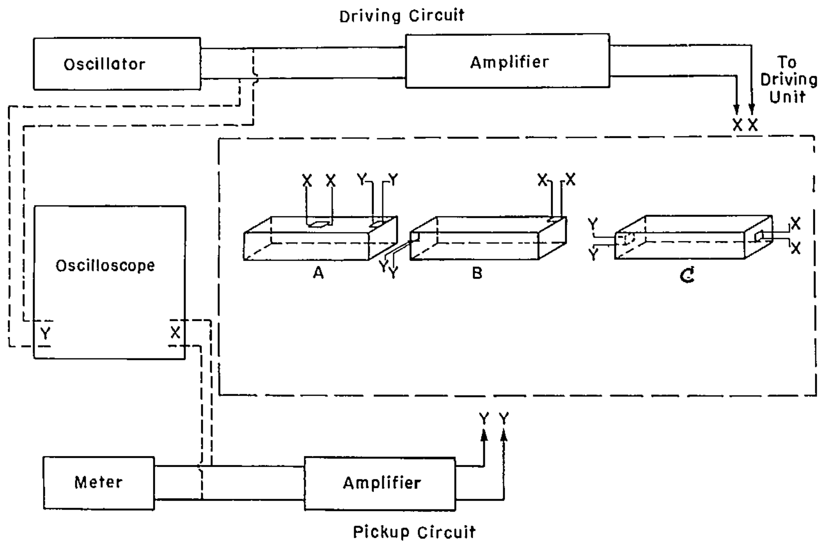

FIGURE 15. Schematic diagram of a typical apparatus, showing driver and pick-up positions for the three types of vibration. After ASTM Standard C 215-60.

A—for transverse resonance
B—for torsional resonance
C—for longitudinal resonance

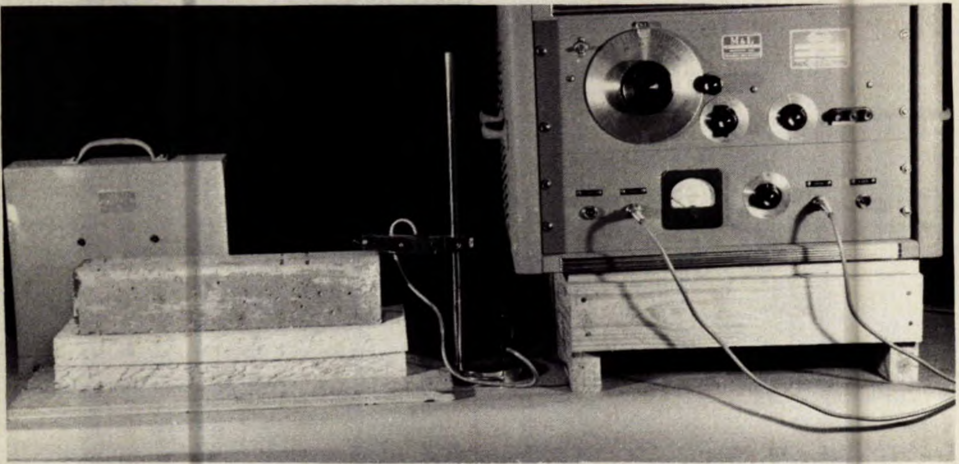

FIGURE 16a. A $3\frac{1}{2}$ x 4 x 16-inch concrete beam being transversely vibrated by a Sonometer manufactured in the United States. The pick-up is shown between the Sonometer and the beam, while the vibrating unit is on the left. Note that the beam is supported on a cushion of soft rubber.

designed by various commercial organizations and is currently being used by several laboratories in Canada and the United States. Commercially available Sonometers from the United States and Great Britain are shown in Figures 16a and 16b.

The testing apparatus consists primarily of two sections, one of which generates mechanical vibrations and the other senses these vibrations.

Vibration Generating Section

The principal part of this section is an electronic audio frequency oscillator, which generates electrical audio frequency voltages. The oscillator output is amplified to a level suitable for producing mechanical vibrations. The relatively undistorted power output of the amplifier is fed to the driver unit for conversion into mechanical vibrations.

Vibration Sensing Section

The mechanical vibrations are sensed by a piezo-electric crystal pick-up. The pick-up is contained in a separate unit and converts mechanical vibrations to electrical frequencies. These frequencies are amplified for the operation of a panel-mounted resonance-indicating meter. As the driver oscillator is turned to the proper frequency, deflection of the meter needle indicates resonance. Visible indications of fundamental modes of different vibrations can be obtained easily through the use of an auxiliary cathode ray oscilloscope, and its use is generally recommended.

Use and Operation of Sonometer

Some skill and experience are needed to determine the resonant frequency as shown by a meter-type indicator or a cathode-ray oscilloscope, because several

FIGURE 16b. A view of the Electrodynamic tester made in Great Britain. Test bench in lower half of photograph shows driver (right) and pick-up (left). (*Courtesy of M. Falk & Co. Ltd., England.*)

maximum frequencies may be obtained. Specimens having either very small or very large ratios of length to maximum transverse direction are frequently difficult to excite in the fundamental mode of vibration. It has been suggested that the best results are obtained when this ratio is between three and five. ASTM Designation C 215-60 specifies that the above ratio be at least as great as two.

The supports for the specimen under test should be of a material having a fundamental frequency outside the frequency range being investigated and should permit the specimen to vibrate without significant restriction. Ideally, the specimens should be held at the nodal points but a sheet of soft sponge rubber is quite satisfactory and is preferred if the specimens are being used for freeze-thaw studies.

The fundamental transverse vibration of a specimen has only two nodal points, at distances from each end of 0.224 times the length. The amplitude is maximum at the ends, about three fifths of the maximum at the centre, and zero at the nodal points. Therefore, movement of the pick-up along the length of the specimen will show whether it is vibrating at its fundamental frequency.

For fundamental longitudinal and torsional vibration, there is a point of zero vibration (node) at the mid-point of the specimen and maximum amplitude at the ends.

The fundamental frequency should be calculated from the formulae in Appendix II and frequencies within $\pm$ 15 per cent of this should be investigated. The equations given in the Appendix are modifications of equations applicable to specimens which are very long in relation to their cross section. The modifications of the equations have been developed and checked by Pickett (29) and Spinner and Tefft (30). It should be remembered that the dynamic modulus of elasticity may range from 2×10^6 psi for low-quality concretes at early ages to 7×10^6 psi for good quality concrete at later ages (31).

Sometime in the sonic testing of concrete, two peaks may appear close together in the frequency response curve. Kesler and Higuchi (32) have referred to it as the double hump phenomenon and believe this to be caused by a non-symmetrical shape of specimen which causes interference with vibration of the specimen in some direction other than that intended. Proper choice of specimen size and shape should practically eliminate this problem; for example in a specimen of rectangular cross section the above problem can be eliminated by vibrating the specimen in the direction parallel to the short side.

The approximate range of fundamental longitudinal and flexural resonant frequencies of standard concrete specimen given by Jones and Gatfield (33) are as follows:

Size of Specimen, inches	Approximate Range of Transverse Resonant Frequency, c/s	Approximate Range of Longitudinal Resonant Frequency, c/s
28 × 6 × 6 beam	550—1150	1800—3200
20 × 4 × 4 beam	900—1600	2500—4500
12 × 6 cylinder	2500—4500	4000—7500

Calculation of Moduli of Elasticity and Rigidity, and Poisson's Ratio

The dynamic moduli of elasticity and rigidity and the Poisson's ratio can be calculated by equations given in ASTM Designation C 215-60. These equations are given in Appendix II.

It is stressed that the corrections to the theoretical equations in all cases involve Poisson's ratio. These corrections are considerably greater for transverse resonant frequency than for longitudinal resonant frequency. For example, a standard 4 × 4 × 20-inch prism requires a correction factor of about 27 per cent at the fundamental transverse resonance, as compared with less than one half per cent at the fundamental longitudinal resonance (34, 35).

Effect of Curing Conditions on Dynamic Modulus of Elasticity

Obert and Duvall (24) have shown that, although the dynamic modulus of elasticity depends on the moisture content, the change in the modulus with drying is rather small after about 3 or 4 days of air drying. Whereas a sufficiently accurate comparison could be obtained with a saturated specimen, it was considered that a better time for testing was after 3 or 4 days of air drying.

Further, it has been shown that a large decrease in the dynamic modulus of elasticity occurs over the first 48 hours of oven drying but the subsequent change is small. Oven drying, even at as low a temperature as 100°F, causes an irreversible reduction of the modulus, the reasons for which are unknown. A possible explanation is that even this low temperature might liberate some of the combined water and thus affect the value of the dynamic modulus of elasticity. This test, therefore, might not give a valid result in studies involving repeated wetting and drying of the specimen.

Kesler and Higuchi (36), in their studies, have concluded:

1. For the same curing conditions, the dynamic modulus of elasticity increases as the strength increases.
2. If the concrete is kept moist the modulus of elasticity increases with age, and if the concrete is allowed to dry the modulus of elasticity decreases with age.

Reproducibility of Test Results

There are limited data available on the reproducibility of test results of dynamic Young's modulus of elasticity. Jones (35) has published data which indicate that for standard-size beams and cylinders the reproducibility of dynamic Young's modulus is very much superior to that obtained in static tests (Table 1).

Table 1
Comparison of Reproducibility of the Standard Methods of Measuring Static and Dynamic Young's Modulus of Elasticity*

Size of Specimen	Young's Modulus	No. of Specimens	Standard Error of 3 Results (lb/sq. in. × 10^6)
12 × 6-inch cylinders	Static	3	0.093
12 × 3 × 3-inch beams	Static	3	0.146
28 × 6 × 6-inch beams	Dynamic	3	0.039
20 × 4 × 4-inch beams	Dynamic	3	0.051
12 × 3 × 3-inch beams	Dynamic	3	0.054

*After Jones (35).

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According to Jones, the greater variability of the results of the static modulus is due to greater errors in the testing rather than to greater variability between specimens; on the other hand, each of the measurements in the resonance method, *i.e.,* resonant frequency, length and density, can be measured to a high order of accuracy.

Resonant Frequency and Durability of Concrete

The determination of flexural resonance has been employed with considerable advantage in North America to study the effects of successive accelerated freezing and thawing cycles on concrete specimens. The advantages of this method are:

(*a*) The repeated tests can be carried out on the same specimen over a very long period, and the number of test specimens to be cast is therefore considerably reduced.

(*b*) The results obtained with flexural resonance methods are more reproducible than those obtained with destructive types of tests (35).

Extensive studies of changes in dynamic modulus of elasticity with the deterioration of concrete subjected to freezing and thawing have been reported by Hornibrook (22), Thompson (23), Willis and de Reus (23), Long and Kurtz (37), and Axon, Willis and Reagel (38). Results of one such study are shown in Figure 17. The ASTM has accepted resonance frequency methods* for studying the deterioration of concrete specimens subjected to repeated cycles of freezing and thawing. These methods require the calculation of the relative dynamic modulus of elasticity and durability factor. The computations involved are given in Appendix III.

Correlation between Dynamic Modulus of Elasticity and Strength Properties of Concrete

Several investigators (36-46) have attempted to establish relations between the dynamic modulus of elasticity and strength of concrete. Some of these correlations appear to hold for the particular type of concrete investigated, but it is doubtful that any generalized relationships can be given. It is therefore considered that if the flexural or compressive strengths of concrete are to be estimated from the dynamic modulus of elasticity, it is most essential first to establish an experimental relationship between these parameters and the dynamic modulus of elasticity.

The following statement by Jones (35) best sums up the state of the art as regards the relationship between the dynamic modulus of elasticity and the strength of concrete:

> In spite of some of the promising results of the early investigations, it must be concluded that no general relationship exists between the dynamic modulus of concrete and its flexural or compressive strength. Nevertheless, limited correlations are obtained when the changes in the dynamic modulus and strength are produced by changes in the age of the concrete, the degree of compaction, the water/cement ratio, or by deterioration.

*ASTM Standards 290-61T and 291-61T.

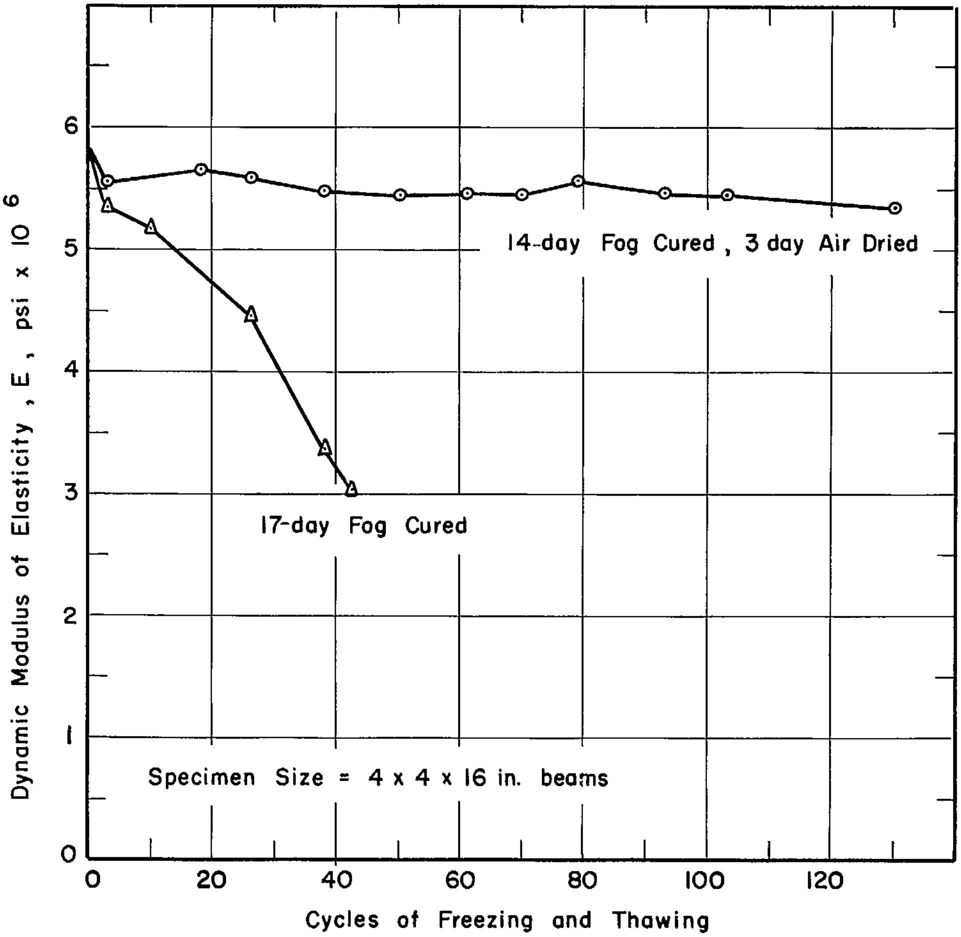

FIGURE 17. Effect of cycles of freezing and thawing on dynamic Young's modulus of elasticity. After Long and Kurtz (37).

Notwithstanding the above limitations and the statement of Jones, relationships between the strength parameters and the dynamic modulus of elasticity have been reported by various researchers, and are given in Appendix IV. Two such relationships are illustrated in Figures 18 and 19.

Comparison of Moduli of Elasticity Determined from Longitudinal and Transverse Frequencies

In normal routine calculation of dynamic modulus, only the transverse frequencies are determined. However, Batchelder and Lewis (47) have shown that excellent correlation exists between the moduli calculated from the transverse and longitudinal frequencies (Figure 20).

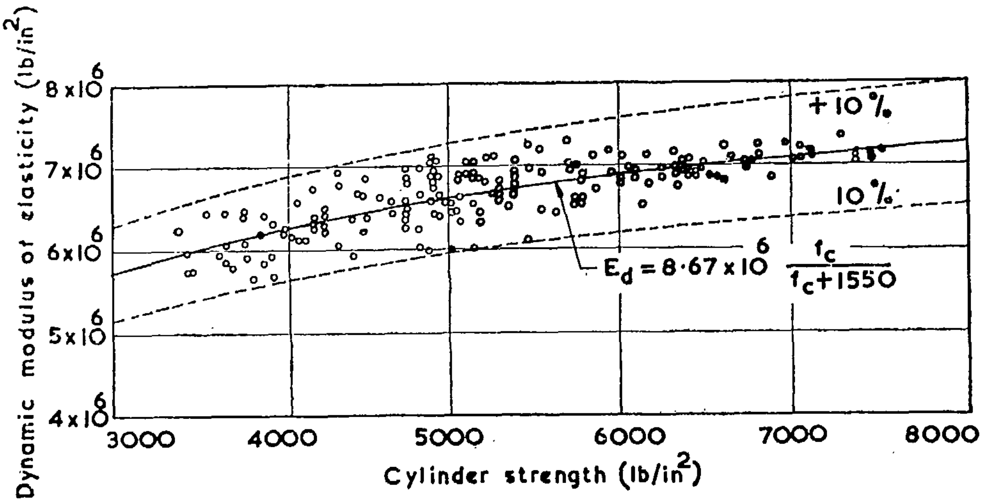

FIGURE 18. Relationship between dynamic modulus of elasticity and compressive strength. After Sharma and Gupta (44).

Jones (34, 35) in his studies found that for wet concretes there was no appreciable difference in the dynamic Young's modulus of elasticity determined from the transverse and longitudinal modes of vibration. However, when the beams were allowed to dry, the Young's modulus calculated from the transverse vibrations was lower than that calculated from longitudinal vibrations. This was attributed to the moisture within the concrete beam.

Comparison of Dynamic and Static Moduli of Elasticity

A considerable amount of work has been carried out by various researchers to establish the relationship between dynamic Young's modulus and static modulus obtained from conventional stress-strain methods conducted at low rates of loading. The following observations may be made from the work reported by Powers (21), Stanton (25), Witte and Price (25), Philleo (28), Sharma and Gupta (44), Whitehurst (48), and Klieger (49):

(*a*) The dynamic modulus of elasticity is generally somewhat higher than the static modulus; the difference depends upon the degree of precautions taken during the conduct of the experiments and the applications of the correction factors allowed for in the equations for the computations of the dynamic modulus.

(*b*) As the age of the specimen increases, the ratio of static modulus to dynamic modulus also increases and more nearly approaches 1.0.*

*On the basis of tests of 2-year-old specimens reported by Witte and Price, the static modulus in compression was equivalent to 89 per cent of the dynamic modulus, while the static modulus in flexure was equal to 88 per cent of the dynamic modulus. When the tests were repeated after the specimens were 3 years old, these values were found to be 96 to 87 per cent respectively.

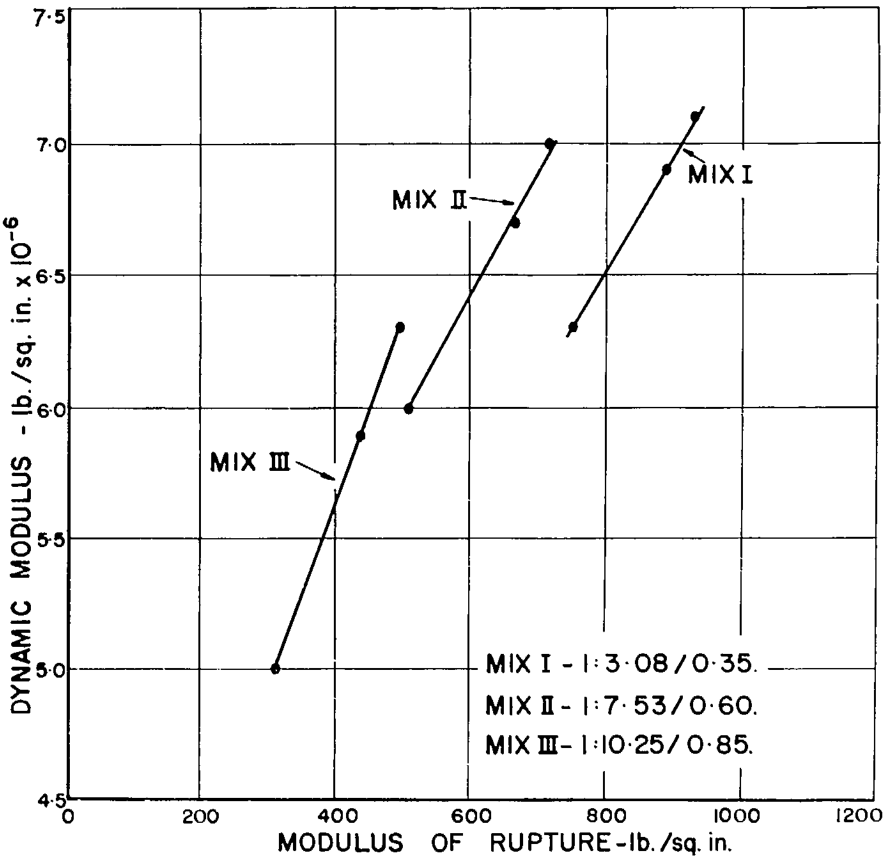

FIGURE 19. Relationship between dynamic modulus of elasticity and flexural strength. After Kaplan (46).

(*c*) For higher static moduli of elasticity, the values for both dynamic and static moduli of elasticity show close agreement.

Figure 21 shows the relationship between static and dynamic moduli for high-strength concretes (44).

Damping Properties of Concrete

Thompson (23) and Obert and Duvall (24) have shown that the quality of concrete can be determined from its damping ability. The measures of damping ability are the logarithmic decrement and the damping constant. These parameters are defined in Appendix V, where the equations for their calculations are also given.

The use of the damping properties of concrete has found little acceptance and there are very few laboratories which carry out these tests as a matter of routine.

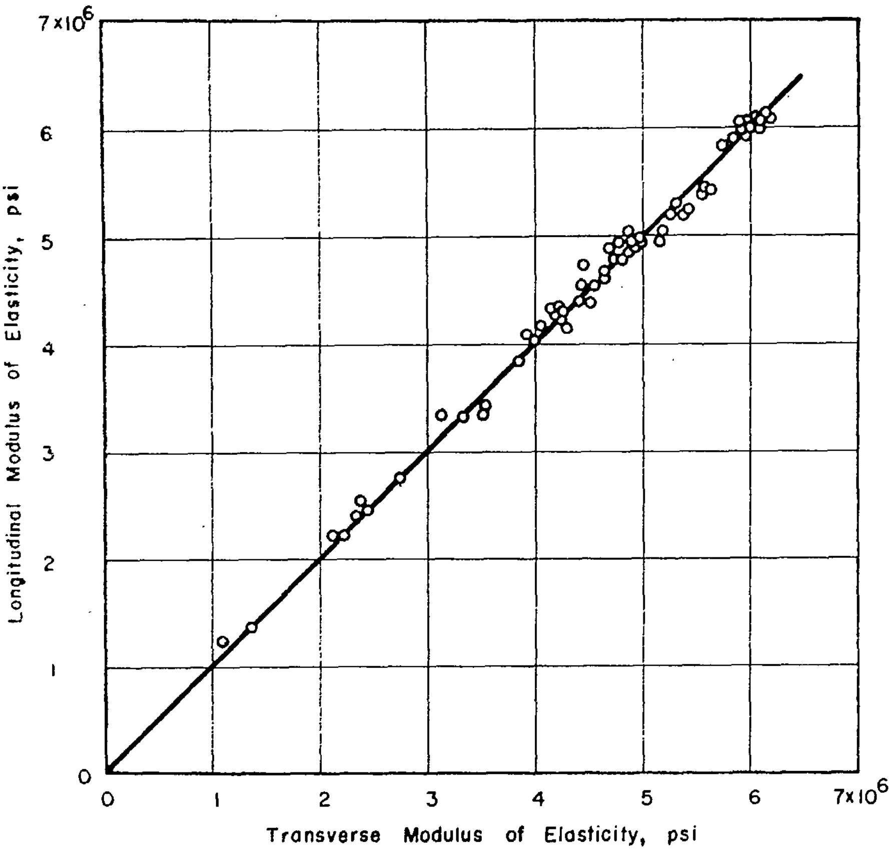

FIGURE 20. Comparison of moduli of elasticity determined from longitudinal and transverse frequencies. After Batchelder and Lewis (47).

LIMITS AND USEFULNESS OF RESONANT FREQUENCY TEST METHODS

Though the basic equipment and testing procedures associated with the resonant frequency techniques have been standardized and commercial units are available, the usefulness of the tests is seriously limited because they can normally only be carried out on laboratory-size specimens.

These methods provide an excellent means for studying the deterioration of concrete specimens subjected to repeated cycles of freezing and thawing, and their use for these studies is recommended.

The resonant frequency test results may be used to calculate dynamic Young's modulus of elasticity of concrete, but the values obtained are somewhat higher than those obtained with standard static tests carried out at lower rates of loading.

Various investigators have published correlations between the strength of

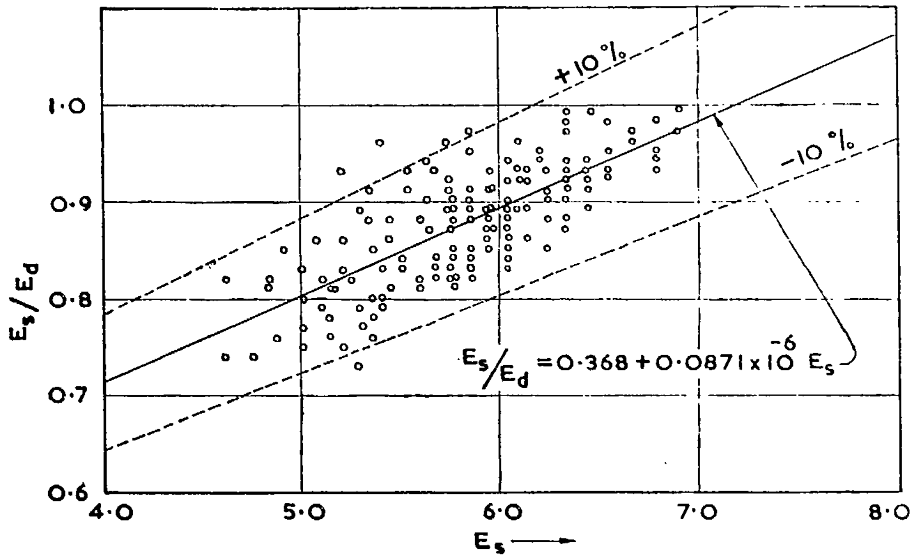

FIGURE 21. Relationship between static modulus of elasticity and ratio of static to dynamic moduli. After Sharma and Gupta (44).

concrete and its dynamic modulus of elasticity. The use of such correlations to predict compressive and/or flexural strength of concrete is strongly discouraged unless similar relationships have been established in the laboratory for the concrete under investigation.

MECHANICAL SONIC PULSE VELOCITY METHODS

Single-Blow Pulse Method

The first detailed application of pulse methods to the testing of concrete *in situ* was reported by Long, Kurtz and Sandenaw (39) in 1945. The basic principle of the method is that a longitudinal or compressional wave is initiated by a single hammer blow, and the time it takes to travel between pick-ups standing on the surface of the concrete is measured.

A schematic diagram of the single-impact test apparatus is shown in Figure 22, and its operation is briefly described below:

An impact is applied in a horizontal direction in a line with two pick-ups. The measurement of the time interval is achieved electronically by an instrument, known as an "interval timer", which is operated from electrical signals produced by the mechanical pulse at each pick-up. The electrical pulse from the first pick-up, after amplification, triggers a thyratron valve which starts an electrical current flowing through a triode and a ballastic galvanometer; the signal from the second pick-up triggers a second thyratron valve to stop the flow of current to the galvanometer. The galvanometer deflection is, therefore, proportional to the time

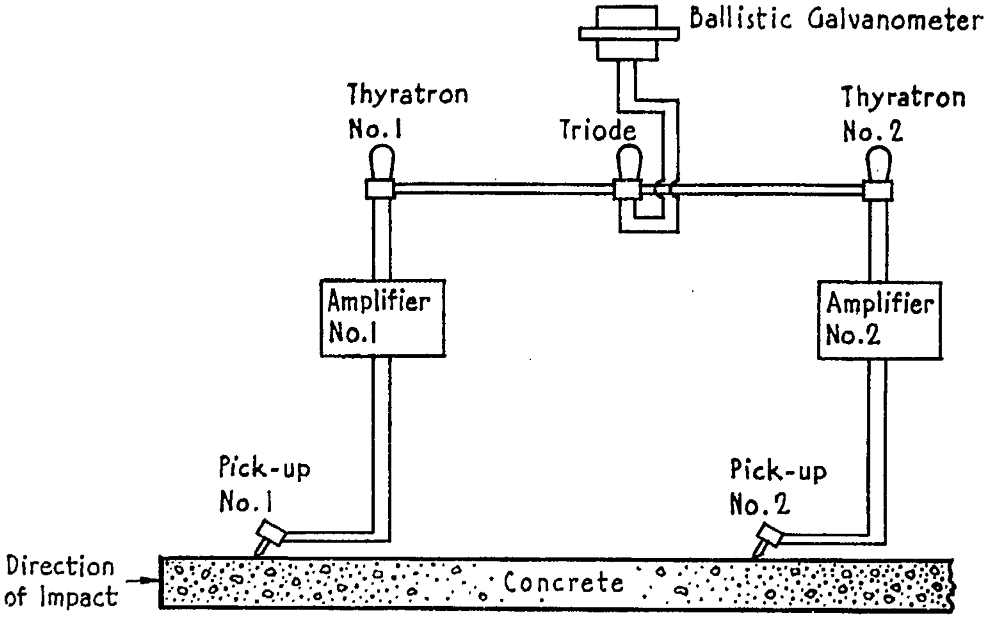

FIGURE 22. Schematic diagram of velocity test apparatus—single blow hammer method. After Orchard (31).

taken for the impulse to pass between the two pick-ups. The impulse velocity is calculated from an equation involving current through the galvanometer, the distance between the pick-ups, the galvanometer constant, and the deflection of the galvanometer.

The results of Long, Kurtz and Sandenaw show a good correlation between flexural strength values and the pulse modulus* determined by the "interval timer."

Mitchell (50) in the United States and Andersen and Nerenst (51) in Denmark, have done considerable work in the use of the single-blow pulse method.

Sources of Error in Single-Blow Pulse Method

In spite of the good correlation between flexural strengths and the pulse modulus, reported by Long, Kurtz and Sandenaw, there are a number of possible sources of error in this method. This may explain the relative lack of popularity of this technique. Some of the sources of errors are:

(*a*) Errors are likely to be introduced because of the assumed value of the Poisson's ratio.

*Modulus of elasticity when obtained using pulse velocity techniques is called the "pulse modulus"; the method for its calculation is given later.

(*b*) The "interval timer" used in the apparatus described above measures the interval between the attainment of specific voltages of each pick-up. The time interval measured may therefore be greater than that taken up by the initial onset of the pulse in travelling between the two pick-ups.

(*c*) The measurement of the travel time may be affected by the intensity and direction of the hammer blow.

(*d*) The method only measures the surface condition of concrete *in situ* and not of the structure as a whole.

(*e*) There is a possible reduction in the amplitude of the pulse as it travels through the concrete.

Repetitive-Blows Pulse Method

Because of the inherent time measurement errors associated with the single-impact devices, experimental work was undertaken in several countries to improve the precision of these measurements. As a result, a new technique was developed in France (52).

In this method, a pulse is produced five times per second by a mechanical hammer. The pulses are received by a single magnetostrictive pick-up, a nickel rod biased by a magnet, and the electrical pulses are displaced on a cathode-ray oscilloscope. The time measurement, which is one of the major sources of error in the single-impact hammer method, is very precise in the current ultrasonic devices.

Figure 23 shows the repetitive-pulse equipment. This method appears to have been used with great success in the Paris region of France, where correlations between the pulse velocity and the strength parameters of concrete have been established.

ULTRASONIC PULSE VELOCITY METHOD

This method was developed in Canada in 1945 by Leslie and Cheesman (53) and at about the same time in England by Jones (34, 35, 54). The Canadian studies, carried out in Toronto at the Hydro-Electric Power Commission of Ontario, were aimed at developing a nondestructive method to examine concrete in dams ranging in thickness up to 40 feet. These studies resulted in the development of an instrument known as the Soniscope. Since then a considerable amount of work has been reported on the use of this instrument, both in Canada (53, 55-58) and in the United States (28, 47, 48, 49, 58-65).

The purpose of the research work carried out at the Road Research Laboratory, England, was to develop a technique for testing laboratory specimens. That work led to the development of an instrument known as the Ultrasonic Concrete Tester. The development and use of this instrument have been reported in great detail by Jones (35, 66), Jones and Gatfield (33) and Kaplan (67, 68).

Several reports have been published in Russia on the use of vibration and ultrasonic techniques. Whitehurst (69) mentions one such report, which describes

FIGURE 23. Repetitive-blows pulse method for testing concrete. After Jones (35).

the design and operation of an ultrasonic testing apparatus in connection with the construction of the Moscow Ring Road.

Figures 24 and 25 show the latest models of the Soniscope and Ultrasonic Concrete Tester.

Basic Principle

The ultrasonic pulse velocity method consists of measuring the time of travel of an ultrasonic wave passing through the concrete to be tested. An electrical impulse from a central unit is transmitted to a sending transducer where it excites a block of crystals. The transducer, through the block, emits an ultrasonic pulse which travels through the concrete under test to the receiving transducer. Here the ultrasonic pulse is converted back into an electric impulse which is then displayed on the face of a cathode-ray oscilloscope. The time of travel between the initial onset and the reception of the pulse is measured electronically. The path length between transducers, divided by the time of travel, gives the average velocity of wave propagation.

Design Features of Soniscope and Ultrasonic Concrete Tester

The fundamental design features of both the Soniscope and the Ultrasonic Concrete Tester are very similar. Both instruments consist of a pulse generator and a pulse receiver. In the Soniscope, pulses are generated by Rochelle salt crystals, and similar crystals are used in the pulse receiver; in the Ultrasonic Concrete

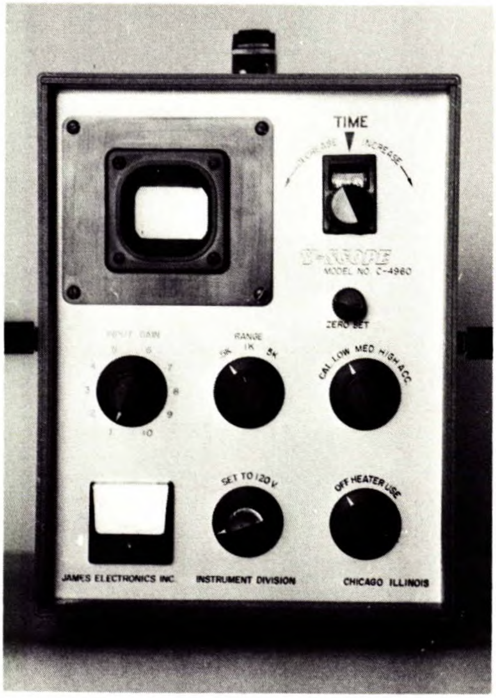

FIGURE 24
A view of the V-scope and two transducers. (*Courtesy of James Electronics Inc., Chicago.*)

Tester the pulses are produced by shock-exciting piezo-quartz crystals, and similar or more excitable Rochelle crystals are used in the receiver. The time taken for the pulse to pass through the concrete is measured by electronic measuring circuits that, though different in detail in the two instruments, perform essentially the same function.

One difference between the two types of equipment is that the resonant frequency of the transducers used with the Ultrasonic Concrete Tester is considerably higher (about 150 kc) than that of the transducers used with the Soniscope (about 20 kc). As a result of the difference in the frequency of the transducers, the Ultrasonic Concrete Tester has a testing range of about 7 feet whereas the Soniscope can be used to test concretes ranging in thickness up to 75 feet.

The time for the pulse to travel between the two transducers is measured on a cathode ray oscilloscope by placing the transducers on opposite faces of the concrete. The vibration of the transducers is transferred across each transducer-concrete interface by a coupling medium such as a thin film of oil, soap, jelly, or kaolin-glycerol paste. If concrete surfaces are very rough these are ground smooth or filled in with a thin coating of plaster.

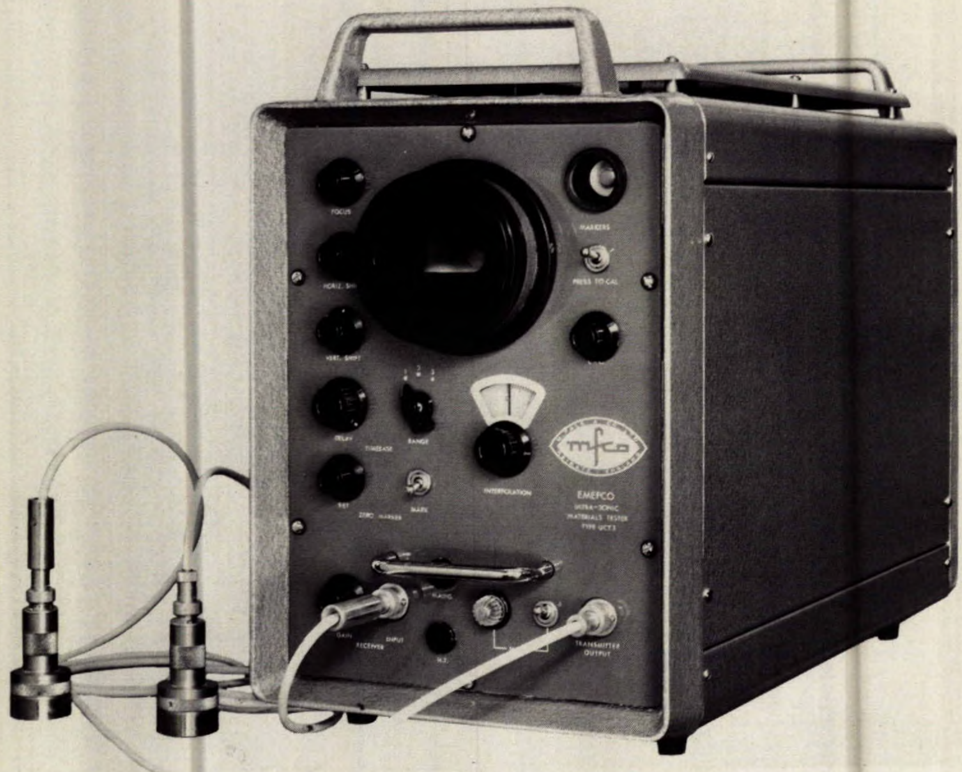

FIGURE 25. Ultrasonic concrete tester. The two 1-inch transducers and a test beam can be seen on the left. (*Courtesy of M. Falk & Co. Ltd., England.*)

Accuracy of Measurement

It is generally agreed that the Ultrasonic Concrete Tester can measure the transit time through small specimens with an accuracy of 0.1 microsecond (0.5 microsecond for the Soniscope using small concrete specimens).

Reproducibility of Test Results

Few published data are available regarding the reproducibility of the test results obtained with the Ultrasonic Concrete Tester. Mather (62) has concluded that the reproducibility of results obtained with normal uncracked concrete using various operators and Soniscope is within 1 per cent.

Application of Pulse Velocity Methods

The Soniscope and the Ultrasonic Concrete Tester have been used to evaluate concrete structures and attempts have been made to correlate the pulse velocity with strength and other properties of concrete. The various applications of the pulse velocity methods are listed and described below:

(*a*) Establishing uniformity of concrete.
(*b*) Establishing acceptance criteria.
(*c*) Determination of pulse modulus of elasticity.
(*d*) Estimation of strength of concrete.
(*e*) Determination of setting characteristics of concrete.
(*f*) Measurement and detection of cracks.
(*g*) Measurement of deterioration of concrete due to fire exposure.

(*a*) *Establishing Uniformity of Concrete*—For establishing the uniformity of concrete, the Ultrasonic Concrete Tester is an ideal tool for laboratory specimens, whereas the Soniscope provides an excellent means for both laboratory and field studies.

Parker (57), Whitehurst (59), Breuning and Bone (65), and Jones (35, 66) have reported results of very carefully conducted surveys for determining the

FIGURE 26
Field testing of mass concrete with the Soniscope. After Leslie (55).

uniformity of concrete of various types of structures. Usually, if material differences in pulse velocity are found within a structure for no apparent reasons (such as changes in materials, concrete mix or construction procedures), then there is strong reason to presume that some defective or deteriorated concrete is present.

Of many such surveys carried out on existing structures both in Canada and the United States, one that deserves mention is that reported in 1953 by the Hydro-Electric Power Commission of Ontario (57). It was carried out on a dam built in 1914. A total of 50,000 readings were taken, most of them at 1-foot spacing. The pulse velocities measured on the structure ranged from below 5,000 to over 17,000 fps and these values were used, with success, to determine the areas of advanced deterioration. Figure 26 shows field testing of mass concrete with the Soniscope.

(*b*) *Establishing Acceptance Criteria*—Generally, high pulse velocity readings in concrete are indicative of concrete of good quality. Leslie and Cheesman (53) have suggested the following pulse velocity ratings for concrete:

Pulse Velocity, fps	General Condition
Above 15,000	Excellent
12,000 to 15,000	Good
10,000 to 12,000	Questionable
7,000 to 10,000	Poor
Below 7,000	Very poor

Jones (35) maintains that it is not possible to specify a minimum acceptable pulse velocity which is applicable under all conditions and all types of structures. He gives the following minimum velocities acceptable for specific structural purposes in Great Britain:

Type of work	Minimum value of pulse velocity for acceptance, fps
Suspended Floor Slabs	15,500
Prestressed Concrete T Sections	15,000
Prestressed Concrete Anchor Units	14,300
Reinforced Concrete Frame Buildings	13,500

Whitehurst (48), while discussing the usefulness of the above ratings, states:

> It is the author's opinion that all of the generalization made above while satisfactory as generalizations must be used with great caution. It is doubtful that sharp lines of demarcation may be rigorously applied to the categories suggested by Leslie and Cheesman and those suggested by Jones. The degree to which any particular concrete will fall into these categories would depend upon the aggregates in concrete, the mix, the conditions of curing and the exposure subsequent to curing. The investigator is thus advised to acquire all possible information concerning the concrete to be evaluated before attempting to interpret pulse velocity tests.

(*c*) *Determination of Pulse Modulus of Elasticity*—Theoretically, the values of the pulse modulus of elasticity calculated from the readings obtained with the Soniscope or the Ultrasonic Concrete Tester should be the same as those obtained with resonant frequency techniques. However, this has not been found to be so.

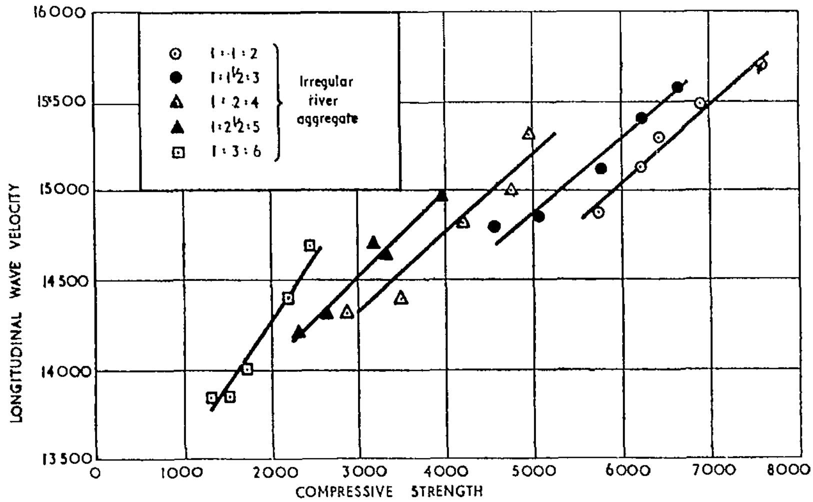

FIGURE 27. Effect of aggregate/cement ratio on the relationship between pulse velocity and cube compressive strength. After Jones and Gatfield (33).

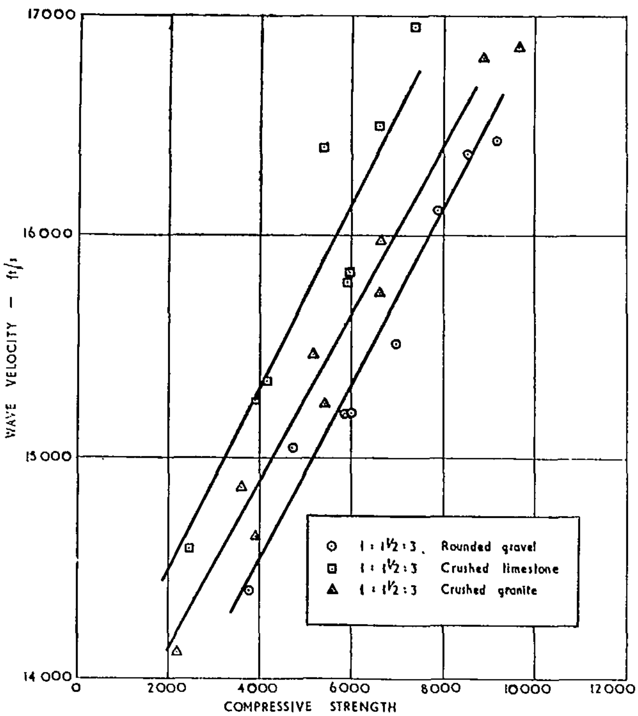

FIGURE 28
Effect of type of aggregate on the relationship between pulse velocity and cube compressive strength. After Jones and Gatfield (33).

For this reason and also because the modulus of elasticity depends upon density and Poisson's ratio, most researchers have attempted to use pulse-velocity itself as a criterion of the quality of concrete without attempting to calculate moduli therefrom. If it is desired to compute modulus of elasticity from the pulse velocity, the formulas given in Appendix VI should be used.

(*d*) *Estimation of Strength of Concrete*—Various researchers have attempted to correlate compressive and flexural strengths of concrete with pulse velocity. Notable among these are Jones and Gatfield (33), and Kaplan (67, 68).

Jones (35) has shown that reasonably good correlation can be obtained between the cube compressive strength and pulse velocity, provided the aggregates and mix proportions are kept constant. Effects of aggregate-cement ratio and the type of aggregates on the above relationships are shown in Figures 27 and 28. The relationships appear to be independent of the water/cement ratio.

Jones and Gatfield (33) have also illustrated the relationship between the pulse velocity and the flexural strength of concrete. The results, (Figure 29), show that the relations, though independent of the water/cement ratio and the aggregate type, are dependent upon the aggregate-cement ratio.

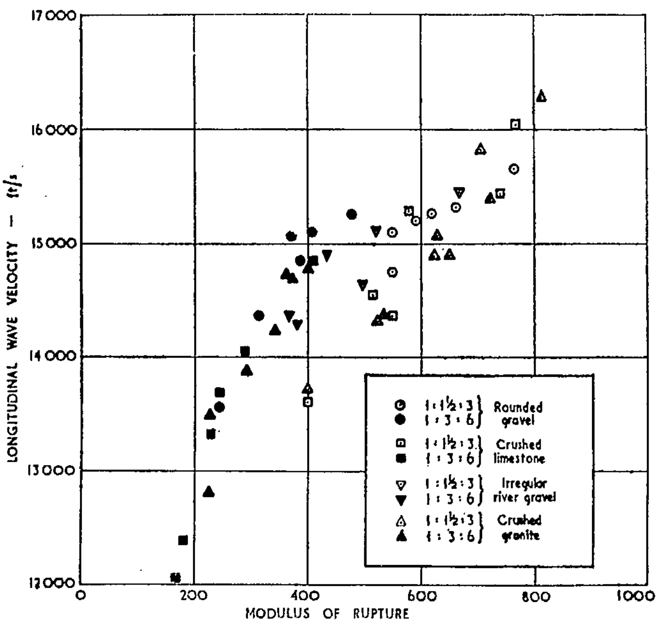

FIGURE 29
Relationship between pulse velocity and modulus of rupture of concrete beams. After Jones and Gatfield (33).

According to Jones (35), some researchers have established relations between pulse-velocity and compressive strength. These relations enable the strength of structural concrete to be predicted within $\pm$ 20 per cent. To obtain this accuracy, allowances must be made for the type of cement, mix-proportions and curing conditions in the structural concrete.

Table 2

Some Typical Values for Pulse Velocity for Concrete made with Different Aggregates*

Type of Concrete Aggregate	Water/Cement Ratio by Weight	Cement, lb/cu yd	Slump, inches	Air, per cent	Unit Weight of Fresh Concrete lb/cu ft	Age of Test, days	Compressive** Strength, psi	Flexural** Strength, psi	Pulse Velocity, ft/sec
Coarse: Expanded Shale.. Fine: Expanded Shale......	0.72	480	13/4	5.0	94.3	120	2110	380	11,238
Coarse: Phonolite............. Fine: Phonolite.................	0.72	479	1	1.2	141.5	118	2640	565	15,804
Coarse: Anorthosite.......... Fine: Anorthosite.............	0.71	482	1½	3.1	148.0	180	2720	515	14,612
Coarse: Ilmenite............... Fine: Ilmenite....................	0.71	471	1	0.7	234.9	95	5960	760	16,077

*After Zoldners, Malhotra and Wilson (70).

**Each result is an average of 15 tests.

Some typical values for pulse velocity for aluminous cement concrete made with different aggregates are given in Table 2. These vary from 11,238 ft/sec for lightweight (expanded shale) concrete to 16,077 ft/sec for heavyweight (ilmenite) concrete. The data in Table 2 clearly bring out the fact that no attempts should be made to predict compressive and/or flexural strength of concrete from pulse velocity values unless similar relations have been previously established for the type of concrete under investigation.

(*e*) *Determination of the Setting Characteristics of Concrete*—The determination of the rate of setting of concrete by means of the Soniscope has been reported by Cheesman (56), Whitehurst (64), and other researchers.

Whitehurst has reported results of tests on 4 x 4 x 16-inch concrete prisms, using various types of cements. The concretes used had zero slump and immediately after casting the end plates of the forms were removed. Pulse velocity tests using the Soniscope were made periodically, from shortly after the specimens were cast until 8 hours or more had elapsed.

Initial velocities of the order of 4,000 fps were observed and during the first few hours the velocities increased at a rapid rate. After a period of time varying from 4½ to 8½ hours, the rate of increase suddenly changed, and continued at a much slower pace. The point at which this occurred was taken as the time of set of concrete.

The results of Whitehurst (64), together with those reported by Cheesman (56), have been reported in Figure 30.

All investigators have reported considerable difficulty in measuring the pulse velocity through concrete at early ages. This perhaps explains the very limited use of pulse velocity techniques in this type of work.

(*f*) *Measurement and Detection of Cracks*—This use of pulse velocity tech-

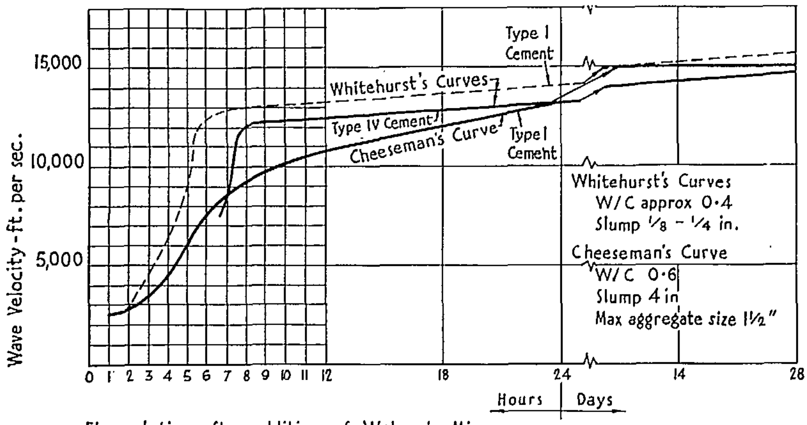

FIGURE 30. Relationship between pulse velocity and setting characteristics of concrete. After Orchard (31).

niques has been described by Leslie and Cheesman (53), Jones (35), and Sturrup (58). The basic principle of crack detection is as follows:

If a crack is of appreciable width and is of considerable depth perpendicular to the test path, the path of the pulse will be blocked and no signal will be received at the receiving transducer. If the depth of the crack is small compared to the distance between the transducers, *i.e.*, the path length, the pulse will pass around the end of the crack and a signal will be received at the transducer. However, in doing so it will have travelled a distance longer than the straight line path upon which the pulse velocity computations are based. The resulting calculated pulse velocity will then be low in comparison with that through uncracked concrete in the same vicinity. The difference in the pulse velocity is then used to estimate the path length and hence the crack depth. It is, of course, assumed that the crack is not filled with water.

This principle has been used by the Hydro-Electric Power Commission of Ontario to determine internal as well as surface cracks (53). According to Sturrup (58), the pulse velocity method differentiates between deep and shallow cracks and will detect internal cracks large enough to cause a significant increase in the transmission time or an abnormal reduction in the pulse amplitude.

Jones (35, 71) has also described in detail studies for crack detections by testing specimens which were being subjected to compressive or tensile stresses.

(*g*) *Measurement of Deterioration of Concrete Due to Fire Exposure*—Zoldners, Malhotra and Wilson (70) have used pulse velocity techniques to measure deterioration due to fire exposure in 3½ x 4 x 16-inch concrete prism specimens. In their investigation, they had exposed concrete prism specimens to fire exposure for one hour at temperatures ranging from 100 to 1,000°C. After the exposure, the specimens were removed from the furnace and allowed to cool to room temperature. Pulse velocity was then measured using the ultrasonic concrete tester;

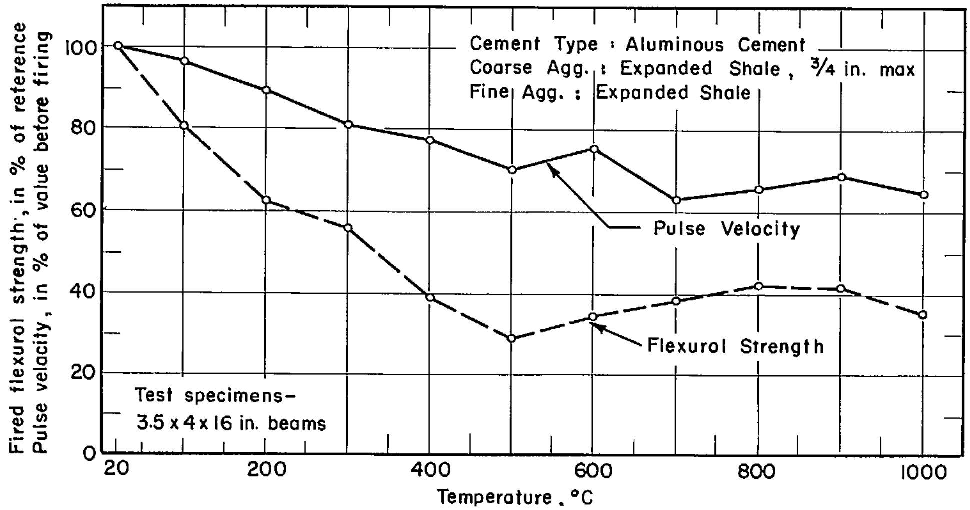

FIGURE 31. Loss in pulse velocity and flexural strength of concrete prism specimens after exposure to temperature to 1,000°C. After Zoldners, Malhotra and Wilson (70).

following this the prisms were tested in flexure. Figure 31 shows results of one such investigation. It will be seen that the deterioration in the prism specimens can be determined by measuring the per cent loss in pulse velocity. Furthermore, the per cent loss in pulse velocity followed very closely the per cent loss in flexural strength of test prisms after fire exposure.

Comparison of Pulse and Static Young's Moduli of Elasticity

Published data comparing pulse and static Young's moduli are sparse. The data reported by Philleo (Figure 32) indicate that the pulse modulus was invariably higher than the static modulus. For 6 x 12-inch cylinders, the ratio of the two ranges from 1.54 to 1.10 as the static modulus varies from 3.92 to 5.50 x 10^6 psi; for 6 x 6 x 30-inch beams, the ratio varies from 1.17 to 1.01 as the static modulus increases from 4.52 to 6.03 x 10^6 psi.

Comparison of Pulse and Dynamic Moduli of Elasticity

The comparison between the two moduli has been reported by Leslie and Cheesman (53) and Philleo (28).

In their original work on the development of the Soniscope, Leslie and Cheesman (53) reported an excellent correlation between the pulse and dynamic moduli of elasticity. The reported pulse modulus averaged about 8 per cent higher than the dynamic modulus. This was based on experiments on three hundred 3½ x 4 x 16-inch prisms.

Philleo (28), reporting on the work carried out at Portland Cement Association, indicated that the pulse moduli were 1 to 47 per cent higher than the dynamic moduli, averaging 15.4 per cent higher. These results were based on the analysis of

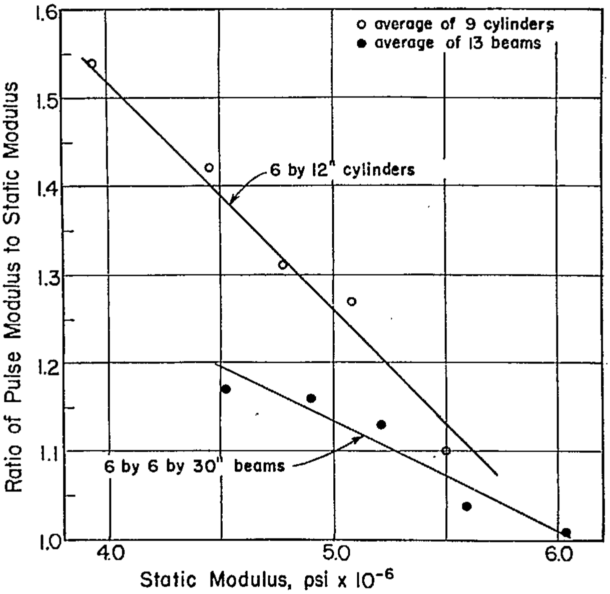

FIGURE 32
Relationship between pulse and static Young's moduli of elasticity. After Philleo (28).
Note: Age at test—28 days
Cement content—3, 4.5 and 6 sacks per cu. yd.
Cylinders—Initial tangent modulus
Beams—Secant modulus at 15 per cent ultimate

one hundred and seventy 6 x 6 x 30-inch beams and the dynamic encountered ranged from 4.75 to 8.0 x 10^6 psi.

LIMITS AND USEFULNESS OF PULSE VELOCITY TEST METHODS

Although ultrasonic pulse velocity equipment is available from several commercial sources, the testing procedures have yet to be standardized. The ASTM has recently issued a tentative standard.*

Ultrasonic pulse velocity techniques provide an excellent means for establishing uniformity of concrete and deserve a definite place in quality control operations.

Within the previously outlined limitations, ultrasonic pulse velocity techniques provide the only available means of delineating both surface and internal cracks in concrete structures.

Ultrasonic pulse velocity tests can be carried out both on laboratory-size test specimens and on completed concrete structures. This fact alone enhances the usefulness of these tests in comparison with resonant frequency tests.

Inasmuch as a large number of variables affect the relations between the strength parameters of concrete and its pulse velocity, *the use of the latter to predict the compressive and/or flexural strengths of concrete is not recommended.*

The equation for the calculation of the pulse modulus of elasticity of concrete involves both its density and its Poisson's ratio. Since the Poisson's ratio of

* ASTM Standard C 597-67T. Test for pulse velocity through concrete.

concrete is seldom known, the use of the pulse modulus is not recommended; instead, the pulse velocity itself should be used as a criterion of the quality of concrete.

RADIOACTIVE METHODS

The use of X-rays and gamma radiography as nondestructive methods for testing some properties of concrete is relatively new. Mullens and Pearson (72) appear to have been the first to employ X-rays for examining concrete to show variations in density and to locate reinforcing bars. The high initial cost and the immobility of the testing equipment in the field were the main limitations of the method.

The first use of gamma radiography appears to have been made by Smith and Whiffin (73) in 1952 and Fackler (74) in 1954. Between 1954 and 1958, Whiffin (75) and Forrester (76, 77) reported an experimental technique to provide a practical method for testing structural concrete and precast products to determine the position and condition of the reinforcement and also the condition of the concrete. More recently, Preiss (78, 79) and Harland (80) have reported work on measurement of concrete density and thickness by gamma ray transmission.

PRINCIPLES OF GAMMA RADIOGRAPHY

The basic principles of gamma radiography are simple. When concrete is placed in the path of radiation emitted from a gamma source, the radiation is partly absorbed, partly scattered, and partly transmitted. Methods have been developed to measure the density of concrete by measuring the absorbed radiation and also by measuring the amount of back-scatter. The radiation that passes through concrete has been used to measure its thickness.

APPLICATIONS OF GAMMA RADIOGRAPHY

A. Measurement of Density by the Absorption of Gamma Rays

The three methods using this approach are described below:

Smith and Whiffin Method

The general arrangement for direct measurement is shown in Figure 33. A modification of this arrangement was used by Smith and Whiffin (73) to measure the variation of density with depth from the surface of a concrete slab compacted by an experimental surface-vibrating machine. Vertical holes of 2-inch diameter were drilled at intervals of 1 foot along the middle of a 4-foot-wide slab, and a source of 130 millicuries of radio-cobalt* was lowered by progressive increments into each hole. Geiger-Müller counters were located at the same height as the source on each vertical face of the slab. To achieve narrow beam conditions, *i.e.,* to record only the radiation coming directly from the source, the Geiger-Müller

*200 milligrams of radium=130 millicuries of radio-cobalt with regard to gamma ray emission. The radioactive source was machined as an approximate cylinder, 14 mm long by 14 mm diameter, enclosed in a thin-walled cylindrical brass housing.

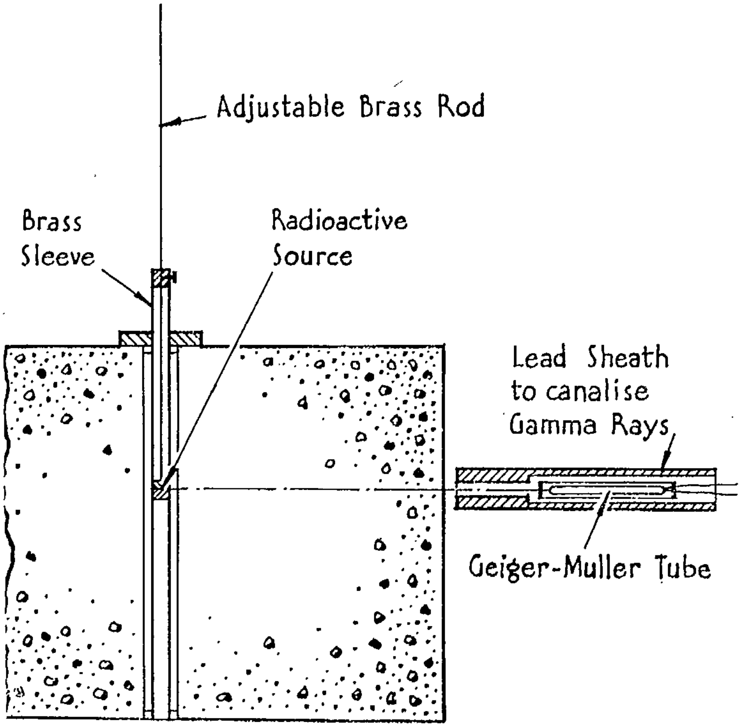

FIGURE 33. General arrangement for measuring density of concrete by means of gamma rays. After Smith and Whiffin (73).

counters were housed in a heavy lead sheath. This sheath was provided with an aperture of 1-inch diameter and 6-inch length, through which the direct radiation could pass.

Calibration of the equipment was effected by drilling cores in the path of the radiation at each location tested and cutting these with a diamond saw into cylindrical sections of known volume, each section being weighed in an oven-dry condition.

Preiss Method

The general arrangement for this method is given in Figure 34. This method has been used to measure the density of concrete of given thickness, or the thickness of concrete of given density.

The gamma radiation source in this method is 5 millicuries of caesium 137 in a lead shield. The radiation passes through a collimating hole in the shield, penetrates the concrete, and is detected by a scintillation counter. The collimating hole in the lead shield is designed so that the radiation beam shines on a

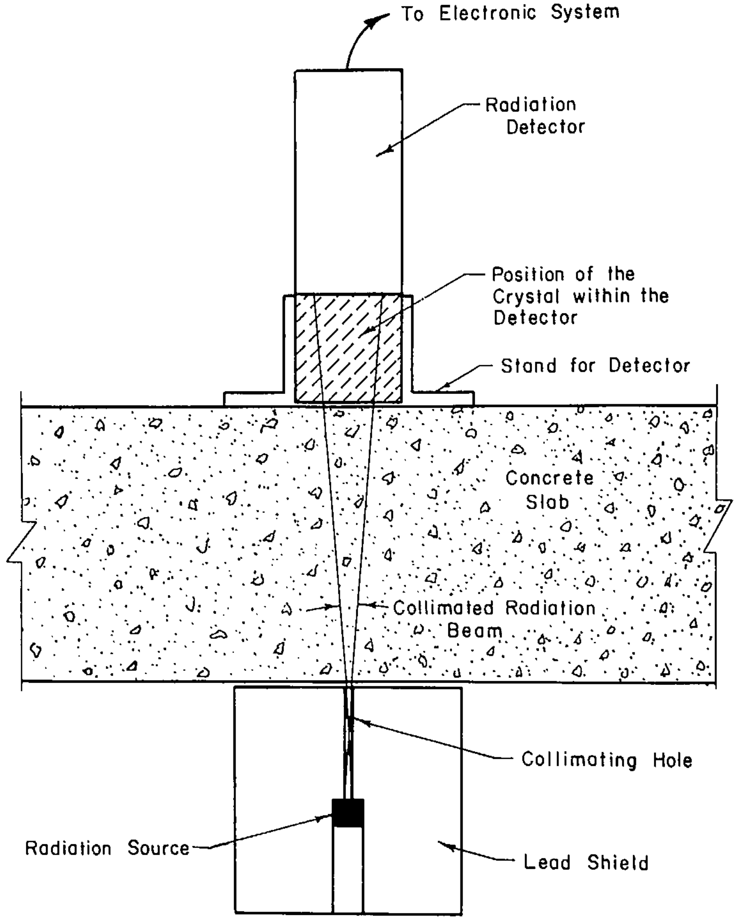

FIGURE 34. Experimental arrangement for measuring the thickness of concrete by gamma rays. After Preiss (79).

limited area of the detector. When taking a reading the detecting device is moved to and fro until a maximum count rate reading is obtained. This count rate is recorded and the density or thickness of concrete is then calculated using the equipment calibration charts.

It is claimed that the amount of scattered radiation detected is not sufficient to upset the method. An accuracy of better than 2 per cent has been claimed for this method in the determination of thickness of slabs that were approximately 4 feet thick.

Harland Method

A general view of the apparatus used in this method is given in Figure 35. This method has been used to measure variations in density in concrete cores,

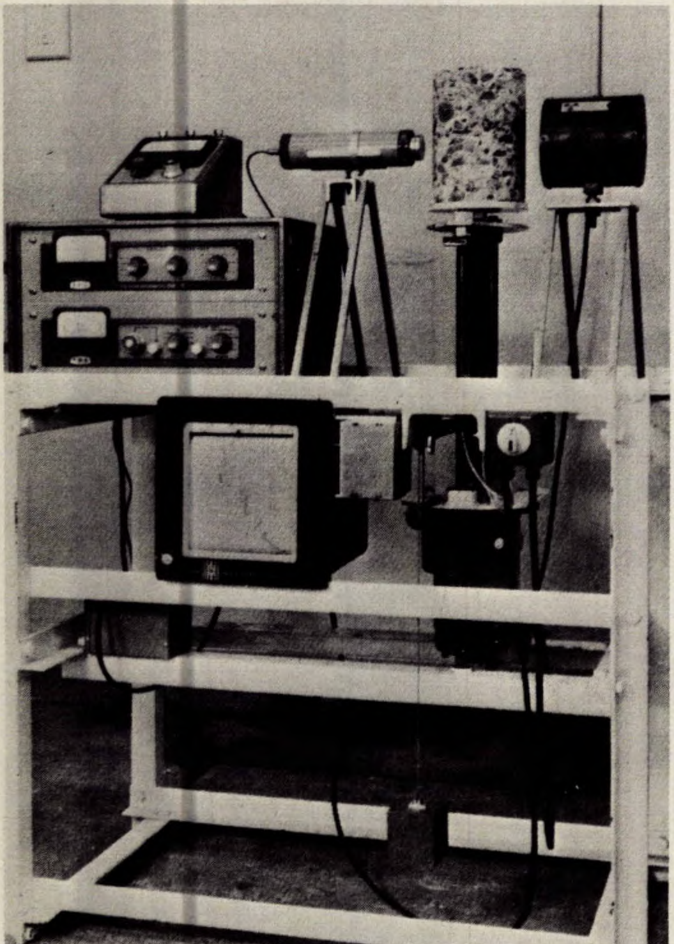

FIGURE 35
A general view of apparatus for measuring variation in density in concrete cores. After Harland (80).

cubes and beams. The variation of density is determined by measuring the change in absorption of gamma radiation (80).

The gamma radiation source is 1 millicurie of caesium 137. The source is sealed in a capsule enclosed by a steel tube which is pushed to the centre of a $5\frac{3}{8}$-inch diameter lead sphere and held securely in position. The lead sphere is supported in such a way that the length of core that may be tested is just under $13\frac{1}{2}$ inches. A scintillation counter acts as a gamma-ray detector. This is connected to a rate meter and chart-recorder which continuously display the intensity of radiation transmitted as the sample traverses along its depth dimension through the gamma beam. The variation in the intensity of the transmitted radiation is translated into density variations by a suitable calculation.

It is claimed that the equipment measures density gradients with a standard deviation of about 1 lb/ft^3.

B. Back-Scatter Method for Determining the Density of Surface Layers of Concrete

An arrangement for the back-scatter method for determining the density of surface layers of concrete is given in Figure 36. Here both the gamma-ray source and the Geiger-Müller counter are close together in a device which is placed on the surface of the concrete under test. In this system the gamma rays propagate through the concrete at an angle to the surface, and the intensity of radiation returning to the surface at a fixed lateral distance from the source is measured.

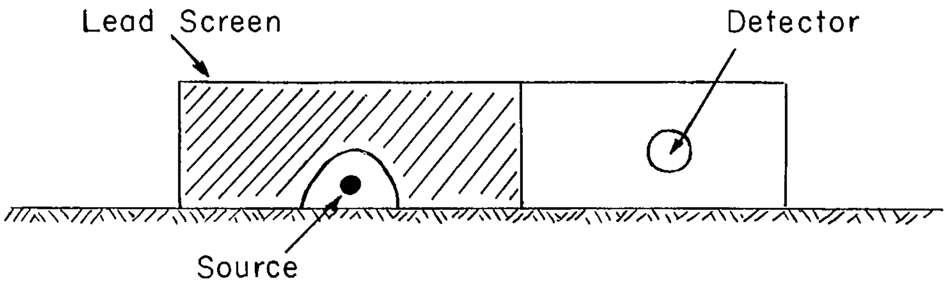

FIGURE 36. Back-scatter method for measuring the density of surface layers of concrete.

This method was originally proposed in the United States as a means of obtaining the density of surface layers of soil (81), and several countries are now using similar techniques to test soil and concrete.

A major disadvantage of this arrangement is that it only measures the density of concrete near the surface; however, a weaker source of radiation can normally be applied than with the direct method.

C. Gamma Radiography for Determining Position and Condition of Reinforcement

The use of gamma radiography to determine position and condition of reinforcement has been reported by Whiffin (75) and Forrester (76). A typical arrangement used by Forrester is given in Figure 37, and the details of the experiment are as follows:

A 170-millicuries source of Cobalt-60 is placed about 18 inches from the front face of concrete member under test and a standard X-ray film is held against the back face. The film is sandwiched between thin lead screens (0.01 inch thick) and these help to intensify the photographic image produced on the film. The gamma radiation passing through the reinforcement is more heavily absorbed than the radiation passing through the concrete so that, on the processed radiograph, the reinforcement is denoted by the lower density parts of the photographic image.

Forrester (77) has also used the gamma-radiography technique to locate cracks, voids, faulty grouting and honeycombed areas in concrete. Voids or honey-

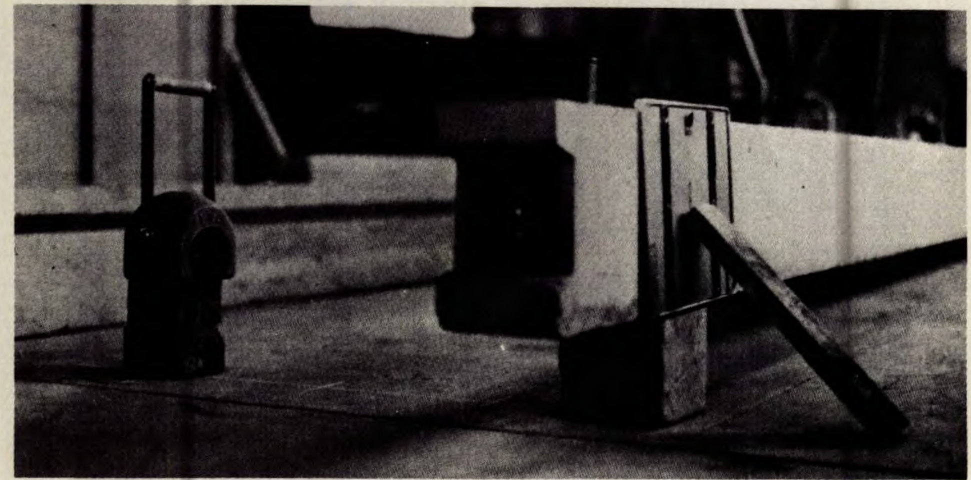

FIGURE 37. A precast concrete structural unit being subjected to gamma radiography. The radioactive source can be seen on the left. After Forrester (77).

combing in the concrete appears as dark patches on the radiograph, and cracks appear as dark lines.

LIMITS AND USEFULNESS OF RADIOACTIVE TEST METHODS

The radioactive methods are still in the development stage. The equipment is relatively expensive and is not commercially available for concrete testing, and test methods have yet to be standardized. Even with the highly penetrative gamma rays, the maximum thickness of concrete which can be penetrated is between 2 and 4 feet. This fact alone limits the usefulness of these methods.

CONCLUSIONS

1. Rapid advances have been made in the art of nondestructive testing of concrete. Apart from the radioactive methods, a large measure of standardization has been achieved in these tests.

2. The nondestructive methods of testing concrete must not be considered as a replacement for the standard destructive tests, but only as additional techniques. When performed by skilled technicians and the results evaluated by experienced engineers, the nondestructive tests do provide a storehouse of information which otherwise cannot be obtained. When carried out in conjunction with standard tests, they can reduce the cost of testing.

3. The various nondestructive methods of testing concrete provide an excellent means for establishing and evaluating uniformity of concrete, and their use in this aspect of concrete quality control is recommended.

4. Unless laboratory correlations have been established between the strength

parameters to be predicted and the results of nondestructive tests, the use of the latter to predict compressive or flexural strength of concrete is strongly discouraged.

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APPENDICES

APPENDIX I

MAIN FEATURES OF INDENTATION TEST HAMMERS*

Impact	Frank Spring Hammer		Einbeck Pendulum Hammer	
	Full	Half	Full	Half
Work performed during impact (kg/cm)	50	12.5	137	68.5
	Plunger Movement		Angle of Swing	
Type of impact	5 cm	2.5 cm	180 degrees	90 degrees
Drop (cm)	—	—	70	35
Diameter of test ball (mm)	10		25	
Range of indentation (mm)	4–7	4–7	7–14	7–14

*After Gaede (2).

APPENDIX 2

CALCULATION OF MODULUS OF ELASTICITY AND POISSON'S RATIO

(*a*) According to ASTM Designation C 215-60, the transverse or flexural dynamic modulus of elasticity may be calculated as follows:

$$E_R = C\,W\,n^2 \qquad \text{........ (Eq 1)}$$

where E_R = dynamic modulus of elasticity, psi
W = weight of specimen, pounds
n = fundamental transverse frequency, cps
$C = 0.00416\,\frac{L^3T}{d^4}$, sec^2 per sq. inch (for a cylinder)
$= 0.00245\,\frac{L^3T}{bt^3}$, sec^2 per sq. inch (for a prism)
L = length of specimen, inches
d = diameter of cylinder, inches
t,b = dimensions of cross-section of prism, inches
t being in the direction in which it is driven; and
T^* = a correction factor which depends on the ratio of the radius of gyration K' to the length of the specimen L, and on the Poisson's ratio.
$K' = d/4$ for a cylinder
$= t/3.464$ for a prism

(*b*) According to ASTM Designation C 215-60, the longitudinal dynamic modulus of elasticity may be calculated as follows:

$$E_R = D\,W(n')^2 \qquad \text{........ (Eq 2)}$$

where E_R = dynamic modulus of elasticity, psi
W = weight of specimen, pounds
n' = fundamental longitudinal frequency, cps
$D = 0.01318\,\frac{L}{d^2}$, sec^2 per sq. inch (for a cylinder)
$= 0.01035\,\frac{L}{bt}$, sec^2 per sq. inch (for a prism)
L = length of specimen, inches
d = diameter of cylinder, inches; and
t,b = dimensions of cross-section of prism, inches

*Values of T for Poisson's ratio of $\frac{1}{6}$ are given in Table 1, ASTM Designation C 215-60. However, the value of E_R is only slightly affected by Poisson's ratio; a change in the ratio from $\frac{1}{6}$ to $\frac{1}{4}$ increases the computed value of E_R by less than 2 per cent.

(*c*) According to ASTM Designation C 215-60, the dynamic modulus of rigidity may be calculated as follows:

$$G_R = B\,W\,(n'')^2 \qquad \text{........ (Eq 3)}$$

where G_R = dynamic modulus of rigidity, psi

W = weight of specimen, pounds

n'' = fundamental torsional frequency, cps

$B = \dfrac{4\,LR}{gA}$, sec^2 per sq. inch

L = length of specimen, inch

R (a shape factor) = 1.0 for circular cylinder

= 1.183 for a square cross-section prism

$$= \frac{a/b + b/a}{4\,a/b - 2.52\,(a/b)^2 + 0.21\,(a/b)^6}$$

for a rectangular prism whose cross-sectional dimensions are *a* and *b* in., with *a* less than *b*

g = gravitational acceleration (386.4 inches per sec); and

A = cross-sectional area of test specimen, sq. inches

(*d*) Poisson's ratio of small, regular-shaped specimens can be found by the resonance method from the formula:

$$\mu = \frac{E_R}{2G_R - 1} \qquad \text{........ (Eq 4)}$$

where μ = dynamic Poisson's ratio, and

E_R and G_R = dynamic moduli of elasticity and rigidity, respectively.

APPENDIX 3

COMPUTATIONS FOR RELATIVE DYNAMIC MODULUS OF ELASTICITY AND DURABILITY FACTOR

Relative Dynamic Modulus of Elasticity: According to ASTM Designation C 291-61T, the relative dynamic modulus of elasticity may be calculated as follows:

$$P_c = \frac{n_2^2}{n_1^2} \times 100 \qquad \text{........ (Eq 5)}$$

where P_c = relative dynamic modulus of elasticity, per cent, after C cycles of freezing and thawing;

n_1 = fundamental transverse frequency at 0 cycles of freezing and thawing; and

n_2 = fundamental transverse frequency after c cycles of freezing and thawing.

Durability Factor: According to ASTM Designation C 291-61T, the relative dynamic modulus of elasticity may be calculated as follows:

$$DF = \frac{PN}{M} \qquad \text{........ (Eq 6)}$$

where DF = durability factor of the test specimen;

P = relative dynamic modulus of elasticity at N cycles, per cent;

N = number of cycles at which P reaches the specified minimum value (60 per cent) for discontinuing the test, or the specified number of cycles at which the exposure is to be terminated, *whichever is least*; and

M = specified number of cycles at which the exposure is to be terminated.

APPENDIX 4

CORRELATIONS BETWEEN DYNAMIC MODULUS OF ELASTICITY AND STRENGTH PROPERTIES OF CONCRETE

1. Flexural Strength

Method of Long, Kurtz and Sandenaw (39)

$$F = A_1 + B_1E_R + C_1E^2{}_R \qquad \text{........ (Eq 7)}$$

where F = flexural strength, psi

E_R = dynamic modulus of elasticity, psi $\times 10^6$; and

A_1, B_1 and C_1 = constants with values of 29.3, 76.9 and 6.9 respectively.

Method of Sweet (40)

$$F = A_2 + B_2E_R \qquad \text{........ (Eq 8)}$$

where F and E_R are the same as in Equation 7, and

A and B are constants with values of −335 and 180 respectively.

Method of Chefdeville (41)

For concrete at least 28 days old:

$$E_R = K_1\sqrt{F} \qquad \text{........ (Eq 9)}$$

For concrete at earlier ages:

$$E_R = \frac{K_2}{1-\mu}\sqrt{F} \qquad \text{........ (Eq 10)}$$

where E_R and F are the same as in Equation 7 but in Kg/cm^2;

μ = dynamic Poisson's ratio; and

K_1 and K_2 are constants and have different values for different concretes and aggregates.

2. Compressive Strength

Method of L'Hermite (42)

$$E_R = K_3\sqrt{f_c} \qquad \text{........ (Eq 11)}$$

Method of Chefdeville (41)

For concretes at least 28 days old:

$$E_R = K_4 f_c^{\frac{1}{3}} \qquad \text{........ (Eq 12)}$$

For concretes at early ages:

$$E_R = \frac{K_5 f_c^{\frac{1}{3}}}{(1-\mu)} \qquad \text{..... .. (Eq 13)}$$

In these equations,

E_R = dynamic modulus of elasticity, kg/cm²
f_c = cube compressive strength, kg/cm²
μ = dynamic Poisson's ratio; and
K_3, K_4 and K_5 are constants.

Method of Sharma and Gupta (44)

$$E_R = 8.67 \times 10^6 \frac{f'_c}{f'_c + 1550} \qquad \text{........ (Eq 14)}$$

where E_R = dynamic modulus of elasticity, psi
f'_c = cylinder compressive strength, psi

APPENDIX 5

LOGARITHMIC DECREMENT AND DAMPING CONSTANT

Logarithmic Decrement. Jones (35) has defined the logarithmic decrement as the ratio between the amplitudes of successive oscillations in the damped sine wave produced by the decay of free vibrations of a specimen, and it is given by the following equation:

$$\delta = \ln \frac{h_1}{h_2} \qquad \text{........ (Eq 15)}$$

where δ = logarithmic decrement, and
h_1 and h_2 = amplitudes of two successive vibrations after the driving force has been removed from the specimen.

The amplitudes h_1 and h_2 can be obtained by recording on a moving film strip the decay of vibrations from resonance after the driving oscillator is turned off. A cathode-ray oscilloscope is used as an indicator. The amplitudes h_1 and h_2 can easily be measured off the film once it has been developed.

Damping Constant. The damping constant is given by the equation:

$$Q = \frac{f_o}{f_2 - f_1} \qquad \text{........ (Eq 16)}$$

where Q = damping constant;
f_o = resonant frequency of vibration; and
f_1, f_2 = frequencies on either side of resonance at which the amplitude is 0.707 times the amplitude at resonance.

The values of f_1 and f_2 can easily be determined if an output meter is employed for resonance indication. After locating the fundamental resonance, the oscillator is de-tuned on each side of the resonance frequency until the output meter reads 0.707 times the reading at resonance. The frequencies at which this occurs are the frequencies f_1 and f_2 (35).

The relationship between the damping constant and the logarithmic decrement is as follows:

$$Q = \frac{\pi}{\delta} \qquad \text{........ (Eq 17)}$$

Substituting the value of Q from Equation 16, we get:

$$\frac{f_o}{f_2 - f_1} = \frac{\pi}{\delta}$$

or

$$\text{Logarithmic decrement} = \pi \left(\frac{f_2 - f_1}{f_o} \right) \qquad \text{........ (Eq 18)}$$

APPENDIX 6

EQUATIONS FOR DETERMINATION OF PULSE MODULUS OF ELASTICITY

For Laboratory Beam Specimens:

$$E_p = \frac{V^2 d_1}{144g} \qquad \text{........ (Eq 19)}$$

For Flat Slabs:

$$E_p = \frac{V^2 d_1}{144g}(1 - \mu'^2) \qquad \text{........ (Eq 20)}$$

For Mass Concrete:

$$E_p = \frac{V^2 d_1 (1 + \mu')}{144g\,(1 - \mu')}(1 - 2\mu') \qquad \text{........ (Eq 21)}$$

where E_p = pulse modulus of elasticity, psi
V = longitudinal pulse velocity of transmission, ft/sec
d_1 = density of concrete, lb/cu/ft
g = 32.2 ft/sec^2; and
μ' = Poisson's ratio.

For $\mu' = 0.24$, the above Equation 21 reduces to:

$$E_p = \frac{V^2 d_1}{144g}(0.849) \qquad \text{........ (Eq 22)}$$

The reason that the value of E_p is not affected by Poisson's ratio when the test is carried out on laboratory specimens is that, in a small beam, concrete is free to expand and contract laterally when subjected to longitudinal strain. This reduces the wave velocity. In mass concrete, however, lateral displacements are suppressed and the wave velocity is slightly increased (31).

The experience of most investigators has been that, even for laboratory specimens, Equation 21 for mass concrete gives better results than do those applying to either slab or long slender member (48).