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Nondestructive Testing of Concrete: History and Challenges

by N.J. Carino

Synopsis: A brief history of nondestructive testing of hardened concrete over the past 50 years is presented. The contributions of V.M. Malhotra towards the development and promotion of nondestructive testing are emphasized. The underlying principles and inherent limitations of the methods are reviewed, and historical highlights of their development are presented. Test methods are grouped into those which assess in-place strength and those which evaluate non-strength characteristics, such as flaws and deterioration. The paper concludes with a discussion of the challenges for the 21st century in the area of nondestructive testing.

Keywords: Concretes; infrared spectroscopy; in-place testing; nondestructive tests; penetration tests; probes; pullout tests; radar; rebound hammer; statistical methods; tests; thermography; ultrasonic tests.

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INTRODUCTION

Concrete differs from other construction materials in that it can be made from an infinite combination of suitable materials and that its final properties are dependent on the treatment after it arrives at the job site. The efficiency of the consolidation and the effectiveness of curing procedures are critical for attaining the full potential of a concrete mixture. While concrete is noted for its durability, it is susceptible to a range of environmental degradation factors, which can limit its service life. There has always been a need for test methods to measure the in-place properties of concrete for quality assurance and for evaluation of existing conditions. Ideally, these methods should be nondestructive so that they do not impair the function of the structure and permit re-testing at the same locations to evaluate changes in properties with time.

Compared with the development of nondestructive test (NDT) methods for steel structures, the development of NDT methods for concrete has progressed at a slower pace, because concrete is an inherently more difficult material to test than steel. Concrete is highly heterogenous on a macroscopic scale, it is electrically nonconductive but usually contains significant amounts of steel reinforcement, and it is often used in thick members. Thus it has not been easy to transfer the NDT technology developed for steel to the inspection of concrete. In addition, there has been little interest by the NDT community (physicists, electrical engineers, mechanical engineers) to develop test methods for concrete. Prior to the 1980s, conferences, symposia and workshops organized by the NDT community rarely included sessions dealing with civil structures. Methods applicable to concrete that may have been developed in military research programs have either been unavailable for civilian use or have been too sophisticated and expensive for practical implementation. Despite these barriers, significant advances were made in the 1970s and 1980s. In addition, the crisis of the "aging infrastructure" has highlighted the needs for reliable NDT methods, and significant research funding has been available. It is an exciting time, and new methods are on the horizon.

The purpose of this paper is to provide a historical perspective on the development of NDT methods for concrete and to outline future directions. In any review of NDT methods for concrete, the name V. Mohan Malhotra will undoubtedly be encountered. Within the past 25 years, he has done more than any other person to advance, on a worldwide basis, the cause of NDT methods for concrete. His accomplishments have included numerous state-of-the-art assessments of technology, original research, and developing educational tools, such as the first videotape on the use of in-place tests. He provided leadership in the development of annotated bibliographies on NDT of concrete (Zoldners and Soles 1985, 1989, 1991, 1992), which provide a wealth of information for researchers and practitioners.

Terminology

There is no standard definition for *nondestructive tests* as applied to concrete. For some persons, they are tests that do not alter the concrete. For some, they are tests that do not impair the function of a structure, in which case the drilling of cores is considered a NDT test. For others, they are tests that do less damage to the structure than drilling of cores. This review deals with methods which either do not alter the concrete or result in superficial local damage. The author prefers to divide the various methods into two groups: (1) those whose main purpose is to estimate strength; and (2) those whose main purpose is evaluate conditions other than strength, i.e., to evaluate integrity. It will be shown that the most reliable tests for strength are those that result in superficial local damage, and the author prefers the term *in-place tests* for this group. The *integrity tests*, on the other hand, are nondestructive.

Scope

Table 1 lists the various test methods that will be considered in this review. Techniques for corrosion testing and locating reinforcing bars, the so-called *permeability* tests, and tests of fresh concrete are not included. Information on these methods is available (Malhotra and Carino 1991, Bungey 1989). The bibliographies developed by the Canada Centre for Mineral and Energy Technology (CANMET) were vital in preparing this paper (Zoldners and Soles 1985, 1989, 1991, 1992).

PRIOR TO WORLD WAR II

Some of the first methods to evaluate the in-place strength of concrete were adaptations of the Brinell hardness[2] test for metals, which involves pushing a high-strength steel ball into the test piece under a given force and measuring the area of the indentation. In the metals test, the load is applied by an hydraulic loading system. Modifications were required to enable this type of test to be made on a concrete structure. In 1934 Prof. K. Gaede (Hanover, Germany) reported on the use of a spring-driven impactor to supply the force to drive a steel ball into the concrete (Malhotra 1976). A non-linear, empirical relationship was obtained between cube strength and indentation diameter. In 1936, J.P. Williams (England) reported on a spring-loaded, pistol-shaped device, in which a 4-mm ball was attached to a plunger (Malhotra 1976). The spring was compressed by turning a screw, a trigger released the compressed spring, and the plunger was propelled toward the concrete. The

[2]The term *hardness* is used routinely in the description of a series of tests of metals and concrete, yet this is not a readily quantified mechanical property. If one considers the nature of the hardness test methods that have been developed for metals, it can be concluded that these tests measure the amount of penetration caused by a specific indentor under a specific load. Therefore, a more descriptive term for these methods might be *indentation* tests.

diameter of the indentation produced by the ball was measured with a magnifying scale.

In 1938 there appeared a landmark paper by D.G. Skramtajev, of the Central Institute for Industrial Building Research, Moscow (Skramtajev 1938). It summarized 14 different techniques, 10 of which were developed in the Soviet Union, for measuring the in-place strength of concrete. This paper should be read by every student of nondestructive testing for its historical content. Skramtajev divided the test methods into two groups: (1) those that required installation of test hardware prior to placement of concrete, and (2) those that did not require pre-installation of hardware. The methods described by Skramtajev included the following: molds placed in the structure to form in-place test specimens; pullout tests of embedded bars; an in-place punching shear test; an in-place fracture test using a pincer-device; penetration of a chisel by hammer blows; guns that fired indentors into the concrete; and penetration of a ball by a spring-driven apparatus. Readers who are familiar with the *modern* in-place test methods (to be discussed later) will recognize that many of them are variations of methods suggested over one-half century ago.

Skramtajev also commented on the need for in-place testing. For example, he noted that (Skramtajev 1938):

- The curing conditions of standard test specimens are not representative of the concrete in the structure.
- The number of standard test specimens is insufficient to assure the adequacy of all members in a structure.
- Standard test specimens that are tested at age of one month provide no information on later-age strength in the structure.
- Surface tests may not provide indication of the actual concrete strength due to the effects of carbonation, laitance, and moisture condition.
- Methods requiring pre-placement of hardware tend to provide more precise estimates of strength than those that do not require pre-placement of hardware, but they lack flexibility for use at any desired location in an existing structure.

It is interesting that fifty years later, the same arguments and limitations are quoted in relation to in-place testing (ACI 228, 1989).

The methods reviewed by Skramtajev are mechanical tests and produce varying degrees of local damage to the concrete. In 1938 another landmark paper was published which dealt with a truly nondestructive test. The paper was by T.C. Powers, of the Portland Cement Association, who reported on the use of resonant frequency testing to establish the modulus of elasticity of concrete (Powers 1938). Prismatic specimens were struck with a hammer, and the resulting resonant frequencies were determined by comparing the tones to those of calibrated steel bars. Subsequent advancements resulted in electronic devices that eliminated the need of matching tones by listening to sounds. Typically, these devices used a special speaker to vibrate the test specimen at variable frequencies, and a transducer (phonograph needle) was used to measure the amplitude of the vibration. When the specimen was vibrated at its resonant frequency, a maximum amplitude was noted.

The resonant frequency method was quickly adapted for monitoring deterioration during cyclic freezing and thawing tests (Hornibrook 1939), and it became the first *nondestructive* test to be standardized by ASTM. A tentative test method for measuring the *transverse* resonant frequency of test specimens was published in 1947 (ASTM C 215-47T). Subsequent modifications added procedures for determining the *longitudinal* and *torsional* resonant frequencies, and provided formulae for computing the elastic constants. The method remained essentially unchanged until a major revision was adopted in 1991, which permitted impact testing as an alternative to the forced resonance technique. The new impact method improved upon Powers' approach by using modern frequency analyzers to determine the natural frequencies of the specimens (Wang et al. 1973, Gaidis and Rosenberg 1986).

The standard resonant frequency method is basically a laboratory procedure and is not applicable to in-place testing. However, impact response measurements are being used to evaluate complete structures (Maguire and Severn 1987). These approaches are not discussed in this review.

POST WORLD WAR II and THE 1950s

Following World War II, there was a great surge in the development of nondestructive test methods for concrete. This review focuses on four methods: ultrasonic pulse velocity; rebound hammer; maturity method; and radioisotopes.

Ultrasonic Pulse Velocity

The ultrasonic pulse velocity method is a stress wave propagation method that involves measuring the travel time, over a known path length, of a pulse of ultrasonic waves. The pulses are introduced into the concrete by a piezoelectric transducer and a similar transducer acts as receiver to monitor the surface vibration caused by the arrival of the pulse. A timing circuit is used to measure the time it takes for the pulse to travel from the transmitting to the receiving transducers. The presence of low density, or cracked, concrete increases the travel time which results in a lower pulse velocity. By conducting tests at various points on a structure, locations with lower quality concrete can be identified by their lower pulse velocity.

The development of a field instrument to measure the pulse velocity occurred nearly simultaneously in Canada and in England (Whitehurst 1967). The developments were outgrowths of earlier successful work by the U.S. Corps of Engineers to measure the speed of a mechanical stress pulse through concrete (Long et al. 1945). The earlier approach involved two receivers attached to the concrete surface. A horizontal hammer blow was applied in line with the receivers, and a specially designed electronic interval timer was used to measure the time for the pulse to travel from the first to the second receiver. The major purpose of this technique was to calculate the in-place modulus of elasticity.

In 1946 and 1947, engineers at the Hydro-Electric Power Commission of Ontario (Ontario Hydro) worked on the development of a device to investigate the extent of cracking in dams (Leslie and Cheesman 1949). The device that was developed was called the *Soniscope*. It had a 20-kHz transmitting transducer, was capable of penetrating up to 15 m of concrete, and could measure the travel time with an accuracy of 3%. The stated purposes of the Soniscope were to identify the presence of internal cracking, determine the depth of surface-opening cracks, and determine the dynamic modulus of concrete (Leslie and Cheesman 1949). It was further stated that the fundamental measurement was the travel time. The amplitude of the received signal was said to be of secondary importance because the transfer of energy between the transducers and the concrete could not be controlled. It was also emphasized that interpretation of results required knowledge of the history of the structure being investigated.

In early uses of the Soniscope on mass concrete, the emphasis was on measuring the pulse velocity rather than estimating strength or calculating the elastic stiffness (Parker 1953). By making velocity readings on a gridwork, the presence of distressed concrete could be easily located. Parker (1953) reported on early attempts at Ontario Hydro to develop relationships between pulse velocity and compressive strength. Forty-six mixtures involving the same aggregate, different cement types, and different admixtures were investigated. The results indicated no significant differences in the velocity-strength relationships for the different mixtures. The results were, therefore, treated as one group, and the best-fit relationship was determined. Figure 1 shows the relationship between estimated strength and pulse velocity and the lower 95% confidence limit for the estimated strength. Due to large scatter, the lower confidence limit was about 45% of the mean strength. Thus the inherent uncertainty in using pulse velocity to estimate strength was established very early. Figure 1 also shows that the change in pulse velocity per unit change in strength decreases with increasing strength. This means that pulse velocity is relatively insensitive to strength for mature concrete.

While work on the Soniscope was in progress in Canada, R. Jones and co-workers at the Road Research Laboratory (RRL) in England were involved in independent research to develop an ultrasonic testing apparatus (Jones 1949b). The RRL researchers were interested in testing the quality of concrete pavements, which involved shorter path lengths compared with the work at Ontario Hydro. As a result, the apparatus that was developed operated at a higher frequency than the Soniscope, and it was called the *ultrasonic concrete tester*. Transducers with resonant frequencies from 60 to 200 kHz were used, depending on the desired penetration (Jones 1953). Besides using a different operating frequency, the RRL device used a different approach than the Soniscope to measure travel time. This was necessary because of the shorter path lengths in the RRL work. It was reported that the ultrasonic concrete tester could measure travel times to within ± 0.2 µs.

Jones reviewed the test program carried out with the newly developed ultrasonic concrete tester (Jones 1949b). Among these studies were the following:

- Investigation of the variation of pulse velocity with height in standard cube specimens and with depth in slabs. This is one of the first studies to document the *top-to-bottom effect* that is mentioned often as a problem when planning and interpreting in-place tests (ACI 228, 1989).

- Investigation of the influence of water-cement ratio, aggregate type and aggregate content on pulse velocity. These studies demonstrated the importance of aggregate type and aggregate content on pulse velocity.
- Investigation of the relationships between pulse velocity and compressive strength. These studies demonstrated that for a given mixture and under uniform conditions, there was good correlation between strength and pulse velocity.

Thus Jones established the inherent problems in using the pulse velocity to estimate concrete strength. Despite these early findings, numerous researchers sought to establish correlations between pulse velocity and strength, and many reached the same conclusions as Jones (Sturrup et al. 1984).

In the United States, a Soniscope was developed in 1947 at the Portland Cement Association in cooperation with Ontario Hydro, and field applications were reported by Whitehurst (1951). In the summary of the U.S. experience, Whitehurst published the following *tentative classification* for using pulse velocity as an indicator of quality:

Pulse velocity, m/s	Condition
Above 4570	excellent
3660 to 4570	generally good
3050 to 3660	questionable
2130 to 3050	generally poor
Below 2130	very poor

This table was quoted in many subsequent publications. However, Whitehurst warned that these values were established on the basis of tests of normal concrete having a density of about 2400 kg/m^3 and that the boundaries between "conditions" could not be sharply drawn. He mentioned that, rather than these limits, the more important comparison should be with the velocity in portions of the structure that are known to be of acceptable quality. Nevertheless, the above table was used often by inexperienced investigators as the sole basis to interpret test results.

After the publication of these landmark papers on the development of the ultrasonic pulse velocity method, a flurry of activity occurred worldwide, and efforts were begun toward developing test standards. In the U.S., a proposed ASTM test method was published by Leslie in 1955, but it was not until 1967 that it finally became a tentative test method (ASTM C 597-67T). In Europe, the International Union of Testing and Research Laboratories for Materials and Structures (RILEM) organized a working group on nondestructive testing (R. Jones as Chairman), and in 1969 draft recommendations for testing concrete by the ultrasonic pulse method were published (Jones and Făcăoaru 1969). In Eastern Europe, the method was used extensively in precast concrete plants.

Rebound Hammer

In 1948 Ernst Schmidt, a Swiss engineer, developed a device for testing concrete based upon the rebound principle (Malhotra 1976). As was the case with the earlier indentation tests, the motivation for this new device came from tests developed previously to measure hardness of metals. In this case, the new device was an outgrowth of the *Scleroscope*[3] test, which involves measuring the rebound height of a diamond-tipped *hammer*, or mass, that is dropped from a fixed height above the test surface.

As noted by Kolek (1958), when concrete is struck by a hammer, the degree of rebound is an indicator of the *hardness* of the concrete. Schmidt standardized the hammer blow by developing a spring-loaded hammer and devised a method to measured the rebound of the hammer. Several models of the device were built (Greene 1954), and Fig. 2 is a schematic of the model that was eventually adopted for field use. The essential parts of the *Schmidt rebound hammer* are the outer body, the hammer, the plunger, the spring, and the slide indicator. To perform the test, the plunger is extended from the body of the instrument, which causes a latch mechanism to grab hold of the hammer (Fig. 2(a)). The body of the instrument is then pushed toward the concrete surface, which stretches the spring attached to the hammer and the body (Fig. 2(b)). When the body is pushed to the limit, the latch is released and the hammer is propelled toward the concrete (Fig. 2(c)). The hammer strikes the *shoulder* of the plunger and it rebounds. The rebound distance is measured on a scale by the slide indicator. The rebound distance is expressed as a *rebound number*, which is the percentage of the initial extension of the spring (Kolek 1958). Currently, different models of the instrument are available, which differ in the mass of the hammer and the rigidity of the spring. Thus different impact energies can be used for different materials.

Due to its simplicity and low cost, the Schmidt rebound hammer is, by far, the most widely used nondestructive test device for concrete. It is reported that about 50,000 rebound hammers were sold worldwide by 1986 (Malhotra 1991). While the test appears simple, there is no simple relationship between the rebound number and the strength of concrete. In principle, the rebound is affected by the movement of the end of the plunger in contact with the concrete. The more the end of the plunger moves, the lower is the rebound. Thus the rebound number is likely to be influenced by the elastic stiffness and the strength of the concrete. As reviewed by Malhotra (1976, 1991), there are many factors that affect the rebound number for a given strength of concrete.

The rebound hammer was constructed and tested extensively at the Swiss Federal Materials Testing and Experimental Institute in Zurich. A correlation was developed between the compressive strength of standard cubes and rebound number, and this correlation was provided with the instrument. However, as other investigators began to develop correlations between strength and rebound number, it became evident that there was not a unique relationship between strength and rebound number (Kolek 1958). This lead to the often-stated recommendation that for the best accuracy a correlation should be developed using the same concrete and forming materials as will be used in construction. Without such a correlation, the rebound hammer is useful only for detecting gross changes in concrete quality throughout a structure.

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[3]In Greek, the word "sklero" means "hard".

Because of the uncertainties associated with strength predictions based upon the rebound hammer, there was considerable time delay between the development of the method and its incorporation into standards. For example, the British Standards Institution included the method in its standards in 1971, a tentative method was adopted by ASTM in 1975 (ASTM C 805 75-T), and draft recommendations were published by RILEM in 1977. Today, the rebound hammer is recognized as a useful tool for performing quick surveys to assess the uniformity of concrete, but it is not generally recommended where accurate strength estimates are needed.

Maturity Method

The *maturity method* is a technique to estimate strength development of concrete during its curing period by measuring the temperature history of the concrete. Historically, it was not classified as a nondestructive test method, but it is now regarded as a useful technique to estimate in-place strength. Its origin can be traced to a series of papers from England dealing with accelerated curing methods (McIntosh 1949, Nurse 1949, Saul 1951). There was a need for a procedure to account for the combined effects of time and temperature on strength development for different elevated temperature curing processes. It was proposed that the product of time and temperature could be used for this purpose. These ideas lead to the famous *Nurse-Saul maturity function*:

$$M = \sum_{0}^{t} (T - T_0)\, \Delta t \tag{1}$$

where M = maturity index, °C-hours (or °C-days),
T = average concrete temperature, °C, during the time interval Δt,
T_0 = datum temperature (usually taken to be 10 °C), and
Δt = time interval.

The index computed by Eq. (1) was called the *maturity*, however, the current terminology is the *temperature-time factor* (ASTM C 1074). Saul (1951) presented the following principle which has become known as the *maturity rule*:

> *Concrete of the same mix at the same maturity (reckoned in temperature-time) has approximately the same strength whatever combination of temperature and time go to make up that maturity.*

Equation (1) is based on the assumption that the rate of strength gain is a linear function of temperature, and it was soon realized that this approximation may not be valid when curing temperatures vary over a wide range. As a result, a series of alternatives to the Nurse-Saul function were proposed by other researchers (Malhotra 1971). However, none of the alternatives received widespread acceptance, and the Nurse-Saul function was used worldwide until an improved function was proposed in the 1970s.

Radioactive Methods

Another major development in the 1950s was the application of radioactive methods to the inspection of concrete. These involve a source of penetrating electromagnetic radiation and a sensor to measure the intensity of the radiation after it has travelled through the object. If the sensor is in the form of special photographic film, the technique is called *radiography*. If the sensor is an electronic device which converts the incident radiation into electrical pulses, the techniques is called *radiometry* (Mitchell 1991).

Initial work in the late 1940s focused on the use of X-rays to produce radiographs that revealed the internal structure of concrete elements, but in the 1950s attention turned to the use of gamma rays. The fundamental differences between these two forms of penetrating radiation are the sources used to generate them and their penetrating ability. X-rays are produced by high-voltage electronic devices, and gamma rays are the by-products of the disintegration of radioactive isotopes. The penetrating ability of gamma rays depends on the radioactive isotope and its age, while the penetration of X-rays depends on the voltage of the generating instrument.

There are two basic methods to use X-rays and gamma rays in nondestructive testing of concrete. In the *transmission* method, the amplitude of the radiation passing through a member is measured. As the radiation passes through a member, the attenuation is dependent on the density of the material and the path length from the source to the sensor. In the *backscatter method*, a radioactive source is used to supply gamma rays and a detector close to the source is used to measure the backscattered rays. The backscatter method was used to develop instruments for measuring in-place density of construction materials.

A summary of the early work involving X-rays and gamma rays is provided by Malhotra (1976). Some of the earliest reported work was at Ontario Hydro (de Hass 1954). Slabs were constructed with artificial flaws, a pipe containing a radioactive isotope of cobalt was placed beneath the slab, and a Geiger-Müller tube was placed on the top surface of the slab to measure the intensity of radiation. The earliest concerted efforts at using gamma ray methods took place in Great Britain during the 1950s, where it was used to locate reinforcing bars, measure density, and locate voids in grouted post-tensioning ducts (Forrester 1970). Eastern Europe and the Soviet Union also conducted early studies that eventually lead to the development of portable density meters for concrete and soil.

While radioactive methods have the ability to *see* into concrete, they are cumbersome and require trained and licensed personnel. In addition, the penetrating abilities are limited and testing is disruptive to a structure in active use. Thus the use of radioisotopes has not developed into a routine method for flaw detection.

THE 1960s and 1970s

Compared with the pioneering developments that occurred during the late 1940s and 1950s, developments during the early 1960s were less historic. Much of the research focused on refining the ultrasonic pulse velocity method and gaining a better understanding of the factors affecting pulse velocity. However, during the late 1960s and 1970s significant advancements were made in the development of new test systems to estimate in-place strength. While these new systems could not, in a strict sense, be classified as nondestructive because they produced surface damage, they were viewed as nondestructive compared with drilling cores. In addition to new methods for estimating strength, this period also witnessed the early stages of development of powerful techniques for flaw detection.

Before proceeding with a review of the various advancements, some of the contributions of V.M. Malhotra during this period are mentioned. In 1976, the classic monograph on nondestructive testing of concrete was published (Malhotra 1976). This monograph proved to be instrumental in educating today's engineers and concrete technologists (including this author) on the principles, applications, and limitations of the available test methods. At about the same time, Malhotra wrote a landmark paper on the subject of testing cores versus in-place test methods (Malhotra 1977). In that paper, a call was made for a change to the status-quo for quality control of the concrete that is actually in a structure. He argued that the current practice based on the standard-cured cylinder strength fails to assure quality of *in-place* concrete. Malhotra suggested a new, three-pronged approach to remedy the situation:

(1) Carry out sufficient inspection to assure that the concrete delivered to the site meets the specifications and that it is properly handled once it arrives at the site.
(2) Use accelerated strength methods to assure that the concrete as delivered meets the strength requirements.
(3) Use in-place test methods to estimate the actual strength achieved in the structure.

Malhotra concluded his paper by stating:

> *"In order to create some semblance of order in an otherwise chaotic situation, a completely new approach has been suggested. This, of course, will involve fundamental changes in our approach to specifications and code writing, and could take some time before the concrete community accepts it."*

Malhotra's suggested approach remains a goal to be achieved, and as predicted, progress towards acceptance of this approach has been slow.

<u>Ultrasonic Pulse Velocity</u>

During this period, considerable attention was devoted to gaining further knowledge about the effects of different factors on pulse velocity. Researchers continued to explore the relationship between compressive strength and pulse velocity. However, there appears to have been a consensus that there is no unique relationship. Numerous studies showed that the type and quantity of aggregate have major effects on pulse velocity, but not on strength. Significant effort was also expended to examine whether attenuation measurements could provide additional information about concrete strength. These results were, in general, found to be impractical in field situations because of difficulties in achieving consistent coupling of the transducers, which is critical for measuring attenuation.

Perhaps the most significant advances during this period were in the development of improved field instrumentation. Due to advances in micro-electronic circuity, the cumbersome instruments developed in the 1940s and 1950s gave way to compact portable devices. In the late 1960s, TNO in Delft, Netherlands developed a portable, battery-operated pulse velocity device that incorporated a digital display of the travel time. In the earlier devices, travel time was measured by examination of oscilloscope displays, which was a time-consuming process. The portable instrument had a resolution of 1 μs, which resulted in low accuracy for short path lengths, and it had limited penetrating ability (Făcăoaru 1969). At about the same time, R.H. Elvery of University College, London developed a similar portable device which was called PUNDIT (Portable Ultrasonic Non-destructive Digital Indicating Tester). It weighed 3.2 kg, had a resolution of 0.5 μs, and could be powered by rechargeable batteries (Malhotra 1976). These, and other, relatively low-cost, portable devices simplified testing, and resulted in a worldwide increase in the number of consultants and researchers who could perform this type of testing. Later models of these devices had resolutions of 0.1 μs and some provided an optional output terminal to allow the received signal to be displayed on an oscilloscope.

Combined Methods

A major finding from the many investigations in the 1950s and 1960 on the correlations between strength and nondestructive test results was that none of the methods could provide the level of accuracy needed to make *accept/reject* decisions about the quality of in-place concrete. Hence efforts were made to develop *combined methods*, in which the results of two or more nondestructive tests are used to estimate concrete strength. The underlying notion is that if the two methods are influenced in different ways by the same factor, their combined use results in a canceling effect that improves the accuracy of the estimated strength. For example, increasing moisture content increases pulse velocity but decreases the rebound number.

Among the various suggestions, the combination of pulse velocity and rebound number have received the most consideration. This combined method was strongly promoted by Dr. I. Făcăoaru of the Building Research Institute in Romania (Făcăoaru 1970) and became known as the *SONREB method*. The SONREB method is based on a correlation obtained using a *standard* concrete mixture. By using multiple-regression analysis, the compressive strength (S) of standard specimens is expressed as a function of the average rebound number (R) and the average pulse velocity (V). A variety of correlation equations have been suggested by various researchers (Samarin 1991). For example, some have used a linear equation as follows:

$$S = A_0 + A_1 R + A_2 V \tag{2}$$

The coefficients A_0, A_1, and A_2 would be determined by least-squares regression analysis. The resulting correlation can be depicted in a two-dimensional graph by a series of contour lines representing the combinations of rebound number and pulse velocity that result in the same estimated strength.

Reported values of rebound number, pulse velocity, and cube strength (Cianfrone and Făcăoaru 1979) are used to illustrate the nature of the SONREB correlation. The published data were entered into a computer spreadsheet by the author and Eq. (2) was fitted to the data. Based on the regression equation, equal-strength contour lines were determined for compressive strengths of 20, 30, 40, 50, 60 and 70 MPa. The data and the contour lines are shown in Fig. 3(a). The data were divided into groups based on measured cube strength. The scatter of the data about the best fit correlation is illustrated in Fig. 3(b). The horizontal axis is the measured compressive strength, and the vertical axis is the estimated strength based on the correlation. In general, the differences between the measured and estimated strengths fall within ±15%.

In the SONREB method (Făcăoaru 1970), the correlation is based on a standard concrete mixture. To improve the accuracy of the estimated strength for other mixtures, a series of *influence coefficients* are applied to the estimated strength obtained from standard mixture correlation. Five coefficients are used to account for the cement type, the cement content, the type of aggregate, the gradation of the fine aggregate and the maximum aggregate size. Thus knowledge about the concrete is needed to improve the accuracy of the estimated strength. If mixture data are not available, a set of cores can be used to obtain a correction factor, which equals the ratio of the measured core strength to the estimated strength obtained from the standard correlation.

From the experience gained in Romania, Făcăoaru (1970) suggested the following levels of accuracy for the SONREB method:

- When composition is known and cores are available, accuracy is ± 10 to 15%.
- When only the composition is known, accuracy is ± 15 to 20%.
- When the composition is unknown and cores are not available, accuracy is ± 20 to 30%.

Other countries investigated the applicability of the SONREB method during the 1970s (Samarin 1991). In North America, the method aroused little interest. Because of the diversity of concreting materials used in North American construction, a large testing program would be needed to develop the *standard* correlation and the accompanying influence coefficients. No interest was shown in organizing and funding such a program. In addition, the method is best suited for quality control in a precast plant, where similar materials are used during production. However, in North America, the majority of concrete construction is cast-in-place.

A RILEM Draft Recommendation on the application of combined methods was published in 1993[4].

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[4]*Materials and Structures* (RILEM), Vol. 26, No. 155, Jan-Feb 1993, pp. 43-49.

Maturity Method

In 1977, an new function was proposed to compute a maturity index from tl recorded temperature history of the concrete (Freiesleben Hansen and Pederse 1977). This function was based on the Arrhenius equation that is used to describe th effect of temperature on the rate of a chemical reaction. The new function allowe the computation of the *equivalent age* of concrete as follows:

$$t_e = \sum_0^t e^{\frac{-E}{R}\left(\frac{1}{T}-\frac{1}{T_r}\right)} \Delta t \tag{3}$$

where t_e = the equivalent age at the reference temperature
E = apparent activation energy, J/mole
R = universal gas constant, 8.314 J/K-mole
T = average absolute temperature of the concrete during interval Δt degrees Kelvin
T_r = absolute reference temperature, degrees Kelvin

By using Eq. (3), the actual age of the concrete is converted to its equivalen age, in terms of strength gain, at the reference temperature. In European practic the reference temperature is usually taken to be 20 °C, whereas in North Americar practice it is usually assumed to be 23 °C. The introduction of this function overcame one of the main limitations of the Nurse-Saul function (Eq. (1)) because it allowec for a non-linear relationship between the rate of strength development and curing temperature. This temperature dependence is described by the value of the *apparen activation energy*. Comparative studies in the early 1980s showed that this new maturity function was superior to the Nurse-Saul function (Byfors 1980, Carino 1982).

New Strength Tests

During this period, several new methods were introduced for estimating the in-place strength of concrete. These techniques produce local damage to the near-surface concrete and are, therefore, not truly nondestructive. Because they produce local failure, these methods result in correlations that are, generally, not very sensitive to changes in such factors as concrete composition, aggregate type, surface texture, and moisture conditions. However, they tend to be more time-consuming and expensive to use than the rebound and pulse velocity methods.

Probe penetration — One of the important developments in the U.S. during this period was the *probe penetration* method. This involves using a gun to drive a hardened steel rod, or *probe*, into the concrete and measuring the exposed length of the probe. In principle, as the strength of the concrete increases, the exposed probe length also increases, and by means of a suitable correlation, the exposed length can be used to estimate compressive strength. This idea for the technique is not original as Skramtajev had mentioned a similar concept in his 1938 summary paper, and Malhotra (1976) mentions that similar techniques were also reported in 1954.

Development of this new probe penetration test system began in about 1964 as a joint undertaking by T.R. Cantor of the Port of New York Authority and R. Kopf of the Windsor Machinery Co. (Arni 1972). The test system that was eventually commercialized became known as the *Windsor probe*. The apparatus is supplied with a table relating exposed probe length to compressive strength for different aggregate hardness as measured by Mohs' hardness scale of minerals. The basis of the values in the tables and their uncertainty were not provided (Arni 1972). In the late 1960s, independent investigations of the reliability of the Windsor probe system were carried out by the National Ready Mixed Concrete Association (Gaynor 1969), the Federal Highway Administration (FHWA) (Arni 1972), and the Department of Energy, Mines and Resources (Canada) (Malhotra 1974). In general, it was found that the probe system had an acceptable within-test variability. However, the scatter in the correlation between compressive strength and probe penetration would lead to rather high uncertainties in the estimated strength. All investigators cautioned against reliance on the manufacturer's correlation tables.

Arni's study of the uncertainties of the probe penetration and rebound hammer tests is very interesting and worth summarizing (Arni 1972). He calculated the number of tests required to detect a strength difference of 1.4 MPa (200 psi) using cylinders, probe penetration, or rebound number. These estimates were based on the variability of test results and the slopes of the correlation equations developed in the FHWA study. For 90% confidence levels, the results were as follows:

Cylinders	8
Rebound	120
Probe	85

Note that these numbers apply for specific data used by Arni. Nevertheless, they point out the inherent inability of in-place tests to detect small differences in concrete strength without performing large numbers of tests. This important concept has been largely ignored.

The Windsor probe test method was adopted as a tentative ASTM standard (C 803-75T) in 1975. In 1990, the standard was modified to include the use of a *pin penetration* device, in which a small pin is forced into the concrete using a spring-loaded driver (Malhotra and Carette 1991).

Pullout test — The pullout test measures the force required to extract an insert with an enlarged head that has been cast into the concrete. When the insert is extracted, a conic frustum of concrete is also pulled out. The ultimate force, therefore, measures a strength property of the concrete. It is this characteristic which makes the pullout test superior to other indirect methods, such as rebound hammer and pulse velocity. During the 1970s, significant progress was made in developing portable pullout test systems that could be used in the field. A comprehensive review of the history and theory of the pullout test is available (Carino 1991b), and only a brief summary is provided here.

As mentioned, ideas for pullout tests originated in the Soviet Union (Skramtajev 1938). In 1944, Tremper became the first American to report on the correlation between pullout force and companion cylinder strength. The insert developed by Volf (of the Soviet Union) and the one used by Tremper are shown in Figs. 4(a) and 4(b), respectively. In both cases, the reaction to the pullout force was applied sufficiently distant from the insert so that there was negligible interaction between the failure surface and the reaction system. As a result, failure was controlled primarily by the tensile strength of the concrete. This explains why Tremper found that the correlation between pullout force and compressive strength was non-linear.

Despite Tremper's encouraging results, there was no additional documented work on the pullout test until 1962, when a comprehensive study began in Denmark (Kierkegaard-Hansen 1975). The objective was to find the optimum geometry for a field test system that would have high correlation between pullout load and compressive strength of concrete. Kierkegaard-Hansen found that the correlation could be improved by constraining the failure surface to follow a predefined path by using a relatively small diameter reaction ring. The study resulted in the pullout test configuration shown in Fig. 4(c), which was eventually incorporated into the LOK-TEST[5] system, the most widely used commercial pullout test system.

Owen Richards, a materials consultant in the U.S., carried out independent studies of a pullout test in the late 1960s and early 1970s. The early version of Richards' pullout test configuration was larger than that being developed in Denmark. The inserts were manufactured from 19-mm threaded rods and washers were used to provide the enlarged head. Nuts were used to add rigidity to the washers and to fix the embedment depth of the washers. The test geometry, which is shown in Fig. 4(d), resulted in an idealized failure surface with an area approximately equal to the area of a standard 152-mm diameter cylinder. Richards preferred to divide the pullout force by the nominal area of the idealized failure to obtain a *pullout strength*, which was a fictitious quantity because the pullout force was inclined to the surface area.

The first reported pullout tests using Richards' early system were performed at the Bureau of Reclamation (Rutenbeck 1973). Strong correlation was obtained between pullout tests performed on slabs and the compressive strength of companion cylinders. Good correlation between pullout strength and compressive strength was also obtained when the inserts were placed in shotcrete panels and compression specimens were cut from the panels. In 1975, Malhotra also reported on the applicability of Richards' pullout test (Malhotra 1975). It was found that the coefficient of variation for three replicate tests was less than 5%, which was very encouraging. In a later study (Malhotra and Carette 1980), it was noted that similar correlations were obtained in different investigations of Richards' system.

Richards' pullout system produced encouraging results, but the large size of the insert required heavy testing equipment and produced significant surface damage. In 1977, a smaller version of the test system was introduced (Richards 1977), as shown

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[5]As explained by Kierkegaard-Hansen (1975), the failure when a small reaction ring is used can be considered as a *punching* type of failure. The Danish word for "punching" is *lokning*, so the term *lok-strength* was used to describe the strength measured by the test.

in Fig. 4(e). The apex angle of the conic frustum was maintained at 67°, but the insert was constructed from one piece of steel. The enlarged end of the shank accommodated a pull-rod that passed through a center-hole tension ram.

In the early 1960s, investigations of the pullout test were also conducted in Great Britain (Te'eni 1970), but the work was apparently never carried to the stage of a practical field test system. A novel feature of the British work was the use of a power function for the correlation equation, rather than a straight line as had been used in Denmark, U.S.A, and Canada.

The usefulness of the pullout test for evaluating early-age strength was quickly recognized. In 1978, ASTM adopted a tentative test method for the pullout test (C 900-78T). In North America, J. Bickley became an early advocate of the pullout test method for achieving construction safety and economy (Bickley 1982a).

Post-installed pullout tests -- A drawback of the pullout test mentioned in the previous section is that the inserts have to be cast into the concrete. This limits the applicability of the method to new construction. In an effort to apply pullout testing to existing structures, various types of *post-installed* pullout tests were investigated in the 1970s.

In the United Kingdom, there was a need for in-place tests to evaluate distressed concrete structures built with high alumina cement. Researchers at the Building Research Establishment (BRE) developed a pullout technique using commercial anchor bolts, as shown in Fig. 5(a) (Chabowski and Bryden-Smith 1980). A 6-mm diameter hole is drilled into the concrete and an anchor bolt is inserted so that the split-sleeve is at a depth of 20 mm. After applying an initial load to expand and engage the sleeve, the bolt is pulled out and the maximum load during the extraction is recorded. Because of the shallow embedment, failure occurs by concrete fracture. Reaction to the pullout load is provided by three *feet* located along the perimeter of a 80-mm diameter ring. As the bolt is pulled, the sleeve imparts vertical and horizontal forces to the concrete. Hence the fracture surface differs from that in the cast-in-place (CIP) pullout test, and the test has been called an *internal fracture test* rather than a pullout test. The correlation between ultimate load and compressive strength was found to have a pronounced non-linearity, indicating that the failure mechanism was probably related to the tensile strength of the concrete. Within-test variability was found to be greater than the CIP pullout test, and the 95% confidence limits of the correlation relationship were found to range between ±30% of the mean curve (Chabowski and Bryden-Smith 1980). The relatively low precision of the internal fracture test has been attributed to two principal causes (Bungey 1981): (1) the variability in the hole drilling and test preparation; and (2) the influence of aggregate particles on the load transfer mechanism and the failure initiation load.

In a study funded by CANMET, the feasibility of several post-installed pullout tests was also investigated (Mailhot et al. 1979). One of these used a split-sleeve and a tapered bolt assembly that was placed in a 19-mm diameter hole drilled into the concrete. As shown in Fig. 5(b), the details differ from the BRE method because the reaction to the pulling force acts through a specially designed high strength, split-sleeve assembly. Thus the force transmitted to the concrete is predominantly a lateral load due to the expansion of the sleeve by the tapered bolt. It is likely that failure occurs by indirect tensile splitting, similar to that in a standard splitting-tension test. As with the BRE test, the variability of this test was reported to be rather high. Another successful method involved epoxy grouting a 16-mm diameter threaded rod into a 19-mm hole to a depth of 38 mm. After the epoxy had cured, the rod was pulled using a tension jack reacting against a bearing ring. This method was also reported to have high variability. The study concluded that these two methods had the potential for assessing the strength in existing construction. However, with the exception of later work in the United Kingdom (Domone and Castro 1986), these methods were not subjected to additional study.

Another method was developed by the manufacturer of the LOK-TEST system, and is referred to as the CAPO test (for Cut And PullOut) (Petersen 1984). The method involves drilling a 18-mm diameter hole into the concrete and using a special milling tool to undercut a 25-mm diameter slot at a depth of 25 mm. An expandable ring is placed in the hole, and the ring is expanded using special hardware. Figure 5(c) shows the ring after expansion. The entire assembly used to expand the ring is pulled out of the concrete using the same loading system as for a CIP pullout test. Unlike the other methods discussed above, the CAPO test subjects the concrete to a similar state of stress as the CIP pullout test. Petersen reported that the performance of the CAPO test in laboratory evaluations was similar to the LOK-TEST, but insufficient independent data have been published to verify this finding.

Break-off test — This test measures the force required to break off a cylindrical core from the concrete mass. The method was developed in the early 1970s by R. Johansen at the Cement and Concrete Research Institute in Norway. In cooperation with contractors, Johansen sought a simple, inexpensive, and robust method to measure in-place strength (Johansen 1977,1979).

Figure 6(a) is a schematic of the break-off test. For new construction, the core is formed by inserting a plastic sleeve into the fresh concrete. When the in-place strength is to be estimated, the sleeve is removed. Then a special, hand-operated, hydraulic loading jack is placed into the counter bore, and a force is applied to top of the core until it ruptures from the concrete mass. The hydraulic fluid pressure is monitored with a pressure gage, and the maximum pressure gage reading in units of bars (1 bar = 0.1 MPa) is referred to as the *break-off number* of the concrete. For existing construction, a special drill bit can be used to cut the core and the counter bore.

The break-off test subjects the concrete to a slowly applied force and measures a static strength property of the concrete. The core is loaded as a cantilever, and the concrete at the base of the core is subjected to a combination of bending and shearing stresses. In early work (Johansen 1977, 1979), test results were reported as the *break-off strength*, by computing the flexural stress at the base of the core corresponding to rupture force. In later applications (see review by Naik 1991), the flexural strength was not computed, and the break-off number (pressure gage reading) was related directly to compressive strength. Because the break-off strength is related closely to flexural strength, one would expect the correlation between break-off number and compressive strength to be non-linear, which agrees with what has been reported (Johansen 1977, 1979).

Failure during the break-off test occurs by fracture at the base of the 55-mm diameter core. The crack initiates at the most highly stressed point. It then propagates through the mortar and, in most cases, around coarse aggregate particles located at the base of the core. The particular arrangement of aggregate particles within the failure region would be expected to affect the ultimate load in each test. Because of the relatively small size of the core and the heterogeneous nature of concrete, the distribution of aggregate particles will be different at each test location. Hence, one would expect the within-test variability of the break-off test to be higher than that of other standard strength tests which involve larger test specimens. One would also expect that the variability might be affected by maximum aggregate size and aggregate shape. The developer of the break-off test reported a within-test coefficient of variation of about 9% (Johansen 1979). This value has generally been confirmed by other investigators (Carino 1992).

The break-off test was standardized by ASTM in 1990 (ASTM C 1150).

Pull-off test — This test involves gluing a 50-mm diameter metal disk to concrete using an epoxy adhesive. After the adhesive has cured, the force required to pull off the disk is measured, and a nominal pull-off tensile strength is calculated by dividing the force by the area of the disk. In a valid test, failure occurs in the concrete (see Fig. 6(b)).

The test appears to have been developed independently in the 1970s in the United Kingdom (Long and Murray 1984) and in Austria, where it was called the *tear-off test* (Stehno and Mall 1977). The test can be performed with or without a partial core as shown in Fig. 6(b). The partial core is useful when the concrete has an outer hard shell, e.g., as a result of carbonation. The partial core technique can also be used to assess the bond strength of overlay and repair materials to the base concrete. It has also been suggested that the presence of the groove may result in a more uniform axial stress distribution (Stehno and Mall 1977).

The pull-off test is essentially a direct tension test of concrete. Therefore, the relationship between pull-off strength and compressive strength is affected by those factors which affect the relationship between tensile and compressive strength. These would include the age of concrete, aggregate type and size, curing conditions, and air entrainment (Long and Murray 1984). In comparative studies, it was reported that the pull-off test was more reliable than the BRE internal fracture test (Fig. 5(a)) and slightly more reliable than the Windsor probe test (Long and Murray 1984). The within-test coefficient of variation in laboratory studies has been reported to be in the range of 11 to 17%, depending on the age of the concrete (Long and Murray 1984).

The pull-off test was incorporated into British Standard 1881-Part 207 in 1992 (BSI 1992). This standard includes the following *near-to-surface tests*: internal fracture, pullout, pull-off, probe penetration, and break-off.

Flaw detection methods

This time period also resulted in significant developments in techniques to detect flaws and deterioration in concrete structures. For example, stress wave methods were developed to verify the integrity of cast-in-place foundation piers being constructed with new methods. During this period, the problem of delaminations due to reinforcement corrosion in bridge decks also became a major concern. Methods were needed to permit rapid evaluation of the extent of delaminations. This need led to developments in the use of infrared thermography and ground penetrating radar for flaw detection in concrete structures.

Stress wave propagation methods — Some of the limitations of the pulse velocity method include the need for access to opposite surfaces and the fact that no information is obtained about the depth of a suspected internal defect. In the testing of metals, these limitations were overcome by development of the *ultrasonic pulse-echo* technique. This method uses a high frequency transducer to emit a pulse of ultrasonic waves, the pulse travels through the material and is reflected by defects[6]. The reflected pulse travels back toward the transducer, which also functions as a receiver to detect the surface motion caused by the arrival of the reflected pulse (echo). By measuring the travel time from the start of the pulse until the arrival of the echo and by knowing the wave speed through the material, the depth of the defect can be determined.

Early attempts at using high frequency pulse-echo equipment to test concrete were futile because of the heterogeneity of concrete (Jones 1949a). The presence of paste-aggregate interfaces, air voids, and reinforcing steel results in numerous reflections that obscure those from real defects. In addition, the high frequency pulses (>500 kHz) used to test metals were quickly attenuated in concrete. When lower frequency (<100 kHz) transducers were used, the depth resolution was poor because the transducer would still be acting as a transmitter when an echo arrived from a reactively shallow defect. Because of these inherent problems posed by the *physics* of stress wave propagation, a commercial pulse-echo system has not been developed for concrete.

In the 1960s, some progress was made in the United Kingdom in development of ultrasonic *pitch-catch* systems for measuring pavement thickness (Jones 1963). The pitch-catch approach involves the use of two transducers placed on the same surface to send and receive the ultrasonic pulse. Despite success in the laboratory, the test system was found to be too cumbersome for field use.

In the U.S., an ultrasonic thickness gage was developed in the mid 1960s at Ohio State University (Mailer 1972). The transmitting transducer was a hollow ring with an outer diameter of 460 mm, and a small receiving transducer was located at the center of the transmitter. In field trials, the device proved capable of making accurate measurements of pavement thickness, but it was too cumbersome and lacked the ruggedness of a practical field instrument (Weber et al. 1976).

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[6]With regard to stress wave propagation, a *defect* represents a material having a different wave speed or density than the surrounding material.

In 1963, R. Muenow of the Portland Cement Association reported on a vibration method to measure pavement thickness. The method was an adaptation of the resonant frequency technique used on small specimens. Two transducers were placed side by side on the pavement surface. One transducer was driven by a variable frequency oscillator, and the response of the second transducer was measured. The signals from the oscillator and receiver were displayed on a two-channel oscilloscope. The frequency of the oscillator was varied until a receiver signal reached a maximum amplitude, at which time the slab was vibrating at the thickness mode, resonant frequency. The thickness of the pavement, T, was calculated from the following simple relationship:

$$T = \frac{C_p}{2f} \tag{4}$$

where C_p = pulse velocity
f = thickness mode resonant frequency.

The pulse velocity was measured on the slab using the Soniscope. Based on extensive field trials, Muenow concluded that pavement thickness could be measured with an accuracy of 5%. However, the technique did not receive widespread acceptance, probably because of the time required to perform the test and the skill required to identify when the resonant condition was attained.

Pile integrity tests — In the late 1960s and 1970s, significant progress was made in the application of stress wave propagation methods for integrity testing of piles and drilled shafts. Piles and shafts present two advantages for the use of stress waves compared with pavements: (1) the geometry results in a wave guide that traps much of the incident energy and permits measurements to be made over long distances, and (2) the long lengths allow sufficient time separation so that the duration of incident pulse does not have to be short. The latter advantage permitted using a hammer rather than a transducer to generate the pulse, which in turn allowed high input energies.

One of the first reported studies in the U.S. on the use of stress waves for integrity testing of piles was conducted at the Illinois Institute of Technology (Steinbach and Vey 1975). The basis of the method was established in laboratory studies, and subsequent field studies demonstrated that the technique could be used in the field. The technique involved striking the top of the pile with a hammer and measuring the surface response using an accelerometer connected to an oscilloscope. An electrical circuit was used to trigger the oscilloscope display at the start of the impact. The time from the start of the impact to when the accelerometer responded to the returning echo was measured. The pile length, or defect location, was obtained by multiplying one-half the travel time by the longitudinal wave speed, which was estimated from measurement of the speed of the surface wave created by the impact. The technique was adopted by others and became known generally as the *sonic-echo* or *seismic-echo method*.

In France, a technique known as the *vibration method* was developed in 1966 (Stain 1982). In principle, the vibration method was analogous to the resonant frequency method used by Muenow, except that more quantitative data were recorded and more information was obtained about the pile. In this technique, an electrodynamic vibrator was attached to the top of the pile. A load cell was included between the vibrator and the pile. The response of the pile was measured with a velocity transducer as the frequency of the vibrator was varied. A feedback system adjusts the amplitude of the vibrator so that the maximum dynamic force remains constant. The amplitude of the surface velocity was measured as the frequency of vibration was varied, and a plot of velocity versus frequency was prepared. The motion of the pile surface is the sum of the motion produced by the vibrator and the motion produced by wave reflection from the pile tip (Stain 1982). As the vibrator frequency is varied, these two components will vary from being in phase or out of phase. Thus the graph of velocity versus frequency (called a mobility plot) was composed of a series of peaks and valleys, which provided information about the pile.

With the advent of digital processing oscilloscopes and digital signal analysis methods in the 1970s, the vibration method was replaced by the *impulse-response method* (also called *transient dynamic response*). In this case, a hammer was used to strike a load cell attached to the pile (Higgs 1979). By using the digital records of the time histories of the load cell and velocity transducer, it was possible to calculate the mobility plot. Figure 7(a) is a schematic of the impulse response method and Fig. 7(b) is the idealized mobility plot of a solid pile. The frequency difference (Δf) between peaks can be related to the pile length and wave speed, and the average amplitude (N) of the peak-and-valley region of the plot can be related to the cross-sectional area of the pile (Davis and Dunn 1974).

<u>Infrared thermography</u> — Infrared thermography is a technique for locating near-surface defects by measuring surface temperature. It is based on the principle that a surface emits electromagnetic radiation with an intensity that depends on its temperature. At about room temperature, the radiation is in the infrared region of the electromagnetic spectrum. If there is heat flow into or out of an object (see Fig. 8), the presence of a defect (having different thermal conductivity than the surrounding material) affects the heat flow. As a result, the surface temperature will not be uniform. Thus, by measuring the surface temperature, the presence of the defect can be inferred. In practice, the surface temperature is measured with an infrared scanner which works in a manner similar to a video camera. The output of the scanner is a *thermographic image* of temperature differences.

In civil engineering applications, the method is used primarily to detect corrosion-induced delaminations in reinforced concrete bridge decks. In North America, early research on this application was performed independently in the late 1970s by the Virginia Highway and Transportation Research Council (Clemeña and McKeel 1978) and by the Ontario Ministry of Transportation and Communication (Manning and Holt 1983). Initial studies involved handheld scanners and photographic cameras to record the thermographic images. This was followed by scanning from a boom attached to a truck and by airborne scanning using a helicopter. Although infrared thermography allowed more rapid surveys than the chain drag technique (ASTM D 4580), it was not as accurate as chain dragging in determining the extent of the delaminations (Manning and Holt 1983).

In 1988, ASTM published a standard test method (ASTM D 4788) on the use of the infrared thermography to locate delaminations in exposed and overlaid concrete bridge decks. Additional information on the factors to consider in performing an infrared survey and representative case histories are provided by Weil (1991).

Ground penetrating radar — Radar (acronym for RAdio Detection And Ranging) is analogous to the pulse-echo (or pitch-catch) technique previously discussed, except that pulses of electromagnetic waves (short radio waves or microwaves) are used instead of stress waves. Radar was developed during World War II to provide early warning of approaching enemy aircraft. During the 1960s, the need for a device to locate non-metallic land-mines provided the impetus for the development of high resolution *ground probing* radar. In the 1970s, radar began to be applied in the field of civil engineering. Before reviewing the early civil engineering applications of radar, some of the principles involved are presented, and additional information may be found elsewhere (Morey 1974, Clemeña 1991, Carino 1992).

In civil engineering applications of radar, relatively short distances and small *targets* are involved, therefore, devices were developed to emit very short pulses of electromagnetic waves. For this reason the technique is often called *short-pulse radar*, *impulse radar*, or *ground penetrating radar* (GPR). In this review it will be called GPR. The operating principle of GPR is analogous to that of the stress wave, echo methods that have been discussed. An antenna emits a short duration pulse (on the order of nanoseconds) of electromagnetic (EM) waves. The pulse travels through the underlying material, and when the pulse encounters an interface between dissimilar materials, some of the energy is reflected back toward the antenna (see Fig. 9). The antenna receives the reflected portion of the pulse and generates an output signal. By measuring the time from the start of the pulse until the reception of the echo, the depth of the interface can be determined if the propagation speed through the material is known.

Whereas the reflection of stress waves at an interface depends on differences in elastic constants and density, the amplitude of reflection of the EM pulse at an interface depends on the difference between the relative dielectric constants[7] of the two materials, as shown in Fig. 9. By definition, the relative dielectric constant of air equals 1, and typical values for other materials are given in Fig. 9. The relative dielectric constants for materials such as concrete and soil increase as the moisture content and salt concentrations increase (Morey 1974). At a metal interface, such as between concrete and steel reinforcement, there is complete reflection of the incident pulse.

An important characteristic of a GPR system is the duration of the pulse. To be able to differentiate echoes from closely spaced objects (resolution), the pulse should be short. A short pulse contains high frequency EM waves. The attenuation

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[7]The relative dielectric constant is related to the alignment of charges that occurs in an insulating material when it is placed in an electric field. The speed of propagation of EM waves in an material is the speed in air ($\approx 3 \times 10^8$ m/s) divided by the square root of the dielectric constant.

of the pulse as it travels through the material is a function of the frequency content: higher frequencies result in more attenuation. Thus a compromise must be reached between resolution and penetration ability. The typical frequencies used in concrete evaluations are between 500 and 1000 MHz. For a given frequency, the penetration is affected by the conductivity of the material (Morey 1974). Since moisture in concrete or soil increases conductivity, the penetrating ability decreases with increasing moisture content.

Interpretation of the results of a radar scan poses difficulties, and several approaches have been used to simplify the process. A common method of presenting the results of a radar scan is by using a graphic (or facsimile) recorder. The graphic recorder operates on the principle of threshold plotting, which is illustrated in Fig. 10. Figure 10(b) is a schematic of the antenna output as a function of time, when the antenna is located over a void in a concrete slab (Fig. 10(a)). When the signal amplitude exceeds a threshold value (selected by the operator), the stylus of the graphic recorder draws a line on the paper. The length of the line corresponds to the time interval during which the threshold value is exceeded. Thus the graphic recorder transforms the time domain waveform into a series of dashes, as shown in Fig. 10(b). Note that each echo is associated with two dashes; the number of dashes depends on the number of cycles in the emitted pulse and the threshold level. As the antenna is scanned along the surface, the paper feeds through the recorder, and the dashes result in a series of horizontal bands, which correspond to the reflecting interfaces. In effect, the recorder produces a cross-section view, or a *sub-surface profile*, of the structure.

The development of commercial short-pulse radar systems appears to have begun in 1970, and its early uses were for geophysical investigations to locate buried pipes and tanks (Morey 1974). This was followed by studies to examine whether voids beneath airfield pavements could be detected.

The first large-scale application to the evaluation of bridge decks appears to have been performed by the Port Authority of New York and New Jersey (Cantor and Kneeter 1978). The radar equipment was mounted on a van which drove across the bridges at about 10 km/h, and the antenna was directly over a wheel track. At the pulse repetition rate that was used, every 75 mm of roadway was documented by a waveform. The antenna output was connected to an oscilloscope and tape recorder to store the waveforms. It was found that the simplest way to examine the voluminous amount of data was to record the oscilloscope traces on 35-mm film using a *topographic* or *waterfall* format, in which each waveform was displayed at a fixed distance from the preceding one. With this form of display, it was possible to identify differences that were indicative of deteriorated concrete.

The encouraging results obtained from the initial field tests led to the start of basic studies to gain an understanding of the effects of different conditions on the radar response (Alongi et al. 1982). In addition, other methods were investigated for coping with the large amount of data and permit analysis without needing a highly experienced engineer (Cantor and Kneeter 1982). One of the successful methods that was developed is a form of pattern recognition called *cluster analysis*. Each waveform was digitized and, by computer analysis, was assigned to a cluster of similar waveforms.

The clustering resulted in groups of traces that were characterized as "good", "distressed" and "intermediate". It was found that the distressed regions in the bridge decks could be determined with 90% reliability (Cantor and Kneeter 1982).

Subsequently, other investigators began to explore the potentials of GPR. Researchers from the Virginia Highway and Transportation Research Council demonstrated the capability to locate voids beneath sidewalks by using a graphic recorder (Clemeña and McGhee 1980). This was followed by applications on bridge decks with asphalt concrete overlays (Clemeña 1983). In 1978, Researchers at Georgia Institute of Technology began analytical and laboratory studies aimed at locating voids beneath pavements (Steinway et al. 1981). The Ontario Ministry of Transportation and Communications began to investigate the use of radar on asphalt-covered bridge decks in 1980 (Manning and Holt 1983). Eventually the system called D.A.R.T. (Deck Assessment by Radar and Thermography) was developed, which was a vehicle incorporating both radar and infrared thermography equipment (Manning and Holt 1985).

THE 1980's and THE PRESENT

During the last decade, numerous workshops, conferences, symposia, and meetings on the subject of *NDT for civil structures* have been conducted, and significant research funds have been spent to enhance the state-of-the-art. The impetus has come largely from the *infrastructure* issue. It has become obvious that reliable tools are needed for accurate assessment of existing conditions so that the most economical strategy of repair/replacement can be selected.

There has also has been an increased appreciation of the importance of in-place testing during construction. The need to assure safety during today's accelerated construction schedules makes in-place testing a necessity. Failure to provide this assurance can result in drastic consequences, such as the tragic construction failure in 1978 of a cooling tower in Willow Island, West Virginia. The accident resulted in 51 deaths, making it the mostly costly construction failure in terms of human lives. Subsequent investigations revealed that the most probable cause of the failure was inadequate concrete strength to support the construction loads (Lew 1980). Had any of the available in-place test methods been used, the failure would probably not have occurred. In response to the need for guidelines on the use of in-place tests, the American Concrete Institute established a committee on nondestructive testing in 1982. The committee issued a report in 1989 (ACI 228R-89) that provided practical information on in-place testing to supplement the ASTM standards.

Another factor that has promoted interest in in-place testing is the introduction of *performance related specifications*, particularly in highway construction. Such specifications emphasize *in-place performance* and the need for reliable in-place test methods is obvious.

In a recent paper, the author reviewed some of the significant advancements in the areas of in-place strength and flaw detection methods (Carino 1992a). To avoid duplication, only a brief summary of these advancements is provided here. Additionally, recent information on these methods may be found in a textbook by Bungey (1989) and a handbook edited by Malhotra and Carino (1991).

In-place strength methods

Pullout test failure mechanism — The pullout test has proven to be one of the most reliable techniques for estimating in-place strength during construction. Ever since the test was first described by Skramtajev (1938), there has been speculation about the underlying failure mechanism and why there is good correlation with compressive strength. Skramtajev noted correctly that the test subjects the concrete to a combination of tensile and shearing stresses. Kierkegaard-Hansen (1975) tried to relate the shape of the extracted conical fragment to the intact cones often observed at the ends of cylinders tested in compression. Malhotra and Carette (1980) suggested that the pullout strength was related to the direct shear strength of concrete.

In 1976, the first analytical study to explain the relationship between compressive strength and the ultimate pullout load was published (Jensen and Braestrup 1976). However, there were questions about the validity of the assumptions used in that analysis. Hence, in the early 1980s, a series of independent analytical and experimental studies were performed (see Carino 1991b for a review). From these studies, it was established that the pullout test subjects the concrete to a nonuniform, three-dimensional state of stress. It was also shown that the failure process involves two circumferential crack systems: a stable system that initiates at the insert head at about 1/3 of the ultimate load and propagates into the concrete along a large apex angle; and a second system which propagates with increasing load and defines the shape of the extracted cone. However, there was no consensus on the failure mechanism at the ultimate load. Some proposed that ultimate load occurred as a result of compressive failure along a *strut* running from the bottom of the bearing ring to the insert head. This mechanism was used to explain why there is good correlation between pullout strength and compressive strength. Others proposed that the ultimate load is carried by aggregate interlock across the conical crack system, and that the ultimate load is reached when sufficient aggregate particles have been pulled out of the mortar matrix. In this case, it was argued that there is correlation between pullout strength and compressive strength because both properties are controlled by the tensile strength of the mortar.

Despite the lack of agreement on the exact failure mechanism, it has been shown that the pullout strength has good correlation with the compressive strength of concrete and that the test has good repeatability. In a review of published data, ACI Committee 228 recommended a coefficient of variation of 8% for the pullout test (ACI 228, 1989).

Maturity method — Beginning in the mid 1970s, there was renewed interest in the maturity method, and both practical and research papers were published (see Carino 1991a). In the 1980s, there was further research to gain a better understanding of the basis and reliability of the technique. Some of this work was carried out in the U.S. at the National Institute of Standards and Technology (NIST) (formerly the National Bureau of Standards). The research at NIST reiterated the inherent limitations of the traditional Nurse-Saul maturity function (Eq. (1)) and emphasized the superiority of the maturity function proposed by Freiesleben Hansen and Pedersen (Eq. (3)).

The work at NIST also led to a procedure to obtain the *apparent activation energy* of a given cementitious mixture (Carino 1982). The procedure is based on determining the effect of curing temperature on the rate constant for strength development. The rate constant is related to the curing time needed to reach a certain fraction of the long-term strength, and it is obtained by fitting an appropriate equation to the strength versus age data acquired under isothermal curing. By using this procedure, the *apparent activation energy* was determined for concrete and mortar specimens made with different cementitious materials (Tank and Carino 1991, Carino and Tank 1992). The results are summarized in Table 2. For concrete with water-cement ratio (W/C) = 0.45, the *apparent activation energy* ranged from 30 and 64 kJ/mole; while for W/C = 0.60 it ranged from 31 to 56 kJ/mole, depending on the type of cement and additives.

The NIST research also lead to the following alternative to the Arrhenius equation for computing equivalent age:

$$t_e = \sum_0^t e^{B(T - T_r)} \Delta t \tag{5}$$

where B = temperature sensitivity factor, 1/°C
T = average concrete temperature during time interval Δt, °C
T_r = reference temperature, °C

It was shown that Eqs. (3) and (5) would result in similar values of equivalent age (Carino 1992a). However, the author believes Eq. (5) has the following advantages over Eq. (3):

- The temperature sensitivity factor, B, has more physical significance compared with the *apparent activation energy*: for each temperature increment of 1/B, the rate constant for strength development increases by a factor of approximately 2.7.
- Temperatures do not have to be converted to the absolute scale.
- Eq. (5) is a simpler equation.

Recent work at NIST has demonstrated the applicability of the maturity method to high performance concrete mixtures, which are characterized by low ratios of water to cementitious materials and the use of silica fume (Carino et al. 1992).

In 1987, ASTM adopted a standard practice on the use of the maturity method to estimate concrete strength (ASTM C 1074). The maturity method is also used in ASTM C 918 for estimating later-age potential cylinder strength based on measured early-age strength.

Statistical methods — Due to the need for in-place testing to provide safeguards against construction failures, such as the one at Willow Island, and due to the work of researchers, such as Malhotra (1976) and colleagues, in-place testing became a *recognized* alternative to testing field-cured cylinders for assessing concrete strength during construction. In the 1983 edition of the ACI Code, the following sentence was added to section 6.2.1.1 dealing with form removal[8]:

> *"Concrete strength data may be based on tests of field-cured cylinders or, when approved by the Building Official, on other procedures to evaluate concrete strength."*

The Commentary to the Code listed acceptable alternative procedures, and further stated that these alternative methods *"require sufficient data using job materials to demonstrate correlation of measurements on the structure with compressive strength of molded cylinders or drilled cores."* Thus, to use the alternative methods, an empirical relationship had to be developed to convert in-place test results to equivalent compressive strength values. In addition, a procedure was needed to analyze in-place test results so that compressive strength could be estimated with a high degree of confidence. Both steps require statistical analysis of test data.

During the 1960s and 1970s, traditional methods of ordinary least squares (OLS) analysis were used to establish best-fit regression equations and their confidence limits (Natrella 1963). In the 1980s, simple procedures were used to estimate lower confidence limits of estimated in-place strength (Bickley 1982b, Hindo and Bergstrom 1985). However, it was recognized that the existing methods were not statistically rigorous and the stated confidence levels were not accurate (Stone et al. 1986). One of the major deficiencies in using OLS to establish correlations is that OLS assumes the X-variable (the in-place test result) has no measurement error. In fact, the within-test coefficient of variation of in-place tests were known to be two to three times those of compression tests of cores or cylinders. To overcome these deficiencies, a study was undertaken at NIST to develop a rigorous method for obtaining the correlation equations and estimating the lower confidence limit of the in-place strength (Stone and Reeve 1986). The procedure used a method for least-squares fitting which accounted for error in the X-variable (Mandel 1984). The rigorous method was incorporated into the report of ACI 228 (ACI 228, 1989), but it found little use due to its complexity. Subsequently, a simplification of the rigorous method was proposed, which could be implemented by using a spreadsheet program (Carino 1992b). This simplified method will be incorporated into the revision of the ACI 228 report.

During the late 1980s and early 1990s, A. Leshchinsky published a series of papers on statistical methods for in-place tests, which were based largely on work at the Institute for Research of Building Structures, in Kiev. These papers compared practices in different countries with those in the former Soviet Union, which had a long history in using in-place methods for quality control in precast plants.

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[8] "Building Code Requirements for Reinforced Concrete," ACI 318-83, American Concrete Institute, Detroit, MI.

In a review of various national standards, Leshchinsky (1990) concluded that: (1) greater numbers of in-place tests are required compared with tests of standard specimens, which makes in-place testing economically unattractive; (2) the number of test specimens to establish the correlation and the number of in-place tests on the structure have been established arbitrarily, based on the experiences of those writing the standards; and (3) the number of replicate tests to establish the correlation differ from the replications at a test location in a structure. Procedures were presented for selecting in-place tests based on consideration of cost and reliability of the estimated strength, and recommendations were made to reduce the cost of in-place testing.

In another paper, procedures for developing correlations were discussed, and criteria were suggested for verifying the correlation at periodic intervals (Leshchinsky and Leshchinsky 1991). The notion of a *stable* correlation was introduced. This refers to a correlation that is little affected by changes in concrete composition and the construction process. It was noted that methods which have a close connection between concrete strength and the quantity measured by the in-place test tend to have more stable correlations, but they also tend to be more costly than the methods which don't have this close connection. Pullout, break-off, and pull-off tests were identified as possessing stable correlations. It was also shown that the correlations may be affected by the location on the test specimens (top, middle bottom) where the in-place test is performed (Leshchinsky 1991a).

Leshchinsky also discussed the factors to consider in deciding whether combined methods are justified (Leshchinsky 1991b). In addition, the following situations were identified where the combined use of a reliable, expensive method could be combined with a less expensive but less reliable method to achieve an overall savings in testing:

- The reliable method is used to *calibrate* the strength correlation of the less reliable method; a correction factor is determined and applied to the strength estimated by the less reliable method.
- The less reliable method is used to identify those areas of lower quality concrete where the reliable tests would be performed.
- For new construction, a less reliable method is used to determine when tests by the more reliable method should be performed.

There have been few instances in North America where in-place testing has been used for acceptance testing. The major barrier is the lack of a consensus-based, statistical procedure for this application. Leshchinsky (1992) discussed some of the considerations in using in-place testing for acceptance testing. Because the strength of the actual concrete in the structure is being measured, the acceptable in-place strength could be less than the design strength. Provisions in existing national standards that allow acceptance based on in-place tests were reviewed. In some of these standards, the required in-place strength depends on the number of tests and the variability of the in-place results. For high variability, the required in-place strength may exceed the design strength. These national standards should be critically reviewed and the methods that strike a balance between practicality and statistical rigor should be considered for incorporation into North American practice.

In 1992, a new RILEM technical committee on in-place testing (TC 126-IPT) was organized with a goal of developing technical recommendations on the use on in-place test methods that would be applicable to any appropriated procedure. The recommendations would deal with development of the correlation and estimating a lower confidence limit for the in-place concrete strength.

Stress wave propagation methods

During the 1980s important advances were made in the application of methods based upon stress wave propagation. Alexander and Thornton (1989) reported on the development of a new *pitch-catch* ultrasonic system and on feasibility studies of a true *pulse-echo* system. However, the area in which there was the greatest advancement was in the application of impact methods. This progress was made possible largely by the developments in computer technology that permitted exploitation of the power of digital signal analysis.

Impedance testing — Integrity testing of piles became widespread in Europe and portable field systems for *seismic-echo* testing were developed in Europe and the U.S (Davis and Hertlein 1991, Rausche et al. 1991). A new approach for signal analysis of *impulse-response* tests was developed in Europe (Davis and Hertlein 1991). The technique became known as *impedance testing*, and it allowed test results to be reported in the form of a cross-section of the pile or shaft, rather than the usual time domain waveforms and frequency domain transformations. Thus the *wiggles* that were common to pile integrity test reports were replaced by *pictures* that could be understood by non-experts.

Impact-echo method — In 1983, research began at NIST to develop a method for flaw detection in slab-like structures. The method that was eventually developed became known as the *impact-echo method* (Sansalone and Carino 1986). Figure 11(a) is a schematic of the method. A small impactor produces a short duration impact which propagates into the member along spherical wavefronts as P- and S-waves. In addition, a surface wave (R-wave) travels away from the impact point along a circular wavefront. A displacement transducer is located close to the impact point. As a result, the transducer response is dominated by the P-wave reflected from the opposite boundary or an internal defect.

A key feature leading to the success of the impact-echo method was the use of *frequency analysis* instead of the traditional time domain analysis to interpret the recorded waveforms (Carino et al. 1986). The principle of frequency analysis is that the P-wave produced by the impact undergoes multiple reflections between the test surface and the reflecting interface. Thus a resonant condition is created in which the frequency, f, of the P-wave arrival is given by

$$f = \frac{C_p}{2\,D} \qquad (6)$$

where C_p is the P-wave speed and D is the depth of the reflecting interface. This frequency is called the *thickness frequency*.

To apply frequency analysis, the recorded waveform is transformed into the frequency domain by using the fast Fourier transform technique. The computed amplitude spectrum shows the dominant frequencies in the waveform. For slab-like structures, the thickness frequency will usually be the dominant peak in the spectrum. The value of the peak frequency in the amplitude spectrum can be used to determine the depth of the reflecting interface by expressing Eq. (6) as follows:

$$D = \frac{C_P}{2f} \tag{7}$$

Figure 11 illustrates the use of frequency analysis of impact-echo tests. Figure 11(b) shows the amplitude spectrum from a test over a solid portion of a 0.5-m thick concrete slab. There is a frequency peak at 3.42 kHz, which corresponds to multiple P-wave reflections between the bottom and top surfaces of the slab. Using Eqs. (6) and solving for C_p, the P-wave speed is calculated to be 3420 m/s. Figure 11(c) shows the amplitude spectrum from a test over a portion of the slab containing a disk-shaped void. The peak at 7.32 kHz results from multiple reflections between the top of the slab and the void. Using Eq. (7), the calculated depth of the void is 3420/(2 x 7320) = 0.23 m, which compares favorably with the known distance of 0.25 m.

In the initial work leading to the impact-echo method (Sansalone and Carino 1986), it was learned that the duration of the impact (the *contact time*) was critical for success of the method. The contact time determines the frequency content of the stress pulse generated by the impact (Carino et al. 1986). As an approximation, the highest frequency component of significant amplitude equals the inverse of the contact time. In order to accurately locate shallow defects, the stress pulse must have components at frequencies greater than the frequency corresponding to the flaw depth (Eq. (6). For example, for a P-wave speed of 4000 m/s, a pulse with a contact time shorter than 100 μs is needed to determine the depth of defects shallower than about 0.2 m.

The early work showed that the impact-echo method could detect a variety of defects, such as voids and honeycombed concrete in members, delaminations in bare and overlaid slabs, and voids in tendon ducts (see Sansalone and Carino 1991, or Carino 1992a for references). Experimental studies were supplemented with analytical studies to gain a better understanding of the propagation of transient waves in bounded solids with and without flaws (Sansalone and Carino 1987, Lin et al. 1991a, 1991b). Recent work (Lin and Sansalone 1992a, 1992b, 1992c) has extended the application of the method to prismatic members, such as columns and beams. It has been found that reflections from the perimeter of these members cause complex modes of vibration. As a result, the amplitude spectra have many peaks, and the depth of the member is not related to the dominant frequency in the spectrum according to Eq. (7). Nevertheless, it has been shown that defects can still be detected within beams and columns, and field applications have been reported (Sansalone and Poston 1992).

As with most methods for flaw detection in concrete, experience is required to interpret stress-wave test results. A recent advance in the interpretation of impact-echo results from tests of slab-like structures has been the application of an artificial intelligence technique known as a *neural network* (Pratt and Sansalone 1992). In this technique, a computer program is *trained* to recognize amplitude spectra associated with flawed and unflawed structures. After this *training*, the program can be used to classify the results of tests on a structure under investigation. This technique was incorporated into the development at Cornell University of the first commercial impact-echo test system (Pratt and Sansalone 1992).

Spectral analysis of surface waves — In the late 1950s and early 1960s, Jones reported on the use of *surface waves* to determine the thickness and elastic stiffness of pavement slabs and of the underlying layers (Jones 1955, Jones 1962). The method involved determining the relationship between the wavelength and velocity of surface vibrations as the vibration frequency was varied. Apart from the studies reported by Jones there seems to have been little use of this technique for testing concrete pavements. In the early 1980s, however, researchers at The University of Texas at Austin began studies of a surface wave technique that involved an impactor instead of a steady-state vibrator. Digital signal processing was used to develop the relationship between wavelength and velocity. The technique was called *spectral analysis of surface waves* (SASW) (Heisey et al. 1982, Nazarian et al. 1983).

Figure 12(a) shows the configuration for SASW testing (Nazarian and Stokoe 1986a). Two receivers, separated by a known distance, monitor the vibrations due to the surface wave (R-wave) produced by the impact. The R-wave contains a range of frequency (or wavelength) components. As in all impact methods, the frequency range depends on the contact time of the impact; a shorter contact time results in a wider range. The longer wavelength (lower frequency) components penetrate more deeply, and this is why surface wave methods can be used to gain information about the underlying layers in layered systems. The propagation speed of these different components is affected by the wave speed of the layers through which the components propagate.

The goal in SASW testing is to determine the speeds of the various frequency (or wavelength) components. The speeds are calculated from the time it takes each component to travel from the first to the second receiver. Travel times are based on the distance between the receivers and the phase differences of the frequency components arriving at the receivers (Nazarian and Stokoe 1986b). The phase differences are obtained from the phase spectrum of the *cross power spectrum* of the received signals (see Fig. 12(b)). The wavelength of each component is calculated, and a plot of speed versus wavelength is obtained as shown in Fig. 12(b). Such a plot is called a *dispersion curve*, and it summarizes the results of an SASW test.

After the *experimental* dispersion curve is obtained, a process called *inversion* is used to obtain the stiffness profile at the test site (Nazarian and Stokoe 1986b). The site is modelled as layers of varying thickness. Each layer is assigned a density and elastic constants. Using these assumed properties, the surface motion at the location of the receivers is calculated using surface wave propagation theory. The calculated responses are subjected to the same signal processing technique as used for the test data, and a *theoretical* dispersion curve is obtained. The theoretical and experimental dispersion curves are compared. If the curves match, the problem is solved and the assumed stiffness profile is correct. If there are significant discrepancies, the properties of the assumed layered system are changed, and a new theoretical dispersion curve is calculated. This process continues until there is agreement between the theoretical and experimental curves.

The SASW method has been used to determine the stiffness profiles of soil sites and of flexible and rigid pavement systems (Nazarian and Stokoe 1986b, Rix and Stokoe 1989). Recently the method has been extended to the measurement of changes in the elastic properties of concrete slabs during curing (Rix et al. 1990).

Infrared thermography and radar — In the 1980s there was considerable progress in developing field systems to permit rapid inspection of pavements and bridge decks, and *routine* inspection services using these methods became available (Kunz and Eales 1985, Holt and Eales 1987, Weil 1991). With the advent of the personal computer, new techniques were developed for analyzing data. However, it became evident that the instrumentation was outpacing the understanding of the *physics* that was involved in the different testing situations that were encountered. Hence, studies were undertaken to gain a better understanding of the interactions of electromagnetic waves with the type of defects encountered in concrete structures, including the effects of reinforcing bars (e.g., Mast et al. 1990, Maser and Roddis 1990).

In 1988, a standard test method was adopted by ASTM for detecting delaminations in bridge decks by using infrared thermography (ASTM D 4788). The standard provides practical guidance on the necessary conditions to be able to detect delaminations in bare and asphalt-covered bridge decks. The results of the inspection are usually reported in terms of delaminated area and percentage of delaminated area. After the delaminated areas are identified in the infrared images, the visible video images should be compared to assure that apparent temperature differences were not due to changes in surface textures or stains (Kunz and Eales 1985). The ASTM standard states that 80 to 90% of the delaminations in a bare bridge deck can be located by this method. It has also been found that the inspection of the same deck by four different operators resulted in a variation of ±5% of the known delaminated area.

In France, a prototype radar system was developed that used tomographic techniques to reconstruct the reinforcement layout in concrete (Pichot and Trouillet 1990). The system was called the *microwave camera*, and it involved an array of receivers to record the reinforcing bar echoes from different directions. The received signals were processed with a mini-computer to create an image of the bars in the concrete. For improved resolution, the emitted pulses were in the range of 7 to 13 GHz, which limited the depth of penetration to about 80 mm.

Due to difficulties in using GPR in reinforced concrete because of the strong reflections produced by the bars, standardized test procedures for flaws have not been devised. However, an ASTM standard was adopted in 1987 (ASTM D 4748) to measure the thickness of the upper layer of a multi-layer pavement system. Basically, the technique involves measuring the transit time of the pulse though the pavement layer, and calculating the thickness from the propagation speed through the top layer. The accuracy of the calculation is dependent on the accuracy of the value of the relative dielectric constant. Thus it is necessary to take occasional cores to determine the appropriate value for the pavement materials. The user is cautioned against using the method on saturated concrete, because of the high attenuation and limited penetration of the pulse. It is stated that inter-operator testing of the same materials resulted in thickness measurements within ±5 mm of the actual thickness. Finally, it is noted that reliable interpretation of received signals can only be performed by an experienced data analyst.

While the majority of the applications of GPR have dealt with locating reinforcing bars in structures, locating delaminations in bridge decks, and measuring the thickness of pavement layers, there are other potential uses. Since the dielectric properties of a material like concrete are strongly dependent on the moisture content, microwave measurements can be used to monitor the progress of hydration (Clemeña 1991). This is made possible because the relative dielectric constant of free water is much higher than that of chemically bound water. The potential application of microwave measurements to determine water content of fresh concrete was also investigated (Clemeña 1991)

THE FUTURE

The previous sections have highlighted the major developments in the history of nondestructive testing of concrete. The paper concludes with personal views on the future directions in this field.

In-place strength

There has been impetus in the highway construction industry to change from *prescriptive* construction specifications to *performance related specifications* (PRS). The argument is that the change will result in innovation by contractors and in better quality products with reduced life-cycle costs (Afferton et al. 1992). An important element of performance related specifications is a payment adjustment schedule based on the quality achieved in the actual structure. Currently, drilled cores are used to measure the thickness and compressive strength of concrete pavements. Clearly, reliable in-place tests can be used to advantage under PRS.

In addition to the movement towards PRS, there is a greater awareness of the importance of proper curing of the *covercrete* (name applied to the concrete covering the reinforcement) to achieve durable concrete. In-place tests are needed to provide assurance that adequate curing has been accomplished. Finally, as has been mentioned, the rebuilding of the Nation's aging infrastructure requires reliable methods to assess existing conditions. Thus in-place test methods are expected to play a critical role in future concrete construction.

Test methods — The ultimate objective is the development of a technique which is reliable, simple to use, and inexpensive. Despite many years of research, there is no method that satisfies all of these criteria. However, when properly used, existing techniques have proven successful on actual construction projects. Therefore, ideas for new methods should be nurtured only if they appear to offer genuine advancements over existing methods. Exploratory research should be designed so that the performance of a prospective test method is evaluated over a wide range of conditions. To avoid drawing faulty conclusions about the suitability of a new method, a statistically-based *screening* test program should be used to permit valid conclusions to be drawn about the effects of different factors on within-test repeatability and the stability of the correlation.

The need is greatest for in-place tests to estimate strength in *existing construction*. The major problem with current technology is the expense required to develop the correlation for the specific concrete under investigation. To obtain a reliable correlation, a relatively large number of cores, representing a wide range of concrete strength, are needed. Unless, the project is large, the cost to establish the correlation will be a barrier to the use of in-place tests. It would be desirable to have a test method that can be readily applied to existing construction which has a stable correlation, i.e., one which is little affected by the actual materials present in the concrete. History suggests that methods which measure some static strength property are likely to be the best candidates for this purpose. This review has mentioned some potential methods, but more field performance data are needed to assess their true capabilities.

Statistical methods — There is a misconception that the use of in-place strength methods simply involves performing tests on the structure, computing and average of the results, and using some formula to convert the average to an equivalent compressive strength. Such an approach fails to account for the uncertainties that are involved in the process; these include: (1) the uncertainty of the correlation, (2) the uncertainty of the average value of the in-place results, and (3) the variability of the actual in-place strength of the concrete. These factors must be taken into account to arrive at a reliable estimate of the in-place strength.

While standard test methods exist for performing in-place tests, there is no standard practice in North America for evaluating the results. ACI Committee 228 and RILEM TC-126 IPT are taking steps to develop such practices. The technique that is finally adopted must be founded on sound technical principles, otherwise it will be of no value. However, the technique must be sufficiently simple so that it can be used by practitioners. Research is needed to understand the true reliability of various statistical approaches that have been suggested or are being used to estimate in-place test strength. This may be achieved by extensive computer simulations and final verification on actual construction projects.

Flaw detection

Barriers and opportunities — Despite the advancements in other fields, progress towards the development of reliable and relatively easy-to-use nondestructive flaw detection methods for concrete has been slow. Some of the reasons for this condition are the difficult technical problems posed by the inherent characteristics of concrete structures and the rugged and varying field conditions that are encountered. These difficult problems can be solved only by multi-disciplinary research teams involving experts in concrete technology, electrical and mechanical engineering, physics, mathematic, and computer science.

On the positive side, the advancements in the microelectronics industry have been major factors leading to recent successes in the development of methods for flaw detection. The continued development of compact, powerful computers and techniques for digital signal analysis have expanded the scope of what is achievable. However, true progress can not be realized solely by developing more powerful and sophisticated hardware. There must be parallel advancements in the understanding of the physical processes involved in the test methods. Only then will it be possible to develop reliable test procedures.

Test methods — Ground penetrating radar and stress wave propagation methods offer opportunities for powerful new techniques. Among these, stress wave propagation methods will likely play a dominant role in terms of flaw detection. This is because stress wave methods are inherently sensitive to changes in density and elastic constants, which are the major characteristics which distinguish flawed from sound concrete. The major drawback to stress wave methods is that they are *point* tests, which makes the inspection of a large structure a slow process. A major breakthrough would be the development of a technique to permit *scanning*, as is currently possible with radar. Analytical studies should go hand-in-hand with hardware development to assure a complete understand the measurements being made.

Radar is well suited for locating reinforcing steel because of the high amplitude reflections that occur at the concrete-steel interface. There is the potential for developing a device that will permit the determination of the configuration of steel in a member. The *microwave camera* developed in France has proven the feasibility of the approach, but difficult obstacles have to be surmounted before such a device can be used in the field. However, is it here where the continuing breakthroughs in computer technology may permit the needed advancements.

New approaches are needed to assist users in dealing with the voluminous amount and complexity of data recorded during an investigation. The need for a highly trained individual to be able to use these methods has to be eliminated. Artificial intelligence methods, such as neural networks and expert systems, should be exploited to assist in the effective use of these methods.

Standards development — Standards are essential to achieve acceptance and proper use of flaw detection methods. This requires an understanding of what can and what cannot be measured reliably by the various techniques. The uncertainty associated with the test methods needs to be quantified. The cost of research to provide the basis for standard test methods is not trivial. Providers of research funding must consider this type of research as being equally important to the *leading edge* research that they have traditionally funded. The development of standards is of paramount importance for effective transfer of the latest technology.

New technology

The fields of *smart structures* and *optical fibers* are considered by some to be the forward looking areas in regard to concrete construction. The term *smart structure* refers to a structure that can sense its environment and take appropriate remedial actions. At present, there have only been conceptual ideas of how this technology might be applied to concrete. For example, it has been suggested that capsules could be embedded into concrete which would provide a substance to *heal* cracks that might develop during the service life. However, the author is not aware of any successful feasibility studies of such conceptual ideas.

Optical fibers provide the opportunity for a new generation of sensors that can be embedded in concrete. The notion is that such sensors can be used to monitor "the long-term health" of a structure or "structural integrity" (Huston et al. 1992, Ansari 1992). Laboratory and field demonstrations have demonstrated that optical fiber sensors can be used to measure internal strains or identify the location of a crack. However, these same studies have not demonstrated how such measurements can lead to the goals of monitoring the structural performance of real structures.

The author is not convinced that these new technologies offer the potential for revolutionary advancements that are claimed by their proponents. However, well thought out research programs should be carried out to explore whether real advancements are possible.

Education

The test methods that have been discussed are *indirect*, i.e., the value the property or the characteristic of interest is inferred from the measurement of some other property or characteristic. Proper use of these methods requires a thorough understanding of the underlying principles and the complicating factors that affect the measurement. Some of the misgivings that have arisen about NDT methods did not occur because the methods were inadequate, but because they were used incorrectly or were used for a purpose for which they were not intended. There is a need to educate potential users about the correct procedures for using existing methods. Such education should cover the underlying principles of the method, the factors affecting the results, and the proper use of the instrument. This education must go beyond information provided in the usual operating manuals provided by manufacturers. It is obvious that the methods that have been discussed in this paper have different levels of complexity, and they are not all suited for use by a *technician* who has been instructed only in the proper sequence to "push the buttons." The proper method of education is through formal courses devoted to this subject.

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- C 215 Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens
- C 597 Standard Test Method for Pulse Velocity Through Concrete
- C 803 Standard Test Method for Penetration Resistance of Hardened Concrete
- C 805 Standard Test Method for Rebound Number of Hardened Concrete
- C 900 Standard Test Method for Pullout Strength of Hardened Concrete
- C 918 Standard Test Method for Measuring Early-Age Compressive Strength and Projecting Later-Age Strength
- C 1074 Standard Practice for Estimating Concrete Strength by the Maturity Method
- C 1150 Standard Test Method for the Break-Off Number of Concrete

1992 Annual Book of ASTM Standards Vol. 04.03

- D 4580 Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding
- D 4788 Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography
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TABLE 1 — NONDESTRUCTIVE AND IN-PLACE TESTS

In-place tests to estimate strength	Nondestructive tests for integrity
Break-off	Ground penetrating radar
Indentation	Infrared thermography
Maturity method	Pulse velocity
Penetration	Radioisotope and X-ray methods
Pullout	Resonant frequency
Pull-off	Stress wave propagation
Pulse velocity	
Rebound	

TABLE 2 — APPARENT ACTIVATION ENERGIES IN kJ/mol OBTAINED FROM ISOTHERMAL STRENGTH DEVELOPMENT OF CONCRETE AND MORTAR SPECIMENS (CARINO AND TANK 1992)

Cementitious Materials	W/C = 0.45		W/C = 0.60	
	Concrete	Mortar	Concrete	Mortar
Type I	63.6	61.1	48.0	43.6
Type II	51.1	55.4	42.7	41.1
Type III	43.6	40.1	44.0	42.6
Type I + Fly Ash	30.0	33.1	31.2	36.6
Type I + Slag	44.7	42.7	56.0	51.3
Type I + Accelerator	44.6	54.1	50.2	52.1
Type I + Retarder	38.7	41.9	38.7	34.1

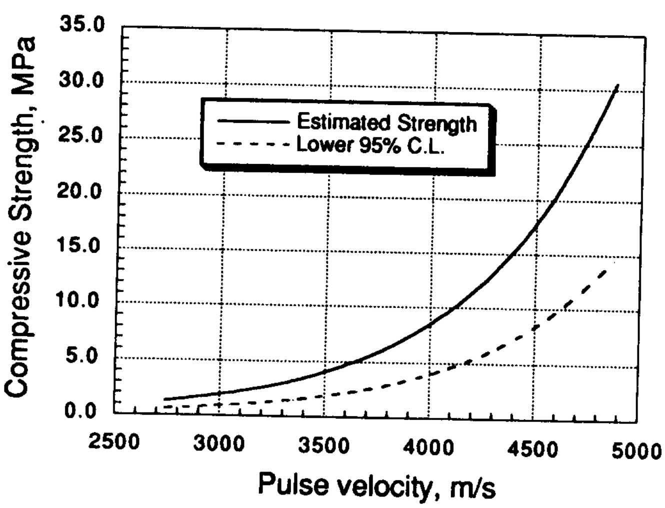


Fig. 1—Compressive strength versus pulse velocity relationship based upon 46 mixtures made with the same aggregate (Parker 1953)

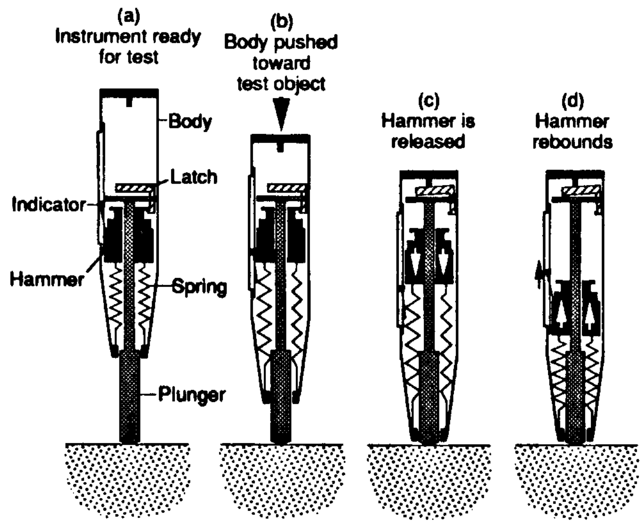


Fig. 2—Schematic cross section of Schmidt rebound hammer showing operating principle

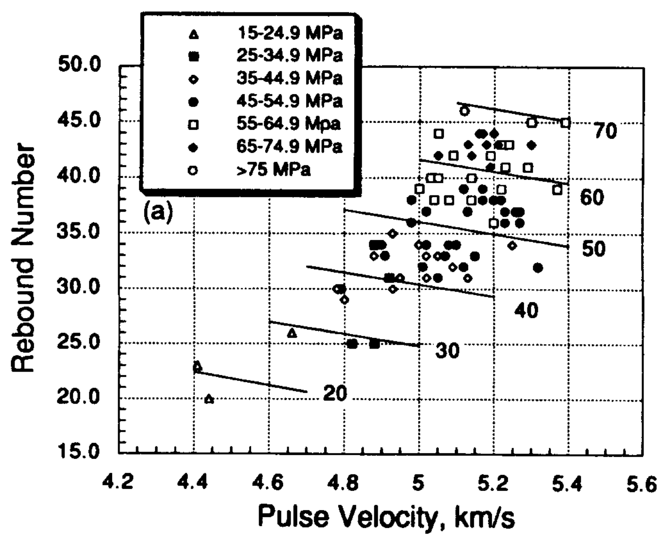


Fig. 3a—Data using the combined method of rebound hammer and pulse velocity (Cianfrone and Făcăoaru 1979): equal strength lines based on linear multiple regression equation

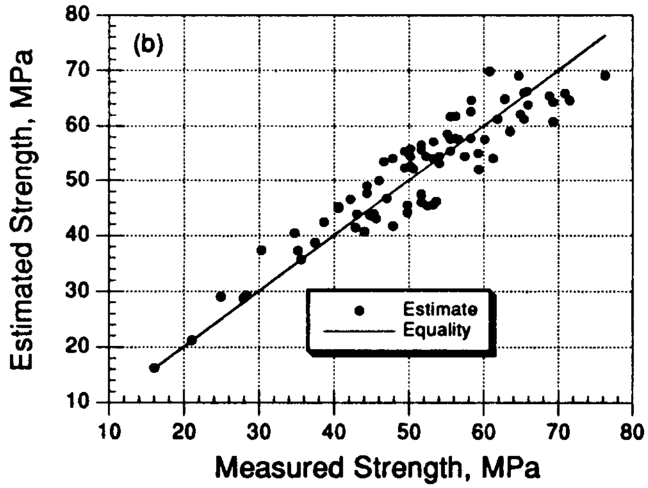


Fig. 3b—Data using the combined method of rebound hammer and pulse velocity (Cianfrone and Făcăoaru 1979): comparison of estimated and measured strength

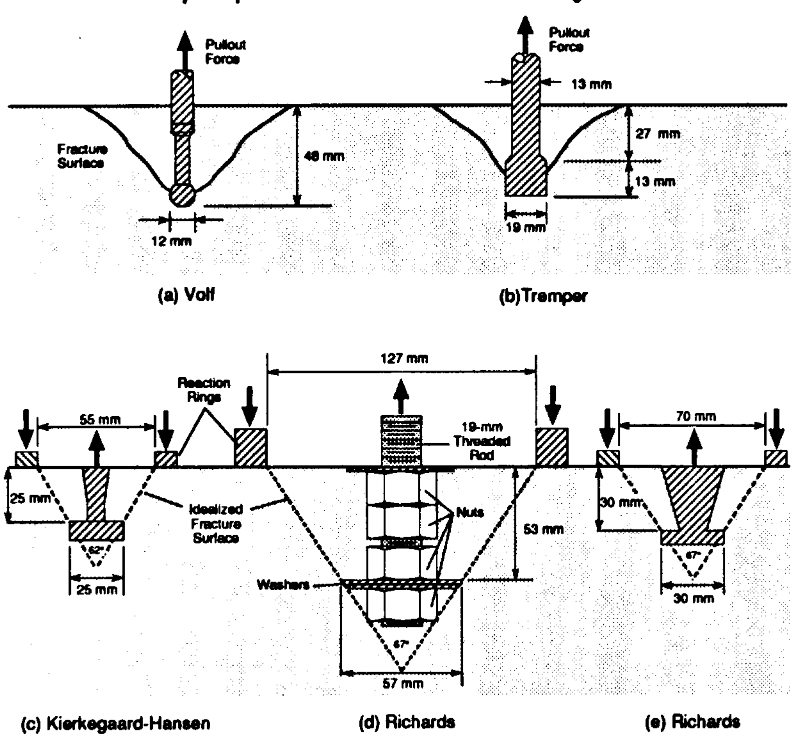


Fig. 4—Various pullout test configurations

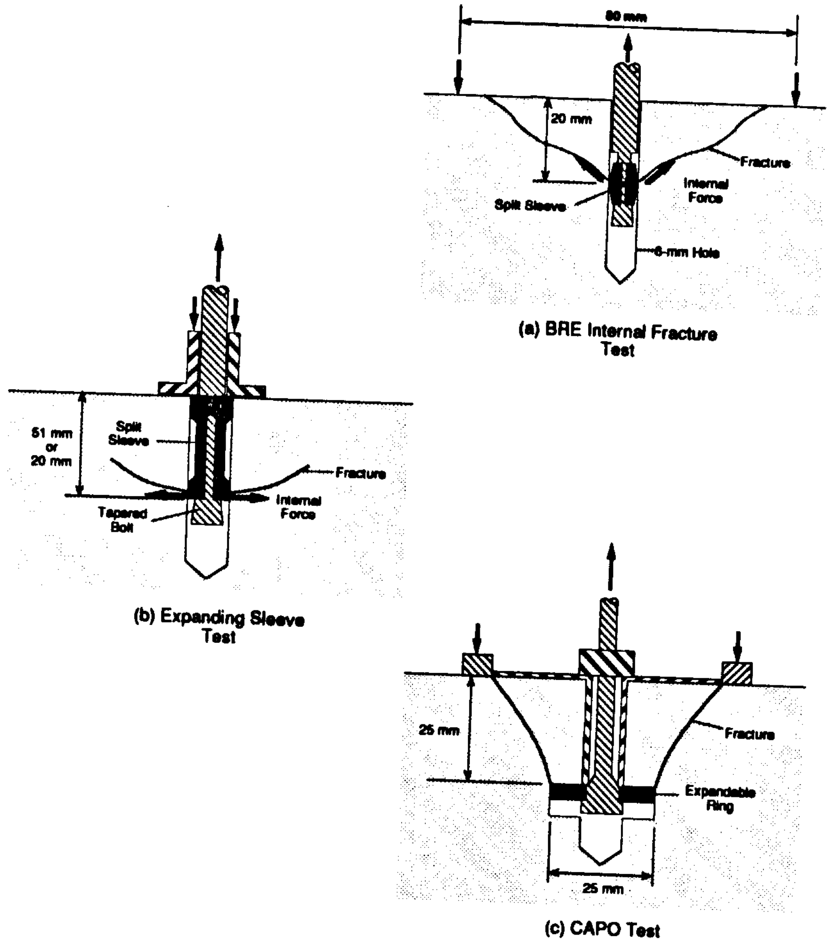
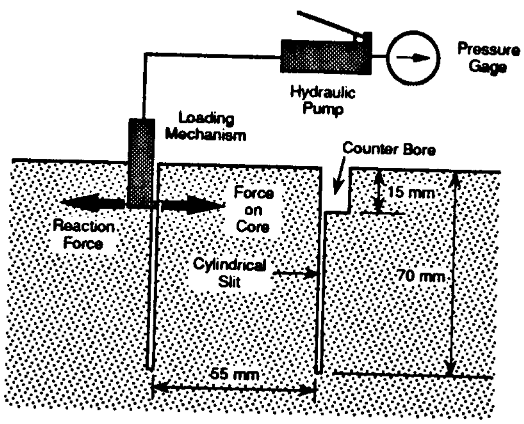


Fig. 5—Examples of post-installed pullout test techniques

(a) Break-off Test



(b) Pull-off Test

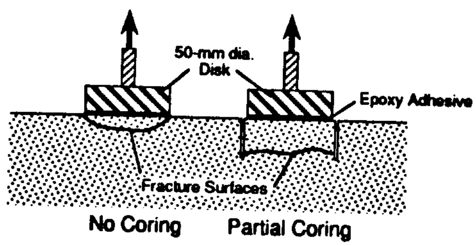


Fig. 6—Schematics of break-off and pull-off methods for in-place strength

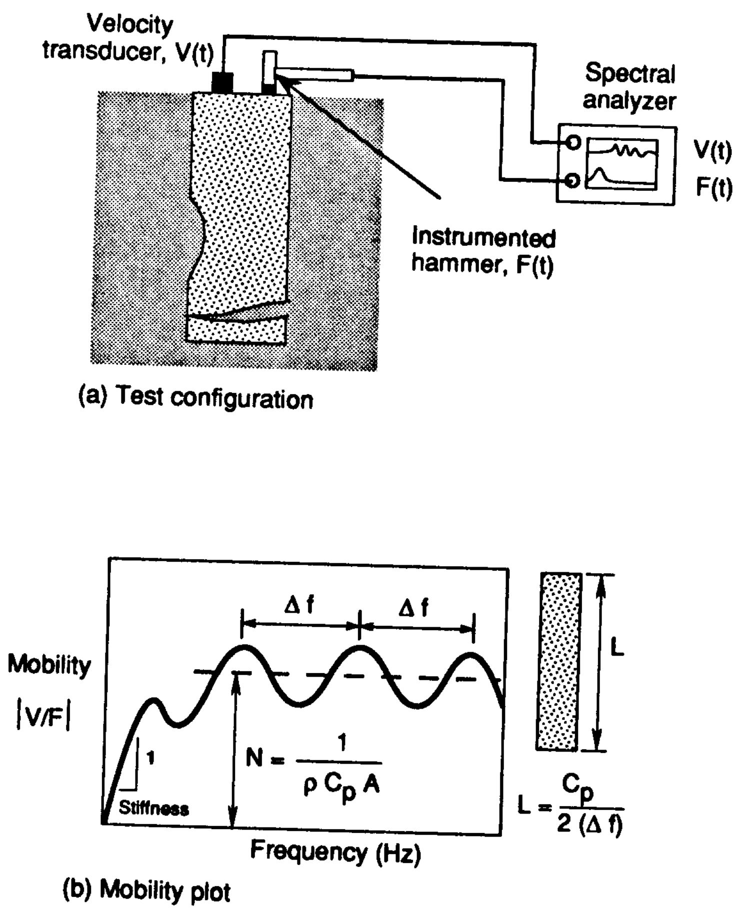
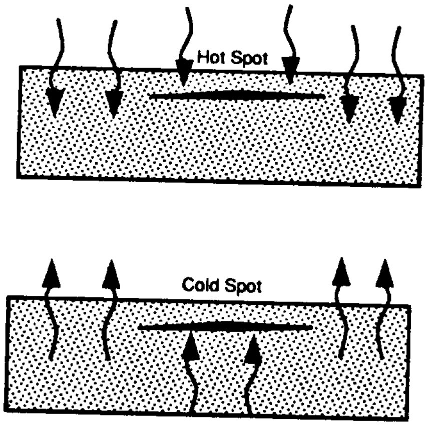


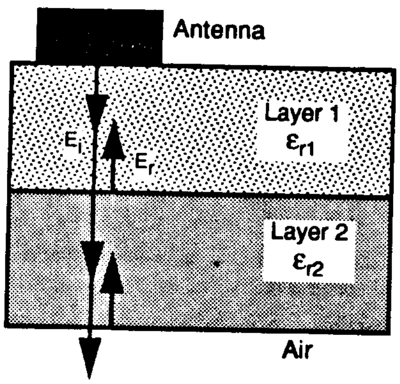
Fig. 7—The impulse response (or transient dynamic response) method: (a) testing configuration for shaft in soil; (b) idealized mobility plot for solid shaft of uniform section



Inward heat flow creates "Hot" spot above flaw

Outward heat flow creates "Cold" spot above flaw

Fig. 8—Effect of void on heat flow through a slab and resulting surface temperature



Reflection at Interface:

$$\frac{E_r}{E_i} = \frac{\sqrt{\varepsilon_{r1}} - \sqrt{\varepsilon_{r2}}}{\sqrt{\varepsilon_{r1}} + \sqrt{\varepsilon_{r2}}}$$

Material	Range of ε_r
Air	1
P.C. Concrete*	6 to 11
Bituminous Concrete*	3 to 5
Gravel*	5 to 9
Sand*	2 to 6
Rock*	6 to 12
Water	80

* ASTM D 4748

Fig. 9—Reflection of electromagnetic waves at interface between materials with different relative dielectric constants

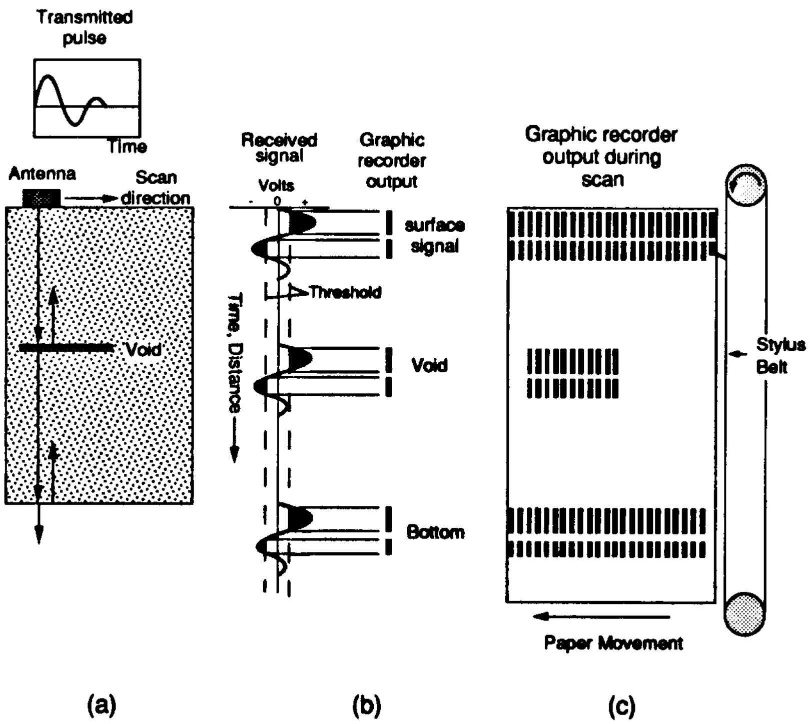


Fig. 10—(a) Reflection of radar pulse within test object with void; (b) waveform from receiving antenna and graphic recorder output based on selected threshold limits; (c) profile of test object shown by graphic recorder display during scan of test object

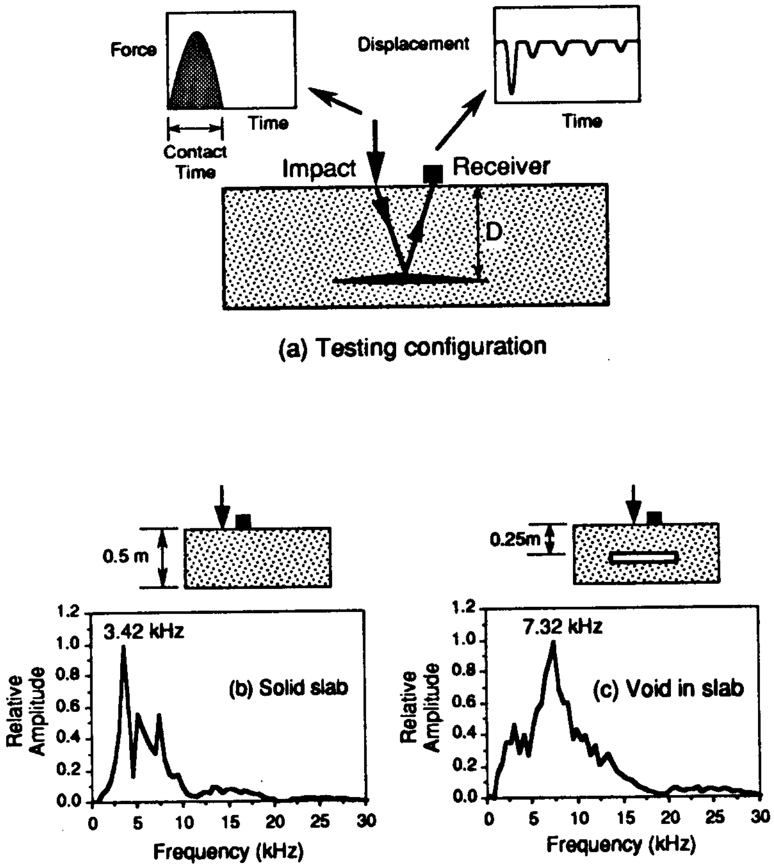


Fig. 11—The impact-echo method: (a) testing configuration; (b) impact-echo response for solid slab; and (c) response for slab with a void

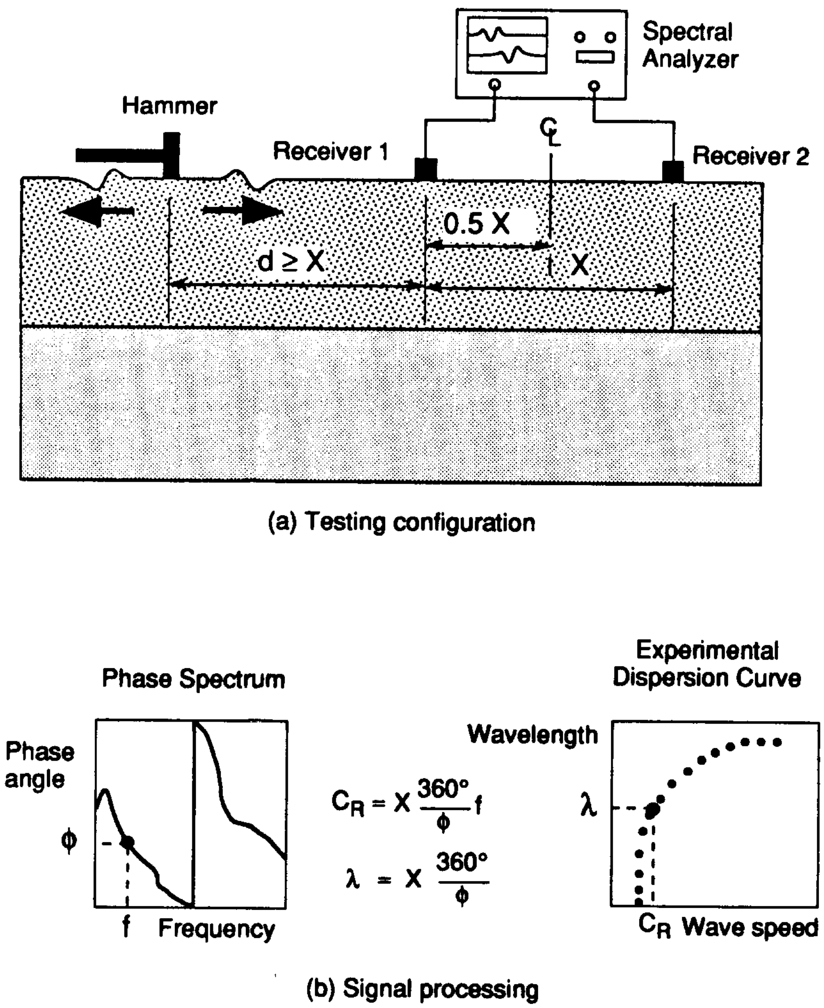


Fig. 12—The SASW method: (a) testing configuration; (b) calculation of dispersion curve from phase of the digital cross power spectrum of the receiver waveforms