

In Situ/Nondestructive Testing of Concrete – A Global Review

By V.M. Malhotra

Synopsis: During the past 40 years in-situ/non-destructive testing of concrete has achieved increasing acceptance for the evaluation of existing concrete structures with regard to their uniformity, durability and other properties. This paper reviews critically the available in-situ/non-destructive tests for estimating concrete strength and for determining properties other than strength, and discusses their implications. The methods discussed for estimating concrete strength include surface hardness and penetration resistance tests, pullout, ultrasonic pulse velocity, break-off, combined methods, and maturity techniques. The tests reviewed for determining properties other than strength include magnetic, electrical, radioactive, pulse echo, radar, microwave absorption, acoustic emission, nuclear, infrared thermography, and permeability methods.

Some of the tests described are relatively easy to perform whereas others require sophisticated equipment and trained personnel, and there are others which are still in the development stage. Regardless of the type of test used, it is emphasized that interpretation of test data must be performed by specialists rather than by technicians performing the tests. Unless comprehensive laboratory correlations have been established between the strength parameters to be predicted and the results of in-situ/non-destructive tests, the use of the latter to predict compressive or flexural strength of concrete is discouraged.

Keywords: acoustic emission; break-off tests; calibrating; compressive strength; concretes; flexural strength; impact hammer tests; measuring instruments; nondestructive tests; penetration tests; pullout tests; radiography; reviews; ultrasonic tests.

2 Malhotra

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INTRODUCTION

In an overall quality assurance program, it must be assured that a finished concrete element is structurally adequate for the purposes for which it has been designed. According to the currently used specifications in major countries of the world, if the 28-day compressive strength of test cylinders or cubes indicates compliance with the specified strength, it is assumed that the concrete represented by the test specimens satisfies the design criteria. In concrete technology, the great emphasis on the determination of strength of concrete, which almost reaches fetish proportions in some instances, is easy to explain: a number of other properties of concrete such as its elastic behaviour and to some extent its service performance can be approximated, directly or indirectly, from its strength characteristics. The strength tests, regardless of the type, are excellent for determining the criteria of quality during manufacture and delivery but they leave a lot to be desired. The main disadvantages of this approach are: the delay in obtaining test results, the fact that the test specimens may not be truly representative of concrete in a structure because of different placing, consolidation and curing conditions, the necessity of testing the specimens to failure and the lack of reproducibility in the test results. These combined with the fact that structural concrete elements are considerably larger and more massive in size, cast doubt whether the 150 x 300-mm cylinders do really represent strength of concrete in a structure. As a result, to ensure designers and owners about quality of concrete in a structure, there have been a large number of attempts over the last 40 years to develop methods for in-situ and non-destructive testing of concrete.

As the direct determination of strength implies that concrete specimens must be loaded to failure, it becomes abundantly clear that in-situ/non-destructive methods of testing concrete cannot be expected to yield absolute values of strength. The currently available in-situ methods can be broadly classified into two categories: the first category consists of those methods which attempt to measure some property of concrete from which an estimate of its strength, its durability, and its elastic parameters are obtained. Some such properties of concrete are its hardness, its rebound number, its resistance to penetration and its ability to allow ultrasonic pulses to propagate through it. The second category consists of those methods which can determine position and size of reinforcement, areas of poor consolidation, voids, cracks, honeycombing and moisture content of in-place concrete.

This paper gives a global review of the above test methods and discusses their implications. The paper is concluded by a list of pertinent references.

TEST METHODS FOR ESTIMATING STRENGTH OF CONCRETE

Two types of test methods are available for estimating compressive or flexural strength of concrete. The first type includes those methods which do not measure strength directly but measure some other property of concrete from which an estimate of strength can be made; these include surface hardness, penetration resistance, ultrasonic pulse velocity and maturity methods. The second type of test methods are those which measure some strength property from which an estimate is then made of the compressive or flexural strength of concrete; these include various types of pullout methods and break-off techniques. All of the above methods are described briefly below.

Surface Hardness Methods

The surface hardness methods consist essentially of impacting the concrete surface in a standard manner, using a given mass activated by a given energy and measuring the size of indentation or rebound. The most commonly used method is the rebound hammer. Although the rebound hammer provides a quick, inexpensive means of checking uniformity, it has many serious limitations and these must be recognized. For example, the results of the rebound hammer are affected by smoothness, carbonation, and moisture condition of concrete surface, size and age of concrete specimens, and type of coarse aggregate.

4 Malhotra

Penetration Resistance Techniques

The techniques to determine penetration resistance of concrete consist essentially of powder activated devices, the currently available system being known as the Windsor probe. In this system a powder activated driver is used to fire a hardened alloy probe into the concrete; the exposed length of the probe is a measure of the penetration resistance of concrete. This method is excellent for measuring strength development of concrete at early ages in order to determine stripping times for formwork and for determining the relative strengths of concrete in different parts of the same structure or relative strengths in different structures. The test results are affected by the hardness of the aggregate used.

Pullout Tests

Briefly, a pullout test consists of pulling out from concrete a specially shaped steel insert whose enlarged end has been cast into the concrete. The pullout force required is measured using a dynamometer. Because of its shape, the steel insert is pulled out with a cone of concrete. The concrete is in shear/tension with generating lines of the cone running at approximately 45° to the direction of the pull. The pullout strength is of the order of 20 percent of compressive strength. Like the penetration resistance technique, the pullout test is an excellent means of determining the strength development of concrete at early ages.

The major drawbacks of the pullout tests are that these have to be planned in advance and, unlike most other in-situ tests, cannot be performed at random after the concrete has hardened. To overcome the above drawbacks, new techniques are being developed in which holes are drilled into hardened concrete, into which either normal pullout inserts or split-sleeve assemblies/wedge anchors are installed which are then pulled out; in the former case a cone of concrete is pulled out and in the latter case internal cracking of concrete is caused. Investigations have also been reported dealing with the pulling out of bolts set by means of epoxy in holes drilled in hardened concrete.

Break-Off Method

This method consists of determining flexural strength in a plane parallel to and a certain distance from the concrete surface. For this purpose, tubular disposable forms are inserted in the fresh concrete. When testing, the inserts are removed and the concrete core is broken

off at the bottom by applying a force to the top and at right angles to the axis of the core.

The test method is rapid and simple, and the test results are not affected by the surface condition of concrete. However, the method suffers from the disadvantage that tests have to be pre-planned, difficulty is experienced in inserting tubes in concrete with slumps of less than 75 mm, and the test cannot be used for concretes incorporating aggregate larger than 19 mm in size. Portable equipment is available commercially.

Ultrasonic Pulse Velocity Method

The ultrasonic pulse velocity method consists of measuring the time of travel of an ultrasonic wave passing through the concrete. The time of travel between the initial onset and reception of the pulse is measured electronically. The path length between transducers divided by the time of travel gives the average velocity of wave propagation.

The pulse velocity technique is excellent for establishing uniformity of concrete, and has often been used for estimating in-situ strength. However the relationships between pulse velocity and strength are affected by a number of variables such as age of concrete, moisture conditions, aggregate to cement ratio, type of aggregate and location of steel reinforcement.

Maturity Concept

The basic principle of the maturity concept is that the strength varies as the function of both time and temperature. The maturity of in-situ concrete can be monitored by thermocouples or by instruments called maturity meters. Two commonly used maturity functions to estimate the strength of concrete are the Nurse-Saul and the Arrhenius, the former being suitable when concrete is cured between 10 and 30°C and the latter being applicable over a larger temperature range. Disposable maturity meters of Danish origin are a refreshing development.

GENERAL COMMENTS ON TEST METHODS FOR ESTIMATING STRENGTH OF CONCRETE

It cannot be overstressed that rebound and penetration resistance tests are not substitutes for standard compression tests. Estimation of strength of concrete by these methods within an accuracy of ± 15 to ± 20 percent may be possible only for specimens cast,

6 Malhotra

cured and tested under identical conditions to those from which the calibration curves are established.

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

The pullout tests, measure tensile/shear strength of concrete and the break-off tests approximate flexural strength of concrete. From the above strength values, estimates of concrete compressive strength may be made. Once again prior laboratory correlations are a prerequisite.

Variability

The within-batch variability of in-situ test results is generally high, the only exception being the pulse velocity techniques. The within-test coefficient of variation for the pulse velocity tests performed in a laboratory is generally of the order of 2 percent whereas for commercially available pullout and penetration resistance tests this value varies between 6 and 10 percent. Other methods such as surface hardness show higher variability. Caution should be exercised to ensure that for the results to be valid, a minimum number of tests are specified for a particular test method to be used.

Use of Calibration Charts Provided by Manufacturers of Equipment

Usually manufacturers of in-situ tests provide correlation charts with their equipment and recommend their use for estimating strength properties of concrete. These charts do not appear to be satisfactory because their development is based on the use of certain types and sizes of aggregates, test specimens, and test conditions. It is therefore essential for users of in-situ tests to prepare their own calibration charts for the type of concrete under investigation. With changes in source of materials new calibration charts become mandatory.

Surface moisture condition of specimens under test affects significantly the in-situ test results. This is especially so for the rebound test method. It is desirable to ensure that the surface of the structural concrete under test is uniformly moist. Needless to say that the use of in-situ tests on frozen concrete is not recommended.

Use of Multiple Methods

In Europe in general and in Romania in particular, the use of more than one in-situ/non-destructive testing technique to improve the accuracy of prediction of strength parameters of concrete has gained some credibility. RILEM* is processing a document giving details of a combined method involving pulse-velocity and rebound hammer. Some researchers have suggested the use of pullout and pulse velocity techniques, while others have suggested the use of other combined methods. The proponents of this approach claim that the use of two methods, each measuring a different property, can overcome the limitations associated with the use of one method. Some case histories have been published supporting this claim; data disputing this claim are available also.

The use of more than one method may provide useful information in some instances but its general use is not advocated because of economy and time requirements, and the possible marginal increase in the accuracy of predicting compressive strength.

Certification of Testing Personnel

One of the serious problems associated with the use of in-situ/non-destructive testing methods for estimating strength of concrete in the field is the lack of trained support personnel. Unlike the metals field where there are well structured facilities available for the certification of non-destructive testing technologists**, no such system exists for certification of concrete personnel. Properly trained and certified technicians can advance considerably the use of in-situ/non-destructive testing methods by giving designers and engineers confidence in the accuracy of test data.

Standardization

A number of leading organizations such as ASTM, BSI, DIN and ISO have issued or are in the process of issuing standards on the use of in-situ/non-destructive testing methods. The ASTM provides the most comprehensive coverage with standards on rebound method, pulse velocity, penetration resistance and pullout tests. There is a fair degree of common ground in the standards of the above

*The International Union of Testing and Research Laboratories for Materials and Structures.

**In Canada, CANMET routinely certifies such personnel.

8 Malhotra

organizations, but occasionally there is a difference in approach. It is suggested that the various organizations writing standards should form joint committees to review the currently available standards to develop a more common approach. The ASTM may be best suited to do so.

Specifications

Canadian and U.S.A. specifications have no provisions for the acceptance of concrete on the basis of in-situ/non-destructive methods. The British and European specifications are no different. The development of well documented detailed field test data by testing organizations is a prerequisite before the specification writing authorities would allow the acceptance of concrete on the basis of the above tests. It is imperative that such data be developed. The organizations best suited to develop such information are Portland Cement Association and Bureau of Standards in the U.S.A., and the Building Research Station and the Cement and Concrete Association in the U.K.

Differences Between North America and European Practices

In North America, in-situ tests have been used primarily to determine development of strength of in-place concrete to provide data for safe form removal times. Also, the in-situ tests are used to assess quality of concrete when the 28-day strength tests fail to meet the specified design criteria. The writer knows of no region of North America where government or private laboratories are engaged on a routine basis to develop data-banks on correlations between the results of in-situ tests and the 28-day compressive strength of concrete with a view to using this data for predicting the latter. On the contrary, in a number of countries in Europe, the in-situ tests are used primarily to predict the 28-day compressive strength of concrete. As a back up, a number of laboratories in Europe and especially in eastern Europe have developed correlation data between parameters of in-situ tests and the 28-day compressive strength of concrete covering a wide range of local aggregates, cements and mix proportions. One such example is the INCERC* in Romania.

The origins of this difference in approach are rooted in history but are probably due to the fact that, in relative terms, the European specifications until recently have been more strength oriented as compared with North American specifications where both strength and durability criteria are specified.

*Institutul de Cercetari in Constructii Si Economia Constructiilor

Safety Aspects

Of all the in-situ/non-destructive testing methods the penetration resistance and pullout tests seem to be the most suitable for monitoring early-age strength development of concrete and this fact can be used to develop correlation data for determining safe form stripping times. In the case of the penetration resistance test, data should be developed that would indicate that if probes fail to penetrate into concrete a predetermined distance, the concrete has reached sufficient maturity, and that forms could be removed safely. In the case of pullout tests, the pullout assemblies need not be torn out of concrete; instead if a predetermined pullout force has been reached on the gauge, the test can be terminated and forms can be removed safely.

Cost Effectiveness

In-situ/non-destructive testing methods provide an effective way of obtaining a considerable amount of preliminary test data at relatively little cost. Both rebound and pulse velocity methods can be used to monitor structural elements to delineate zones of weaker concrete that can then be subjected to core testing if necessary. Core testing and loading tests should only be performed as a last resort, and only after the structural elements under investigation have been fully evaluated using in-situ/non-destructive testing methods.

Research Needs

In-situ/non-destructive testing methods for estimating strength of concrete have reached a standstill as to new developments. Very little research and development work is in progress in this area in North America and Europe. There seems to be no major breakthrough on the horizon. The only promising aspect appears to be that international standardization organizations either have, or are in the process of, writing standards for known available tests. There are perhaps two possible types of research needs, the immediate ones and the long-term ones. The immediate research needs consist of refining penetration resistance, pullout and maturity tests; for example the inserts for the commercially available pullout tests are too small and too close to the surface and the within-test variability is too high. These problems need solutions. The development of multi-channel digitized maturity meters would encourage use of the maturity technique. The long-term research needs are for more fundamental work in the properties of materials to help develop tests based directly upon these properties.

TEST METHODS FOR DETERMINING PROPERTIES OTHER THAN STRENGTH

In a number of instances in concrete technology, strength is not the most critical parameter to be investigated; instead information is needed to determine position and size of reinforcement, to assess moisture content, to delineate cracks and discontinuities, and to locate areas of poor consolidation, voids and honeycombing in concrete. Over the years attempts have been made to develop methods to assist in resolving some of the above problems. These methods vary from simple magnetic devices to highly sophisticated infrared thermography; some of these have reached maturity while others are still in an experimental stage. These methods are described briefly below.

Magnetic Methods

A number of portable battery-operated magnetic devices that can measure the depth of reinforcement in concrete and detect the position of reinforcement bars are available commercially. The devices manufactured in the U.K. and in Holland are known as cover meters and a French device is called a pachometer.

Cover meters and pachometers are magnetic devices and are based on the principle that the presence of steel affects the field of an electromagnet. In a typical cover meter or pachometer, the probe unit consists of a highly permeable U-shaped magnetic core on which two coils are mounted. An alternating current is passed through one of these coils and the current induced in the other coil is measured. The induced current depends upon the mutual inductance of the coils and upon the nearness of the steel reinforcing bars. A moving coil meter measures the induced current. British Standard (BS 4408, Part 1) gives guidance for the use of these devices. Cover meters and pachometers give satisfactory results if structural members are lightly reinforced. In heavily reinforced sections, the effect of secondary reinforcement cannot be eliminated and the satisfactory determination of the cover to steel becomes difficult. The performance of the magnetic devices is adversely affected at temperatures below 0°C.

Electrical Methods

Electrical methods are gaining increasing acceptance as a tool for evaluation of in-place concrete to determine reinforcement corrosion and thickness of concrete pavements. They also offer potential for determining moisture content of and moisture penetration through hardened concrete.

The principle of determining the likelihood of corrosion activity of steel reinforcement consists of estimating the electrical half-cell potential of reinforcing steel with reference to a copper/copper-sulphate reference electrode. The measurement presented in an equipotential contour provides a graphical delineation of areas in a concrete element where corrosion activity may be occurring; the measurements presented as a cumulative frequency diagram provide an indication of the magnitude of affected areas of a concrete member. ASTM standard C876 describes the test approach in detail and the method is used routinely for determining corrosion activity of reinforcing steel in concrete parking garages and bridge decks.

The electrical methods have been used also to determine the thickness of concrete pavements. These methods are based on the principle that a material under test offers resistance to the passage of an electric current when the latter is made to pass through it. A concrete pavement has a resistivity characteristic that usually differs from that of the underlying subgrade layers; thus a change in the slope of the resistivity versus depth curve is used to estimate the depth of concrete pavement.

In electrical resistivity methods the testing equipment and test procedures are simple. However the resistivity of concrete is highly dependent on moisture and salt content, and its temperature. Thus the presence of moisture, salt and/or temperature gradients in concrete pavements may pose problems in the interpretation of test results.

Radioactive Methods

The radioactive methods can be classified broadly into two types; the radiography and radiometry. In radiographic methods, a graphic picture of the interior is taken employing a radioactive source to reveal position and condition of reinforcement, air voids, segregation, grouting of post-tensioned concrete members and cracks. In the radiometry methods, gamma rays generated by a variable radioisotope are made to pass through concrete, and the intensity of the emerging radiation is detected by geiger or scintillation counters. Prior calibration charts are then used to determine in-situ density of structural concrete members and thickness of concrete.

Of the radiography methods, X-ray radiography has serious limitations due to the necessity of costly and dangerous high voltage equipment, and offers little hope for use in the field. Gamma radiography on the

12 Malhotra

hand is being accepted increasingly in the U.K. and Europe. The equipment is relatively portable and concrete of up to 450 mm in thickness can be examined. Above this thickness, the long exposure times needed make the methods uneconomical. British standard (BS 4408, Part 3, 1970) outlines testing procedures dealing with gamma radiography. Again, the limiting factors in the use of gamma radiography appear to be the relatively high cost and safety aspects.

Ultrasonic Pulse Velocity and Pulse Echo Techniques

The principle of ultrasonic pulse velocity has already been discussed earlier. In addition to estimating strength of concrete, the pulse velocity method is ideal for determining cracks and voids in mass concrete. It has been used with advantage to monitor integrity of in-place concrete after a fire.

Pulse echo techniques are used primarily in analyzing the reflected waves to delineate voids and internal discontinuities in concrete. The principal advantage is that the tests can be performed when only one surface is available. The echo may be created by means of a blow from a hammer or by mechanical means. This technique probably originated in the testing of concrete piles and is now being used increasingly in North America to delineate honeycombing, cracks and lack of consolidation in concrete. The interpretation of the reflected waves displayed on an oscilloscope is not easy and depends on the experience of the operator. This method, when standardized and when portable equipment becomes available commercially, will have considerable potential in evaluating in-place concrete. A number of leading organizations in North America are initiating development work in this method.

Microwave Absorption and Radar* Scanning Techniques

Microwave are electromagnetic in nature and are about 1 mm in wavelength and about 10^{12} Hz in frequency. Because of their electromagnetic nature they can be reflected, diffracted and absorbed. The absorption of these waves by water has led to development of a method for determining the moisture content of concrete. The basic principle lies in the measurement of the attenuation of a microwave beam passing through concrete, the attenuation being considered to be characteristic of the moisture content of the material.

*Initial letters of radio detection and ranging

The microwave absorption technique is still in the development stage and considerably more research is needed before the method can be used with confidence for determining the moisture content of in-place concrete.

Radar techniques are based on the principle of reflection of electromagnetic waves by concrete. This technique has been used with success by The Port Authority of New York and New Jersey, U.S.A. to evaluate concrete pavements and bridge decks. One of the attractive features of this technique is the speed (approximately 17 km/h) at which pavements can be scanned. However, great planning and skill are needed to evaluate the data. Portable equipment is available commercially, but the method has yet to be standardized.

Acoustic Emission Methods

The acoustic or stress emission methods have been used in recent years to study the initiation and growth of cracks in concrete under stress. Briefly, acoustic emissions are small amplitude elastic stress waves created by localized deformation in concrete at points being strained beyond their elastic limit. At the surface they are detected as small displacements by sensors positioned on the surface of the test specimens. The variations in the time of arrival of stress waves at each position are used to locate the source of deformation.

The major drawback of these methods is that observations can be made only during a period of increasing deformation and stress, and the method cannot be used for individual or comparative measurements of concrete under static loading conditions.

Nuclear Methods

The use of nuclear techniques for determining the moisture and cement content of concrete is of relatively recent origin. These consist of neutron scattering methods for moisture content determination and neutron activation analysis for the determination of cement content.

Neutron moisture gauges, an application of neutron scattering methods, are based on the principle that hydrogen containing materials (water in case of concrete) act as excellent moderators for fast neutrons i.e. such materials produce a rapid decrease in neutron energy depending on the hydrogen present in the materials. Thus, counting the slowed down or thermalized neutrons resulting from the interaction of fast neutrons with a hydrogen

14 Malhotra

containing matrix gives a measure of hydrogen (water in concrete) content of the matrix.

Neutron activation methods are still largely underdeveloped for application to in-situ concrete and little data are available on their field use. The major potential use of these methods lies in the determination of the cement content of hardened concrete. As the calcium* and silicon* contents of portland cement are relatively high compared to its other constituents, these elements are useful when using activation analysis. The basic principle of this method is that most elements become radioactive on neutron bombardment. The radioisotopes that are unstable but decay to a stable state with the emission of energy in the form of beta and gamma radiation of characteristic energies and half-life factors that allow characterization of the radioisotope and hence of its stable parent nuclide. The test is performed on a comparative basis.

Infrared Thermography

Infrared thermography or infrared scanning measure and record the heat emission from a structure. As the rate of heat emission is affected by cracks and other discontinuities such as delamination in concrete bridge decks, the scanners will show a difference between heat emissions from sound and unsound concrete. This technique has been used with success for determining deterioration in chimneys. In the U.S.A. and Canada, the infrared scans have been used with limited success to detect delaminations in concrete bridge decks. Test data are affected considerably by the physical conditions of concrete such as its moisture condition. Though equipment is available commercially, considerable research is needed to iron out operational problems. No standards are yet available.

Permeability Methods

Permeability of concrete, which has always been an important design criterion for water retaining structures, is now considered equally important for concrete exposed to aggressive elements such as deicing salts. Advances are being made in the development of permeability tests for small laboratory specimens but in-situ permeability tests are still in an experimental stage. A portable equipment of Danish origin employing a gas permeability technique appears promising.

*The presence of calcium and silicon in aggregates does create difficulties in the interpretation of test data.

GENERAL COMMENTS ON TEST METHODS FOR DETERMINING
PROPERTIES OTHER THAN STRENGTH

Most of the test methods described above for determining properties of concrete other than strength are still in their infancy. Apart from the cover meters and radiographic and radiometric techniques for which guidance is provided in the British standards and European codes, and electrical half-cell potential method to determine corrosion activity which has been standardized by the ASTM, no national guidelines and standards are available on any of the other methods. This is because: (a) most of these methods are still in development stage, (b) test equipment is expensive and requires highly skilled personnel, and (c) interpretation of test data depends to a considerable degree on the skill and experience of the operator. Of the methods under discussion, radar scanning techniques followed by the pulse echo method seem to be most promising and have potential for applications for a variety of situations in the field. Major research efforts should be directed to resolve some of the problems associated with the above two techniques as to the interpretation of test data and the development of portable test equipment. Because of the increased appreciation of durability of concrete, especially in the U.K. and Europe, development of reproducible in-situ permeability tests needs serious consideration and infusion of research funds.

The nuclear, microwave absorption, acoustic emission, and infrared thermography techniques are specialized methods of limited interest and use.

CONCLUSIONS

A large measure of standardization has been achieved for tests used for estimating the strength parameters of concrete; however, apart from magnetic and radioactive methods, there is a need for considerable research before standards can be developed for tests which are used to determine properties other than strength. Radar scanning and pulse echo techniques appear to be the most promising of these types of tests.

In-situ/non-destructive tests must not be considered as a replacement for the standard cylinder or prism test, but only as additional techniques. When performed in conjunction with standard core tests, they can reduce the cost of testing.

Most of those tests which are used for estimating the strength parameters of concrete provide an excellent means of establishing and evaluating uniformity of concrete, and their use in this aspect of control of quality of concrete is recommended. Unless comprehensive laboratory correlations have been established between the strength parameters to be predicted and the results of in-situ/non-destructive tests the use of the latter to predict compressive or flexural strength of concrete is discouraged.

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Study of the Stress Waves in the Plunger of a Rebound Hammer at the Time of Impact

By Toyoki Akashi and Syouji Amasaki

Synopsis: The nature of the stress waves found in the Schmidt Hammer after impact during the testing of concrete were examined experimentally. Using a specially designed plunger, the authors have been shown that the impact of the hammer mass produces a large compressive wave σ_i and a large reflected stress wave σ_r at the centre of the plunger. The ratio, σ_r/σ_i of the amplitudes of these waves and the time T between their appearance was found to depend upon the surface hardness of cured concrete. The rebound number was found to be approximately proportional to the ratio of the two stresses and was not significantly affected by the moisture conditions of the concrete. The magnitude of the first stress wave at the centre of the plunger is almost constant and is approximately 80 percent of the value calculated by Smith's numerical solution, which does not consider the efficiency of the impact of the hammer.

The authors have concluded that the principal of operation of the N-type Schmidt test hammer may be more complex than is assumed when consideration is given only to the simple problem of applying Newton's laws to impacting bodies. It may involve considerable components of longitudinal wave transmission.

It is further concluded that, to correctly measure the rebound number of hardened concrete, the Schmidt hammer can be calibrated by testing a material with a constant hardness and measuring the resulting impact stress wave. By observing the behaviour of the impact stress in the plunger the surface hardness of concrete can be measured with higher accuracy.

Keywords: amplitude; calibrating; concretes; hardness; impact; impact hammer tests; nondestructive tests; stress waves.

18 Akashi and Amasaki

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INTRODUCTION.

The rebound number determined by the Schmidt Hammer is a well known method of assessing the uniformity of concrete in situ, and to obtain an approximate estimate of its compressive strength. However it is known that rebound numbers differing by as much as three units may be obtained using different hammers. Therefore it is normally recommended that tests be made with the same hammer in order to compare results from one test to another (1).

The accuracy of an estimate of compressive strength determined by the Schmidt Hammer depends upon the correlation obtained between the rebound number and a destructive test performed on the same specimen. Malhotra (2) has noted that the accuracy of estimation of compressive strength determined on specimens cast, cured and tested in the laboratory lies between 15 and 20 percent. He estimated that the probable accuracy of prediction of concrete strength in a structure is +25 percent.

The principal of operation of the N type Schmidt Hammer is based upon Newton's Law of Collision. However, the plunger of the hammer is an elastic medium and thus on impact generates a stress wave in the same manner as occurs during the driving of a pile. It is thus apparent that the behaviour of the hammer may be affected by the reflected stress wave from the tip of the plunger. Further, the stress wave may be significantly affected by the surface hardness and the deformation properties of the concrete.

Gaede and Schmidt (3) measured the stress waves caused by the mass of the hammer impacting the plunger, but their observations were significantly influenced by the bending of the plunger caused by eccentric impact of the hammer.

The present paper describes some empirical relationships



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between the surface hardness of concrete and the behaviour of the impact stress waves produced when using a N type Schmidt Hammer.

EXPERIMENTAL PROCEDURES

Because it is not possible to mount a strain gauge on the plunger of a Schmidt Hammer as it is supplied a modified plunger was made. The experimental plunger was of the same shape and dimensions as the normal plunger except that it was 180.6 mm in length. Figure 1 illustrates the form and dimensions of the modified plunger. The Young's modulus of the modified plunger was measured and found to be 190 GPa.

Figure 2 illustrates the experimental method used to measure the impact stress waves. Four semi-conductor strain gauges, of 2 mm length each, were mounted axially to the plunger, each at 90° to the other. They were connected in parallel to minimize any effects caused by bending of the plunger resulting from eccentric impact of the body of the hammer.

A Wheatstone bridge circuit was used to measure the output from the gauges, which was amplified and sent to a digital transient memory, an oscilloscope and a paper tape punch.

The surface hardness of the concrete was estimated by determination of the depth of the surface indentation produced by the hammer on impact. The depth of the indentation being determined by measurement of its diameter and calculation as follows:

$$y = -0.00731r + 0.0183r^2 \quad (\text{mm})$$

where, y = depth of the indentation;
 r = radius of the indentation.

Two series of tests were made, in Series 1, the effects of mix proportions and curing of the concrete upon the behaviour of the impact stress waves were observed. In Series 2, the effects of different hammers and the Brinell hardness of materials were examined.

20 Akashi and Amasaki

Series 1

Six different concrete mixes were examined, the mix proportions for which are given in Table 1. The following materials were used for these concretes:

- Yasu river sand, specific gravity = 2.57, fineness modulus = 2.81;
- Takatsuki crushed stone, specific gravity = 2.68, maximum size = 20mm.
- Ordinary Portland cement.

Cube specimens were made and cured for 3, 7, 28 and 91 days in water at $20 \pm 3^{\circ}\text{C}$ and in air at $20 \pm 3^{\circ}\text{C}$, 90% RH. Cylinders, 200 x 100(dia) mm were also made for determination of compressive strength. Schmidt Hammer tests, using Hammer N-13, were conducted horizontally under a compressive stress of 0.74 MPa.

Series 2

Metal plates, 160 x 160 mm of the five materials shown in Table 2 were embedded in concrete cubes. Schmidt Hammer tests using four hammers, N-13, N-34, 4923 and K-X, were used to conduct tests vertically and horizontally under a compressive stress of 0.74 MPa.

RESULTS AND DISCUSSION

Stress wave testing on concrete

Figures 3 and 4 show the oscilloscope output and the results of computer analysis of the data from typical impact stress waves at the centre of the plunger generated when the hammer is used on concrete.

In general, when the body of the hammer strikes the plunger, a compressive stress wave is produced at the head of the plunger in the same manner as occurs when a pile is being driven. This compressive stress wave travels along the plunger, with some attenuation, at a velocity given by:

$$c = \sqrt{E/\rho}$$

where c = velocity of the stress wave, in m/sec;
 E = Young's modulus of the plunger material, Pa;
 and ρ = the density of the plunger material, kg/m³.

When the compressive stress wave reaches the point of the plunger, it is reflected back in some manner determined by the resistance of the concrete. If the point of the plunger experiences little resistance from the concrete, so that longitudinal displacement of the point can occur, a tensile stress wave will be reflected back up the plunger. If the resistance of the concrete at the point of the plunger is substantial, the initial compressive stress wave travelling down the plunger also will be reflected back as a compressive stress wave (4).

The reflected wave is itself further reflected from the free head of the plunger and from the body of the plunger where changes in cross section occur. These secondary reflections travel as either compressive or tensile stress waves and may overlap at certain points. The net stress at a particular point on the plunger at an instant in time will be the algebraic sum of the initial and reflected waves (4).

In the present tests, it was found that after the compressive stress wave had been produced and the reflected waves discussed above had resulted, a large compressive stress wave was observed. The reason for the appearance of this second wave is not clear from the data from these experiments but it is assumed that locally the concrete may be destructively deformed after the impact and the plunger tip may then behave as a fixed point. The rise time of the compressive stress wave at hammer impact was approximately 20 microseconds and the time required for the wave to travel down the plunger and return was 36.6 microseconds, thus the peak of the initial compressive stress could not have been affected by a reflected wave from the tip of the plunger.

In this paper, an initial stress wave is defined as the first stress wave and a regenerated large stress wave is termed the reflected stress wave. Figure 3 shows that the amplitude σ_1 and the shape of the first stress wave does not vary much for concrete as curing time increases, but the reflected stress wave

22 Akashi and Amasaki

is significantly changed in amplitude σ_r and the time T at which it appears.

The relationship between the amplitudes of the two stress waves is of some interest, and a stress ratio σ_r/σ_i can be defined the relationship of which to the time T from the first to the reflected wave is shown in Figure 5. It is clear that in these experiments the stress ratio varied from 0.6 to 1.05 and that there was an inverse relationship between the magnitude of the stress ratio and the time T . When the stress ratio was larger than 0.8, the relationship was found to be affected by the moisture content of the concrete. T was observed to be less for wet concrete than for dry concrete for the same stress ratio. This may be considered to indicate that wet concrete behaves more like an elastic medium than does dry concrete because it exhibits a larger value for Young's modulus.

Effect of concrete properties on the behaviour of stress waves

In Figures 6 and 7 respectively the stress ratio and T have been plotted versus the depth of indentation. Both are seen to be influenced by moisture and curing conditions such that the stress ratio is greater for wet concrete and the time T is shorter for the same depth of indentation. It is apparent that the local deformation on impact is more destructive to wet concrete than to dry concrete.

The relationship between the stress ratio and the rebound number was not found to be affected by moisture. As can be seen from Figure 8 the rebound number was almost directly proportional to the stress ratio. This would indicate that the stress ratio is a measure of the surface hardness of concrete and, as is discussed below, that the Schmidt test hammer can be calibrated by using a material of constant hardness as an anvil and measuring the impact stress wave.

As is shown in Figure 9, the stress ratio correlates well with compressive strength for concrete. Although the relationship between rebound number and compressive strength was influenced by moisture, the relationship to the stress ratio was not. The differences observed in Figure 9 that result from moisture content are interpreted as resulting from variations in surface hardness of the concrete.

Anvil and metal sample tests

According to the operating instructions (5), the N type Schmidt hammer must give readings between 78 and 82 on the plunger testing anvil (weight =16 kg; Brinell hardness HB =4.9 GPa). Table 3 shows the results from anvil tests, each of which are the average of six readings. Hammers N-13 and N-34 gave reading between 78 and 82, but hammers 4923 and K-X gave values below 70 and these could not be adjusted. Although hammers N-13 and N-34 gave readings between 78 and 82 on the anvil with the guide tube, the readings without the guide tube or using the test plunger were smaller. It appears that the tip of the plunger may jerk on impact when the guide tube is not used and that the hardness of steel and the length, which was different for the test plunger than for the original plunger, may affect the readings. Table 4 shows test results obtained with the original plunger.

In Figure 10 typical stress results from tests on the anvil and on five different types of metals are presented. These data are different from those observed with concrete and show the development of a large wave between the first stress wave and the reflected wave. For concrete both the amplitude and the time T of the reflected waves were found to be affected by hardness. For metals, only the amplitude showed a dependence on hardness. This is considered to result from differences between local deformation properties between metals and concrete. This intermediate wave was ignored in the present study.

Analysis of data by Smith's numerical solution

The stress wave in a plunger during hammer impact is influenced by several parameters some of which are:

- The weight and Young's modulus of the hammer;
- The spring constant and expansion at the release of the impact spring;
- Young's modulus and the dimensions of the plunger;
- Friction between the hammer and the hammer guide bar.

24 Akashi and Amasaki

Using the values of the hammer mass, spring constant and the expansion at the release of the impact spring provided by the supplier, the behaviour of the impact stress wave at the centre of a test plunger was calculated using Smith's numerical solution (6,7). This solution is based on driving the distributed mass of the plunger into a number of concentrated weights which are connected by weightless springs and point resistances that act upon it. The calculated results are shown in Figure 11. By Smith's solution, the reflected wave and the large wave (ignored in these tests) observed in the experiments were not formed, and the amplitude of the first wave was larger than the experimental value.

As discussed above, hammers N-13 and N-34 gave anvil reading between 78 and 82, however a large difference was observed in rebound numbers on different metals. The relationship between rebound numbers obtained using the test plunger and the stress ratios σ_r/σ_1 and σ_1/σ_{cal} (where σ_{cal} is the value of the first large stress calculated by Smith's numerical solution) is shown in Figure 12. In spite of the large differences in the rebound numbers, values for σ_1 are consistently approximately 80 percent of σ_{cal} and are constant for each hammer. This appears to be because the energy of impact of the hammer is not completely propagated to the plunger.

In pile-driving problems, it is suggested that the impact efficiency of a hammer, e , is approximately 60 percent for single-acting steam hammers (8,9). Assuming that the impact energy of a Schmidt hammer is also 60 percent, σ_1 can be calculated as 74.1 MPa which agrees approximately with experimental values for σ_1 . Thus it would seem that the proper operation of the Schmidt hammer requires precise release of the impact so that σ_1 is a constant value at all times.

CONCLUSIONS

It is concluded that the principal of operation of the N-type Schmidt test hammer may be more complex than is assumed when consideration is given only to the simple problem of applying Newton's laws to impacting bodies. It may involve considerable components of longitudinal wave transmission.

It is further concluded that, to correctly measure the rebound number of hardened concrete, the Schmidt hammer can be calibrated by testing a material with a constant hardness and measuring the resulting impact stress wave.

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Table 1
Concrete Mix Proportions

M.S. (mm)	Slump (mm)	W/C (%)	s/a (%)	Unit weight (kg/m ³)				
				C	W	S	G	WRA
20	75±1	70.0	51.1	276	193	925	921	0
		60.0	49.1	322	193	870	939	0
		50.0	47.1	386	193	810	947	0
		40.0	45.1	482	193	740	937	0.45
		30.0	43.1	643	193	650	893	4.21
		25.5	43.0	600	153	712	981	7.37

WRA(MT-150) used to adjust slump.

Table 2
Properties of Metals Used for Tests

Materials	thickness (mm)	Brinell hardness HB (Gpa)
free cutting copper	20.1	0.62 (10/500)
structural hot rolled steel	19.2	1.75 (10/1000)
carbon tool tempering steel	19.5	2.41 (10/1000)
"	19.4	3.29 (10/1000)
carbon tool quenching steel	19.6	5.96 (10/1000)

Table 3
Data from Anvil Tests

Hammer	vertically downward			horizontally	
	original		specific plunger	plunger	
	with guide	without guide		original	specific
N-13	79.7	76.5	67.1	77.1	66.1
N-34	80.0	78.4	59.0	79.1	57.8
4923	68.6	60.0	55.2	----	53.4
K-X	63.8	60.6	----	72.0	----

Table 4
Rebound Numbers Determined on
Five Types of Metals

brinell hardness	vertically downward				horizontally			
	N-13	N-34	4923	K-X	N-13	N-34	4923	K-X
0.62	40.5	37.9	27.3	35.8	36.5	33.4	33.9	27.3
1.75	49.3	50.0	37.6	44.1	51.0	45.8	46.8	40.4
2.41	57.7	56.5	46.5	53.6	55.3	54.5	51.1	55.5
3.29	61.7	60.8	53.6	57.7	62.3	61.6	55.5	50.4
5.96	68.3	68.0	53.3	62.2	65.7	65.1	58.8	50.0

Table 5
Relationship Between Rebound Number and Maximum Stress
of the First Stress Wave for Tests on Metals

Brinell Hardness HB (GPa)	vertically		horizontally	
	Rebound Number	Max. Stress (MPa)	Rebound Number	Max. Stress (MPa)
Anvil	67.0	78.0	66.1	72.9
0.62	31.6	77.2	36.3	72.8
1.75	39.1	74.9	45.0	71.0
2.41	46.6	75.3	51.7	71.3
3.29	54.6	74.2	60.6	72.9
5.96	59.5	75.1	59.2	71.3

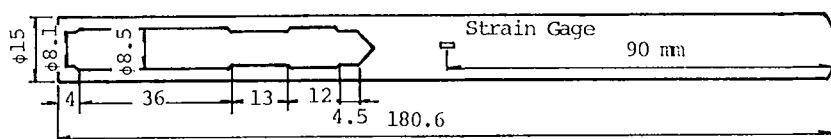


Fig. 1--Sectional view of a test plunger

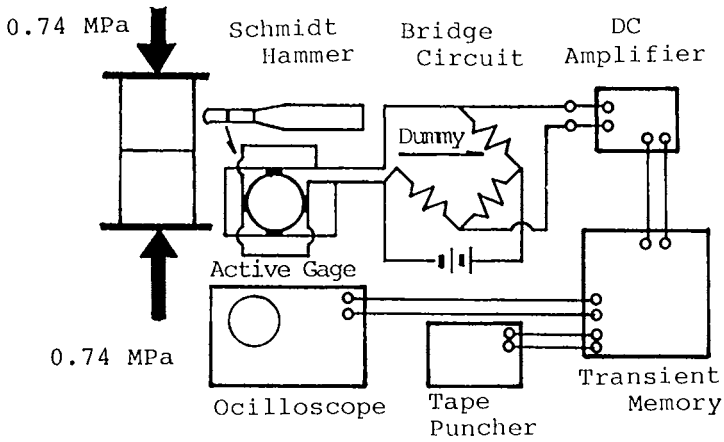


Fig. 2--Schematic diagram of the equipment used for observing stress waves

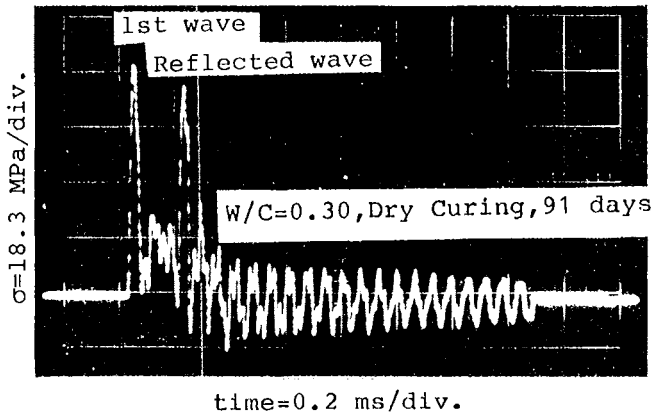


Fig. 3--Oscilloscope trace of an impact stress wave in the test plunger when testing concrete

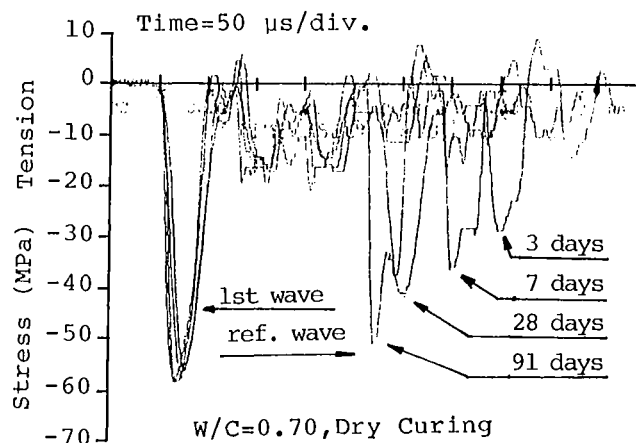


Fig. 4--Variation of impact stress waves with the age of hardened concrete

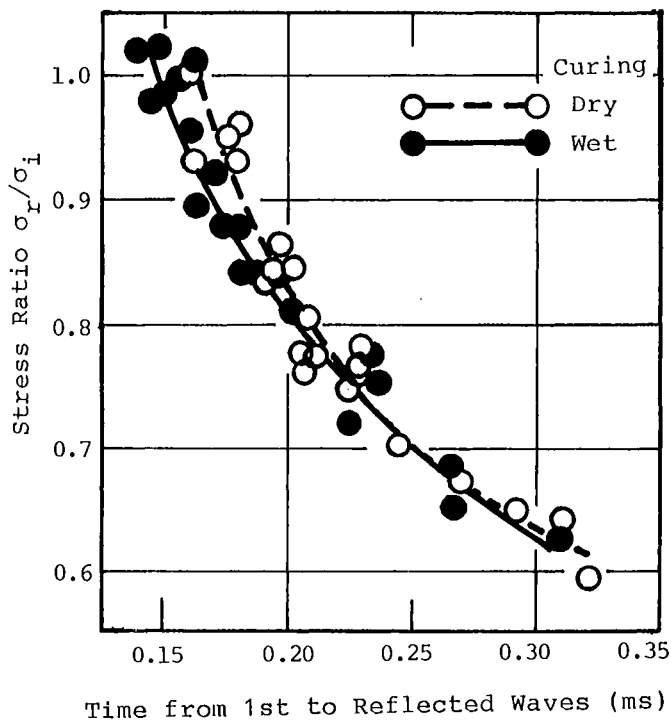


Fig. 5--Relationship between stress ratio and the time from the first to second waves

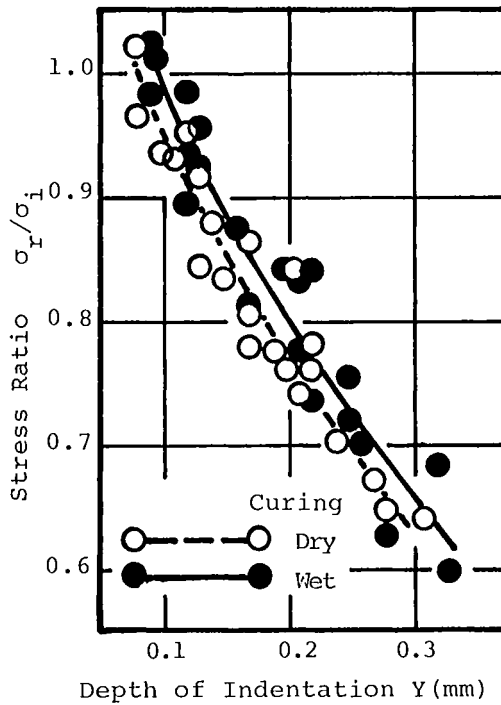


Fig. 6--Relationship between stress ratio and depth of indentation

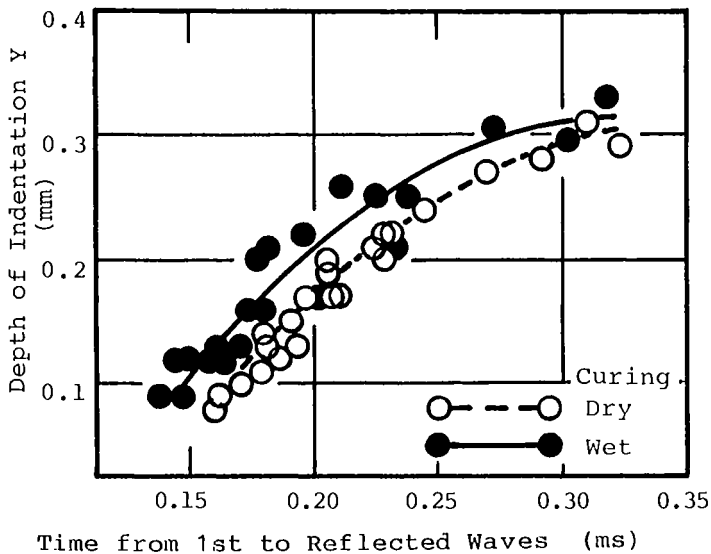


Fig. 7--Relationship between depth of indentation and the time from the first to the second stress waves

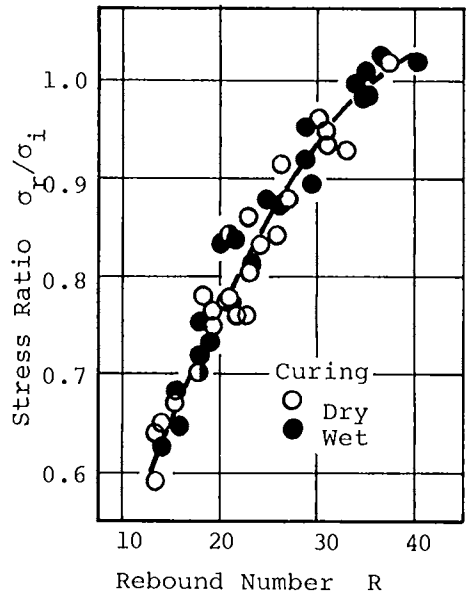


Fig. 8--Stress ratio versus rebound number

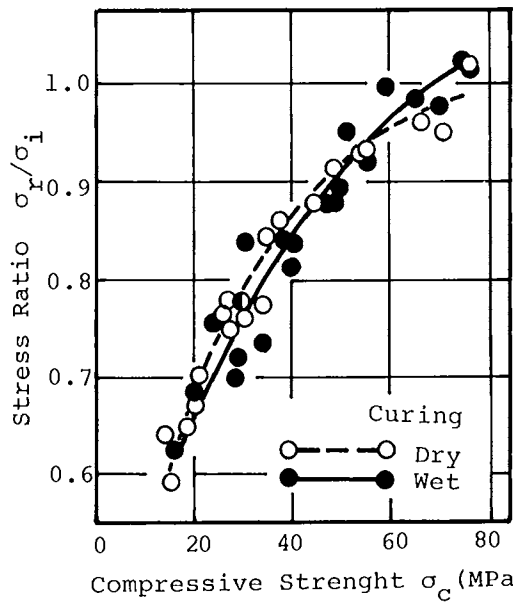
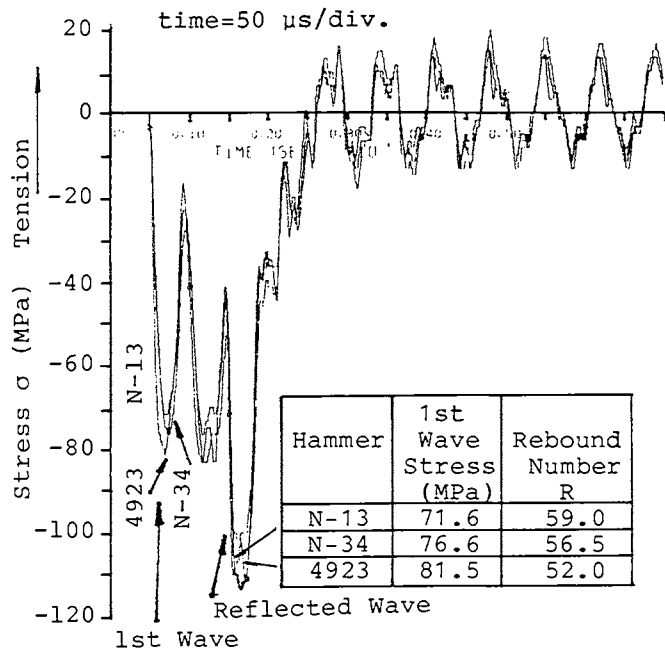
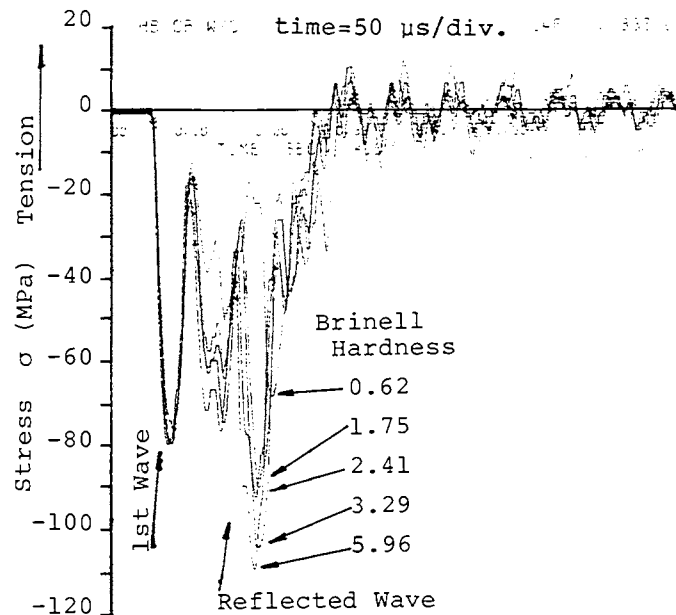


Fig. 9--Stress ratio versus compressive strength



(a):Difference with Hammer on the Anvil



(b):Difference with Brinell Hardness by Hammer N-34

Fig. 10--Variation of impact stress at center of the test plunger

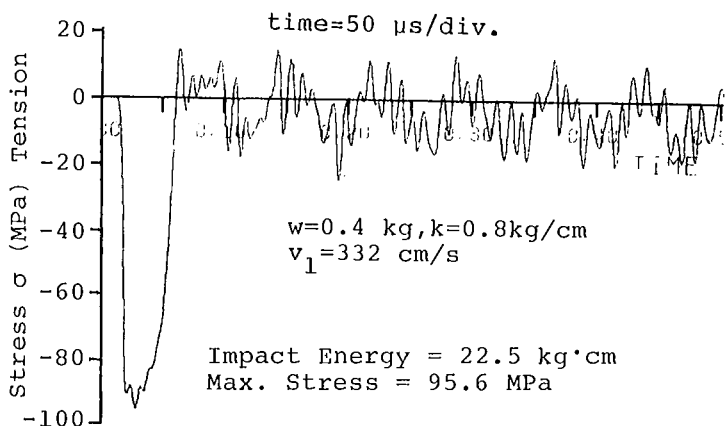


Fig. 11--Variation of stress wave by Smith's numerical solution

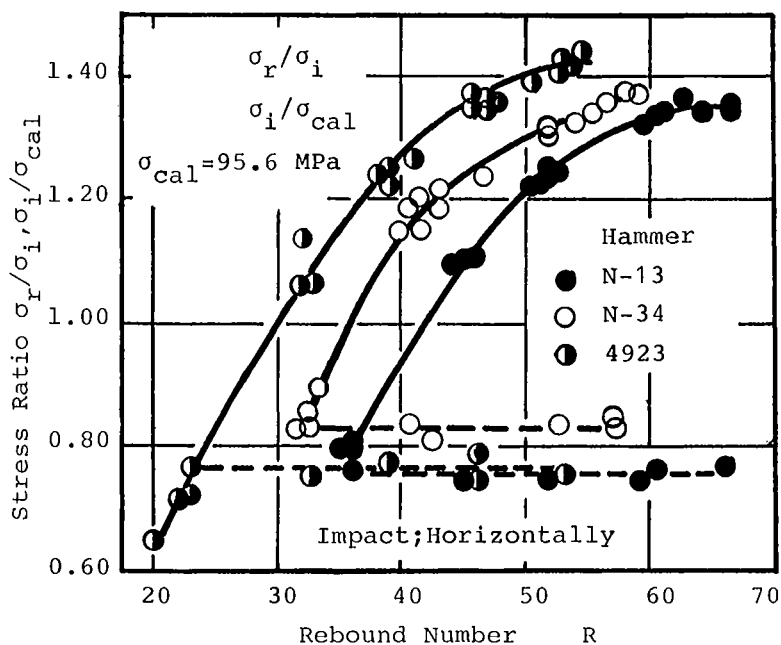


Fig. 12--Relationship between rebound number and stress ratio

Romanian Achievements in Nondestructive Strength Testing of Concrete

By Ioan Facaoaru

Synopsis: This paper summarizes Romanian experience concerning in-situ concrete strength estimation by non-destructive methods. The methods presented include "classical" non-destructive methods such as ultrasonic pulse method and rebound method. The philosophy of transforming the measured property into concrete strength, taking into account concrete composition and maturity, is given. Details about original developments of non-destructive methods for concrete strength estimation are also presented. Such developments are:

- a) combined SONREB method based on pulse velocity and rebound index measurements,
- b) sonic coring method, an immersion method using radial transducers, adapted for the inspection of deep foundations,
- c) acoustic method by shock, a method based on the measurement of the natural period for the quality control of precast units.

Keywords: compressive strength; concretes; impact hammer tests; nondestructive tests; ultrasonic tests.

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INTRODUCTION

In Romania non-destructive testing methods are primarily used for in-situ concrete strength estimation due to following reasons /2, 3/:

- a) A large number of structures are made of concrete.
- b) There is a weak correlation between the quality of concrete, in specimens and in a structure.
- c) The strength of concrete is not a constant during the service life of a structure, especially when exposed to adverse external conditions.

Romania initiated researches in this field, about 30 years ago and has official regulations, which have been renewed several times, during the past 20 years /4, 5, 6/. This paper is aiming to summarize the experience acquired especially in the field of in-situ testing.

COMBINED SONREB METHOD

This method is based on simultaneous measurements of longitudinal pulse velocity and rebound number /1/. It is the most popular non-destructive method in Romania due to the following advantages:

- a) The accuracy of results is, within large limits, not influenced by moisture content and maturity of concrete and less influenced by cement content and aggregate grading, in respect to ultrasonic method, and less influenced by carbonation effects, in respect to rebound method.
- b) The results provide information along the tested thickness of concrete.
- c) There is no need for a special preparation, before casting the concrete and no local damage is brought to the unit, after testing.

The method is based on the definition of a reference concrete, for which a family of iso-compressive strength curves, in a system of pulse velocity - rebound number axes, are plotted. The reference concrete, for the combined method is defined through the following factors: type and amount of cement, nature and grading of aggregates. An example of iso strength curves for the reference concrete adopted in Romania, is given in Fig. 1. In order to get different experimental points the following factors can be varied: water-cement ratio, degree of compaction, moisture content and maturity of concrete.

If the concrete to be tested is not the reference one, in order to get the real strength R_{ef} , the result obtained according Fig. 1 (R_{ref}) has to be corrected, by a correction factor, named total coefficient of influence (C_t), according to the relation (1):

$$R_{ef} = R_{ref} \cdot C_t \quad (1)$$

The ways of determining the C_t value are different depending on the following:

a) When test specimens or cores from the tested concrete are available. In this case the experimental determination of the C_t value is possible by testing, non-destructively and destructively, these specimens and taking the mean value of results according relation (2):

$$C_t^{exp} = \frac{1}{n} \sum_{i=1}^n \frac{R_{ef,i}}{R_{ref,i}} \quad (2)$$

where: $R_{ef,i}$ - is the strength obtained destructively on specimen i .

$R_{ref,i}$ - the strength given by graph 1, for the non-destructive measurements on specimen i .

n - the number of tested specimen.

b) When composition of tested concrete is known. In this case a semi-theoretical value of the C_t coefficient can be computed, according relation (3):

$$C_t^{sth} = C_c \cdot C_d \cdot C_a \cdot C_g \cdot C_\phi \quad (3)$$

where: C_c - is the coefficient of influence of cement type

C_d - the coefficient of influence of cement content

C_a - the coefficient of influence of aggregate nature

C_g - the coefficient of influence of aggregate fine fraction (0.1 mm)

C_ϕ - the coefficient of influence of maximum size of aggregate

If certain admixtures are used, a coefficient of their influence has to be added. An example of the values used for these partial coefficients of influence is given for C_d values in Table 1, column 1.

c) When both composition of concrete and test specimens are available. In this case the two total coefficients of influence are computed and compared.

If their difference is less than 20%, a weighted mean value has to be adopted, according relation (4):

$$C_t = \frac{2 C_t^{\text{exp}} + C_t^{\text{sth}}}{3} \quad (4)$$

If their difference is greater than 20%, an analysis should be developed, in order to find the less applicable value. It is usually the C_t^{sth} value which is under question mark.

d) When neither cores, specimens are available nor the composition of concrete is known. In such cases auxiliary information has to be taken into consideration such as: concrete grade, time of construction, analysis of a broken corner, in order to decide the value of C_t to be adopted. The experience of the investigator leading the tests play an important role, under such circumstances. The accuracy (range in which 90% of results are comprised) is considered to be:

a) 10-14% when specimens or cores are available and composition is known.

b) 12-16% when only specimens or cores are available.

c) 15-20% when only composition is known.

d) above 20%, according the experience of the investigator and the auxiliary data, if neither specimens nor the composition of concrete are available.

ULTRASONIC PULSE METHOD

This method is based actually on the exclusive measurement of longitudinal pulse velocity. The method also allows the measurement of other interesting properties of concrete such as attenuation of ultrasonic pulses, period of the first half wave, transversal pulse velocity or the so called reverberation time but none of these have found practical applications. Even with these limitations the method is very useful, when testing concrete of well known composition, for which a calibration curve has been derived. Such examples are: the quality control in precast factories or on large building sites. The method is adequate to control large units of known maturity of concrete, kept under well known conditions of humidity. It can be also used when only a limited number of specimens are available (for example 3) according the following procedure.

The correlation pulse velocity (v) - compressive strength (R_c) is described by an exponential relation of the type:

$$R_c = a e^{1.1 v} \quad (5)$$

where pulse velocity (v) is expressed in km/s and constant (a) has to be determined according the concrete composition.

For reference concrete, the constant (a) has the value:

$$a_r = 0.24 \text{ MPa} \quad (6)$$

The reference concrete used in ultrasonic method is described in terms of cement type and content, aggregate nature and grading, concrete maturity and humidity, absence of admixtures. Its transformation curve is given in Fig. 2, under the index $C_t = 1.00$. If the tested concrete is not the reference one, the coefficient (a) is given by the relation:

$$a = a_r \cdot C_t \quad (7)$$

where the total coefficient C_t can be determined in the following ways:

a) Experimentally if test specimens or cores are available. The specimens are tested non-destructively and destructively and the obtained data are plotted on Fig. 2. The center of gravity of the polygon formed by

40 Facaoaru

the points gives the value of C_t^{exp} coefficient, to be adopted for the concrete under test. Alternatively, one can obtain the ratios:

$$C_{t,i} = \frac{R_{c,i}^{\text{exp}}}{R_{c,i}^{\text{ref}}} \quad (8)$$

for each specimen and average the value for all tested specimens. In relation (8) $R_{c,i}^{\text{exp}}$ is the compressive strength of specimen i , obtained destructively and $R_{c,i}^{\text{ref}}$ is the strength obtained on base of pulse velocity measurement, using the curve of reference concrete ($C_t=1.00$) from Fig. 2.

b) Semi-theoretically if the composition, humidity and maturity of the tested concrete are known.

In such cases tables have been prepared for all the factors affecting the pulse velocity - strength correlation. An example of such table, for the amount of cement influence is given in Table 1, column 2. If several factors are different from the reference concrete composition, the total coefficient of influence is computed according the formula:

$$C_t^{\text{sth}} = C_c \cdot C_d \cdot C_a \cdot C_\phi \cdot C_g \cdot C_h \cdot C_m \cdot C_e \quad (9)$$

where: C_c - is the type of cement coefficient of influence

C_d - the amount of cement coefficient of influence

C_a - the aggregate nature coefficient of influence

C_ϕ - the maximum size of aggregate coefficient of influence

C_g - the fine fraction (0-1 mm) of aggregate coefficient of influence

C_h - the humidity of concrete coefficient of influence

C_m - the maturity of concrete coefficient of influence

C_e - the admixture coefficient of influence.

c) When specimens and composition of concrete are available.

In such cases the two values C_t^{exp} and C_t^{sth} can be computed, including their weighted mean value, according

relation (4). In case of disagreement, an analysis of the causes should be developed, in order to detect the less applicable value, which in most cases is the semi-theoretical one.

The accuracy of strength estimation by ultrasonic pulse method is as follows:

- a) 12-16% when specimens or cores are available and composition is known.
- b) 14-18% when only specimens or cores are available.
- c) 18-25% when only composition is known.
- d) well above 30%, depending upon the experience of the investigator and the existence of auxiliary data if neither specimens nor the composition of concrete are available.

The equipment used for ultrasonic testing in Romania is of local manufacture, characterised by the following:

- a) Equipment with cathod ray tube (Betonoscop N 27o1):
 - weight : 9 kg
 - accuracy of time measurement : $\pm 1\%$
 - signal amplification: adjustable up to 80 dB and measurable
 - maximum concrete thickness investigated: about 3 m
 - heads: emitter magnetostrictive and receiver piezoelectric of 50 or 100 kHz frequency
 - supply: battery or main.
- b) Equipment with digital display (Betonoscop N 27o2):
 - weight: 1,5 kg
 - accuracy of time measurement: $\pm 1\%$
 - signal amplification: constant 100 dB
 - heads: emitter magnetostrictive and receiver piezoelectric of 50 or 100 kHz frequency
 - supply: battery.
- c) Equipment with cathod ray tube and digital display of measurement (Betonoscop N 27o3):
 - weight: 7kg
 - accuracy of time measurement: $\pm 1\%$
 - signal amplification: adjustable and measurable up to 100 dB
 - maximum concrete thickness investigated: about 4 m
 - heads: emitter magnetostrictive and receiver piezoelectric of 50 and 100 kHz frequency
 - supply: battery or main.

SURFACE HARDNESS METHOD

From the large class of surface hardness methods, in Romania, we prefer the Schmidt hammer method, based on the rebound measurement. The reason for this choice is that the manufacturer was able to produce the most reproducible equipment of its class and its use facilitates an easy measurement.

The main development, specific for our country, concerns the way of transforming the rebound number measurement into concrete compressive strength. We have recognised that it is no unique correlation for rebound number - strength. Like in the case of previous non-destructive methods, we have defined a reference concrete, characterised by cement type and amount, nature of aggregate, maturity and moisture content of concrete. The rebound number - compressive strength correlation for this concrete is given in Fig. 3, under the value $C_t = 1$. Such a curve is represented by the relation:

$$R_{ref} = 0.01 N^{2.2} \quad (10)$$

where the rebound number N is expressed in scale divisions, in order to obtain the strength in MPa.

If the tested concrete is not the reference one, the result given by relation (10) has to be corrected according relation (11):

$$R_{ef} = R_{ref} \cdot C_t \quad (11)$$

where C_t is again a coefficient of influence of concrete composition and maturity and can be obtained as follows:

a) Experimentally if specimens or cores are available. In this case they are tested non-destructively and destructively. The obtained results are plotted in Fig. 3 and the center of gravity of the figure formed by the points defines the experimental value of C_t named C_t^{exp} . Alternatively a procedure based on the computation of the ratios:

$$C_t^{exp} = \frac{R_{exp,i}}{R_{ref,i}} \quad (12)$$

for each specimen and their mean value can be adopted. We remember that $R_{exp,i}$ is the strength obtained when testing destructively specimen i and $R_{ref,i}$ is the

strength of specimen i , derived from reference concrete curve (Fig. 3), on the basis of the measured rebound number values for specimen i .

b) Semi-theoretically if the composition of the tested concrete is known. The relation is of the form:

$$C_t^{\text{sth}} = C_c \cdot C_d \cdot C_a \cdot C_m \cdot C_h \quad (13)$$

where: C_c - is the type of cement coefficient of influence

C_d - the cement amount coefficient of influence

C_a - the nature of aggregate coefficient of influence

C_m - the concrete maturity coefficient of influence

C_h - the concrete humidity coefficient of influence

An example for the C_d values, for the rebound method, is given in Table 1, column 3. The values of the other partial coefficients of influence have to be taken from similar tables. Different correlation curves, rebound number - compressive strength for different C_t values, are given in Fig. 3.

c) Combined way using both the experimental way leading to C_t^{exp} value and the semi-theoretical one leading to C_t^{sth} . The final value adopted in this case for the C_t coefficient is the one given by relation (4), on condition that the difference between the two coefficients is not larger than 25%. If it is not the case, an analysis should be performed, in order to eliminate the less applicable value.

d) Neither specimens, cores nor the composition of the tested concrete are available. In such cases auxiliary informations should be collected in order to compensate for this lack of data. Because of the influence of carbonation on the measurements it is not recommended to test old buildings using exclusively the rebound method.

The accuracy of Schmidt rebound method is as follows:

a) 12-18% if specimens or cores and concrete composition are available

b) 15-20% if only specimens or cores are available

44 Facaoaru

c) 18-28% if only the composition of concrete is known

d) above 30% when only auxiliary data are available but on condition that concrete is not older than 1 year.

SONIC CORING METHOD

The method was derived from ultrasonic pulse method but adapted to the control of concrete quality in deep foundations such as drilled piles or columns, slurry walls etc. /7/. In order to facilitate the ultrasonic inspection, closed tubes are inserted together with the reinforcement cage, in the digged trench, before concrete casting. After concreting, the tubes are opened and filled with water. Imersion transducers with radial sensitivity are lowered along the tubes (Fig. 4).

In this way it is possible to measure the transit time of ultrasonic pulses across the concrete between the tubes. Certain corrections have to be applied to the direct time measurement to substract the propagation time in water and in tube walls, both at the emitting and at the receiving transducer. Once the propagation time is known, the computation of longitudinal pulse velocity and hence the estimation of concrete strength can be done, according the procedure described earlier. Nevertheless we want to point on some details which allow a higher accuracy in concrete strength estimation:

a) It is necessary to preserve at least 3 specimens, to be tested by classical ultrasonic pulse technique and destructively, at about the same maturity as the concrete in the units.

b) When choosing the final velocity-strength correlation it should be remembered that usually the moisture content and sometimes the maturity of concrete in specimens and in the units is different.

c) Information should be collected about the composition of in place concrete in order to be able to check the experimental value of the total coefficient of influence. In Fig. 5 there are illustrated the pulse velocity and concrete strength variations along the height of a slurry wall cast with concrete (class B20) of special composition. Measurements have been performed down to a depth of 25 m, with distance between tubes up to 2 m. The measurement interval in our country is usually 25 cm but if a detailed inspection is required it can be decreased to 12.5 cm and even below this. The number of embeded tubes depends on the desired accuracy of the inspection and on the size of unit. This number is usually between 2 and 6 or even more. The internal diameter

of the tubes has to be between 50 mm and 80 mm. Tubes of plastics or steel can be used but the latter are preferred for their rigidity. The method is able to detect defects along the unit depth, locating their position and intensity.

ACOUSTIC METHOD BY SHOCK

This method is a close relative of resonance frequency method but instead of using the excitation by an electrodynamic generator, of adjustable frequency, it uses simply a mechanical shock applied to the tested unit. The receiver chain is composed from an accelerometer and an electronic block, enabling to measure the natural period of vibration of the unit, when the unit enters in a stable regime of vibration (Fig. 6). In respect to resonance method it has the advantage to be applied to any size and shape of unit and to enjoy a much simpler emission chain. All the three main types of vibrations can be excited: longitudinal, flexural or torsional (Fig. 7) but on large size units we concentrate on the use of longitudinal vibrations as being the most reproducible. The intensity of the shock plays an important role, in getting reproducible results. A correlation between the mass of the unit and the mass of the impact hammer has been derived. It is also important to know the order of magnitude of the expected natural period of the unit, in order to remove false informations. When testing concrete units, the following approximate formula can be used to find this order:

$$\text{- Longitudinal vibrations: } T_L \simeq \frac{L}{2} \quad (14)$$

$$\text{- Flexural vibrations: } T_F \simeq 0.6 \frac{L}{a} T_L \quad (15)$$

$$\text{- Torsional vibrations: } T_T \simeq 1.7 T_L \quad (16)$$

where all periods are obtained in μs if the length (L) and the height (a) of the unit are expressed in mm.

The testing technique consist in applying a mechanical shock of adequate intensity, in the adequate place, to the unit and in measuring automatically, by using the receiver chain, the natural period of the unit, when this enters in its own regime of vibration. The result is displayed with three digits.

If the natural period T_L is known the following stages have to be covered:

- computation of dilatation wave velocity:

$$v_D = \frac{L}{T_L} \quad (17)$$

- computation of longitudinal wave velocity:

$$v_L = \sqrt{\frac{1-\nu_d}{(1+\nu_d)(1-2\nu_d)}} v_D = \alpha v_D \quad (18)$$

where, according our experience, the values of α vary between 1.11 - 1.14.

- computation of concrete strength (R_c) taking into account all considerations mentioned for ultrasonic pulse method.

It is still a difference of information, between acoustic method by shock and ultrasonic pulse method. The informations given by the first method are influenced by the whole unit, where as for the second, only by the properties along the path length.

Table 2 gives an image about the concrete strength obtained on specimens (S), beams (B), columns (C), diaphragms (D) and plates (P), of different concrete composition and maturity using the acoustic method by shock (column S), in certain cases in parallel with resonance method results (column R).

An attempt has been made to use the flexural period method also for estimating the stresses in a prism, loaded in compression. The results are represented in Fig. 8a and b and show that it is a marked decrease of the natural period of the specimen as the stress increases. The period decrease is very sharp at small stresses and tends to diminish as the stress increases, at about 70% of the fracture load being almost null and very close to fracture ($0.9 R_{pr}$) even reversing the sense of variation. In order to avoid the two values (σ/R_{pr}) corresponding to the same value T_f (Fig. 8b) a representation $\Delta T/\Delta \sigma$ as function of σ/R_{pr} has been proposed (Fig. 9). The method seems very accurate for small stresses estimation.

CONCLUSIONS

Romanian experience in non-destructive testing of concrete for in-situ compressive strength estimation has led to the following conclusions:

a) None of the existing non-destructive methods can estimate concrete strength without additional information about concrete composition and sometimes its maturity.

b) Influence of concrete composition and maturity can be expressed either by using the test specimens or cores from the concrete under test or by knowing explicitly its composition and using a system of coefficients of influence depending on this composition, to correct a relation established for a reference or standard concrete.

c) One of the original developments in Romania is the SONREB method based on pulse velocity and rebound number measurements. The main advantage of this method is that the relationship between the measured properties and strength is independent of concrete maturity and humidity. To a lesser degree it is also independent of aggregate grading and cement amount as compared with the ultrasonic pulse method.

d) Another original development is the equipment and the method for measuring concrete strength in deep foundations by sonic coring. For this method immersion transducers with radial sensitivity which can be connected to a usual ultrasonic concrete tester have been developed. The method is an equivalent of ultrasonic pulse method but enabling to control units buried under ground.

e) Still another original development is the equipment and the method for quality control of precast units of any shape and size by acoustic method with shock. This method is an equivalent of resonance frequency method but allows the testing of any unit, regardless of its dimensions and shape by using a much simpler system of excitation.

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Table 1 - C_d Values for non-destructive methods

Cement content, kg/m ³	C_d		
	SONREB method	Ultrasonic method	Rebound method
0	1	2	3
100	-	0.46	0.67
200	0.88	0.75	0.88
300	1.00	1.00	1.00
400	1.15	1.21	1.12
500	1.27	1.38	1.22
600	1.38	1.50	1.30

Table 2 - Tests by longitudinal shock

Type,	L, cm	Section, cm x cm		T_L , μ s	f_L Hz		v_D , m/s	v_L , m/s	R_c MPa
					S ⁺	R			
0	1	2		3	4	5	6	7	8
S	10	10 x 10		51	19607	-	3940	4330	55
S	20	20 x 20		110	9090	-	3670	4200	40
S	30	10 x 10		143	6990	7008	4010	4460	32
S	55	10 x 10		275	3630	3620	3950	4350	29
B	299	34 x 38		1920	521	-	3120	3460	12
B	530	34 x 50		2600	385	-	4080	4520	38
C	790	30 x 60		4020	249	-	3930	4360	29
P	397	160 x 12		2020	495	-	3760	4170	27
D	300	255 x 14		1605	623	-	3740	4150	27

+ Note: The values of frequency given in column (4) has been deduced from the measured values of the natural period of vibration given in column (3).

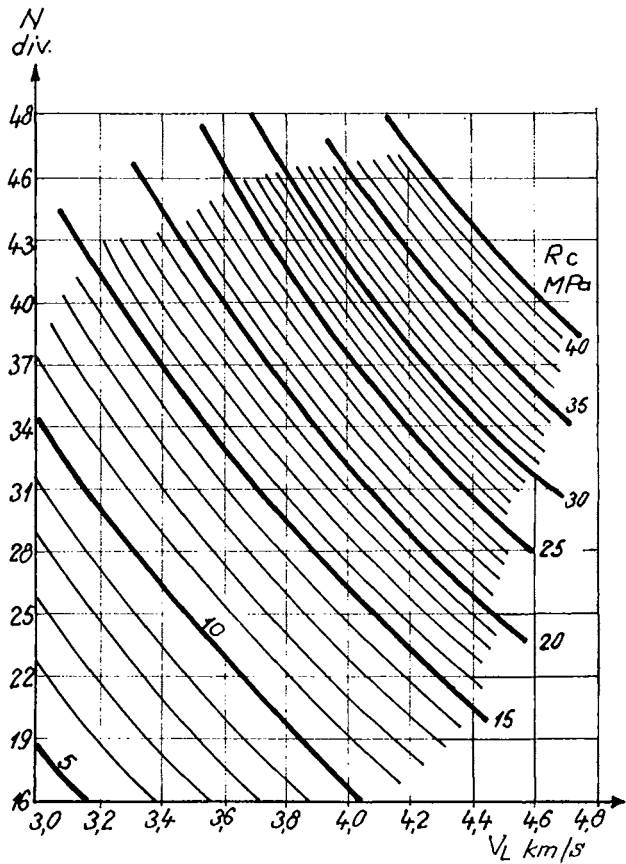


Fig. 1--ISO-strength curves for reference concrete in SONREB-method