SPECIAL REPORT

17

State of the Art:
Non-Destructive Testing Techniques of Concrete Bridges

IRC HIGHWAY RESEARCH BOARD
NEW DELHI
1996

SPECIAL REPORT

State of the Art: Non-Destructive Testing Techniques of Concrete Bridges

Copies can be had by sending advance payment from the Secretary, Indian Roads Congress, Jamnagar House, Shahjahan Road, New Delhi - 110011
Price : Rs.180
(Plus packing and postage)

IRC HIGHWAY RESEARCH BOARD
NEW DELHI
1996

Reprinted : October, 2019

ACKNOWLEDGEMENTS

The financial assistance received from the All India Council for Technical Education for publication of this Report is thankfully acknowledged.

Edited and Published by Secretary, IRC Highway Research Board, New Delhi-110011,
Printed by I. G. Printers Pvt. Ltd., New Delhi - 110 020
300 Copies

MEMBERS OF I.R.C. HIGHWAY RESEARCH BOARD

(As on 24th November, 1994 when the Report was approved)

Chairman: The Director General (Road Development), Ministry of Surface Transport (Roads Wing), Transport Bhawan, NEW DELHI-110001 ... **Shri. M.V. Sastry**

Secretary: The Secretary, Indian Roads Congress, Jamnagar House, Shahjahan Road, NEW DELHI-110011 ... **Shri S.C. Sharma**

Members:

The President,
(Shri M. Shivananda)
Indian Roads Congress,
Jamnagar House,
Shahjahan Road,
NEW DELHI-110011

The Director,
(Prof. D.V. Singh)
Central Road Research Institute,
P.O. CRRI, Okhla,
NEW DELHI-110020

The Additional Director General (Roads),
Ministry of Surface Transport
(Roads Wing), Transport Bhawan,
NEW DELHI-110001

The Additional Director General (Bridges),
Ministry of Surface Transport
(Roads Wing), Transport Bhawan,
NEW DELHI-110001

The Director General Border Roads,
(Lt. Gen. V. Shinghal)
Kashmir House, DHQ P.O.
NEW DELHI-110011

Shri H.P. Jamdar
Secretary of the Govt. of Gujarat,
R&B Deptt., New Sachivalaya,
GANDHINAGAR-382010

Shri B.V. Raghavaswamy
Engineer-in-Chief (R&B),
Errum Manzil,
HYDERABAD-500482

Shri U. Borthakur
Secretary to the Govt. of Meghalaya,
P.W.D., (R&B), Lower Lachumiere,
SHILLONG-793001

Shri G.C. Mitra
Engineer-in-Chief-cum-Secretary,
Works Department, Nirman Soudha,
BHUBANESWAR-751001

Shri D.N. Handa
Chief Engineer,
National Highways,
HP PWD,
SHIMLA-171001

The Director,
Highways Research Station
Guindy,
MADRAS-600025

Dr. A.K. Gupta
Prof. & Co-ordinator, COTE,
Deptt. of Civil Engg.,
University of Roorkee,
ROORKEE-247667

Prof. G.V. Rao
Prof. of Civil Engg.,
Indian Institute of Technology,
Hauz Khas,
NEW DELHI 110016

Dr. T.N. Subba Rao
18-E, Block, Dhanraj Mahal,
C.S.M. Marg,
MUMBAI-400039

Shri K.K. Sarin
DG(RD) & Addl. Secy.,
MOST (Retd.),
S-108, Panchsheel Park,
NEW DELHI - 110017

Co-opted Members:

The Deputy Director General (Designs),
(Brig. Kulwant Singh)
E-in-C's Branch, Army Headquarters,
Kashmir House, DHQ P.O. ,
NEW DELHI-110011

The Executive Director (Civil Engg.),
Railway Board,
Rail Bhawan,
NEW DELHI-110001

Shri N.V. Merani
Principal Secretary (Works) (Retd.),
A-47/1344, Adarsh Nagar,
MUMBAI-400025

Shri M.K. Agarwal
Engineer-in-Chief, Haryana PWD, B&R (Retd.),
H.No.40, Sector 16,
PANCHKULA-134109 (Haryana)

Shri D.P. Gupta
DG(RD) & Addl. Secy., MOST (Retd.),
E-44 (Ist Floor), Greater Kailash
Enclave - Part-I,
NEW DELHI-110048

Shri C.R. Alimchandani
Chairman & Managing Director,
STUP Consultants Ltd.,
1004-5&6, Raheja Chambers,
313, Nariman Point,
MUMBAI-400021

FOREWORD

The need for conducting non-destructive testing for condition assessment of bridges has grown considerably in recent times due to increasing number of bridges showing signs of distress. Today we require more advanced assessment techniques and precision instruments for bridge inspection. A large variety of techniques and methods have been developed world over to assess material properties and member responses. Scope, applicability and accuracy vary considerably from instrument to instrument and many of them are at different levels of development.

The HRB Dissemination Committee entrusted the work of preparation of draft State-of-the-Art Report on 'Non-Destructive Testing of Concrete Bridges' to Dr. M.G. Tamhankar of SERC, Ghaziabad. Subsequently, a Working Group was constituted with the following composition :

Dr. M.G. Tamhankar, SERC	Convenor
Shri S.C. Gupta, CPWD	Member
Shri R.C. Wason, NCB	Member
Shri M.V.B. Rao, CRRI	Member
Dr. N.S. Rengaswamy, CECRI	Member
Jt. Director General (B&S), RDSO	Member
Dr. A.R. Santhakumar, Anna University	Member

The draft report prepared by Dr. M.G. Tamhankar with assistance from S/Shri S.S. Gaharwar and V.K. Singh from SERC was circulated in October, 1991 to members. Based on the suggestions made in the subsequent meetings and the inputs received from the members, the draft was modified and circulated in May, 1993. Techniques for Monitoring Reinforcement Corrosion compiled by Dr. Rengaswamy and his colleagues from CECRI were subsequently included. Comments on this revised draft were considered by the Group in their meeting held in March, 1994. The report was discussed and approved by HRB's Identification, Monitoring and Research Application Committee on 22nd July, 1994. Thereafter, the Highway Research Board approved the report during its 31st Meeting held on 24th November, 1994.

The State-of-the-Art Report brings out the operating principle, specifications, limitations and calibration of the known techniques to the extent available in the literature. Inspite of advancement in techniques of modern gadgets for testing, the role of visual inspection cannot be ignored. In fact it constitutes the first step in physical assessment which forms the basic input for detailed assessment. It hoped that the report will be of great use to members of the profession.

M.V. Sastry
Chairman,
IRC Highway Research Board &
Director General (Road Development),
Ministry of Surface Transport (Roads Wing)

New Delhi
May, 1995

HRB IDENTIFICATION, MONITORING AND RESEARCH APPLICATION COMMITTEE

(As on the 22nd July, 1994 when the Report was approved)

Prof. D.V. Singh	*... Convenor*
V.K. Sood	*... Member-Secretary*

Members

Dr. S.M. Chakravarty	Dr. A.K. Gupta
Dr. C.E.G. Justo	Dr. N.B. Lal
P.K. Lauria	K.N. Kale
N.V. Merani	Prof. T.S. Nagaraj
P.R. Nair	Dr. B.B. Pandey
Dr. S. Raghavachari	Dr. C. Rajkumar
Dr. A.C. Sarna	Dr. S.P. Srivastava
Prof. C.G. Swaminathan	Chief Engineer (Roads) Std/R, Ministry of Surface Transport, New Delhi
Chief Engineer (T&T), Ministry of Surface Transport, New Delhi	Director, R & B Research Institute, 24-Parganas (W.B.)
Engineer-in-Chief, Haryana P.W.D., B&R, Chandigarh	Engineer-in-Chief, A.H.Q., New Delhi
Chief Engineer (Designs), C.P.W.D., New Delhi	Director, Highways Research Station, Madras
	D.D.G. (DGBR), New Delhi
Indian Institute of Technology, Mumbai (Prof. P.K. Sikdar)	Director, Gujarat Engineering Research Institute, Vadodara
Director, SERC, Ghaziabad	

Ex-Officio Members

President, IRC	(M. Shivananda)
DG(RD), MOST	(M.V. Sastry)
Secretary, IRC	(S.C. Sharma)

MEMBERS OF WORKING GROUP FOR PREPARATION OF STATE-OF-THE-ART REPORT ON "NON-DESTRUCTIVE TESTING TECHNIQUES OF CONCRETE BRIDGES"

1. Dr. M.G. Tamhankar ....Convenor
   Deputy Director,
   Structural Engineering Research Centre,
   Central Govt. Enclave, Sector - 19,
   Kamla Nehru Nagar, Post Bag No.10,
   GHAZIABAD-201002

2. Dr. A.R. Santhakumar,
   Professor of Civil Engineering,
   College of Engineering, Anna University,
   MADRAS-600005

3. Shri S.C. Gupta,
   Chief Engineer (Designs), C.P.W.D.,
   Room No.221, A-Wing, Nirman Bhawan,
   NEW DELHI-110011

4. Shri R.C. Wason,
   Scientist, National Council for Cement and Building Materials,
   BALLABHGARH
   (Haryana)

5. Dr. N.S. Rengaswamy,
   Scientist, Central Electrochemical Research Institute,
   KARAIKUDI-623006 (Tamil Nadu)

6. Shri M.V.B. Rao,
   Head, Bridges Division,
   Central Road Research Institute,
   P.O. CRRI, Okhla,
   NEW DELHI-110020

7. The Joint Director Research (B&S)
   Research, Designs & Standards Organisation,
   Ministry of Railways, Manak Nagar,
   LUCKNOW-226011

CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

Non-destructive testing is an activity to assess variation from the design and monitor performance. It covers a wide field ranging from assessment by the naked eye to the techniques which are yet in the research stage. With the increasing number of structures showing signs of distress a greater need is felt to apply more advanced assessment techniques and equipment in bridge inspection. A large variety of techniques and methods have been developed to assess material properties and member responses [119]. The list is ever increasing. Scope, applicability and accuracy vary considerably from instrument to instrument and many of them are at different levels of development.

This report presents the State-of-art and brings out the operating principle, specifications, limitations and calibration of the known techniques to the extent available in the literature. In the midst of advanced micro-processor controlled array of emerging gadgets, role of Visual Inspection cannot be ignored. In fact it constitutes the first step in physical assessment to identify the common symptoms of distresses, such as, cracking, spalling, honeycombing, rusting, water leakages, etc. However, for meaningful visual inspection, built-in access, availability of mobile inspection unit and understanding of the behaviour of the structure are desirable requisites.

2. EXECUTIVE SUMMARY

An overview of the parameters which can be monitored and the range of available techniques is presented in Table 2.1. The Table gives the parameters to be monitored and the possible non-destructive techniques for their measurement, to facilitate field personnel to make selection.

Potential and Limitations of Various techniques cited in Table 2.1. are briefly discussed in the following paragraphs to appraise users of their relevance in field applications.

Table 2.1. Summary of Parameters vs Measuring Techniques

Parameter	Reported Techniques
(I) CONCRETE	
Compressive Strength	Rebound Hammer Test Windsor Probe Test Core Test Lok Test Capo Test North American Pull-out Test
Flexural Strength	Break-off Test
Direct Tensile Strength	Pull-off Test
Homogeneity	Ultrasonic Pulse Velocity Test Accoustic Emission Method Pulse-echo Method Gamma Ray Radiography Endoscopy Integrity Testing of Piles
Permeability	Initial Surface Absorption Test Figg's Air and Water Permeability Tests Visual Inspection for Water Leakages and Accumulation
Concrete Lamination	Radar Technique Infrared Thermography
Chemical Composition	Wet Chemical Analysis for Total Chlorides Potential Measurement Technique for Chlorides Determination Carbonation Test X-ray Diffractometry (XRD) X-ray Fluorescence Spectroscopy (XRF) Differential Thermal Analysis (DTA)
Microscopic Examination	Petrography

Parameter	Reported Techniques
Cracks	Ultrasonic Pulse Velocity Test Accoustic Crack Detector Dye Penetration Test X-ray Radiography Gamma Radiography Portable Crack Measuring Microscope Visual Inspection for Location and Patterns of Cracks
(II) STEEL	
Stresses	Stressmeter Based on Magnetic Properties vs Mechanical Stress VW Wire Load Gauge
Snapping	Sensor Indicating Snapping Through Change in Polarity of Magnetised H.T. Wire
Location and Cover	Magnetic Detector Radar Technique
Condition	Endoscope/Borescope
Corrosion	Open Circuit Potential Surface Potential Polarisation Resistance Impedence Technique Electrochemical Noise Concrete Resistivity Electrical Resistance Probe Electrical Resistance of Prestressing Steel Visual Inspection for Stains
(III) GLOBAL BEHAVIOUR	
Movements	Geodetic Instruments Dial Gauge Hydrostatic Levelling Deflectometer Displacement Transducer Visual Inspection for Gaps in Hand-Rails/Parapets
Strain	VW Acoustic Strain Gauge Mechanical Strain Gauge Electrical Strain Gauge
Rotation	Tiltmeter Inclinometer
Pressure	Pressure Transducer
Temperature	Thermometer Thermocouple VW Temperature Sensor

Parameter	Reported Techniques
Overall Behaviour, Load Carrying Capacity, Elasticity, etc.	Load Test Structural Integrity Test (Signature analysis)

2.1. Potential and Limitations of Various Techniques

2.1.1. Strength of concrete

Knowledge of strength of concrete in structures is important for predicting not only the load carrying capacity of the structure but also other features related to serviceability and durability. This is because many a properties of concrete, such as, modulus of elasticity, water and air impermeability and durability resistance to environmental attack, etc. are closely related to the same set of in-structure properties of ingradients of concrete which determine its strength. By the same token, strength can be indirectly measured by measuring other properties. However, it is impossible to establish mathematically exact relationship between the property and its direct estimator due to multiplicity of contributing factors and almost infinite combinations in which they exist in nature. Various NDT techniques have two major sources introducing variability in the measured value (i) Inherent variability of in-structure property, and (ii) Method of measurement (i.e., within-test variability). The second factor includes amongst others the characteristics of instruments, skill of operator, details of method, and sensitivity of measurement to changes in the property being measured[150]

Strength of in-structure concrete is sought to be controlled by controlling the concrete mix and by following good practices of workmanship. The mix is controlled by preparing works samples in form of cubes or cylinders following standardised methods of consolidating, curing and testing at required age. In this process of standardisation, all variables except the material variability (including relative ratios of cement contents, water/cement ratio, etc.) are sought to be eliminated. Thus, the standard cube or cylinder strength is an estimator of the basic strength of concrete material and does not represent truly the in-structure strength of concrete. The strength of concrete when placed in-structure is further affected by various factors. These include consolidation, curing, moisture contents, maturity (time and temperature effects), size of member (geometric scale effect), state of stress (uniaxial or multiaxial), loading history (including duration and rate of loading) and its cumulative residual effects, state of permanent and temporary imposed deformations and internal cracking, effects of internal stresses between steel and concrete, environmental attack and many other factors. It is, therefore, extremely difficult to know the in-structure strength with any degree of certainty. At best, it is possible to estimate the most likely range of strength[150,151].

In NDT methods the strength is sought to be measured by measuring another property related to compressive strength. The ultimate aim is to establish the strength grade of concrete used in the structure (in terms of characteristic strength at 28 days) as measured by standard works cube. This is necessary since methods of design, and evaluation criteria used in the process of re-evaluation of capacity and retrofitting are built around this estimator of strength. Hence, use of reliable correlation of in-structure strength to standard cube (cylinder) strength is important. Surprisingly, this correlation is not known with adequate level of confidence and any hope of establishing this with measurable degree of certainty lies with large scale application and studies of in-structure strength using NDT techniques on structures for which works cube/cylinder strengths are also known[150]. Properly conceived and executed statistical experiment is required to be designed to separate the effects of different factors contributing to variability and to establish proper correlations between estimators.

Estimation of strength of concrete by **Rebound Hammer** cannot be viewed to be very accurate and the probable accuracy of prediction of concrete strength in a structure is ±25 per cent. Rebound indices are indicative of compressive strength of concrete to a limited depth from the surface[7,8,10,11,44]. The results are affected by many factors including the angle of test, surface smoothness, mix proportions, type of coarse aggregate, the moisture content of the concrete and carbonation of the surface[8]. If the concrete in a particular member has internal microcracking, flaws or heterogeneity across the cross-section, rebound hammer indices may not indicate the same. The rebound hammer method is suitable only for close texture concrete. Open texture concrete typical of masonry blocks, honeycombed concrete or no-fines concrete are unsuitable for this test. All correlations assume full compaction, as the strength of partially compacted concrete bears no unique relationship to the rebound numbers. Troweled surfaces are harder than moulded surfaces and tend to overestimate the strength of concrete. A wet surface may give rise to underestimation of the strength of concrete calibrated under dry conditions. In structural concrete, this can be about 20 per cent lower than in equivalent dry concrete[11]. Carbonated concrete gives an overestimation of strength which in extreme cases can be put 50 per cent[11]. Rebound Hammer test when properly calibrated on site with cubes, can be useful for measuring in-structure magnitude and variability of strength. It is most commonly used due to its simplicity and low cost.

The **Windsor Probe Test** basically measures hardness over a certain depth of concrete. Accuracy in the prediction of compressive strength is about ± 30 per cent[11,1,2,4]. The test results are, however, affected by the edge distance and the hardness of aggregates. Type of aggregate also affects the penetration depth[119]. The reliability of the test results on concrete is measured in terms of average co-efficient of variation which is 4 per cent. Shape of aggregates as well as the moisture condition also affect the calibrations, although, these two effects are small.

The **Core Test** helps to predict the compressive strength of concrete and can be used to establish the quality of in-situ concrete by cutting cores from structural members. The core strength depends upon various factors, such as, slenderness ratio, diameter of core, coring direction, core location (i.e., depth of core, exposed surface condition and position of core in the structural element). The equivalent cube compressive strength of concrete can be obtained from crushing strength fcy of core samples of 100 to 150 mm diameter having a slenderness ratio of 2 through the following relationships suggested by various investigators:

(i) fc = 1.44 fcy - 0.0066 fcy **2 (Sangha & Dhir)

(ii) fc = 1.25 fcy (BS 1881 & IS 516)

(iii) fc = A fcy - B fcy **2 (Munday et. al.)

According to IS:1189-1159, a core specimen shall have a diameter atleast three times the maximum nominal size of coarse aggregate used in the concrete and in no case shall be diameter of the specimen be less than twice the maximum nominal size of coarse aggregate. Strength of horizontal core = 0.92 (strength of vertical cores)[16]. It has been observed that strength of core is lower in the top 20 per cent portion as compared to any other part of the structural member[23]. Tassio[151] has recommended simple divisors for converting core test results to cube strengths, these divisors, approximate as they are, attempt to account for various factors affecting in-structure strength.

The **Lok, Capo, and North American Pull-out Tests** are used to measure the in-situ quality of concrete and its compressive strength through calibration. Strength calibration is claimed to be more dependable in Lok type tests than for most other non-destructive or partially destructive methods[15]. The depth of penetration is greater in this method[11,1,45,55]. The results are very sensitive to tensioning methods and procedures. In Capo Test, the test result actually depends directly on the compressive strength. This test is claimed to be more reliable in terms of its repeatability. Its limitation lies in the fact that it can be employed for near surface strength measurements only[153].

The **Break-off Test** measures the flexural strength of concrete which is correlated with the compressive strength of the core. The rupture zone is located 70 mm from the surface. Accuracy of prediction is ± 20 per cent with the aid of appropriate calibrations[11,70,71,72,73,74,75,76] The method is regarded as especially suitable for very young concrete and although, leaving a sizable damage zone, may gain acceptance as an in-situ quality control test where tensile strength is important[72]. Although, quicker than compression testing of cores, the use of results for strength estimation of old concrete may be unreliable unless a specific calibration relationship is available. The test has to be preplanned, difficulty is experienced in inserting tubes in concrete with slumps of less than 75 mm and the test cannot be used for concrete incorporating aggregates larger than 19 mm.

The **Pull-off Test** actually measures nominal tensile strength of concrete and is correlated with the compressive strength of concrete. It can measure the strength within ±20 per cent accuracy. However, fair degree of precision is required in using the instrument and in correlation of strength[66,68,69]. The test gives reproducible results and does not require planning in advance of placing the concrete. However, it is confined to testing of the surface layers of concrete.

2.1.2. Homogeneity

The **Ultrasonic Pulse Velocity Method** is used to establish homogeneity of concrete, changes in the structure of the concrete which may occur with time, values of elastic modulus of concrete, quality of one element of concrete in relation to another and also the presence of cracks, voids and other imperfections. Variations of the concrete temperature between 5^0C to 30^0C do not significantly affect the pulse velocity measurements in concrete[9]. At temperatures between 30°C to 60°C, there can be reduction in pulse velocity upto 5 per cent. Below the freezing temperature, the free water freezes within concrete resulting in an increase in pulse velocity upto 7.5 per cent. In general the pulse velocity through concrete increases with increased moisture content of concrete. This influence is more for low strength concrete than high strength concrete. The pulse velocity of saturated concrete may be upto 2 per cent higher than that of similar dry concrete. In general, drying of concrete may result in somewhat lower velocity. The shape and size of the concrete member does not influence the pulse velocity unless the least lateral dimension is less than a certain minimum value, e.g., the minimum lateral dimension of about 80 mm for 50 kHz natural frequency of the transducer. When concrete is subjected to a stress which is abnormally high for the quality of the concrete, the pulse velocity may be reduced due to the development of micro-cracks. The influence is generally insignificant unless the stress is greater than about 60 per cent of the ultimate strength of the concrete. The pulse velocity measured in reinforced concrete in the vicinity of reinforcing bars is usually higher than in plain concrete of the same composition. This is because the pulse velocity in steel is 1.2 to 1.9 times the velocity in plain concrete[9]. Although, the ultrasonic techniques[8] have the advantages of being both rapid and truly non-destructive, there are serious practical limitations to their use. The heavy reinforcement of highway bridges can make interpretation of the results very difficult. It is also difficult to align transducers properly and measure the path length accurately in the direct transmission mode. The overall within-test variability of this method is better than other NDT tests.

Within-test variability is established by following appropriate statistical techniques. Although, it is desirable to minimise this variance by using the same equipment and operators for a given set of measurements and carrying out analysis of variance, in practice this approach is not always possible and it is necessary to accept estimation of the within-test variance as established by other investigators.

The **Acoustic Emission Method (AE)** can be used to assess the homogeneity, deformation fracture and fatigue of metals and composite materials, such as, concrete. It depends on the phenomenon of material or structure transient vibrations when stressed. As these acoustic emissions are indicators of increasing stress levels in a structure, they could also possibly be used to determine the degree of change which a structure has suffered. Acoustic emissions are pulses of elastic strain energy released spontaneously during deformation. These vibrations or sound waves can be detected by placing a sensor on the surface of the material. The AE method can only detect changes in materials. Those defects already present, such as, crack cannot be revealed unless the size of the crack increases under the load. AE is, therefore, used as a supplement to other NDT methods[11,49,50,51,52,87]. It has been reported[31] that as the load level on a concrete specimen increases, the emission rate and signal level both increase slowly and consistently until failure approaches and there is then a rapid increase upto failure whilst this allows crack initiation and propagation to be monitored during a period of increasing stress. Mindess[52] has also shown that mature concrete provides more acoustic emission on cracking than young concrete, but confirms that emissions do not show a significant increase until about 80 per cent to 90 per cent of the ultimate stress. The absence of the Kaiser effect for concrete effectively rules out the method for establishing a history of past stress levels.

The **Pulse-echo Method** is used for locating defects, such as, voids, cracks and zones of deterioration within hardened concrete structures. It is based on monitoring the interaction of acoustic (or stress) waves with the internal structure of an object. The simplest version commercially available is the instrumented delamination device (IDD), which measures the amplitude of the reflected shock waves caused by a surface hammer blow[61,62,63,64]. The contact time of the impact source should be shorter than the arrival time of the first ray. However, if the contact time is made too short, the recorded signal becomes complex and difficult to interpret. The reliability of the techniques used in this work appears to decrease with increasing concrete thickness. Further work is required on the inter-relationship between concrete thickness and impact source. Pulse-echo testing using a mechanically generated stress pulse is applicable only if the test object dimensions exceed certain minimum requirements. The thickness must be large enough so that specular reflections arrive at the receiver after the surface wave. The lateral dimensions must be large enough so that reflected surface waves do not interfere with the initial portions of signal from specular reflections.

The **Radiography Method** is used to determine the position of cable, voids in grouting and in-situ density of concrete. There are three basic methods currently in use for testing concrete: X-ray radiography, Gamma ray radiography and Gamma ray radiometry. The radiographic methods consist essentially of a 'photograph' taken through a specimen to reveal picture of the interior, whereas, radiometry involves the use of a concentrated source and a detector to pick up and measure the received emissions at a localized point on the member. The choice of isotope depends upon the thickness of concrete involved[11,87]. Radiography has the edge over the other

NDT methods in situations where visible images assessed by the eye and data analysis techniques have yielded more precise information than other. However, it is expensive. It requires stringent safety precautions and is also limited by member thickness. Although, 600 mm is sometimes quoted as an upper limit, for thickness greater than 450 mm, the exposure time becomes unacceptably long[11].

Petrography is conducted through microscopic examination of thin section of concrete. It is used[112] for (a) determining whether the concrete in a construction was or was not as specified. In this case, other tests may be required in conjunction with petrographic examination; (b) determining whether alkali - silica or alkali - carbonate reactions or cement - aggregate reactions or reactions between contaminants and the matrix have taken place and their effects upon the concrete; (c) determining whether concrete has been subjected to and affected by sulphate attack or other chemical attack or early freezing or to other harmful effects of freezing and thawing; and (d) core inspection which reveals the information about the fresh or old fractures, reacted particles, reaction products, change in size or type of coarse and fine aggregates, distribution of coarse aggregate, honeycomb, segregation of components, etc.

2.1.3. Permeability

The **Initial Surface Absorption Test (ISAT)** is used as a quality control test for precast units[82]. Initial Surface Absorption is the rate of flow of water into concrete per unit area from the start of the test at a constant applied head and temperature[82,83]. The test on ovendried specimens gives reasonably consistent results, but that in other cases, the results are less reliable. Particular difficulties have been encountered with in-situ use in achieving a watertight fixing. Levitt[82] has suggested that specific limits could be laid down as an acceptability criterion for various types of construction, but the data is insufficient. The test has been found to be very sensitive to changes in quality. The method can be applied to exposed aggregates or profiled surfaces provided that an effective seal can be obtained, but is not suitable for porous or honeycombed concretes.

The **Figg's Air and Water Permeability Tests** are used as an alternative to the ISAT for quality control checking in relation to the durability[11]. For air permeation, the pressure within the system is firstly reduced to a standard value and the time for the pressure to rise to another standard value is recorded. This time is taken as a measure of the air permeation index of concrete. For water permeation, water is forced into the concrete and the time for the meniscus in a capillary tube to travel 150 mm is recorded . This time is taken as a measure of the Water Absorption Index of Concrete[11,80,81,84]. Using this method, the relationship between air pressure and time, meniscus movement and time, were both found to be nearly linear. The air and water permeability measured by this method correlate well with water/cement ratio, strength and ultrasonic pulse velocity. Aggregate characteristics have a profound effect on results, limiting the potential usage to comparative testing, but variations of drilling and plugging of the test hole are less significant. As with the Initial Surface Absorption Method, the moisture condition of the concrete will considerably influence the results. This seriously restricts the in-situ usages.

2.1.4. Concrete lamination

The high frequency pulse **Radar** has been used to detect deterioration in concrete pavements and bridge decks[8]. Radar when used on bituminous surfaced bridge decks, can measure the thickness of the surfacing[40,41]. The echoes produced from the pavement surface and the interface with the bridge deck concrete are very distinct such that the thickness can be determined accurately. It is claimed that features, like, reinforcing bars, voids, ducts, zones of varying moisture content and the thickness of slabs can be identified. The radar, however, does not work effectively if the deck surface is wet or if there is significant moisture in the bituminous surfacing because of attenuation of the signal. It should also be noted that vertical saw cut joints or cracks perpendicular to the surface are generally not readily visible to radar since the cross section of the joint or crack is quite small compared to the area of radar illumination[39,40,41]. Analysis of correlation of large amount of data and different radar signals to physical distress can, however, pose problem.

The **Infrared Thermography** has been used to detect the delamination in bridge decks and asphalt covered deck slabs[36,38]. The difference in surface temperature can be measured using sensitive infrared detection systems. The method works on the principle that a discontinuity within the concrete, such as, a delamination parallel to the surface, interrupts the heat transfer through the concrete[36,38]. In addition to the constraints imposed by weather the main disadvantage of thermography is that while a positive result is valid, a negative result may mean that the deck is free from deterioration or that it contains deterioration that could not be detected under the conditions prevailing at the time of test. This problem is expected to become less serious as more experience is gained in predicting the conditions under which thermography will identify deterioration. Infrared thermography can also be used to identify delamination and scaling in asphalt - covered deck slabs, although, the technique is much more sensitive to weather conditions than when applied to bare decks. Nevertheless, the method does have considerable promise as a rapid screening tool on both asphalt-covered and exposed concrete deck slabs to assess priorities and determine if a more detailed investigation is required.

2.1.5. Chemical composition

The **Wet Chemical Analysis** is used to measure the chloride concentration of a solution. In this test a standard solution containing 5 gms of concrete is prepared. To this solution a specified quantity of Silver Dichromate is added. As a result white coloured Silver Chloride is formed and the colour of the solution is changed. This colour of the solution is correlated with the chloride concentration of the solution. The method can be suitable for site use by non-specialised staff[85,86].

The **Potential Measurement Technique** commercially known as RCT (Rapid Chloride Test), is used for rapid determination of total chloride in hardened concrete. The test is reported to have similar precision as standard laboratory titration[104,105]. Pulverized concrete is obtained by hammer drilling and the chlorides are extracted by a special acid. The amount of acid soluble chlorides, expressed as per centage weight of concrete, is determined directly by a chloride sensitive electrode connected to an electrometer.

The carbonation of concrete can be assessed by **Carbonation Test**. Carbonation of concrete by attack from atmospheric carbon dioxide results in reduction in alkalinity/pH of the concrete and increases risk of reinforcement corrosion. The extent of carbonation can be assessed by treating with phenolphthalein indicator the freshly exposed surface of a piece of concrete which has been broken from a member to give surfaces roughly perpendicular to an external surface[106,107]. The colour change of phenolphthalein corresponds to a pH of about 9.5, whereas, the concrete is no longer protective to reinforcing steel when the pH drops below about 10[108].

X-ray Differactrometry (XRD), X-ray Fluorescence Spectroscopy (XRF) and Differential Thermal Analysis (DTA) are available for determining the Hydration characteristics of the hardened concrete and can be used for assessing the cause of deterioration[113]. The samples obtained from in-situ concrete either by drilling, coring or chiselling can be used for such studies.

In XRD technique, the powdered sample of concrete is bombarded by high energy X-rays and from the presence or absence of particular reflected beams (peaks) at the respective angles of incidence of the X-ray beams (20^{o}), different phases present or absent in the concrete can be studied. These peaks generally indicate calcium hydroxide, calcite, ettringite, calcium silicate hydrate, etc. in the concrete.

In XRF technique, a sample of concrete is bombarded by high energy X-rays and the fluorescent emission spectrum so caused is collimated into a parallel beam, directed on to the analyzing crystal within a spectrometer and reflected into a detector. The wave-lengths and densities of the fluorescent emission are measured and the constituent elements, together with their properties, can be calculated from this data. The results obtained in this method are compared with samples of known properties.

DTA technique is concerned with the rate of change of temperature of a sample as it is heated at a constant rate of heat input and involves heating a small sample of powdered concrete in a furnace together with a similar sample of inert material. The inert sample is controlled to be as nearly uniform as possible. The DTA graph has a series of peaks at particular temperatures which are characteristic for the minerals in the concrete sample under test. DTA studies are particularly useful for fire damaged concrete structures for assessment of temperatures to which the concrete was exposed during fire and also for assessment of the depth of affected concrete.

2.1.6. Cracks

The **Dye Penetration Examination** is used to reveal defects which reach the surface of non-porous materials, such as, cracks, porosities, clevages and leaks[87]. In this method a penetrating liquid, which is a dye or fluorescent is applied to the cleaned surface of the component. The penetrant is allowed to act for a period of time depending, among other things, upon the temperature and the component under examination. Excess of penetrant is carefully removed from the surface of the component, after which a developing liquid is applied and dried off. The developer acts like a blotter, drawing the penetrant out of the defect. After some time indications appear in the developer which are wider than the defect and which, therefore, can be seen directly or under ultraviolet light due to the enhancement contrast results between the penetrant and the developer. The method is widely used in aerospace industries, however, its application in case of concrete is rarely reported[87].

The **Portable Crack Measuring Microscope** measures the crack width. Apart from the necessary optical excellence required in the lens system, the heart of the Microscope lies in the reticle arrangement. The reticle itself may involve almost any type of plane outline, including scales, grids and lines. The images of the reticle and the work are superimposed making direct comparison possible[100].

2.1.7. Steel stresses

A **Sensor**[6] is reported to have been developed for measuring stress in the prestressing tendons of concrete structures, ground anchors, cables and ordinary reinforcement. It is a transducer which works on the principle that the intensity of magnetization of ferromagnetic body changes under the action of applied mechanical stress. The stress measurements made on the basis of the standard calibration allow a value to be obtained for the stress in the steel. However, a recess must be provided as soon as the structure is designed in which the pick-up can be housed. Preferably, it must be positioned before the concrete is cast and before stressing work is commenced. A technique to scan change in polarity of magnetised H.T. wires is reportedly used in Germany to detect snapped wire in the case of pretensioned concrete roof.

2.1.8. Cover

The **Magnetic Detector** or **Cover Meter** is used to measure the thickness of cover to reinforcing bars. It is claimed to give reliable results and can be successfully used for bridge decks, slabs, columns and other reinforced concrete structures[5]. A battery is used to generate a magnetic field which is distorted whenever there is steel in the vicinity of the probe. Cover meter is claimed to measure cover to within 6 mm in the range of 0 to 75 mm[5]. The instrument gives satisfactory results in lightly reinforced members, but in heavily reinforced members, the effect of deeper steel cannot be eliminated. Parallel bars and strands also influence the instrument reading if their spacing is less than two to three times the depth of cover. A further complication arises when some of the constituents of concrete are magnetic in nature.

2.1.9. Condition

Endoscope is reported to have been used in conjunction with an innovative drilling procedure to examine the condition of prestressing steel. The drill is equipped with magnetic sensors that shut down the drill as soon as it is on the point of contact with steel. The instrument consists of rigid or flexible viewing tubes that can be inserted into holes. Light is provided by glass fibres from an external source. In the rigid tubes, viewing is provided through reflecting prisms and in flexible tubes fibre optic system is used. These scopes allow close examination of parts of the structure which could not be otherwise viewed. Some equipments have attachments for camera or television monitor. The angle and the field of view of endoscope being narrow, its visibility is limited.

2.1.10. Corrosion

Corrosion monitoring techniques are broadly classified in two groups based on (a) electrochemical, and (b) electrical principles as follows:

a) Electrochemical Techniques

- Open Circuit Potential Measurements
- Surface Potential Measurements
- Polarisation Resistance Technique
- Impedance Technique
- Electrochemical Noise Analysis

b) Electrical Techniques

- Resistivity of Concrete
- Electrical Resistance Probe
- Corrosion Monitoring of Prestressing Steel by Electrical Resistance

Open Circuit Potential (OCP) Method is used for qualitative assessment of corrosion in reinforced concrete structures. Whenever the reinforcement surrounded by concrete comes in contact with the corrosive environment, it develops a potential which varies from place to place depending upon the corrosive environment and this corrosion potential of rebar is measured with respect to a standard reference electrode. This technique, however, gives only a qualitative information. As the OCP values are influenced by moisture content in concrete, it should be prewetted at the points where OCP is to be measured. The values are temperature dependent and the measurements on coated rebars may not reflect the real condition of the rebar. Stray currents and delamination of concrete can affect the potential measurements.

Surface Potential Measurement is used for identifying anodic and cathodic regions in concrete structures and indirectly detecting the probability of corrosion of rebar in concrete. During corrosion process an electric current flows between the cathodic and anodic sites through the concrete and this flow can be detected by measurement of potential drop in the concrete. The surface potential measurements are only for comparative studies. The measurement by itself does not indicate corrosion behaviour of rebar embedded in concrete. It needs to be coupled with the resistivity measurements to obtain a parameter "Corrosion Cell Ratio" for assessing the probability of corrosion of reinforcement. It has been reported that whenever the potential difference obtained on concrete surface is not more than 30 mv, it indicates that steel remains in passive condition, whereas, if surface potential difference exceeds 100 mv, it indicates active corrosion condition[126].

Polarisation Resistance Technique is among the best known techniques for evaluation in the laboratory of instantaneous corrosion rate of rebar in concrete. There is a linear relationship between potential and applied current at potentials only slightly shifted from the corrosion potential. Based on the kinetics of electrochemical reactions and the concept of the mixed potential theory, an equation has been derived which relates quantitatively the slope of the polarisation curve in the vicinity of the corrosion potential to the corrosion current density. The method suffers from the limitation that in the field measurements the working electrode, viz, steel reinforcement grid is very large, compared to the auxilary electrode, thus, posing problems with regard to area of influence. Further, the electronic equipment is likely to get damaged due to the mechanical vibration generated during its transportation.

In recent years, **A.C. Impedance Spectroscopy** has been applied to rebars for quantifying corrosion of steel rebars embedded in concrete. In this technique an A.C. signal is applied to the embedded rebar and the response is monitored in terms of the phase shift of the current and voltage components and their amplitudes. Impedance is the ratio of A.C. voltage to A.C. current. An alternating voltage of about 10 to 20 mv is applied to the rebar and the resultant current and phase angle are measured for various frequencies. Accessibility of rebar network and interference effects may lead to practical problems. The most difficult part is in obtaining the Rt, i.e., the charge transfer resistance value at each location of the probe sensor since the degree of polarisation induced on the rebar gradually decreases with the distance from the position of the counter electrode. The usefulness of the technique is limited, if the rebar is essentially passive. If the rebar is in active condition, then only well developed impedance data is obtained.

Electrochemical Noise Technique is an emerging tool for monitoring corrosion of reinforced concrete structures. It enables information about the mechanism and rate of corrosion processes at areas identified in concrete structures. A low amplitude of variation of the corrosion potential (range of 1 mv to 10 mv, 10 μHz to 1Hz) of steel in concrete is measured to obtain a noise data as a record of potential fluctuations in the form of power spectra. Noise source is located within the probable corroding area. A time record of sufficient interval is monitored over the frequency range (10 μHz to 1Hz). Noise data as a record of potential fluctuation is obtained. Noise signal is transformed from time domain to frequency domain displaying in the form of amplitude and frequency based on either Fast Fourier Transform or maximum entropy method of spectral analysis. Since fluctuations are in microvolt range, a highly sensitive equipment is necessary.

Electrical Resistivity of concrete is an important parameter which can be related to various other aspects, such as, strength, porosity, deterioration, etc. it is well known that the reinforcing steel embedded in concrete is protected by the concrete cover and that this protection is mainly due to high alkalinity and the fairly high electrical resistance of concrete. During the corrosion process, the corrosion current has to flow from anode to cathode through electrolyte and the resistivity of concrete has an influence on the flow of this corrosion current. However, correlation between resistivity of concrete with deterioration of concrete and corrosion of steel in concrete is not fully established. Porous concrete can also give high resistivity. This necessitates careful interpretation of the obtained data.

Electrical Resistance Probe Technique has been used for monitoring the corrosion of mild steel element embedded in concrete. The method is based on the fact that when a conductor corrodes, the metal lost is replaced by an insoluable non-conducting film which adheres to the metal or is carried away by the corrosive medium. Metals and alloys have much lower specific electrical resistance than their corrosion products. Since the electrical resistance of a metal depends on its cross sectional area, a decrease in thickness of a specimen due to uniform corrosion may be evaluated. Since the probe is a representative element for steel reinforcement, the exposed element of the probe is kept as close as possible to the reinforcement and the same cover is maintained. Accuracy of measurement, however, depends on the form of corrosion. If it is a uniform corrosion, then fairly accurate value can be obtained. As the probe gives the corrosion data for the specified location, number of probes need to be installed at carefully selected locations. Due to the embedment of probe involved, the technique is not suitable for existing structures.

Corrosion damages in **prestressing cable** can be quantitatively assessed by making **direct Electrical Resistance Measurements.** For a known length, the electrical resistance of a cable is inversely proportional to its cross sectional area as the electrical resistivity being constant for the particular material. As the cross sectional area of cable reduces due to corrosion, an increase in resistance with time is indicative of the progress of corrosion. The measured resistance value can be compared with the initial value. Thus, by periodic measurements of resistance values, corrosion rate can be evaluated. The technique requires that the cable ends are made accessible for making electrical connections. In the existing bridge, this may be possible only in those prestressing cables which are anchored in the deck. If the corrosion is highly localised then the technique will not be able to indicate the exact condition of the prestressing cable. Inspite of these limitations and uncertainty factors, this technique can still provide some information about the condition of the prestressing steel. The measurements need to be taken periodically. The electrical circuit for the initial condition of the prestressing strands, cable sheaths, anchor plates, etc., should be precisely established. Influence of cement grout also needs to be considered.

Overall behaviour of the bridge, periodic variation, elasticity, load carrying capacity, etc. can be monitored through load testing of the bridge. IRC:SP:37-1991[124] provides guidelines in detail for evaluation of load carrying capacity of bridges. Static response measurement under predetermined loading by moving crane-mounted trucks loaded with concrete blocks, has been attempted when a large number of bridges are to be assessed. Structural integrity through in-situ dynamic test to monitor changes in the dynamic characteristics, such as, natural frequencies, damping, mode shapes, etc. has also been attempted in several countries, although no quantitative estimation has so far been achieved. The current state of monitoring structural dynamic characteristics to assess structural integrity of bridges has been given[120]

2.1.11. Global behaviour

Geodetic Instruments are used for both horizontal as well as vertical distance measurements. EDM instruments fall under this category. The instruments function by sending light wave or microwave along the path to be measured and measuring the time involved in traversing the required distance as with microwaves or in measuring the time involved in returning the reflecting light wave back to source[78], whereas, the measurement of distance may be reasonably accurate, its accuracy in measuring body deformations may depend upon the distance involved. Computer controlled tracking stations to monitor deformations, although, very expensive may find application in future.

Linear Variable Differential Transformer (LVDT) is the most commonly used inductive transducer. It provides an AC voltage output proportional to the displacement of the core passing through the windings[100]. **Dial Gauges** are used to measure the deformation. A stationary base is, however, necessary in case of LVDT and dial gauge.

Deflectometer is a device fabricated to monitor deflections of bridges under dynamic loads. It consists of a steel cantilever plate attached to a heavy metal base. The free end of the cantilever is connected to a tensioned thin steel wire which is attached to the top of the structure. An electrical resistance strain gauge which is attached to the cantilever, is calibrated for deflection of the point at which the wire is attached. The deflections of the structure are indirectly measured as strains of the cantilever. The output of the deflectometer could either be recorded on an oscillograph or a strip chart. The technique requires facility for installing the heavy metal base.

VW Acoustic Strain Gauges are based on the principle that the resonant frequency of a taut wire will vary with changes in tension. A length of steel wire is tensioned between two end blocks that are firmly in contact with the mass of concrete. Deformation in the concrete will cause the two blocks to move relative to each other, altering the tension in the steel wire. This change in tension is measured as a change in the resonant frequency of vibration of wire[11]. Although, suited for measurements on long term basis, sensor needs to be embedded in the body and hence suitable for new construction. Sensor mounted on concrete surface may not reflect the body strains as accurately as in the case of embedded one.

'Pfender' Mechanical Strain Gauge which is an improvement over the conventional strainmeter uses steel balls as marks of the gauge lengths for testing successive extensions of as many test objects or locations as required, by means of one handy instrument. The measuring length is adjustable between 20 and 500 mm as required. Maximum variation of length that can be measured is $\pm$ 0.5 mm. Accuracy is 1/1000 mm[96,97,98]. Use of spherical balls makes better and stable contact with the measuring instrument. Strainmeters mounted with LVDT are available for auto-recording of movements.

Tiltmeter measures tilt in structures, in which the component to be measured is rotation in a vertical plane. In this instrument, the sensor provides an electrical signal proportional to the sine of the angle of inclination of base plate from horizontal or vertical, which is indicated by a digital readout unit[94]. It has potential application in superstructures, subsidence adjacent to building excavations. The technique can be used as a cross-check to the deflection measuring system.

Inclinometer is a high precision instrument for measuring subsurface displacement or deformation[95]. Applications include measurement of lateral movement of bridge piers and well foundations. The instrument is normally lowered down a grooved plastic or aluminium inclinometer casing installed near vertical in the mass under observation. The grooves control the orientation (azimuth) of the instrument in a predetermined direction. Inclinometer readings are taken at frequent intervals of depth and are subsequently converted to displacements. Embedding of the duct makes the technique suitable for new constructions.

Pressure Transducer consists of a Piezometer, attached to the fluid filled Pressure Cell. Soil pressure on the flat walls of the cell is converted to fluid pressure and measured by the piezometer. Pressure transducers are fixed to the contact surface between walls and soil. The typical application includes measuring pressure at soil - concrete interface in concrete well foundations[103]

Thermocouples and Thermometers are used for the temperature measurements. Thermocouples make use of the principle that when two dissimilar metals are brought in contact with each other either by twisting them together and brazing or by welding, an electromotive force exists across the junction, which along with other factors also depends upon the junction temperature. A minimum of two conductors will be necessary to form a circuit. The net EMF is a function of two materials used to form the circuit and the temperature at the two junctions. The actual relations, however, are empirical and the temperature - EMF data is to be based on experiment. A **Vibrating Wire Temperature Sensor** can also be used for temperature measurement. It has a stainless steel transducer body to which a vibrating wire element is attached. Different coefficients of thermal expansion of the body and wire make it a simple, sensitive temperature measuring device. The output signal from the vibrating wire temperature gauge is in the form of a frequency[103].

Overall behaviour of the bridge, periodic variation elasticity, load carrying capacity, etc. can be monitored through load testing of the bridge. IRC:SP:37-1991[124] provides guidelines in detail for evaluation of the load carrying capacity of bridges. Static response measurement under predetermined loading by moving crane-mounted trucks loaded with concrete block, has been attempted when a large number of bridges are to be assessed. Guidelines for carrying out tests on materials and for the global behaviour have been explained at length in IRC:SP:40-1993 in context of rehabilitation of bridges[148]

Structural integrity through **in-situ dynamic test** to monitor changes in the dynamic characteristics, such as, natural frequencies, damping, mode shapes, etc. has also been attempted in several countries, although, no quantitative estimation has so far been achieved. The current status of monitoring structural dynamic characteristics to assess structural integrity of bridges has been reported by SERC[120].

Low strain non-destructive **integrity testing technique for concrete piles** based on one dimensional stress wave approach is simple, quick and requires minimal interference with site activity[155,157,158]. Major defects, like, cracking, necking, soil inclusions and change in cross-section produce their own unique signal in velocity reflectograms which can be monitored soon after the construction. The pile length can also be estimated to a fairly reasonable degree of accuracy, if it is not too long or skin friction is not too high. However, minor deficiencies, like, local loss of cover to pile reinforcement, small inclusions or type of debris present at pile base are not detectable. Analysis of velocity reflectograms offer both qualitative and quantitative information. For proper interpretation, however, knowledge of sub-soil investigation data, construction details, quality of concrete is essential. Qualified personnel having knowledge of pile-soil interaction and piling construction techniques are required both for conducting tests in the field and for interpretation of the test results.

3. NON-DESTRUCTIVE TESTING TECHNIQUES

Operating principle, specifications, limitations and calibration of the known techniques referred in the above summary are given in the following paragraphs.

3.1. Rebound Hammer Method

Principle

Schmidt's Rebound Hammer[7] consists of a spring controlled mass that slides on a plunger within a tubular housing. When the plunger is pressed against the surface of the concrete, the spring controlled mass rebounds and the extent of such rebound depends upon the surface hardness of concrete (Fig. 3.1.1). For the purpose of this method, the surface hardness and, therefore, the rebound is taken to be related to the compressive strength of the material. The rebound is read off along a graduated scale and is designated as the 'rebound number' or 'rebound index'.

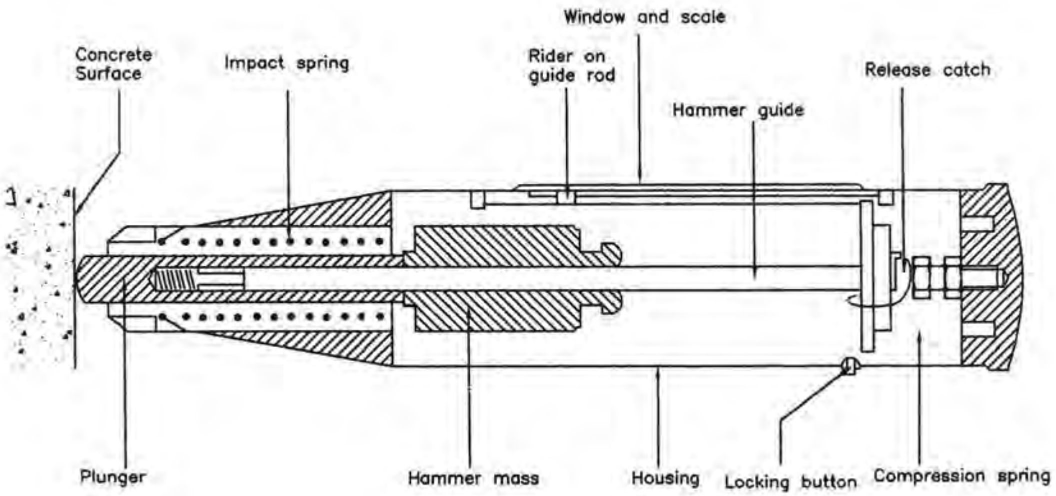


Fig. 3.1.1. Typical Rebound Hammer[11]

The rebound hammer method can be used for[10]:

(i) estimating the compressive strength of concrete with the help of suitable correlations between rebound index and compressive strength,

(ii) establishing the uniformity of concrete,

(iii) assessing the quality of one element of concrete in relation to another

Specifications

Depending upon the impact energy required for different applications, Schmidt's test hammers are classified into four types shown in Table 3.1.1[44].

The point of impact should be atleast 20 mm away from any edge or shape discontinuity. For taking a measurement, the rebound hammer should be held at right angles to the surface of the concrete member. If the situation demands, the rebound hammer can be held at intermediate angles also, but in each case, the rebound number will be different for the same concrete.

Table 3.1.1. Impact Energy for Rebound Hammers for Different Applications[44]

Type	Applications	Approximate Impact Energy Required for the Rebound Hammers (N-M)
N	For testing normal grades of concrete in ordinary buildings and bridge constructions	2.25
L	For light-weight concrete or small and impact sensitive parts of concrete or artificial stones	0.75
M	For testing mass concrete in roads, air-field pavements and hydraulic structures	30.0
P	For testing cement mortars and plasters, concrete of low strength (5 to 25 N/mm^2)	0.90

Limitations

The estimation of strength of concrete by rebound hammer method cannot be held to be very accurate and the probable accuracy of prediction of concrete strength is ±25 per cent. The results are affected by many factors including the angle of test, surface smoothness, mix proportions, type of coarse aggregate, the moisture content of the concrete and carbonation of the surface8.

It is also pointed out that rebound indices are indicative of compressive strength of concrete to a limited depth from the surface. If the concrete in a particular member has internal microcracking, flaws or heterogeneity across the cross-section, rebound hammer indices will not indicate the same. The rebound hammer method is suitable only for close texture concrete. Open texture concrete typical of masonry blocks, honeycombed concrete or no-fines concrete are unsuitable for this test. All correlations assume full compaction, as the strength of partially compacted concrete bears no unique relationship to the rebound numbers. Troweled and floated surfaces are harder than moulded surfaces and tend to overestimate the strength of concrete. A wet surface will give rise to underestimation of the strength of concrete calibrated under dry conditions. In structural concrete, this can be about 20 per cent lower than in equivalent dry concrete[11]. Carbonated concrete gives an overestimation of strength which in extreme cases can be put 50 per cent[11]. It is possible to establish correction factors by removing the carbonated layer and testing the concrete with the rebound hammer on the uncarbonated concrete.

Calibration

It is necessary that the rebound hammer is frequently calibrated and checked against the testing anvil to ensure reliable results for each position in which it is to be used on the structure[8,10]. The testing anvil should be of steel having Brinell hardness of about 5000 N/mm^2.

The most satisfactory way of establishing[44] a correlation between compressive strength of concrete and its rebound number is to measure both the properties simultaneously on concrete cubes. The concrete cube specimen are held in a compression testing machines under a fixed load, measurements of rebound number taken and then the compressive strength determined as per IS:516-1959.

The fixed load required is of the order of 7 N/mm^2 when the impact energy of the hammer is about 2.2 Nm. The load should be increased for calibrating rebound hammers of greater impact energy and decreased for calibrating rebound hammers of lesser impact energy. The test specimen should be as large as possible in order to minimize the size effect on the test result of a full scale structure. The 150 mm cube specimens are preferred for calibrating rebound hammers of lower impact energy (2.2 Nm), whereas, for rebound hammers of higher impact energy for example 30 Nm, the test cubes should not be smaller than 300 mm. Atleast, nine readings should be taken on each of the two assessable vertical faces in the compression testing machine when using the rebound hammers. The same points must not be impacted more than once.

Rough surfaces resulting from incomplete compaction, loss of grout, spalled or tooled surfaces do not give reliable results and should be avoided.

Around each point of observation, six readings of rebound indices are taken and average of these readings after deleting outliners (extreme values whose probabilities of occurrence by chance alone are very small) becomes the rebound index for the point of observation.

3.2. Windsor Probe Test

Principle

The test estimates the strength of concrete[4] from the depth of penetration by a metal rod driven into the concrete by a given amount of energy generated by a standard charge of powder.

The underlying principle is that for standard test conditions, the penetration is inversely proportionate to the compressive strength of concrete, but the relation depends on the hardness of the aggregate. The frictional resistance to the probe and the energy absorbed by cracking of concrete are said to be negligible.

Penetration depth is a direct measure of the compressive strength of concrete. Charts of strength vs penetration for length of exposed probe are available for aggregates with hardness of between 3 and 7 on Moh's scale (Figs. 3.2.1., 3.2.2., 3.2.3.). However, in practice, the penetration resistance should be correlated with the compressive strength of a standard test specimens or cores of the actual concrete used. The various applications[11] of the method are - Compressive Strength Estimation, Quality Control and Strength Development Monitoring.

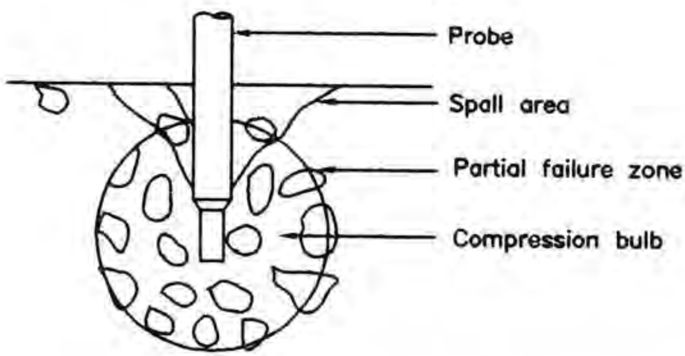


Fig. 3.2.1. Compression Bulb Produced by Penetration Probe[1]

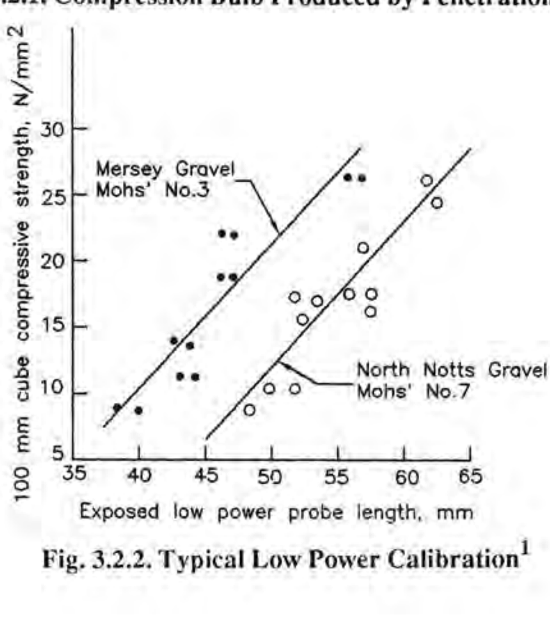


Fig. 3.2.2. Typical Low Power Calibration[1]

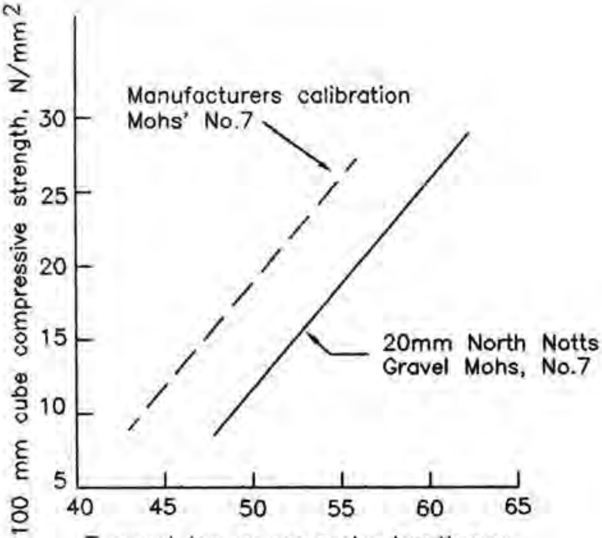


Fig. 3.2.3. Comparison of Calibration[1]

Specifications[1]

Diameter of probe	7.94 mm
Length of probe	79.5mm
Velocity of cartridge	183 m/s
Variation in diameter	±1 per cent

The Windsor Probe conforms to BS:1881 (Part 207), ASTM C 803

Limitation

The Windsor Probe Test basically measures hardness and cannot yield absolute values of strength, but the advantage of the test is that hardness is measured over a certain depth of concrete and not just at the surface. The other limitations[1] are as follows:

(i) The test results are affected by the edge distance and the hardness of aggregates. Type of aggregate also affects the penetration depth.

(ii) The reliability of the test results on concrete is measured in terms of average co-efficient of variation which is 4 per cent.

(iii) Shape of aggregates as well as the moisture condition also affect the calibrations, although, these two effects are small.

Calibration

The indentations[1] are measured by a spring loaded calibrated depth gauge to the accuracy of 0.5 mm. Alternatively, the probe in sets of three can be measured by using a triangular template system with probes at 177 mm centers and using triangular measuring plates to get the average of three readings.

A low power drive	-	for a strength upto 26 N/mm^2
A standard power drive	-	for higher strength
Recommended edge distance	=	100 mm

Centre-to-centre distance of 175 mm for individual probe is recommended by Bungey[11]. For low power drive, edge distance of 75 mm is not adequate to prevent splitting. The probed specimen, when used for compressive strength calibration, give values which vary as much as 17.5 per cent and hence give false calibration. It is recommended that a minimum number of six specimen be used - three for probe test and other three for the determination of compressive strength. Also, the least thickness of the members to be tested has to be twice the expected penetration depth. It is advisable to use Ultrasonic Pulse Velocity Technique to verify whether the specimen cast for calibration and the in-situ concrete have similar mix characteristics and age at test.

3.3. Core Test

Principle

Cores are cut from structural members in order to establish the quality of in-situ concrete, particularly in terms of uniaxial compressive strength[3]. The most common occurrence of Core Testing is when the results of standard specimen (cubes or cylinders) cast from the concrete used for forming a structural member, fail to comply with the specified strength and as a result of this the quality of concrete is disputed. The cores[4] can also be used to detect segregation or honeycombing or to check the bond at construction joints.

Specifications

Maximum aggregate size	= 10 mm to 20 mm
Core Diameter	= 50 mm to 150 mm
As per the ASTM C42-82[12] recommendations,	
Core Diameter	= 3 {Nominal maximum size of aggregate}
As per the BS:1881[13] recommendations,	
Core Diameter	= 100 mm to 150 mm

The 150 mm diameter is prefered, although, BS:6089[14] makes provisions in exceptional cases for core diameter less than 100 mm.

Limitation

The main drawback of this test is the difference in the intrinsic quality of concrete in structure and controlled specimen in the laboratory.

Calibration

(i) Effect of slenderness ratio

As the slenderness ratio[3] decreases below 2, strength increases at an increasing rate, for the ratio between 2 to 3, strength remains sensibly constant and beyond 3, it shows reduction.

In general, the slenderness ratio in core testing is limited to between 1 and 2 and the measured strength is expressed as the equivalent strength value for a specified (L/d). This is achieved by using correction factors, such as, those specified by ASTM[12] and BSI[13,15], which are presented in Tables 3.3.1., 3.3.2., 3.3.3., 3.3.4. and 3.3.5. together with those reported by Sangha and Dhir[16].

(ii) Effect of core diameter

There is considerable conflicting evidence of the effect of the core diameter itself on the measured strength for a given slenderness. Much of the conflict is thought to have arisen due to variations in the location of the different sizes of test cores in concrete section as reported by Meininger[17] and Lewis[18].

Table 3.3.1. Effect of Core Diameter on Measured Concrete Strength, Core l/d = 2[3]

Specimen Number	Strength, MPa		
	Core	Diameter	mm
	50	75	100
1	33.5	35.0	33.5
2	37.0	32.5	35.5
3	35.0	36.5	37.0
4	33.5	36.0	34.0
5	32.0	35.0	35.0
6	34.0	34.0	33.0
Mean	34.0	35.0	34.5
S.D.	1.7	1.4	1.5

Table 3.3.2. Influence of Core Diameter on the Strength of Concrete made with Aggregate of Maximum Size 10 mm[3]

Statistics	Strength, MPa			
	Core Diameter, mm			
	25	50	75	100
a) Slenderness Ratio - 1				
Mean Value	61.5	59.0	59.0	58.0
S.D.	3.8	1.2	0.5	0.8
b) Slenderness Ratio - 2.5				
Mean Value	44.0	47.0	48.5	46.5
S.D.	4.5	0.9	0.5	0.8

Table 3.3.3. Strength Correction Factors for Different Length/Diameter Ratios[3]

Length/diameter ratio	Correction Factors			
	ASTM	BSI		Sangha Dhir
		Current[11]	Previous[13]	
2.00	1.00	1.00	1.00	1.00
1.75	0.98	0.97	0.98	--
1.50	0.96	0.82	0.96	0.95
1.25	0.93	0.87	0.94	--
1.00	0.87	0.80	0.92	0.83

It is suggested by Munday and Dhir[3] that provided the core diameter is confined to certain sensible range the effect of this factor on the measured strength could be ignored for practical purposes.

(iii) Effect of coring direction

Because of the inherent anisotropy of concrete[19] the strength of cores obtained by drilling in the direction of casting (vertical cores) will be different to that of cores obtained at right angles to it (horizontal cores). The results of a study[20] undertaken to examine this effect showed that although, individual strength ratio vary from 0.88 to 0.96 similar to those obtained by Petersons[21,22], these variations are of random nature and cannot be attributed to either the position

Table 3.3.4. Effect of Coring Direction on Measured Concrete Strength[3]

Position of cores in test Columns		Strength ratio of horizontal to vertical									
		Nominal Concrete Strength, 25 MPa					Nominal Concrete Strength, 65 MPa				
mm from top	Column Height (mm)	200	400	600	800	Mean	200	400	600	800	Mean
50		89	90	92	90	90	90	92	96	91	92
100		88	90	91	91	90	89	92	94	90	91
150		89	91	93	91	91	91	93	96	93	93
250		-	91	95	91	92	-	94	96	93	94
350		-	94	95	94	94	-	94	95	91	93
450		-	-	95	90	92	-	-	94	90	93
550		-	-	94	90	91	-	-	94	93	93
650		-	-	-	91	91	-	-	-	89	89
750		-	-	-	92	92	-	-	-	89	89
	Mean	89	91	94	91	91	90	93	95	91	92

Table 3.3.5 . Effect of Lift Depth on Strength Reduction in Concrete between the Bottom and Top of a Lift[3]

Lift depth (mm)	Strength Reduction (per cent)
200	8
400	12
600	16
800	19
1600	21
>1600	23

of the core or the strength of concrete and that a mean value of 0.92 be adopted. Thus, the strength of core taken in horizontal direction may to a good approximation be taken as 8 per cent less than that value of vertical cores. This figure has been recommended by the Concrete Society Report[3] and adopted in the recently revised version of BS:1881[13].

(iv) Effect of core location

It is very easy to visualize that the strength of concrete will vary in the vertical direction due to water gain effect and this will produce concrete weaker nearer the top. However, it is extremely difficult to establish the nature of this variation along the entire height of structural member, as in practice additional factors, such as, mix composition, type of aggregate (angular, rounded), workability, method and degree of compaction and environment all become involved. It has been suggested that the actual strength is usually the lowest in the top 20 per cent of the lift[23]. In fact others[20,24,25] have shown quite clearly that the strength of concrete varies continuously with depth.

(v) Effect of soaking of cored specimen

It should be noted that the standard procedures[12,13] require the cores taken from in-situ concrete to be soaked for about 2 days prior to strength testing. It is recognized that due to the 2 days pre-test soaking, air cured concrete will register a lower strength than if it was tested in its in-situ moisture state and accordingly allowances should be made in interpreting the results. A value of about 16 per cent strength reduction due to soaking air-cured specimen has been reported[20,27]

(vi) Cube/Core strength relationship

The cube/cylinder strength relationships, as suggested in the various investigations are given below:

1. Sangha and Dhir[20] proposed the strength relationship as:

$$fc = 1.44\ fcy - 0.0066\ fcy ** 2$$

where, fc = Cube strength

fcy = Strength of cylindrical core specimen

2. BS:1881[13,14] recommends the following relationship:

$$fc = 1.25\ fcy$$

3. Munday et.al[3], based on their investigations, proposed the following relationship:

$$fc = A\ fcy - B\ fcy ** 2$$

where, A and B are the constants, whose values depend upon shape of the test specimen and the (L/D) ratio. A graph[3] showing relationship between cube and cylinder strength is given in Fig. 3.3.1.

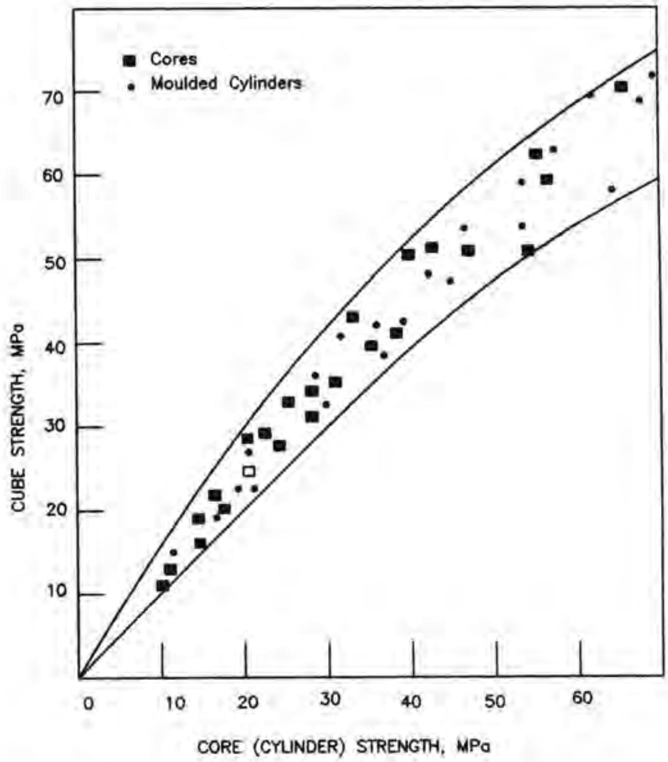


Fig. 3.3.1. Cube/Core (Cylinder) Strength Relationship[3]

4. Based on his limited tests and from the observations of other researchers, Gupta[121] indicated that the ratio of universal mean value of equivalent core strength to cube strength of 0.6 or above may be regarded as an index of satisfactory quality of construction. However, more tests are necessary to establish such a generalised relationship. NCCBM, Ballabhgarh have also recently taken up field studies to develop correlation between core and cube strengths.

5. IS:456-1978 states that concrete in the member represented by a core test shall be considered acceptable, if the average equivalent cube strength of the cores is equal to atleast 85 per cent of the cube strength of the grade of concrete specified for the corresponding age and no individual core has a strength less than 75 per cent.

6. IS:516-1959 states that the measured compressive strength of the test specimen should be corrected using a suitable correction factor given in the code (Fig. 3.3.2.). The corrected compressive strength represents the equivalent strength of a cylinder having (height/diameter) ratio of two. The equivalent cube strength of the concrete shall be determined by multiplying the corrected cylinder strength by 5/4.

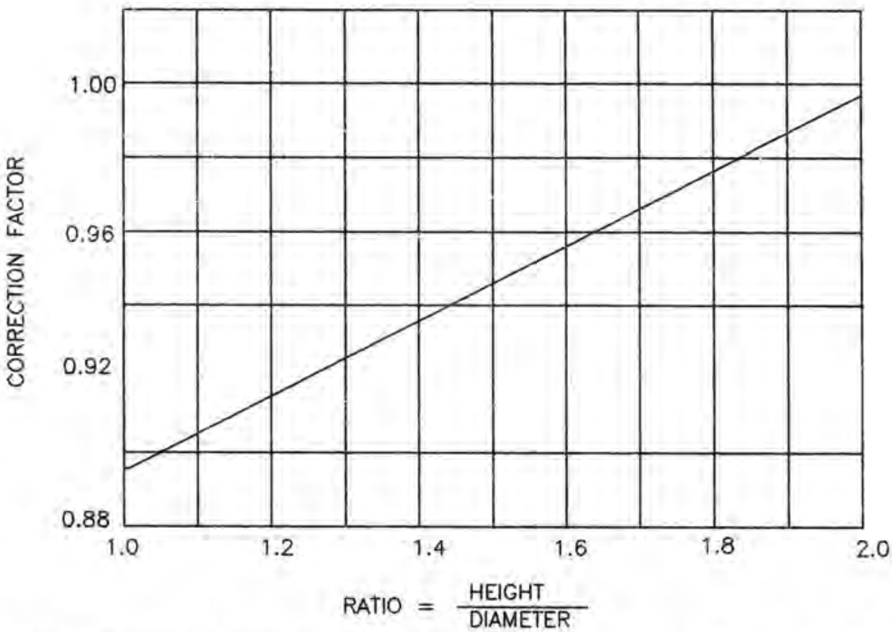


Fig. 3.3.2. Correction Factor for Height/Diameter Ratio of Core (as per IS:516-1959)

3.4. LOK and CAPO Tests

(a) LOK Test

Principle

This test was developed at the Danish Technical University in the late 1960s and is now accepted as equivalent to cylinder for acceptance testing by public agencies in Denmark[1]. The test consists of measuring the force required to pull out a previously cast-in steel rod with an embedded enlarged end[4] from concrete (Fig. 3.4.1.). This force is taken as a measure of concrete quality. For a given concrete and a given test apparatus, the pullout strength is related to other strength test result[114]. Such strength relationships depend on the configuration of embedded insert, bearing ring dimensions, depth of embedment, and level of strength development in that concrete. Prior to use, this relationship must be established for each system and each new combination of concreting materials. Such relationships tend to be less variable where both pullout and other test specimen are of consistent size and cured under similar conditions.

Other applications[15] include determination of stress time in post-tensioned construction, in-situ strength monitoring, etc. The Construction Industry and Research Information Association (CIRA) of Great Britain recommends the LOK Test as suitable method of assessing formwork striking time.

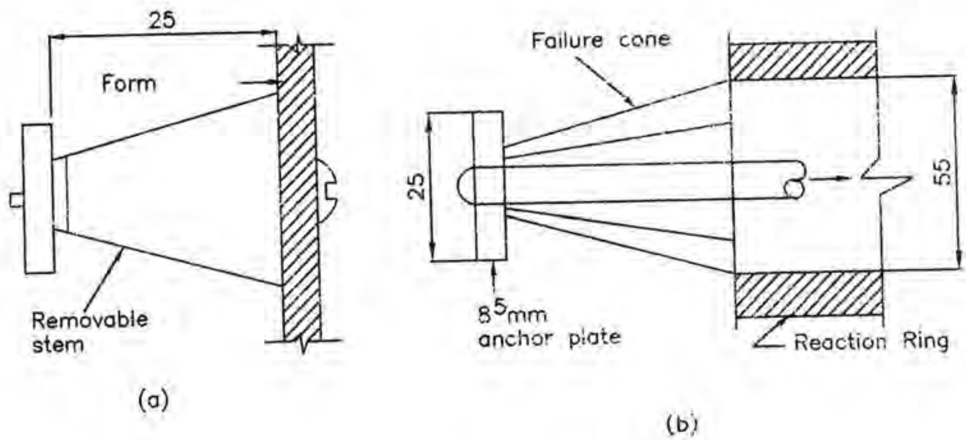


Fig. 3.4.1. LOK Test Assembly[1]

Specifications

Diameter of anchor plate	= 25 mm
Thickness of anchor plate	= 8.5 mm
Inner diameter of reaction ring	= 55 mm
Length of removable stem	= 25 mm
Rate of loading	= 30 ±10 kN/min

Calibrations

Calibration charts are usually provided by the Manufacturers (Fig. 3.4.2.). However, Bunge[11] and Peterson[45] have also given the typical calibration charts. The reliability[11] of the method is reported to be good with correlation coefficients for laboratory calibrations of about 0.96 on straight line relationship and a corresponding coefficient of variation of about 7 per cent. Comparison with rebound hammer and ultrasonic pulse velocity strength calibrations shows that the slope is much steeper, hence, the test is more sensitive to strength variation [55]. Strength calibration is more dependable than for most other non-destructive or partially destructive methods and generalized correlations may be acceptable. However, for large projects it is recommended that specific calibration is developed for the concrete actually to be used. It should also be noted that artificial light weight aggregates are likely to require specific calibration. When planning pullout tests and analyzing test results, consideration should given to the normally expected decrease of concrete strength with increasing height within a given concrete placement in a structural element[114].

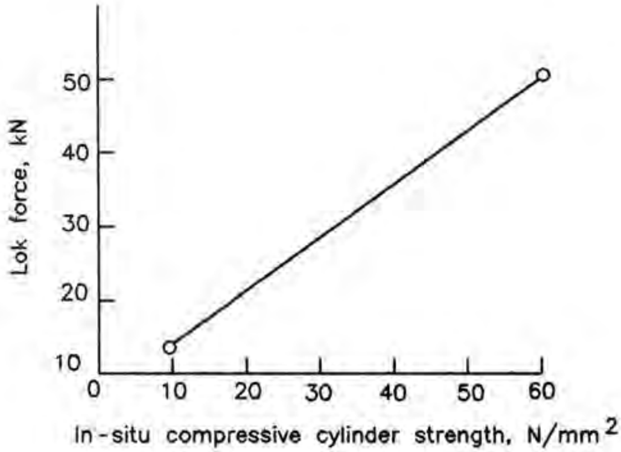


Fig. 3.4.2. Calibration Chart for LOK Test[1]

(b) CAPO Test

It is based on the same principle as LOK Test but allows hardened concrete to be tested at random without having a pullout disc embedded in the fresh concrete before hand. The test was developed in Denmark in 1970's. The procedure involves drilling a hole in hardened concrete, cutting a groove at a specified depth of the hole by milling operation and then expanding the 'CAPO' insert in a similar way as in the LOK Test. Fig. 3.4.3. shows internal state of stress in CAPO Test[153].

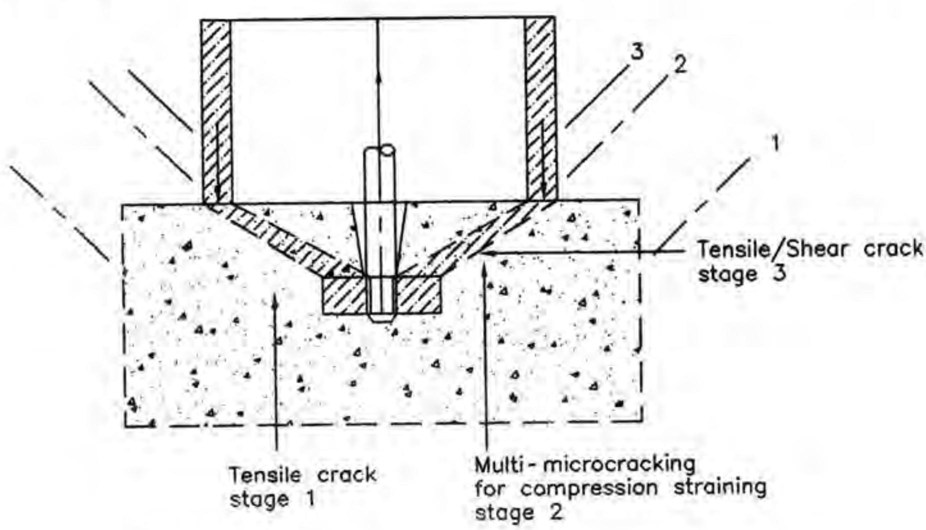


Fig. 3.4.3. Internal Cracking Stages during a Pull-out[153]

3.5. North American Pull-out Method

Principle

The principle[1] is similar to LOK Test (Fig. 3.5.1.). The test assembly bears some modifications. The method is described by ASTM C 900-2[4].

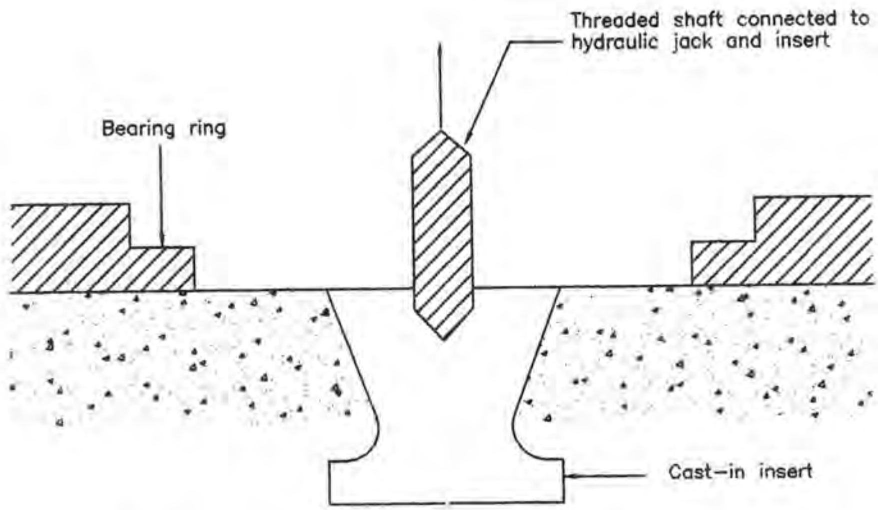


Fig. 3.5.1. North American Pull-out Method Assembly[1]

Specifications

The depth of penetration is greater in this method. The appex angle of failure surface is desirably set at 64°.

Calibration

The pullout strength has to be circulated as follows[1]

$$fp = \frac{F}{A}$$

where, F = force on the arm

A = failure surface area

$A = (d3 + d2)*(n/4)*[4h^2 + (d3\text{-}d2)^2]^{\frac{1}{2}}$

d2 = diameter of pullout insert head

d3 = inside diameter of reaction ring
h = distance from insert head to the surface

The result of the test is claimed to have a correlation with core strength with a correlation co-efficient of 0.99.

The relationship suggested is :

Pullout strength = 0.21*(core strength)

3.6. Break-off Test

Principle

Break-off test has been developed as a method for the in-situ determination of the flexural strength of concrete by Norwegian Technical University and A/S NORCEM[70,71]. A tubular disposable form is inserted into the fresh concrete or alternatively a shaped hole can be drilled to form a slot of the type shown in Fig. 3.6.1.[72]. Using the hydraulic testing device, a transverse force is applied at the top surface to fracture the resulting core left after removal of the insert. The force required to rupture the core obtained from the instruments manometer is taken as a measure of the flexural strength of the concrete and can be correlated with the compressive strength through the use of calibration charts. The small concrete core obtained during the test may also be taken to the laboratory for further examination.

Specifications

Diameter of concrete core	= 55 mm
Depth of concrete	= 70 mm

Calibrations

The break-off strength[72] calculated from the results has been shown to give a linear correlation with the modulus of rupture measured on prism specimen, although values were 30 per cent higher on average. A correlation with compressive strength has been developed which covers a wide range of concrete, but this is likely to be less reliable than a tensile strength correlation in view of the factors influencing the tensile/compressive strength relationship. Compressive strength estimation to within ±20 per cent should be possible with the aid of appropriate calibrations.

It is claimed that the method is quick and uncomplicated, taking less than two minutes per test. Results are not significantly affected by the surface condition or local shrinkage and temperature effects. Field experience in a variety of situations has been reported by Carlsson et. al.[72].

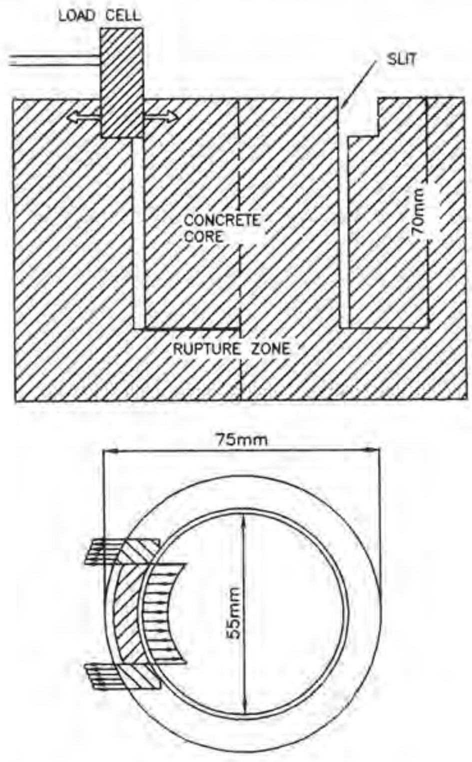


Fig. 3.6.1. Principle of Operation of the Break-off Test[72]

Limitations

The in-situ testing method measures the flexural strength of concrete on a core with the rupture zone located 70 mm from the surface. The method is regarded as especially suitable for very young concrete and although leaving a sizable damage zone, may gain acceptance as an in-situ quality control test where tension strength is important. Althoûgh, quicker than compression testing of cores, the use of results for strength estimation of old concrete may be unreliable unless a specific calibration relationship is available.

References of their application

Dahl - Jorgensen et. al.[73] described a modified version of the test in which a steel cylinder is glued to the surface and jacked against a counter-support. This is intended for bond testing of concrete overlays or epoxy surface coatings, and lower test variability is claimed than with pull-off methods. Charistiansen et. al.[73] have examined relationships between break-off values and bending tensile strengths and have shown water/cement ratio, age, curing and cement type to be significant. Johnsen et. al.[76] obtained break-off values for an airfield pavement contract and have suggested coefficients of variation of 6.4 per cent for laboratory samples and 12.6 per cent for in-situ samples. Eeg. Inge.R. et. al.[76] used break-off tester in concrete work for the new "Bank of Norway" building in Oslo. The break-off tester was used to control the time for safe formwork removal and the other curing control of concrete. Depending on the temperature and other curing conditions compressive strength of this concrete varied from 14 to 23 N/mm^2 after 2-3 days. After 28 days of curing the compressive strength was approximately 55 N/mm^2.

3.7. Pull-off Test

Principle

The pull-off test, as a means of predicting the compressive strength of concrete, involves bonding a circular steel probe to the surface of the concrete under test by means of an epoxy resin adhesive (Fig. 3.7.1.)[67]. Prior to this operation the surface of the concrete is abrased using sand paper to remove laitance and then degreased using a suitable solvent. After sufficient time has elapsed for the epoxy resin adhesive to cure a slowly increasing tensile force is applied to the probe and as the tensile strength of the bond is greater than that of concrete, the latter eventually fails in tension. The amount of overbreak is usually small so that area of failure can be taken as being equal to that of the probe. From this area and the force applied at failure, it is possible to calculate a nominal tensile strength for the concrete specimen. This parameter is of little use on its own to structural engineers but with the aid of comprehensive calibration graphs, based on a large number of pull-off tests and corresponding cube/cylinder compressive tests, it is possible to make a reliable estimate (with acceptable confidence limits) of the equipment cube/cylinder strength.

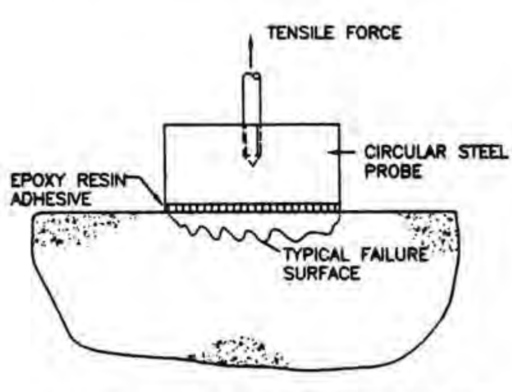
Several factors which affect the tensile/compressive strength ratio are age of concrete, aggregate type and size, air entrainment, compressive stress normal to the axis of pull-off load application and curing conditions.

Specifications

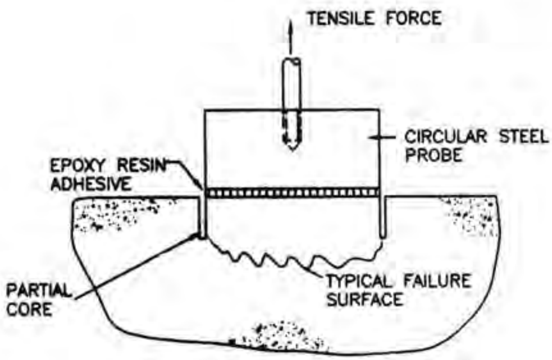
Probe diameter (on uncored specimen)	= 50 mm (approx.)
Min. capacity of hydraulic apparatus	= 10 kN
Co-efficient of variation between individual pull-off results	= 8 per cent to 20 per cent
Depth of partial core	= 10 mm

Calibration

An inspection of the probe, after the test, reveals whether or not the failure has been in the concrete. Results from unsatisfactory failures can, therefore, be discounted.



(a) Arrangement for testing uncored specimens



(b) Arrangement for testing partially cored specimens

Fig. 3.7.1. Pull-off Test [67]

References of their applications

Long et. al.[67] used the pull-off test in the construction of Multi-storey Car Park which consists of in-situ beams, columns and slabs with pad foundations. Concrete with maximum aggregate sizes of 10 mm and 20 mm were used and pull-off tests were undertaken on both types at 7 and 28 days. Wherever, possible, three pull-off tests were planned for each member, although, the average number of successful results throughout the programme was only two per member. Tests were carried out using an hydraulic apparatus and 50 mm diameter steel probes. Pull-off tensile strengths and details of the failures (where they were not fully successful) were also recorded. They also carried out pull-off test along with a considerably greater number of Schmidt Hammer tests to make an assessment of a multi-span flyover made up of precast pretensioned concrete beams.

3.8. Ultrasonic Pulse Velocity (UPV) Test

Principle

The Ultrasonic Pluse Velocity method can be used to establish[9].

(i) The homogeneity of the concrete

(ii) The presence of cracks, voids and other imperfections

(iii) Changes in the structure of the concrete which may occur with time

(iv) The quality of the concrete in relation to standard requirements

(v) The quality of one element of concrete in relation to another

(vi) The values of elastic modulus of the concrete

The ultrasonic pulse is generated by an electro-acoustical transducer. When the pulse is induced into the concrete from a transducer, it undergoes multiple reflections at the boundaries of the different material phases within the concrete. A complex system of stress waves is developed which includes longitudinal (compressional), shear (transverse) and surface (rayleigh) waves. The receiving transducer detects the one set of the longitudinal waves, which is the fastest.

The velocity of an ultrasonic pulse through any material depends upon its density, modulus of elasticity, presence of reinforcing steel and Poisson's ratio. The underlying principle of assessing the quality of concrete is that comparatively higher velocities are obtained when the quality of concrete in terms of density, homogeneity and uniformity is good. In case of poorer quality, lower velocities are obtained. If there is a crack, void or flaw inside the concrete which comes in the way of transmission of the pulses, the pulse strength is attenuated and it passes around the discontinuity, thereby, making the path length longer. Consequently, lower velocities are obtained. The actual pulse velocity obtained depends primarily upon the materials and mix proportions of concrete. Aggregates through their density and modulus of elasticity also make significant contribution to the pulse velocity.

After traversing a known path length (L) in the concrete, the vibration pulse is converted into an electrical signal by a second electro-acoustical transducer held in contact with the other surface of the concrete member and an electronic timing circuit enables the transit time (T) of the pulse to be measured, from which the UPV (V) can be calculated[7] as,

$$V = L/T$$

Specifications

The apparatus for ultrasonic pulse velocity measurement consists of the following[9]:

(i) Electrical pulse generator

(ii) Transducer - one pair

(iii) Amplifier

(iv) Electronic timing device

The natural frequency of transducers should preferably be within the range of 20 to 150 kHz (Table 3.8.1.). Generally, high frequency transducers are preferably chosen for short path lengths and are more sensitive to detecting voids and low frequency transducers where the path length is longer.

Table 3.8.1. Natural Frequency of Transducers for Different Path Lengths[9]

Path length (mm)	Natural frequency of transducer (kHz)	Min. transverse dimensions of members (mm)
upto 500	150	25
500-700	≥ 60	70
700-1500	≥ 40	150
above 1500	≥ 20	300

The apparatus should be capable of measuring transit times to an accuracy of ± 1 per cent over a range of 20 microseconds to 1 milliseconds.

There are three ways of measuring pulse velocity through concrete (Fig. 3.8.1.[8]):

a) The direct method (cross probing) is preferred wherever access to opposite sides of the component is possible.

b) The semi-direct method is used whenever access to different but not opposite sides of the component is possible.

c) The surface method is the least satisfactory and should only be used when access to only one surface is possible. This method only indicates the quality of the concrete near the surface and is influenced by the presence of reinforcement parallel to the surface.

Limitations

Variations of the concrete temperatures between 5°C to 30°C do not significantly affect the pulse velocity measurements in concrete[9]. At temperatures between 30°C to 60°C, there can be reduction in pulse velocity upto 5 per cent. Below the freezing temperature, the free water freezes within concrete resulting in an increase in pulse velocity upto 7.5 per cent.

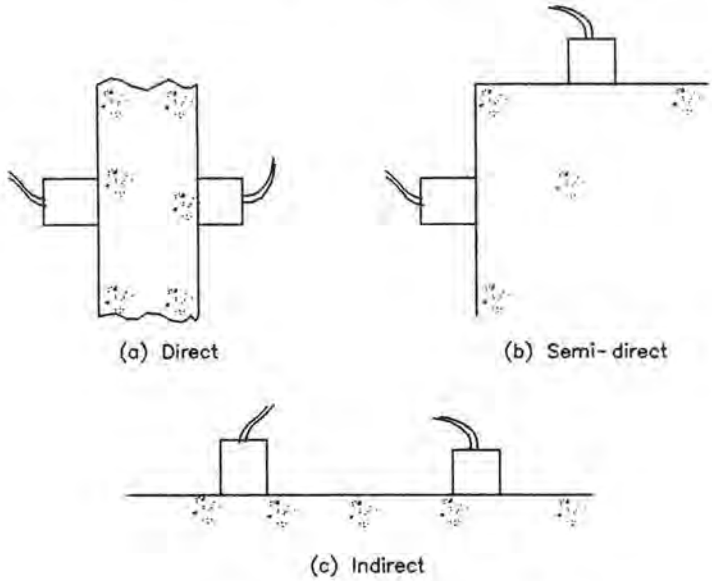


Fig. 3.8.1. Types of Reading[8]

In general, the pulse velocity through concrete increases with increased moisture content of concrete. This influence is more for low strength concrete than high strength concrete. The pulse velocity of saturated concrete may be upto 2 per cent higher than that of similar dry concrete. In general, drying of concrete may result in somewhat lower velocity.

The shape and size of the concrete member does not influence the pulse velocity unless the least lateral dimension is less than a certain minimum value, e.g., the minimum lateral dimension of about 80 mm for 50 KHz natural frequency of the transducer.

When concrete is subjected to a stress which is abnormally high for the quality of the concrete, the pulse velocity may be reduced due to the development of micro-cracks. The influence is generally insignificant unless the stress is greater than about 60 per cent of the ultimate strength of the concrete.

The pulse velocity measured in reinforced concrete in the vicinity of reinforcing bars is usually higher than in plain concrete of the same composition. This is because the pulse velocity in steel is 1.2 to 1.9 times the velocity is plain concrete[9].

Although, the ultrasonic techniques[8] have the advantages of being both rapid and truly non-destructive, there are serious practical limitations to their use. The heavy reinforcement of highway bridges can make interpretation of the results very difficult. It is also difficult to align transducers properly and measure the path length accurately in the direct transmission mode.

Nevertheless, unltrasonic methods can be used with advantage to identify deterioration in concrete bridges and further development is underway[28].

Calibration

A number of qualitative scales have been published to relate pulse velocity measurements to the quality of concrete[8]. The quality of concrete in terms of uniformity, incidence or absence of intial flaws, cracks and segregation, etc. indicative of the level of workmanship employed, can thus be assessed using the guidelines given in Table 3.8.2., which have been evolved for characterising the quality of concrete in structure in terms of the ultrasonic pulse velocity.

Table 3.8.2. Pulse Velocity Ratings for Concrete Quality Grading

	Pulse Velocity		
Quality	After Malhotra[31] km/sec	After Leslie and Chessman[29] km/sec	As per Bureau of Indian Standards[9] km/sec
Excellent	> 4.6	-	> 4.5
Good	3.7 to 4.6	> 5.0	3.5 to 4.5
Fair/Medium	3.0 to 3.7	4.0 to 5.0	3.0 to 3.5
Poor	2.1 to 3.0	3.0 to 4.0	< 3.0 **
Very poor	< 2.1	-	-

** In case of ultrasonic pulse velocity less than 3.0 km/sec, concrete is designated as 'Doubtful' and it may be necessary to carry out further tests.

Generally, it is desirable to choose path lengths that avoid the influence of the reinforcing steel. Where it is not possible to do so, the measured values have to be corrected. Methods of calculating correction factors are usually contained in equipment manuals and simple procedures[30,31,32]. A summary of the conditions that influence the transmission of sound waves in concrete is given in Fig. 3.8.2.[8].

3.9. Acoustic Emission (AE) Method

Principle

In certain materials, changes of condition can occur which cause the emission of sound waves[87] due to thermal, mechanical or other effects. The changes of condition can be fractures, crack formation, crack growth or metallurgical changes, such as, plastic deformation, dislocation

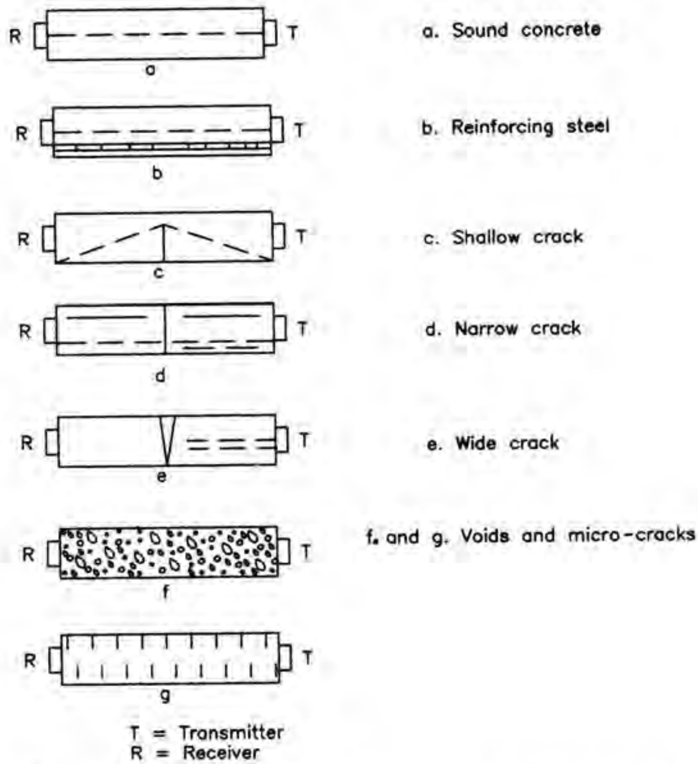


Fig. 3.8.2. Factors Affecting the Transmission of Sound Wave through Concrete[8]

diffusion and changes in crystal structure. During such changes in the condition, energy is released which propagates in the surrounding material as elastic vibrations. These vibrations or sound waves can be detected by placing a sensor on the surface of the material. AE sensors (or transducers) are in principle high frequency microphones which are glued or fixed in some other manner to the clean surface so that the best possible acoustic contact is achieved. AE technology is thus used for production control of the material and condition monitoring of constructions consisting of materials for which ability to emit sound is present. Because not all materials emit sound wave from defects under load and since the propagation of sound is damped depending on the material, it is necessary to evaluate the possibility of using the AE method in each individual case.

The AE method can only detect changes in materials. Thus defects already present, i.e., a crack, cannot be revealed unless the size of the crack increase under load. AE is, therefore, used as a supplement to other Non-Destructive Testing Techniques, where critical regions with defects detected by means of other NDT methods can be monitored using AE. And vice-versa, AE can indicate regions where other NDT examinations are required.

Specifications

The signal detected by the piezo-electric transducer is amplified, filtered, processed and recorded in some convenient form (Fig. 3.9.1.[11]). Specialist equipment for this purpose is available in the U.K. as an integrated system in modular form[63] and lightweight portable models may be used in the field. The results are most conveniently considered as a plot of emission count rate against applied load (Fig. 3.9.2.[11]).

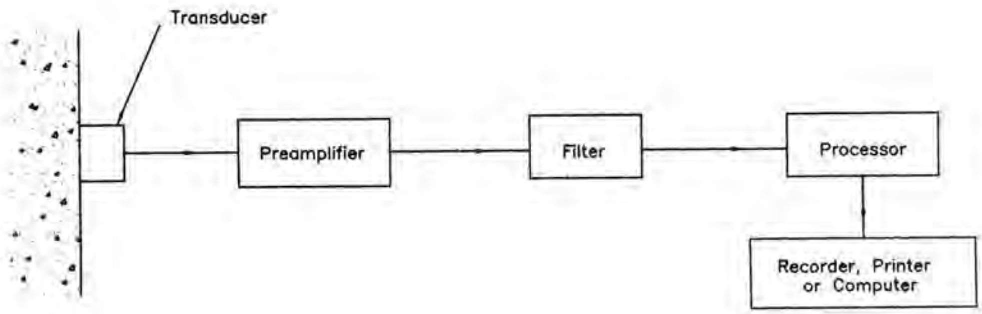


Fig. 3.9.1. Acoustic Emission Equipment[11]

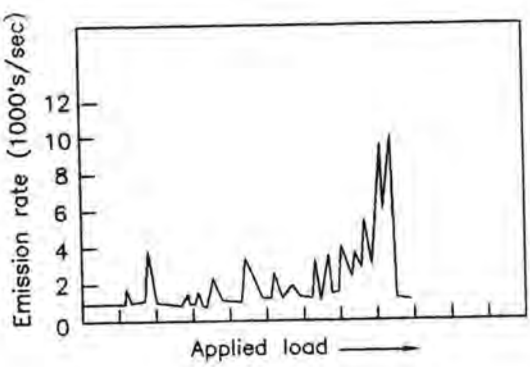


Fig. 3.9.2. Typical Acoustic Emission Plot[11]

Limitations

It has been reported[51] that as the load level on a concrete specimen increases, the emission rate and signal level both increase slowly and consistently until failure approaches and there is then a rapid increase upto failure whilst this allows crack initiation and propagation to be monitored during a period of increasing stress. Mindess[52] has also shown that mature concrete provides more acoustic emission on cracking than young concrete, but confirms that emissions do not show a significant increase until about 80 per cent to 90 per cent of the ultimate stress. The absence of the Kaiser effect for concrete effectively rules out the method for establishing a history of past stress levels.

3.10. Pulse-echo Method

Principle

Untrasonic Examination by means of the Pulse-echo method[87] can be described in its simplest form as a type of a material radar, which transmits a very short, high frequency pulse of mechanical oscillations into a material (Figs. 3.10.1., 3.10.2. and 3.10.3.). The pulse has a character of sound oscillations, but it cannot be heard because of its high frequency and is, therefore, termed as Ultrasonic. The pulses propagate in the material in a narrow beam until they strike an interface, such as, opposite surface of the test object or an internal defect. The pulses are entirely or partly reflected back to the transmitter, which now functions as a receiver. The time interval between the transmission and the reception of pulse depends upon the distance traversed, for the propagation, speed is constant for a particular type of material. If one sends a pulse normal to the surface of a plate and measures the transmission time, one can compute the thickness of the plate. If one wants to localize internal defects in an object, one must know the starting point of the sound pulse and direction in which it has travelled as well as the distance traversed. Larger defects are mapped by moving the probe or transducer which transmits and receives the sound pulses across the surface of the object. This search pattern is called a scanning and often follows a certain pattern. The emission is reported so many times per second that the cathode ray tube of the apparatus displays an apparently stationary image. On the other hand, there must be sufficient time between pulses and the reflected pulses, which can sometimes be reflected back and forth many times in objects with smooth surfaces and low attenution, will have had time to dissipate before the next pulse is emitted. Ultrasonic examination is used mostly today in the steel industry but also to a lesser extent for examining concrete.

Limitations

(i) The contact time of the impact source should be shorter than the arrival time of the first ray. However, if the contact time is made too short, the recorded signal becomes complex and difficult to interpret.

(ii) The reliability of the techniques used in this work appears to decrease with increasing concrete thichness. Further work is required on the inter-relationship between concrete thickness and impact source.

(iii) Pulse-echo testing using a mechanically generated stress pulse is applicable only if the test object dimensions exceed certain minimum requirements. The thickness must be large enough so that specular reflections arrive at the receiver after the surface wave. The lateral dimensions must be large enough so that reflected surface waves do not interfere with the initial portions of signal from specular reflections.

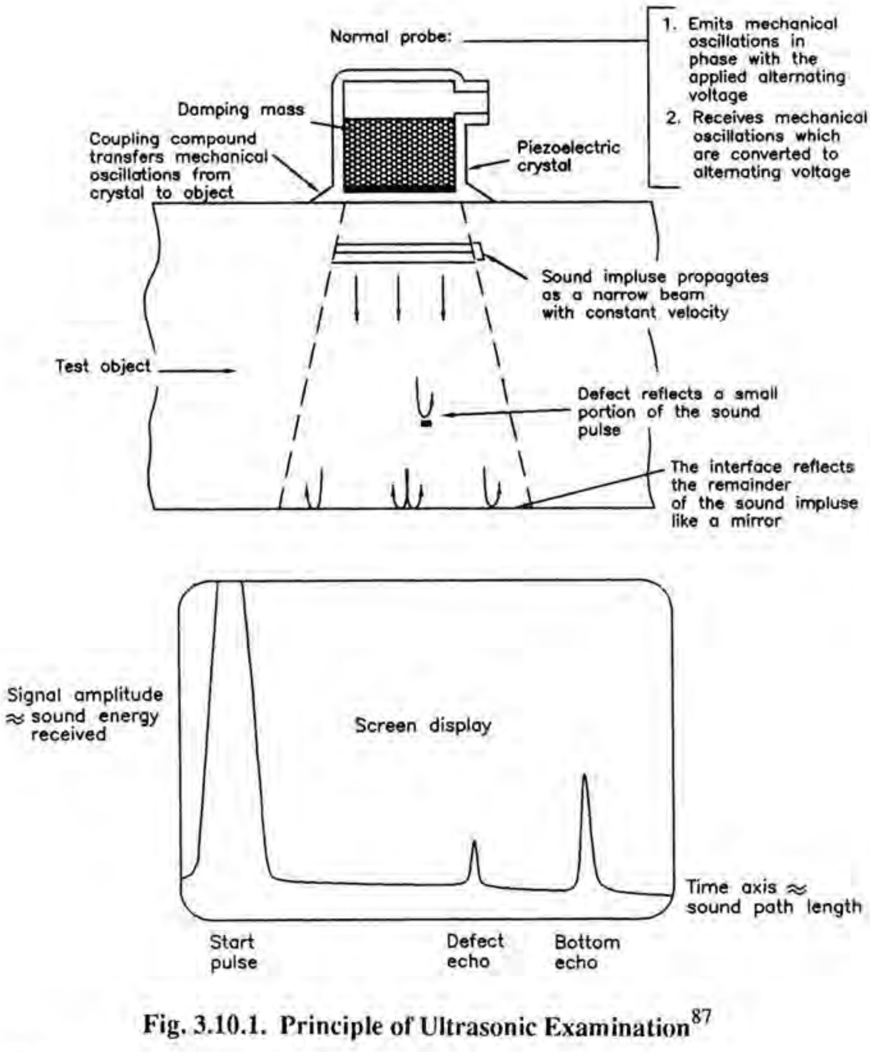


Fig. 3.10.1. Principle of Ultrasonic Examination[87]

References of their application

Nicholass J. Carl et. al.[61] carried out experiments using artificial flaws in large concrete slab. They discussed type of impact source, distance from impact plane to receiver, type of receiving transducer, depth of reflecting interfaces, and diffraction effects by sharp edges. The contact time of the impact is shown to be an important parameter for the success of the technique.

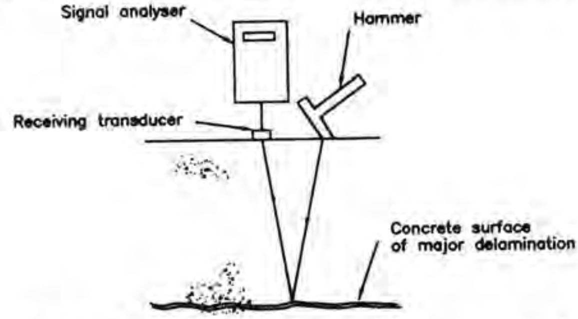


Fig. 3.10.2. Instrumented Delamination Device[11]

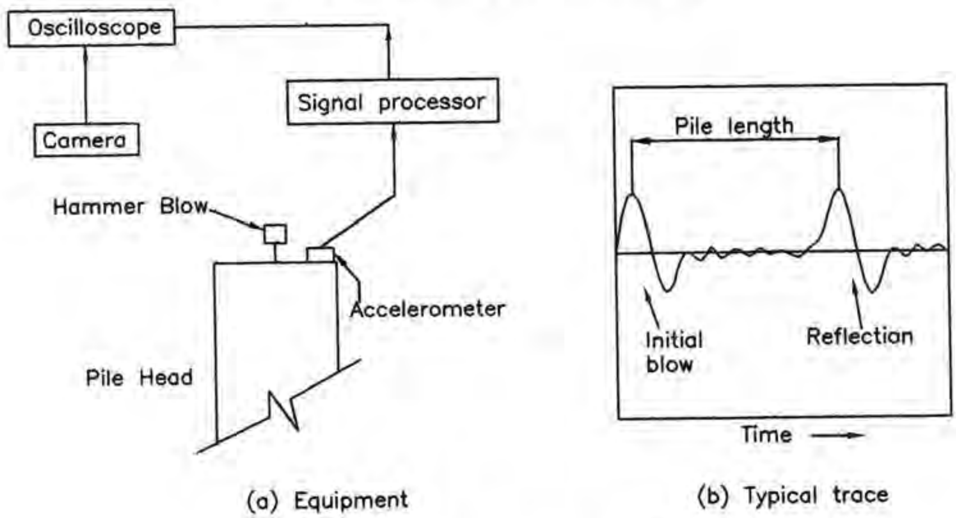


Fig. 3.10.3. Pulse Echo Method[11]

3.11. Initial Surface Absorption Test (ISAT)

Principle

This method is detailed in BS:1881: [Part 5(iii)] and Levitt[82] has discussed the theory and application of the technique (Fig. 3.11.1.). ISA test is defined as the rate of flow of water into concrete per unit area at a stated interval from the start of the test at a constant applied head and temperature. Results are expressed as ml/m^2/s at a stated time from the start of the test.

When water comes into contact with dry concrete it is absorbed by capillary action at a rate which is high initially, but decreases as the water filled length of capillaries increases. Levitt[82] has shown that this may be described mathematically by the expression:

$$P = \frac{a}{t^n}$$

where,

P = initial surface absorption

t = time from start

a = a constant

n = a parameter between .3 and .7 depending on the degree of silting or flushing mechanisms, but constant for a given specimen.

The ISAT can be used as a quality control test for precast units.

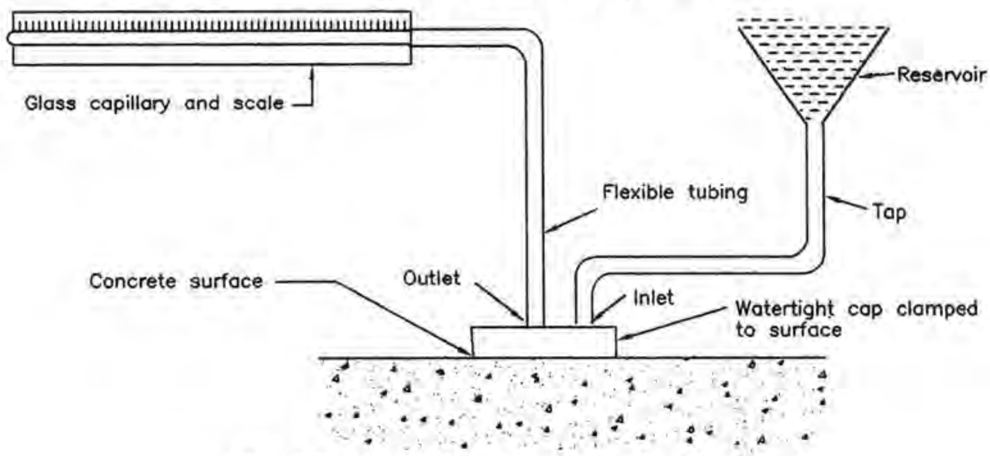


Fig. 3.11.1. Initial Surface Absorption Test[11]

Specifications

According to BS:1881, Pressure Head = 200 mm

The readings are to be taken at 10 minutes, 30 minutes, 1 hour and 2 hours intervals.

Water contact area	= 5000 mm^2
Height of reservoir and horizontal capillary from the surface	= 200 mm ±5 mm (above)
Length of capillary tube	= 100 mm to 1000 mm
Bore of capillary tube	= 0.4-1.0 mm radius
Minimum edge distance proposed from the internal circumference of the cap	= 30 mm

Limitations

The test on oven dried specimen gives reasonably consistent results, but that in other cases, the results are less reliable. Particular difficulties have been encountered with in-situ use in achieving a watertight fixing. Levitt[82] has suggested that specific limits could be laid down as an acceptability criterion for various types of construction, but insufficient evidence is available. The test has been found to be very sensitive to changes in quality and to correlate with observed weathering behaviour. The method can be applied to exposed aggregates or profiled surfaces provided that an effective seal can be obtained, but is not suitable for porous or honeycombed concretes.

Calibration

(1) Effect of Moisture

BS:1881 recommends that the test concrete is either oven dried at 105°C until constant weight is reached, or is kept in a normal laboratory environment for at least 48h. For site use, the measurement should not be made within 48h of rain. Figs. 3.11.2. and 3.11.3. show that ISAT readings increase at a rate influenced by the loss of moisture from the concrete and its final moisture condition[80]. Air drying period before the test can influence the results. A better alternative for site concrete could be to take cores from the in-situ concrete and test them in a laboratory after drying them at 105°C.

(2) Repeatability of Test Results

BS:1881 recommends the use of a knife edge cap with the edges sealed with modelling clay for rough concrete surfaces and a cap with a rubber gasket for smooth surfaces. However, the knife edge cap would give more variable results, both within a batch and between batches than the gasketed cap. Fig. 3.11.4 shows the repeatability of Test Results. The use of "model aircraft engine elastic" (similar to an elastic band) to seal the gasketted cap has been found to be better than an O-ring on rough surfaces.

(3) Factors affecting ISA

(i) Strength and curing

For the same materials, the strength and durability of concrete at a particular age are largely governed by its water/cement ratio and the type and duration of curing used. The duration of curing affected the compressive strength and surface absorption of concrete as shown in Fig. 3.11.5. There is a close relationship between strength and ISA for any given curing regime, Fig. 3.11.6, as would be expected, since both properties are affected by the degree of hydration of the cement paste and its gel/space ratio.

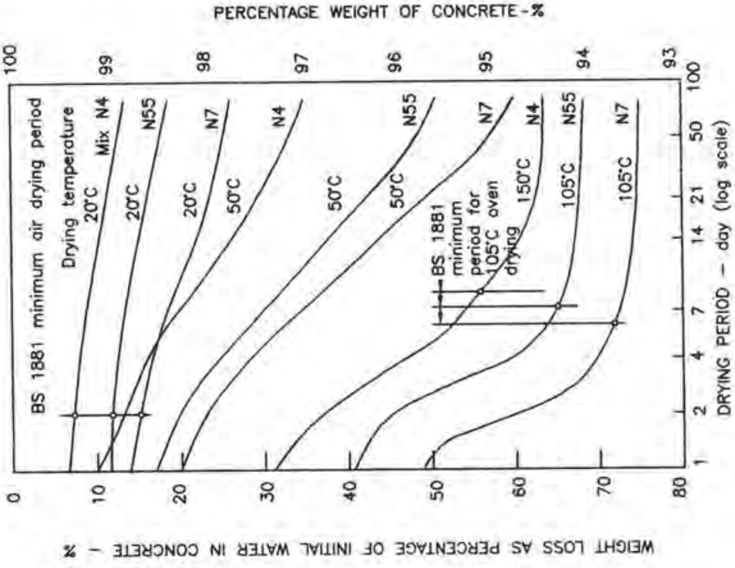


Fig. 3.11.3. Moisture Loss under various Drying Conditions[80]

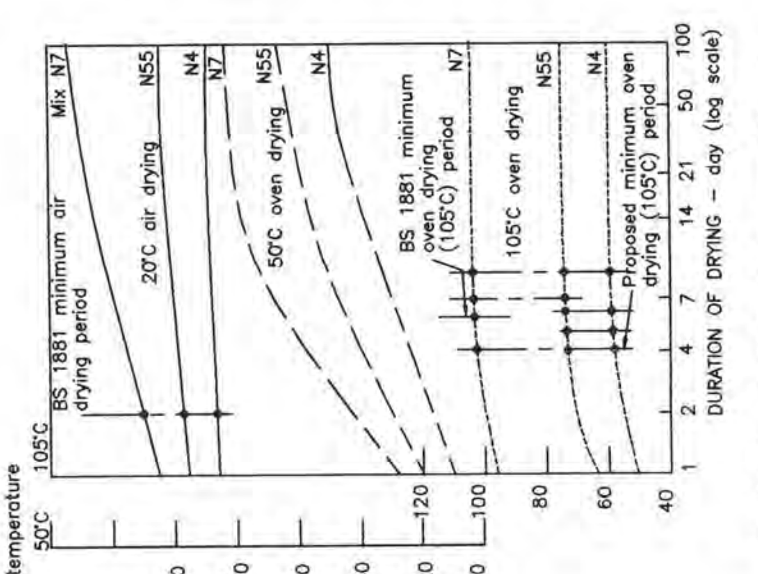


Fig. 3.11.2. Relationship between ISA and Duration of Drying[80]

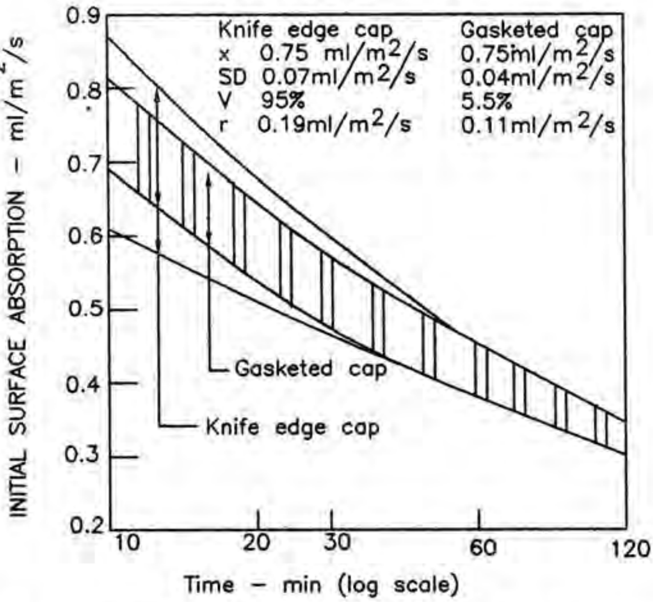


Fig. 3.11.4. Repeatability of ISA Test, 10 min. Values. Test Concrete: Water/Cement Ratio 0.55 Cured 3 Days in Water then in Air, Oven Dried at 105°C[80]

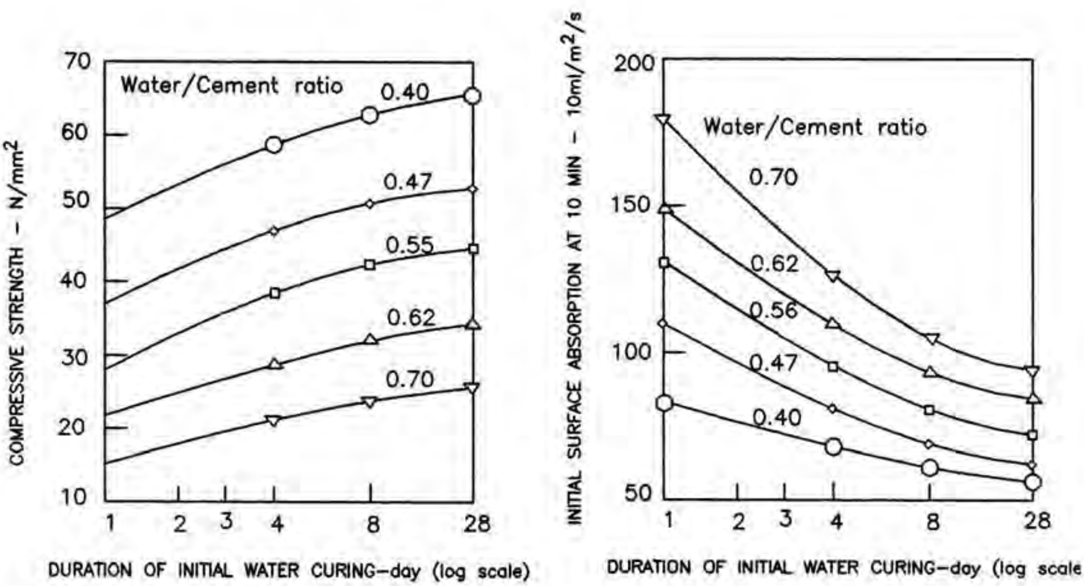


Fig. 3.11.5. Influence of Initial Moist Curing on Strength and ISA, Test Age 28 Days[80]

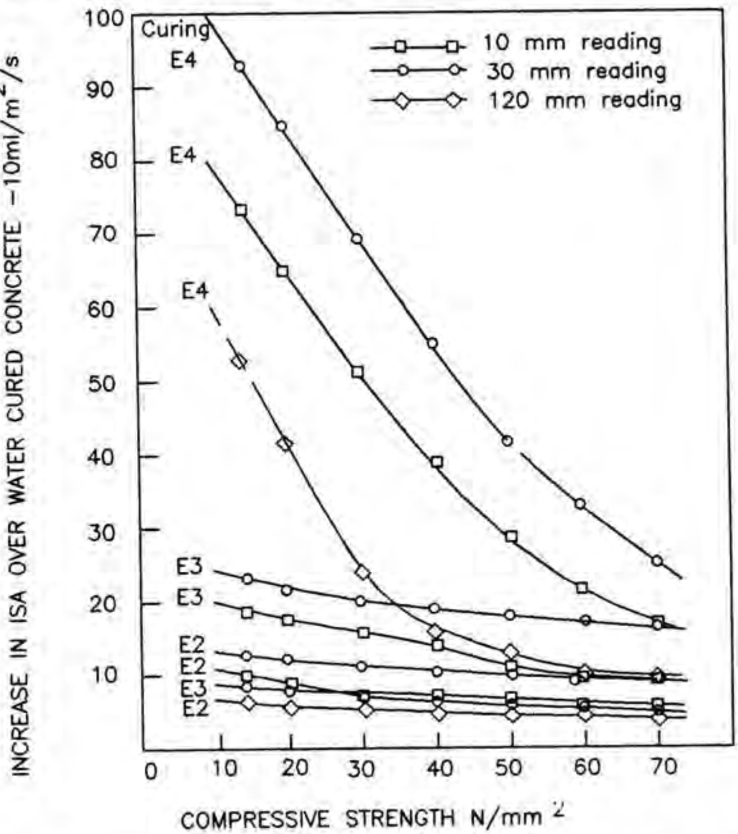


Fig 3.11.6. Influence of Concrete Grade on the Increase in ISA under various Curing Conditions with respect to Water Cured Concrete[80]

(ii) Workability

A series of tests conducted on concrete mixes with a constant water/cement ratio of 0.55, but with different workabilities obtained by varying either the water content and accordingly the cement concrete or by using a plasticizing admixture of the dispersing type, showed the effects of concrete slump on its ISA (Fig.3.11.7.). It is seen that ISA increases with workability at a greater rate when the increase in workability is associated with an increase in the water content (and hence cement content to maintain constant w/c ratio) than when the increase in workability is obtained by the use of plasticizing admixtures.

(iii) Specimen size

Initial work showed that the use of 100 mm cubes rather than 150 mm cubes lowered the ISAT values and caused larger variations in the results (Table 3.11.1.). It is proposed that a minimum distance of 30 mm is required between the edge of the specimen and the internal circumference of the cap to ensure that edge effects are minimal and to minimize the possibility of water leakage through channels connecting the test area and the side surface.

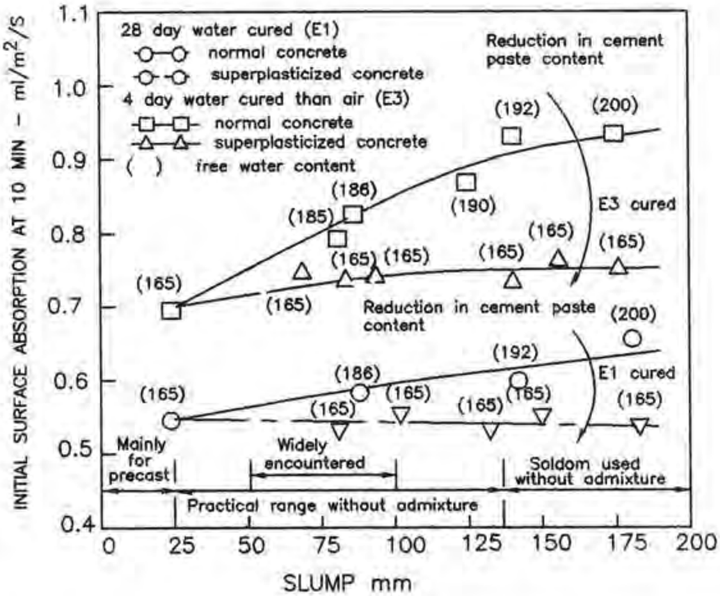


Fig. 3.11.7. Effects of Concrete Slump on its Initial Surface Absorption[80]

Table 3.11.1. Effect of Specimen size on ISA for a 45 N/mm² Mix with E3* Curing [80]

Specimen Size	Mean 10 min ISA ($x10\text{-}2ml/m^2/s$)	Coefficient of variation (per cent)
100 mm cube	64	7.9
150 mm cube	77	5.4
200 mm cube	76	5.1

* Curing environment after demoulding at 24h: 3 days water, then in air at 20°C and 55 per cent RH

3.12. Figg's Air and Water Permeability Test

Principle

The test was developed at the Building Research Establishment, U.K. in the early 1970s[81] to enable measurements of either an air permeation index or water absorption of concrete to be made. The most widely accepted procedure is that proposed by Cather, Figg, Marsden and

O'Brien[84] on the basis of extensive experience with the method. This is commonly known as the modified Figg's method.

In this test, a hole is drilled into the concrete surface and after thorough cleaning, it is plugged from outside surface by polyeather foam and then sealed with a catalyzed silicon rubber. After the hardening of rubber, a hypodermic needle is pushed through the silicon plug, Fig. 3.12.1.

For air permeation, a mercury filled manometer and a hand vacuum pump is attached to the needle. The pressure within the system is firstly reduced to a standard value and then after isolating the pump, the time for the pressure to rise a standard value is recorded. This time is taken as a measure of the air permeation index of concrete.

For water absorption, a water head of 100 mm is used. Water is forced into the assembly using a syringe and one minute after first contact between water and concrete, the syringe is shut off. The time for the meniscus in a capillary tube to travel 150 mm is recorded and is taken as a measure of the water absorption index of concrete.

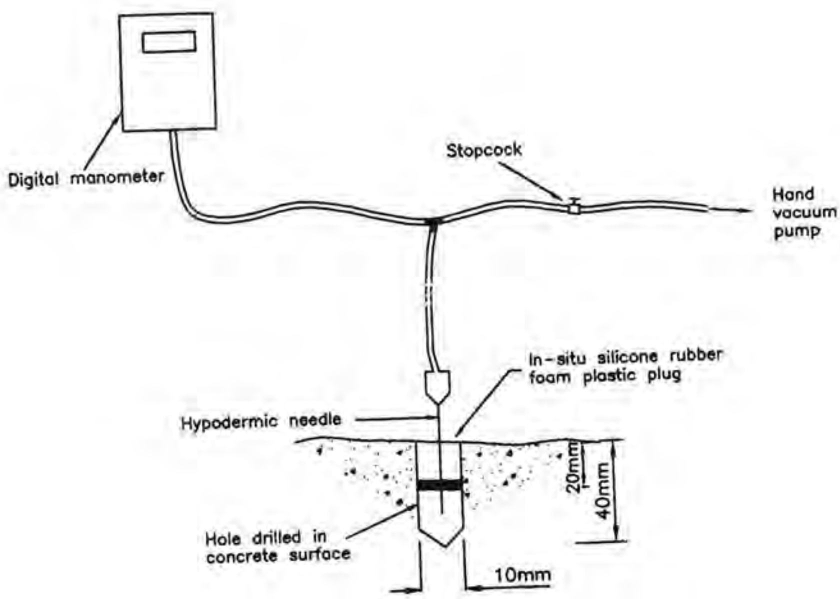


Fig. 3.12.1. Modified Figg's Ai .'ermeability Test[11]

In case of the air permeability measurements, the higher the index, the less permeable the concrete. The advantage of using air rather than water is that readings may be repeated on the same specimen without delay, since the passage of air through the specimen does not change the specimen in any way. The principle use[11] of this method, which involves cheap and simple techniques, is an alternative to the Initial Surface Absorption Method for quality control checking in relation to durability.

Specifications

Diameter of the hole	= 5.5 mm
Depth of hole	= 30 mm
16 gauge hypodermic needle is used.	

In the original work for air permeation,

Initial Pressure Drop	= 0.015 N/mm^2
Final Pressure Drop	= 0.02 N/mm^2
For water permeation, Pressure Head	= 100 mm of water

Limitations

Using this method, the relationship between air pressure and time, meniscus movement and time, were both found to be nearly linear. The air and water permeability measured by this method correlate well with water/cement ratio, strength and ultrasonic pulse velocity. Aggregate characteristics have a profound effect on results, limiting the potential usage to comparative testing, but variations of drilling and plugging of the test hole are less significant. As with the Initial Surface Absorption Method, the moisture condition of the concrete will considerably influence the results. This seriously restricts the in-situ usages, but a general classification for dry concrete is given in Table 3.12.1. It has been suggested that use of a larger hole will improve repeatability[11].

Table 3.12.1. General Relationship Between Permeability and Absorption Test Results on Dry Concrete[11]

Method	Concrete permeability/absorption		
	Low	Average	High
'Intrinsic permeability' $k(m^2)$	$< 10^{-19}$	10^{-19} - 10^{-17}	$> 10^{-17}$
ISAT-10min ($ml/m^2/s$)	< 0.25	0.25 - 0.50	>0.50
Figg water (s)	> 200	100-200	<100
Modified Figg air(s)	> 300	100-300	<100
BS Water absorption 30 min (per cent)	< 3	3-5	> 5
DIN:1048-4 day (mm)	<30	30 - 60	> 60

(1) Effect of Moisture Condition of Concrete

The moisture condition of test specimen has been found to markedly affect the results obtained. Fig. 3.12.2 shows air permeation test results.

(2) Repeatability of Test Results

The variation in the air permeation results of tests repeated in the same test hole (after allowing for the pressure to return to atmospheric each time) were found to be negligible. This suggests that the design of the equipment is intrinsically sound. However, tests on different specimen taken from the same batch of concrete and then from different batches cast using the same mix proportion, consistently produced very high variations with the co-efficient of variation within a batch and between different batches being 27 per cent and 32 per cent respectively. The main reasons for this are the variation in the area of aggregate surface exposed at the periphery of the small hole drilled for the test, the hetrogenity of the concrete and the aggregate properties adjacent to the hole, and to a lesser extent, the disturbance caused to the aggregate matrix bond by the drilling action. Table 3.12.2. shows the repeatability of the test results.

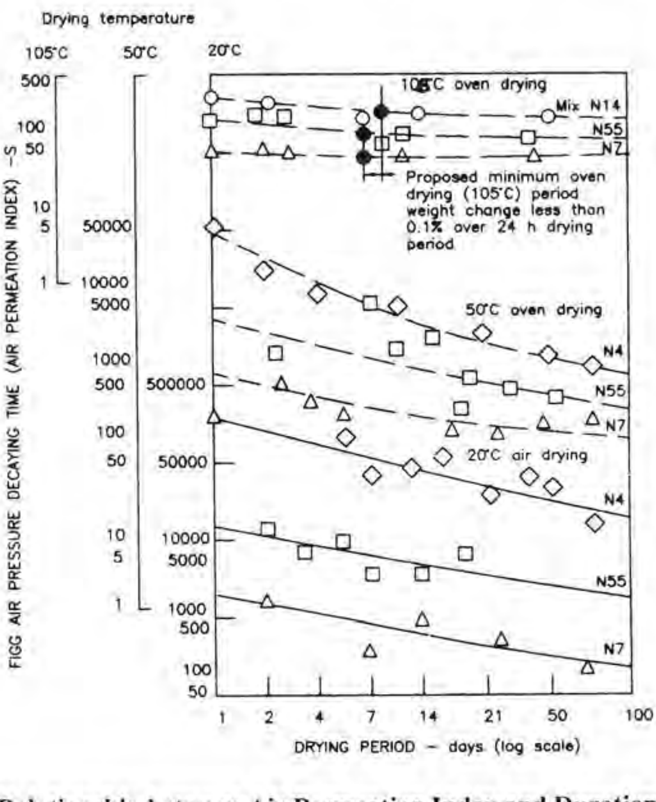


Fig. 3.12.2. Relationship between Air Permeation Index and Duration of Drying[80]

Table 3.12.2. Repeatability of Figg Air Permeability and Figg Water Absorption Test Results, for E3 Cured Concrete with w/c Ratio of 0.55[80]

Test criteria		Coefficient of variation, v (per cent)			
		Air permeability		Water absorption	
Test hole dimensions (diameter x height) (mm)	Drying method	Within batch	Between batch	Within batch	Between batch
10x40	50°C weight change < 0.1 per cent over 24h	38*	49*	36	37
10x40	50°C weight change < 0.05 per cent over 24h	30*	37*	33	40
10x40	105°C weight change < 0.1 per cent over 24h	27*	32*	36	39
13x40	105°C weight change < 0.1 per cent over 24h	17*	17*	32	38
13x50	105°C weight change <0.1 per cent over 24h	111+	111^{+}	35	39

Note: * Pressure range 45kPa→ 50 kPa
+ Pressure range 45kPa → 55 kPa

(3) Factors Affecting Air Permeation Index of Concrete

Tests were carried out on concrete with a range of water/cement ratios (Table 3.12.3), workabilities (Slump, 25 mm to collapse) and constituent materials curing (Table 3.12.4). The results showed similar general trends, but to a different degree to those observed for Initial Surface Absorption Test (Figs. 3.11.5 and 3.12.3). The effect of workability was much pronounced in the case of air permeation. This is mainly due to the different mechanisms governing fluid permeation under pressure (or vacuum) in the case of Figg's Test and water flow mainly due to capillary action in the case of ISAT. For the same porosity, a reduction in pore size will greatly reduce the rate of fluid flow under pressure, which is not so in the case of capillary absorption. The difference in the permeation property between cement paste and the normal aggregates used, is less. Hence the larger portion of the cement paste in the high workability concrete does not have the same degree of influence on the air permeation index results as on the ISAT values. Furthermore, segregated water, particularly bleed water, tends to cause a weak layer near the surface, with the Figg's Test measuring mainly the quality of the inner concrete (the outer 20 mm of the concrete is plugged with silicon rubber), the weak surface layer has less influence on the result.

Table 3.12.3. OPC Concrete Mixes, Designed for 75±5 mm Slump[80]

Mix	N4	N5	N5.5	N6	N7
Water/cement-ratio	0.40	0.47	0.55	0.62	0.70
Nominal 28 day strength (N/mm^2)	65	55	45	35	30

Table 3.12.4. Special Concretes Included in the Test Programme[80]

Concrete	Description
OPC	OPC control N55
RHPC	Rapid-hardening cement
PFA	OPC with pfa, C+F = $390 kg/m^3$ and F/(C+F) = 0.27
MS	OPC with 16 per cent microsilica
AEA	OPC with 5 per cent air entrainment
SA	Superplasticizing admixture

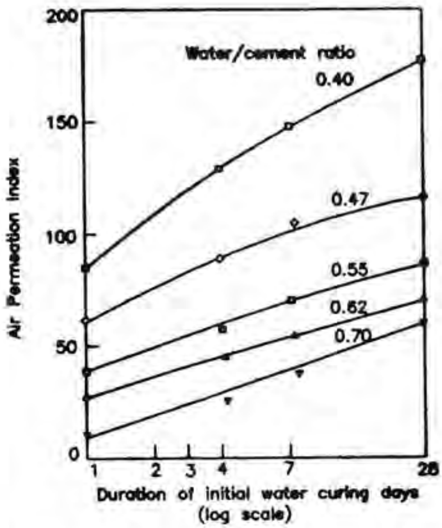


Fig. 3.12.3. Influence of Curing Conditions on Concrete Air Permeation Index, Modified Figg's Test, Test age 28 days[80]

3.13. Radar Technique

Principle

High frequency pulsed radar has been used to detect deterioration in concrete pavements and bridge decks[8]. Its use on bituminous surfaced bridge decks in measuring the thickness of the surfacing has also been exploited. The echoes produced from the pavement surface and the interface with the bridge deck concrete are very distinct such that the thickness can be determined accurately[40,41]. Short duration pulses of radio frequency energy are directed into the deck, a portion is reflected from any interface and the output is displayed on an oscilloscope and can be recorded and examined. Reinforcing bars, voids and ducts can be identified as well as zones of varying moisture content and the thickness of slabs can be estimated. An interface is any discontinuity or differing dielectric, such as, air to surfacing, to concrete, or cracks in concrete[39]. A permanent record is usually stored on magnetic tape. The unit is normally mounted in a vehicle and the data are collected as the vehicle moves slowly across the deck. Equipment consists of transmitting and receiving antennae together with a control unit and recorder, and is available in portable form, (Fig. 3.13.1.).

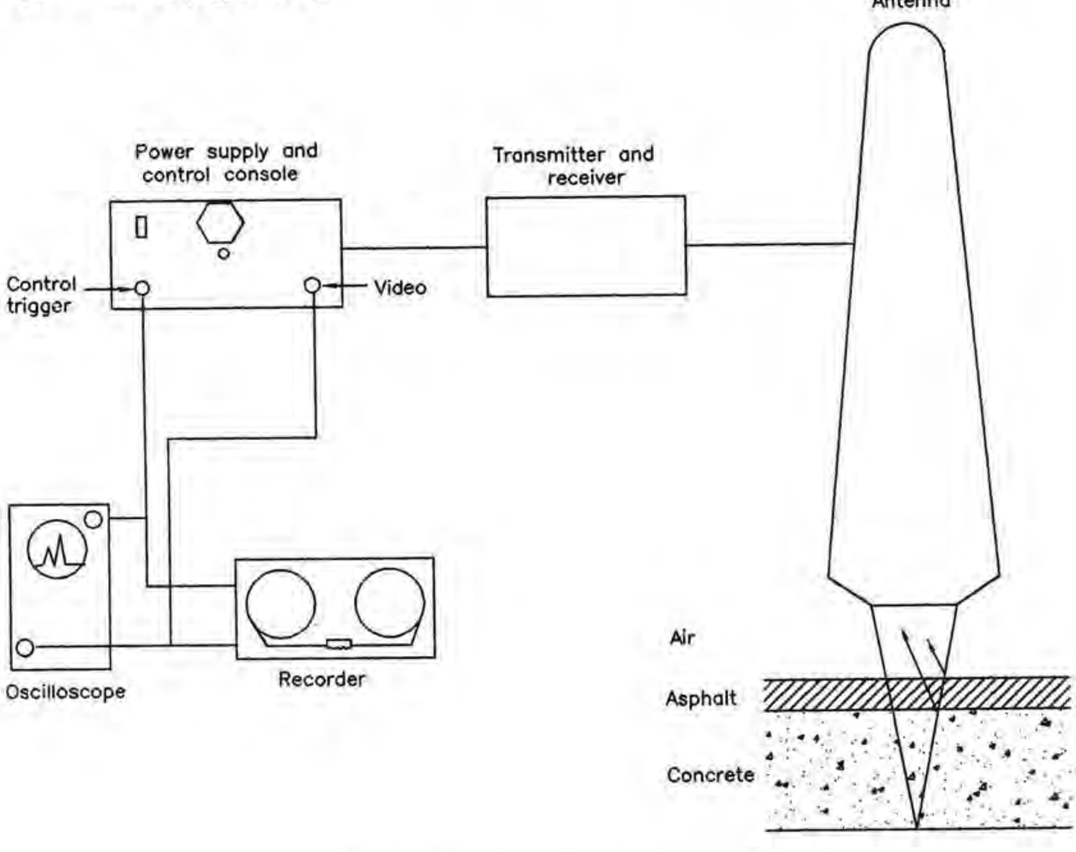


Fig. 3.13.1. Elements of a Radar System[8]

The application of radar technique to rebar location may provide advantages over other standard techniques, such as, covermeters because a graphical plot provides a paper record of the bar spacing and cover variations[110]. Profiles taken on parallel and perpendicular lines rapidly build up a picture of the overall reinforcement provision.

Specifications

Electromagnetic wave frequency = 100 to 1200 MHZ

Limitations

Analysis of the large amount of data collected and to relate the different radar signals to physical distress create practical problems. Further, the radar does not work effectively if the deck surface is wet or if there is significant moisture in a bituminous surfacing because of attenuation of the signal.

Calibration

When a change in material dielectric occurs at an interface, it is indicated by change in wave shape. The signal goes positive, when travelling from lower to higher dielectric material (air to concrete) and negative from higher to lower dielectric material, such as, a void under concrete (concrete to air). The changes in direction as the radiation penetrates from air to asphalt to concrete to air (bridge deck) or soil (roadway) are generally visible as peaks on the display of the radar trace and thin location along the trace are an indication of their length. Since this is a complex wave, peaks are not always clearly discernible.

The resolution that can be obtained, will depend upon the frequency used, which will in turn influence the depth of penetration possible which may typically be of the order of about 300 mm. Very small changes over short distances require a higher frequency and wider band width, whereas, lower frequency and narrower band width provides greater depth of penetration with lesser ability to see small changes. For micro studies, such as, location of voids or delaminations in thin sections, such as, roadways, bridge-decks, etc. higher frequency radar is used.

3.14. Infrared Thermography

Principle

Infrared thermography has been found to be capable of detecting delaminations in bridge decks[36,38]. The differences in surface temperature can be measured using sensitive infrared detection systems. The essential components of such a system are an infrared scanner, control unit, battery pack and display screen. The images can be recorded on photographic plates or videotape.

The method works on the principle that a discontinuity within the concrete, such as, a delamination parallel to the surface, interrupts the heat transfer through the concrete. This means that in periods of heating, the surface temperature of delaminations is higher than the surrounding concrete. At night, when there is usually a loss of heat from the concrete to the surrounding air, the surface of delamination is cooler than the average temperature of the solid concrete.

A number of operation modes have been used ranging from hand-held scanner at deck level to boom trucks and helicopters. A truck mounted plate from 4 to 6 m above the deck has been found to give best results with respect to definition, accuracy and speed. This configuration also permits a lane width to be scanned at a single pass and this is very convenient in the field.

Limitations

In addition to constraints imposed by weather the main disadvantage of thermogaphy is that while a positive result is valid, a negative result may mean that the deck is free from deterioration or that it contains deterioration that could not be detected under the conditions prevailing at the time of test. This problem is expected to become less serious as more experience is gained in predicting the conditions under which thermography will identify deterioration.

Infrared thermography can also be used to identify delamination and scaling in asphalt - covered deck slabs, although, the technique is much more sensitive to weather conditions than when applied to bare decks. Nevertheless, the method does have considerable promise as a rapid screening tool on both asphalt - covered and exposed concrete deck slabs to assess priorities and determine if a more detailed investigation is required.

Specifications

Height of bucket truck to scan the deck	= 5 to 20 m
Surface temperature difference for better result	= > 2°C
Height of helicopter (if equipment is mounted on it)	= < 400 m
Speed of truck	= 15 kmph (approx.)

Calibration

During the hottest part of the day the concrete temperature decreases with depth below the surface, whereas, at night the situation is reversed. Delamination will affect these temperature gradients, thus, enabling surface temperatures of sound and unsound portions to be compared by infrared measurements. Because of the small temperature differentials involved, the results cannot be directly recorded by infrared film and the image from the camera must be displayed on a cathode ray tube. The temperature of the surface is indicated by a range of greys, although, thermal contours can be automatically superimposed, and colour monitors are also available. This image is normally recorded on film to provide a hard copy[11].

It was found that surface temperature differentials existed in delaminated decks at most times, were greatest in summer, peaking in mid-afternoon. Attenuation of the reflected heat by the atmosphere, however, poses a major problem and this is increased by wind. Moisture on the surface was also found to mask surface temperature differences.

3.15. Chloride Determination : The Quantab Test

Principle

The test uses a commercially available "Quantab Test" strip[86] to measure the chloride concentration of a solution. A standard solution containing 5 gm of powdered concrete is prepared. A plastic strip, approximately 75 mm long and 15 mm wide, with a vertical capillary column impregnated with silver dichromate is used. At the top of the column is a horizontal air vent containing a yellow moisture sensitive indicator which changes to blue when the capillary is full. The lower end of the test strip is placed in the chloride solution until the capillary is full and the reddish brown silver dichromate in the capillary tube reacts with the chloride to form white silver chloride. The tip of this colour change is related to a vertical scale and reading converted to mg chloride ion/litre by reference to calibration tables. Although, the facilities for sample preparations are required, the method should be suitable for site use by staff without specialist experience and will be of sufficient accuracy to indicate the presence and level of significant chloride contents for most practical purposes.

3.16. Carbonation Test

Principle

Carbonation of concrete by attack from atmospheric carbon dioxide to form calcium carbonate forms a hard carbonated skin. The carbonation rates depend upon gas/air permeability of concrete and are also influenced by moisture level.

Carbonation of concrete by attack from carbon dioxide results in the reduction in alkalinity of the concrete. The extent of carbonation can easily be assessed by treating with phenolphthalein indicator the freshly exposed surface of a piece of concrete which has been broken from a member to give surface roughly perpendicular to the external face[106,11,107]. Incrementally, drilled powdered samples can be sprayed or allowed to fall on indicator-impregnated filter paper. Direct chemical tests may be used to determine the carbonation with greater precision. These include measurement of evolved carbon dioxide from slices of cores, about 5 mm thick using a range of specialised techniques, such as, thermogravimetry. Microscopy is probably the most precise method of measuring carbonation[1]. Drilled cores may also be split and sprayed with the indicator to measure the carbonation. A purple-red colouration will be obtained where the highly alkaline concrete has been unaffected by carbonation but no change in colour will be noticed in carbonated zone. A solution of 1 per cent phenolphthalein in 70 per cent ethyl alcohol is generally suitable to test the degree of carbonation. The colour change of phenolphthalein corresponds to a pH of about 9.

3.17. X-ray Diffractrometry (XRD), X-ray Fluorescence (XRF) Spectroscopy and Differential Thermal Analysis (DTA)

Principle

In XRD technique, the powdered sample of concrete is bombarded by high energy X-rays and from the presence or absence of particular reflected beams (peaks) at the respective angles of incidence of the X-ray beams ($2\theta^{o}$), different phases present or absent in the concrete can be studied. These peaks generally indicate calcium hydroxide, calcite, ettringite, calcium silicate hydrate, etc. in the concrete. An XRD pattern for an old concrete sample is shown in Fig. 3.17.1[113].

In XRF technique, a sample of concrete is bombarded by high energy X-rays and the fluorescent emission spectrum so caused is collimated into a parallel beam, directed on to the analyzing crystal within a spectrometer and reflected into a detector. The wave-lengths and densities of the fluorescent emission are measured and the constituent elements, together with their properties, can be calculated from this data. This method is a comparative one and the results obtained are compared with samples of known properties.

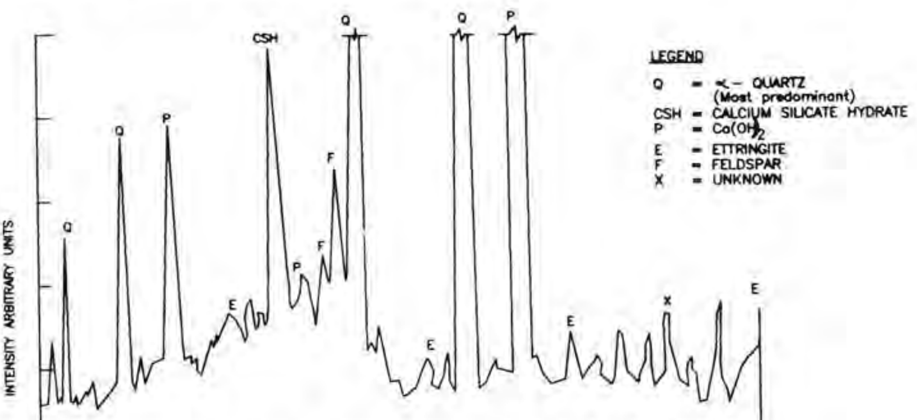


Fig. 3.17.1. X-ray Diffraction Pattern of an Old Hardened Concrete Sample

DTA technique is concerned with the rate of change of temperature of a sample as it is heated at a constant rate of heat input and involves heating a small sample of powdered concrete in a furnace together with a similar sample of inert material. The rate of temperature rise of the inert sample is controlled to be as nearly uniform as possible. The DTA graph has a series of peaks at particular temperatures which are characteristic for the minerals in the concrete sample under test. Fig. 3.17.2[113] shows a DTA graph for an old concrete sample. DTA graphs can be used for studying the status of hydration of concrete in the same way as XRD patterns. DTA studies are particularly useful for fire damaged concrete structures for assessment of temperatures to which the concrete was exposed during fire and also for assessment of the depth of affected concrete.

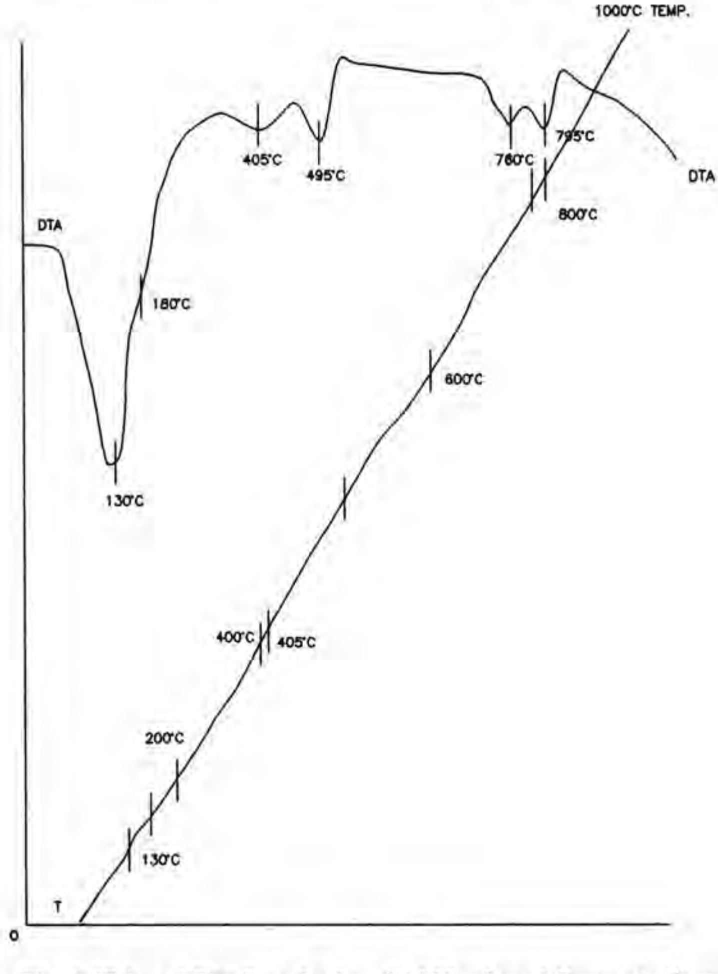


Fig. 3.17.2. DTA Graph of an Old Hardened Concrete Sample

3.18. Dye Penetration Examination

Principle

Examination with penetrants[87] is used to reveal defects which reach the surface of non-porous materials, such as, cracks, porosities, clevages and leaks in steel, cast iron, plastics, ceramics, etc. The examination (Fig. 3.18.1) is carried out in such a manner that the penetrating liquid (penetrant), which is dyed or fluorescent, is applied to the cleaned surface of the component. The penetrant must be allowed to act for a period of time depending, among other things, upon the temperature and the component under examination. Excess of penetrant is carefully removed from the surface of the component, after which a developing liquid is applied and dried off. The developer acts like a blotter, drawing the penetrant out of the defect. After some time indications appear in the developer which are wider than the defect and which, therefore, can be seen directly or under ultra-violet light due to the enhancement of contrast results between the penetrant and the developer. The method is used not only for the detection of surface defects but also to find leaks.

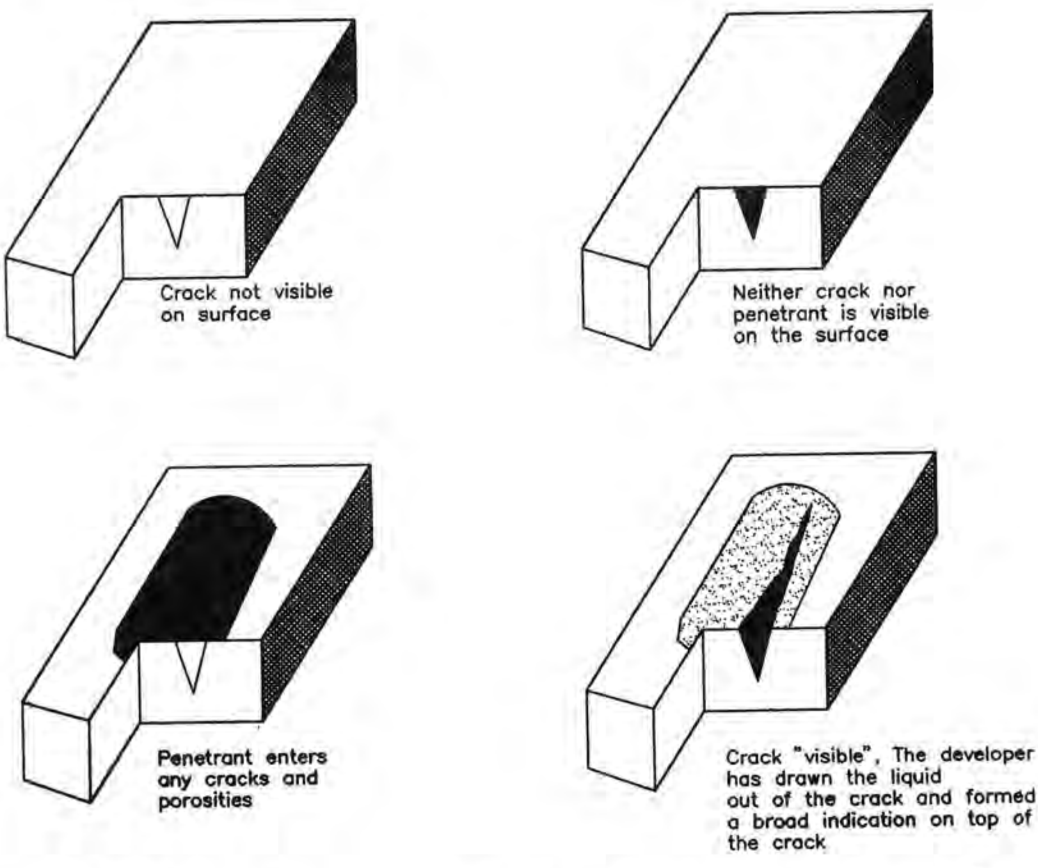


Fig. 3.18.1. Procedure useu when Performing an Examination with a Penetrant[87]

1. Pre-clean, remove greese and dry the component
2. Penetrant is applied to the component and acts for a brief period
3. Excess penetrant is completely removed from the surface
4. A developer is applied and dried off. Inspect for indication of defects.

Specifications

Penetrant types: The various penetrants which are used, may broadly be classified into the following two main categories:

(i) Dye Penetrants - The liquids are coloured so that they provide good contrast against the developer. The liquids are as a rule red with white developer. Observation is performed in ordinary daylight or good indoor illumination.

(ii) Fluorescent Penetrants -The liquids contain certain additives to give fluorescence under ultra-violet light. Inspectibn is performed protected from visible light. The indications are easy to see in dark because any developed penetrant emits light under the action of ultra-violet light. Furthermore, combined types are available which are both coloured and fluorescent.

Developer Types : The developer can be classified into three main groups:

* Dry Powder Developers
* Water Based Wet Developers
* Non-Water Based Wet Developers

In most cases wet developers are used. On some occasions it can be advantageous to use dry developers, i.e., on rough surfaces and for objects with concave corners and threads, where there is a tendency for the wet developer to form a layer which is too thick. The dry developer is then used in connection with a fluorescent penetrant. Among the non-water soluble wet developers organic solvent or alcohol based developers can be mentioned.

Inspection

Inspection for defect indications, when coloured dye penetrants are used, must be carried out in sufficiently bright daylight or artificial illumination. Most norms specify that the area to be observed must be illuminated with a luminous intensity of atleast 500 lux, corresponding roughly to the light from a new 80 W lamp one metre away. On the other hand, if fluorescent penetrants are used, the room in which the inspection is to be made should be darkened, hence, the illumination level should be no more than 10 lux.

Limitations

Method is suitable for all materials which are not porous. The penetrant method can prove to be quite slow for examining large surfaces.

3.19. Radiography

Principle

X- ray or gamma ray radiation[87] is used to perform a radiographic examination. In order to examine an object, it is irradiated with X-rays or gamma rays radiation. The radiation will be absorbed in the object to varying degrees depending upon the thickness of the object, the composition of the material and the wave length of the radiation. That portion of radiation which penetrates the object can be registered by recording it on a film (Fig. 3.19.1). The more radiation penetrating the object and striking the film, the darker the film appears when developed. By examining differences in exposure of the film, it is thus possible to determine difference in thickness and composition of the object. When the exposed object is concrete, it is possible to observe the steel reinforcing bars because different materials attenuate the radiation differently and thus expose the film accordingly.

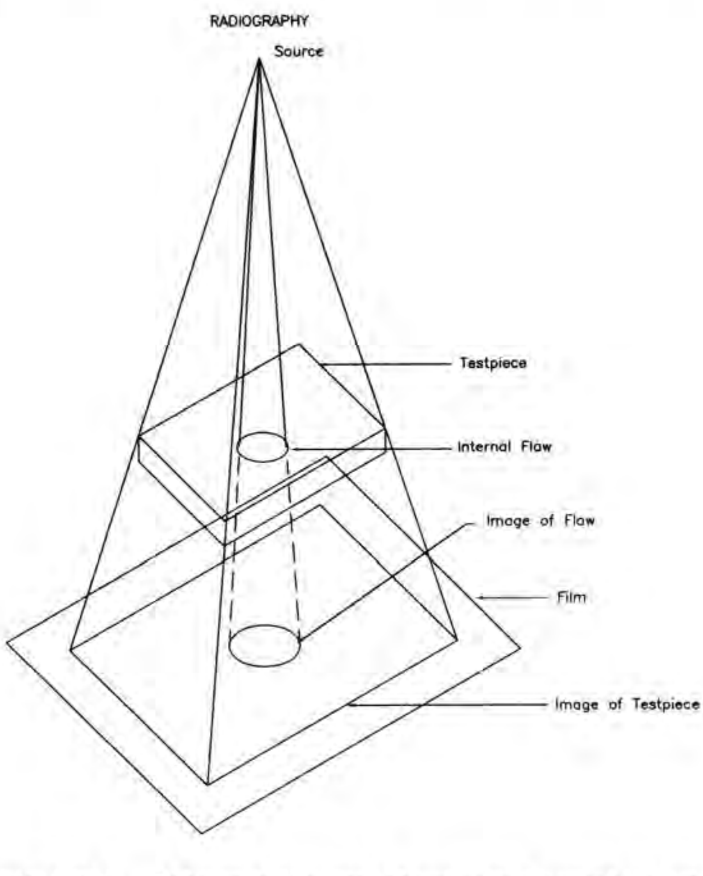


Fig. 3.19.1. Schematic Diagram showing Radiographic System[90]

Specifications

The choice of isotope as source of radiation[11] will depend upon the thickness of concrete involved. 192 Iridium for 25-250 mm thickness and 60 Cobalt for 125-500 mm thickness are most commonly used. In cases where particularly precise details are required, such as, specific identification of reinforcing bars or grouting voids, the image can be intensified by sandwitching the film between very thin lead screens. After the development of film, reinforcement will appear as light areas due to the high absorption of rays by the high density material, whilst voids will appear as dark areas. If it is required to determine the size and position of reinforcement or defects, photogrammetric techniques can be used in conjunction with the stereoscopic radiographs.

Limitations

It is expensive. It requires stringent safety precautions. It is also limited by member thickness. Although, 600 mm is sometimes quoted as an upper limit, for thickness greater than 450 mm, the exposure time becomes unacceptably long. Access to both sides of an object is generally necessary for placing film on one side and for transmitting the radiation from the other[11]. Radiography has the edge over other NDT methods in situations where visible images assessed by the eye and data analysis techniques yield more precise information than other methods.

References of applications

The first use of radiometric technique to measure in-situ density seems to have been in the early 1950s, when Smith and Whiffin[57], Hass[58] and others reported applications. More recently, Simpson[59] has described developments of the back scatter technique, whilst Honig[60] has outlined methods adopted in Czechoslovakia.

Pullen et. al.[77] gathered important engineering data in an examination of the road bridge over the Swaythling Brook on the A27 truck road at Southampton.

3.20. Portable Crack Measuring Microscope

Principle

Crack widths can be measured with a hand-held illuminated optical microscope, which is powered by a battery and is held against the concrete surface over the crack. The surface is illuminated by a small internal light bulb and the magnified crack widths can be measured directly by comparison with an internal graduated scale which is visible through the eye piece. Basically, the instrument[99] consists of (Fig. 3.20.1) an objective cell containing the objective lens, an ocular cell containing the eye and field lenses, and a reticle mounting arrangement, all assembled in optical and body tubes. The ocular cell is adjustable in the optical tube, thereby, allowing the eyepiece to be focussed sharply on the reticle. The complete optical tube is adjustable in the body tube by means of a rack and pinion, for focussing the microscope on the work.

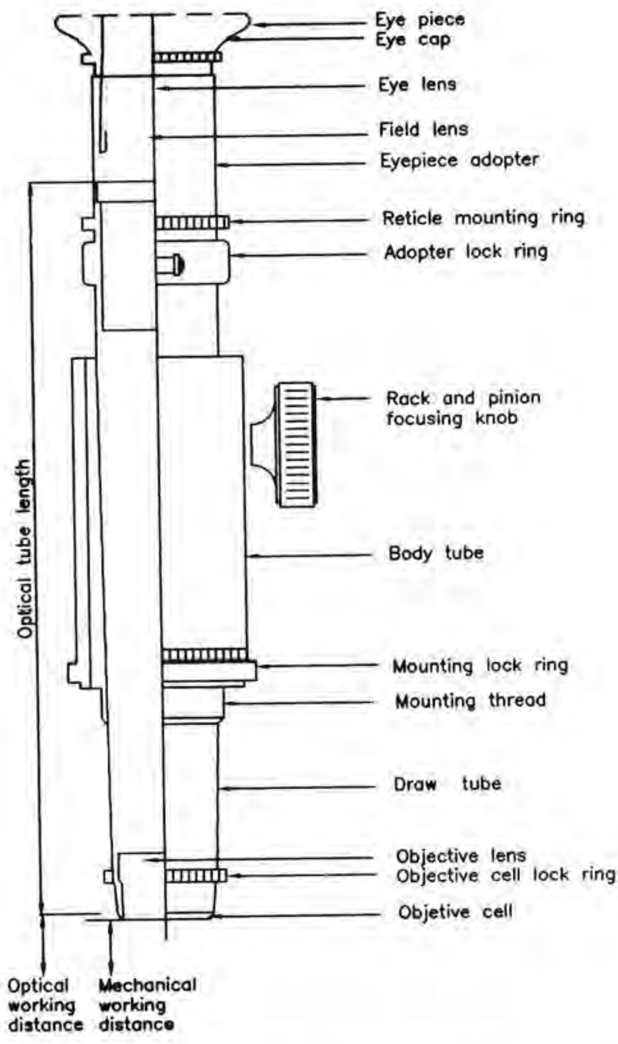


Fig. 3.20.1. Section through a Simple Low-Power Microscope[99]

Specifications

The least count of the instrument is usually of the order of 0.02 mm.

3.21. Stressmeter Based on Magnetic Properties vs Mechanical Stress

Principle

It has been known that the intensity of magnetization of ferro-magnetic body changes under the action of applied mechanical stress. The change is apparent in the incremental magnetic permeability which is a measure of the gradient of a minor hysteresis loop.

The Tensiomag[6] is a transducer claimed to be essentially designed for site needs and particularly applicable for measuring stress in the prestressing tendons, ground anchors, ordinary reinforcement, etc.

Specifications

The instrument consists of a transformer of appropriate type wrapped around the prestressing steel which fulfills the role of connecting core between the primary winding and the secondary winding of a transformer.

The primary, when supplied with current, produces sufficient magnetization in the steel tendon, while the secondary emits an electrical signal, dependent upon the magnetic state of the tendon and, therefore, of the stress imposed on it. The signal output level of the Tensiomag is sufficient to allow processing without amplification. In each case, the output signal from the Tensiomag can be related to the tensile stress by a calibration process.

A recess must be provided as soon as the structure is designed in which the pick-up can be housed. Preferably, it must be positioned before the concrete is cast and before stressing work is commenced.

The measuring appliance contains the necessary equipment for supplying electric power and excitation to the transducer and the measuring experiment itself.

Calibration

The Tensiomag is calibrated during the stressing work or on an identical tendon in the laboratory. The stress must be raised to its final value in stages (ten stages for example). For each intermediate value of stress, as read off the hydraulic tensioning system, the output signal from the Tensiomag is plotted. This data allows the calibration curve to be drawn. The signal from the Tensiomag allows the residual stress in the prestressing steel in the concrete structure to be read at any time during its lifetime by reference to the calibration curve.

Main characteristics

No amplification is required in processing of the output signal of Tensiomag. It indicates the total stress applied directly to the tendon. The relationship between the output signal and the stress to be measured is generally almost linear upto and even slightly beyond the stress corresponding to the conventional elasticity limit of 0.2 per cent strain. The sensitivity of Tensiomag is high, which depends on many parameters, i.e., the nature and the dimension of the element stressed, the form and size of Tensiomag, the characteristics of the magnetic field set up, etc. The tensile stress imposed on wires or strands is measured with an accuracy of 1 per cent. When measuring the tensile stress imposed on a bundle of strands, the accuracy obtained in the laboratory is in the order of 2 to 3 per cent. The method is sensitive to variation in temperature. The error in measuring stress introduced by a variation in temperature is 0.25 per cent per degree centigrade. The structural elements of Tensiomag do not deteriorate with time, i.e., they are not subjected to magnetic creep or aging.

Limitations

Application of the technique in bridges and performance of the sensor are not yet known.

3.22. Cover Thickness Measuring Technique[5]

Principle

When a metallic object is placed in the varying magnetic field of a coil, the field induces eddy currents in the metallic object. These eddy currents in turn produce an additional magnetic field in the vicinity of the metallic object. The magnetic fields, thus, get superimposed, and the magnetic field near the coil also gets modified if a metal is present. This magnetic field modification has the same effect as would be obtained if the characteristics of the coil itself had been changed.

The change depends upon the electrical conductivity, dimensions, i.e., diameter of rod, magnetic permeability, presence of discontinuity, such as, cracks or cavities, frequency of the field of the coil, size and shape of the coil and distance of the coil from the metallic object.

It is possible to measure the cover thickness for a known diameter by keeping all the other parameters constant. By placing the coil at two different distances from the rebar, both the cover thickness and diameter of the rebar can be found out.

Specifications

The electronic circuit consists of an oscillator, probe coil and amplifier. The oscillator generates a stabilized sine wave of 1KHz frequency. This is applied to the probe coil through a resistance. The probe coil has an inductance and along with a condenser forms a parallel resonant circuit. The voltage across the probe coil is connected to a voltage follower which acts as a high impedance buffer. The A.C. voltage is rectified and a D.C. amplifier stage follows.

When there is no metallic object in the vicinity of the probe coil, the voltage across the probe coil is maximum and the output of the amplifier stage is maximum. The cover thickness in such a case is infinity. When the probe is placed over a concrete cube containing a rebar, the voltage across probe coil gets reduced and the output of D.C. amplifier is also reduced. This reduction depends on the cover thickness and diameter of rebar.

Portable battery operated cover thickness meters based on eddy current are available[5]. The instrument is useful in measuring the actual concrete cover provided over embedded steel reinforcement. Orientation and profile of prestressing cables in case of prestressed members can also be ascertained. Most covermeters consist of a unit containing the power source, amplifier and meter, and a separate scanner unit containing the electromagnetic, which is coupled to the main unit by a cable.

Calibration

Fig. 3.22.1[5] gives the relationship between the output of the D.C. amplifier and the cover thickness for various rebars with diameter 6 mm, 10 mm and 16 mm. It is also seen from the graph that the sensitivity decreases as the cover thickness increases.

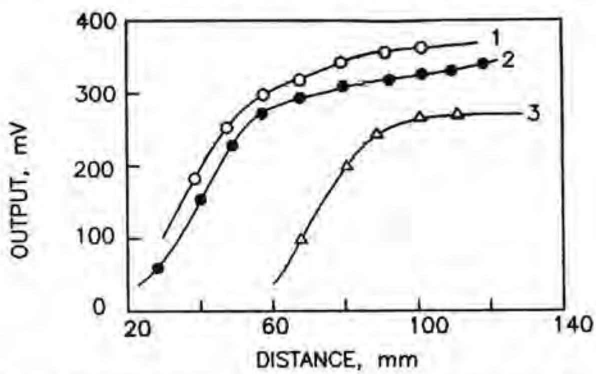


Fig. 3.22.1. Relation between Cover Thickness (distance in mm) with Output Voltage (mv)[5]

If the rebar diameter is known, from the output voltage of D.C. amplifier (Fig. 3.22.1) the cover thickness can be calculated. When both the cover thickness and rebar diameter are not known, the following procedure can be adopted.

The probe is placed on the concrete surface and the output of D.C. amplifier stage 'E1' is noted. Then a 20 mm spacer is placed between the test probe and the concrete surface and the output voltage 'E2' is noted. With the help of chart giving relationships between E1 and E2-E1 for different bar diameters (Fig. 3.22.2[5]), the line on which the point (E1, E2-E1) lies, indicates the diameter of rod. Then knowing the bar diameter the cover thickness can be found out. It is possible to get cover thickness upto 110 mm for the rebars having diameter 6 mm,10 mm and 16 mm.

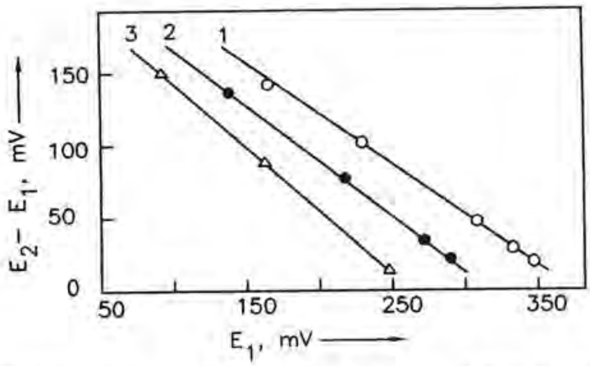


Fig. 3.22.2. Relation between E1 and (E2-E1) to get Rebar Dia (1) 6 mm Dia, (2) 10 mm Dia, and (3) 16 mm Dia[5]

Limitations

Influence of steel on the induced current is non-linear in relation to the distance and is also affected by the diameter of the bar, which makes calibration difficult. For bars less than 10 mm of greater than 32 mm to be measured, it may be necessary to develop a special calibration.

Calibration can be influenced by presence of more than one reinforcing bar. Lapse, transverse steel for closely spaced bars can cause misleading results. Variations in the iron content of the cement, and the use of aggregates with magnetic properties may cause reduced covers to be indicated.

BS:1881 (Part 204) suggests that an average site accuracy at covers less than 100 mm of about ±15 per cent may be expected with a maximum of ±5 mm.

3.23. Electronic Geodetic Techniques

Principle

Electronic distance measuring (EDM) instruments[78] function by sending a light wave or microwave along the path to be measured and measuring the time involved in traversing the required distance as with microwaves, or in measuring the time involved in returning the reflecting light wave back to source. The microwave systems require a transmitter - receiver at both ends of the measured line, whereas, light wave systems use a transmitter at one end and a reflector at the other end.

Long-range land measurements can be taken with laser instruments that have a distance capability in the range of 60 km. Most short range instruments (0-3000 m) now in use are infra-red instruments (gallium-arsenide diode). In such instruments a wave of wave length λ, travels along the X-axis with a velocity V of 299792.5±0.4 km/s(in vacuum). The frequency f of the wave is the time taken for one complete wave length, thus

$$\lambda = V/f$$

The modulated electromagnetic wave leaves the EDM and gets reflected (light wave) or retransmitted (microwaves) back to the EDM. It can be seen that the double distance (2L) is equal to a whole number of wave lengths ($n\lambda$), plus the partial wave length (ϕ) occurring at the EDM

$$L = \frac{n\lambda + \phi}{2} \text{ meters}$$

The partial wave length (ϕ) is determined in the instrument by noting the phase delay required to precisely match up the transmitted and reflected or retransmitted waves. The commercially available instruments can count the number of full wave lengths or, instead the instrument can send out a series (three or four) of modulated waves at different frequencies. By substituting the resulting values of λ and ϕ into the above equation, the value of 'n' can be found out. The instruments are designed to carry out this procedure in a matter of seconds and then to display the value of 'L' in digital form.

The following instruments are commonly used:

(1) DISTOMAT
(2) Electronic Theodolite Instrument
(3) Electronic High Precision Level

Specifications

Typical EDM characteristics[78] are as follows:

Range	: 75 - 1000 m (Single Prism)
Accuracy	: ±(5 mm + 5 ppm)
Operating temperature range	: -20 to 50°C
Slope reduction ______________________	manual or automatic

Typical features of high precision level are as follows:

- Provided with micrometer and internal illumination
- Precision 0.1 mm
- Accessories: Tripod, Suitably high Invar staff with circular spirit level, Ground plate, etc.

3.24. Linear Variable Differential Transformer (LVDT)[99]

The LVDT provides an A.C. voltage output proportional to the relative displacement of the transformer core to the windings[99].

The central coil is energized from an external A.C. power source and two end coils, connected together in phase position, are used as pick-up coils. Output amplitude or phase depends on the relative coupling between the two pick-up coils and the power coil. Relative coupling is, in turn, dependent on the position of the core. Theoretically, there should be a core position for which the voltage induced in each of the pick-up coils will be of the same magnitude, and the resulting output should be zero. In actual practice, this condition is difficult to attain, typical differential transformer features are shown in Fig. 3.24.1, which shows output vs. core movement. Within limits on either side of the null position, core displacement results in proportional output. In general, the linear range is primarily dependent on the length of the secondary coils. Although, the output voltage magnitudes are ideally the same for equal core displacements, on either side of null balance, the phase relation existing between power source and output changes 180° through null. It, is therefore, possible, through phase determination or the use of phase sensitive circuitary to distinguish between the outputs resulting from displacements on either side of null. As displacement recording instrument, it requires a stationery base.

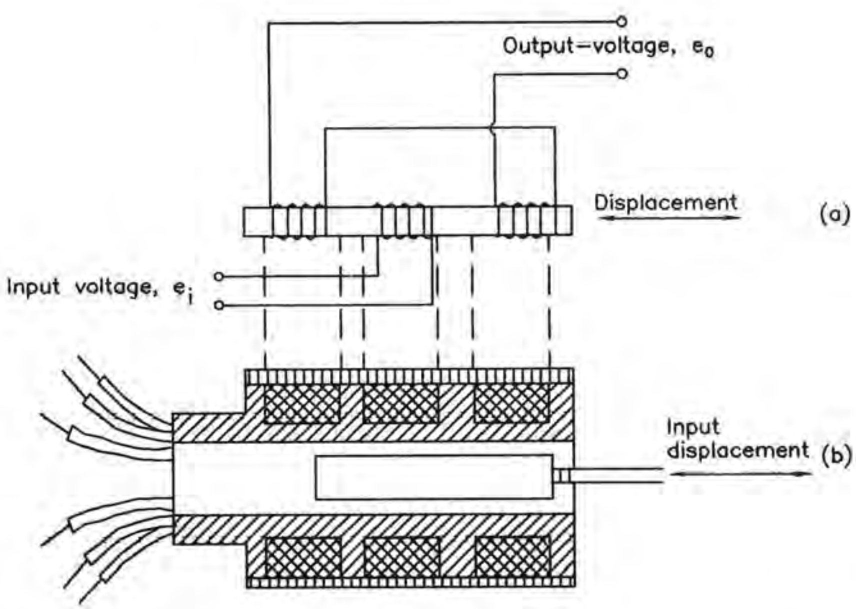


Fig. 3.24.1. Differential Transformer (a) Schematic Arrangement and (b) Section through a Typical Transformer[99]

3.25. Vibrating Wire Strain Gauge[103] (Surface Mounted Type & Embedment Type)

Surface Mounted Type Strain Gauges are designed primarily for arc welding them to steel structures, such as, bridges, piles, tunnel linings and supports. By modification of the end blocks they can be adopted for attachment to concrete surfaces. Performance of these gauges would, however, very much depend upon the fixing detail of the sensor on the concrete surface. In these strain gauges, a length of steel wire is tensioned between two end blocks that are welded to the steel surface being studied. Deformations (i.e., strain changes) of the structure will cause the two end blocks to move relative to each other, thus altering the tension in the steel wire. The tension is measured by plucking the wire and measuring its resonant frequency of vibration using an electro-magnetic coil.

In case of embedded type strain gauges, a length of steel wire is tensioned between two end blocks that are embedded directly in concrete. The principle is the same as that of surface mounted type gauge.

Accessories used with the vibrating wire strain gauge provide the necessary voltage pulses to pluck the wire and convert the resulting frequency reading directly into strain units by means of an internal micro-processor. The advantage of the vibrating wire strain gauge over more conventional electrical resistance gauges lies mainly in the use of frequency rather than a voltage as the output signal from the strain gauge. Frequencies can be transmitted over a long cable lengths without appreciable degradation caused by variations in cable resistance, contact resistance or leakage to ground. They are suited more for long term monitoring.

Typical specifications

- Gauge length as per requirement
- L.C. of the order of 1 microstrain
- Fitted with temp. monitoring device
- Signal cable of adequate length, robust, waterproof
- Operating temp. and strain range to be specified
- Rechargeable portable VW Readout unit with in-built data recording device, and cables to connect sensor leads or junction box
- Gauge body of stainless steel
- Junction box to connect sensors
- Multi-channel automatic data recording system

SERC, Ghaziabad extensively used embedded type of VW gauges in their Instrumentation projects on Mandovi bridges in Goa and Ganga bridge at Varanasi[169]. Fig. 3.25.1 shows typical fixing details of various types of strain gauges.

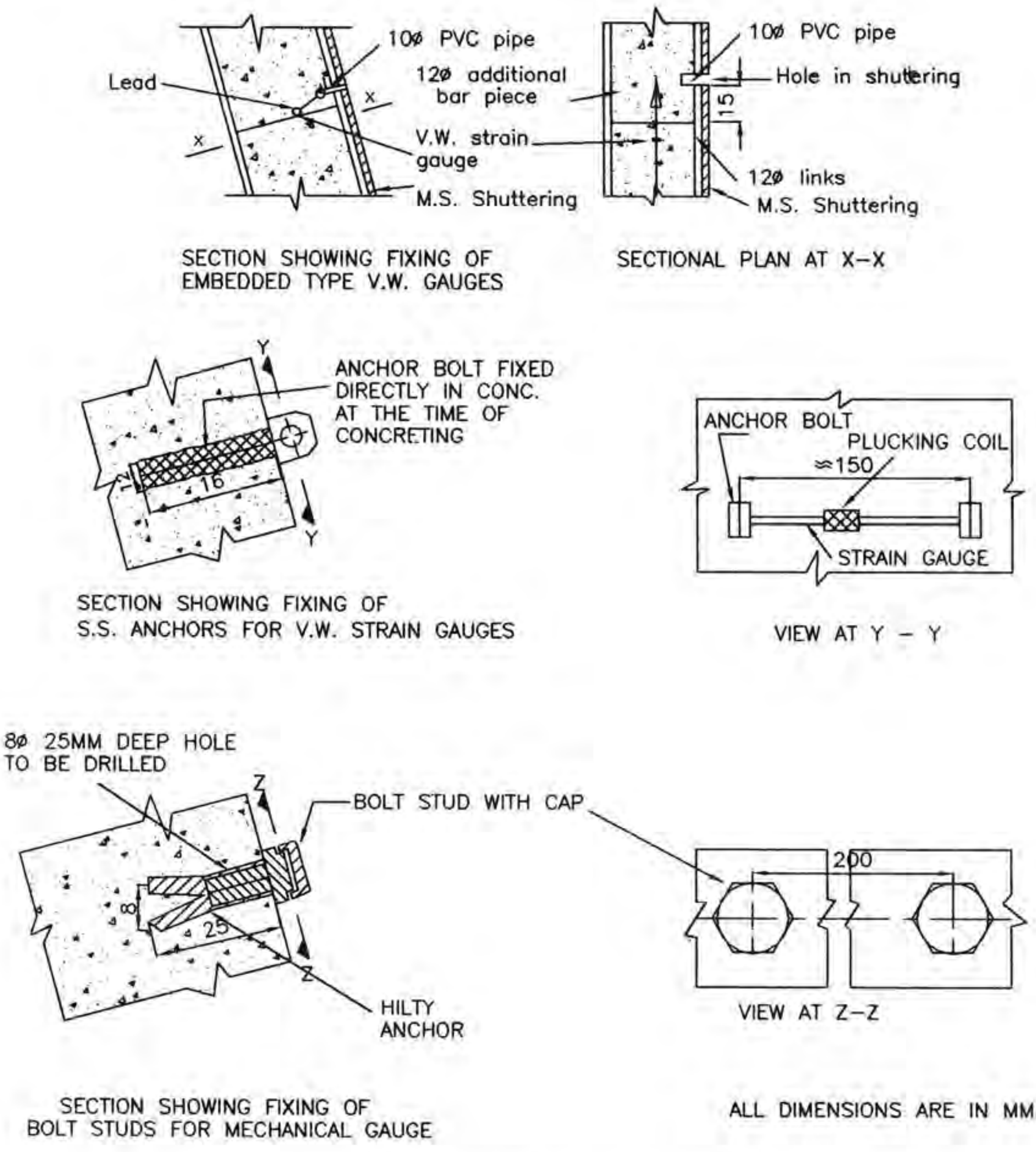


Fig. 3.25.1. Typical Fixing Details of various Types of Strain Gauges[169]

3.26. Contact -Type Strain Gauge

The Contact-Type Strain Gauge designed by Pfender[96,97,98] is set with its bolt contact points on the measuring tract (Fig. 3.26.1). One contact point is stationary, the other contact point is movable, forming the short arm of an angular lever. If the length of the measuring tract changes, the movable contact point is shifted according to the variation in length. The free end of the lever arm moves five times the variation of length, corresponding to a lever transmission ratio of 1:5. The position of the lever will be fixed and the dial reading is taken. Later deformation of the previously stressed measuring tract can be observed by repeated applications of the measurement to the same measuring tract. The difference of length is ascertained by comparing the 2nd and 1st indication of the dial with each other. Measuring tracts are marked exactly by driving small hardened steel balls (1/16" dia) into the material to be monitored[149]. The manner of marking is claimed to make correct measurement possible even after a long time. The special advantage of the instrument is that many measuring tracts can be measured by an instrument.

Fig. 3.26.1. Pfender Gauge held on Standard Bar

Spherical ball type contact with the instrument at the sensing points provides relatively steady readings compared to other conventional mechanical strain meters. L.C. of instrument is of the order of 10 microstrains.

Specifications

Measuring length variable :	L	= 20....100 mm
(Extension by means of additional holders possible)		
Max. variation of length :	Lmax	= ± 0.5 mm
Min. measurable extension :	Lmin	= ± 0.001 mm

SERC, Ghaziabad extensively used such type of sensors alongwith embedded VW gauges in their instrumentation projects on Mandovi bridges in Goa[169].

3.27. Tiltmeter[94]

Principle

Tiltmeters measure tilt in natural and man made masses and structures, in which the measure component is rotational in a vertical plane. General application examples include settlements induced by construction, subsidence over mines, differential compression in earthen dams, other man made embankments and land slides. More specific applications involve observations in open pit mines, retaining walls, bridge piers and ground surfaces over underground mines and adjacent to foundation excavations.

A typical Tiltmeter System includes Tiltplates, Tiltmeter Sensor and Indicator. The Tiltplates are bonded to the surface of the mass or structure. The sensor is oriented on three pegs of the Tiltplate and senses change in tilt of the tiltplate in two orthogonal directions. The portable indicator displays the reading.

Specifications

Range of inclination	: $\pm 30^{\circ}$ from horizontal
Sensitivity (Smallest change in tilt angle)	: 1 in 10000

SERC, Ghaziabad used tiltmeter sensors alongwith other deflection measuring techniques in their instrumentation projects on Mandovi bridges at Goa[169].

3.28. Inclinometer[95]

Principle

Inclinometer is a high precision instrument for measuring subsurface displacement or deformation[95]. Lateral movements within landslides, earth dams, foundations can generally be measured more precisely and economically than with other types of instruments. The instrument is normally lowered down a grooved plastic or aluminium inclinometer casing installed in the structure near vertical. The grooves control the orientation (azimuth) of the instrument in a predetermined direction. Inclinometer readings are taken at frequent intervals of depth and are subsequently converted to displacements.

Specifications

Range of Inclination	: $\pm 53^{\circ}$ from vertical
Sensitivity (Smallest change in inclination angle)	: 1 in 20,000
Accuracy (Error in deflection measurement)	: $\pm$ 7.5 mm

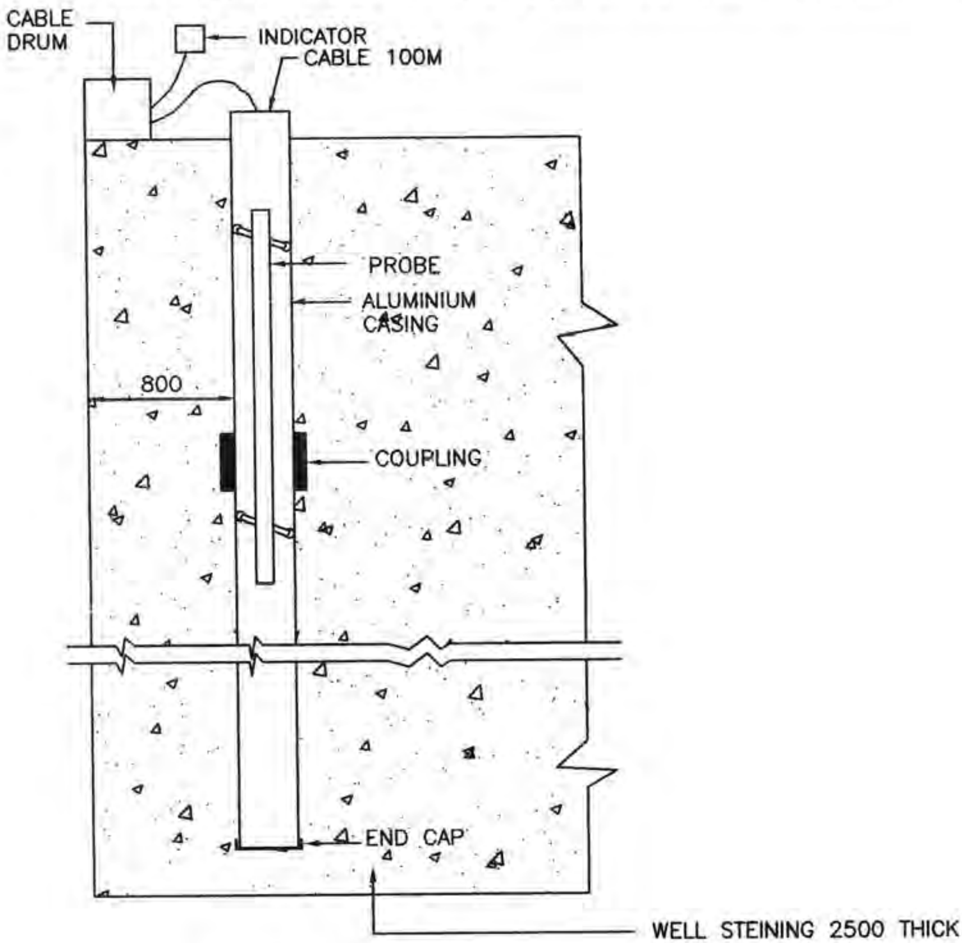


Fig. 3.28.1. Inclinometer System (Schematic)[169]

SERC, Ghaziabad used this technique by embedding ducts in one of the foundation wells of the Ganga bridge at Varanasi and monitoring tilts during its construction phase[169]. Fig. 3.28.1 shows the inclinometer system in position.

3.29. Pressure Transducer[103]

Principle

A pressure transducer[103], usually a piezometer, is attached to the fluid filled pressure cell. Soil pressure on the flat walls of the cell is converted to fluid pressure and measured by the piezometer. Pressure cells are fixed at the interface between wall and soil. The typical applications include measurement of pressure at soil-concrete interface, foundation bearing pressures, and the orientation and magnitude of stress within dams and embankments.

Typical specifications

- Pressure cell with piezometer fitted with temp. sensing device
- Signal cable factory fitted, robust, waterproof and of adequate length (Resolution 0.025 per cent F.S.)
- Gauge body of stainless steel
- Compatible readout unit

SERC, Ghaziabad extensively used this technique by embedding pressure cells at the interface of soil and well steining along the well periphery at different heights[169]. Fig. 3.29.1 shows the pressure cell assembly in position.

Fig. 3.29.1. Pressure Cell Assembly in Position before Concreting[169]

3.30. Thermocouple, Thermometer and VW Temperature Sensor

Principle

It was discovered that an electromotive force exists across a junction formed between two unlike metals[101]. It was further shown[102,103] that the potential actually results from two different sources: (a) resulting solely from contact of the two dissimilar metals and the junction temperature (Fig. 3.30.1, where A and B are the two conductors, T1 and T2 are the junction temperatures at 'p' and 'q' respectively); (b) due to temperature gradients along the conductors in the circuit. In most cases EMF due to temperature gradient along the conductors in circuit, is quite small relative to the EMF due to the contact of two dissimilar metals and the junction temperature and with proper selection of materials can be disregarded. These effects form the basis for temperature measuring element[100]. Because of its small size, reliability and relatively large range of usefulness, the thermocouple has the potential for field applications[99].

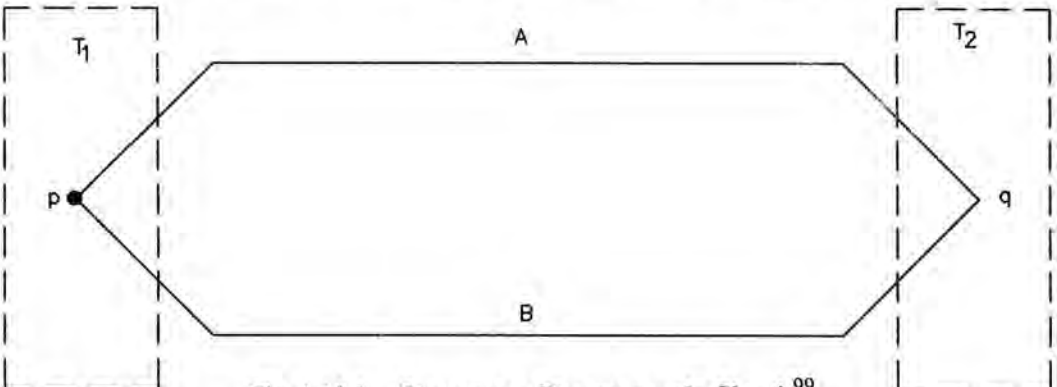


Fig. 3.30.1. Elementary Thermocouple Circuit[99]

The ordinary thermometer is an example of liquid in glass tube. Essential elements consist of a relatively large bulb at the lower end, a capillary tube with scale and liquid filling both the bulb and a portion of the capillary or stem of the thermometer and the height of rise in the tube is used as a measurement of the temperature[100].

The Vibrating Wire Temperature Gauge consists of a stainless steel transducer body to which a vibrating wire element is attached. Different co-efficients of thermal expansion of the body and the wire make it a simple, sensitive temperature measuring device. The gauge is protected by a stainless steel housing which allows it to survive in the most severe environments. The output signal from the vibrating wire temperature gauge is in the form of a frequency, the stability and accuracy of which is unaffected by changes in cable resistance caused by water penetration, temperature variations and contact resistance. These gauges can be used conveniently where other types of vibrating wire transducers are employed as all of them may require the same logging device.

Typical specifications of VW sensor

- Embedment type VW temp. sensor fitted with temp. sensing device
- Signal cable robust, waterproof and of adequate length (Resolution 0.025 per cent F.S.)

- Gauge body of stainless steel
- Compatible readout unit
- Accuracy 0.05°C
- Temp. range: as per site requirement

SERC, Ghaziabad extensively used VW temperature sensors in the flanges and webs of the box girders of the Mandovi bridges in Goa[169].

3.31. Open Circuit Potential Measuring Technique

Principle

The tendency of any metal to react with an environment is indicated by the potential it develops on contact with that environment. In reinforced concrete structures, concrete acts as an electrolyte and the reinforcement develops a potential depending on the concrete environment which may vary from place to place. The principle involved in this technique is essentially of measuring of corrosion potential of rebar with respect to a standard reference electrode. The electrical circuit for open circuit potential measurement is shown in Fig. 3.31.1. The steel rebar in concrete structure should be accessible in few locations for giving electrical connections. The positive terminal of high impedance voltmeter is connected to exposed rebar and negative terminal (common) to reference half cell. The surface of concrete is divided into number of grids. The reference electrode is moved along the nodal points and corresponding potentials are recorded. These are referred to as either open circuit potential or corrosion potential. As per ASTM standards, the probability of reinforcement corrosion is as follows :

OCP VALUES in terms of mV vs. SCE	mV vs. CSE	Probability (per cent) of corrosion
more -Ve than -275	more -Ve than -350	< 90
Between -275 & -125	Between -350 & -200	uncertain
More +Ve than -125	More +Ve than -200	< 10

Specifications

The equipment for open circuit potential measurement consists of the following :

(i) High impedance voltmeter

A voltmeter with an input impedance of more than 10 Mn with an accuracy of ± 10 mV should be used.

(ii) Reference electrodes

Three reference electrodes commonly used for potential monitoring of steel in concrete are:

(a) Saturated Calomel Electrode (SCE)
(b) Copper/Copper Sulphate Electrode (CSE)
(c) Silver/Silver Chloride Electrode

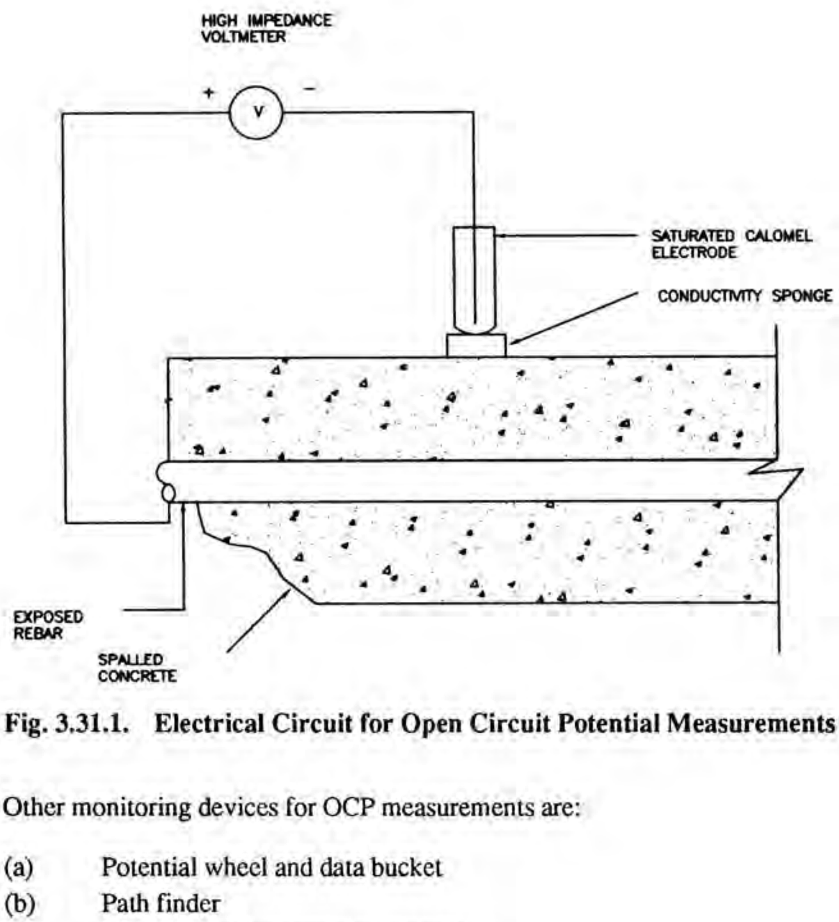


Fig. 3.31.1. Electrical Circuit for Open Circuit Potential Measurements

Other monitoring devices for OCP measurements are:

(a) Potential wheel and data bucket
(b) Path finder
(c) Bloodhound for corrosion mapping

Potential wheel[146]

Potential wheel can be used to identify areas of high and low corrosion risk by surveying the structure. It enables a thorough and quick investigation of the structure. Potential wheel is actually a half cell with a wheel at tip. Wheel is placed in contact with the concrete. It is then drawn on the surface of the structure to provide a continuous record of corrosion potential by moving a water saturated tyre of the wheel, water being used as the conductive medium. The data bucket is carried by the operator and is connected to the potential wheel by a flexible cable. The operator communicates with the microprocessor in the data bucket using a small hand-held terminal incorporating keyboard and display.

Bloodhound for corrosion mapping[147] is another commercially available instrument for corrosion mapping. It is reported to have features, like, data on potentials and electrical resistance, estimation of the cover condition, computerised readings, $Cu/CuSO_4$ or Ag/AgCl measuring wheel, etc.

Path finder

In situation, where large number of potential measurements are to be carried out, it is convenient to use path finder, which is incorporated with 8 reference half-cells to enable continuous monitoring rapidly. Measurements are made against copper/copper sulphate electrode.

Limitations

This technique gives only a qualitative data. As OCP values are influenced by moisture content in concrete, valid electrical potentials can be obtained, with proper precautions at any time of the year but during dry season. Concrete should be prewetted at the points where OCP is to be taken. Monitoring of reinforcement corrosion by this technique is not reliable in submerged zones. OCP values are temperature dependent. Potential measurements on coated rebars will not give the real OCP values and hence will not reflect the real condition of the rebar. Stray currents in the surrounding area may also affect the potential of the structure. The large surface area of the steel grid acts as an effective radio antenna, picking up high frequency signals. These signals can affect the operation of a solid state, high impedance voltmeter attached to the structure, resulting in erratic readings. Delamination in the concrete can also affect the potential measurements.

Field applications

It has been reported by many workers that this technique has been extensively used to assess bridge deck corrosion in USA[115-117]. Potential studies were carried out by California Division of Highways on three different bridges in USA[118]. As the deck slab was covered with asphaltic concrete overlay, it was attempted whether there will be a correlation between potential measurements on sealed deck and actual probability of corrosion. Open circuit potential measurements obtained on the deck were subject to much uncertainty that a definite relationship between the top deck surface and soffit potential could not be established. Potential studies made on asphaltic concrete overlay on bridge deck showed that potential values varied with time leading to large errors in the potential value.

CECRI has also reported extensive work using open circuit potential measurement. Potential studies in the case of a bridge structure I, which was under distress indicated more than 50 per cent probability of corrosion[129]. Similar potential measurements when made on a bridge structure II which was in good condition showed values in the range -26 to -40 mV vs. SCE at 3 locations indicating less than 10 per cent probability of corrosion. Extensive studies carried out on different members of various bridges indicated 60 - 80 per cent probability of corrosion in Bridge III while in another Bridge IV, deck slab showed 70 - 90 per cent probability of corrosion. In a bridge structure V, showing advanced stage of reinforcement corrosion a potential value as high as -1000 mV[123] was noted.

3.32. Surface Potential Measuring Technique

Principle

During corrosion process an electric current flows between the cathodic and anodic sites through the concrete and this flow can be detected by measurement of potential drop in the concrete. Hence, surface potential measurement is used as a non-destructive testing for identifying anodic and cathodic region in concrete structures and indirectly detecting the probability of corrosion of rebar in concrete. Two reference electrodes are used for surface potential measurements. The electrical circuit for this system is shown in Fig. 3.32.1. No electrical connection to the rebar is necessary in this technique.

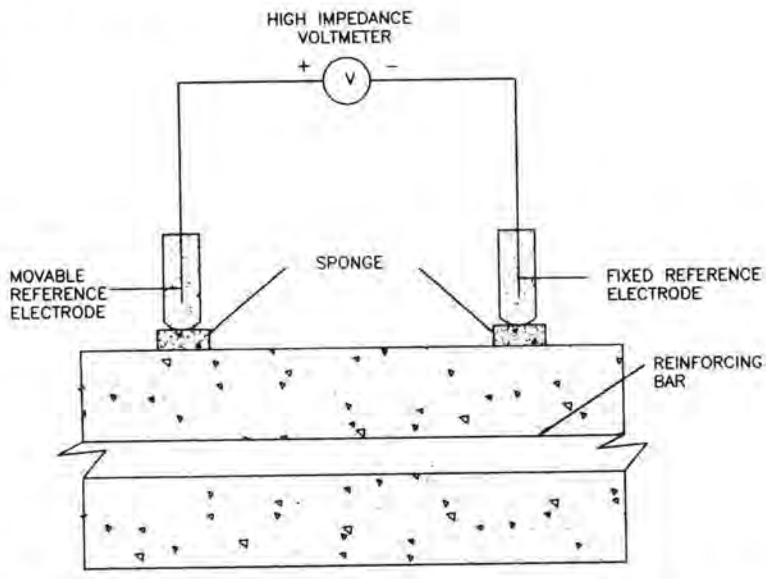


Fig. 3.32.1. Electrical Circuit showing Two Reference Electrodes used for Surface Potential Measurements

In this technique one electrode is kept fixed on the structure on a symmetrical point. The other electrode called moving electrode is moved along the structure on the nodal points of the grid as mentioned in OCP measurements. The potential of movable electrode when placed at nodal points is measured against the fixed electrode using a high impedance voltmeter. Equipotential contours are plotted as shown in Fig. 3.32.2 to form a contour map of potential gradient. A more positive potential reading represents anodic areas where corrosion is possible. Greater the potential difference between anodic and cathodic areas, greater is the probability of corrosion.

Specification

Equipment needed for surface potential measurements is as follows:

(i) A voltmeter with an input impedance of more than 10 Mn with an accuracy of $\pm$ 10mV,

(ii) Two standard reference electrodes, such as, Calomel, Copper - Copper Sulphate Electrodes, etc.

Limitations

Eventhough this technique does not rely on any electrical continuity in the reinforcement, choosing of a sound area of structure should be made for the static electrode position to avoid any problems due to delaminations in concrete, which may increase the variability of measured potentials.

Although, anodic and cathodic areas are identified, the surface potential measurements recorded are only for comparative studies which would help in understanding the changes occurring in the structure. This measurement by itself will not indicate corrosion behaviour of rebar embedded in concrete. It is to be coupled with the resistivity measurements to obtain a parameter "corrosion cell ratio" for assessing the probability of corrosion of reinforcements.

Field applications

Stratful has adopted surface potential measurements as a method of detecting active corrosion area in San Manteo Hayward Bridge, California[125]. Potential distribution pattern identified the affected areas of corrosion in concrete pilings, caps and main deck beams of the bridge structure.

It has been reported that whenever a potential difference obtained on concrete structure is not more than 30mV, it indicates that the steel is in passive condition, whereas, if surface potential difference exceeds 100mV vs. CSE, it indicates active corrosion condition[122].

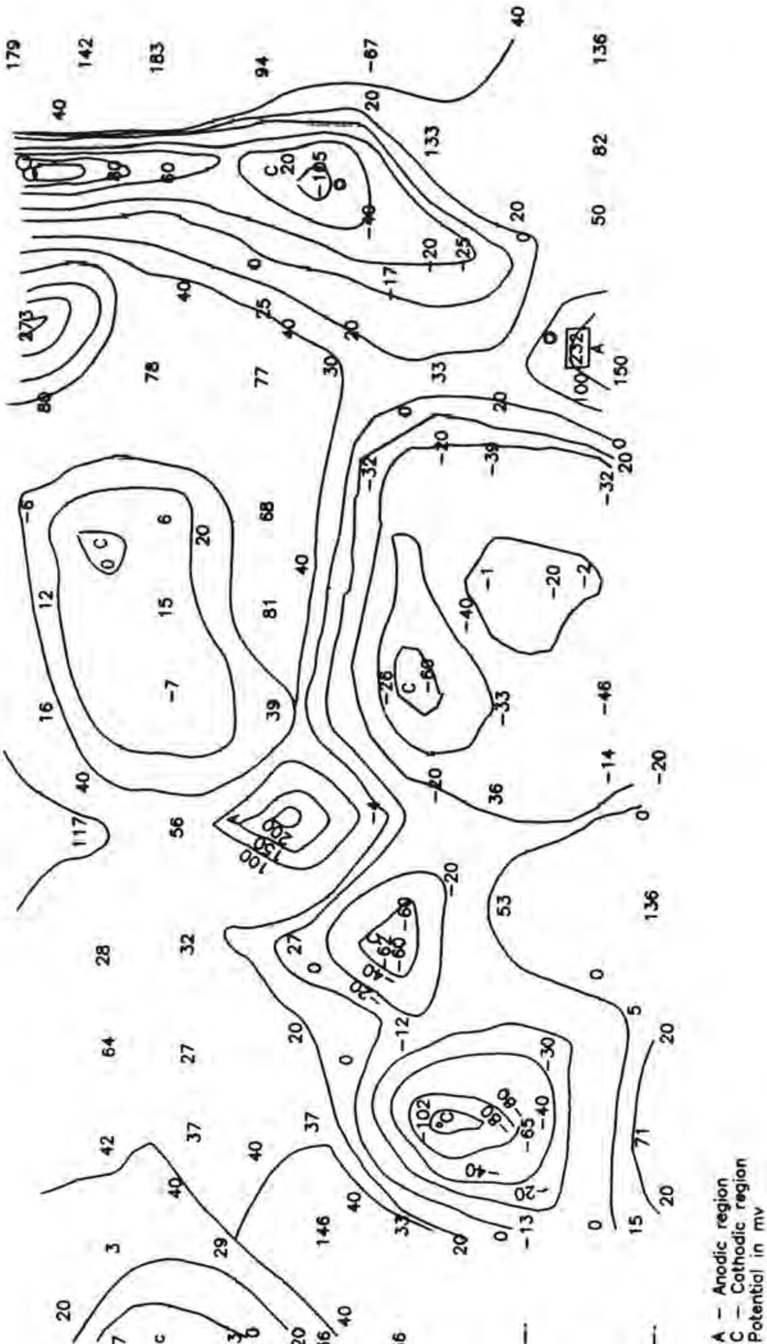


Fig. 3.32.2. Typical Equipotential Contours Obtained on a Bridge

Surface potential technique was employed by CECRI in different members of bridge structures for identifying the more anodic areas in the structure[128,129]. Maximum surface potential difference values were obtained on girders, diaphragms and soffit of deck slabs which indicated that active corrosion had set in many places in those components.

3.33. Polarisation Resistance Technique

Principle

Among the electrochemical techniques, the best known technique for evaluation of instantaneous corrosion rate in the laboratory is the resistance polarisation method. There is a linear relationship between potential and applied current at potentials only slightly shifted from the corrosion potential. Based on the kinetics of electrochemical reactions and the concept of the mixed potential theory postulated by Wagner and Traud, an equation has been derived which relates quantitatively the slope of the polarisation curve in the vicinity of the corrosion potential to the corrosion current density (i_{corr}) as follows:

$$i_{corr} = \frac{ba \times bc}{2.303(ba+bc)} \times \frac{1}{R_p} = \frac{K}{R_p}$$

Where, ba = Anodic tafel slope constant
bc = Cathodic tafel slope constant
Rp = $\Delta E/\Delta I$ = Polarisation resistance

This principle can be applied for estimating corrosion rate (i_{corr}) of rebars embedded in concrete.

In this technique, a small amount of D.C. current (ΔI) is applied to the embedded rebar and the corresponding potential (ΔE) is monitored. This is called polarisation and this can be carried out from -10 mV to +10 mV in the vicinity of open circuit potential (OCP). There are three methods to carryout this polarisation.

(i) Galvano-static method

(ii) Potentio-static method

(iii) Potentio-dynamic method

(i) Galvano-static method

By applying a small increment of current, the change in potential is monitored. For each increment of current a waiting time of 10 minutes is necessary in order to obtain corresponding stabilised (ΔE) values.

(ii) Potentio-static method

By applying a small increment of potential, the change in current is measured for each increment of potential (ΔE), the current value (ΔI), is recorded after 30-60 seconds.

(iii) Potentio-dynamic method

By using potentiostat coupled with voltage scan generator, the polarisation can be carried out at a particular sweep rate. The best result can be obtained at the scan rate of 5-10 mV/min.

Polarisation can be carried out by any one of the above methods and E vs. I plot obtained. From this plot, Rp value can be calculated, which is the slope of the curve near zero current.

Limitations

In actual field measurements, the working electrode, viz, steel reinforcement grid is very large, compared to the auxilary electrode. It poses some problems with regard to area of influence.

As the resistivity of the concrete is very high, Ohmic drop (IR) between the reference and working electrodes should be eliminated. The equipment used to carryout these measurements should hence have provisions for IR compensation. If IR is not compensated, Rp value will be overestimated and calculated i_{corr} will be smaller than the real one.

By the use of assumed 'B' values in calculating corrosion current density, corrosion rate of steel in concrete with different compositions and exposed to different environmental conditions may yield misleading results.

Corrosion reactions are usually very slow, i.e., they have large time constants and field measurements require considerable measurement period. Further, in field application, the electronic equipment may get damaged due to the mechanical vibrations generated during transportation between laboratory and the field.

Field application

Escalaute measured the corrosion rate using computerised model on three different bridges constructed at different periods[131]. The results are presented in Table 3.33.1. From the studies it has been concluded that it is difficult to assess the reliablity or accuracy of data obtained. However, the results can be compared with the visual appearance of the bridge deck in the immediate vicinity of the electrochemical measurements.

Cigna used this technique for measuring on-site corrosion rate in a viaduct[132]. From the measurements, it was concluded that the measured corrosion rates were probably too high because the technique mainly depended upon the local aggressivity of the environment.

Table 3.33.1. On-site Corrosion Rate on Three Bridges Located at Frederick County, USA

Bridge	Average corrosion rate (mdd)	Visual observation
54 years old	0.5	Small cracks randomly scattered over the surface of the deck
17 years old	1.1	Crack free
12 years old	1.9	Cracking of concrete, rebar exposed

mdd = Milligrams per square decimeter per day

3.34. Impedance Technique

Principle

In recent years, A.C. impedance spectroscopy is being experimented as an useful non-destructive technique for quantifying corrosion of steel rebars embedded in concrete. Impedance technique offers tremendous advantage for analysing the reaction mechanisms occurring on the corroding interfaces. In this technique, an A.C. signal is applied to the embedded rebar and the response is monitored in terms of the phase shift of the current and voltage components and their amplitude. This is done in the time or frequency domain using a spectrum or frequency response analyser. The equivalent circuit of a corroding reinforcing bar embedded in concrete can be represented as shown in Fig. 3.34.1.

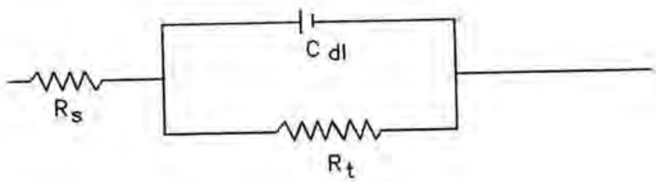


Fig. 3.34.1. Equivalent Circuit of a Corroding Rebar Embedded in Concrete

R_s is the resistance of concrete
C_{dl} is double layer capacitance, and
R_t is the charge transfer resistance

Impedance z is the ratio of A.C. voltage to A.C. current. An alternating voltage of about 10 to 20 mV is applied to the rebar and the resultant current and phase angle are measured for various frequencies.

As per the circuit, the cell impedance,

$$|Z| = R_s + \frac{R_t}{1 + jw\,C_{dl}\,R_t} \qquad \text{where, } w = 2\,\pi f \text{ and } j = -1$$

As W - >O, the cell impedance $|Z| = Rs + Rt$ and as W - >α the cell impedance $|Z| = Rs$. Thus, by subtraction of cell impedance z at high frequency from that of low frequency gives Rt. Corrosion current i_{corr} can be calculated from Rt using well known Stern Geary equation.

$$i_{corr} = \frac{ba \times bc}{2.303(ba+bc)} \times \frac{1}{R_t},$$

where, ba the anodic tafel slope and bc, the cathodic tafel slope are to be determined experimentally.

From practical situation, it may be possible to get Rt from just two measurements, one at high frequency and another at low frequency. The Nyquist plot for steel in concrete is shown in Fig. 3.34.2. Typical impedance plots are shown in Fig. 3.34.3.

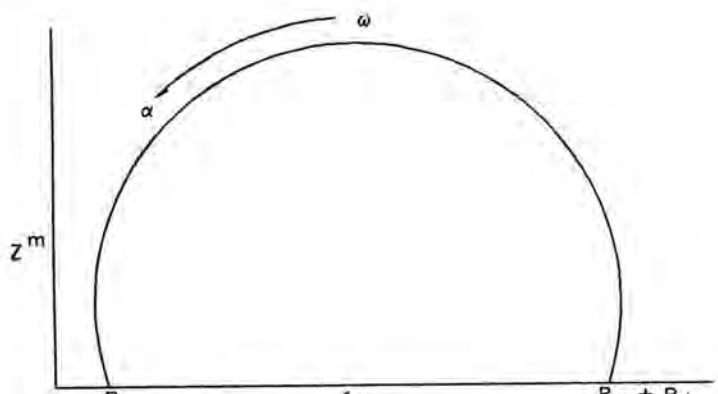


Fig. 3.34.2. Nyquist Plot for Steel in Concrete

Specifications

Impedance measuring system usually consists of a frequency response analyser (FRA), potentiostat, and a micro-computer (CPU). Data can be stored on a floppy disk.

Limitations

Accessibility of rebar network and interference effects may lead to practical problems. The most difficult part is of obtaining the true Rt value at each location of the probe sensor since the degree of polarisation induced on the rebar gradually decreases with the distance from the position of the counter electrode. Thus, the main problem is the irregular distribution of the electrical signal applied with a counter electrode of much smaller dimension compared to that of the structure.

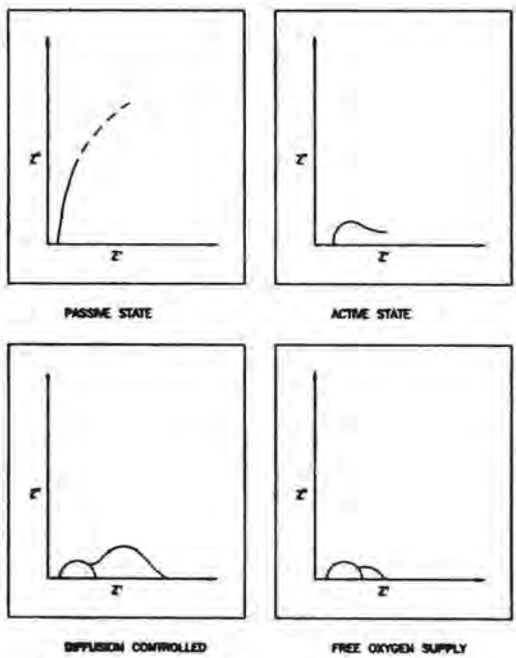


Fig. 3.34.3. Typical Impedance Plots

Eventhough this technique is fairly reproducible its usefulness becomes limited if the rebar is essentially passive. If the rebar is in active condition then only the well developed impedance data are obtained. Measurements at low frequencies make this technique time consuming.

3.35. Electrochemical Noise Analysis

Principle

Electrochemical noise technique is an emerging technique for monitoring corrosion of reinforced concrete structures. This technique enables information on the mechanism and rate of corrosion processes at areas identified in concrete structures. A low amplitude variation of the corrosion potential (range of 1mV to 10mV, 10 μHz to 1 Hz) of steel in concrete is measured to obtain a noise data as a record of potential fluctuations in the form of power spectra.

Noise source is located within the probable corroding area. A time record of sufficient interval is monitored over the frequency range (10 μHz to 1 Hz). Noise data as a record of potential fluctuation is obtained. Noise signal is transformed from time domain to frequency domain displaying in the form of amplitude and frequency based on either fast fourier transform or maximum entropy method of spectral analysis. The measurement interval is usually between 2-10 seconds depending upon the frequency range applied to the specimen. The block diagram of noise analysis is shown in Fig. 3.35.1.

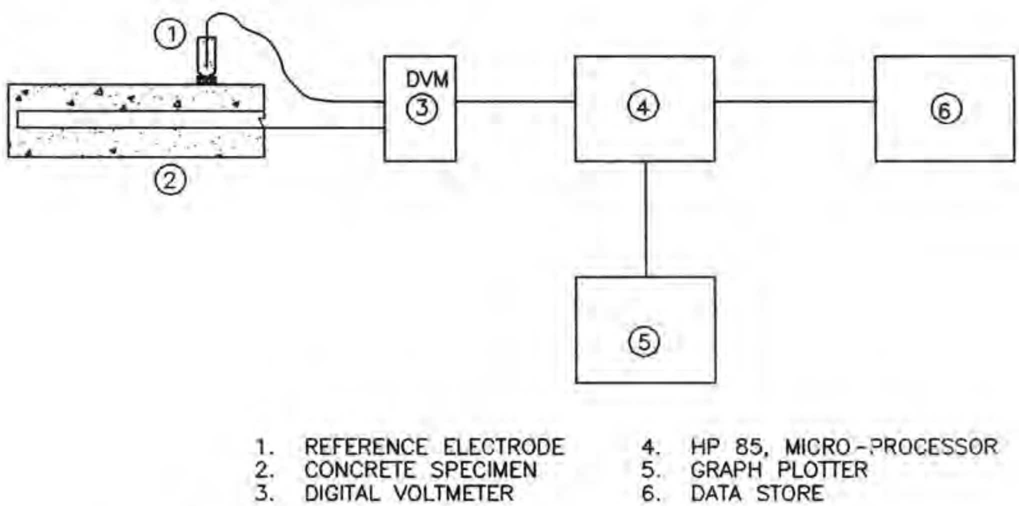


Fig. 3.35.1. Block Diagram of Noise Measuring Instrumentation

Specifications

For noise analysis technique following equipments are needed:

1. Solartron 7055
2. Digital voltmeter
3. Data logger (Potential time recorder)
4. Hewlett Packard HP-85 Micro Computer or an equivalent unit "Solartron 1200 Signal Processor" which combines the above 4 individual units.

Limitations

Since fluctuations are in microvolt range, a highly sensitive equipment is necessary.

Field investigations

Electrochemical noise technique was applied to galvanised prestressed wires in a prestressed concrete retaining wall. Noise data indicated that the attack was localized which may be due to break down of galvanized layer. This observation was consistent with the pitting attack visually assessed.

3.36. Resistivity of Concrete

Principle

Electrical resistivity of concrete is an important parameter which can be related to various other aspects, such as, strength, porosity, deterioration, etc. It is well known that the reinforcing steel embedded in concrete is protected by the concrete cover and that this protection is mainly due to the higher alkalinity and the fairly high electrical resistance of concrete. During any corrosion process, corrosion current has to flow from anode to the cathode site through the electrolyte and the resistivity of the concrete has an influence on the flow of this corrosion current. In the case of reinforced concrete structures, the high electrical resistance can impede the flow of such currents. However, resistivity of concrete has been found to vary considerably depending on the moisture content and other soluble salts present in the concrete. Hence, depending upon the resistivity of concrete, the corrosion process can be stiffled or accelerated.

It can be seen from Fig. 3.36.1 that four metallic probes are placed over the concrete surface at an equal spacing of "a". A known current "I" is impressed, on the outer probes and the resulting potential drop "V" between the inner probes is measured. The influence of current flow is such that it spreads out vertically and horizontally and takes the shape of hemispherical surface

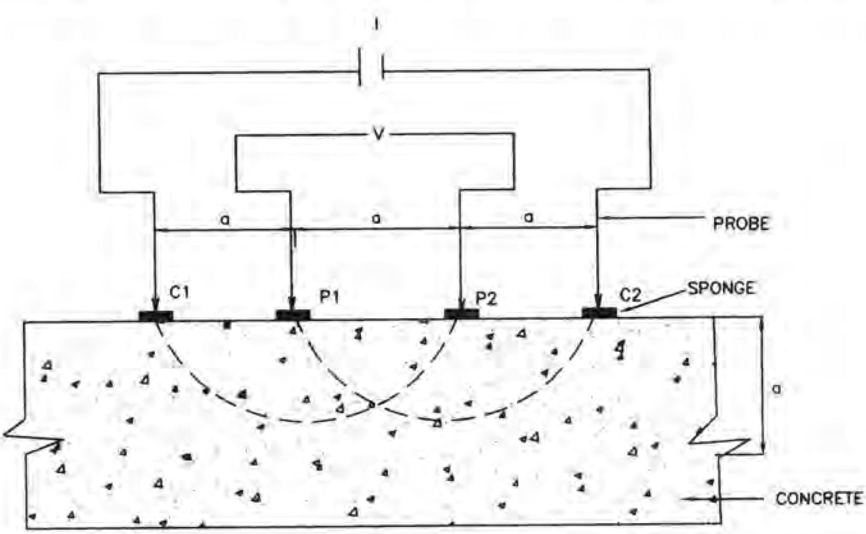


Fig. 3.36.1. Circuit of FPR Meter

and depth of its influence is proportional to the distance "a" between the probes. Resistance "R" is given by V/I. The equation relating resistivity to measured resistance has been derived from 4 probe method[127].

Resistivity of concrete (P) = $2\ \pi\, a\, R$

Where, a = Inter-electrode distance in cm
R = Measured resistance in ohm

The tips of the 4 probes of the resistivity meter are wrapped with sponge and saturated with potable water for making effective contact with the concrete surface. The four probes when pressed against the concrete surface indicates the resistivity of concrete directly on the digital panel provided in the meter as shown in Fig. 3.36.2. This meter gives the average resistivity of concrete in the cover portion provided that the inter electrode spacing "a" is less than or equal to cover thickness. If "a" is more than the cover thickness value, then one has to take care to avoid interference effect from steel reinforcements.

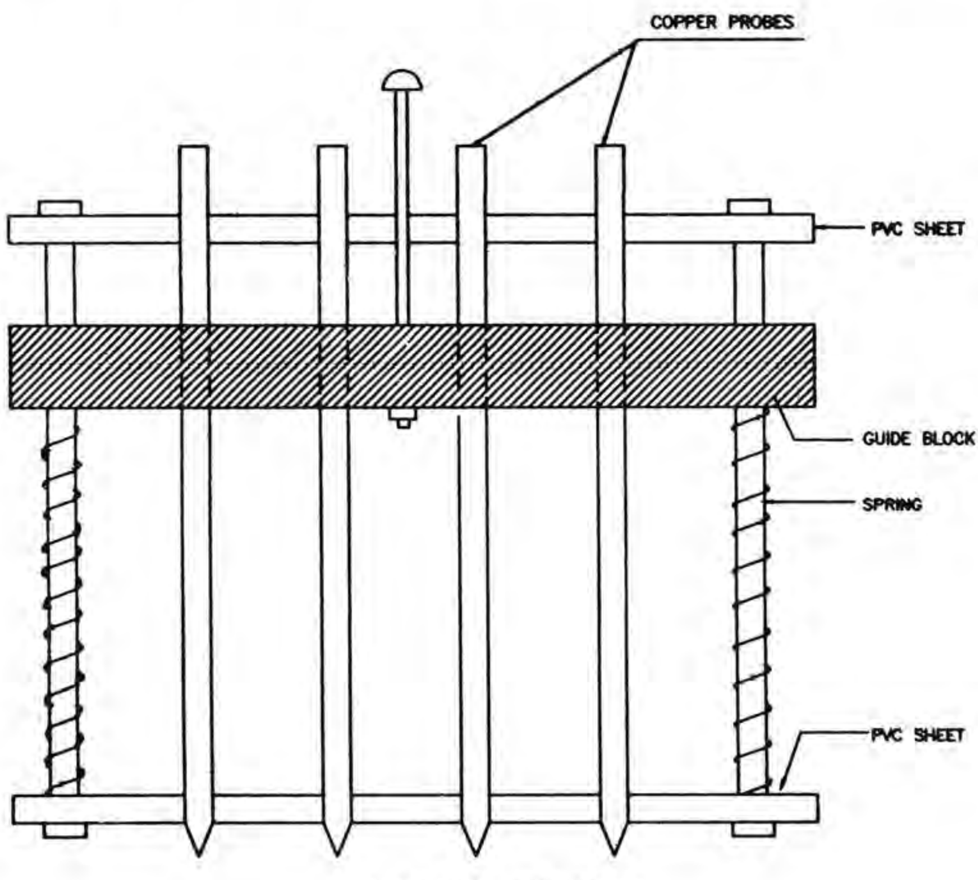


Fig. 3.36.2. FPR Meter

Specifications

A four probe resistivity meter[12] developed by CECRI is employed for monitoring resistivity of concrete structure[136], (Fig. 3.36.3). The instrument consists of two units:

Fig. 3.36.3. Four Probe Resistivity Meter for Monitoring Resistivity of Concrete Structures

(a) Four Probe Unit:

This unit is provided with tips wrapped with sponge or cotton wetted to have contact with concrete when pressed against its surface.

(b) An Electric Circuit Board:

This system converts current voltage data in terms of resistivity and scaling of the data to display the resistivity directly. Resistivity meter has also been developed in the UK.

Limitations

Correlation between resistivity of concrete with deterioration of concrete and corrosion of steel in concrete is not fully established. Porous concrete can also give high resistivity. This necessitates careful interpretation of the obtained data.

Field applications

An investigation of Scan Mateo Hayward Bridge, California revealed that amount of concrete cracking occurring in the structure varied with resistivity of concrete[125]. Electrical resistivity measurements made on concrete beams of 12 different spans revealed that at zero deterioration or when concrete is uncracked, resistivity value was more than 60 K-ohm-cm[137]. It has also been reported that the resistivity of concrete decreases as the deterioration increases.

Resistivity studies carried out by CECRI on bridge structures revealed that resistivity values of concrete in a distressed bridge structure, "A" was on the lower side in the range 13-36 K-ohm-cm indicating the poor quality of concrete. In another bridge "B" which was apparently under good condition, resistivity values were on the higher side 700-800 K-ohm-cm indicating 0 per cent deterioration.

Electrical resistivity technique was tried by CECRI, as a tool for studying the porosity of concrete structures. Electrical resistivity of such porous concrete will be very high under dry condition, "ρ dry" because of pores. When saturated with water, pores are filled with water and hence resistivity ρ wet will be very low[138]. Higher ρ dry/ρ wet ratio indicates higher porosity of concrete. Typical resistivity ratios obtained from RCC column under good and deteriorated conditions are shown in Table 3.36.1.

Table 3.36.1. Resistivity Ratio Values Obtained from Existing Concrete Structures

Details	ρ dry	ρ wet	ρ dry /ρ wet
RCC column (good condition)	90±5	55±5	1.65
RCC electric post (highly deteriorated)	58±13	6.5±1.5	8.92

This clearly brings out that the ratio ρ dry/ρ wet is about 8 times higher for deteriorated column when compared to undeteriorated column. Thus, ρ dry/ρ wet ratio gives an idea about the porosity of concrete.

This technique can also be used to monitor the performance of coating on concrete surface[139]. A typical evaluation data is presented in Table 3.36.2.

Table 3.36.2. Electrical Resistivity of Coated Concrete

Sl.No.	Paint system	Before test	After 70 days of alternate wetting of 3 per cent NaCl solution and drying
1.	Control without coating	30±9	4±2
2.	Paint System 1	77.5±35.5	33±16
3.	Paint System 2	49.5±28.5	50±28
4.	Paint System 3	10±6	47±19
5.	Paint System 4	16.5±11.5	57±27
6.	Paint System 5	12.5±7.5	50±29

3.37. Electrical Resistance Probe Technique

Principle

Electrical resistance probe technique has been used for monitoring the corrosion of mild steel element embedded in concrete. The method is based on the fact that when a conductor corrodes the metal lost is replaced by an insoluble non-conducting film which adheres to the metal or is carried away by the corrosive medium. Metals and alloys have much lower specific electrical resistances than their corrosion products. Since the electrical resistance of a metal depends on its cross-sectional area, a decrease in thickness of a specimen due to uniform corrosion may be evaluated.

In the case of a cylinderical rod/wire, electrical resistance 'R' is related to the cross section by the equation,

$$R = \rho\rho l/\pi r^2 \qquad \ldots.1$$

where, ρ is the specific resistance, l is the length of the wire and r is the radius of the wire. If corrosion is expressed as a percentage then

$$\text{Per cent Corrosion} = \frac{r_o - r}{r_o} \times 100 \qquad \ldots.2$$

where, r_o is the initial radius and r is the final radius of wire element.

By combining equations (1) and (2), one gets,

$$\text{Per cent Corrosion} \quad = \quad 100 \times \left[1 - \sqrt{\frac{R_o - R_u}{R - R_u}} \right]$$

where, R_o is the initial resistance
R_u is the fixed resistance of unexposed portion
R is the resistance at any given time.

It is apparent that in using resistance probes one must balance sensitivity with length of the test. In order to achieve increased sensitivity, it is convenient to use thin wire or ribbon elements.

Suitable locations for installation of probes are first selected by the concerned bridge authorities. Probes are normally installed prior to concreting after completion of all formwork and after laying of steel reinforcement network. The probe is kept in position and tied up with reinforcement rod to avoid any dislocation (Fig. 3.37.1). Since the probe is a representative element for steel reinforcement, the exposed element of the probe is kept as close as possible to the reinforcement and the same cover is maintained.

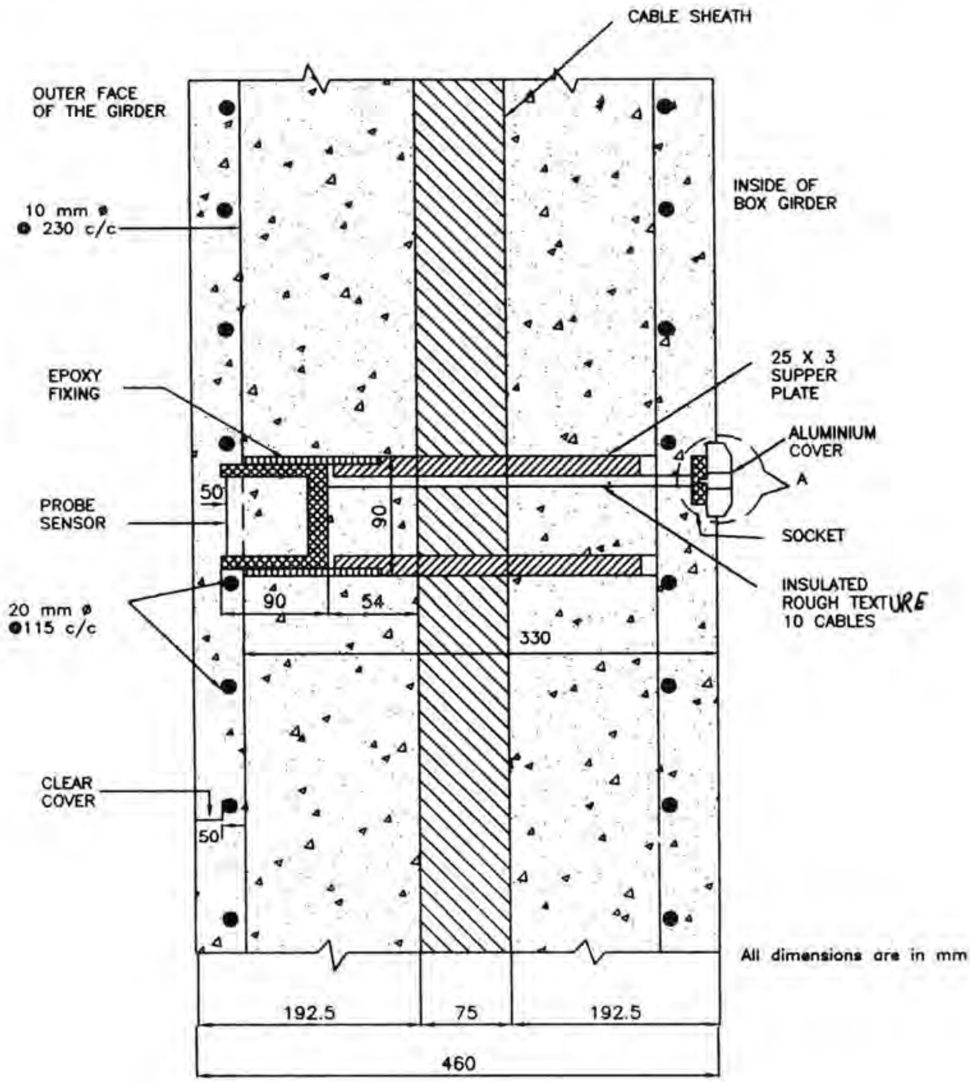


Fig. 3.37.1. Plan showing the Position of Corrosion Monitoring Probes and Sockets to be Installed in Web Portion of the Box Girder

The exposed probe element thus, represents a steel reinforcement rod of smaller diameter but exposed to the same concrete medium at a specific location. A protected PVC cable brings the signal from the probe. The changes in resistance of the exposed element with respect to the protected reinforced element is measured by a transmitter and sent to a centralised remote location for further processing. Thus, a number of probes installed in various parts of the bridge structure can be centrally monitored. The system uses a micro-processor and all the signals are handled in digits.

Specifications

For electrical resistance probe technique following equipments are needed:

(i) Corrosion monitoring probe

(ii) Corrosion monitor

CECRI carried out detailed studies on various probe designs and worked out an optimum probe design which can be conveniently used in reinforced concrete bridges as digital corrosion monitor[140,141]. In this design, one section of the probe is protected from the environment by encasement in epoxy resin while the other section is exposed to the same concrete environment as that of the reinforcement.

If d_p and d_e are the diameters of the protected and exposed elements respectively, then the resistance of protected portion,

$$Rp = K/(d_p)^2 \qquad \ldots.3$$

resistance of the exposed portion,

$$Re = K/(d_e)^2 \qquad \ldots.4$$

combining (3) and (4)

$$\frac{d_e}{d_p} = \sqrt{\frac{R_p}{R_e}} \qquad \ldots.5$$

$$\text{Therefore, per cent reduction in diameter} = 100 \times \left[1- \sqrt{\frac{R_p}{R_e}} \right] 1 \qquad \ldots.6$$

Thus, by measuring the resistance R_p and R_e, the percentage reduction in diameter due to corrosion can be obtained.

CECRI has developed a digital corrosion monitor for measuring the percentage reduction in diameter.

Limitations

(1) Accuracy of measurement depends on the form of corrosion. If it is uniform corrosion then fairly accurate value can be obtained.

(2) Installation of probes needs to be done skillfully.

(3) Since the probe gives the corrosion data for the specified location where it is installed, number of probes are to be installed in carefully selected locations.

(4) This technique is not suitable for existing structures.

Field applications

CECRI initially installed some experimental probes in few girders of Pamban Bridge, Tamilnadu and monitored its performance for a period of 18 months. It has also installed corrosion monitoring probes in one instrumented span of each of the new and recommissioned old Mandovi Bridges, Goa. Typical installation of corrosion monitoring probe is shown in Fig. 3.37.2.

In other countries, electrical resistance probes are being extensively utilised for monitoring cathodic protection of steel in concrete[134].

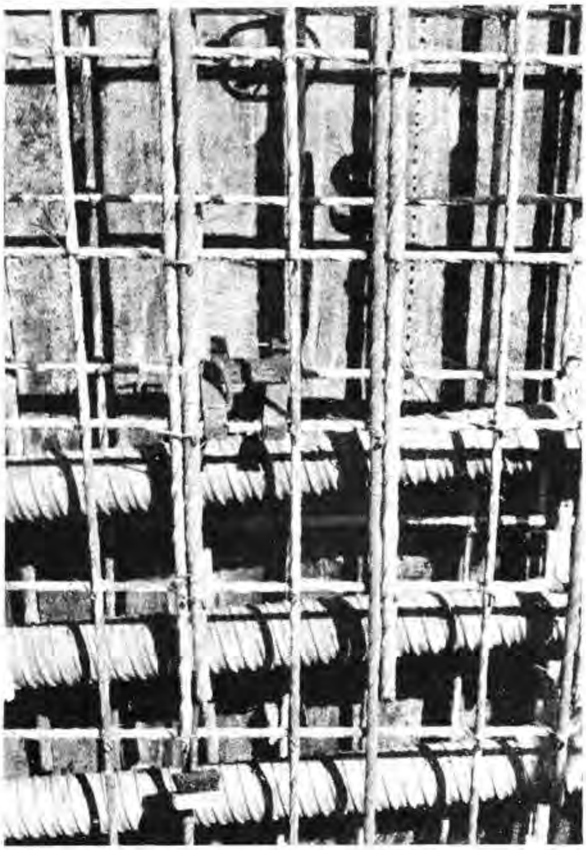


Fig. 3.37.2. Typical Installation of Corrosion Monitoring Probe

3.38. Electrical Resistance Technique for Prestressing Steel

Principle

A critical examination of the available literature shows that no fool-proof technique is available to monitor the condition of prestressing steel wires in prestressed concrete bridges. Corrosion damages in the prestressing cable can be quantitatively assessed by making direct electrical resistance measurements. Electrical resistance is an important parameter of any metallic material and in the case of prestressing cables, the resistance R of a cable of length L is given by the relation,

$$R = \rho L/A$$

where, ρ is the resistivity of the metal, A is the cross sectional area of the cable.

From this it is clear that the resistance of a known length of a cable depends only on the cross sectional area of the cable, the resistivity being constant for the particular material. This fact could be exploited to monitor corrosion of the cables as the area of cross-section reduces due to corrosion, thus, giving a higher resistance value than the original value. Hence, an increase in resistance with time is indicative of the progress of corrosion. The resistance value measured can be compared with the initial resistance R_T which is calculated with the known initial diameter. By periodic measurements of resistance values, corrosion rate is calculated. Percentage reduction due to corrosion can be arrived at after actually measuring the resistance, Rm.

$$\text{Per cent reduction } = 100 \times \left[1 - \sqrt{\frac{R_T}{R_m}} \right]$$

The technique to be made use of in measuring the resistance of the cable is the four probe technique where the contact resistance is eliminated and the resistance of the connecting wires is not included in the measurements.

Specifications

For electrical resistance measurement technique following equipments are needed:

(1) Micro ohm meter

(2) Multimeter

CECRI has developed a portable battery powered instrument specifically for cable resistance measurement[144]. The instrument consists of a constant current source, a differential amplifier and a LC display. The system uses four probe technique. Alternatively, any suitable resistance meter can be used.

Limitations

Eventhough this technique looks very simple, lot of care needs to be taken while making the measurements. Cable ends are to be made accessible for making electrical connections. In the existing bridge, this may be possible only in those cables which are anchored at the deck. If the corrosion is highly localised then this technique will not be able to indicate the exact condition of the prestressing cable. Inspite of all these limitations and uncertainty factors, this technique can still provide some information about the condition of the prestressing steel. Instead of one time measurement, it is desirable to take these measurements periodically. The electrical circuit for the initial condition of the prestressing strands, cable sheets, anchor plates, etc. should be precisely established. In addition to this, influence of cement grout is also to be considered. Another point to be noted is that only under stressed and locking condition, perfect electrical short circuiting is established, thereby, making it possible to work out the initial electrical circuitry.

The data collected during monitoring needs to be interpreted with due precautions and expertise.

Field applications

CECRI has applied this technique on a few bridges in the country[136]. In bridge "A" which was a new bridge under construction, the cable under measurement should have a theoretical resistance value R_T of 14.2 milliohms. The measured resistance R_m, which was measured in that girder just after prestressing was found to be 14.7 milliohms, indicating that "R_m" in a new girder is in good agreement with "R_T".

In bridge "B" which was worst affected, cable 1 with a R_T value of 33.9 milliohms showed a R_m value of 43 milliohms after 16 years. The actual diameter of the cable was measured to be 6 mm at a visible location instead of 7 mm. Similarly, cable 2 with a R_T value of 35.8 milliohms showed a R_m value of 70 milliohms after 16 years of construction and the actual diameter of this cable measured to be 4.7 mm at a visible location instead of 7 mm.

Currently, CECRI is engaged in monitoring of one span each of new and recommissioned old Mandovi Bridges, Goa (Fig. 3.38.1).

3.39. Integrity Testing of Concrete Piles[155,157,158]

Principle

It is a method of investigating soundness of a pile with respect to continuity, shape, dimensions, cross-sectional variations and quality of concrete used in the piles. Need for integrity testing to assure quality of piles shortly after their construction and prior to laying caps using a definite and economical non-destructive procedure has been emphasised from time to time. Various non-destructive integrity testing techniques, e.g., sonic methods, vibration methods, sonic logging technique, etc. have been tried during the past 15 to 20 years in different parts of the world. However, the method based on one dimensional stress wave approach known as sonic integrity testing or low strain integrity testing or sonic echo testing has been used successfully, in various parts of the world for investigating structural integrity of cast-in-situ as well as precast concrete piles. The method is simple and quick enabling dozen of piles to be examined in a single working day, without disturbing site activities.

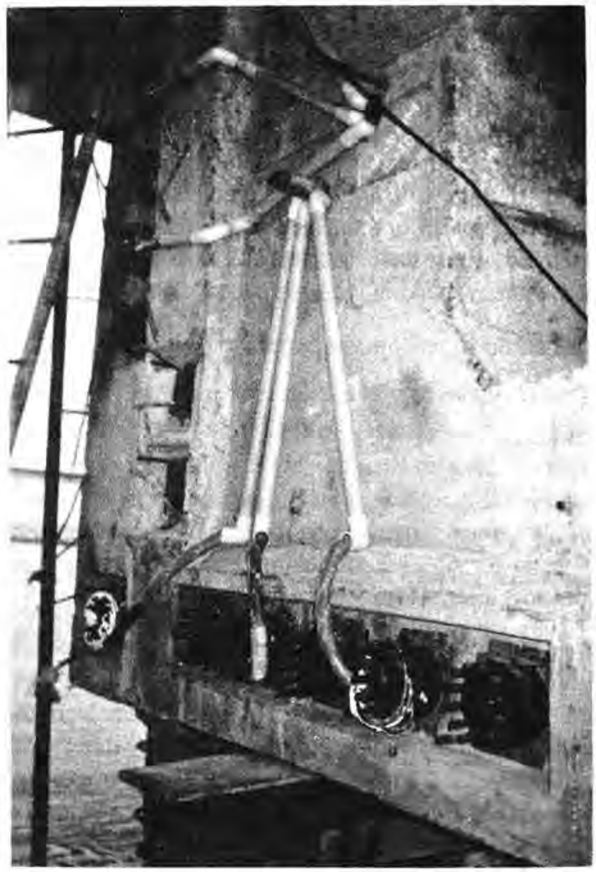


Fig. 3.38.1. Various Arrangements for Cable Resistance Measurements

The test provides information about continuity of installed piles, defects, such as, cracks, necking, soil incursions, changes in cross-section and approximate pile length unless the pile is very long or its skin friction is too high.

The test is conducted by striking the pile head with a small hammer. The transmitted force generates a compression wave that travels down the pile shaft. This wave is reflected from the location of any defect, level of changes in cross-section and is picked up by an accelerometer held at the pile top close to the location of hammer impact. The reflected stress wave can be monitored using either analogue or digital data processing technique. In digital data processing technique, the observed signals are amplified and converted into digital display as velocity versus length records which provides input for the integrity testing. The stress wave velocity is dependent on the Young's modulus and mass density of pile concrete.

Schematic conceptual diagram alongwith a typical record obtained for structurally sound bored cast-in-situ concrete pile is depicted in Fig. 3.39.1[158]. It is a time-domain reflectometry, wherein the pile acts as a one dimensional medium in which a wave is propagated. The wave is originated by a short hammer blow to the head of pile in the axial direction. It travels with the

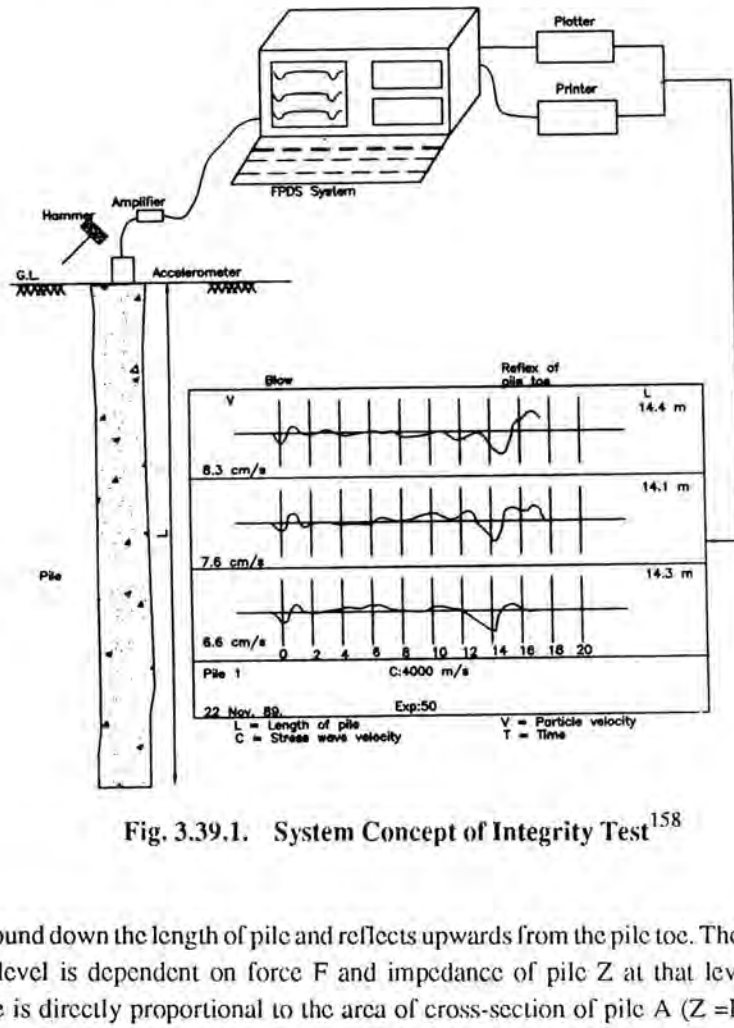


Fig. 3.39.1. System Concept of Integrity Test[158]

speed of sound down the length of pile and reflects upwards from the pile toe. The partical velocity V at any level is dependent on force F and impedance of pile Z at that level (V=F/Z). The impedance is directly proportional to the area of cross-section of pile A ($Z = EA/C = A\sqrt{E\rho}$, C = Stress wave velocity, E = Young's modulus and $\rho = \lambda$ density of pile material) and any change in it. If on the way down, variations in the impedance of pile as a result of increase or decrease in cross section, cracks, inclusions of foreign material and skin friction are met, a part of the wave reflects from this impedance variation and returns to the pile top. Figs. 3.39.2 and 3.39.3[155] represent the velocity-time reflectograms for various types of piles. The reflected wave signal

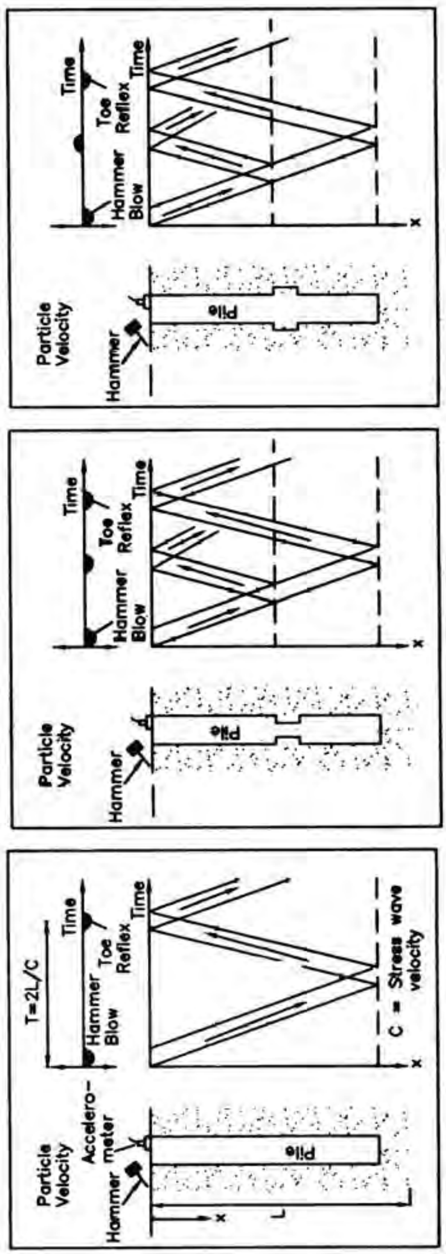


Fig. 3.39.2. Stress Wave Pattern of Reflections[155]

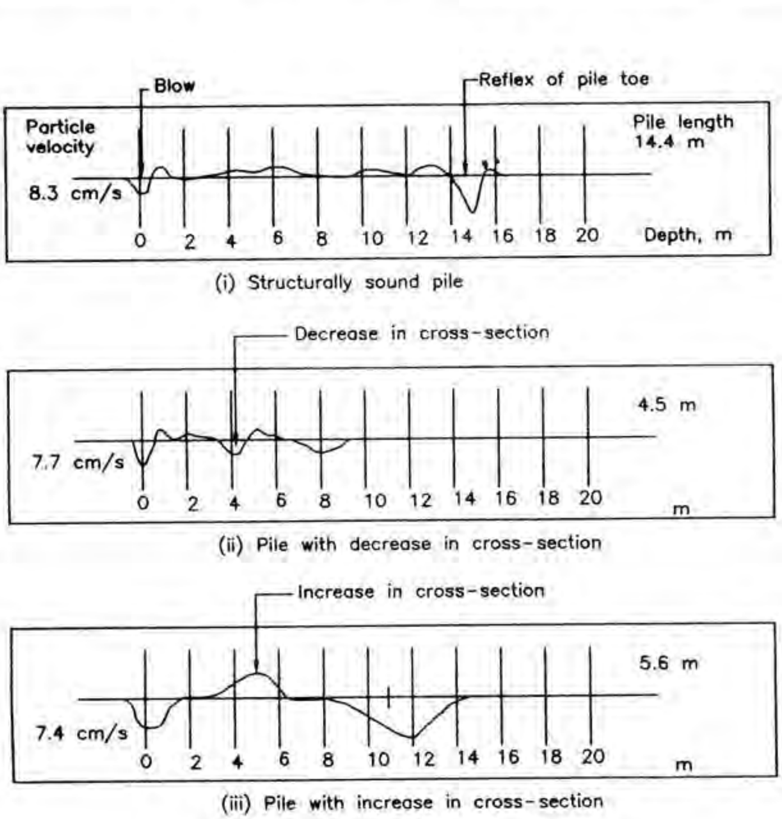


Fig. 3.39.3. Typical Integrity Test Results[155]

picked up by the accelerometer is amplified, converted into digital form and displayed on screen of the equipment as velocity-time record (time is converted into length scale by $T = 2L/C$, where L is the length of pile, C is stress wave velocity and T is time) providing information about defects. These signals are stored in computer for subsequent analysis. Several commercial standard computer programme packages are available. For making a quantitative estimate of the exact locations and dimensions of defects, the results are processed using suitable computer programme[159,160].

The pile driving analysis test (PDA) is conducted for the following :

(i) Quality control of the pile driving process

(ii) Check on stress levels in the pile material

(iii) Evaluation of performance of pile driving hammer

(iv) Resistance offered to penetration by the bearing layers

(v) Check on integrity of piles during driving

(vi) Collection of data for prediction of driveability of piles to be driven later

The test is conducted by using specially designed combined acceleration and strain transducers for measuring velocity and force. The system concept for the test is shown in Fig. 3.39.4[157]. Two sensors are mounted on sides of pile opposite to each other near the pile top. The

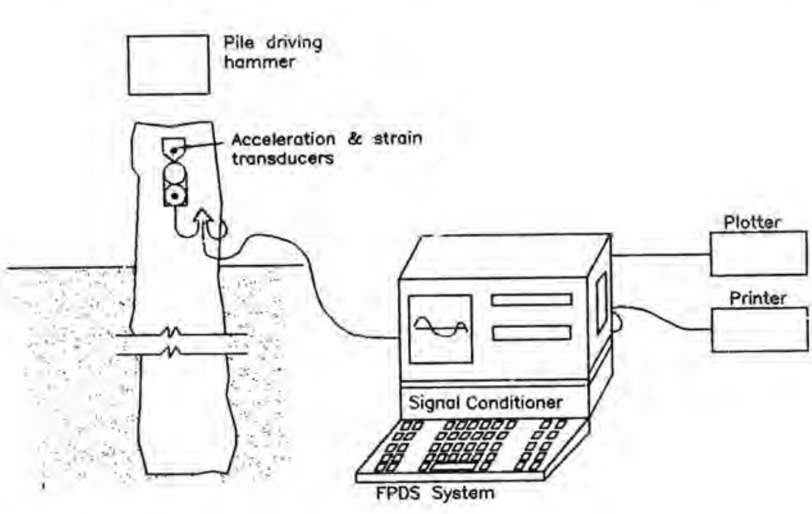


Fig. 3.39.4. System Concept for PDA/DLT[157]

signals obtained through the mounted transducers during driving are passed on for processing to the signal conditioning sub-system and then to the computer. The processed signals for each blow are stored in the hard disk. The signals and other information are available immediately on the screen as a function of time, scaled in engineering units. On completion of pile driving a full record of all the relevant parameters, such as, blow count, blow rate, maximum compression and maximum tension in pile, driving resistance, etc. are available.

The dynamic load test on piles is conducted to predict static bearing capacity of a pile by means of dynamic test. The test is performed by giving an impact load on the head of pile already driven or cast in the ground. The system concept is the same as for pile driving analysis test as shown in Fig. 3.39.4[157]. A guided drop weight is used to strike the head of pile. For this test precast concrete piles or steel piles, need no further preparation except that their heads are intact. In case of cast-in-place piles the top is levelled and a heavy steel plate is fixed on the pile top with anchor bolts. The gap is filled with a quick hardening resin. If normal pile driving rig is not available, a drop weight having a central hole to accommodate a guide rod fixed in the centre of plate is used. The test is normally done after a lapse of adequate time required for soil set up which is disturbed during the pile installation. The signals are obtained and processed in this test in the same way as in the PDA test. However, with dynamic load testing also a signal matching technique[157] is involved.

Specifications

The head of pile where test is to be performed should be readily accessible and free from standing water. It should be free from laitence, loose concrete, etc. In case of working piles, it can be trimmed back and levelled as far as practicable.

The test should be carried out generally after 14 days of pile installation. In no case, this test should be performed prior to 10 days have elapsed since casting of piles in case of cast-in-situ piles. In case of precast concrete piles, this period may be reduced to 4 to 5 days after installation. The test should generally be carried out on all the piles where feasible.

The hand held hammer in integrity test generates accelerations in the 10 to 100 g range, pile strains around 10^{-3}, velocities near 3 cm/s and displacements less than 0.0025 cm. The velocities contained the most useful and usable information[159].

Limitations

The method does not identify all the imperfections in a pile, but indicates faults existing within the effective length evaluated by the test. The method of testing involves high skill and use of computerised equipment, and the test needs to be performed by experienced personnel.

There are some situations where the integrity tests is partially ineffective[159]. Firstly, reflections from mechanical splices in concrete driven piles are usually present because of physical gap (discontinuity). Therefore, records of mechanical spliced piles may not show pile toe reflections as the pile's integrity can only be checked down to the first splice. Pile with greatly varying cross-sectional area may make it difficult to distinguish between reflections from significant discontinuities and those caused by the construction method. Also, because the impact wave is dampened out by high skin friction, the pile embedment limits the ability to obtain clearly recognisable pile toe reflection.

Calibration

It is important that the results of integrity testing be interpreted by the personnel of requisite specialist experience. Integrity testing may also identify minor defects which will not necessarily affect pile performance and the specialist will have to exercise his judgement as to the acceptability or otherwise of such a pile.

The stress wave velocity value generally lies between 3000 m/s and 4000 m/s depending on the grade of concrete used (M15 to M25). Normally, three records are taken and the repeatability in signals is achieved. Sometimes averaging of signals is also done to achieve more informative signals. The test is to be repeated at two to three places on a pile, particularly for the suspected pile. In case of large diameter piles, the test should be conducted at 4 to 5 places to cover entire section of the pile.

The tests should be conducted initially on test piles along with few other piles whose length is correctly recorded, to determine the probable value of stress wave velocity and characteristic or reference signal for comparing the observed signals of piles to be tested later at that site.

A clearly indicated toe signal together with a fairly steady velocity trace between the impact and toe signal are signs of a sound pile. Traces with strong variations may indicate the presence of a pile cross-section change or soil resistance changes.

References of their applications

Integrity tests have been reported[157] on bored cast-in-situ concrete piles of 375 mm, 400 mm, 500 mm and 760 mm diameter and depth ranging from 5.0 m to 15.0 m at four sites in and around Delhi. The PDA tests have been reported on precast concrete piles of 400 mm x 400 mm cross-section and 22 m deep (single length) and about 34 m deep (two segments) at Kakinada to monitor performance of piles during driving.

Integrity tests have been reported [158] on (i) especially cast bored concrete defective piles, (ii) bored cast-in-situ concrete production piles, (iii) driven cast-in-situ concrete production piles, and (iv) precast concrete production piles, at various sites covering different types of soil deposits.

3.40. Visual Inspection

Visual inspection helps to identify the common symptoms of distresses, such as, location and pattern of cracks, spalling, honeycombing, leaching in concrete, corrosion stains on concrete surface, spots having exposed reinforcement besides condition of bearings, expansion joints and wearing coat.

Convenient access to the area under inspection and understanding of the behaviour of the structure are, however, the essential requisites for meaningful inspection.

Following are the possible accesses[154]

- Manholes in the deck
- Access ladder to the pier top
- Catwalks
- Platform gantry
- Equipment operating under the bridge from boat/barge
- Mobile inspection unit (snoopers)

Standard tools[154] which are useful in visual inspection are :

- Markers, pocket tapes
- Pocket knife, wirebrush, scrapper
- Spirit level, plumb bob, feeler gauges
- Flash light, binocular, magnifying glass, camera

4. CONCLUSION

i) Potential and limitations of a wide range of techniques presented above would reveal that with the increasing number of NDTs,- some very exotic or in developmental stage - there is immediate need to build up a few Nodal Centres to acquire, calibrate and promote use of NDTs in condition survey of structures - both under construction or distress and for predicting reserve strength. The appropriate NDT method on its own or in combination can be selected on the basis of the knowledge of its limitations and applicability. The required number of in-situ NDT tests for achieving accuracy comparable to compressive strength of cube can also be determined. Correlation equations and correction factors can be established on important and major projects for equipment and procedures being actually used using appropriate curve fitting techniques.

ii) User instructions for various techniques, recommended sets of instruments for quality assurance and control, and acceptability criteria for different techniques are the immediate requirements.

iii) Corresponding software to process the mass of data and an Expert System for diagnostics incorporating the input from the NDTs can encourage use of NDTs in an efficient manner.

iv) While the use of NDTs invariably leads to a mass of data, it is to be noted that it cannot replace the human judgement related to structural mechanism and global behaviour, it can only supplement this process.

REFERENCES

1. Santhakumar, A.R., Satyanarayan, S., Venkatasubramanian, C.K., and Reddy, R.N., "Semi-Destructive Method of Testing in-situ Concrete ", Indian Concrete Journal, May 1986, pp. 127- 133.

2. Hulshizer, A.J., Edgar, M.A., Daniels, R.E., Suminsby, J.D. and Myers, G.E., "Maturity Concept Proves Effective in Reducing Form Removal Time and Winter Curing Cost", SP-82-18, ACI Detroit, pp. 351-376.

3. Munday, John, G.L. and Dhir, Ravindra K., "Assessment of In-situ Concrete Quality by Core Testing", SP-82-20, ACI Detroit, pp. 393-410.

4. Neville, A.M. and Brooks, J.J., Concrete Technology, University Press, Belfast, p.319.

5. Iyer, Mahadeva Y., Rengaswamy, N.S., Rajagopalan, Srividya C., Nayak, N.U., Srinivasan, S. and Suresh, Bapb R.H., " Non-destructive Cover Thickness Measurements using Eddy Currents Principle ", The Transaction of the SAEST, Vol.23, No. 2-3, 1988.

6. Stamm, Alian,'Tensiomag', Freyssinet Magazine, No. 1,Paris, 1984.

7. Non-Destructive Testing of Concrete, (Part 1), CRI Technology Digest, August 1985, National Council of Cement and Building Materials.

8. "Methods of Diagnosing Deterioration ", Organisation for Economic Co-operation and Development, Paris, August 1989, pp. 65-86.

9. Indian Standard IS:13311 (Part I)-1992, "Methods of Non- Destructive Testing of Concrete, (Part-I), Ultrasonic Pulse Velocity", Bureau of Indian Standards, DOC -CED - 2(3889) pp. 1-9 .

10. Indian Standard IS:13311 (Part II)-1992, "Methods of Non - Destructive Testing of Concrete, (Part II), Rebound Hammer", Bureau of Indian Standards, DOC - CED - 2 (3890), pp. 1-7.

11. Bungey, John H., " Testing of Concrete in Structures", Surrey University Press.

12. * American Society for Testing of Materials, C 42- 82, "Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete", (Part 14), Philadelphia, pp. 32-35.

13. * British Standards Institution, B.S.: 1881, (Part 120), "Method for Determination of the Compressive Strength of Concrete Cores", BSI, London, 1983.

14. * British Standards Institution, B.S.:6089, "Guide to Assessment of Concrete Strength in Existing Structures", B.S.I., London, 1981.

15. * British Standards Institution, B.S.: 1881, (Part 4), "Method of Testing Concrete for Strength", BSI, London, 1970.

16. * Sangha, C.M. and Dhir, R.K., " Strength and Complete Stress -Strain Relationships for Concrete Tested in Uniaxial Compression under Different Test Conditions ", Materiaux et constructions, Vol. 5, No. 30, Nov-Dec, 1972, pp. 360-371.

17. * Meininger, R.C., " Effect of Core Diameter on Measured Concrete Strength ", Journal of Materials, Vol. 3, No.2, June 1968, pp. 320-336.

18. * Lewis, R.K., " The Compressive Strength of Concrete Cores, Commonwealth Scientific and Industrial Research Organisation, Report 1.18, Melbourne, 1972, p.15.

19. * Dhir, R.K. and Sangha, C.M., " Development and Propogation of Microcracks in Plain Concrete, Materiaux et constructions, Vol. 7, No. 7, Jan-Feb, 1974, pp. 17-23.

20. * Sangha, C.M. and Dhir, R.K., " Core-Cube Relationships of Plain Concrete ", Advances in Ready Mixed Concrete Technology, Ed. Dhir, R.K., Pergamon Press, 1976, pp. 193-202.

21. Petersons, N., " Strength of Concrete in Finished Structures ", Swedish Cement and Concrete Institute, Reprint 26, Stockholm, 1964, p.189.

22. Peterson, N., "Concrete Strength in Precast Elements", Swedish Cement and Concrete Institute, Applied Studies No. 9, Stockholm, 1973, p.74.

23. Concrete Core Testing for Strength, Technical Report No. 11, The Concrete Society, London, 1976, p.44.

24. * Bhargava, J., " Strength of Concrete Members Cast in Deep Forms ", National Swedish Institute for Building Research, Document No. 5, Stockholm, 1969, p.108.

25. * Bhargava, J., " Strength of Concrete in High Strength Revibrated Walls ", National Swedish Institute for Building and Research, Document No.6, Stockholm, 1969, p.48.

26. * Parkinson, J., " Poor Tests Blamed for Faulty Mixes", New Civil Engineer, London, 26th January, 1984, p.17.

27. * Bloem, D.L.," Concrete Strength in Structures", ACI Proceedings, Vol. 65, March 1968, pp. 176-187.

28. * Ingvarsson, H. and Wiberg, U., " Development of Ultrasonic Technology for Detection of Damage in Concrete", Project Proposal, Royal Institute of Technology, Stockholm, 1986.

29. * Leslie, J.R. and Cheesman, W.J.," An Ultrasonic Method of Studying Deterioration and Cracking of Concrete Structures", Journal of American Concrete Institute, Proceedings, Vol. 46, No.1, September 1949, pp. 17-36.

30. * Naik, T.R., " Ultrasonic Testing to Determine Concrete Properties", Engg. Methods in Concrete Structures for Practitoners, American Concrete Institute, ASCE, 1979.

31. * Malhotra, V.M., " Testing Hardened Concrete: Non-destructive Methods", American Concrete Institute, Monograph No. 9, 1976.

32. * Jones, R. & Facaoaru, I., " Recommendations for Testing Concrete by the Ultrasonic Pulse Method ", Materials and Structures / Research and Testing, Vol. 2, No. 10, Paris, July-August 1969, pp.275-284.

33. * Spellman, D.L. and Stratfull, R.F., " An Electrical Method for Evaluating Bridge Deck Coatings", Highway Research Record No. 357, Washington DC, 1971, pp. 64-71.

34. * Champion, M. and Dufay, J.C., " Naissance du Scorpion, Systeme de radioscope televisee par rayonnement pour inspection des overages en biton ", Revue Generale des routes et des acrodromes., No. 589 September 1982.

35. * "Durability of Concrete Bridge Decks ", NCHRP, Synthesis of Highway Practice 57, Transportation Research Board, Washington, DC., 1979.

36. Manning, D.G. and Holt, F.B., " Detecting Deterioration in Asphalt-Covered Bridge Decks ", Concrete International, Vol. 2, No. 1, 1981.

37. * Clemena, G.G. and Mckeel, JR., W.T., " The Application of Infrared Thermography in the Detection of Delamination in Bridge Decks ", Rep. NO VEHTRC - 78 - R27, Virginia Highway and Transportation Research Council, 1977.

38. * Manning, D.G.and Holt, F.B., " Detecting Deterioration in Asphalt-Covered Bridge Decks ", Transportation Research Record No. 899, Washington DC, 1988.

39. * Alongi, A.V., Cantor. T.R., Kneeter, C.P. and Alongi J.R., " A Concrete Evaluation by Radar Theoretical Analysis ", Transportation Research Record No. 853, Washington DC, 1982.

40. * Cantor, T. and Kneter, C.," Radar and Acoustic Emission Applied to Study of Bridge Decks ", Suspension Cables and Masonary Tunnel ", Transportation Research Record No. 676, Washington DC, 1978.

41. * Steliway, W.J, Echard, J.D. and Luke, C.M., " Locating Voids Beneath Pavement Using Pulsed Electromagnetic Waves ", NCHRP Report 237, Washington DC., November 1981.

42. * Cantor, T.R. and Kneeter, C.P., " Radar as Applied to Evaluation of Bridge Decks ", Transportation Research Record No.853, Washington DC, 1982.

43. * Clemena, G.G., "Non-destructive Inspection of Overlaid Bridge Decks with Ground Penetrating Radar", Transportation Research Record No. 899, Washington DC, 1983.

44. " Equipments of Non-destructive Testing of Bridges ", GIMRET & Co., New Delhi.

45. * Peterson, C.G., "Lok Test and Capo Test Development and Their Applications", Proc.ICE, Pt 1, 76, May 1984, pp.539-549.

46. * Peterson, C.G., " In-situ Strength Testing with Lok Test and Capo Test ", The Danish Pull-out Tests ", Proc. Symp. on Modern Concrete Technology, Venezuelan Cement Producers Association, Corasas., Venezuela, 1980.

47. * Davis, R.E. et.al. " Model and Prototype Studies of Box Girder Bridge ", Journal of Structural Division, ASCE., Vol 9.8, No.STI, January 1972, pp. 251-256.

48. * Gumour, R.S. and Waison, C.K., " The Construction and Testing of a one-tenth Scale Model of a Prestressed Concrete Bridge ", The 7th Conference of the Australian Road Research Board., Vol. 7 Party, Adelaide, 1974, pp. 277 - 291.

49. * Nielsen, J. and Griffin, D.F.," Acoustic Emission of Plain Concrete ", Journal of Testing and Evaluation (JTEVA) U.S.A., 5, No. 6, Nov.77, pp. 476-483.

50. * Dunegan/ENDEVCO, 3000 Series Acoustic Emission Instrumentation System. ENDEVCO (U.K. Division), Melbourn, Royston., Herts.

51. * Malhotra, V.M., " Testing Hardened Concrete : Non-Destructive Methods, Monograph, American Concrete Institute, Detroit, 1976.

52. * Mindess, S., " Acoustic Emission and Ultrasonic Pulse Velocity of Concrete", Int. Journal of Cement Composites and Light Weight Concrete, 4, No. 3, August 1982, Construction Press, pp. 173-179.

53. Carino, N.J. Sansalone, M. and Hsu, N.N., " A Point Source - Point Receiver, Pulse Echo Technique for Flaw Detection in Concrete", ACI Journal, 83-20, March/April 1986, pp. 199-208.

54. Sansalone, M. and Carino, N.J., " Impact Echo Method ", Concrete International, Vol. 10, No.4, April 1988, pp. 38-46.

55. * Acoustic Pile Integrity Testing Automatic Logger, Instruction Manual, CNS Electronics Ltd., Holmes Rd., London, pp. 61-63.

56. * Byles, R. Dutch, " Strike a Blow for Integrity Testing " , New Civil Engineer, 11 June 1981, p.26.

57. * Smith, E.E. and Whiffin, A.C.," Density Measurement of Concrete Slabs using Gamma Radiation ", The Engineer, 194, August 1952, London, pp. 278-281.

58. * de Hass,E.,"Radio-Isotope Techniques in Concrete Research", ACI Journal Proceedings, Vol. 50, No. 10., June 1954, pp. 890-891.

59. * Simpson, J.W., " A Non-destructive Method of Measuring Concrete Density using Back Scattered Gamma Radiation ", Building Science. 3, No.1, Aug. 1968, Pergamon, Oxford, pp. 21-29.

60. Honig, A., " Radiometric Determination of the Density of Fresh Shielding Concrete in - situ", SP 82-30, ACI, Detroit, 1984, pp. 603-618.

61. Carino, Nicholas J., " Laboratory Study of Flaw Detection in Concrete by the Pulse-Echo Method ", SP 82-28, ACI, Detroit, pp 557-579.

62. * Carino, N.J. and Sansalone, M., "Pulse-Echo Method for Flaw Detection in Concrete ", Submitted for publication in ACI Journal.

63. * Krautkramer, R.L., " The Radiation Pattern from a Rotationally Symmetric Stress Source on a Semi-infinite Solid ", Part II of Ph.d. Thesis, Graduate Division of Applied Mathematics, Brown U., May 1951, pp. 1-29.

64. Ogilvy, J.A. and Temple, J.A.G., " Differaction of Elastic Waves by Cracks : Application of Time-of-Flight Inspection", Ultrasonics, V.21, No. 6., Nov. 1983, pp. 259-269.

65. Mlakar, P.F., Walker, R.E., Sullivial B.E. and Chiarito. V.P., " Acoustic Emission Behaviour of Concrete", SP 82-31, American Concrete Institute, Detroit, pp. 619-637.

66. * Browne, R.D. et. al., "Long Term Performance of Concrete in the Marine Environment", Off-shore Structures, ICE, 1974, pp. 49-60.

67. Long, A.E. and Murray, A.McC., "The Pull-off Partially Destructive Test for Concrete", SP 82-17, ACI, Detroit, pp. 327- 350.

68. * Chabowski, A.J. and Bryden Smith, D.W., " A Simple Pull- out Test to Assess the Strength of In-situ Concrete", BRE Current Paper CP 25/77, June, 1977.

69. * Denison, INc., " Windsor Probe Test System ", Technical Data Manual, Elmwood, Connnecticut, U.S.A.

70. * NORDTEST, Nordtest Method, NT Build 212, Approved, 1983-06, p.5.

71. * SWEDISH STANDARD, SS 137239, Approved, 1983-06, p.3.

72. * Carlsson, M., Eeg, I.R. and Jahren, P., " Field Experience in the Use of Break off Tester", SP 82-14, ACI, Detroit, 1984, pp. 277-292.

73. * Dahl, Jorgensen, E., and Johansen, R., " General and Specialized Use of the Break-off Concrete Strength Testing Method", SP-82-15, ACI, Detroit, 1984, pp. 293-308

74. * Christiansen, V.T., Thorpe, J.D. and Yener, M., "In-situ Testing of Concrete Using Break-off Method', Proceeding, Structural Faults and Repair-87, Engineering Technics Press, Edinburgh, 1987, pp, 101-112.

75. * Johansen, R., "In-situ Strength Evaluation of Concrete - the Break-off Method", Concrete International, Vol. 1, No. 9, Sept. 1979, pp. 45-51.

76. * Eeg., Inge R., "Use of the B-O Tester during Construction of the New Bank of Norway Building", A/S Scabcem, Slemmestad, Norway, 1983-12-12, p.3.

77. Pullen, D. A.W., Clayton Ron , " The Radiography of Swaythling Bridge", pp. 3-8.

78. Kavanagh, Barry F., Bird, Glenn S.J., "Surveying Principles and Applications", Second Edition, Prentice Hall, 1989, pp. 218- 243.

79. Cantor, T.R., "Review of Penetrating Radar as Applied to Non-destructive Evaluation of Concrete", SP 82-29, ACI, Detroit,1984, pp. 581-602.

80. Dhir, R.K., Hewlett, P.C. and Chan, Y.N., " Near Surface Characteristics of Concrete : Assessment and Development of in- situ Test Methods", Magazine of Concrete Research. Vol. 39, No. 141, December 1987.

81. * Figg, J.W., " Methods of Measuring the Air and Water Permeability of Concrete", Magazine of Concrete Research, Vol. 25, No. 85, December 1973, pp. 213-219.

82. * Levitt, M., "Non-destructive Testing of Concrete by Initial Surface Absorbtion Method", Proceedings. Symposium on Non-destructive Testing of Concrete and Timber, ICE, London, 1969, pp 23-36.

83. * BS :1881 (Part 5), " Methods of Testing Hardened Concrete for other than Strength ", British Standards Institution, London.

84. * Cather, R., Figg, J.W., Marsden, A.F. and O'Brien, T.P., " Improvements to Figg Methods for Determining the Air Permeability of Concrete ", Magazine of Concrete Research, Vol. 36, No. 129, Dec. 1984.

85. * " Simplified Method for the Detection and Determination of Chloride in Hardened Concrete", Information Sheet, IS 12/77, Building Research Establishment, Garston, 1977.

86. * " Quantab Chloride Filterations ", Miles Laboratories Ltd., Stoke Court, Stoke Poges, Slough, Bucks.(U.K.).

87. Knud G.Boving, "NDE Handbook, Non-destructive Examination Methods for Condition Monitoring, Butterworths", London, 1989.

88. * Programmed Instruction Handbook Non-destructive Testing PI-4-2 Liquid Penetrant Testing, General Dynamics" Convair Division.

89. * Oforstorande Provning, Sandvikens Handbook, (Non-destructive Testing Handbook Available from Sandviken AB in Sweden).

90. Hull, Barry and Vernon, John, "Non-destructive Testing of Concrete ", Macmillan, 1988, pp. 7-17.

91. Najjar S., Walid and Hover C. Kenneth, " Neutron Radiography for Microcrack Studies of Concrete Cylinders Subjected to Concentric and Eccentric Compressive Loads", ACI Materials Journal / July - August 1989, pp. 354-359.

92. Walids, Najjar, Howard, C., Aderhold Kenneth C. and Hover, " The Application of Neutron Radiography to the Study of Microcracking in Concrete ", Cement Concrete and Aggragates, Vol.8, No. 2, Winter 1986, pp. 103-109.

93. * Najjar, W.G., " The Development and Application of Neutron Radiography to Study Concrete with Emphasis on Microcracking ", Ph.D. Thesis, Cornell University, Ithaca, 1987, p.174.

94. Literature on Tiltmeter : M/S Slope Indicator Co., U.S.A.

95. Literature on Digi Tilt Inclinometer : M/S Slope Indicator Co., U.S.A.

96. * Siebel,E., and Pfender,M., Arch. Eisenhuettenwesen, 7(1933/34), p.407.

97. * Ruhl,K., "Neuere Hilfsmittel der Festigkeitsforschung (Modern Means in the Strength Research)", Die Technik, Vol. 3, No. 7, 1948.

98. * Emschermann, H.H. and Kruse, U.J.," Messung der Flaechenveraenderung an Papier mit Setzdehnungsmesser (Measurement of Variations in the Surface Area of Paper by Means of Contact - Type Strain Gauge)", Das Papier 5, No. 13/14, 1951.

99. Beckwith,T.G., Lewis, Buck, N. and Marangoni, R.D., "Mechanical Measurements ", Narosa Publishing House, India, 1989.

100. * Seebeck, J.J., " Evidence of the Thermal Current of the Combination Bi-Cu by Its Action on Magnetic Needle", Berlin, Abt. d. Konigl, Akad. d. Wiss., 1822-23, p.265.

101. * Peltier, M, " Investigation of the Heat Developed by Electric Currents in Homogeneous Materials and at the Junction of Two Different Conductors ", Ann. Chim. Phys. 56: 371, 1834p

102. * Thomson, W., " Theory of Thermoelectricity in Crystals ", Trans. Edinb. So. 21:153, p.1847.

103. Literature from M/S Slope Indicator Company, U.S.A.

104. Operational Manual - RCT, Rapid Chloride Test, M/S Germman Peterson Alps, Denmark.

105. National Council for Cement and Building Materials, "Investigations of Indian Oil Bhawan to Assess the Probable Causes of Distress and to Suggest Suitable Remedial Measures", Unpublished Project Report, CDI - SP 455, May 1989.

106. RILEM TC 56 : Measurement of Hardened Concrete Carbonation Depth (CPC - 18), Materials and Structures, Research and Testing, November 1988, 21(126), 453-6.

107. National Council for Cement and Building Materials, "Investigations to Assess Causes of Distress in the Sub - structure of Luni Bridge and to Suggest Suitable Remedial Measures", (Unpublished Project Report), March 1988.

108. Plowman, C. "A Logical Approach to the Investigation of a Concrete Structure". The International Journal of Construction Maintenance & Repair, 4 (3), May / June 1990, pp.110 - 113.

109. Petersons, C.G. and Poulsen, E., "In-situ NDT Methods for Concrete with Particular Reference to Strength, Permeability, Chloride Content and Disintegration"., Proc. Ist International Conference on Deterioration and Repair of Reinforced Concrete in the Arabian Gulf 1, CIRIA, 1985, pp.495 - 508.

110. Anon , "Radar Used for Structural Investigation, Construction Repair ", Sept. / Oct. 1988, pp. 20.

111. IS.: 456 - 1978 : "Code of Practice for Plain and Reinforced Concrete".

112. IS.: 516 - 1959 : "Methods of Test for Strength of Concrete".

113. Cement Research Institute of India, "Assessment of Strength of Concrete in the Training Walls and Apron Slab of the Bhakra Dam Spillway, (Unpublished Project Report), September 1982.

114. ASTM - C 900 - 87.

115. R.T.Stratful, Highway Research Record, No.443-12 (1973), 21.

116. Standard Test Method for Half-cell Potentials of Reinforced Steel in Concrete, ASTM C 876 - 80, Annual Book of ASTM Strandards.

117. Gronvold, F.O. and Arup, H., "Localisation of Corroding Reinforcement by Electrochemical Potential Surveys", Proceedings of the RILEM Symposium on Quality Control of Structural Concrete, Stockholm, June 1979, pp. 251-258.

118. Boulware, R.L.and Stewart, C.F., "Field Electrical Measurements for Bridge Deck Membrane Permeability and Reinforcing Steel Corrosion", Report No.CA H7 - BD 7120, pp. 234 - 569/2.

119. Tamhankar, M.G., Gaharwar S.S. and Singh V.K., "Non-destructive Testing of Bridges-State of Art", unpublished report, SERC, Ghaziabad, August 1991.

120. Kapla, M.S., Gaharwar, S.S. and Tamhankar, Dr. M.G., "Development of Structural Integrity Test to Monitor Bridge Degradation Through Vibration Response", unpublished report of SERC, Ghaziabad, 1994.

121. Gupta, S.C., "In-situ Strength of Concrete: Engineer, Builder and the Contractor", Indian Concrete Journal, February 1986

122. Rengaswamy, N.S., Srinivasan, S., Balasubramanian, T.M., Mahadeva Iyer, Y., Nayak, N.V. and Suresh Babu, R.H, "Corrosion Survey of Reinforced and Prestressed Concrete Structures - Methodology of Approach", Trans. SAEST, Vol.23, 2-3, 1988, pp.207-212.

123. Rengaswamy, N.S., Saravanan, K. and Srinivasan, S., "Electrochemical and Electrical Techniques for Monitoring Reinforcement Corrosion, Unpublished Report, CECRI, Karaikudi, Aug. 1993.

124. Guidelines for Evaluation of Load Carrying Capacity of Bridges, IRC Special Publication No.37, 1991.

125. Stratful R.F., "The Corrosion of Steel in Reinforced Concrete Bridges", Corrosion, 1957, 13, pp. 173 - 178.

126. Deptt. of Industry, Industrial Corrosion Monitoring, Committee on Corrosion, HMSO, London, 1978.

127. Escalaute, E. and I to S.R, "Measuring the Rate of Corrosion of Steel in Concrete", Proceedings of Corrosion 81, International Symposium, Toronto, April, 1981, p.44.

128. Rengaswamy, N.S., Balasubramanian, T.M., Srinivasan, S., Mahadeva Iyer, Y. and Suresh Bapu, R.H., "Corrosion Survey of Bridges" Indian Concrete Journal, June,1987, pp. 147 - 160.

129. Rengaswamy N.S., Srinivasan S., Balasubramanian TM., Mahadeva Iyer Y., Nayak N.V.and Suresh Bapu R.H., "Corrosion Survey of Reinforced and Prestressed Concrete Structures - Methodology of Approach", Trans. SAEST, 23, 1988, 2-3, pp. 201 - 212.

130. Flor, L.L.and Keith, W.T," Potential Survey Method to Detect Corrosion of Prestressed Concrete Tanks", Mat. Prot. Nov. 1965, pp.49-52.

131. Escalaute E., et al., "Measuring the Rate of Corrosion of Reinforcing Steel in Concrete", Final Report NBSIR 86 - 3456, Sponsored by FHWA, Washington, DC 205 90, Oct., 1986.

132. Cigna, R., et al, "Behaviour and Corrosion Rate Monitoring of Reinforced Concrete in Various Environments"; Proc. of 11th International Corrosion Congress, 2-6, April, 1990, Florence, Italy, p. 2.443 - 2.430.

133. John, D.G, et. al. "Corrosion Measurements on Reinforcing Steel and Monitoring of Concrete Structures", Proc.Corrosion/87, Symposium on Corrosion of Metals in Concrete 9 - 70, March, 1987, San Fransisco, p.159-167.

134. John, D.J. et. al., "Electrochemical Techniques for the Investigation Monitoring of Corrosion of Reinforcing Steel in Concrete", p.485 - 499.

135. Scott, P. Ewing and Hutchinson, "Corrosion 9", p.221, 1953.

136. A Direct Reading From Probe Resistivity Meter, Indian Patent No.2521, Dec 88, dated 20.8.88.

137. Stratful, R.F., "How Chloride Affect Concrete Used with Reinforcing Steel?", Materials Protection, Vol.7, 1968, pp. 29-34.

138. Subramaniyan, D., Srinivasan, S., Rengaswamy, N.S. and Balakrishnan, K., "A Study on Porosity of Concrete using Electrical Resistance Technique", Proceedings of National Seminar on Corrosion in Concrete and other Building Materials, Bhuvanesvaram, March 1990.

139. Saravanan , K., Vedalakshmi, R., Srinivasan, S., Rengaswamy, N.S. and Balakrishnan, K., "Evaluation of Surface Coatings for Concrete : An Acceptance Test, Trans. SAEST, Vol.23, 2-3, 1988, pp. 213 - 223.

140. Rengaswamy, S. and Rajagopalan, K.S., "Measuring Corrosion using Electrical Resistance Technique", Proc. Twelth Seminar on Electro-chemistry, March 1972, CECRI, Karaikudi, pp. 287-301.

141. Venkoba Rao, H.N., Mahadeva, Iyer Y., Rengaswamy, N.S. and Suresh Bapu, R.H., "Novel Resistance Probe Corrosion Monitor for Concrete Reinforcements", B.Electro-chem., 1-1, 1985, 77-79.

142. Francis Xavier, A.,"Corrosion Monitor for Reinforced Concrete Structure", Trans. SAEST, Vol.23, No.2-3 1988, pp.229 - 231.

143. Schutt, William R.,"Cathodic Protection Applied to Bridge Decks", Proc. Conference on Cathodic Protection of Reinforced Concrete Bridge Decks, NACE, 1985, pp. 46 - 54.

144. Suresh, Bapu R.H., Nayak, N.V., Srividya, Rajagopalan, S., Srinivasan, S., Rengaswamy, N.S.and Mahadeva, Iyer Y., "Monitoring of Corrosion of Prestressing Steel Enables in Prestressed Concrete Bridges," Trans. SAEST, 23, 2-3, 1988, pp.203 - 206.

145. Unpublished Report, CECRI, Karaikudi.

146. "Potential Wheel and Data Bucket", Publication of C.N.S. Electronics Ltd., London.

147. "In-situ Test Systems for Quality Control of Concrete", Publication of Germann Instruments Ltd. Copenhagen.

148. "Guidelines on Techniques for Strengthening and Rehabilitation of Bridges", IRC Special Publication 40, 1993.

149. "Contact Type Strain Gauge", Literature of Fritz Staeger, Berlin 47.

150. Joglekar, S.G.,"Non-destructive Measurements of Strength of Concrete, Accuracy, Variation and Adequacy of Estimators in Different Methods", 3rd Round Table Conference on NDT Techniques, IIBE, Bombay, 1994.

151. Tassios,T.P., "A Posteriori Estimation of Conventional Concrete Strength from in-situ Measurements", Testing during Concrete Construction', Proceedings of an International RILEM, The International Union of Testing and Research Laboratories for Materials and Structures Workshop 1990).

152. * "In-Place Methods for Determination of Strength of Concrete", Report ACI 228.1R-89, American Concrete Institute, Detroit, 1989.

153. Petersen, C.G., "In-place Testing for Quality Assurance and Early Loading Operations with Pull-out Testing and Maturity Measurements".

154. Guidelines for Inspection and Maintenance of Bridges, IRC Special Publication 35, New Delhi, 1990

155. Revised Draft Code of BIS on Non-destructive Integrity Testing of Concrete Piles.

156. IS:9716 - 1981, BIS Code on Lateral Dynamic Load Test on Piles.

157. Chandra Prakash, Rastogi, P.C. & Sharma, A.K. , "Use of Stress Wave Measurements for Diagnosis and Analysis of Piles", Civil Engineering and Construction Review, July 1989.

158. Chandra Prakash, Bhandari, R.K. , Sharma, A.K. & Rastogi, P.C. , "Pile Diagnostics by FPDS", Developments in Geotechnical Aspects of Embankments", Excavations and Buried Structures, AIT, Bangkok, 1991.

159. Mehta, D.B., "Integrity Testing of Piles and Drilled Shafts", IIBE Conference on NDT of Concrete Structures, Bombay, July 1994.

160. * Middendorp, P. and Brederode Van, P.J. , "A Field Monitoring Technique for the Integrity Testing of Foundation Piles", International Symposium on Field Measurements in Geomechanics, Zurich, September 5-8, 1983, pp. 493-502, 1983.

161. * Reiding, F.J., Middendorp, P. and Brederode Van, P.J. , "A Digital Approach to Sonic Pile Testing", Second International Conference on Application of Stress Wave Theory on Piles, Stockholm, pp. 85-93, 1984.

162. * Smith, E.A.L. , "Pile Driving Analysis by the Wave Equation", Journal Soil Mechanics and Foundations, ASCE, 86, August, pp. 35-61, 1960.

163. * Middendorp, P. and Reiding, F.J., "Determination of Discontinuities in Piles by TNO Integrity Testing & Signal Matching Techniques", 3rd Int. Conf. on the Application of Stress Wave Theory to Piles, Ottawa, 1988.

164. * Rausche, F., Likins, G.E. and Hussein, M., "Pile Integrity by Low and High Strain Impacts", 3rd Int. Conf. on Application of Stress Wave Theory to Piles, Ottawa, 1988.

165. * Sliwinski, Z.H. & Fleming, W.G.K. , "The Integrity and Performance of Bored Piles", Advances in Piling and Ground Treatment for Foundations Conference, I.C.E., Thomas Telford Ltd., London, 1984.

166. * Fellenius, Bengt H. (editor), "Application of Stress-Wave Theory to Piles", Proceedings of the Third International Conference, Ottawa, Canada, May, 1988.

167. * Goble, G.G., Rauche, F, and Likins, G.E., "The Analysis of Pile Driving State-of-the-Art", The 1st Seminar on the Application of Stress Wave Theory on Piles, Stockholm, Sweden, 1980.

168. * Berger, J.A. and Cotton, D.M., "Low Strain Integrity Testing of Deep Foundations", Proceedings of Deep Foundation Institute, Annual Meetings Seattle, October 1990.

169. Tamhankar, M.G. et. al., "Response Monitoring of Bridges during Construction through Built-in-Sensors", Journal of Indian Roads Congress, Vol.54-2, 1994.

'*' indicates sub-references.