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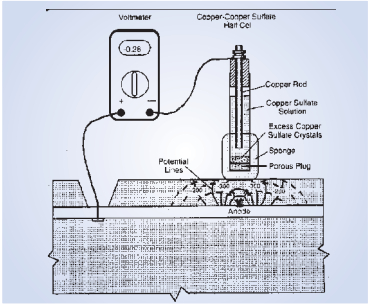
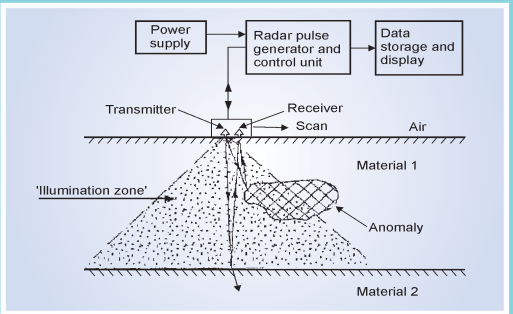
*To Beam As A Beacon of Knowledge*

# NON DESTRUCTIVE TESTING OF BRIDGES




**October 2014**

**INDIAN RAILWAYS INSTITUTE OF CIVIL ENGINEERING**
**PUNE 411001**

# NON DESTRUCTIVE TESTING
# OF
# BRIDGES

**(Revised & Reprinted October 2014)**

**INDIAN RAILWAY INSTITUTE OF CIVIL ENGINEERING**
**PUNE 411001**

# Forward to the Second Edition

The book "**NON DISTRUCTIVE TESTING OF BRIDGES"** was first published in June 2005. The book was intended to help field engineers for periodically monitoring the health of very important bridges as prescribed by Indian Railways Bridge Manual (IRBM). Since then , lot of new NDT techniques have been used world wide. This edition incorporates new NDT testing techniques like Nuclear Method, Structural Scanning Equipment (GPR), Spectral Analysis of Surface Waves for Unknown Foundation etc. and many more, which were not the part of the first edition.

I hope this edition will be a useful source of information for Railway engineers dealing with Non Destructive Testing of Bridges.

Pune
October 2014

**Vishwesh Chaubey**
Director / IRICEN, Pune
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# Preface to the Second Edition

The first edition of the book **"NON DISTRUCTIVE TESTING OF BRIDGES"** was published in June 2005 by Indian Railways Institute of Civil Engineering, Pune. The book was out of print and there was great demand from field engineers.

In August 2009, B & S Directorate of RDSO has issued "GUIDELINES ON NON - DESTRUCTIVE TESTING OF BRIDGES, BS – 103". This edition of book incorporates the new testing methods included in the RDSO guidelines, BS-103.

I am thankful to Shri N.C.Sharda, Senior Professor / Track I, for his contribution in publication of this book. I am grateful to Shri Vishwesh Chaubey, Director IRICEN, Pune for his encouragement and guidance in bringing out this publication.

Although care has been taken to include details as per reference, still there may be some errors. I would be thankful to the readers for their suggestions , which may be sent to IRICEN at mail@iricen.gov.in . These will be helpful for rectifying them.

Pune
October 2014

J.M.Patekari
Lecturer/Track-1
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# PREFACE

Non-destructive testing of bridges has assumed a greater significance in the present scenario because our existing inspection system is not adequate to identify the internal defects in the structures. With the recent collapses of bridges in India and other countries the objective inspection of bridges has become the need of the hour.

Indian Railway Bridge Manual (IRBM) prescribes periodical Health Monitoring of Very Important Bridges by an independent agency which includes corrosion monitoring, deterioration of material, system damage, retrofitting etc. All these can be done by using suitable NDT methods. Since the work will be executed by the independent agency, it is important to understand the details of the testing procedures as the engineer may have to supervise the work at the site. Since non-destructive testing methods do not form part of IRBM, the various methods used for testing different types of bridges have been included in this publication.

I hope this book will be found very much useful by field engineers, who are entrusted with the work of non-destructive testing of bridges, and will help as guide for implementation of testing methods for inspection and testing of bridges.

**Shiv Kumar**
**Director**
IRICEN/PUNE

# ACKNOWLEDGEMENT

The subject “non-destructive testing of bridges” is being taught during various courses at IRICEN. In the present scenario, it needs special attention. Even though a lot of information is available on this subject , yet when it pertaining to various types of bridges i.e. concrete, steel and masonry bridges, it is not available at one place.

The IRICEN publication is an attempt to compile all the relevant information regarding various NDT methods commonly in use for testing of all types of bridges on Indian Railways. Even though the publication is primarily aimed at Railway Engineers, the basic concepts are equally applicable to road bridges also.

It would not be out of place to acknowledge the support and assistance rendered by IRICEN faculty and staff in the above efforts. I am particularly thankful to Shri. Praveen Kumar, Prof. (Computers), who has provided logistic assistance for printing of this book. The word processing of the manuscript and repeated editings thereof has been done by Mrs. Gayatri. I also acknowledge the help of drawing staff of IRICEN who have assisted in preparation of drawings.

Above all, the author is grateful to Sri Shiv Kumar, Director, IRICEN for his encouragement and advice for improving the document.

**A.K. Gupta**
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# CONTENTS

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# CHAPTER 1

## NON-DESTRUCTIVE TESTING OF BRIDGES - GENERAL

## 1.1 Introduction

There are about 1,27,000 bridges of different types with varying spans on Indian Railways. About 40% of these bridges are over 100 years old and have completed their codal life. The present method of bridge inspection is mostly visual and give only subjective assessment of the condition of bridge. Moreover present inspection system is not capable of assessing hidden defects, if any.

We may group the various bridges mainly in three types, based upon material of construction.

(i) Concrete bridges
(ii) Steel bridges
(iii) Masonry bridges

Various types of bridges are having their own strengths, weaknesses and maintenance related problems. Each type of bridge is having different properties i.e. concrete is a heterogeneous material but the steel is a denser and homogeneous material. Similarly in masonry structures, the condition of joints and material of construction is of utmost importance. Considering the uniqueness of each type of bridge , there are different methods adopted for inspection and maintenance based on material of construction. In this book, various Non-Destructive Testing (NDT) methods for testing concrete , steel and masonry bridges have been discussed separately.

## 1.2 Concrete Bridges

The quality of new concrete structures is dependent on many factors such as type of cement, type of aggregates, water cement ratio, curing, environmental conditions etc. Besides this,

the control exercised during construction also contributes a lot to achieve the desired quality. The present system of checking slump and testing cubes, to assess the strength of concrete, in structure under construction, are not sufficient as the actual strength of the structure depend on many other factors such as proper compaction, effective curing etc.

Considering the above requirements, need of testing of hardened concrete in new structures as well as old structures, is there to asses the actual condition of structures. Non-Destructive Testing (NDT) techniques can be used effectively for investigation and evaluating the actual condition of the structures. These techniques are relatively quick, easy to use, cheap and give a general indication of the required property of the concrete.This approach will enable us to find suspected zones ,thereby reducing the time and cost of examining a large mass of concrete. The choice of a particular NDT method depends upon the property of concrete to be observed such as strength, corrosion, crack monitoring etc. The subsequent testing of structure will largely depend upon the result of preliminary testing done with the appropriate NDT technique.

**Purpose of Non-destructive Tests:** The non-destructive evaluation techniques are being increasingly adopted in concrete structures for the following purposes:

(i) Estimating the in-situ compressive strength

(ii) Estimating the uniformity and homogeneity

(iii) Estimating the quality in relation to standard requirement.

(iv) Identifying areas of lower integrity in comparison to other parts.

(v) Detection of presence of cracks, voids and other imperfections.

(vi) Monitoring changes in the structure of the concrete which may occur with time.

(vii) Identification of reinforcement profile and measurement of cover, bar diameter, etc.

(viii) Condition of prestressing /reinforcement steel with respect to corrosion.

(ix) Chloride, sulphate, alkali contents or degree of carbonation.

(x) Measurement of Elastic Modulus.

(xi) Condition of grouting in prestressing cable ducts.

Many of NDT methods used for concrete testing have their origin to the testing of more homogeneous, metallic system. These methods have a sound scientific basis, but heterogeneity of concrete makes interpretation of results somewhat difficult. There could be many parameters such as materials, mix, workmanship and environment, which influence the result of measurements. Moreover the test measure some other property of concrete (e.g. hardness) yet the results are interpreted to assess the different property of the concrete e.g. (strength). Thus, interpretation of the result is very important and a difficult job where generalization is not possible. Even though operators can carry out the test but interpretation of results must be left to experts having experience and knowledge of application of such non-destructive tests.

Variety of NDT methods have been developed and are available for investigation and evaluation of different parameters related to strength, durability and overall quality of concrete. Each method has some strength and some weakness. Therefore prudent approach would be to use more than one method in combination so that the strength of one compensates the weakness of the other. The various NDT methods for testing concrete bridges are listed below –

**A. For strength estimation of concrete**

(i) Rebound hammer test

(ii) Ultrasonic Pulse Velocity Tester

(iii) Combined use of Ultrasonic Pulse Velocity tester and rebound hammer test

(iv) Pull off test

(v) Pull out test

(vi) Break off test

(vii) Penetration Resistance Test (Windsor Probe)

(viii) Core Drilling Method

(ix) Permeability Test

(x) Bond Test

(xi) Maturity Method

(xii) Complete Structural Testing

**B. For assessment of corrosion condition of reinforcement and to determine reinforcement diameter and cover**

(i) Half Cell Potentiometer

(ii) Resistively Meter Test

(iii) Test for Carbonation of Concrete

(iv) Test for Chloride Content of Concrete

(v) Endoscopy Technique

(vi) Profo Meter

(vii) Micro Cover Meter

**C. For detection of cracks/voids/ delamination etc.**

(i) Infrared Thermographic Technique

(ii) Acoustic Emission Techniques

(iii) Short Pulse Radar Methods

(iv) Stress Wave Propagation Methods

- pulse echo method
- impact echo method
- response method

(v) Crack Detection Microscope

(vi) Boroscope

(vii) Nuclear Method

(viii) Structural Scanning Equipment

(ix) Spectral Analysis of Surface Waves for Unknown Foundation

All the above said methods have been discussed in detail in this book.

## 1.3 Steel Bridges

On Indian Railways, the superstructure of the large number of major bridges are of steel, and substructure is generally of concrete/masonry. These steel bridges are fabricated using structural steel section i.e. channels, angles, plates and I-sections etc. The bridges are subjected to severe dynamic stresses under passage of traffic and because of these stresses, the deterioration of the materials takes place.

In our system of inspection, we are mainly carrying out the visual inspection of the various parts of bridges, rivet testing and inspection of bearings etc. But all these methods do not give any indication about the microcracking, presence of flaws/internal blow holes/lamination etc. in the bridge members. Moreover some of the members of the bridge girders are difficult to inspect because of inaccessibility and in those cases, the NDT technique can be used effectively for inspection and evaluation of structures.

The various NDT methods for testing steel bridges are listed below:

(i) Liquid Penetrant Testing

(ii) Magnetic Particle Testing

(iii) Electromagnetic Testing or Eddy Current Testing

(iv) Radiography

(v) Ultrasonic Testing

(vi) Complete Structural Testing

(vii) Acoustic Emission Techniques

All the above said methods have been discussed in detail in this book.

## 1.4 Masonry Bridges

A large no. of bridges on Indian Railways are masonry bridges in which foundation or substructure is of either stone or brick masonry. In addition, a large no. of bridges are masonry arch bridges which have become quite old and already outlived their design life. The weakest location in a masonry bridge is the joint, as the deterioration gets initiated from the joints. With the passage of loads and over a period of time the deterioration of the material itself take place due to which the strength of the masonry structures gets affected. At the time of inspection, normally the condition of joints or the material on the outer surface is noted but it does not give any indication about the inherent defects within the structures. Moreover the present system of inspection is not about detecting the deterioration in strength of the stone/brick masonry because of the weathering action and other factors. In India, the NDT of masonry structures is still in necessant stage. There are lot of methods available for NDT of masonry structure, as indicated below:

(a) Flat Jack Testing

(b) Impact Echo Testing

(c) Impulse Radar Testing

(d) Infrared Thermography

(e) Boroscope

As the application of the above said NDT methods for masonry inspection is not very common in India, details given in this book are just for general guidance.

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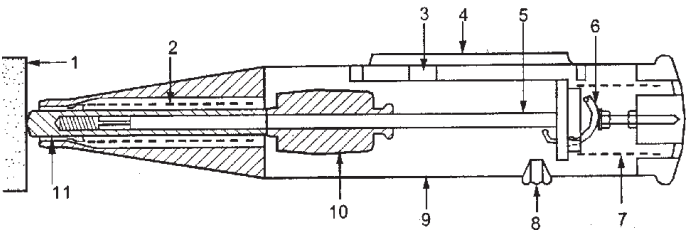
# CHAPTER-2

## NON-DESTRUCTIVE TESTS FOR STRENGTH ESTIMATION OF CONCRETE

## 2.1. Rebound Hammer Test (Schmidt Hammer)

This is a simple, handy tool, which can be used to provide a convenient and rapid indication of the compressive strength of concrete. It consists of a spring controlled mass that slides on a plunger within a tubular housing. The schematic diagram showing various parts of a rebound hammer is given as Fig 2.1.1




1. Concrete surface; 2. Impact spring; 3. Rider on guide rod; 4. Window and scale; 5. Hammer guide; 6. Release catch; 7. Compressive spring; 8. Locking button; 9. Housing; 10. Hammer mass; 11. Plunger

**Fig. 2.1.1 Components of a Rebound Hammer**

### 2.1.1 Object

The rebound hammer method could be used for –

(a) Assessing the likely compressive strength of concrete with the help of suitable co-relations between rebound index and compressive strength.

(b) Assessing the uniformity of concrete

(c) Assessing the quality of concrete in relation to standard requirements.
(d) Assessing the quality of one element of concrete in relation to another

This method can be used with greater confidence for differentiating between the questionable and acceptable parts of a structure or for relative comparison between two different structures.

**Fig. 2.1.2 Rebound Hammer**

## 2.1.2 Principle

The method is based on the principle that the rebound of an elastic mass depends on the hardness of the surface against which mass strikes. When the plunger of rebound hammer 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. The surface hardness and therefore the rebound is taken to be related to the compressive strength of the concrete. The rebound value is read off along a graduated scale and is designated as the rebound number or rebound index. The compressive strength can be read directly from the graph provided on the body of the hammer.

The impact energy required for rebound hammer for different applications is given below –

| Sr. No. | Application | Approximate impact energy required for the rebound hammers (N-m) |
|---|---|---|
| 1. | For testing normal weight concrete | 2.25 |
| 2. | For light weight concrete or small and impact sensitive part of concrete | 0.75 |
| 3. | For testing mass concrete i.e. in roads, airfield pavements and hydraulic structures | 30.00 |

Depending upon the impact energy, the hammers are classified into four types i.e. N, L, M & P. Type N hammer having an impact energy of 2.2 N-m and is suitable for grades of concrete from M-15 to M-45. Type L hammer is suitable for lightweight concrete or small and impact sensitive part of the structure. Type M hammer is generally recommended for heavy structures and mass concrete. Type P is suitable for concrete below M15 grade.

### 2.1.3 Methodology

Before commencement of a test, the rebound hammer should be tested against the test anvil, to get reliable results. The testing anvil should be of steel having Brinell hardness number of about 5000 N/mm$^2$. The supplier/manufacturer of the rebound hammer should indicate the range of readings on the anvil suitable for different types of rebound hammer.

For taking a measurement, the hammer should be held at right angles to the surface of the structure. The test thus can be conducted horizontally on vertical surface and vertically upwards or downwards on horizontal surfaces (Fig.2.1.3).

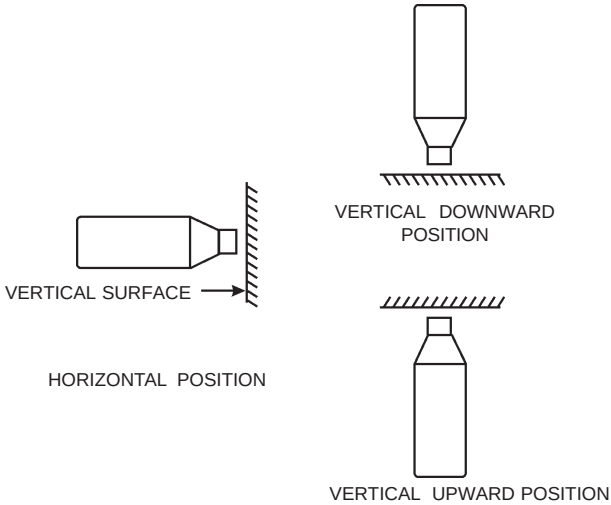



**Fig. 2.1.3 Various positions of Rebound Hammer**

If the situation so demands, the hammer can be held at intermediate angles also, but in each case, the rebound number will be different for the same concrete.

The following should be observed during testing.

(a) The surface should be smooth, clean and dry

(b) The loosely adhering scale should be rubbed off with a grinding wheel or stone, before testing

(c) Do not conduct test on rough surfaces resulting from incomplete compaction, loss of grout, spalled or tooled surfaces.

(d) The point of impact should be at least 20mm away from edge or shape discontinuity.

Around each point of observation, six readings of rebound indices are taken and average of these readings after deleting outliers as per IS 8900:1978 is taken as the rebound index for the point of observation.

### 2.1.4 Procedure for obtaining correlation between compressive strength of concrete and rebound number

The most satisfactory way of establishing a correlation between compressive strength of concrete and its rebound number is to measure both the properties simultaneously on concrete cubes. The concrete cubes specimens are held in a compression testing machine 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 specimens should be as large a mass as possible in order to minimize the size effect on the test result of a fullscale structure. 150mm cube specimens are preferred for calibrating rebound hammers of lower impact energy (2.2Nm), whereas for rebound hammers of higher impact energy, for example 30 Nm, the test cubes should not be smaller than 300mm.

If the specimens are wet cured, they should be removed from wet storage and kept in the laboratory atmosphere for about 24 hours before testing. To obtain a correlation between rebound numbers and strength of wet cured and wet tested cubes, it is necessary to establish a correlation between the strength of wet tested cubes and the strength of dry tested cubes on which rebound readings are taken. A direct correlation between rebound numbers on wet cubes and the strength of wet cubes is not recommended. Only the vertical faces of the cubes as cast should be tested. At least nine readings should be taken on each of the two vertical faces accessible in the compression testing machine when using the rebound hammers. The points of impact on the specimen must not be nearer an edge than 20mm and should be not less than 20mm from each other. The same points must not be impacted more than once.

### 2.1.5 Interpretation of results

After obtaining the correlation between compressive

strength and rebound number, the strength of structure can be assessed. In general, the rebound number increases as the strength increases and is also affected by a number of parameters i.e. type of cement, type of aggregate, surface condition and moisture content of the concrete, curing and age of concrete, carbonation of concrete surface etc. Moreover the rebound index is indicative of compressive strength of concrete upto a limited depth from the surface. The internal cracks, flaws etc. or heterogeneity across the cross section will not be indicated by rebound numbers.

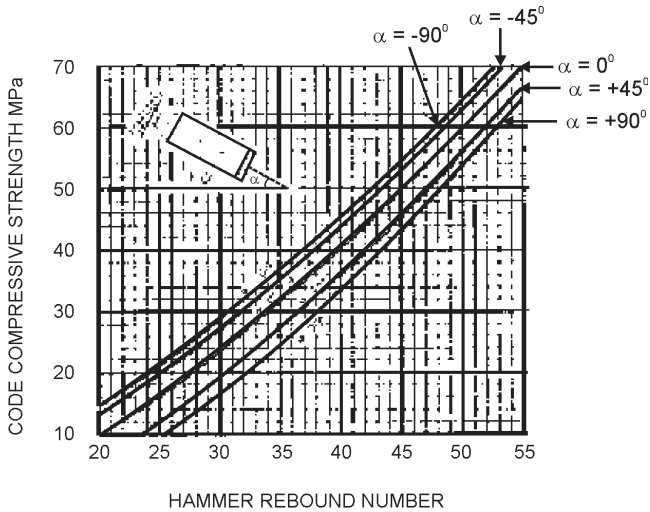



**Fig. 2.1.4 Rebound number vs compressive strength**

As such the estimation of strength of concrete by rebound hammer method cannot be held to be very accurate and probable accuracy of prediction of concrete strength in a structure is ± 25 percent. If the relationship between rebound index and compressive strength can be found by tests on core samples obtained from the structure or standard specimens made with the same

concrete materials and mix proportion, then the accuracy of results and confidence thereon gets greatly increased.

### 2.1.6 Standards

The rebound hammer testing can be carried out as per IS-13311 (Pt.2).

### 2.1.7 Other types of Rebound Hammer are:

**- Concrete Test Hammer (Pendulum Type):**

This is a new type of test hammer. In addition to testing of concrete, this measures the strength of masonry structures as well, although approximately. The equipment is very handy and to the fair extant reliable also. Further more, this is only equipment which is most pre-dominantly used in the field. It's new addition is having so many additional features.

**- Digital Concrete Test Hammer**

The digital concrete test hammer is a microprocessor operated standard unit equipped with electronic transducer which converts the rebound of the hammer into electric signal and displays it in the selected stress unit. It has capability of setting of test of the testing angle, selection of units in use ($Kg/cm^2$ , Mpa or Psi ). It is battery operated instrument and can be easily connected to a PC and has large memory to store up-to 5000 results.

## 2.2 Ultrasonic Pulse Velocity Tester

Ultrasonic instrument is a handy, battery operated and portable instrument used for assessing elastic properties or concrete quality. The apparatus for ultrasonic pulse velocity measurement consists of the following (Fig. 2.2.1) –

(a) Electrical pulse generator
(b) Transducer – one pair
(c) Amplifier
(d) Electronic timing device

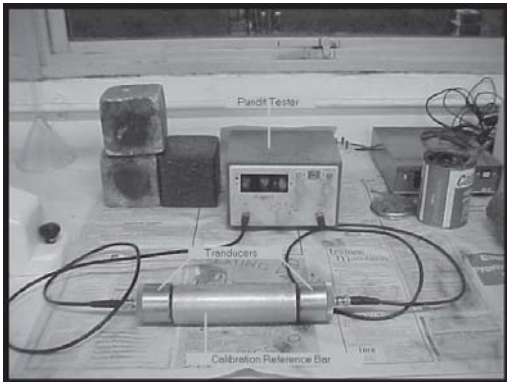



**Fig. 2.2.1 Apparatus for UPV measurement**

## 2.2.1 Object

The ultrasonic pulse velocity method could be used to establish:

(a) The homogeneity of the concrete

(b) The presence of cracks, voids and other imper fections

(c) Change in the structure of the concrete which may occur with time

(d) The quality of concrete in relation to standard requirement

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

(f) The values of dynamic elastic modulus of the concrete

## 2.2.2 Principle

The method is based on the principle that the velocity of an ultrasonic pulse through any material depends upon the density, modulus of elasticity and Poisson's ratio of the material. Comparatively higher velocity is obtained when concrete quality is

good in terms of density, uniformity, homogeneity etc. 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 (compression), shear (transverse) and surface (Reyleigh) waves. The receiving transducer detects the onset of longitudinal waves which is the fastest.

The velocity of the pulses is almost independent of the geometry of the material through which they pass and depends only on its elastic properties. Pulse velocity method is a convenient technique for investigating structural concrete.

For good quality concrete pulse velocity will be higher and for poor quality it will be less. 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 passed 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. Density and modulus of elasticity of aggregate also significantly affects the pulse velocity.

Any suitable type of transducer operating within the frequency range of 20 KHz to 150KHz may be used. Piezoelectric and magneto-strictive types of transducers may be used and the latter being more suitable for the lower part of the frequency range. Following table indicates the natural frequency of transducers for different path lengths –

| Path length (mm) | Natural Frequency of Transducer (KHz) | Minimum transverse dimensions of members (mm) |
|---|---|---|
| Upto 500 | 150 | 25 |
| 500 – 700 | $\geq$60 | 70 |
| 700 – 1500 | $\geq$40 | 150 |
| above 1500 | $\geq$20 | 300 |

The electronic timing device should be capable of measuring the time interval elapsing between the onset of a pulse generated at the transmitting transducer and onset of its arrival at receiving transducer. Two forms of the electronic timing apparatus are possible, one of which use a cathode ray tube on which the leading edge of the pulse is displayed in relation to the suitable time scale, the other uses an interval timer with a direct reading digital display. If both the forms of timing apparatus are available, the interpretation of results becomes more reliable.

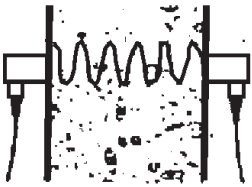
### 2.2.3 Methodology

The equipment should be calibrated before starting the observation and at the end of test to ensure accuracy of the measurement and performance of the equipment. It is done by measuring transit time on a standard calibration rod supplied along with the equipment.

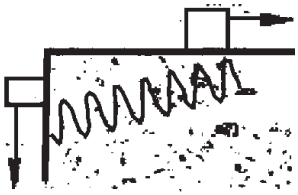
A platform/staging of suitable height should be erected to have an access to the measuring locations. The location of measurement should be marked and numbered with chalk or similar thing prior to actual measurement (pre decided locations).

#### Mounting of Transducers

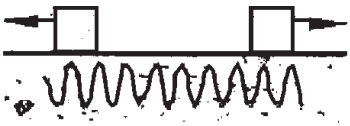
The direction in which the maximum energy is propagated is normally at right angles to the face of the transmitting transducer, it is also possible to detect pulses which have travelled through the concrete in some other direction. The receiving transducer detects the arrival of component of the pulse which arrives earliest. This is generally the leading edge of the longitudinal vibration. It is possible, therefore, to make measurements of pulse velocity by placing the two transducers in the following manners (Fig.2.2.2)



Direct Transmission
(Cross probing)



Semi-direct Transmission



Indirect Transmission
(Surface probing)

**Fig.2.2.2 Various Methods of UPV Testing**

(a) Direct Transmission (on opposite faces) – This arrangement is the most preferred arrangement in which transducers are kept directly opposite to each other on opposite faces of the concrete. The transfer of energy between transducers is maximum in this arrangement. The accuracy of velocity determination is governed by the accuracy of the path length measurement. Utmost care should be taken for accurate measurement of the same. The couplant used should be spread as thinly as possible to avoid any end effects resulting from the different velocities of pulse in couplant and concrete.

(b) Semi-direct Transmission: This arrangement is used when it is not possible to have direct transmission (may be due to limited access). It is less sensitive as compared to direct transmission arrangement. There may be some reduction in the accuracy of path length measurement, still it is found to be sufficiently accurate. This arrangement is otherwise similar to direct transmission.

(c) Indirect or Surface Transmission: Indirect transmission should be used when only one face of the concrete is accessible (when other two arrangements are not possible). It is the least sensitive out of the three arrangements. For a given path length, the receiving transducer get signal of only about 2% or 3% of amplitude that produced by direct transmission. Furthermore, this arrangement gives pulse velocity measurements which are usually influenced by the surface concrete which is often having different composition from that below surface concrete. Therefore, the test results may not be correct representative of whole mass of concrete. The indirect velocity is invariably lower than the direct velocity on the same concrete element. This difference may vary from 5% to 20% depending on the quality of the concrete. Wherever practicable, site measurements should be made to determine this difference.

There should be adequate acoustical coupling between concrete and the face of each transducer to ensure that the ultrasonic pulses generated at the transmitting transducer should be able to pass into the concrete and detected by the receiving transducer with minimum losses. It is important to ensure that the layer of smoothing medium should be as thin as possible. Couplant like petroleum jelly, grease, soft soap and kaolin/glycerol paste are used as a coupling medium between transducer and concrete.

Special transducers have been developed which impart or pick up the pulse through integral probes having 6mm diameter tips. A receiving transducer with a hemispherical tip has been found to be very successful. Other transducer configurations have also been developed to deal with special circumstances. It should be noted that a zero adjustment will almost certainly be required when special transducers are used.

Most of the concrete surfaces are sufficiently smooth. Uneven or rough surfaces, should be smoothened using carborundum stone before placing of transducers. Alternatively, a smoothing medium such as quick setting epoxy resin or plaster

can also be used, but good adhesion between concrete surface and smoothing medium has to be ensured so that the pulse is propagated with minimum losses into the concrete.

Transducers are then pressed against the concrete surface and held manually. It is important that only a very thin layer of coupling medium separates the surface of the concrete from its contacting transducer. The distance between the measuring points should be accurately measured.

Repeated readings of the transit time should be observed until a minimum value is obtained.

Once the ultrasonic pulse impinges on the surface of the material, the maximum energy is propagated at right angle to the face of the transmitting transducers and best results are, therefore, obtained when the receiving transducer is placed on the opposite face of the concrete member known as Direct Transmission.

The pulse velocity can be measured by Direct Transmission, Semi-direct Transmission and Indirect or Surface Transmission. Normally, Direct Transmission is preferred being more reliable and standardized. (various codes gives correlation between concrete quality and pulse velocity for Direct Transmission only). The size of aggregates influences the pulse velocity measurement. The minimum path length should be 100mm for concrete in which the nominal maximum size of aggregate is 20mm or less and 150mm for aggregate size between 20mm and 40mm.

Reinforcement, if present, should be avoided during pulse velocity measurements, because the pulse velocity in the reinforcing bars is usually higher than in plain concrete. This is because the pulse velocity in steel is 1.9 times of that in concrete. In certain conditions, the first pulse to arrive at the receiving transducer travels partly in concrete and partly in steel. The apparent increase in pulse velocity depends upon the proximity of the measurements to the reinforcing bars, the diameter and number of bars and their orientation with respect to the path of propagation. It is reported that the influence of reinforcement is generally small

if the bar runs in the direction right angle to the pulse path for bar diameter less than12 mm. But if percentage of steel is quite high or the axis of the bars are parallel to direction of propagation, then the correction factor has to be applied to the measured values.

### 2.2.4. Determination of pulse velocity

A pulse of longitudinal vibration is produced by an electro-acoustical transducer, which is held in contact with one surface of the concrete member under test. After traversing a known path length(L) in the concrete, the pulse of vibration is converted into an electrical signal by a second electro-acoustical transducer, and electronic timing circuit enable the transit time (T) of the pulse to be measured. The pulse velocity (V) is given by

$$V = L / T$$

where,

V = Pulse velocity

L = Path length

T = Time taken by the pulse to traverse the path length

### 2.2.5 Interpretation of Results

The ultrasonic pulse velocity of concrete can be related to its density and modulus of elasticity. It depends upon the materials and mix proportions used in making concrete as well as the method of placing ,compacting and curing of concrete. If the concrete is not compacted thoroughly and having segregation, cracks or flaws, the pulse velocity will be lower as compare to good concrete, although the same materials and mix proportions are used. The quality of concrete in terms of uniformity, can be assessed using the guidelines given in table below:

**Table: Criterion for Concrete Quality Grading**
**(**As per IS 13311(Part 1) : 1992**)**

| Sr. No. | Pulse velocity by cross probing (km/sec.) | Concrete quality grading |
|---|---|---|
| 1 | Above 4.5 | Excellent |
| 2 | 3.5 to 4.5 | Good |
| 3 | 3.0 to 3.5 | Medium |
| 4 | Below 3.0 | Doubtful |

***Note****: in case of doubtful quality, it will be desirable to carry out further tests.*

Since actual value of the pulse velocity in concrete depends on a number of parameters, so the criterion for assessing the quality of concrete on the basis of pulse velocity is valid to the general extent. However, when tests are conducted on different parts of the structure, which have been built at the same time with similar materials, construction practices and supervision and subsequently compared, the assessment of quality becomes more meaningful and reliable.

The quality of concrete is usually specified in terms of strength and it is therefore, sometimes helpful to use ultrasonic pulse velocity measurements to give an estimate of strength. The relationship between ultrasonic pulse velocity and strength is affected by a number of factor including age, curing conditions, moisture condition, mix proportions, type of aggregate and type of cement.

The assessment of compressive strength of concrete from ultrasonic pulse velocity values is not accurate because the correlation between ultrasonic pulse velocity and compressive strength of concrete is not very clear. Because there are large number of parameters involved, which influence the pulse velocity and compressive strength of concrete to different extents. However, if details of material and mix proportions adopted in the particular structure are available, then estimate of concrete strength can be made by establishing suitable correlation between the pulse velocity and

the compressive strength of concrete specimens made with such material and mix proportions, under environmental conditions similar to that in the structure. The estimated strength may vary from the actual strength by ± 20 percent. The correlation so obtained may not be applicable for concrete of another grade or made with different types of material.

### 2.2.6. Factors influencing pulse velocity measurement

The pulse velocity depends on the properties of the concrete under test. Various factors which can influence pulse velocity and its correlation with various physical properties of concrete are as under:

**Moisture Content:** The moisture content has chemical and physical effects on the pulse velocity. These effects are important to establish the correlation for the estimation of concrete strength. There may be significant difference in pulse velocity between a properly cured standard cube and a structural element made from the same concrete. This difference is due to the effect of different curing conditions and presence of free water in the voids. It is important that these effects are carefully considered when estimating strength.

**Temperature of Concrete:** No significant changes in pulse velocity, in strength or elastic properties occur due to variations of the concrete temperature between 5° C and 30° C. Corrections to pulse velocity measurements should be made for temperatures outside this range, as given in table below:

**Table – Effect of temperature on pulse transmission**
BS 1881 (Pt 203 Year 1986)

| Temperature °C | Correction to the measured pulse velocity in % | |
|---|---|---|
| | Air dried concrete | Water saturated concrete |
| 60 | +5 | +4 |
| 40 | +2 | +1.7 |
| 20 | 0 | 0 |
| 0 | -0.5 | -1 |
| - 4 | -1.5 | -7.5 |

**Path Length:** The path length (the distance between two transducers) should be long enough not to be significantly influenced by the heterogeneous nature of the concrete. It is recommended that the minimum path length should be 100mm for concrete with 20mm or less nominal maximum size of aggregate and 150mm for concrete with 20mm and 40mm nominal maximum size of aggregate. The pulse velocity is not generally influenced by changes in path length, although the electronic timing apparatus may indicate a tendency for slight reduction in velocity with increased path length. This is because the higher frequency components of the pulse are attenuated more than the lower frequency components and the shapes of the onset of the pulses becomes more rounded with increased distance travelled. This apparent reduction in velocity is usually small and well within the tolerance of time measurement accuracy.

With indirect transmission, there is some uncertainty regarding the exact length of the transmission path. It is, therefore, preferable to make a series of measurements with placing transducers at varying distances to eliminate this uncertainty. To do this, the transmitting transducer should be placed in contact with the concrete surface at a fixed point “x” and the receiving trans-

ducer should be moved at fixed increments ‘$x'_n$ along a chosen line on the surface. The transmission times recorded should be plotted as points on a graph showing their relation to the distance separating the transducers.( Fig.2.2.3).

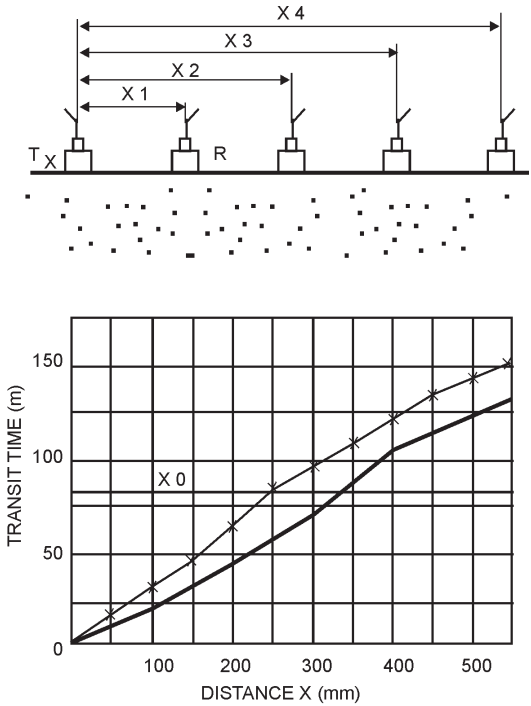



**Fig.2.2.3 Method for determination of pulse velocity**

The slope of the best fitted straight line drawn through the points should be measured and recorded as the mean pulse velocity along the chosen line on the concrete surface. Where the points measured and recorded in this way indicate a discontinuity, it is likely that a surface crack or surface layer of inferior quality is present and the pulse velocity measured in such case is unreliable.

**Shape and Size of Specimen:** The velocity of pulses of vibrations is independent of the size and shape of specimen, unless its

least lateral dimension is less than a certain minimum value. Below this value, the pulse velocity may be reduced appreciably. The extent of this reduction depends mainly on the ratio of the wavelength of the pulse vibrations to the least lateral dimension of the specimen but it is insignificant if the ratio is less than unity. Table given below shows the relationship between the pulse velocity in the concrete, the transducer frequency and the minimum permissible lateral dimension of the specimen.

**Table: Effect of specimen dimension on pulse transmission**

BS 1881 (Part 203 Year 1986)

| Transducer Frequency in KHz | Minimum lateral dimension in mm for Pulse specimen velocity in concrete in Km/s | | |
|---|---|---|---|
| | $V_c = 3.5$ | $V_c = 4.0$ | $V_c = 4.5$ |
| 24 | 146 | 167 | 188 |
| 54 | 65 | 74 | 83 |
| 82 | 43 | 49 | 55 |
| 150 | 23 | 27 | 30 |

This is particularly important in cases where concrete elements of significantly different sizes are being compared.

**Effect of Reinforcing Bars:** The pulse velocity in reinforced concrete in vicinity of rebars is usually higher than in plain concrete of the same composition because the pulse velocity in steel is almost twice to that in plain concrete. The apparent increase depends upon the proximity of measurement to rebars, their numbers, diameter and their orientation. Whenever possible, measurement should be made in such a way that steel does not lie in or closed to the direct path between the transducers. If the same is not possible, necessary corrections needs to be applied. The correction factors for this purpose are enumerated in different codes.

## 2.3. Combined use of Rebound hammer and Ultrasonic Pulse Velocity Method

In view of the relative limitations of either of the two methods for predicting the strength of concrete, both ultrasonic pulse velocity (UPV) and rebound hammer methods are sometimes used in combination to alleviate the errors arising out of influence of materials, mix and environmental parameters on the respective measurements. Relationship between UPV, rebound hammer and compressive strength of concrete are available based on laboratory test specimen. Better accuracy on the estimation of concrete strength is achieved by use of such combined methods. However, this approach also has the limitation that the established correlations are valid only for materials and mix having same proportion as used in the trials. The intrinsic difference between the laboratory test specimen and in-situ concrete (e.g. surface texture, moisture content, presence of reinforcement, etc.) also affect the accuracy of test results.

Combination of UPV and rebound hammer methods can be used for the assessment of the quality and likely compressive strength of in-situ concrete. Assessment of likely compressive strength of concrete is made from the rebound indices and this is taken to be indicative of the entire mass only when the overall quality of concrete judged by the UPV is ‘good’. When the quality assessed is ‘medium’, the estimation of compressive strength by rebound indices is extended to the entire mass only on the basis of other collateral measurement e.g. strength of controlled cube specimen, cement content of hardened concrete by chemical analysis or concrete core testing. When the quality of concrete is ‘poor’, no assessment of the strength of concrete is made from rebound indices.

## 2.4 Pull Off Test

Pull off tester is microprocessor based, portable hand operated and mechanical unit used for measuring the tensile strength of in situ concrete. The tensile strength obtained can be correlated with the compressive strength using previously established

empirical correlation charts. The apparatus for pull off test shall consist of the following -

(a) 50mm dia steel disc with threaded rod screw
(b) Pull off tester

One commercially available pull off tester is shown in Fig 2.4.1.


**Fig. 2.4.1 Pull Off Tester**

### 2.4.1 Object

The pull off test could be used to establish:

(a) The compressive strength of concrete
(b) Tensile strength of in situ concrete
(c) The adhesive strength of all kinds of applied coatings
(d) The bond strength of repairs and renovation works on concrete surfaces

### 2.4.2 Principle

The pull off test is based on the concept that the tensile force required to pull a metal disk, together with a layer of concrete, from the surface to which it is attached, is related to compressive strength of concrete There are two basic approaches that can be used. One is where the metal disk is glued directly to

the concrete surface and the stressed volume of the concrete lies close to the face of the disk, and the other is where surface carbonation or skin effect are present and these can be avoided by use of partial coring to an appropriate depth. Both the approaches are illustrated in Fig. 2.4.2.

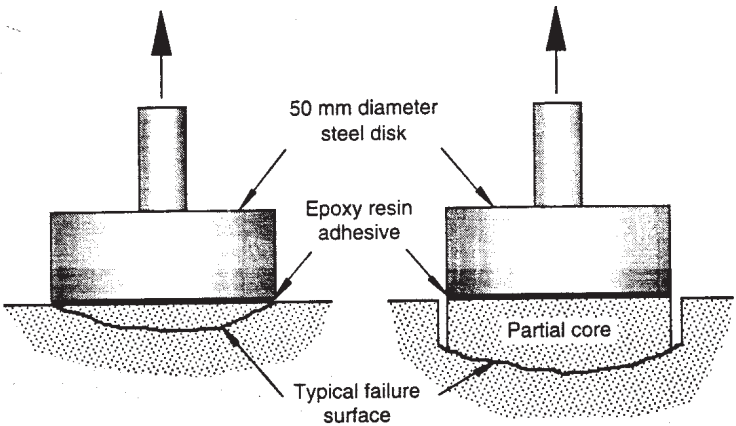



**Fig. 2.4.2 Various Methods of Pull Off Testing**

### 2.4.3 Methodology

The first step is to remove any laitance from the concrete surface to expose the top of the coarse aggregate particles. This is usually done using some sort of abrasion, using typically a wire brush. The exposed concrete surface and metal disk are then degreased to ensure good bonding of the adhesive. The adhesive is generally a two part epoxy system. A thin layer of adhesive is spread over the disk area and the metal disk is pressed firmly onto the concrete surface. Excessive adhesive that is squeezed out during this process should be removed before it sets. The curing time needed for the adhesive depends upon the type of epoxy used and surrounding environmental condition, although in most of the situations a curing time of not more than 24 hours is required. After the adhesive has cured sufficiently, the metal disk is “pulled” from the concrete surface. The apparatus used for applying and recording this tensile force is known as “Limpet” and

this applies a tensile force through a threaded rod screwed into the metal disk.

Equipments are available from 5 kN to 100 kN tensile force capacities. The instrument mechanism makes it possible to pre-select the rate of loading and actual tensile force applied is displayed on LCD monitor. The memory allows transferring of recorded data to PC.

From the recorded tensile force a nominal pull off tensile strength is calculated on the basis of the disk diameter (usually 50mm). To convert this pull off tensile strength into a cube compressive strength, a previously established empirical correlation chart is used.

### 2.4.4 Advantages and Limitations

The main advantage of pull off test is that it is simple and quick to perform. The entire process of preparing the surface and bonding the steel disk normally doesn't take more than 15 minutes. The damage caused to concrete surface after conducting the test is very minor and can be repaired easily.

The main limitation of this method is, the curing time, required for the adhesive. In most situations, it is normal practice to apply the disk one day and complete the test next day. The another problem is the failure of adhesive. The adhesive may fail because of inferior quality of adhesive or improper surface preparation or unfavourable environmental conditions. If during testing, the adhesive fails, the test result becomes meaningless.To compensate for this type of problem, it is recommended that at least six disks be used to estimate the compressive strength and, if necessary, one of the individual test results can be eliminated if an adhesive failure has occurred.

Another aspect of the pull off test is the correlation used to determine the compressive strength. The single factor that has the greatest effect on this relationship is the type of coarse aggregate used in the concrete, the greatest difference being between natural gravels and crushed rocks. Therefore, care should always

be taken to ensure that the correlation being used is applicable in that situation.

### 2.4.5 Standards

The test is conducted as per BS-1881 part 207.

## 2.5 Pull Out Test

The pull out test measures the force needed to extract an embedded insert from a concrete mass. By using a previously established relationship, the measured ultimate pullout load is used to estimate the in place compressive strength of the concrete. The equipments required for pull out test shall consist of the following:

(a) Specially shaped steel rod or discs
(b) Dynamometer to apply the force
(c) Loading ram seated on a bearing ring for applying pull out force

### 2.5.1 Object

The pull out test can be used to determine the following properties.

(a) Compressive strength of concrete
(b) Normally planned for new structures to help decide, whether critical activities such as form removal, application of post tensioning etc. can be started
(c) Can be used for existing structures and surveys of matured concrete

### 2.5.2 Principle

The pullout test measures the force required to pull an embedded metal insert with an enlarged head from a concrete specimen or a structure. The Fig 2.5.1 illustrates the configuration

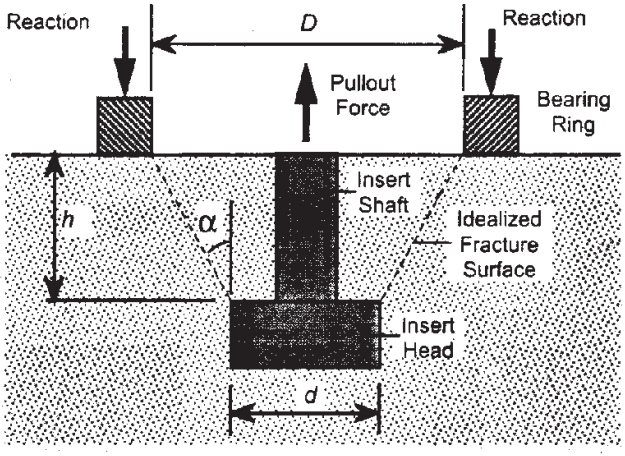



**Fig. 2.5.1 Configuration of Pull Out Test**

of a pull out test. The test is considered superior to the rebound hammer and the penetration resistance test, because large volume and greater depth of concrete are involved in the test. The pull out strength is proportional to the compressive strength of concrete. The pull out strength is of the same order of magnitude as the direct shear strength of concrete, and is 10 to 30% of the compressive strength. The pull out test subjects the concrete to slowly applied load and measures actual strength property of the concrete. The concrete is subjected, however, to a complex three dimensional state of stress, and the pull out strength is not likely to be related simply to uniaxial strength properties. Nevertheless, by use of a previously established correlation, the pull out test can be used to make reliable estimates of in situ strength.

### 2.5.3 Methodology

The pull out tests falls into two basic categories

(i) Those in which an insert is cast along with concrete i.e. the test is preplanned for new structures and

(ii) Those in which insert is fixed by under cutting and subsequent expanding procedure in the hardened concrete of existing structures.

These methods are generally known as “cast-in-method” and “drilled hole method” respectively. There are various variants in cast - in - in methods like LOK test as well as in drilled hole method like “CAPO” (cut and pull out). In the “CAPO” test method, an expanding ring is fixed into an under reamed groove, producing a similar pull out device to that used for “LOK”.

This insert is pulled by a loading ram seated on a bearing ring that is concentric with the inner shaft. The bearing ring transmits the reaction force to the concrete. As the insert is pulled out, a conical-shaped fragment of concrete is extracted from the concrete mass. The idealized shape of extracted conic frustum is shown in Fig 2.5.1. Frustum geometry is controlled by the inner diameter of the bearing ring (D), the diameter of the insert head (d), and the embedded depth (h). The apex angle ($\alpha$) of the idealized frustum is given by

$$2\alpha = 2\tan^{-1}\frac{D-d}{2h}$$

The ultimate pullout load measured during the in place test is converted to an equivalent compressive strength by means of a previously established relationship.

As per ASTM C-900-82, following are the requirements for metal insert –

(a) Embedment depth 1.0d
(b) Bearing ring 2.0d to 2.4d
(c) Apex angle 53° to 70°
where d = insert head diameter

### 2.5.4 Advantages & Limitations

The relationship between pullout strength and compressive strength is needed to estimate in place strength. Studies suggested that for a given test system there is a unique relationship. Therefore the recommended practise is to develop the strength relationship for the particular concrete to be used in construction. A large number of correlation studies have reported that

compressive strength is linear function of pull out strength.

The locations and number of pullout tests in a given placement should be decided very carefully. The inserts should be located in the most critical portions of the structure and sufficient number of tests should be conducted to provide statistically significant results.

The test is considered superior to the rebound hammer and the penetration resistance test, because large volume and greater depth of concrete are involved in the test.

### 2.5.6 Standards

The pull out test is conducted as per ASTM C 900-01 & BS-1881 Part 207.

## 2.6 The Break-Off Test

This test is a variant of the pull out test in which the flextural strength of concrete is determined in a plane parallel to and at a certain distance from the concrete surface. The break off stress at failure can then be related to the compressive or flextural strength of the concrete using a predetermined relationship that relates the concrete strength to the break off strength.

The BO tester, consists of a load cell, a manometer and a manual hydraulic pump capable of breaking a cylindrical concrete specimen having the specified dimensions.

### 2.6.1 Object

This test can be used both for quality control and quality assurance. The most practical use of the BO test method is for determining the time for safe form removal and the release time for transferring the force in prestressed post-tensioned members. This test can be planned for new structures as well as for existing structures.

### 2.6.2 Principle

The method is based upon breaking off a cylindrical specimen of in place concrete. The test specimen has a 55mm diameter and a 70mm height. The test specimen is created in the concrete by means of a disposable tubular plastic sleeve, which is cast into the fresh concrete and then removed at the planned time of testing, or by drilling the hardened concrete at the time of the break off (BO) test. Fig 2.6.1 and 2.6.2 show tubular plastic sleeves and a drill bit, respectively.

**Fig. 2.6.1 Tubular Plastic Sleeves**

**Fig. 2.6.2 Drill Bit**

Both the sleeve and the drill bit are capable of producing a 9.5mm wide groove (counter bore) at the top of the test specimen (see Fig 2.6.3).A force is applied through the load cell by means of a manual hydraulic pump. Fig 2.6.3 is a schematic of a BO concrete cylindrical specimen obtained by inserting a sleeve or drilling a core. The figure also shows location of the applied load at the top of the BO test specimen. In principal, the load

configuration is the same as a cantilever beam with circular cross section, subjected to a concentrated load at its free end. The force required to break off a test specimen is measured by mechanical manometer. The BO stress can then be calculated as:

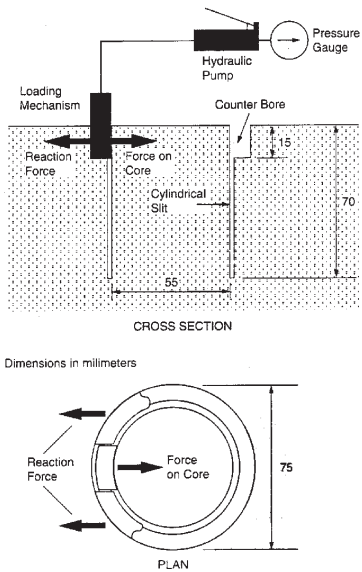



**Fig. 2.6.3 Schematic dig. of BO concrete specimen**

$$f_{BO} = \frac{M}{S}$$

where $M = P_{BO}h$,

$P_{BO}$ = BO force at the top

$h$ = 65.3 mm

$S = \frac{d}{32}$

$d$ = 55mm

In this case the cracks are initiated at the point 55mm away from the concrete surface.

### 2.6.3 Methodology

The load cell has two measuring ranges: low range setting for low strength concrete up to approx. 20 MPa and high range setting for higher strength concrete up to abut 60 MPa. The equipment used for this test is shown in Fig 2.6.4.

**Fig. 2.6.4 Equipment for Break Off Test**

A tubular plastic sleeve of diameter 55mm and geometry shown in Fig. 2.6.1, is used for forming cylindrical specimen in fresh concrete. A sleeve remover as shown in Fig. 2.6.5 is used for removing the plastic sleeve from hardened concrete.

**Fig. 2.6.5 Sleeve Remover**

A diamond tipped drilling bit is used for drilling cores for the BO test in hardened concrete (Fig. 2.6.2). The bit is capable of producing a cylindrical core, along with a reamed ring (counter bore) in the hardened concrete at the top with dimensions similar to that produced by using a plastic sleeve.

The sleeves should be at center to center and edge distance of minimum 150mm. Concrete inside the sleeve and the top of plastic sleeve itself should then be tapped by fingers to ensure good compaction for the BO specimen. Sleeves should then be moved gently up and down in place and brought to the same level as the concrete surface at its final position. For stiff mixes (i.e. low slump concrete) a depression may occur within the confines of the sleeve during the insertion process. In such cases the sleeve should be filled with additional concrete, tapped with fingers. For high slump concrete, the sleeve may move up ward due to bleeding. For such cases, sleeves should be gently pushed back in place, as necessary, to the level of finished concrete surface. Grease or other similar material, should be used to lubricate the plastic sleeves for its easier removal after the concrete hardens.

For core drilling from hardened concrete the concrete surface should be smooth in order to fix the vaccum plate of the core drilling machine. The core barrel should be perpendicular to the concrete surface. The length of the drilled core should be 70mm and in no case shorter than 70mm.

At the time of conducting BO test, remove the inserted plastic sleeve by means of key supplied with the tester (Fig 2.6.5). Leave the plastic ring in place. Remove loose debris from around the cylindrical slit and the top groove. Select the desired range setting and place the load cell in the groove on the top of the concrete surface so that load is applied properly. The load should be applied to the test specimen at a rate of approx. one stroke of hand pump per second. After breaking off the test specimen, record the BO manometer reading. The BO meter reading can then be translated to the concrete strength using curves relating the BO reading to the desired concrete strength.

Before conducting the test, the BO tester should be calibrated as per the procedure given by the manufacture. The BO manufacturer provides correlation curves relating the BO reading and the compressive strength of the standard 150mm cubes. However, it is desireable that the user should develop his own correlation curves for a particular concrete batch. Developing correlation curves for different types of concrete would increase the accuracy and dependability of method in predicting the in-place strength. The following precaution should be taken when developing data for correlations:

(a) Keep the center-to-center and edge distance of at least 150mm in the process of inserting sleeves or drilling
(b) Obtain a minimum of 5 readings and three corresponding strength test specimens values i.e. cubes for compressive strength, and beams for flextural strength, for each test age.
(c) An average of the five BO readings and the average of the three standard cube test results represent one point on the graph relating the BO reading to the desired standard strength of the concrete.
(d) Cover the range of concrete strengths expected during execution of the project, at early as well as later stages, such as 1,3,5,7,14 and 28 days.

### 2.6.4 Advantages & Limitations

The main advantage of BO test is that it measures in-place concrete (flextural) strength. The equipment is simple and easy to use, the test is fast to perform, requiring only one exposed surface. The BO test does not need to be planned in advance of placing the concrete because drilled BO test specimen can be obtained.

The two limitations of this test are :

(i) The max. aggregate size and
(ii) The minimum member thickness for which it can be used. The max. aggregate size is 19mm and min. member thick ness is 100mm. However, the principle for the method can be applied to accommodate large aggregate sizes or smaller members. The test cause the damage to the con crete which needs repair after conducting the test.

### 2.6.5. Standards

The break off test is conducted as per ASTM C 1150.

## 2.7 Penetration Resistance Test (Windsor Probe).

Amongst the penetration methods presently available, the most well known and widely used is Windsor Probe test. Penetration resistance methods are based on the determination of the depth of penetration of probes (steel rods or pins) into concrete. This provides a measure of the hardness or penetration resistance of the material that can be related to its strength.

The Windsor probe consists of powder – actuated gun or driver, hardened alloy steel probes, loaded cartridges, a depth gauge for measuring the penetration of probes and other related equipment (Fig. 2.7.1).

The probes have a tip dia of 6.3 mm, a length of 79.5mm, and a conical point. Probes of 7.9mm dia are also available for the testing of concrete made with light weight aggregates.

### 2.7.1 Object

The Windsor probe test is used to determine

(a) Compressive strength of in situ concrete

(b) For ensuring quality control

(c) For determining safe form removal time
(d) The uniformity of concrete and to delineate zones of poor quality or deteriorated concrete in structures.

### 2.7.2 Principle

The Windsor probe, like the rebound hammer, is a hardness tester, and the penetration of probe can be related to the compressive strength of concrete below the surface, using previously developed correlations between strength properties and penetration of the probe. The underlying principal of this penetration resistance technique is that for standard test conditions, the penetration of probe in to the concrete is inversely proportional to the compressive strength of the concrete. In other words larger the exposed length of the probe, greater the compressive strength of concrete.

### 2.7.3 Methodology

The method of testing is simple and is given in the manual supplied by the manufacturer. The area to be tested must have a smooth surface. To test structure with coarser finish, the surface must be first ground smooth in the area of the test. The powder actuated driver is used to drive a probe into concrete. If flat surfaces are to be tested, a suitable locating template to provide 158mm equilateral triangular pattern is used and three probes are driven into the concrete at each corner. The exposed length of individual probes are measured by a depth gauge. For testing structures with curved surfaces, three probes are driven individually using the single probe locating template. In either case, the measured average value of exposed probe length may then be used to estimate the compressive strength of concrete by means of appropriate correlation data.

The manufacturer of the Windsor probe test system supply tables relating exposed length of the probe with compressive strength of concrete. For each exposed length value, different values of compressive strength are given, depending upon the hardness of aggregate. However the manufacturer's table do not always give satisfactory results. Sometimes they considerably

over estimate the actual strength and in some cases they underestimate the strength. It is, therefore, imperative to correlate probe test result with the type of concrete being used. In addition to hardness of the coarse aggregate, the type and size of coarse aggregate also have a significant effect on probe penetration. The degree of carbonation and the age of concrete may also affect the probe penetration strength relationship.

### 2.7.4 Advantages & Limitations

Windsor probe testing method is basically hardness method, and like other hardness methods, should not be expected to yield absolute values of strength of concrete in a structure. However, like surface hardness tests, penetration tests provide an excellent means of determining the relative strength of concrete in the same structure, or relative strength in different structures.

One of the limitation of this test is minimum size requirements for the concrete member to be tested. The minimum distance from a test location to any edges of the concrete member or between two given test locations is of the order of 150mm to 200mm, while the minimum thickness of the member is about three times the expected depth of penetration. The test also causes some minor damage to the surface, which generally needs to be repaired.

The main advantages of this test are the speed and simplicity and only one surface is required for testing.

### 2.7.5. Standards

The penetration resistance test is coducted as per ASTM C 803 / C 803-03 and BS 1881 Part 207.

## 2.8 Core Drilling Method

Core drilling method is the most direct way of measuring the actual strength of concrete in the structure. It mostly involves proper selection of location and number of samples to be obtained.

Core should be taken so as to avoid the reinforcement. If avoidance of secondary reinforcement or surface reinforcement is inescapable, strength of Core can be taken as 10% less than measured strength. Cylindrical specimen of 100mm or 150mm diameter are common; other sizes may also be permitted but the least lateral dimension should not be less than 3 times the maximum size of the aggregates used. The core specimen to be tested should preferably have height of specimen as twice the diameter. If there are difficulties of obtaining samples of such size, the length to diameter ratio is permitted to be lower, but in no case lower than 0.95. The samples are to be stored in water for two days prior to testing and are to be tested in moist condition. The ends of specimens are trimmed and flatten and capped with molten sulphur or high alumina cement or some other permissible capping material to obtain a true flat surface. The specimen is then tested in compression.

Although drilling of cores and compressive strength test are quite simple (and are covered in IS:1199 and IS:516), but the procedures and influencing factors are to be carefully understood as they affect the measured value and therefore the assessment of the quality of in-place concrete. The provision of IS 456: 2000 vide clause 17.4.3 in this regard is given below:

"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 at least 85 percent of the cube strength of the grade of concrete specified for the corresponding age and no individual core has strength less than 75 percent."

## 2.9 Permeability Test:

### 2.9.1 Introduction

The permeability tester is a measuring instrument which is suitable for the determination of the air permeability of cover concrete by a non destructive method.

The fundamental significance of thin surface layer for the durability of concrete structures is attracting more and more

attention among researchers and engineers since it has been recognized that owing to the small distance between form work and reinforcement and as a result of process such as segregation and bleeding , finishing and curing, the formulation of micro-cracks, etc., the composition and properties of the cover concrete may differ very considerably from those of the good quality of cover concrete. In addition, the concrete test specimens used for quality controls can never represent the quality and properties of the cover concrete since they are produced and stored in a completely different manner.


**Fig. 2.9.1 General arrangement of Permeability Tester**

Bearing capacity of a structural element in a concrete structure is based on the mechanical properties of the total element and its durability under aggressive environmental influences depends essentially on the quality of a relatively thin surface layer ( 20-50 mm) . This layer is intended to protect the reinforcement from corrosion which may occur as a result of carbonation or due to ingress of chlorides or other chemical effects. The influence mentioned is enhanced by damage due to frost/thaw/salt.

The process which cause damage to concrete structure are so varied and include so many different and often interlined mechanisms( Physical, Chemical , Physi cochemical , Electrochemical, Mechanical) that it cannot be expected that only one or two parameters of the cover concrete quality will be

sufficient for predicting the durability. This aside, there is general agreement that the permeability of the cover concrete is the most relevant property for measuring the potential durability of and individual concrete.

There is no generally accepted method to characterize the pore structure of concrete and to relate it to its durability. However, several investigations have indicated that concrete permeability both with respect to air and to water is an excellent measure for the resistance of concrete against the ingress of agressive media in the gaseous or in the liquid state and thus is a measure of the potential durability of a particular concrete.

There is at present no generally accepted method for a rapid determination of concrete permeability and of limiting values for the permeability of concrete exposed to different environmental conditions.

Though concrete of a high strength class is in most instance more durable than concrete of a lower strength class, compressive strength alone is not a complete measure of concrete durability, because durability primarily depends on the properties of the surface layer of a concrete member, which have only a limited effect on concrete compressive strength.

The Permeability Tester permits a rapid and non-destructive measurement of the quality of the cover concrete with respect its durability. The general arrangement of the permeability tester is shown in fig 2.9.1.

**2.9.2 Principle:**

Significance of permeability in addition to compressive strength in assessing quality of concrete has become more important due to increasing instances of corrosion in reinforcement concrete. The rate at which the air from the concrete cover may extracted, is a measure of permeability of concrete. This method can be used to assess the resistance of concrete to carbonation, penetration of aggressive ions and quality of grout in post tension ducts.

### 2.9.3 Description of equipment:

The technical details of the instrument are given below:

**(1) Display Unit**

- Non volatile memory for upto 200 measured objects
- Display on 128 X 128 graphic LCD
- RS 232 C interface
- Integrated software for printout of measured objects and transmission to pC
- Operation with 6 batteries LR6 1.5 v for about 60 hrs. or commercial power unit 9 VDC/0.2 A.
- Temperature range $-10^0$C to $+60^0$C
- Carrying case 320 x 285 x 105 mm, total weight 2.1 Kg

**(2) Control Unit and vacuum cell**

- The volume of inner chamber and hose and the cross sectional area of the inner chamber are terms in the formula for calculating kT and L. They must therefore not be changed.
- Vacuum connection – small flange 16 KF
- Carrying case 520 x 370 x 125 mm , total weight 6.3 kg

**(3) Resistance probe WENNER – PROCEQ**

- Electrode spacing 50 mm

**(4) Vacuum pump:**

- The instrument is operated with a commercial vacuum pump.
- Technical data as per DIN 28400
- Suction capacity : 1.5 $m^3$/h
- Final total pressure : approx 10 bar
- Suction side connection : small flange 10 KF / 16KF
- High water vapour toleration

### 2.9.4 Methodology:

It operates under vacuum and can be used at the site and also in the laboratory.

The essential features of the method of measurement are a two chamber vacuum cell and a pressure regulator which ensures an air flow at right angles to the surface and into the inner chamber.

Dry surface without cracks should be selected for test. It should be insured that inner chambers should not be located above the reinforcement bar. Pressure loss is calibrated from time to time and after a large change in temperature and pressure. 3 to 6 measurements of electrical resistance of the concrete and its mean value is taken for the measurement of coefficient of permeability. This permits the calculation of the permeability coefficient kT on the basis of theoretical model.

In case of dry concrete, the results are in good agreement with laboratory methods, such as oxygen permeability, capillary suction, chloride penetration and others. The quality class of the cover concrete is determined from kT using a table as shown below.

**Table - Quality class of cover concrete**

| Quality of cover concrete | Index | kT ($10^{-16}$ $m^2$ ) |
|---|---|---|
| Very Bad | 5 | > 10 |
| Bad | 4 | 1.0 - 10 |
| Normal | 3 | 0.1 – 1.0 |
| Good | 2 | 0.01 – 0.1 |
| Very Good | 1 | < 0.01 |

The humidity, a main influence on the permeability, is compensated by additionally measuring the electrical resistance $\rho$ of the concrete. With kT and $\rho$ the quality class is obtained from a monogram shown in fig 2.9.2.

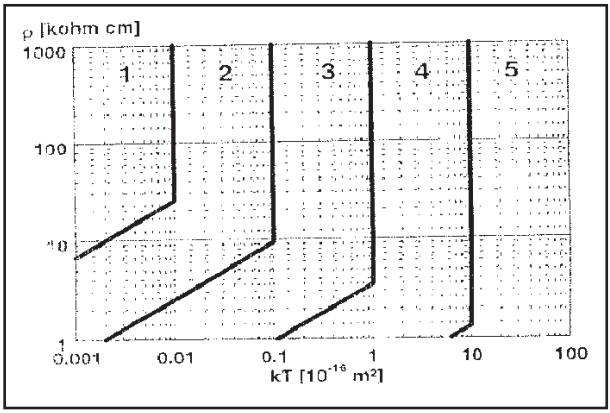



**Fig. 2.9.2 Monogram for quality class of cover concrete**

### 2.9.5 Limitations:

- The determination of kT and p should not be carried out on wet surfaces (the moisture entering the unit could damage the membrane in the pressure regulator).
- The most accurate values are obtained for dry concrete (p measurement is superfluous).
- In order to obtained an exact idea of the quality of the cover concrete of a structure or of a finished component, several measurements must always be carried out .
- The quality classification of cover concrete from table and the monogram from figure related to young concrete i.e. concrete age about 1-3 months. Some measurements on concrete a few years old have shown that the classification in Table and the monogram cannot be directly applied.
- The moisture content of the concrete has a major effect on the gas permeability. The correction of this effect by the measurement of the electrical resistance generally leads to satisfactory results in the case of young concrete. For old concrete, further investigations must be carried out.
- The investigations were performed using a vacuum pump with

a suction capacity of 1.5 $m^3/h$ and a motor power of 0.13 kW, this pump makes it possible to achieve a vacuum of a few mbar. Pumps of lower power do not reach the same vacuum and it is therefore advisable to use only pumps of similar power.

- There may be three further reasons why the desired vacuum (10-50 mbar ) is not reached.

- The concrete cover is too permeable (normal function of the unit).
- The concrete surface is too uneven: the rubber seals can compensate only a certain degree of unevenness (abnormal function).
- The unit has a leak (abnormal function).

## 2.10 Bond Test:

The bond testing equipment measures in place bonding or direct tensile strength between two layers e.g. a repair overlay and the parent concrete material or adhesion of shot Crete and membrane. The test location is so selected with the help of a metal detector that reinforcement disturbance, if any, is controlled and minimised during cutting operation.

The test consists of drilling a 100 mm nominal diameter core through the overlay into the parent concrete material. The drilled core is left without breaking. The top surface of the core is cleaned and dried and a cylindrical steel disc 85 mm in diameter is fastened to it with epoxy resin and adhesive. After hardening of the epoxy, the counter pressure ring with an inside diameter of 105 mm is placed concentric with the core on the overlay surface and connected to the pull bolts with countering plate and coupling. Load is applied by turning the instrument handle to a required pull force up to failure of the core in tension. Here also the disadvantage is that a small damage to the concrete is required to be repaired.

## 2.11 Maturity Method :

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

Compressive strength of well cured concrete increases with time. But this increase is dependent on the temperature of curing also. The combined influence of time and temperature is considered as the maturity. It is thus defined as the integral of time multiplied by temperature with a datum temperature of – $10^0$ C, since below this temperature cement in concrete ceases to hydrate.

The maturity of in-place can be monitored by thermocouples or by instruments called “maturity meters” The strength of in-place concrete is then estimate using the established correlation graph between maturity and compressive strength of concrete. The advantage of maturity concept is that by prior placing of maturity meters in the formwork at the time of the construction, the strength of early age concrete can be monitored and accordingly formwork can be removed confidently.

## 2.12 Complete Structural Testing:

### 2.12.1 Structural Testing System-

This integrated technique provides an approach that can be used todetect damage, general evaluation and development of load ratings for all type of st eel, concrete and timber structures.

The technique is based on the principle of “semi- static “live load test. It consists of instrumentation removal. In this technique upto 65 recessable strain transducers are indented to the structural members of the bridge and strains recorded vehicle with known weight crosses at crawling speed. High speed passes are also needed to conduct in order to determine actual impact.

**2.12.1.1 Advantage:**

- In this technique no holes or welds are required.
- Each strain sensors needs very short period of time (about 5 minutes) to install.
- In need, only one lane to be closed at a time.

The subsequent analytical modeling techniques range from a simple planner grid model to a three dimensional finite element representation. The response of the model in systematically compared with the field test results using multiple gauge location and load configurations. Structural parameters such as lateral deck stiffness, rotational restraints are then modified through an interactive process until the analytical responses closely match the field measurement. This calibrated model can be used to predict stress levels at critical locations due to rating and overloaded vehicle. Rating factors can then be developed using either allowable stress method (ASD) or the load factor method (LFD).

This approach is suitable to use on highway bridges, rail road bridges and other structures. Where the live load can be easily applied and load stresses significant, by approaching to this method steel, pre-stressed concrete, reinforce concrete and timber structures can be tested successfully.

**2.12.2** Indian Railways has planned to procure the complete structural testing equipment for performing live load tests on short to medium span Railway Bridge for static load, low speed, full sectional speed i.e. upto 200 kmph, braking and acceleration of trains so as to complete the full test for load rating of bridge.

■ ■ ■

# CHAPTER-3

## NON–DESTRUCTIVE TESTS FOR CORROSION ASSESSMENT, LOCATION AND DIAMETER OF REINFORCEMENT AND COVER THICKNESS OF CONCRETE BRIDGES

## 3.1 Introduction

For effective inspection and monitoring of concrete bridges, the condition assessment of reinforcement is an important step. Even for deciding appropriate repair strategy for a distressed concrete bridge, the determination of corrosion status of reinforcing bars is a must. Most of NDT methods used for corrosion assessment are based on electrochemical process. But apart from the process, it is advisable that the persons involved for conducting the tests should have enough experience in this particular field. The following methods are normally used for the condition assessment of reinforcement in concrete structures :

(a) Half cell Potential Measurement (Corrosion Analyzing Instrument)
(b) Resistivity test
(c) Test for carbonation of concrete
(d) Test for chloride content of concrete
(e) Endoscopy Technique

The following methods are generally used for determining the location / diameter of reinforcement bars and cover thickness in concrete bridges

(a) Profometer
(b) Micro covermeter

Each of the above said methods has been discussed in detail in this chapter.

## 3.2 Half-cell Potential Measurement Method

When there is active corrosion, current flow through the

concrete between anodic and cathodic sites is accompanied by an electric potential field surrounding the corroding bar. The equipotential lines intersect the surface of the concrete and the potential at any point can be measured using the half potential method. Apparatus for half cell potential measurement is shown in Fig.3.2.1.

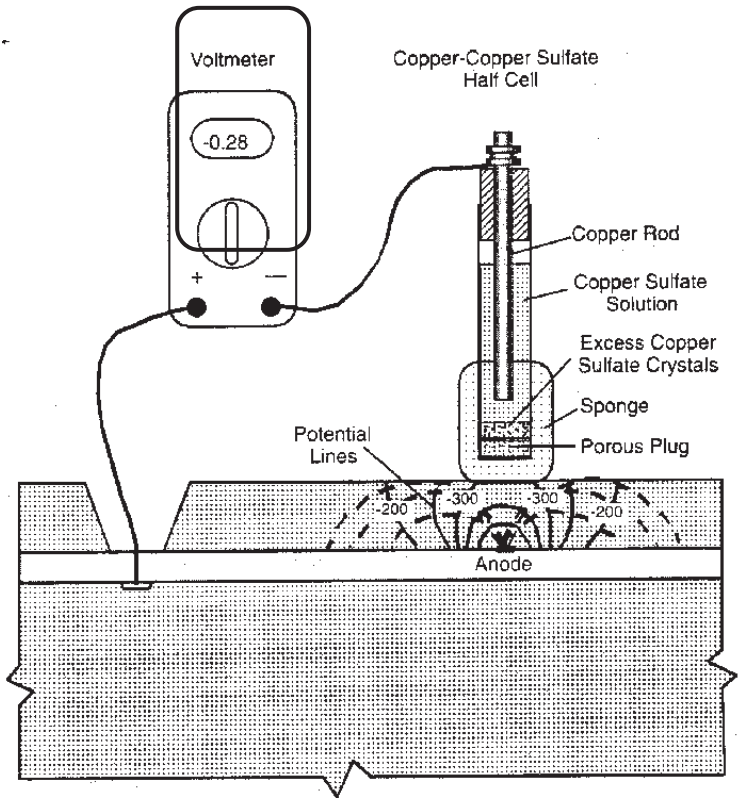



**Fig. 3.2.1 Apparatus for Half-Cell Potential Measurement**

The apparatus includes copper-copper sulphate half-cell, connecting wires and a high impedance voltmeter. This half-cell is composed of a copper bar immersed in a saturated copper sulphate solution. It is one of the many half cells that can be used as a reference to measure the electrical potential of embedded bars. A high impedance voltmeter (normally greater than 10MW) is used

so that there is very little current through the circuit. The copper-copper sulphate half-cell makes electrical contact with the concrete by means of porous plug and a sponge that is moistened with a wetting solution (such as liquid detergent).

One of the instrument available in the market is CANIN corrosion analyzer, which is a computer based device for making half cell potential measurement. This particular instrument stores data acquired at different test points and display equipotential contours.

### 3.2.1 Object

This test is used to assess the corrosion conditions in a reinforced concrete structure. The method detects the likelihood of corrosion of steel but can not indicate the rate of corrosion. By making measurements over the whole surface, a distinction can be made between corroded and non-corroded locations.

### 3.2.2 Principle

Corrosion analyzer is based on electro-chemical process to detect corrosion in the reinforcement bars of structure. It represents a galvanic element similar to a battery ,producing an electrical current, measurable as an electric field on the surface of concrete. The potential field can be measured with an electrode known as half cell. The electrical activity of the steel reinforcement and concrete leads them to be considered as one half of battery cell with the steel acting as one electrode and concrete as electrolyte. The name half cell surveying derives from the fact that the one half of the battery cell is considered to be the steel reinforcing bars and surrounding concrete. The electrical potential of a point on the surface of steel reinforcing bar can be measured comparing its potential that of copper – copper sulphate reference electrode/silver- silver nitrate reference electrode on the surface.

The positive terminal of the voltmeter is attached to the reinforcement and the negative terminal is attached to the copper-copper sulphate half cell. If there is any corrosion in the bars, the

excess electrons in the bar would tend to flow from the bar to the half cell. Because of the way the terminals of the voltmeter are connected in the electrical circuit (Fig. 3.2.1), the voltmeter indicates a negative voltage. The measured half cell potential is the open circuit potential, because it is measured under the condition of no current in the measuring circuit. A more negative voltage reading at the surface is to interpreted to mean that the embedded bar has more excess electrons, and there is, therefore, a higher likelihood that the bar is corroding.

The half cell potential readings are indicative of the probability of corrosion activity of the reinforcing bars located beneath the copper-copper sulphate reference cell. However, this is true only if the reinforcing steel is electrically connected to the bar attached to the voltmeter.

### 3.2.3 Methodology

The corrosion analyzing instrument CANIN operates as digital voltmeter. Voltage of + 999 mV DC can be measured using this instrument. The potential in millivolts is measured with rod electrodes at different locations on the structure. The measured voltage depends upon the type of the half-cell, and conversion factors are available to convert readings obtained with other half cells to copper-sulphate half cell.

Testing is usually performed at points arranged in a grid. The required spacing between test points depends on the particular structure. Excessive spacing can miss points of activity or provide insufficient data for proper evaluation, while closer spacing increase the cost of survey. In surveying bridge decks, ASTM C 876 recommends a spacing of 1.2 meter. If the difference in voltage between adjacent points exceed 150 mV, a closer spacing is suggested. A key aspect of this test is to ensure that the concrete is sufficiently moist to complete the circuit necessary for a valid measurement. If the measured value of the half cell potential varies with time, pre wetting of the concrete is required. Although pre wetting is necessary, there should be no free surface water between test points at the time of potential measurement. The concrete is sufficiently moist if the measured potential at a test point does not

change by more than $\pm$ 20 mV within a 5 min. period. If stability cannot be achieved by pre-wetting, it may be because of stray electrical currents or excessive electrical resistance in the circuit. In either case, the half cell potential method should not be used. Testing should be performed between temperature range of 17 to $28^0$C.

### 3.2.4 Interpretation of test results

As per ASTM C 876, two techniques can be used to evaluate the results (i) the numeric technique (ii) the potential difference technique.

In the numeric technique, the value of the potential is used as an indicator of the likelihood of the corrosion activity. The potential measured at the surface of concrete can be interpreted as per table given below:-

| Phase of Corrosion Activity | Potential as measured by Copper Half Cell |
|---|---|
| 1. Initial Phase – Corrosion activity not taking place | < - 200 mV |
| 2.Transient Phase – Corrosion activity uncertain | - 200 mV to - 350 mV |
| 3. Final phase – corrosion occurring positively | > - 350 mV |

Source: Appendix of ASTM C 876

The numeric method should not be used in the following conditions –

(a) Carbonation extends to the level of reinforcement

(b) Comparison of corrosion activity in outdoor concrete with highly variable moisture or oxygen content.

(c) Evaluation of indoor concrete that has not been subjected to frequent wetting.

(d) To formulate conclusions about changes in corrosion activity due to repairs which changed the moisture or oxygen content at the level of the steel.

In the potential difference technique, the areas of active corrosion are identified on the basis of the potential gradients. In the equipotential contour plot, the closer spacing of the voltage contour indicates regions of high gradients. The higher gradient indicates, higher risk of corrosion.

The potential difference technique is considered more reliable for identifying regions of active corrosion than is the use of numerical limits.

### 3.2.5 Limitations

For conducting this test access to the reinforcement is must. The method cannot be applied to epoxy coated reinforcement or concrete with coated surfaces. The concrete should be sufficiently moist for conducting this test.

This test only indicates the liklyhood of corrosion activity at the time of measurement. It does not furnish direct information on the rate of corrosion of the reinforcement.

### 3.2.6 Standards

This method is covered under ASTMC 876-91 (Reapproved 1999).

## 3.3 Resistivity Test

This test is used to measure the electrical resistance of the cover concrete. Once the reinforcement bar loses its passivity, the corrosion rate depends on the availability of oxygen for the cathodic reaction. It also depends on the concrete, which controls the ease with which ion migrate through the concrete between

anodic and cathodic site. Electrical resistance, in turn, depends on the microstructure of the paste and the moisture content of the concrete.

The combination of resistance measurement by resistivity meter and potential measurement by corrosion analyzing instrument give very reliable information about the corrosion condition of the rebar.

The equipment used for this test is a portable, battery operated, four probe device which measures concrete resistivity. ( Fig. 3.3.1)


**Fig. 3.3.1 Resistivity Meter**

### 3.3.1 Object

This test is used to assess the probability or likelihood of corrosion of the reinforcement bar. The resistivity increases as the capillary pore space in the paste is reduced. So high resistivity also indicates the good quality of concrete.

### 3.3.2 Principle

The corrosion of steel in concrete is an electrochemical process, which generates a flow of current and can dissolve metals.

The lower the electrical resistance, the more readily the corrosion current flows through the concrete and greater is the probability of corrosion. The resistivity is numerically equal to the electrical resistance of a unit cube of a material and has units of resistance (in ohms) times length. The resistance (R) of a conductor of area A and length L is related to the resistivity ρ as follows:

$$R = \frac{\rho L}{A}$$

The schematic diagram showing the set up for measurement of concrete resistivity is shown in Fig. 3.3.2.

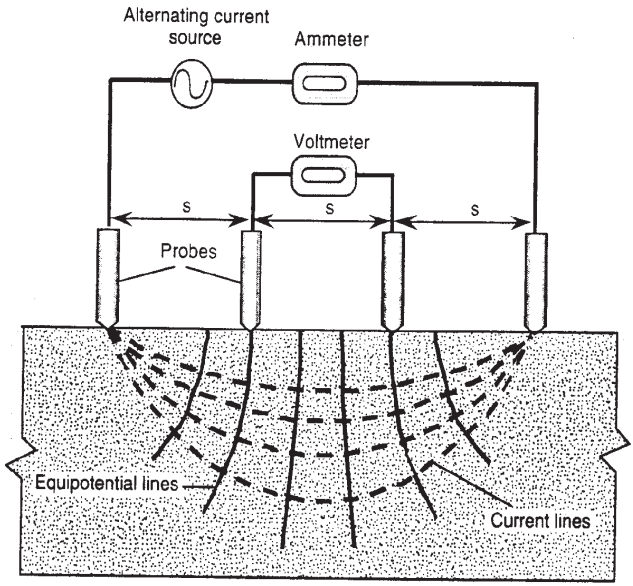



**Fig. 3.3.2 Setup for Measurement of concrete resistivity**

This is based on the classical four electrode system in which four equally spaced electrodes are electrically connected to the concrete surface. The outer electrodes are connected to a source of alternating current, and the two inner electrodes are connected to voltmeter.

### 3.3.3 Methodology

One of commercial equipment available for measurement of resistivity is Resistivity Meter which is a four probe device used for measuring resistivity. The set of four probes are fitted with super conductive foam tips to ensure full contact on irregular surfaces. Once the probes are kept in contact with the concrete surface, the LCD display will indicate the resistivity directly on the screen. The limits of possible corrosion are related with resistivity as under:

| | | |
|---|---|---|
| 1. | With $\rho$ =12 K W cm | Corrosion is improbable |
| 2. | With $\rho$ = 8 to 12 K W cm | Corrosion is improbable |
| 3. | With $\rho$ = 8 K W cm | Corrosion is fairly certain |

where $\rho$ (rho) is the resistivity

### 3.3.4 Limitations

The method is slow because it covers small area at a time. The system should not be used in isolation because it gives better indication of corrosion in reinforced concrete if used in combination with half - cell potentiometer.

## 3.4 Tests for Carbonation of Concrete

Carbonation of concrete in cover results in loss of protection to the steel against corrosion. The depth of carbonation can be measured by spraying the freshly fractured concrete surface with a 0.2% solution of phenolphthalein in ethanol. Since phenolphthalein is a pH indicator, the magenta (pink colour) area presents uncarbonated concrete and the remaining (colourless) portion, the carbonated area. The change in colour occurs at around pH 10 of concrete.

The test must be applied only to freshly exposed surfaces,

because reaction with atmospheric carbon dioxide starts immediately. Relating carbonation depth to concrete cover is one of the main indicators of corrosion.

## 3.5 Test for Chloride Content of Concrete

The presence of chloride in the concrete is the contributory factor towards corrosion of reinforcement.

Portable equipments are available in the market, which can be used for rapid on site measurement of chloride content of concrete. The chloride content of concrete can also be determined by chemical analysis of concrete in the laboratory.

A rotary percussion drill is used to collect a pulverized sample of concrete and a special acid extracts the chlorides. The amount of acid soluble chloride is determined directly by a chloride sensitive electrode connected to a electrometer.

If different samples are obtained from different concrete depths, it can be established whether the chloride contamination was there in the original concrete or the same has come from the environment.

## 3.6 Endoscopy Technique:

Endoscopy consists of inserting a rigid or flexible viewing tube into holes drilled into concrete bridge components or cable ducts and view them with light provided by optical glass fibers from an external source. This is a most useful method for inspecting or detecting voids in the grout and corrosion in steel in the cable ducts. It is also useful for detail examination of other part of the bridge structure, which could not other wise be assessed. Endoscopes are available as attachments for a camera or a TV monitor. It, however, needs an experienced engineer to make assessment of most likely locations of voids in the grout and probable points of entry of chlorides into the ducts.

## 3.7 Profometer

In any RCC/PSC structure, adequate cover thickness is essential to prevent corrosion of the reinforcement. In old structures, sometimes the detailed drawings are not traceable due to which it becomes very difficult to calculate the strength of the structure which is essentially required for finalizing the strengthening scheme. Sometimes, the bridges are to be checked from strength point of view to permit higher axle load and in absence of reinforcement details it becomes very difficult to take a decision.

To overcome all these problems, the methods have been developed for investigation and evaluation of concrete structures. Profometer is a small versatile instrument for detecting location, size of reinforcement and concrete cover. This instrument is also known as rebar locator. This is a portable and handy instrument which is normally used to locate the reinforcement on LCD display. This instrument is available with sufficient memory to store measured data. Integrated software is loaded in the equipment for carrying out and printing statistical values. One of the equipment which is commercially available in the market is shown in Fig. 3.7.1.


**Fig. 3.7.1. Profometer**

The equipment is quite handy and weighing less than two kgs. It works on normal batteries and thus does not require any electrical connection.

### 3.7.1. Object

This test is used to assess the location and diameter of reinforcement bars and concrete cover. This equipment can be used effectively for evaluation of new as well as old structures. The method can be used both for quality control as well as quality assurance.

### 3.7.2. Principle

The instrument is based upon measurement of change of an electromagnetic field caused by steel embedded in the concrete.

### 3.7.3. Methodology

To ensure satisfactory working of profometer and to get accurate results, it should be calibrated before starting the operations and at the end of the test. For this purpose, test block provided with the instrument should be used. To check the calibration accuracy, the size and cover of the reinforcement of the test block is measured at different locations on test block and the recorded data should match with the standard values prescribed on the test block.

Path measuring device and spot probes are together used for path measurements and scanning of rebars. These are connected with profometer with cables and are moved on the concrete surface for scanning the rebars and measuring the spacing. As soon as the bar is located, it is displayed on the screen. Once the bar is located, it is marked on the concrete surface.

Diameter probe is used for measuring the dia of bars. It is also connected with profometer by one cable. After finding out the location of rebar, the dia probe is placed on the bar parallel to bar

axis. Four readings are displayed and mean value of these readings is taken as diameter of bar.

Depth probe of the profometer is used to measure the cover. It is also connected with profometer by cable and is placed exactly on the bar As soon as, the depth probe is above a rebar or nearest to it, it gives an audio signal through a short beep and visual display. Simultaneously, the measured concrete cover is stored in memory.

For carrying out this test, the proper assess is essential. For this purpose, proper staging, ladder or a suspended platforms may be provided. Before actual scanning, marking is done with chalk on the concrete surface by dividing it into panels of equal areas.

### 3.7.4. Advantages and Limitations

This is a purely non-destructive test for evaluation of concrete structure particularly old structures. The methods is very fast and gives quite accurate results if the reinforcement is not heavily congested. The equipment is very light and even one person can perform the test without any assistance.

The equipment is not being manufactured in India and needs to be imparted. Some of the Indian Firms are marketing the instrument and this is a costly equipment.

## 3.8. Micro Covermeter

This is a portable and handy instrument weighing about 0.5 kg. This is normally provided with two types of search heads one for parallel bars having range approx. 360mm and other for mesh and close spaced bars having range of approx. 120mm. It can function over the temp. range of 0°C to 45°C. One of the micro cover meter commercially available in the market is shown in Fig. 3.8.1.

**Fig. 3.8.1. Micro Covermeter**

The equipment is of 180mm x 100mm x 45mm size approximately. The equipment is available with volatile memory that helps in storing data while taking measurements.

### 3.8.1. Object

The test is aimed mainly to detect the location and cover of reinforcement. The test may be used for analyzing the integrity of the structures as the cover thickness is an important aspect of construction. In the coastal areas, the test can be used for deciding the effectiveness of cover as well as rehabilitation measures required.

### 3.8.2 Principle

The equipment consists of a highly permeable U-shape magnetic core on which two coils are mounted. When an alternating current is passed through one of these coils, the current induced in the other coil can be measured. The presence of steel affects the electromagnetic field. The induced current depends upon the mutual inductance of the coils and the nearness of the steel bars. A moving coil meter measures the current. For measurement of the cover, the probe is placed directly over the concrete member and moved slowly until reading is obtained on the dial. The probe should be kept parallel to the length of rebar.

Depending upon the diameter of the bar, the dial readings gives directly the cover to the reinforcement.

### 3.8.3. Methodology

The equipment should be calibrated before starting and at the end of the test to get accurate results. For this purpose, one spacer is provided with each equipment. For calibration, the cover should be measured at one location and then it is remeasured after placing the spacer between the concrete surface and probe. The difference between two readings should not vary more than +/- 5% of the thickness of the spacer.

For locating the reinforcement bar, the search head should be placed on the surface of concrete in such a way so that the length of the search head should be parallel to the reinforcement provided in the structure. The location of the main reinforcement should be decided based upon the geometry of the structure. The search head should be moved from one end to other end in a direction perpendicular to the main reinforcement. The sound of the buzzer /beep will be strongest when the bar will come just above or below the probe.

For measurement of cover, the search head is moved on the surface. While moving, the cover displayed on the screen reduces and sound of the buzzer/beep increases when probe comes near reinforcement bars. The minimum reading displayed will be the cover and the sound of the buzzer is strongest when the reinforcement bar is just below the search head.

### 3.8.4. Advantages and Limitations

The method is very fast and large area can be covered within short time. The instrument work on batteries and does not require any electric supply. Since the equipment is very small and portable, the test can be conducted by single person without any assistance.

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# CHAPTER 4

## NON-DESTRUCTIVE TESTS FOR DETECTION OF CRACKS/VOIDS/ DELAMINATIONS ETC. IN CONCRETE BRIDGES

## 4.1 Introduction

For the assessment of the actual condition of a concrete bridge, the detection of internal crack, void, lamination etc. is very much necessary. The NDT methods for testing surface hardness, strength and for checking the condition of reinforcement do not indicate the internal condition of concrete. Sometimes these internal cracks, voids etc. may lead to the corrosion of reinforcement, cracking of section which may ultimately result into reduced life of bridge. The following methods are normally used for detection of cracks/voids/laminations etc. in concrete bridges:–

(i) Infrared thermographic techniques
(ii) Acoustic Emission technique
(iii) Short pulse radar methods
(iv) Stress wave propagation methods
(v) Crack Detection Microscope
(vi) Boroscope
(vii) Nuclear Method
(viii) Structural scanning equipment
(ix) Spectral Analysis of surface waves for unknown foundation

Some of these methods are used extensively all over the world for condition assessment of various components of concrete bridges while some methods are still in the laboratory trial stage. All the above methods are discussed in detail in this chapter.

## 4.2 Infrared Thermographic Techniques

For conducting this test, high resolution infrared thermographic radiometers (Fig. 4.2.1) are used to inspect large areas of concrete efficiently and quickly. This type of equipment

allows larger areas to be scanned, and the resulting data can be displayed as pictures with areas of differing temperatures designated by differing grey tones in black and white image or by various colours on a colour image. A complete thermographic

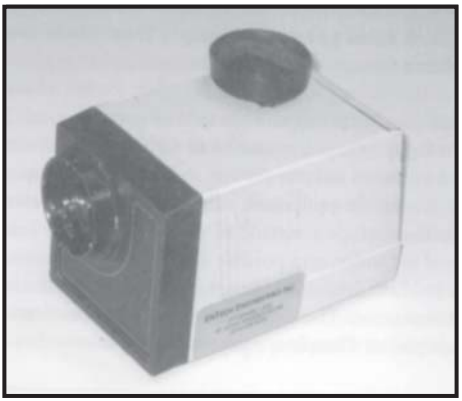


**Fig. 4.2.1 Infrared Thermographic Radiometer**

data collection and analysis system can be divided into four main subsystems. The first is the infrared sensor head that normally can be used with interchangeable lenses. It is similar in appearance to a portable video camera.

The second major component of the infrared scanning system is a real-time microprocessor coupled to a black-and-white or colour display monitor. With this, component, cooler items being scanned are normally represented by darker grey ones, and warmer areas are represented by lighter gray tones. Now-a-days, colour monitors are also used, which displays the different temperature levels as contrasting colours and patterns, which are easier to decipher.

The third major component of the infrared scanning system is the data acquisition and analysis equipment. It is composed of an analog-to-digital converter for use with analog sensors, a computer with a high resolution colour monitor, and data storage

and analysis software.

The fourth major component consists of various types of image recording and retrieving devices. These are used to record both visual and thermal images.

### 4.2.1 Object

This technique can be used to detect concrete subsurface delamination on bridge decks. This can also detect the internal voids, cracks and honeycombing in concrete structures. This method gives the fairly accurate picture about the condition of concrete inside and can be effectively applied for larger surfaces.

### 4.2.2 Principle

Infrared thermographic investigation techniques are based on the principle that the materials with subsurface anomalies, such as voids caused by corrosion of reinforcing steel, or voids caused by poor compaction called honeycombing, in a material affect heat flow through that material. These changes in heat flow cause localized differences in surface temperatures. Thus, by measuring surface temperatures under conditions of heat flow into or out of the material, one can determine the presence and location of any subsurface anomalies.

An infrared thermographic scanning system measures surface temperature only, but the surface temperatures of a concrete mass depend on three factors.

(a) The subsurface configuration
(b) The surface conditions
(c) The environment

Normally the testing should be conducted during times of the day or night when the solar radiation or lack of solar radiation would produce the most rapid heating or cooling of the concrete surface. The test should not be conducted when sky is cloudy. The measurements should be taken when then wind speed is lower than 25kmph. The test should not be conducted when the

temperature is below 0°C. If the concrete surface is covered with standing water, the test should not be conducted.

### 4.2.3 Methodology

For performing this test efficiently, a movement of heat must be established in the structure. Normally the inspection should be conducted during the sunny day, i.e. the testing should be avoided during monsoon. The inspection may begin either 2 to 3 hours after sunrise or 2 to 3 hours after sunset, both are times of rapid heat transfer. The surface to be tested should be cleaned thoroughly.

The next step is to locate a section of sound concrete. This work can be done by chain dragging (sounding), coring, or ground penetrating radar or by using some other suitable method. Image the reference area and set the equipment controls so that an adequate temperature image is viewed and recorded.

The next step is to image the area, which is known to have defects i.e. voids/delamination/cracks etc. and images of this defective area are viewed and recorded. Now the setting of the equipment should be done in such a way to allow viewing of both the sound and defective reference areas in the same image with the widest contrast possible.

If a black and white monitor is used, better contrast images will normally be produced when the following convention is used, black is defective concrete and white is sound material. If a colour monitor or computer enhanced screen is used, three colours are normally used to designate definite sound areas, definite defective areas and indeterminate areas. When tests are performed during day light hours, the defective concrete areas will appear warmer, whereas during test performed after dark, defective areas will appear cooler.Once the control are set, the recording of images can be done and stored.

### 4.2.4 Advantages & Limitations

The main advantage of this method is that it is an area testing technique, whereas other destructive and non-destructive methods are points testing or line testing methods. This method is completely non-destructive, repeatable, accurate, efficient and economical. The method can cover large areas within short time. One of the biggest advantage of this method is that this is absolutely safe and equipment does not emit any radiation.

The main limitation is that the depth or thickness of a void cannot be determined using this method. It cannot be determined whether a subsurface void is near the surface or away from the surface. When used in combination with ground penetrating radar method this method gives useful information about the internal defects.

## 4.3 Acoustic Emission Technique

Acoustic emission is the sound (both audible and sub-audible), that are generated when a material undergoes irreversible changes, such as those due to cracking. In general, acoustic emissions are defined as the class of phenomena whereby transient elastic waves are generated by the rapid release of energy from localized sources within a material. These wave propagate through the material, and their arrival at the surface can be detected by the piezoelectric transducers.

The main elements of a modern acoustic emission detection system are shown in Fig. 4.3.1.

The brief description of the most important parts of this system is as follows:

a) Transducers – Piezoelectric transducers (generally made of lead zirconate titanate, PZT) are used to convert the surface displacements into electric signals. There are mainly two types of transducers – wide band transducers and narrow band transducers. The transducers must be properly coupled to the specimen,

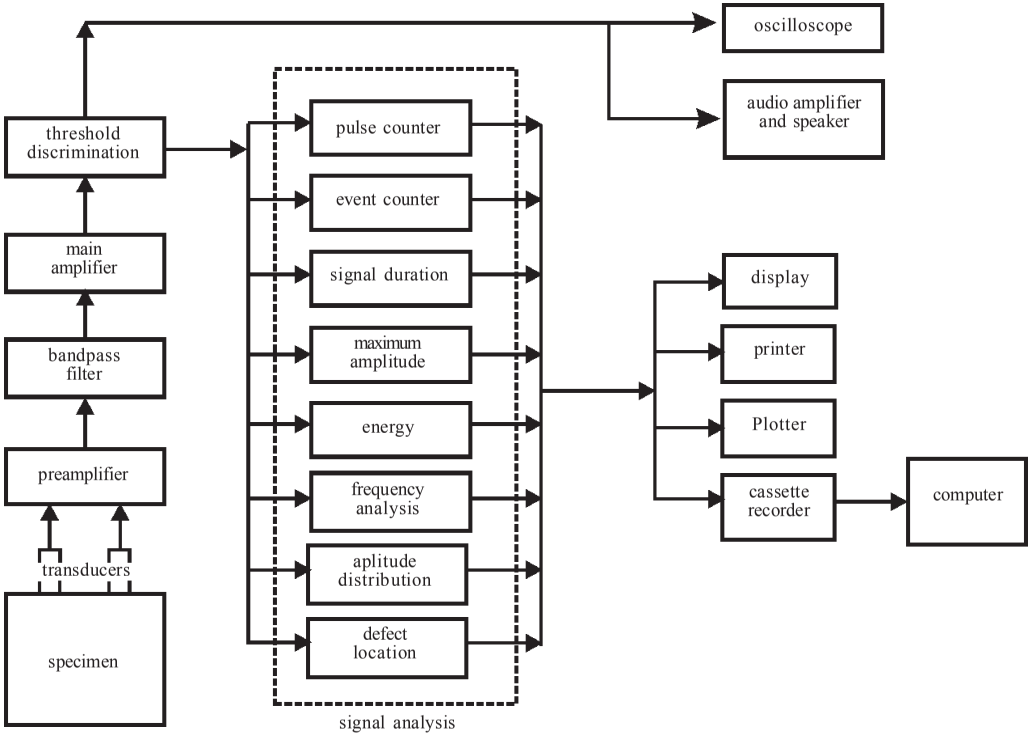



**Fig. 4.3.1 Elements of Modern Acoustic Emission Detection System**

often using some form of silicon grease as the coupling medium.

(b) Preamplifier – Because of the low voltage output, the leads from the transducer to the preamplifier must be as short as possible. Sometimes the preamplifier is integrated within the transducer itself. This amplifies the output signals.

(c) Passband filters – These are used to suppress the acoustic emission signals that lie outside the frequency range of interest.

(d) Main amplifier - This further amplifies the signals, typically within a gain of 20 to 60 dB.

(e) The discriminator – It is used to set the threshold voltage above which signals are counted.

### 4.3.1 Object

This method is used mainly to detect the cracking in concrete, whether due to externally applied loads, drying shrinkage or thermal stresses. This method can be helpful in determining the internal structure of the material and to know the structural changes during the process of loading.

The method can also be used to establish whether the material or the structure meet certain design or fabrication criteria. In this case, the load is increased only to some predetermined level. The amount and nature of acoustic emissions may be used to establish the integrity of the specimen or structure and may also be used to predict the service life.

### 4.3.2 Principle

When an acoustic emission event occurs at a source with the material, due to inelastic deformation or cracking, the stress

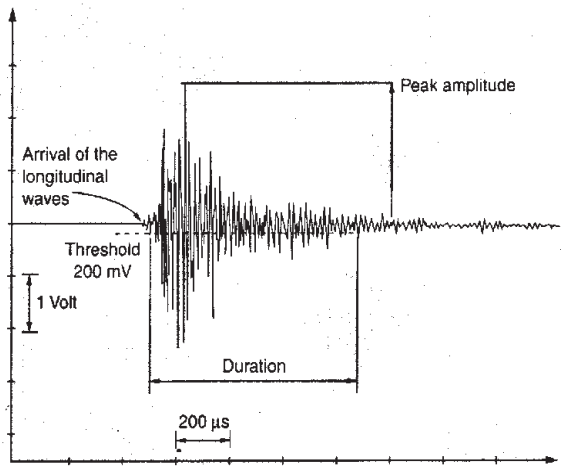



**Fig. 4.3.2 Acoustic Emission Signal from concrete**

waves travel directly from the source to the receiver as body waves. Surface waves may then arise from mode conversion. When the stress waves arrive at the receiver, the transducer responds to the surface motion that occurs. A typical acoustic emission signal from concrete is shown in Fig. 4.3.2.

By using a number of transducers to monitor acoustic emission events, and determining the time differences between the detection of each event at different transducer positions, the location of acoustic emission event may be determined by using triangulation techniques.

### 4.3.3 Methodology

Acoustic emission test may be carried out in the laboratory or in the field. Basically one or more acoustic emission transducers are attached to the specimen. The specimen is then loaded slowly, and the resulting acoustic emissions are recorded. The test is generally conducted in two ways.

(a) When the specimens are loaded till failure. (to know about internal structure/to study about structural changes during loading)

(b) When the specimens are loaded to some predetermined level (to ascertain whether the material or structure meet certain design or fabrication criteria).

### 4.3.4 Limitations

The acoustic emission techniques may be very useful in the laboratory to supplement other measurement of concrete properties. However, their use in the field is still very limited. Another draw back is that acoustic emissions are only generated when the loads on a structure are increased and this create considerable practical problems.

## 4.4 Short Pulse Radar Method

Short Pulse radar systems are used in applications related to inspection of concrete. This is the electromagnetic analog of sonic & ultrasonic pulse echo methods. In this method, the electro magnetic waves propagates through materials of different dielectric constants.

A basic radar system consists of a control unit, a monostatic antenna (i.e. an antenna that is used for both transmitting & receiving) , an oscillograph recorder, and a power convertor for DC operation. ( see Fig. 4.4.1)

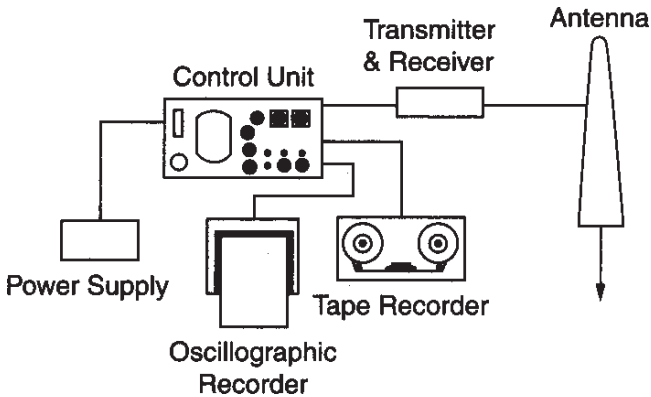



**Fig.4.4.1 Short Pulse Radar System**

For inspection of concrete structures, it is desirable to use a radar antenna with relatively high resolution or short pulse width.

### 4.4.1 Object

The method can be effectively used for detection of delamination in concrete. The method can also be used for determination of degree of hydration of concrete, water content in fresh concrete, and for measurement of concrete layer thickness etc. Method can also be used for locating the position of rebars.

### 4.4.2 Principle

When an electromagnetic wave (such as microwave) strike an interface, or boundary between two materials of different dielectric constants, a portion of the energy is reflected and the remaining penetrates through the interface into the second material ( Fig. 4.4.2 )

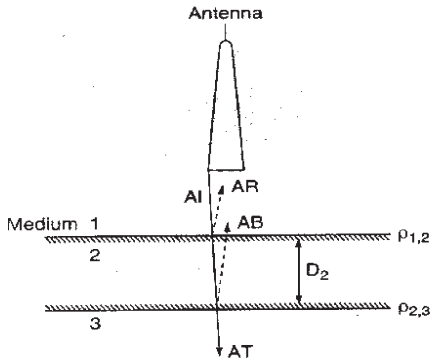



**Fig. 4.4.2 Movment of Electromagnetic Wave**

The intensity of reflected energy (AR), is related to the intensity of the incident energy (AI) by the following relationship -

$$\rho_{1,2} = \frac{AR}{AI} = \frac{\eta_2 - \eta_1}{\eta_2 + \eta_1} \text{.....(1)}$$

Where $\rho_{1,2}$ is the reflection coefficient at the interface, & are the wave impedances of the material 1,2 respectively, in ohms. For non-metallic material, such as concrete or soil the wave impedance is given by

$$\eta = \sqrt{\frac{\mu i}{\in}} \quad \text{.....(2)}$$

where $\mu$ is the magnetic permeability of air, which is 4 x$10^{-7}$ henry /meter and $\in$ is the dielectric constant of material in farad / meter

Since the wave impedance of air, $\eta_0$ is equal to $\sqrt{\frac{\mu_0}{\in_0}}$

where $\in_0$ is dielectric constant of air which is 8.85 x $10^{-12}$ farad/meter.

If we define the relative dielectric constant $\in_r$ of a material as

$$\in_r = \frac{\in}{\in_0}$$

Then the equation (2) can be written as

$$\eta = \frac{\eta_0}{\sqrt{\in_r}}$$

and equ. (1) can be written as

$$\rho_{1,2} = \frac{\sqrt{\in_{r_1} - \in_{r_2}}}{\sqrt{\in_{r_1} + \in_{r_2}}} \quad \text{.....(3)}$$

where $\in_{r_1}$ and $\in_{r_2}$ are the relative dielectric constants of material 1 an 2 respectively.

So depending upon the relative dielectric constants of material 1 and 2, the amount of reflected energy from interface of material 1 &2 is determined, and the remaining energy will be penetrated into material 2. When this remaining microwave energy reaches another interface after traveling to material 2, a portion will be reflected back as shown in equation (3).

This reflection of energy from various interfaces is being measured in Short Pulse Radar method for determination of various defects.

### 4.4.3 Methodology

For inspection of concrete bridges, an antenna is placed with its transmitting face parallel to and at a distance from the surface of the concrete. However, if the concrete member is relatively thick and the expected deterioration is deep or if the antenna does not have sufficient power or penetration, the antenna may be placed directly over the concrete surface. With the adjustments in the control unit the radar signals are recorded with a properly calibrated oscillograph recorder. This is called the static mode of measurement, since the antenna is stationary with respect to the concrete being tested.

If relatively large concrete area has to be inspected the antenna is mounted on the front or rear of an inspection vehicle. The vehicle will make several passes over the area to be tested to cover the entire area. During each pass the antenna scans a different area. The stream of the radar signals are recorded continuously with an instrumentation tape recorder. With two antenna or multi antenna system, the requirement of number of passes will be less. These recordings are later on studied and interpreted for detection of defects.

### 4.4.4 Advantages and Limitations

This methods is very much effective for inspection of deck slab of road bridges, provided with asphalt overlays. The distruption to road traffic is minimal while inspection is done. There are no restrictions, regarding the timing of inspection or the availability of certain ambient conditions during inspections and inspection can be done any time and in any type of ambient conditions.

The major limitation is that the interpretation of signals received at radar is very cumbersome because of the presence of interfering signals.

## 4.5 Stress Wave Propagation Methods

There are several test methods, based on stress wave propagation, used for non-destructive testing of concrete. The echo methods (impact echo and pulse echo) are used for thickness measurements, flaw detection and integrity testing of piles. The impulse response method is also used to test piles and slab like structures. The following stress wave propagation method are generally used for testing of concrete.

(a) Pulse Echo method
(b) Impact Echo method
(c) Impulse response method

All the three methods are discussed in detail in this chapter.

### 4.5.1 Pulse Echo Method

In this method, a transmitter introduces a stress pulse into an object at an accessible surface. The pulse propagates into the test object and is reflected by flaws or interfaces. The surface response caused by the arrival of reflected waves, or echoes, is monitored by either the transmitter acting as a receiver (true pulse echo) or by a second transducer located near the pulse source (pitch-catch). (Fig. 4.5.1.)

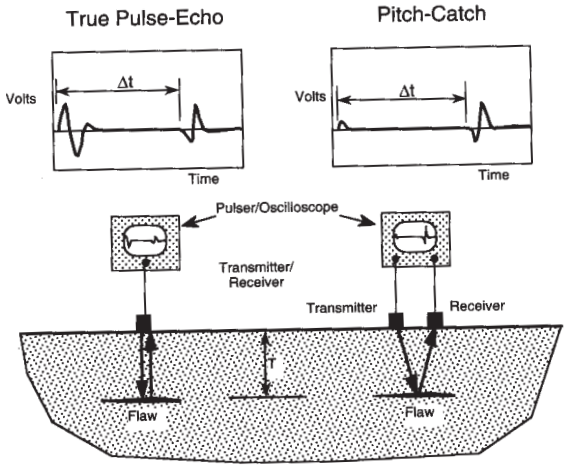



**Fig. 4.5.1 Pulse Echo Methods**

The output is displayed on a display device and the display is called a time domain waveform. By using the time base of display, the travel time of the pulse is determined. If the wave speed in the material is known, this travel time can be used to determine the depth of the reflecting interface using the following equation:.

$$T = \tfrac{1}{2}\,\Delta t\, C_p$$

Where $\Delta t$ = the round trip travel time

$T$ = the depth

$C_p$ = the wave speed

The main components of a pulse echo or pitch catch test system are the transmitting and receiving transducer and the system that is used to record and display wave forms. For testing concrete, low frequency transducers are required. Because of the practical problem of manufacturing large size transducers, the pulse echo technique is not used for testing of concrete in the field.

## 4.5.2 Impact Echo Method:

### 4.5.2.1 Introduction:

The impact-echo method is a technique for flaw detection in concrete based on stress wave propagation. A team of researchers at the National Institute of Standards and Technology (formerly the National Bureau of Standards) initiated a study in 1983 that developed the rudimentary basis for this method. Subsequently, research carried out at Cornell University, under the direction of Dr. Mary Sansalone, has refined the theoretical basis of the method, and extended its applications to a broad spectrum of problems and lead to the development of a field test system. Studies have shown that the impact-echo method is effective for locating voids, honeycombing, delaminations, depth of surface opening cracks, and measuring member thicknesses.

### 4.5.2.2 Basic Principle

The principle of the impact-echo technique is illustrated in Fig.4.5.2.1. A transient stress pulse is introduced into a test object by mechanical impact on the surface. The stress pulse propagates into the object along spherical wavefronts as P- and S-waves (Fig. 1(a)). In addition, a surface wave (R-wave) travels along the surface away from the impact point. The P- and S- stress waves are reflected by internal interfaces or external boundaries. The arrival of these reflected waves at the surface where the impact was generated produces displacements which are measured by a receiving transducer (Fig.1 (b)). If the receiver is placed close to the impact point, the displacement waveform is dominated by the displacements caused by P-wave arrivals. The displacement waveform can be used to determine the travel time, t, from the initiation of the pulse to the arrival of the first P-wave reflection. If the P-wave speed, Cp, in the test object is known, the distance, T, to the reflecting interface can be determined.

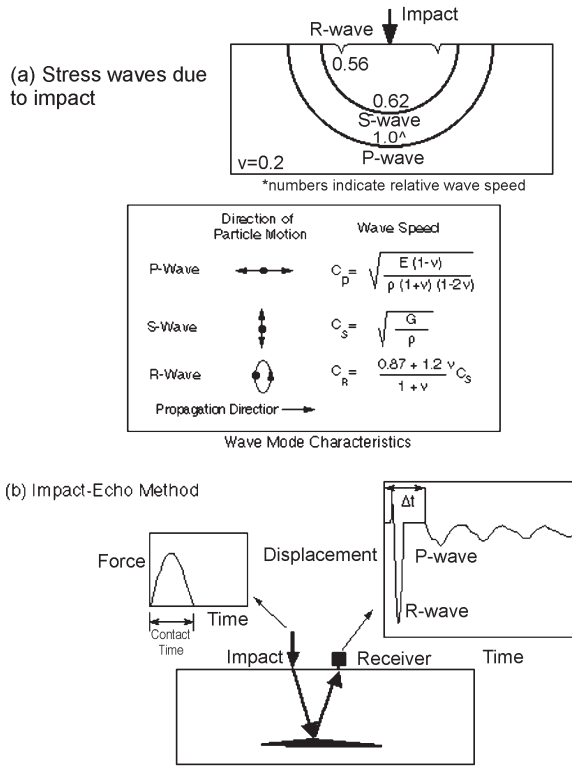



**Fig. 4.5.2.1 -** Principle of the impact-echo method : (a) Impact on solid results in stress waves that propagate into the solid and along its surface; (b) A receiver near the impact point sense the arrival of the R-wave and reflected P-wave.

### 4.5.2.3 Instrumentation:

An impact-echo test system is comprised of three components:

- An impact source.
- A displacement transducer.
- A computer or waveform analyzer.

A commercial impact-echo test system was developed in 1990 at Cornell University. It includes a rugged laptop style computer that functions as the data acquisition system. Special software has been written to set up the data acquisition parameters and perform the data analysis. The system also includes a handheld unit that houses the receiving transducer and a series of different-sized impactors. A suitable system can also be assembled from off-the-shelf components. The waveform analyzer (or data acquisition card) must have a high sampling frequency (500 kHz as a minimum), and the receiving transducer should preferably be a broadband displacement transducer. Accelerometers have been used but they must not have resonant frequencies in the range of those measured during impact-echo testing and additional signal processing is required.

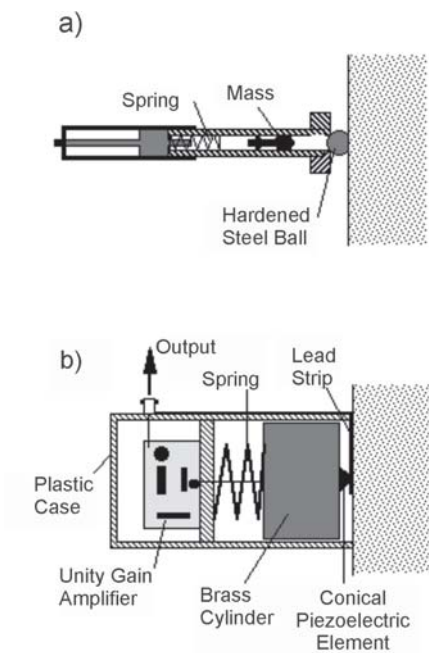



**Fig. 4.5.2.2** - (a) Spring-loaded mechanical impactor ; (b) Conical displacement transducer in housing to permit testing vertical surfaces

The force-time history of the impact may be approximated as a half-sine curve, and the duration of the impact is the contact time. The contact time is an important variable because it determines the size of the defect which can be detected by impact-echo testing. As the contact time decreases, smaller defects can be detected. However, as contact time decreases the penetrating ability of the stress waves also decreases. Thus the selection of the impact source is a critical aspect of a successful impact-echo test system. For the development work performed at NIST, contact times on the order of 30 to 60 microseconds were obtained by dropping steel spheres of different diameters. In later work at NIST, a commercially available impactor was adapted for use as the impact source. The components of the impactor are shown in Fig.4.5.2.2. The spherically-tipped mass is propelled by the spring-loaded device. A hardened steel ball is attached to the end of the device and contacts the test surface. The ball serves to increase the repeatability of the input pulse and shorten the contact time of the impact. Typically the impactor produces impacts with contact times ranging from 30 to 50 microseconds, depending on the characteristics of the concrete at the impact point. The commercially available impact-echo test system uses steel spheres on spring rods for the impact source. The user selects the size of the impactor based on the depth and size of flaw that is to be detected.

The receiving transducer must be capable of accurately measuring surface displacement. A special housing was built to hold the transducer so that it could be used on vertical surfaces (Fig. 4.5.2.2(b)). A thin lead strip is used to provide acoustic coupling between the transducer and the test surface.

A waveform analyzer, or computer with high-speed digital data acquisition hardware, is used to capture the transient output of the displacement transducer, store the digitized waveforms, and perform signal analysis. A suitable waveform analyzer, or data acquisition card, should have a sampling frequency of at least 500 kHz.

### 4.5.2.4 Signal Analysis Method

During the initial development of the impact-echo technique, interpretation of the recorded waveforms was performed in the time domain. This required establishing the time of impact initiation and the arrival time of the first P-wave echo. While this was feasible, it was found to be time-consuming. An alternative approach is frequency analysis of the displacement waveforms.

The principle of frequency analysis is illustrated in Fig. 4.5.2.3, which shows a solid plate of thickness T subjected to an impact-echo test.

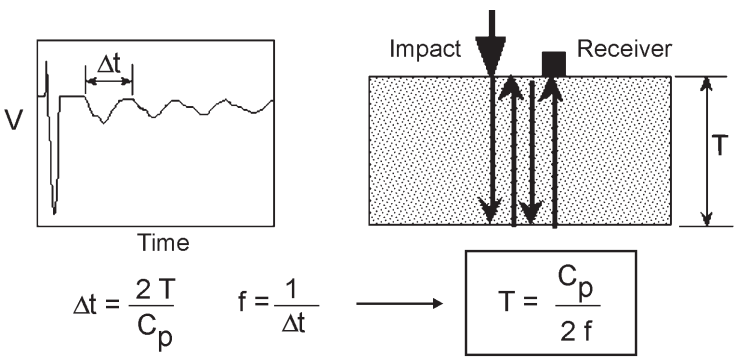



**Fig. 4.5.2.3 -** Frequency analysis is based on the principle that the P-wave undergoes multiple reflections between the reflecting surfaces. The P-wave arrives at the test surface at periodic intervals, resulting in a characteristic frequency in the waveform that is dependent on the distance to the reflecting surface.

The P-wave generated by the impact propagates back and forth between the top and bottom surfaces of the plate. Each time the P-wave arrives at the top surface it produces a characteristic displacement. Thus the waveform is periodic, and the period, t, is equal to the travel path, 2T, divided by the P-wave speed. Since frequency is the inverse of the period, the frequency, fp, of the characteristic displacement pattern is:

fp = Cp/2T **Eq. (1)**

Thus, if the frequency of an experimental waveform can be determined, the thickness of the plate (or distance to a reflecting interface) can be calculated:

$T = Cp/2fp$ **Eq. (2)**

Note that Eq. (2) is an approximation that is suitable for most applications in plate-like structures. When using the method to measure plate thickness, a correction factor is needed. For prismatic members, the value of the correction factor depends on the geometry of the member. In practice, the frequency content of the recorded waveforms is obtained using the fast Fourier transform (FFT) technique to obtain the amplitude spectrum. Appendix A gives additional background information on digital frequency analysis.

### 4.5.2.5 Illustrative Example

Figure 4.5.2.5 illustrates how frequency analysis is used in impact-echo testing. In Fig. 4.5.2.5(a) an impact- echo test was performed over a solid portion of a 0.5-m thick concrete slab. In the amplitude spectrum there is a frequency peak at 3.42 kHz. This frequency corresponds to multiple reflections between the bottom and top surfaces of the slab. Using Eq. (1) and solving for Cp, the P-wave speed is calculated to be 3410 m/s. Figure 4.5.2.5(b) shows the amplitude spectrum obtained from a test over a portion of the slab containing a disk-shaped void. The peak at 7.32 kHz results from multiple reflections between the top of the plate and the void. Using Eq. (2), the calculated depth of the void is 3410/ (2 * 7320) = 0.23 m, which compares favorably with the known distance of 0.25 m.

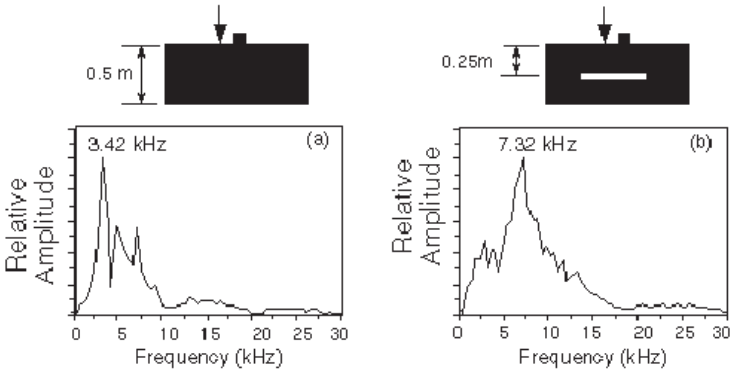



**Fig. 4.5.2.5 -** Examples of amplitude spectra : a) for a test over a solid portion of a 0.5- m thick concrete slab; b) for a test over a disk-shaped void embedded in another portion of the same slab.

Appendix A explains that the resolution in the amplitude spectrum, that is, the frequency difference between adjacent points, is equal to the sampling frequency divided by the number of points in the waveform record. This imposes a limit on the resolution of the depth calculated according to Eq. (2). Because depth and frequency are inversely related, it can be shown that for a fixed resolution in the frequency domain, the resolution of the calculated depth improves as the frequency increases, that is, as depth decreases

### 4.5.2.6 ASTM Standard

In 1998, ASTM adopted a standard test method on using the impact-echo method to measure the thickness of concrete members (ASTM C 1383, Standard Test Method for Measuring the P-wave Speed and Thickness of Concrete Plates Using the Impact-Echo Method). The method involves two procedures. Procedure A is to determine the P-wave speed in the concrete by measuring the travel time between two surface receivers separated by a known distance. Procedure B is to measure the thickness of the member by measuring the thickness frequency using the impact-echo procedure. The method is applicable to

plate-like structures in which the smallest lateral dimension is at least six times the thickness of the member. The standard includes procedures for estimating the systematic errors associated with the thickness measurement due to the digital nature of the data. In the background research leading to the standard, researchers at Cornell University found that Eq. (2) has to be modified by a factor of 0.96 to correctly estimate the thickness based on the measured P-wave speed from Procedure A and the measured frequency from Procedure B.

### 4.5.2.7 APPENDIX A -- FREQUENCY ANALYSIS:

An impact on the top surface of an infinite plate results in multiple reflections of stress waves between the top and bottom surfaces. The multiple reflections give a periodic character to the displacement response at points close to the impact point. In finite solids containing flaws, multiple reflections occur between a variety of interfaces and free boundaries. As a result, time domain waveforms become complex and difficult to interpret. However, if the waveforms are transformed into the frequency domain, multiple reflections from each interface become dominant peaks in the amplitude spectrum -- at frequency values corresponding to the frequency of arrival of reflections from each interface. These frequencies can be used to calculate the location of the interface at each test point. It has been found that, for impact-echo testing, data interpretation is much simpler and quicker in the frequency domain than in the time domain.

The transformation from the time to the frequency domain is based on the idea that any waveform can be represented as a sum of sine curves, each with a particular amplitude, frequency, and phase shift. This transformation is carried out using the principles of the Fourier transform. As an example, Fig. A1 (a) shows the digital time domain waveform, g(t), given by the function:

**g(t) = sin (2 (pi) 20 t) + sin (2 (pi) 40 t) + sin (2 (pi) 60 t)** Eq. (A.1)

**where t = time, s.**

This function is composed of three sine curves of different amplitudes having frequencies of 20, 40, and 60 Hz.

The digital sample in Fig. 4.5.2.7(a) is made up of discrete points. The time interval between points is 0.001 seconds; this is equivalent to a sampling frequency of 1000 Hz.

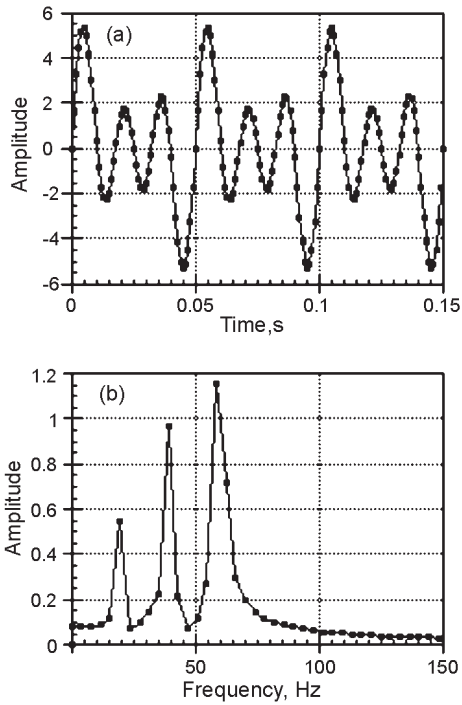



**Fig. 4.5.2.7 -** Example of frequency analysis using the fast Fourier transform technique : a) digital waveform, b) initial portion of corresponding amplitude spectrum.

The objective of frequency analysis is to determine the dominant frequency components in the digital waveform. This is most easily accomplished by using the fast Fourier transform (FFT)

technique. The FFT results can be used to construct the amplitude spectrum, which gives the amplitudes of the various frequency components in the waveform. The amplitude spectrum obtained by the FFT contains half as many points as the time domain waveform, and the maximum frequency in the spectrum is one-half the sampling rate, which for this example is 500 Hz. Figure 4.5.2.7(b) shows the initial portion of the computed amplitude spectrum; the peaks occur at 20, 40, and 60 Hz. Each of the peaks corresponds to one of the component sine curves in Eq. (A.1).

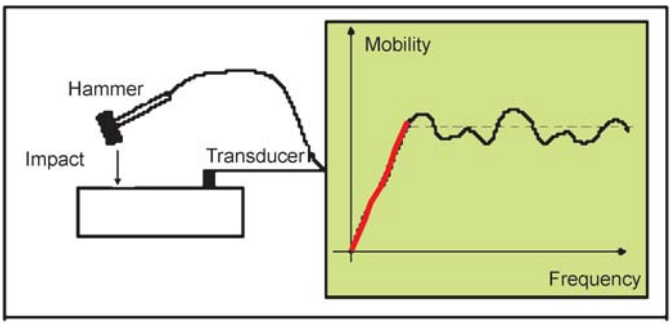
In the FFT technique, the frequency interval, $\Delta f$, in the spectrum is equal to the sampling frequency divided by the number of points in the waveform. For this example, there are 256 points in the complete time domain waveform, and the frequency interval is equal to 1000 Hz divided by 256, or 3.9 Hz. Since the frequency interval is proportional to the sampling frequency, a slower sampling rate enhances resolution in the frequency domain. However, slower sampling rates leads to longer record lengths which can result in complex spectra due to reflections from side boundaries of the test object. Experience has shown that a sampling frequency of 500 kHz with 1024 points per record is desirable in most applications.

### 4.5.3 Impulse Response Method:

Impulse response equipment is used to produce a stress wave in the considered component. The stress wave may e.g. be produced by an impact with an instrumented rubber tipped hammer. The impact causes the component to act in bending mode. A velocity transducer placed adjacent to the impact point measures the response of the component.

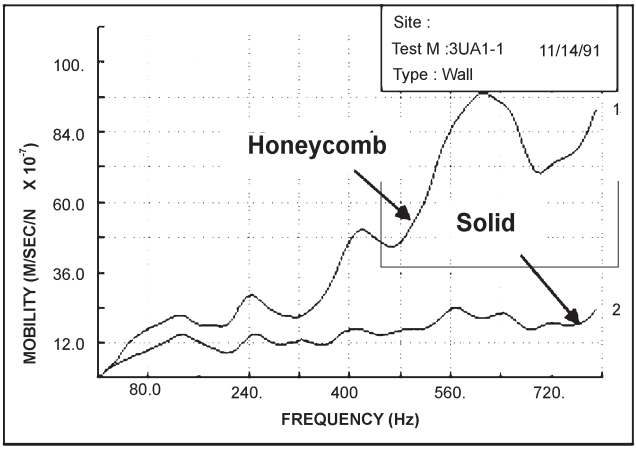
In contrast to the Impact-Echo method the impulse response equipment does not measure the reflection of the impact. Furthermore, the impact used to produce the response is considerably larger than the impulse used for the Impact-Echo method.

The hammer used to produce the impact and the transducer used to measure the response of the component are both connected to a laptop PC. The laptop performs a spectral analysis of the impact as well as the response. Dividing the resultant velocity spectrum by the force spectrum then derives the "mobility". An example of a mobility graph is shown below.

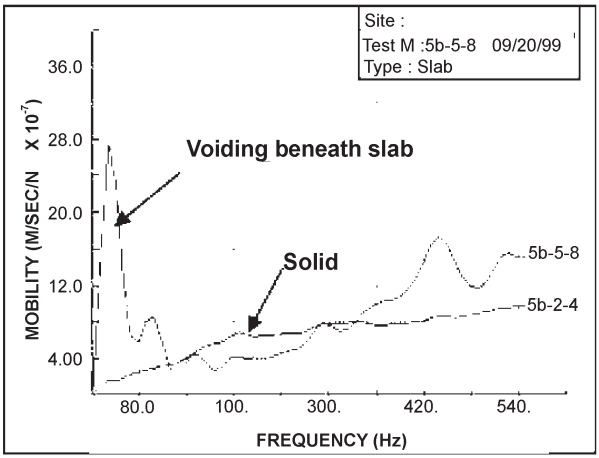



For each measurement the resulting mobility graph is shown. On the basis of the mobility graph the following parameters are determined:

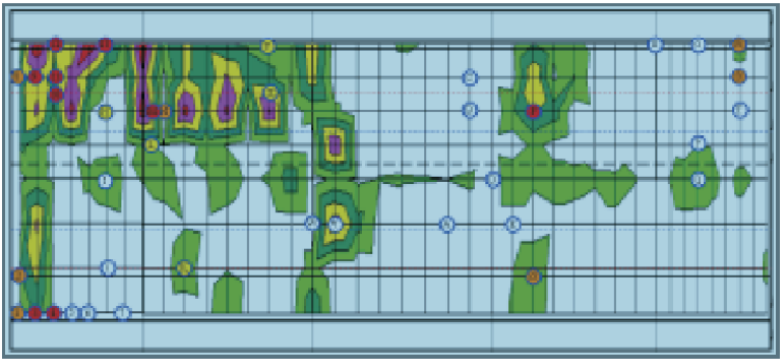
- **Average mobility:** The average mobility is shown as the green line in the figure above. The average mobility depends on the thickness of the material. If the thickness is reduced the average mobility is increased. This implies that laminated concrete has a higher average mobility than non-laminated concrete.
- **Stiffness:** The stiffness is determined as the inverse of the inclination of the part of the mobility graph below 80 Mz, the red line in the figure above. The stiffness depends on the stiffness of the material, the thickness of the material and it depends on how the component is supported. Based on a comparison of the stiffness at a number of different locations potential "weak" areas may be located.
- **Mobility slope:** The presence of honeycombs in the concrete will reduce the damping of the signal. This implies that the mobility graph will be increasing within the considered frequency range, see figure below.




Voids index: The voids index is defined as the ratio between the initial maximum of the mobility and the average mobility. If the component is laminated the initial maximum of the mobility will be considerably higher than the average mobility. If the voids index is higher than 2 – 4 it indicates a potentially “weak” area, see figure below.



The impulse response method is a fast method which may be used to screen a relatively large area within a short period of time. The equipment delivers surface graphs of the measured parameters. In the figure below a surface graph of the average mobility of a bridge deck is shown.



The results of the impulse response testing shall always be calibrated on the basis of e.g. cores, break-ups or a visual inspection using a boroscope. The locations of these tests are selected on the basis of the surface graphs of the measured parameters.

The method is used mainly for testing of piles. Testing of piles by this method is covered in ASTM test method D 5882 and is known as transient response method.

## 4.6 Crack Detection Microscope:

It is a high quality product designed for measuring crack width, both in concrete and other materials. The high definition microscope is connected to an adjustable light source which provide a well – illuminated image under all working conditions. The image is focused by turning the knob at the side of the microscope and the eye-piece graticule can be rotated through $360^0$ to align with the direction of the crack under examination. The 4 mm range of measurement has a lower scale divided into 0.2 mm divisions. These 0.2 mm divisions are

sub-divided into 0.02 mm divisions. Current Codes of practice, state that calculated maximum crack widths should not exceed certain values: e.g.0.3 mm in BS 8110 : Part 2 for most types of environment . This value is 15 divisions on the graticule.

The Crack Detection microscope is very easy to use and comes with simple instructions in a wooden carrying box.

Specifications:

| | |
|---|---|
| Magnification | = 40 times |
| Measuring Range | = 4 mm |
| Divisions | = 0.02 mm |
| Weight including battery and box | = 560 gr |
| Dimensions of box | = 150x100 50 mm deep |

**Advantages :**

It can measure the width of fine cracks. Can measure visible cracks only.

**Disadvantages :**

It can not measure the depth of cracks.

**Operational Procedure :**

Place the scale of Crack Detection Microscope on the crack. Read the number of divisions which falls on the crack.

Multiply the number of division by L.C. of the microscope. This will give the width of the crack in mm. L.C. stands for least count and its value is 0.02 mm.

## 4.7 Boroscope:

This method can be used for concrete, steel and masonry structures. The method is most commonly used on concrete and masonry structures.

A boroscope is used to look inside inaccessible or small voids. For example, if cable ducts are not injected, it is possible to inspect the strands by means of an endoscope through a contact drilling (here a drilled hole from the surface to the cable duct).

For steel structures the method is usually used for investigation of closed profiles to gain information regarding the condition of the interior surfaces of the closed profiles.

For masonry structures the boroscope can be used to gain information of the depth of the outer layer of bricks or natural stones and it can provide information of the filling material in between the arches. It may also be used to examine the mortar between bricks or natural stone.

The boroscope equipment includes a lighting source and a fibre optic cable to transfer the light to the boroscope.

A system of lenses enables the boroscope to be used as a monocular. A camera or video camera can also be mounted on the boroscope for photo documentation.

Generally speaking, the method is appropriate and may also be used for inspections of structural components such as expansion joints, honeycombs and cracks/slots.

The many variations and features which can be obtained for boroscopes make them an almost universal tool for internal inspections. These include a wide range of lengths and diameters, solid tubular or flexible bodies, lenses for forward, sideways or

retro viewing, still and video camera attachments, and mains or battery power supplies.

### 4.7.1 Specification

Video Boroscope Remote High Resolution CCD camera is comprising of three items as below.

**(i) Videoscope 3M 6MM dia**

Industrial videoscope with diameter 6 mm, length 3m, tungsten sheet, deflection 150 degree (up/down) and 120 degree (right/left), direction of view 0 degree, field of view 50 degree, consisting of VP06030AE Videoscope 3m x 6 mm dia, V0625WAE Interchangeable lens 0 degree, 80135A Carrying case

**(ii) Interchangeable lens 90 degree**

Interchangeable lens 90 degree, interchangeable lens for videoscope with 6 mm diameter. Direction of view 100 degree, depth of field 1 to 15 mm.

**(iii) Portable Pack , PAL**

Consisting of 81040020 Portable pack base unit, PAL 81040030 Portable pack battery 81040031, Portable pack battery charger 81040040 Portable pack carrying strap 81040045 Support cover type 1 81040050 Support cover type 2 81040055 Portable pack soft carrying bag 88003001 C Compact – flash card 128 MB, 88002013 Card reader.

**Portable Pack**

24W Xenon light source, 6.4” TFT monitor, integrated battery with life of 2.5 hrs, 1GB CF card as standard for image recording

### 4.7.2 Operational Procedure :-

Each company specifies the operational procedure. The operational procedure of KARL STORZ Borescope is given for information as below.

**(i) System startup**

Insert video connector into the camera port of the camera

control unit / the KARL STORZ TECHNO PACK.

Insert light connector into the light tapping point of the cold light projector / the KARL STORZ TECHNO PACK.

Turn on camera control unit / KARL STORZ TECHNO PACK, light source and monitor (if a separate monitor is used).

**(ii) System Shutoff**

Turn off monitor (if a separate monitor is used), light source and camera control unit / KARL STORZ TECHNO PACK.

Disconnect light connector from the light tapping point of the cold light projector / the KARL STORZ TECHNO PACK.

Disconnect video connector from the camera port of the camera control unit / the KARL STORZ TECHNO PACK.

**(iii) White Balance**

The white balance adjusts the colour reproduction of the system to match the colour temperature of cold light projector used. The white balance remains in memory after the base unit is turned off. When the system is started again , a new white balance must be set whenever a different light source or a different lamp or a different cable is used.

Point VIDEOSCOPE towards a white surface while the camera control unit is turned on. Ensure that no objects with other colour can be seen in the image section.

**(iv)** Press the white balance button ( blue function button on hand-piece ) for a long time. If the white balance is correct, the screen display is briefly inverted. If this does not happen, the white balance was not completed correctly. Either too much or too little light was received. In this case, increase or, as the case may be, decrease the distance between the VIDEOSCOPE tip and the white surface.

**(v)** Repeat the white balancing.

**• Controlling the viewing direction**

**(vi)** The viewing of the VIDEOSCOPE may be adjusted by deflecting the movable tip in four directions.

**(vii)** By rotating the larger of the two hand wheels on the handpiece, the movable tip of the VIDEOSCOPE is deflected downward or upward. The viewing direction that was set either downward or upward can be locked in with a locking lever between the large hand wheel and the handpiece.

**(viii)** By rotating the smaller of the left or right wheels on the handpiece, the movable tip of the VIDEOSCOPE is deflected to the left or right. The viewing direction that was set either to the left or right can be locked in with the rotary dial on the small hand wheel.

## 4.8 Nuclear Methods:

Neutron Moisture Gauges- These are used to measure moisture content in concrete. They are based on the principle that hydrogen containing materials (water) act as excellent moderators for fast neutrons, i.e. such materials produce a rapid decrease in neutron energy, depending on amount of hydrogen. Thus, counting of the slowed down neutrons gives a measure of the hydrogen content of the concrete. Isotopic neutron sources (such as radium with beryllium) are generally used in moisture gauges.

## 4.9 Structural Scanning Equipment:

### 4.9.1 Introduction:

Structural scanning equipment consists of SIR –3000 Ground Penetrating Radar System, High resolution 1500 MHz antenna, handy cart, RADAN 6.0 processing & interpretation software, 3D Quick DRAW software with, Structure Scan and Bridge Deck evaluation Modules and data collection grid. Structure Scan is a complete concrete inspection system. The system is designed to be user-friendly, just four simple steps. Set up survey

area, collect the data and view it in real time. Download the data and create 3D image for easy and accurate analysis. The Structure Scan System is a very rugged, compact and lightweight system. Flash memory data transfer capability (up to 1 GB), high – resolution color monitor, very easy to operate with single person operation. No laptop is required in the field for data acquisition, initial data processing steps and viewing the data.

### 4.9.2 Ground Penetrating Radar:

Ground penetrating radar (GPR) is a geophysical imaging technique used for subsurface exploration and monitoring. It provides an ideal technique for concrete evaluation. Concrete evaluation studies utilizing GPR include the inspection of various foundation floor systems such as structurally suspended slabs, post tensioned or conventionally reinforced slab-on-grade foundation systems, retaining walls, decks, tunnels, balconies and garages. Ground penetrating radar covers a wide area in a relatively short period of time for concrete evaluation studies. Due to recent hardware and software advances, real time cursory analysis can be performed at the site. Because of these and other reasons; GPR has become an increasingly attractive method for the engineering in particular for shallow, high resolution applications such as concrete evaluation studies. Standard test methods and guides involving GPR have been derived by the American Society for Testing and Materials (ASTM). ASTM D 4748-87 is a standard test method for the exploratory use of GPR for the determination of pavement layer(s) thickness. A more recent and broad guide for GPR usage for subsurface investigation is standardized in ASTM D 6432-99. This ASTM guide provides a compendium of related GPR information useful for a wide range of applications including concrete evaluation studies.

GPR is a non-destructive technique that emits a short pulse of electromagnetic energy, which is radiated into the subsurface. When this pulse strikes an interface between layers of materials with different electrical properties, part of the wave reflects back, and the remaining energy continues to the next interface. GPR evaluates the reflection of electromagnetic waves at the interface between two different dielectric materials. The

penetration of the waves into the subsurface is a function of the media relative dielectric constants ($\in$). If a material is dielectrically homogeneous, then the wave reflections will indicate a single thick layer.

Ground penetrating radar directs electromagnetic energy into the subsurface. The propagation of electromagnetic energy is described by Maxwell's equation with the electric component (E) orthogonal to the magnetic component (H) (Reynolds, 1997). For concrete evaluation studies, both components are equally important. Concrete material is a low conductivity, non-metallic medium that is ideal for GPR signal propagation. However, concrete typically has steel reinforcement, which is a metallic and therefore completely reflects the GPR signal and shadows anything directly below the metal. If applicable, the sub-base beneath a concrete unit is non-metallic. The sub base may be highly conductive soils (such as expansive clays) that effectively attenuate the GPR signal propagation thereby limiting depth penetration. The relative dielectric constant ($\in$) of non-metallic medium is a function of three different materials within the medium- solid, fluid and gas (Lytton, 1995). Therefore, for example, the relative dielectric constant for an unsaturated soil is a combination of the relative dielectric constant of the air, relative dielectric constant of water, relative dielectric constant of soil, porosity and degree of saturation.

The velocity in which electromagnetic energy propagates through any medium is a function of the relative dielectric property, speed of light (c=0.3 meters/nanosecond) and magnetic permeability ($\mu$). The magnetic permeability is equal to one ($\mu$ =1) in a non-metallic medium and therefore is not a factor for the wave propagation velocity. Wave propagation velocities (V) through a given medium are important to convert a time domain radargram model into a distance domain radargram model.

#### 4.9.2.1 Operation Principles:

There are several antenna manufacturers, antenna types,

signal pre- and post- setting options; operating frequencies software packages, etc. to consider for a specific application within the engineering and construction industry, geological, environmental and/or archaeological fields. Each radar system must be designed to meet the objective(s) of a given project. For concrete evaluation studies, there are several options available – all of which have certain advantages and disadvantages. For the evaluation of various concrete structures, which include streets/highways, parking lots, bridge decks, pools, tilt wall panels, sidewalks, various foundation systems and retaining walls, a versatile and highly portable radar system with a ground coupled, monostatic antenna is suitable. However, for specialized projects, such as road condition evaluation, an air launched (horn) antenna is commonly used due to the efficient data collection, characteristic of this antenna. Currently, GPR data can be collected with this air-launched antenna at highway speeds.

A typical radar system for concrete evaluation studies generally consists of a control unit (computer), pulse generator, transmitting and receiving antennae and video monitor. A bistatic antenna describes a radar system with two antennae, one to transmit and the other to receive. An antenna that both transmits and receives is defined as a monostatic antenna. There are advantages and disadvantages of each antenna type for a given application; however, for concrete evaluation studies, monostatic antennae are typically more advantageous due to higher data collection and processing efficiency.

GPR is a non-destructive technique that uses electromagnetic (EM) waves to “look” into a material. GPR systems operate in a similar manner to sonar – i.e. by emitting a series of brief pulses and estimating distance to objects from the time it takes to detect reflections. Figure 4.9.2.1 shows a schematic of a GPR system in operation. Transmitting and receiving antennas are used to emit the EM pulse and detect the reflections.

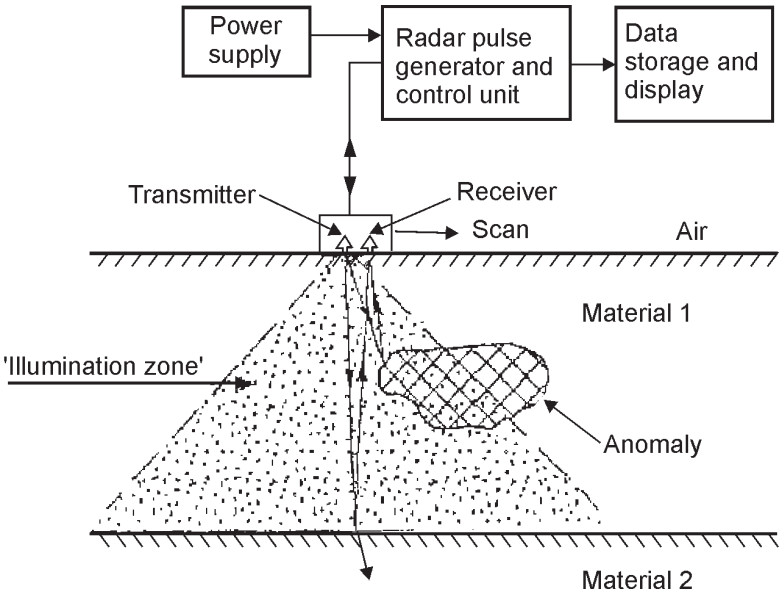



**Fig. 4.9.2.1 - Schematic of GPR operation**

Reflections occur when the EM wave passes from one material into another material with contrasting electrical properties. The strength of the reflection depends on the electrical contrast between materials (i.e. a strong contrast produces a strong reflection).

The GPR records the strength of reflections detected for a set duration after each pulse. A plot of this data is called a “trace”. Figure 4.9.2.2 shows a typical trace and illustrates how the EM reflections correspond to the material boundaries.

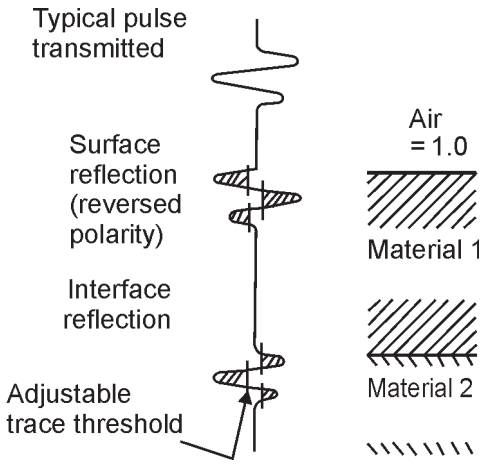



**Fig. 4.9.2.2-GPR "Trace" and corresponding material boundaries**

As the EM waves travel very quickly, the time duration the GPR 'listens' for after each pulse is very brief. The trace signal is usually plotted against a time scale measured in nanoseconds. (i.e. billionths of a second).

GPR systems generate a rapid succession of traces, which are displayed as a "radargram". A radargram is simply a display of one trace after another with the intensity of the reflected signal represented by different colours or shades of grey. Figure 4.9.2.3 shows an unprocessed radargram of the end of a girder shown with a grey scale palette.

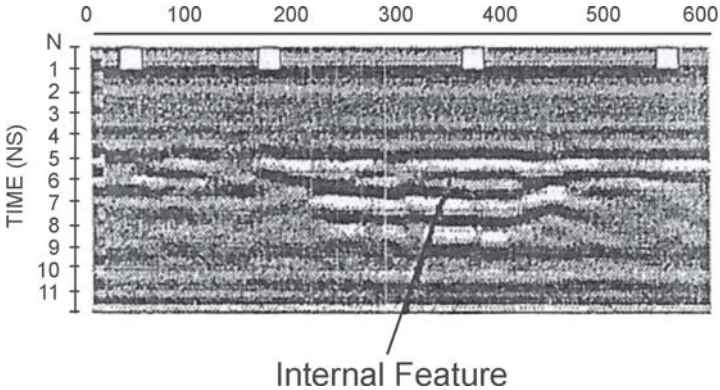



**Fig.4.9.2.3-Unprocessed Radargram**

In figure 4.9.2.3 the black line near the top is the near side of the girder and the far side of the girder is around 9ns (somewhat difficult to see). The location of an internal feature has been highlighted. The white squares near the top are markers that were entered at metre intervals.

The depth to the feature can be estimated as follows:

$D=V_r\ t_r\ /2$ Eqn. (1)[2]

Where $V_r$ = Velocity of the EM wave

$t_r$ = time to detect reflection

d = depth to feature

The velocity ($V_r$ ) of the EM wave is dependant on the material properties, but is fairly constant in sound timber. As the thickness of the girder is known from site observation, the average velocity can be calculated from the time it takes for the signal to reflect from the far side of the girder.

The appearance of the radargram can often be improved by signal processing of the raw data. Figure 4.9.2.4 shows the same radargram shown in Figure 4.9.2.3 after basic signal processing, including corrections for distance along the girder, zero depth and with the depth scale shown.

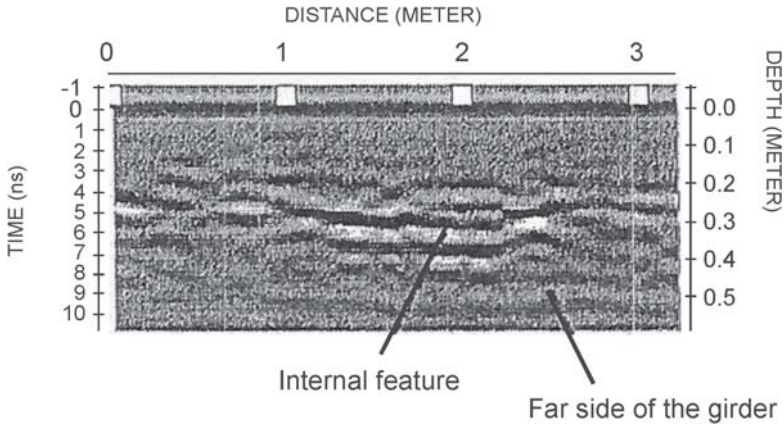



**Fig.4.9.2.4-Processed radargram**

After signal processing it is now easier to see the far side of the girder, which was previously obscured because of signal artifacts.

### 4.9.3 Performance and Other Considerations:

GPR surveys, signal processing and interpretation of the results should be undertaken by personnel experience in using these techniques. Poorly configured systems will lead to poor results.

Interpretation of the results is often an involved process. Successful interpretation requires a good understanding of the underlying principles and limitations of these techniques.

GPR is suitable for investigating relatively non-conductive materials such as concrete, timber and road pavements. It is not suitable for investigating materials with high conductivities such as metals, or materials with high moisture contents.

As GPR uses very weak non-ionising radiation there are no particular safety issues for site personnel (i.e. similar but much weaker than a mobile phone)

### 4.9.4 Bridge Decks:

On bridge decks, various issues can be addressed with the help of GPR. Some examples are asphalt pavement thickness, concrete cover of re-bar, position of pre-stressing-tendons or tendon ducts and bridge deck deterioration.

### 4.9.5 Concrete Cover Assessment Technique for New Bridge Decks:

A procedure for analyzing ground penetrating radar (GPR) data to accurately locate and determine the depth of reinforcement bars (REBAR) in new concrete bridge decks. This technique involves collecting GPR data with a ground coupled antenna attached to a distance encoder. The GPR data is subsequently transferred to a processing computer. One calibration core is then drilling to a rebar located along the data collection profiles line and the depth of the top of the rebar with respect to the concrete surface is recorded. The location and depth of the rebar in the calibration hole are entered into a data processing program which performs the following:

(1) Locates the horizontal position of all the rebar with respect to the start of the profile line.
(2) Calculates the depth of the top of each rebar using the calibration core information.

### 4.9.6 Conclusions:

GPR provides an efficient and versatile means for concrete evaluation studies. Ideal electrical properties of concrete make exploratory studies using GPR extremely efficacious. Data collection and on-site cursory analysis are increasingly becoming easier with recent hardware improvements. More importantly, readily available GPR software has improved significantly, in particular for the concrete evaluation usage. Significant research and development has recently been applied to the determination of density and water content of each layer using GPR (Lytton, 1995). This is an added benefit for concrete evaluation studies. Three-dimensional modeling of GPR data is relatively new, but recent software advances using 3D processing and modeling are

becoming more feasible and user-friendly. Current concrete evaluation studies involve the identification, qualification and/or quantification of reflected GPR signatures. These GPR signatures include, but may not be limited to, reflection strength, signal polarity, two-way travel time, signal attenuation and hyperbolic reflection, which is necessary for subsurface feature identification and/or delineation.

### 4.9.7 Details of Equipments:

**(A) Structure Scan (SIR-3000)**

SIR 3000 is the newest GPR product from the world leader in Ground Penetrating Radar. This rugged, high-performance, single-channel radar system provides unrivaled scan rates with low noise.

- Rugged, compact and lightweight system (Weight 4.1kg including Battery, dimension 12.4” x 8.7” x 4.1”)
- Flash Memory data transfer capability,
- High resolu tion color monitor,
- Easy to use operating system,
- Simple single-person operation,

**Features:**

- Inspect floors, decks, slabs, tunnels, and balconies.
- Locate rebar, tension cables, conduits, voids, PVC pipes, and measure slab thickness.
- Locate a target depth of 18 inches and more in concrete.
- Detect and map the relative concrete condition for rehab planning.

**(B) SIR SYSTEM ANTENNAS**

The structure Scan Standard system comes with a very high resolution antenna model 5100 of Central frequency 1500 MHz that is specially configured for access to small areas up to a depth range of 0 – 18”.

**Built for durability and reliability**

- Rugged, military-style connectors.
- Coated, sealed electronics.

- Shielded to eliminate above-ground interference.
- All temperature conditions, $20^0$C to $50^0$C.
- Low resistance, long-life replaceable wear skids.
- Rugged, high-density molded cases.
- Heavy duty cable.

**Physical Properties:**

- Depth Range 0-0.5m (0-18in),
- Dimensions 3.8 x 10 x 16.5 cm,
- Weight 1.8 kg (4 lbs),

**(C) SOFTWARE**

**(i) RADAN 6.0 Processing & Interpretation Software**

It is Advanced Post Processing Software that works on Windows-XP and Windows 2000 Professional Program, projects processing which includes 16 million colors, horizontal scaling, distance normalization, surface normalization, static corrections, zero position adjustments, arithmetic functions, range gain, gain restoration, vertical and spatial filters, predictive deconvolution, 2D constant and variable velocity migration, interactive interpretation and structure identification.

Now RADAN for Windows has applications specific add-on modules with more new features and capabilities. Some of them are as below:

**(ii) Software 3D Quick Draw Mapping Module**

This add-on module features 3D presentations of data with simple manipulations of the entire data “cube” so that it can be “sliced and diced” along various x-y, y-z, or x-z planes.

This module also uses some of the variable velocity migration capabilities that are featured in the RADAN main program to appropriately “size” point targets that appear like hyperbolic shapes in the raw data.

By first performing a migration operation, the final 3-D data set is now ready for linear feature recognition-a capability that enables the module to assist in identification and display of linear

features such as walls or utilities that may be embedded in the earth.

**New Features:**

- Ability to create multi-segmented “pipes”
- Remap color transform, contrast and gain in real time
- New improved form for easy assembly of 3D survey data Stretch survey in any direction: X, Y and Z
- Movie Mode – Easy control of movie mode in Z, Y or Z directions. Automatically “slice” through your data for easy target identification.
- Simulated Borehole toll

**(iii) Structural Identification Module**

The Structure Identification module is the heart of GSSI?s Structure Scan systems. This powerful tool allows for easy creation of planview slices to aid in interpretation of Structure Scan data files.

The versatility of this module allows for a broad range of civil/structural applications, including structures with different types of reinforcement. It can also be used to automatically find point targets, such as utility crossings or archaeological features with lower frequency antennas.

- Semi-automatic mapping of rebar locations and depths on simple concrete structures.
- Interactive location mapping of conduits within concrete structures.
- Semi-automatic mapping of deterioration zones within concrete structures.

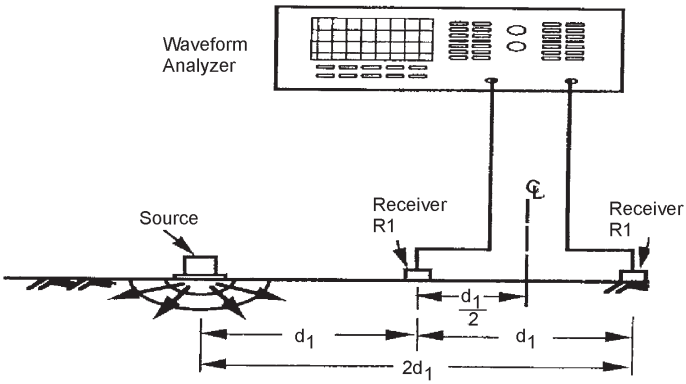
**(iv) Bridge Assessment Module**

Capable of identifying rebar and calculating concrete cover over rebar on new deck structures; performs deterioration-mapping using GSSI’s patented data analysis method, and is designed so that post-processing and analysis are streamlined specifically for bridge deck data. Perfect software for large bridge deck structures with typical two-layer orthogonal grid reinforcement patterns.

## 4.10 Spectral Analysis of Surface Waves for Unknown Foundation:

**4.10.1 Introduction:** Spectral Analysis of Surface Wave (SASW) method is a NDT method used for determining the depth of unknown foundations in bridges and structures. Earlier this method was used to determine the shear wave velocity profile versus depth from ground surface without drilling a boring. Subsequently, SASW tests are useful in determining depth of unknown foundations of more massive abutments, piers and footing provided the substructure geometry allows for proper access. Access for SASW test in term of unknown bridge foundations means that the foundation is more massive and has an exposed fairly flat ledge or top surface on which impacts are applied and a pair of receivers placed. Lately, SASW testing has been adopted for offshore/underwater use.

**4.10.2 Principle:** This method is based on the principle that foundation sub-structure materials have different stress wave velocities (stiffness) than the underline supporting soil and bed blocks, which typically have slower velocities, i.e. they are less stiff than foundation material. SASW method measures variation in surface wave velocity with depth in layered material. The bottom depths of exposed substructures or footings are indicated by slower velocities of surface wave travel in underlying soils. The brief description of the method is given in the figure below:




The SASW method has unique capabilities to nondestructively determine layer thicknesses and velocity (stiffness) versus depth for soft over stiff over soft layers that other methods such as Seismic Refraction are not capable of doing unless velocity increases with depth. One advantage of the SASW method for investigation unknown foundation depths of bridges is that measurements are performed using a source and two receivers which can be placed on top of a horizontal surface such as the exposed surface of an abutment.

**4.10.3 Process & Method:** A source and two receivers placed in a line on surface such that distance from Source to first receivers (d1) is equal to distance between two receivers. Testing is performed by impacting the surface and recording the passage of predominant Rayleigh (surface) wave energy past the two receivers.

A series of receiver spacing is used and testing is performed in forward & reverse directions at each receivers spacing.

Surface A dynamic Signal analyzer is used to capture and process receiver outputs. Time domain outputs are then transformed to the frequency domain using a Fast Fourier Transform. This is used to calculate cross power spectrum between two receivers. The surface wave velocity and wavelength

associated with each frequency are then calculated and plot of surface wave velocity versus wavelength, called dispersion curve, is prepared. Wave velocity is calculated from phase plots (for one receiver spacing).

By dispersion curve, shear wave velocity profile of structure or soil being tested can be obtained by process called forward modeling. A Computer programming for forward modeling has been developed. A good match obtained between experimental and theoretical dispersion curves, assumed profile is considered to be a good representation of actual profile. Accuracies of velocity profiles and layer thickness vary with variability of pavement/soil/bedrock or other layers being tested, but theoretically modeled values are typically accurate to within 10-15% of actual values.

### 4.10.4 Applications:

The SASW tests has applications for unknown foundation depth determination where flat, wide structure access is available for geometry determination of abutment wall thickness and exposed footings/pilecaps, for determining substructure material properties v.s. depth, and for measurement of the variation of stiffness (velocity) of soil and bedrock with depth. The equipment for SASW method includes hammers from 1-lb to a 4-lb hand sledge to a 12-lb sledge hammer (vibrators can also be used), a dynamic signals analyzer, and two seismic accelerometers (or suitable geophones for greater depths and for testing of soils).

### 4.10.5 Capabilities & Uses in Unknown Foundation:

i) This method has been found to be capable of determining depth of shallow abutments, pier walls and other solid substructure with a flat surface from which testing can be performed. Such flat surfaces could be the top of an abutment between girders, a ledge or step, or even the top of exposed footing or pile cap.

ii) It can also be used to determine unknown thickness of abutments breast walls and wing walls, exposed footing, pile caps etc. and indicate material properties in terms of stiffness (velocity) for substructures and surrounding soils and rock.

iii) SASW method is only NDT method (method without drilling a bore) to provide data on change in stiffness of foundation materials with depths.

### 4.10.6 Limitations:

Main limitation of method is geometric. Flat access is required to generate surface wave energy. Maximum foundation depths that can be determined are estimated to be not much deeper than longest test horizontal surface on the tested substructure. Substructure must be solid for surface wave energy to travel down through it and interact with underlying soil.

■ ■ ■

# CHAPTER-5

## NON-DESTRUCTIVE TESTING OF STEEL BRIDGES

## 5.1 Introduction

Flaws and cracks can play havoc with the performance of structures and for improving the performance, the timely detection of these defects is very much necessary. Our present system of inspection of bridges mainly emphasize on the visual inspection which does not give correct picture of internal structural defects. By using non-destructive testing methods, the structures can be evaluated to a greater degree of accuracy, without damaging them. These methods can be used as quality control measures at the time of construction of structures as well as tool for detection of defect during the service. The biggest advantage of NDT methods is that these are quick and large no. of structure can be covered to evaluate their in service performance without causing any damage to the structure. In this chapter, only the following methods which are commonly used for evaluating the steel structures, are discussed –

(a) Dye Penetrant Inspection OR Liquid Penetrant Inspection
(b) Magnetic Particle Inspection
(c) Eddy current testing
(d) Radiography testing
(e) Ultrasonic testing
(f) Complet structral testing
(g) Acoustic Emission Techniques

## 5.2 Dye Penetrant Inspection OR Liquid Penetrant Inspection (LPI)

### 5.2.1 Introduction and History of Penetrant Testing:

Dye penetrant inspection is a method that is used to re-

veal surface breaking flaws by bleed out of a colored or fluorescent dye from the flaw. The technique is based on the ability of a liquid to be drawn into a "clean" surface breaking flaw by capillary action. After a period of time called the "dwell," excess surface penetrant is removed and a developer applied. This acts as a "blotter." It draws the penetrant from the flaw to reveal its presence. Colored (contrast) penetrants require good white light while fluorescent penetrants need to be used in darkened conditions with an ultraviolet "black light".

The advantage that a dye penetrant inspection offers over an unaided visual inspection is that it makes defects easier to see for the inspector. There are basically two ways that a penetrant inspection process makes flaws more easily seen. First, LPI produces a flaw indication that is much larger and easier for the eye to detect than the flaw itself. Many flaws are so small or narrow that they are undetectable by the unaided eye. Due to the physical features of the eye, there is a threshold below which objects cannot be resolved. This threshold of visual acuity is around 0.003 inch for a person with 20/20 vision.

The second way that LPI improves the detect ability of a flaw is that it produces a flaw indication with a high level of contrast between the indication and the background which also helps to make the indication more easily seen. When a visible dye penetrant inspection is performed, the penetrant materials are formulated using a bright red dye that provides for a high level of contrast between the white developer that serves as a background as well as to pull the trapped penetrant from the flaw. When a fluorescent penetrant inspection is performed, the penetrant materials are formulated to glow brightly and to give off light at a wavelength that the eye is most sensitive to under dim lighting conditions.

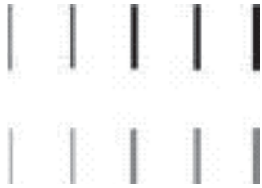


**Fig. 5.2.1: Example of different contrasts.**

### 5.2.2 Basic Processing of a Dye Penetrant Testing:

1. **Surface Preparation:** One of the most critical steps of a dye penetrant inspection is the surface preparation. The surface must be free of oil, grease, water, or other contaminants that may prevent penetrant from entering flaws. The sample may also require etching if mechanical operations such as machining, sanding, or grit blasting have been performed. These and other mechanical operations can smear the surface of the sample, thus closing the defects.

2. **Penetrant Application:** Once the surface has been thoroughly cleaned and dried, the penetrant material is applied by spraying, brushing, or immersing the parts in a penetrant bath.

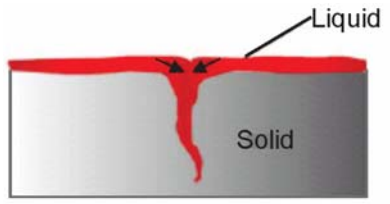



**Fig. 5.2.2.1: Penetrant application.**

3. **Penetrant Dwell:** The penetrant is left on the surface for sufficient time to allow as much penetrant as possible to be drawn from or to seep into a defect. Penetrant dwell time is the total time that the penetrant is in contact with the part surface. Dwell times are usually recommended by the penetrant producers or required by the specification being followed. The times vary depending on the application, penetrant materials used, the material, the form of the material being inspected, and the type of defect being inspected. Minimum dwell times typically range from 5 to 60 minutes. Generally, there is no harm in using a longer penetrant dwell time as long as the penetrant is not allowed to dry. The ideal dwell time is often determined by experimentation and is often very specific to a particular application.

4. **Excess Penetrant Removal:** This is a most delicate part of the inspection procedure because the excess penetrant must be removed from the surface of the sample while removing

as little penetrant as possible from defects. Depending on the penetrant system used, this step may involve cleaning with a solvent, direct rinsing with water, or first treated with an emulsifier and then rinsing with water.

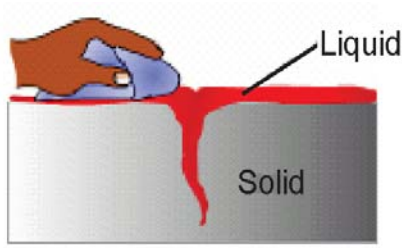



**Fig. 5.2.2.2: Excess Penetrant Removal.**

5. **Developer Application:** A thin layer of developer is then applied to the sample to draw penetrant trapped in flaws back to the surface where it will be visible. Developers come in a variety of forms that may be applied by dusting (dry powdered), dipping, or spraying (wet developers).

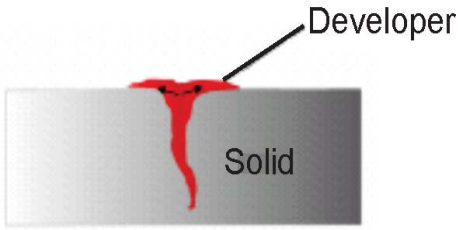



**Fig. 5.2.2.3:** ***Developer.***

6. **Indication Development:** The developer is allowed to stand on the part surface for a period of time sufficient to permit the extraction of the trapped penetrant out of any surface flaws. This development time is usually a minimum of 10 minutes and significantly longer times may be necessary for tight cracks.

7. **Inspection:** Inspection is then performed under appropriate lighting to detect indications from any flaws which may be present.

8. **Clean Surface:** The final step in the process is to thoroughly clean the part surface to remove the developer from the parts that were found to be acceptable.

### 5.2.3 Common Uses of Dye Penetrant Inspection:

Dye penetrant inspection is one of the most widely used nondestructive evaluation (NDE) methods. Its popularity can be attributed to two main factors, which are its relative **ease of use** and its **flexibility**. LPI can be used to inspect almost any material provided that its surface is not extremely rough or porous. Materials that are commonly inspected using LPI include the following:

* Metals (aluminum, copper, steel, titanium, etc.)
* Glass
* Many ceramic materials
* Rubber
* Plastics

Dye penetrant inspection is used to inspect for flaws that breaks the surface of the sample. Some of these flaws are listed below:

- Cracks
- Overload and impact fractures
- Porosity
- Laps Seams
- Pin holes in welds
- Lack of fusion or braising along the edge of the bond line

As mentioned above, one of the major limitations of a penetrant inspection is that flaws must be open to the surface.

### 5.2.4 Advantages and Disadvantages of Dye Penetrant Testing:

Like all nondestructive inspection methods, dye penetrant inspection has both advantages and disadvantages. The primary advantages and disadvantages when compared to other NDE methods are summarized below.

**Primary Advantages:**

- The method has high sensitive to small surface discontinuities.
- The method has few material limitations, i.e. metallic and nonmetallic, magnetic and nonmagnetic, and conductive and

nonconductive materials may be inspected.

- Large areas and large volumes of parts/materials can be inspected rapidly and at low cost.
- Parts with complex geometric shapes are routinely inspected.
- Indications are produced directly on the surface of the part and constitute a visual representation of the flaw.
- Aerosol spray cans make penetrant materials very portable.
- Penetrant materials and associated equipment are relatively inexpensive.

**Primary Disadvantages:**

- Only surface breaking defects can be detected.
- Only materials with a relative nonporous surface can be inspected.
- Precleaning is critical as contaminants can mask defects.
- Metal smearing from machining, grinding, and grit or vapor blasting must be removed prior to LPI.
- The inspector must have direct access to the surface being inspected.
- Surface finish and roughness can affect inspection sensitivity.
- Multiple process operations must be performed and controlled.
- Post cleaning of acceptable parts or materials is required.
- Chemical handling and proper disposal is required.

### 5.2.5 Dye Penetrant Testing Materials:

The penetrant materials used today are much more sophisticated than the kerosene and whiting first used by railroad inspectors near the turn of the 20th century. Today's penetrants are carefully formulated to produce the level of sensitivity desired by the inspector. To perform well, a penetrant must possess a number of important characteristics. A penetrant must

- spread easily over the surface of the material being inspected to provide complete and even coverage.

- be drawn into surface breaking defects by capillary action.
- remain in the defect but remove easily from the surface of the part.
- remain fluid so it can be drawn back to the surface of the part through the drying and developing steps.
- be highly visible or fluoresce brightly to produce easy to see indications.
- must not be harmful to the material being tested or the inspector.

Penetrant materials come in two basic types. These types are listed below:

- Type 1 - Fluorescent Penetrants
- Type 2 - Visible Penetrants

Fluorescent penetrants contain a dye or several dyes that fluoresce when exposed to ultraviolet radiation. Visible penetrants contain a red dye that provides high contrast against the white developer background. Fluorescent penetrant systems are more sensitive than visible penetrant systems because the eye is drawn to the glow of the fluorescing indication. However, visible penetrants do not require a darkened area and an ultraviolet light in order to make an inspection. Visible penetrants are also less vulnerable to contamination from things such as cleaning fluid that can significantly reduce the strength of a fluorescent indication.

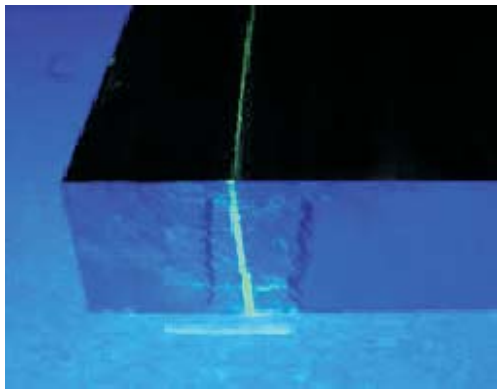


**Fig. 5.2.5: Inspection under ultraviolet light.**

Penetrants are then classified by the method used to remove the excess penetrant from the part. The four methods are listed below:

- Method A - Water Washable
- Method B - Post Emulsifiable, Lipophilic
- Method C - Solvent Removable
- Method D - Post Emulsifiable, Hydrophilic

Water washable (Method A) penetrants can be removed from the part by rinsing with water alone. These penetrants contain some emulsifying agent (detergent) that makes it possible to wash the penetrant from the part surface with water alone. Water washable penetrants are sometimes referred to as self-emulsifying systems. Post emulsifiable penetrants come in two varieties, lipophilic and hydrophilic. In post emulsifiers, lipophilic systems (Method B), the penetrant is oil soluble and interacts with the oil-based emulsifier to make removal possible. Post emulsifiable, hydrophilic systems (Method D), use an emulsifier that is a water soluble detergent which lifts the excess penetrant from the surface of the part with a water wash. Solvent removable penetrants (Method C) require the use of a solvent to remove the penetrant from the part.

Penetrants are then classified based on the strength or detectability of the indication that is produced for a number of very small and tight fatigue cracks. The five sensitivity levels are shown below:

- Level ½ - Ultra Low Sensitivity
- Level 1 - Low Sensitivity
- Level 2 - Medium Sensitivity
- Level 3 - High Sensitivity
- Level 4 - Ultra-High Sensitivity

### 5.2.6 Penetrants:

The industry and military specification that control the penetrant materials and their use all stipulate certain physical properties of the penetrant materials that must be met. Some of these requirements address the safe use of the materials, such as toxicity, flash point, and corrosiveness, and other requirements address storage and contamination issues. Still others delineate properties that are thought to be primarily responsible for the performance or sensitivity of the penetrants. The properties of penetrant materials that are controlled by AMS 2644 and MIL-I-25135E include flash point, surface wetting capability, viscosity, contact angle, color, brightness, ultraviolet stability, thermal stability, water tolerance, and removability.

How some of these properties can affect penetrant testing are described next.

**Some properties of a penetrant Capillary Action:**

Capillary action is the tendency of certain liquids to travel or climb when exposed to small openings. In nature there are many examples of capillary action. Plants and trees have a network similar to capillary tubes that draw water upward supplying nourishment. The earth brings water to the surface through the capillary action of the earth's exterior.

**Dye Penetrant Inspection:**

Capillary action is the phenomena that makes dye penetrant inspection possible. All of the steps that are taken in the process of conducting a penetrant test, from precleaning through the actual evaluation of the results, is done to promote capillary action.

**Precleaning:**

When a part is precleaned, everything is removed that will prevent the penetrant from entering discontinuities and therefore, interfere with capillary action. Once the surface is clean and dry, the penetrant is applied. The penetrant is then drawn into the discontinuities through capillary action, see figure below.

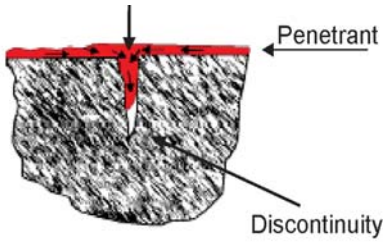



**Fig. 5.2.6.1: Penetrant entering a discontinuity through capillary action.**

If the part is not clean and dry, less penetrant and possibly none, will be drawn into the discontinuities. Discontinuities that would have been revealed may be overlooked.

**Applying Developer:**

Following the removal of the excess penetrant, a developer is applied. The developer induces reverse capillary action to take place. Penetrant is drawn from the discontinuities into the developer in the same way that the fibers of a paper towel absorb or blot a liquid, see figure below.

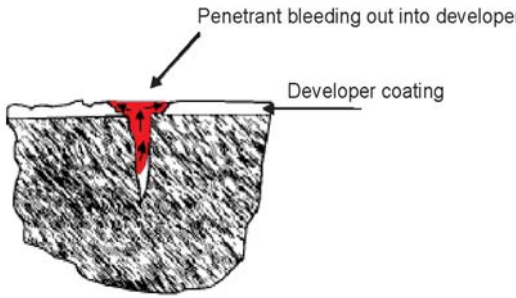



**Fig. 5.2.6.2: Blotting action of developer draws penetrant from discontinuity.**

**Surface Tension:**

There are many factors in capillary action; among these are surface tension, cohesion, wetting ability, adhesion and contact angle. Each of these factors has a strong influence in the performance of capillary action. Of these, surface tension is one of the two most important factors. Water in a pond exhibits surface tension when it supports the weight of an insect - a spider or mosquito for example. The insect is supported by a molecular membrane created by the attraction (cohesiveness) of one water molecule to another. Each water molecule is attracted laterally and vertically (above and below) to adjacent molecules. The molecules on the surface are attracted only laterally and below because of the absence of molecules above them. This change in attraction between surface molecules creates the effect of a stretched membrane on the surface of the water strong enough to support small objects. Water has high surface tension because of the strong cohesive attraction between the molecules of water. The amount of surface tension will vary between different liquids depending upon how cohesive the molecules are.

**Wetting Ability and Contact Angle:**

The second most important factor in capillary action is wetting ability. How well a liquid wets the surface of a specimen is referred to as its wetting ability. The wetting ability of a liquid is determined by the contact angle produced when a liquid meets a

surface. The cohesive force that determines surface tension competes with the adhesive properties of a liquid producing a specific degree of contact angle.

**Adhesion:**

Adhesion is how strongly the molecules of a liquid are attracted to a particular surface. If a capillary tube is placed in a beaker of water, the water will rise in the tube to a level higher than the water surrounding the tube. The water climbs in the tube because the molecules of water are more strongly attracted to the inside surface of the tube than they are to each other. The stronger the attraction between the molecules of a liquid and a surface, the smaller will be the contact angle and the higher a liquid will rise in a capillary tube.

### 5.2.7 Developers:

The role of the developer is to pull the trapped penetrant material out of defects and to spread the developer out on the surface of the part so it can be seen by an inspector. The fine developer particles both reflect and refract the incident ultraviolet light, allowing more of it to interact with the penetrant, causing more efficient fluorescence. The developer also allows more light to be emitted through the same mechanism. This is why indications are brighter than the penetrant itself under UV light. Another function that some developers performs is to create a white background so there is a greater degree of contrast between the indication and the surrounding background.

### 5.2.8 Use and Selection of a Developer:

The use of developer is almost always recommended. One study reported that the output from a fluorescent penetrant could be multiplied by up to seven times when a suitable powder developer was used. Another study showed that the use of developer can have a dramatic effect on the probability of detection (POD) of an inspection. When a Haynes Alloy 188, flat panel specimen with a low-cycle fatigue crack was inspected without a developer, a 90 % POD was never reached with crack

lengths as long as 19 mm (0.75 inch). The operator detected only 86 of 284 cracks and had 70 false-calls. When a developer was used, a 90 % POD was reached at 2 mm (0.077 inch), with the inspector identifying 277 of 311 cracks with no false-calls. However some authors have reported that in special situations the use of a developer may actually reduce sensitivity. These situations primarily occur when large, well defined defects are being inspected on a surface that contains many nonrelevant indications that cause excessive bleedout.

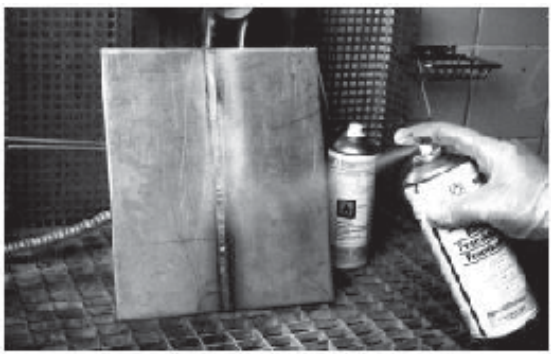


**Fig. 5.2.8: Nonaqueous Wet Solvent Developer.**

**Type of Developer Used and Method of Application:**

Nonaqueous developers are generally recognized as the most sensitive when properly applied. There is less agreement on the performance of dry and aqueous wet developers but the aqueous developers are usually considered more sensitive. Aqueous wet developers form a finer matrix of particles that is more in contact with the part surface. However, if the thickness of the coating becomes too great, defects can be masked. Also aqueous wet developers can cause leaching and blurring of indications when used with water washable penetrants. The relative sensitivities of developers and application techniques are shown in the table below. There is general industry agreement with this table, but some industry experts feel that water suspendible developers are more sensitive than water-soluble developers.

| Ranking | Developer Form | Method of Application |
|---|---|---|
| 1 | Nonaqueous Wet Solvent | Spray |
| 2 | Plastic Film | Spray |
| 3 | water-soluble | Spray |
| 4 | Water Suspendible | Spray |
| 5 | water-soluble | Immersion |
| 6 | Water Suspendible | Immersion |
| 7 | Dry | Dust Cloud (Electrostatic) |
| 8 | Dry | Fluidized Bed |
| 9 | Dry | Dust Cloud (Air Agitation) |
| 10 | Dry | Immersion (Dip) |

*Sensitivity Ranking (highest to lowest) Developer Form Application Technique.*

## 5.3 Magnetic Particle Inspection (MPI)

Magnetic particle inspection is a NDT method used for defect detection in steel structures. This is a fast and relatively easy method to apply in field. MPI uses magnetic fields and small magnetic particles, such as iron fillings to detect flaws in components. The component being inspected must be made of a ferromagnetic particle such as iron, nickle, cobalt or some of their alloys. Ferromagnetic materials are materials that can be magnetic to a level that will allow the inspection to be effective.

The method may be used effectively for inspection of steel girders and other bridge parts made of steel.

### 5.3.1 Principle

When ferromagnetic material or component (weld) is magnetized, magnetic discontinuities that lie in direction approx. perpendicular to the field direction, will result in formation of a

strong leakage field. This leakage field is present at and above the surface of magnetized component and its presence can be visibly detected by the cluster of finely divided magnetic particle i.e. when crack is met to magnetic field direction it will form local magnet and will attract fine particles along the crack when sprayed. Magnetization may be induced in the component by using permanent magnet or electromagnet. For simple illustration, consider a bar magnet. It has a magnetic field in and around the magnet. Any place, that a magnetic line of force exits or enters the magnet, it called a pole. A pole where magnetic line of force exits is called a north pole and where a line of force enters the magnet is called a south pole (Fig 5.3.1)


**Fig. 5.3.1 Magnetic Line of Force**

When a bar magnet is broken in the center of its length, two complete bar magnets with magnetic poles at each end of each piece will form. If the magnet is just cracked but not broken completely in two, a north and south pole will form at each edge of the crack. The magnetic field exits the north pole and reenters at the south pole. The magnetic field spreads out when it encounters the small air gap created by the crack, because the air cannot support as much magnetic field per unit volume as a magnet can. When the field spreads out, it appears to leak out of the material and, thus it is called a flux leakage field (Fig 5.3.2.)

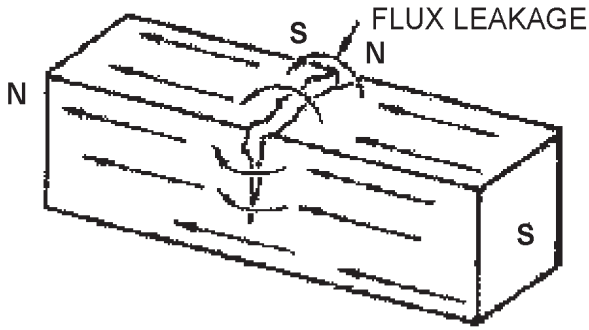



**Fig. 5.3.2 Magnetic flux leakage field**

If iron particles are sprinkled on a cracked magnet, the particles will be attracted to and cluster will be formed not only at the ends of the magnet (poles) but also at poles formed at the edges of the cracks. This formation cluster of particles along the edges of the cracks can be easily detected. (fig. 5.3.3)

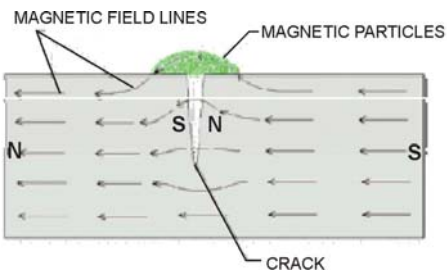



**Fig. 5.3.3 Formation of cluster of particles**

The direction of magnetic field in an electro magnetic circuit is controlled by the direction of current flow. The effectiveness of the defect indication will depend on the orientation of the flaw to the induced magnetic field and greatest when defect is perpendicular. Hence to detect a defect in a member/weld, it has to be tested in both axis i.e. xx-axis as well as yy-axis.

### 5.3.2 Equipments and Methodology

For proper inspection of a component, it is important to establish a magnetic field in at least two directions, so that defects lies in the different directions can be detected. For establishing magnetic field in component various types of equipments are available which are discussed below –

(a) Permanent magnets – The primary types of permanent magnets are bar magnets and horseshoes (yoke) magnets. These are very strong magnets and difficult to handle in the field because large force is required to remove them from the component being inspected. Normally these are used for underwater inspection and for the explosive environment, where electromagnets cannot be used.

(b) Electromagnets – These type of magnets are most widely used in the MPI equipments. In this equipment, electrical current is used to produce the magnetic field. An electromagnetic yoke is a very common piece of equipment which is made by wrapping an electrical coil around a piece of soft ferromagnetic steel. A switch is included in the electrical circuit so that current as well as magnetic field can be turn on and off. This type of magnet generates a very strong magnetic field in a local area where the poles of magnet touch the part to be inspected.

(c) Prods – These are hand held electrodes that are pressed against the surface of the component being inspected to make contact for passing electrical current through steel. The current passing between the prods creates a circular magnetic field around the prods that can be used for magnetic particle inspection. Prods are made from copper and have an insulated handle. One of the prods have a trigger switch so that the current can be quickly and easily turned on and off. Sometimes the two prods are connected by any insulator to facilitate one hand operation. This is known as dual prod also and generally used for weld inspection.

(d) Portable coils and conductive cables – These are used to establish a longitudinal magnetic field within a component, when a preformed coil is used, the component is placed against the inside surface on the coil.

All the above said equipments are portable equipments and can be used in the field without any handling problems.

Following methodology to be followed while performing the test in the field –

(1) The surface should be cleaned before inspection. The surface must be free of grease, oil or other moisture.

(2) Apply the magnetizing force using permanent magnets, a electromagnetic yoke, prods, a coil or other means to establish the necessary magnetic flux.

(3) Dust on the light layer of magnetic particles with the magnetizing force still applied, remove the excess powder from the surface with few gentle puf of dry air.

If wet suspension is used, then the suspension is gently sprayed or flowed over the surface to be tested. Immediately after the application of wet suspension the magnetizing force should be applied.

(4) After this the area should be inspected carefully for finding out the cluster of particles. Surface discontinuities will produce a sharp indication.

### 5.3.3 Magnetic Particles:

(1) As mentioned previously, the particles that are used for magnetic particle inspection are a key ingredient as they form the indications that alert the inspector to defects. Particles start

out as tiny milled (a machining process) pieces of iron or iron oxide. A pigment (somewhat like paint) is bonded to their surfaces to give the particles colour. The metal used for the particles has high magnetic permeability and low retentivity. High magnetic permeability is important because it makes the particles attract easily to small magnetic leakage fields from discontinuities, such as flaws. Low retentivity is important because the particles themselves never become strongly magnetized so they do not stick to each other or the surface of the part. Particles are available in a dry mix or a wet solution.

Dry magnetic particles can typically be purchased in red, black, grey, yellow and several other colours so that a high level of contrast between the particles and the part being inspected can be achieved. The size of the magnetic particles is also very important. Dry magnetic particle products are produced to include a range of particle sizes. The fine particles are around 50 µm (0.002 inch) in size are about three times smaller in diameter and more than 20 times lighter than the coarse particles (150 µm or 0.006 inch), which make them more sensitive to the leakage fields from very small discontinuities. However, dry testing particles cannot be made exclusively of the fine particles. Coarser particles are needed to bridge large discontinuities and to reduce the powder's dusty nature. Additionally, small particles easily adhere to surface contamination, such as remanent dirt or moisture, and get trapped in surface roughness features producing a high level of background. It should also be recognized that finer particles will be more easily blown away by the wind and, therefore, windy conditions can reduce the sensitivity of an inspection. Also, reclaiming the dry particles is not recommended because the small particles are less likely to be recaptured and the "once used" mix will result in less sensitive inspections.

The particle shape is also important. Long, slender particles tend to align themselves along the lines of magnetic force. However, research has shown that if dry powder consists only of long, slender particles, the application process would be less than desirable. Elongated particles come from the dispenser in clumps and lack the ability to flow freely and form the desired "cloud" of particles floating on the component. Therefore, globular particles

are added that are shorter. The mix of globular and elongated particles results in a dry powder that flows well and maintains good sensitivity. Most dry particle mixes have particle with L/D ratios between one and two.

**Wet Magnetic Particles:**

Magnetic particles are also supplied in a wet suspension such as water or oil. The wet magnetic particle testing method is generally more sensitive than the dry because the suspension provides the particles with more mobility and makes it possible for smaller particles to be used since dust and adherence to surface contamination is reduced or eliminated. The wet method also makes it easy to apply the particles uniformly to a relatively large area.

Wet method magnetic particles products differ from dry powder products in a number of ways. One way is that both visible and fluorescent particles are available. Most nonfluorescent particles are ferromagnetic iron oxides, which are either black or brown in colour. Fluorescent particles are coated with pigments that fluoresce when exposed to ultraviolet light. Particles that fluoresce green-yellow are most common to take advantage of the peak colour sensitivity of the eye but other fluorescent colours are also available. (For more information on the colour sensitivity of the eye, see the penetrant inspection material.)

The particles used with the wet method are smaller in size than those used in the dry method for the reasons mentioned above. The particles are typically 10 µm (0.0004 inch) and smaller and the synthetic iron oxides have particle diameters around 0.1 µm (0.000004 inch). This very small size is a result of the process used to form the particles and is not particularly desirable, as the particles are almost too fine to settle out of suspension. However, due to their slight residual magnetism, the oxide particles are present mostly in clusters that settle out of suspension much faster than the individual particles. This makes it possible to see and measure the concentration of the particles for process control purposes. Wet particles are also a mix of long slender and globular particles.

The carrier solutions can be water- or oil-based. Water-based carriers form quicker indications, are generally less expensive, present little or no fire hazard, give off no petrochemical fumes, and are easier to clean from the part. Water-based solutions are usually formulated with a corrosion inhibitor to offer some corrosion protection. However, oil-based carrier solutions offer superior corrosion and hydrogen embrittlement protection to those materials that are prone to attack by these mechanisms.

### 5.3.4 Suspension Liquids:

Suspension liquids used in the wet magnetic particle inspection method can be either a well refined light petroleum distillate or water containing additives. Petroleumbased liquids are the most desirable carriers because they provided good wetting of the surface of metallic parts. However, water-based carriers are

used more because of low cost, low fire hazard, and the ability to form indications quicker than solvent-based carriers. Water-based carriers must contain wetting agents to disrupt surface films of oil that may exist on the part and to aid in the dispersion of magnetic particles in the carrier. The wetting agents create foaming as the solution is moved about, so anti-foaming agents must be added. Also, since water promotes corrosion in ferrous materials, corrosion inhibitors are usually added as well.

Petroleum based carriers are primarily used in systems where maintaining the proper particle concentration is a concern. The petroleum based carriers require less maintenance because they evaporate at a slower rate than the water-based carriers. Therefore, petroleum based carriers might be a better choice for a system that only gets occasional use and adjusting the carrier volume with each use is undesirable. Modern solvent carriers are specifically designed with properties that have flash points above 200 degrees F and keep nocuous vapours low. Petroleum carriers are required to meet certain specifications such as AMS 2641.

### 5.3.5 Testing Practices:

**Dry Particle Inspection:**

In this magnetic particle testing technique, dry particles are dusted onto the surface of the test object as the item is magnetized. Dry particle inspection is well suited for the inspections conducted on rough surfaces. When an electromagnetic yoke is used, the AC or half wave DC current creates a pulsating magnetic field that provides mobility to the powder. The primary applications for dry powders are ungrounded welds and rough as-cast surfaces.

Dry particle inspection is also used to detect shallow subsurface cracks. Dry particles with half wave DC is the best approach when inspecting for lack-of-root penetration in welds of thin materials. Half wave DC with prods and dry particles is commonly used when inspecting large castings for hot tears and cracks.

**Steps in performing an inspection using dry particles:**

Prepare the part surface:- The surface should be relatively clean but this is not as critical as it is with liquid penetrant inspection. The surface must be free of grease, oil or other moisture that could keep particles from moving freely. A thin layer of paint, rust or scale will reduce test sensitivity but can sometimes be left in place with adequate results. Specifications often allow up to 0.003 inch (0.076 mm) of a nonconductive coating (such as paint) and 0.001 inch max (0.025 mm) of a ferromagnetic coating (such as nickel) to be left on the surface. Any loose dirt, paint, rust or scale must be removed.

Apply the magnetizing force: Use permanent magnets, an electromagnetic yoke, prods, a coil or other means to establish the necessary magnetic flux.

Dust on the dry magnetic particles: Dust on a light layer of magnetic particles.

Gently blow off the excess powder: With the magnetizing force still applied, remove the excess powder from the surface

with a few gently puffs of dry air. The force of the air needs to be strong enough to remove the excess particle but not strong enough to dislodge particle held by a magnetic flux leakage field.

Terminate the magnetizing force: If the magnetic flux is being generated with an electromagnet or an electromagnetic field, the magnetizing force should be terminated. If permanent magnets are being used, they can be left in place.

Inspect for indications: Look for areas where the magnetic particles are clustered.

**Wet Suspension Inspection:**

Wet suspension magnetic particle inspection, or more commonly wet magnetic particle inspection, involves applying the particles while they are suspended in a liquid carrier. Wet magnetic particle inspection is most commonly performed using a stationary, wet, horizontal inspection unit but suspensions are also available in spray cans for use with an electromagnetic yoke. A wet inspection has several advantages over a dry inspection. First, all the surfaces of the component can be quickly and easily covered with a relatively uniform layer of particles. Second, the liquid carrier provides mobility to the particles for an extended period of time, which allows enough particles to float to small leakage fields to form a visible indication. Therefore, wet inspection is considered best for detecting very small discontinuities on smooth surfaces. On rough surfaces, however, the particles (which are much smaller in wet suspensions) can settle in the surface valleys and loose mobility rendering them less effective than dry powders under these conditions.

**Steps in performing an inspection using wet suspensions:**

Prepare the part surface: Just as is required with dry particle inspections, the surface should be relatively clean. The surface must be free of grease, oil and other moisture that could prevent the suspension from wetting the surface and preventing the particles from moving freely. A thin layer of paint, rust or scale will reduce test sensitivity, but can sometimes be left in place with adequate results. Specifications often allow up to 0.003 inch (0.076 mm) of a nonconductive coating (such as paint) and 0.001 inch max (0.025 mm) of a ferromagnetic coating (such as nickel) to be left on the surface. Any loose dirt, paint, rust or scale must be removed.

Apply the suspension: The suspension is gently sprayed or flowed over the surface of the part. Usually, the stream of suspension is diverted from the part just before the magnetizing field is applied.

Apply the magnetizing force: The magnetizing force should be applied immediately after applying the suspension of magnetic particles. When using a wet horizontal inspection unit, the current is applied in two or three short busts (1/2 second) which helps to improve particle mobility.

Inspect for indications: Look for areas where the magnetic particles are clustered. Surface discontinuities will produce

a sharp indication. The indications from subsurface flaws will be less defined and loose definition as depth increases.

## 5.4 Eddy Current Testing:

### 5.4.1 Introduction:

The most basic eddy current testing instrument consists of an alternating current source, a coil of wire connected to this source, and a voltmoter to measure the voltage change across the coil. An ammeter could also be used to measure the current change in the circuit instead of using the voltmeter.

Eddy current equipment can be used for a variety of applications such as detection of cracks (discontinuity), measurement of metal thickness, detection of metal thining due to corrosion and erosion, determination of coating thickness and the measurement of electrical conductivity and magnetic permeability.

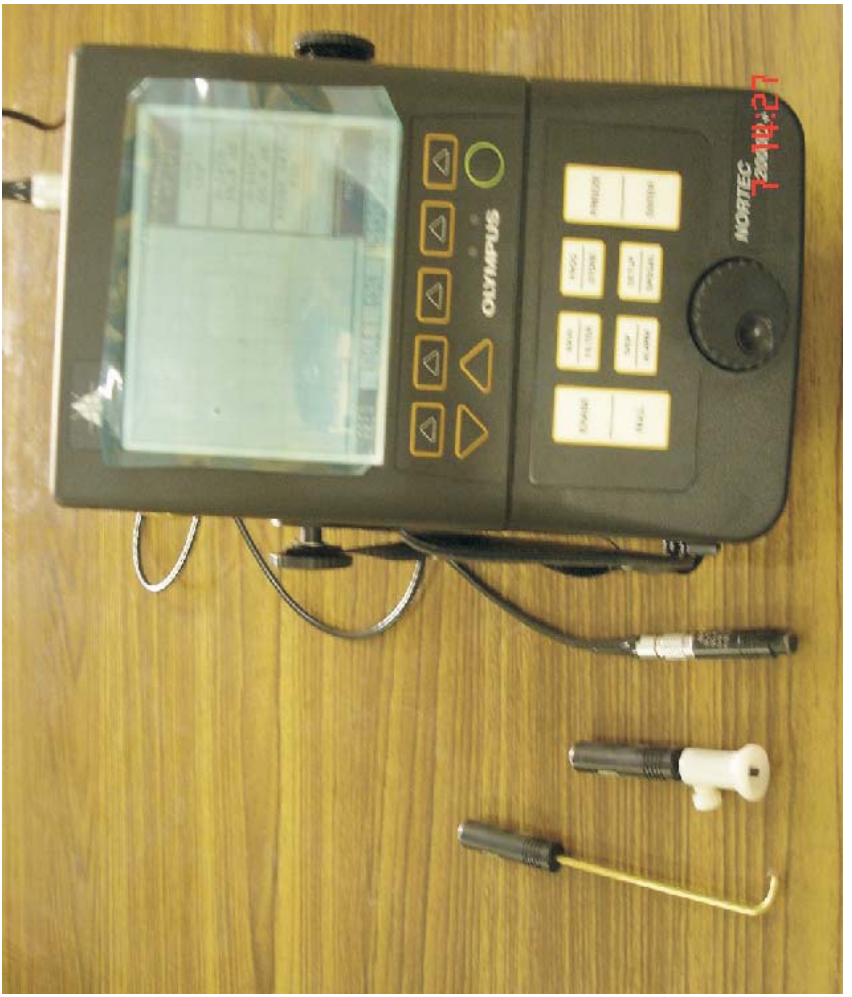
For inspection of bridge girder, this technique can be used for detection of surface breaking cracks. This is an excellent method for detecting surface and near surface defects when the probable defect location and orientation is well known. Defects such as cracks are detected when they disrupt the path of eddy current and weaken their strength.

### 5.4.2 Principal:

This is one of the several NDT methods that use the principle of electromagnetism as the basis for conducting the test. Eddy currents are created through a process called electromagnetic induction. When alternating current is applied to the conductor, such as copper wire, a magnetic field develops in and around the conductor. This magnetic field expands as the alternating current rises to maximum and collapses as the current is reduced to zero. If another electrical conductor is brought into the close proximity to this changing magnetic field, current will be induced in this second conductor. Eddy current are induced electrical currents that flow in a circular path. They get their names

from “eddies” that are formed when a liquid or gas flows in a circular path around obstacles.

For generating eddy current, a “probe” is used which consists of electrical conductor formed into a coil and housed inside the probe. These probes are available in a large variety shapes and sizes. In fact, one of the major advantages of eddy current inspection is that probes can be custom designed for a wide variety of applications. Eddy current probes are classified by the configurations and mode of operation of the test coils. The configuration of probes generally refers to the way the coil or coils are placed with reference to test area. An example of different configuration of probes would be bobbin probes, which are inserted into a piece of pipe to inspect from inside out. While in encircling probes, the coil or coils encircle the pipe to inspect from outside in. The mode of operation refers to the way the coil or coils are wired and interface with the test equipment. The mode of operation of a probe generally falls into one of four categories, absolute, differential, reflection and hybrid. Normally differential probes are used for flaw detection in steel members.


OLYMPUS
MORITEC
FREEZE
CURSOR
MENU
SETUP
SPECIAL
SAVE
FILE
SCAN
ALARM
ERASE
TRIGG

### 5.4.3 Technical Specification:

The technical specification of eddy current instrument, which RDSO has procured, is divided in 4 categories -

(i) Basic Performance.

(ii) Inputs/ Outputs

(iii) Additional Features

(iv) General.

**(i) Basic Performance.**

| | |
|---|---|
| Frequency Range | 100 Hz –12 MHz |
| Gain | 0 – 90.0 dB |
| Sensitivity | Adjustable to 200 Volts per ohm. |
| Flaw Response | 0 – 2000 Hz nominal |
| Digitizing Rate | 6000 samples/ see (max.) |
| Rotation | variable $0^0$ – $360^0$ in $1^0$ steps |
| Sweep | Variable 0.005 – 4 seconds/ division |
| Low Pass Filter | 10 Hz – 500 Hz with 1 Hz increments, 500 Hz – 2000 Hz in 50 Hz increments, WB setting of 4 kHz. Greater than 4 pole response. |
| High Pass | Filter 0 (dc), 2, 4, 6, 8 and 10 –500 Hz in 1 Hz increments. Two pole response available in frequency 1 only. |
| Prove | Drive Adjustable in 3 steps (low, mid, high) corresponding to 2, 6 and 12v peak to peak into 75 ohms, + 0.5v. |
| Null | Three stage digital null. |
| Variable persistence | Screen persistence varies 0.1 – 5 seconds in 0.1 steps. Operator may select |

| | |
|---|---|
| | periodic screen erase at intervals of up to 1 minute. Basic stored screen time is infinite with manual erase. |
| Probe Types | Absolute and differential in either bridge or reflection Configuration, I.D., O.D., bolt hole, sliding and custom special product orders (SPO). |
| Scanner Drive | Drives PS – 5AL, Mini Mite, Spit fire 2000 or RA 2000 scanners at the following speeds.<br>PS - 5AL – 40 to 240 rpm<br>MiniMite – 600 to 3000 rpm<br>Spit fire 2000 – 600 to 3000 rpm<br>RA 2000 – 600 to 2400 rpm |
| Alarms | Can be set to trigger when signal enters the alarm area or is outside alarm area. |
| Alarms Mode | Impedance Plane Display 1 – 3 independent box gates. One polar gate. Seep display – high and low threshold gates. |
| Waterfall Display | 4 – 60 lines with 32 lines max. display simultaneously 105 – 210 points/ sweep recorded. Active with PS - 5AL only. |
| Conductivity Accuracy | ± 0.5% IACS accuracy from 0.9% -2% IACS + 1.0% IACS accuracy over 62% Liftoff accuracy from 0 – 15 mil + 1 mil. Conductivity measurements with greater than 10 mil of liftoff have recorded accuracy. |
| Non-Conductive Coating | Measures non conductive coating. |
| Thickness | thickness from 0.00" – 0.025". Accuracy + 0.001" over 0.00 – 0.015". |

| | |
|---|---|
| Trace Storage | Stores upto 20 traces for recall. Locations may store any combination of frozen screens or movement of the impedance spot for upto 60 seconds. |
| Program Storage | 120 instrument setup may be stored and recalled. The date and time of storage are recorded and available to the operator. Each location may be labeled with alphanumeric names upto 29 characters. The instrument can maintain storage upto one year with batteries removed (with a functional internal memory backup battery). |
| Serial Interface | RS232 compatible serial interface, adjustable from 2400 – 57600 baud, with 9600 baud default.<br><br>Communicates with external computer or serial printer. Instrument parameters except power ON/OFF can be controlled through the serial interface. All programs and stored screen locations one accessible through this interface. |
| Printout | Printout available with optional serial printer. Additional printers may be compatible with optional serial to parallel converter. Printout text may be customized from either the font panel or an external computer. |
| Supported Printers | Cannon Inkjet, Epson compatible, Pentax Pocket Jet, Pentax II, Pocket Jet and other HP-PCL Compatibles. |

**Inputs/ Outputs**

| | |
|---|---|
| Power | 7 pin connector (2000D), 8-pin connector (2000D+) to operate from AC power and charge internal batteries. |
| RS232C | DB9P connector for bidinectional serial data by way of RS232C links to external computers, terminals or printer. |
| Outputs | DB9F connector provides analog out for vertical and horizontal signals ( + 5V, 10mA max. 1V per div) on both frequency 1 and frequency 2 and 5TTL Compatible (3V logic) active high alarm output. |
| Probe Connectors | 16 pin LEMO connector with flush mount adapters. |

**Additional Features**

| | |
|---|---|
| Report Fields | User defined 40-character report header, upto 7 user defined 40-character report fields and 26 character labels, upto 3 user defined 26 character report entries may be down load to the RS232 port or entered through the report edit function. Clock, calendar time, date stored and printed with each wave form instrument ID, manufacturer'. name and model printed with each waveform. International menu selectable languages including English, Spanish, French and German. |
| Conductivity | Frequency: 60 kHz or 480 kHz<br><br>Probe Type: Conductivity Probe.<br><br>Digital conductivity: Digital conductivity display from 0.9% - 110% IACS. |

Accuracy within + 0.5% IACS from 0.9% - 62% IACS and within + 1.0% of **values** over 62%. Meets or exceeds BAC5651. Conductivity measurements with greater than 10 mil of lift off have reduced accuracy.

Alarms: Independent high and low limit alarms can be set for conductivity lift off. Alarms can be set to trigger when the signals one inside the limits or outside the limits.

Dual Frequency

Second frequency: 100Hz – 3MHz, 2nd frequency is an exact division of the first frequency in rations of 1/2, 1/4 and even divisors to 1/32

Display:

Frequency 1 only, Fequency 2 only, sum of frequencies 1 and 2, difference between frequencies 1 and 2, split screen with selected combinations of frequencies 1 and 2 and mixed frequencies.

**General**

Dimensions

9.5" L x 5.5" W x 3.6" D (241mm x 140 mm x 92mm)

Weight

2000D : 6lbs (2.7kg) with batteries

2000D+ : 4.6 lbs (2.09 kg) with batteries. (3.6 lbs or 1.6 kg without batteries)

Display (Color LCD)

For 2000D +, color LCD, 3.0" x 3.8" (5" diagonal) (76mm x 96mm, 122mm diagonal), 320H x 240V pixel. 72 Hz refresh rate. Adjustable backlight.

Display (LCD)

Monochrome LCD, 3.0" x 3.8" (5" diago-

nal) (76mm x 96mm, 122mm diagonal), 320H x 240V pixel. 72 Hz refresh rate. Adjustable backlight.

Operating Temperatures - $4^0$F to $140^0$F ($-20^0$C to $60^0$C).

Storage temperature - $40^0$F to $176^0$F ($-40^0$C to $80^0$C).

| | |
|---|---|
| Power | DC: Two 10.8V DR35 Ni MH batteries (2000D), one 10.8V Li-Ion battery (2000D+), optional D cell pack or external DR 35 pack.<br><br>AC: External charger/ adapter. Fully charges batteries in appox 6 hours which operating instrument. Line voltage 100 – 240V AC, 47-63 Hz. |
| Operating Time | Eight hours (typical) @ $75^0$F ($24^0$C) with EL display. Seven hours (typical) with LCD display. Estimated operating time remaining indicated by icon on status display. With optional belt mounted battery pack, eight hours (typical). |

### 5.4.4 Methodology:

While performing the inspection with surface probe following methodology should be adopted:

(i) Connect the probe with the system. Power Link screen will appear. Rotate the smart knobs to confirm and press enter. The screen will display the information about the probe. There are three different types of probes. These are denoted as LOW, MID, HI. The approximate peak to peak voltages for each are 2, 6 and 12 volts respectively. Mid probe drive is normally sufficient for most eddy current testing. Now press the MAIN key to proceed with test setup.

(ii) Adjust the frequency as required. Press the FREQ menu soft key. Rotate the smart knobs for the required frequency until it appears in the frequency box.

(iii) Adjust the phase angle as required. Press the ANGLE menu soft key on the instrument. Rotate the smart knobs until the same appears in the angle box.

(iv) Adjust the horizontal gain and vertical gain as required. If horizontal gain and vertical gain to be kept same than press GAIN from the menu. Rotate the smart knobs key until both reaches to the desired value. If horizontal gain and vertical gain to be kept different than press HGAIN menu soft key and rotate the smart knobs to the required value. Similarly select the VGAIN and adjust the required value.

(v) Place the probe on the specimen to be tested away from cracks. Press NULL and ERASE keys for positioning of the probe and to clean the screen.

(vi) Slide the probe on the specimen smoothly.

It has an extensive selection of data storage capabilities. Data may be stored using one of three different methods and once stored, the information and instrument setups may be accessed.

Data may be stored as either frozen data, captured data and waterfall data.

Frozen data captured an image as if is displayed on the screen, frozen at that instant of time.

Captured data saves data for a period of time 2.5 to 60 seconds and can be replay this data back to screen, complete with alarms.

The captured data may be recorded using one of two modes.

(a) Capture once starts recording the capture as soon as operation is selected and will automatically end when the selected time has been reached.

(b) Capture continuous stops recording data when the operation is ended and automatically stores the preceding data for the allotted time period. This capture is ended manually by pressing the ENTER key. When the continuous capture is ended the information stored is the information that was being recorded at the end of the capture .

Waterfall data is data captured from a waterfall (multiple sweep) display using a PS-5AL scanner. It is the only data that can be modified after it is captured. All methods of saving will record the current data and time of storage.

Scan the probe over part of the surface in a pattern that will provide complete coverage of the area being inspected. Care must be taken to maintain the same probe to surface orientation as probe wobble can affect interpretation of the signal.

### 5.4.5 Advantages and Limitations:

This method is very sensitive to small cracks and other defects. This method detects surface and near surface defects very efficiently. The results can be obtained immediately after inspection. The equipment required for test is very small and portable and minimum part preparation is required for carrying out this test. During the test, the test probe does not need to contact the part.

One of the major limitation is that the depth for penetration is limited so it cannot detect the internal defects which are located away from the surface. The surface to be tested must be accessible to the probe. Flaws such as delamination that lie parallel to the probe coil winding and probe scan direction cannot be detected easily.

## 5.5 Radiographic Testing

This is the technique of obtaining a shadow image of a solid using penetrating radiation such as X-rays or gamma rays. These rays are used to produce a shadow image of an object on film. Thus if X-ray or gamma ray source is placed on one side of a specimen and a photographic film on the other side, an image is obtained on the film which is in projection, with no details of depth within the solid. Images recorded on the films are also known as radiographs.

The contrast in a radiograph is due to different degrees of absorption of X-rays in the specimen and depends on variations in specimen thickness, different chemical constituents, non-uniform densities, flaws, discontinuities, or to scattering processes within the specimen.

Some of the other closely related methods are Tomography, Radioscopy, Xerography etc.

### 5.5.1 Methodology

First step of the method is to examine carefully the specimen and to decide on the direction to examine the object considering the probable orientation of defects and the thickness of the specimen in relation to the diverging beam of X-rays.

Considering the thickness of object, density of the material etc., the wavelength of X-ray to be used should be decided.

The images can be observed on an image intensifying tube with remote viewing or recorded on film with or without intensifying screens. Grid or blocking materials should be used to reduce scattering effects. The optimum time of exposure need to be determined by experimental trials.

The last but the most important step is the interpretation of radiograph. Radiographs are projections, providing no information about depth within the specimen. While interpreting following factors normally should be considered –

(a) Orientation of the object to permit any discontinuity or defect to show maximum contrast.
(b) Use of radiation so that to have largest possible differences in the relative absorption coefficients of the different compositions present.
(c) The selection of wavelength of X-ray to control sharpness and contrast of the image.
(d) Use of well illuminated viewing screen under optimum lighting conditions.
(e) Probable defects in the specimen.
(f) The X-ray target to film distance should not be less than 10 x the thickness of specimen.
(g) The greatest dimension of the suspected flaw should be parallel to X-ray beam.

### 5.5.2 Advantages & Limitations

For conducting this test access to opposite side of the object is required. X-rays and gamma-rays are dangerous and lot of safety precautions to be taken while conducting the test. A discontinuity of thickness less than 2% of the overall thickness of the specimen is difficult to observe. This method is relatively expansive method. Normally this method is used for detecting internal flaws in the welds especially butt welds.

## 5.6 Ultrasonic Inspection:

This method can be used for steel structures.

### 5.6.1 Definition of Ultrasound:

Mechanical vibrations of different kinds can travel through solids due to their elastic properties. A good example is a spring, which is tightened at one end. The other end is able to expand up and down. If it makes enough oscillations per second, you will be able to hear a sound. This is due to the fact that the air also starts vibrating as compression waves. The human ear can hear these compression waves, if the frequency is higher than the lowest audible range, which is about 20 oscillations/sec (Hz).

The faster the spring oscillates the higher the sound. Over a certain number of oscillations, we are not able to hear anything. We have then reached the upper audible level, which is about 20000 oscillations per second (Hz).

Sound waves with a frequency higher than 20kHz are called ultrasound waves.

After changing to the use of ultrasound the method became useful in a greater scale. Ultrasonic waves gives due to their higher frequency and smaller wave length a much better possibility of finding defects and determine their size and their position. The vibrations are normally generated by the use of a piezoelectric crystal, which can be excited by an electrical pulse.

We are going through the two most common test methods, the through transmission technique and the pulse echo technique.

### 5.6.2 Through Transmission Technique:

When using this technique you have a transmitter on one side and a receiver on the other side of the object to be tested. The transmitter sends out ultrasonic waves either as continuous oscillations or as short pulses, each consisting of a few oscillations. In the last case the pulses are send out with an interval, which is long compared to the duration of the pulse itself. The wave travels through the object and is then received by the receiver. The signal from the receiver shows the sound energy, which has travelled from the transmitter to the receiver. If the sound beam hits a discontinuity in the object, the received sound energy will be less. The signal from the receiver will then be smaller. This signal can be registered and used in different ways. For example the signal can automatically activate an alarm, if the sound beam hits a defect over a certain size, by which the received sound energy goes down under an equivalent fixed level.

The signal can also be registered with ordinary ultrasonic equipment, which contains an oscilloscope. It is seen as vertical reflections of the signal on the screen at a distance to the right of

the deflection on the left side of the screen. This reflection to the left is called the initial pulse.

The ultrasonic equipment's way of working will be discussed later. If there is a defect between the transmitter and the receiver, it will prevent a larger or smaller part of the sound beam from reaching the receiver, which will weaken the signal. This is seen on the screen as a smaller deflection as seen on **Figure 5.6.2.**

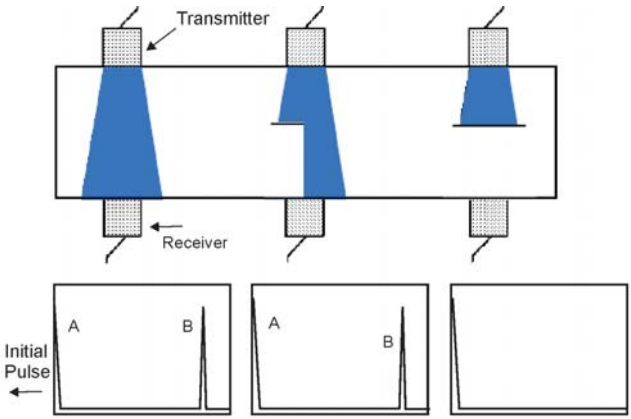



**Fig. 5.6.2: Principles of through transmission technique.**

### 5.6.3 The Pulse Echo Technique:

This is the most common used technique.

The principle in this technique is almost the same as used in an echo sounder. A transmitter sends out a short pulse consisting of a few oscillations into the object to be tested.

The sound wave travels through the object with a constant speed, the sound velocity, which is always the same in the same material, regardless of the frequency. If the object is without defects, the pulse continues until it hits the back wall of the object, from where it is reflected like light beam from a mirror.

The pulse then travels back through the object - still with the same velocity - and is received by a receiver. As the pulse travels with a constant speed, the time, the sound pulse has travelled from the transmitter till it returns is equivalent to twice the thickness of the object. After a while a new pulse is send out, which travels exactly like the first one.

In order to measure the very short time from sending out one pulse till it is received again, the ultrasonic equipment is provided with an oscilloscope or a digital display. An electron beam makes a bright spot to travel horizontally across the screen with a constant velocity from left to right. The movement begins at the same time as the pulse is send out from the crystal. The initial pulse gives a vertical deflection on the left side of the screen.

After that the bright spot continues to the right with a speed that can vary from about 1/200 to 5 times the velocity of sound in steel. See Figure 5.6.3.1.

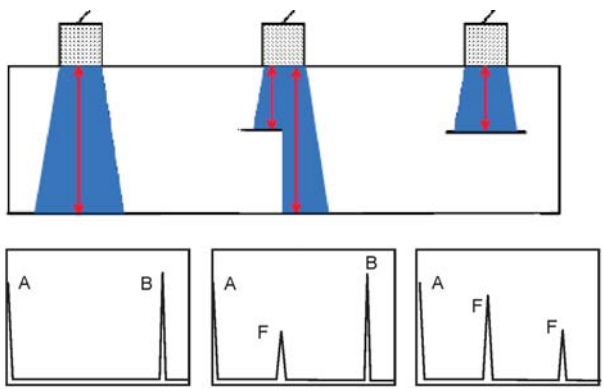



**Fig. 5.6.3.1: Principles of pulse echo technique.**

If you adjust the velocity of the bright spot on the screen, to be the same as the sound velocity in the test piece, it will travel to the right on the screen with the same speed as the pulse travels inside the object. When it returns to the receiver, the bright spot has travelled a distance, which is twice the thickness of the test piece.

The moment the pulse hits the receiver, it sends an electrical signal to the ultrasonic equipment. On the screen it is seen as a brief, vertical reflection of the bright spot. This is called a bottom echo. The distance on the screen between the initial pulse and the bottom echo is in this case twice the thickness of object.

Changing the speed of the bright spot on the screen, the distance between the initial pulse and the bottom echo can be adjusted. You can change it in such a way that the thickness of the steel object between approximately 2 mm and 10 m can be read off on the screen.

If the sound wave hits a reflecting surface during its way through the object for example a crack, a part of the sound will reflect back and will be seen as a vertical reflection before the bottom echo. This deflection is called the defect echo. By its position on the screen the distance from the surface of object can be determined quite accurately. The height and shape of the flaw echoes might give some information about the size and type of the defect.

The sending out of a pulse and the movement on the screen is repeated many times a second. The single instant pictures appear on the screen as constantly shining lines, which only moves when the probe is moved across the surface of the object. However, there is a distance between the pulses, which allows the first to die out before a new pulse is send out. In most ultrasonic equipment it means that a pulse can move backwards and forwards in a 10 m long steel bar, before a new pulse is send out.

Figure 5.6.3.2 and Figure 5.6.3.3 show some common ultrasonic equipment.


**Fig. 5.6.3.2: USM 35 and EPOCH IV.**

**Fig. 5.6.3.3:**

## 5.6.4 Probes:

### Normal probes:

A normal probe generates longitudinal waves, which leaves the probe at a right angle to its contact surface. If the probe is in contact with a specimen, the sound wave penetrate into it. It travels in straight lines, with a certain beam spread. See Figure 5.6.4.1.


**Fig. 5.6.4.1: Normal probes.**

**Construction:**

A normal probe is constructed as shown in Figure 5.6.4.2 below:

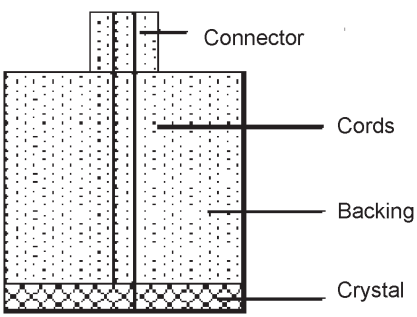



**Fig. 5.6.4.2: Inside of a Normal probe.**

The crystal must be damped in order to quickly stop the oscillations after it has been excited, either by an electrical pulse or by a reflected sound wave. In this way the initial pulse and the echoes on the screen of the equipment are prevented from being too wide

**Dual probe (TR-probes):**

The near resolution can be increased considerably by using a probe with two separate crystals one for transmission and one for receiving. Figure 5.6.4.3 shows the inside of a TR- probe.

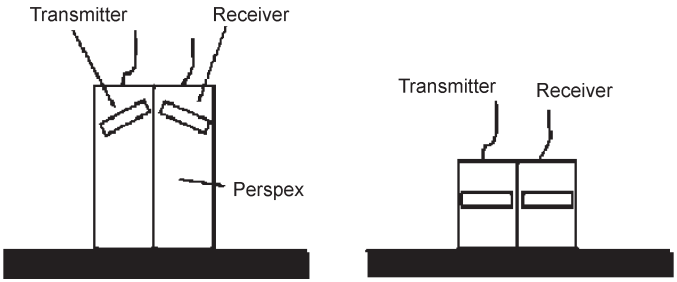



**Fig. 5.6.4.3:** ***TR-probes.***

The piezoelectric crystal is glued to perspex blocks, which works as a delay line for the sound. The crystals are placed in a slight angle to the surface of the object, and turned against one another. Due to that you can detect defects right under the surface. Unfortunately this construction may give spurious echoes from surface waves.

This can be avoided using a probe where the crystals are parallel to the surface of the object. In return you have a minor sensitivity for defects right under the surface. Beyond these common types you have special probes developed for specific tasks e.g. waterproof types and heat resistant types.

**Angle probes:**

Angle probes are normally manufactured with frequencies

between 2 and 5 MHz and the angles 35°, 45°, 60°, 70° and 80° for testing in steel. Other frequencies and angles are available. The angles are always stated in proportion to the normal. See **Figure 5.6.4.4**.

The probe index is marked on the side of the probe with a line or a dot mark.


**Fig. 5.6.4.4: Angle probes.**

Testing a plate can be done manually with a plate tester as shown on Figure 5.6.4.5 or in an automatic ultrasonic testing installation, where the plate is moved past a row of probes for example 40-80 probes according to the width of the plate. Each probe scans the plate along a line and the results are registered on a paper slip.

**Fig. 5.6.4.5: Plate tester with mounted ultrasonic equipment.**

### 5.6.5 Application of Ultrasonic Testing Method:

This method is not suitable for testing of coarse grained structures. Most of the castings cannot be scanned using this method because of the coarse grained structure which leads to scattering and attenuation of waves. This method is suitable for scanning of fine grained structure. Rolled sheets and plates are suitable for ultrasonic testing. Normal probes are used for detecting laminations and angle probes for fast inspection of plates for most of the discontinuities except smooth laminations parallel to the surface.

Before starting the test, the equipment should be tested with a calibration block. The surface to be tested should be smooth and plain. The couplant must be used for testing purpose. The thickness of couplant layer must be as less as possible. A thick layer will affect the direction of the ultrasonic beam in the test piece. For detection of smaller flaws, small diameter high frequency transducers are used.

### 5.7 Complete Structreal Testing :

Details are given at S. No. 2.12

### 5.8 Accoustic Emission Techniques :

Details are given at S. No. 4.3

■ ■ ■

# CHAPTER-6

## NON-DESTRUCTIVE TESTING OF MASONRY BRIDGES

## 6.1 Introduction

As already explained in chapter 1, the use of NDT methods for testing of masonry structures is not very common in India. However there is a necessity of adopting suitable NDT methods for evaluation of masonry structures as existing system of inspection is not sufficient to cover all the aspects of inspection. Sometimes very critical defects like, deterioration of masonry materials, internal cavities formed due to rat holes etc. and cracking of structures due to overstressing go unnoticed which may proved to be very fatal for the safety of structure. The NDT methods have a large potential to be part of system for inspection and monitoring of structures. This includes quality assurance during and after construction, identification of damages in an early stage and to decide the repair strategy for rehabilitation of the structures. Some of the NDT methods which are used for evaluation and inspection of masonry structures are listed below –

(a) Flat Jack Testing

(b) Impact Echo Testing

(c) Impulse radar testing

(d) Infrared thermography

(e) Boroscope

## 6.2 Flat Jack Testing

This test can determine the engineering properties of older structures for structural evaluation. This method is used to determine the in situ stress and compressive strength of masonry structures.

A flat jack is a flexible steel envelope, thin enough to fit within a masonry mortar joint. During testing, the flat jack is hydraulically pressurized and applies stress to the surrounding masonry.

This method directly measures the actual state of compressive stress present within the masonry and is useful for determining stress gradients present within a masonry wall or column. The in situ stress test works on basic principle of stress relief, when mortar is removed from a joint, compressive stress within the masonry forces the slot thus formed to close by a small amount. A flat jack is inserted into the slot and pressurized to restore the slot to its original opening dimension. The pressure at which the original opening is restored is adjusted by the flat jack calibration constant, providing a measure of the in situ masonry compressive stress.

The other test is, in situ deformability test which is used for direct measurement of masonry deformability properties and to estimate the masonry compressive strength. For conducting this test, two parallel flat jacks are used which subject the masonry between them to compressive stress. The stress strain curve which is obtained during the test, is used for obtaining both compressive modulus and an estimate of compressive strength.

**6.3 Impact Echo Testing** :
Details are given at S. No. 4.5.2.

**6.4 Impulse Radar Testing** :
Details are given at S. No. 4.4.

**6.5 Infrared Thermography** :
Details are given at S. No. 4.2.

**6.6 Boroscope** :
Details are given at S. No. 4.7

# CHAPTER No 7

## Summary of NDT equipments available and their use

| S.N. | Measurement | Application | Equipment |
|---|---|---|---|
| **2.0** | **Strength evaluation of Concrete** | | |
| 2.1 | Surface strength (rebound number) | Surface Zone Strength Assessment | Rebound Hammer |
| 2.2 | Homogeneity of Concrete | Quality of Concrete | Ultrasonic pulse velocity meter |
| 2.3 | Combined ultrasonic and rebound number determination | Uniformity/ homogeneity, Location of | Ultrasonic Pulse velocity tester internal defects |
| 2.4 | Pull-off strength (bond strength) | Surface Zone Strength Assessment | Pull off Tester |
| 2.5 | Pull out force | Surface zone strength assessment | Pull out “Lok” Test (Construction Stage) Pull Out “Capo” Test (after construction) |
| 2.6 | Break off test | The break off test at failure can be related to compressive or flexural strength | Break off tester |
| 2.7 | Penetration resistance | Surface Zone Strength Assessment | Windsor Probe |
| 2.8 | Core strength (Micro core) | Localized in-situ strength assessment | Micro Core Test Apparatus |
| 2.9 | Permeability test | To determine air permeability of cover concrete | Permeability tester |
| 2.10 | Bond test | It measures bonding or direct tensile strength between two layers. | Bond tester |

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|---|---|---|---|
| 2.11 | Maturity method | It estimate concrete strength | Maturity meters. |
| 2.12 | Complete Structural | To detect damage and general evaluation of structures | Complete structural testing equipment technique |
| **3.0** | **Corrosion assessment, location and dia of rebar and cover** | | |
| 3.2 | Corrosion potential (half-cell) | Status of Corrosion activity | Half Cell Potential Meter |
| 3.3 | Resistivity | Rate of probable corrosion | Resistivity Meter |
| 3.4 | Carbonation depth | Corrosion risk and cause | Carbonation Test Kit |
| 3.5 | Chloride content | Corrosion risk and cause | Chloride Field Test System |
| 3.6 | Voids and Corrosion | Viewing interior of concrete | Endoscopy |
| 3.7 | Scanning and dia of rebar and cover | It is used for locating rebars, diameter of rebars and concrete cover | Profometer |
| 3.8 | Cover and re-bar measurement | Corrosion risk and cause | Micro Cover Meter |
| **4.0** | **Crack measurement, length changes, deflection in, buildings and structures** | | |
| 4.1 | Length changes | Strain measurement | Measurement and digital strain gauges |
| 4.2 | Infra Red Images | Cracks, delamination | Infra Red Thermal Imaging Systems |
| 4.3 | Acoustic Emission technique | To measure the location and activity of cracks | Acoustic Emission Detection System |

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|---|---|---|---|
| 4.4 | Short pulse radar | Detection of delamination in conc., Degree of hydration of conc., water content in fresh conc. and measurement of conc. layer thickness. | Short pulse radar system. |
| 4.5 | Stress wave propagation method | Based on stress wave propagation, used for non destructive testing of conc. | Stress wave propagation equipment. |
| (a) | Pulse Echo method | Thickness measurements, flow detection and integrity testing of piles | |
| (b) | Impact Echo method | -do- | |
| (c) | Impact response method | To test piles and slab like structures | |
| 4.6 | Crack width measurement | Measurement of crack width and angles | Microscopes, crack width gauges for walls |
| 4.7 | Endoscope Examination | Internal condition of concrete, condition of pre-stressing Tendons | Bore Scope, Core Cutter |
| 4.8 | Nuclear method | To measure moisture content in concrete | Neutron moisture gauge |
| 4.9 | Structural Scanning equipment | It is complete concrete inspection system. It is also used for inspection of foundation | Ground penetrating radar |
| 4.10 | Spectral Analysis of surface wave of unknown foundation | Measurement of depth of unknown foundation | SASW equipment |

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|---|---|---|---|
| **5.0** | **Non Destructive Testing of Steel Bridges** | | |
| 5.2 | Dye-penetrating method | It is used to reveal surface breaking flaws by bleed out of a colored or fluorescent dye from the flaw | Dye – penetrating equipment |
| 5.3 | Magnetic particle inspection (MPI) | To detect defects in the structures | Magnetic particle inspection equipments |
| 5.4 | Eddy Current | Cracks, Voids, Honey combing | Eddy Current Meter |
| 5.5 | Radiographic testing | It gives shadow image of an object or film by using penetrating radiation such as X-rays or Gamma rays | Radiographic tester |
| 5.6 | Cross hole pile integrity testing | Integrity Testing of pile | Ultrasonic Cross pile Integrity Testing Equipment. |
| 5.7 | Complete Structural technique | To detect damage and general evaluation of structures | Complete structural testing equipments. |
| 5.8 | Acoustic Emission technique | To measure the location and activity of cracks | Acoustic Emission Detection System |
| **6.0** | **Non Destructive Testing of Masonry Bridges** | | |
| 6.2 | Stress modulus of deformation | In-situ stress deformability and resistance of brickwork or stone masonry | Flat Jacks and accessories |
| 6.3 | Impact Echo method | Thickness measurements, flow detection and integrity testing of piles | Stress wave propagation equipment. |

| | | | |
|---|---|---|---|
| 6.4 | Short pulse radar | Detection of delamination, Degree of hydration, water content and measurement of layer thickness. | Short pulse radar system. |
| 6.5 | Infra Red Images | Cracks, delamination | Infra Red Thermal Imaging Systems |
| 6.6 | Endoscope Examination | Internal condition of Material, condition of pre-stressing Tendons | Borescope |

**LIST OF FIRMS DEALING WITH NDT EQUIPMENTS :**
**(NOTE : The list is not exhaustive, there are many more manufacturer and distributes available all over India)**

1. M/s AIMIL LTD., A-8, MOHAN COOPERATIVIE INDUSTRIAL ESTATE, MATHURA ROAD, NEW DELHI.

2. M/s HILTI INDIA PVT. LTD.,8 LSC PUSHPA VIHAR COMMUNITY CENTRE, NEW DELHI

3. M/s ULTRA TECHNOLOGIES PVT. LTD, B-85, KALKAJI, NEW DELHI.

4. M/s. ENCARDIO RITES, LUCKNOW

5. M/s. JAMES INSTRUMENTS LUC. 3727, NORTH AEDZIE AVENUE, CHICAGO ILLINOIS 60618, U.S.A.

6. PROSEQ U.S.A., RIESHASH STRASSE 57 CH - 8034 ZURICH, SWITZERLAND.

7. ELE INTERNATIONAL LTD., EAST MAN WAY, HEMEL HAMPSTEAD HERTFORDSHIRE, HP2 7HB, ENGLAND.

8. M/S S.G. MARKETING PVT. LTD., 15, BIRBAL ROAD, JANGPURA XTN., NEW DELHI 110014

9. M/S NDT TECHNOLOGIES PVT. LTD., PLOT NO. 11, SECTOR 23, TURBHE, NEW BOMBAY 400705

10. M/S J.MITRA & CO. PVT. LTD., A 180-181, OKHLA INDUSTRIAL AREA – I, NEW DELHI 110020

11. M/S KARL STORZ ENDOSCOPY INDIA PVT. LTD., C-126, OKHLA INDUSTRIAL AREA PHASE – I, NEW DELHI 110020.

**REFERENCES:**


1. RDSO Guidelines on non-destructive testing of bridges BS - 103

2. IS-1311 (Part-1) : 1992 Non-Destructive Testing of Concrete methods of test, Part-I, Ultrasonic Pulse Velocity.

3. IS 13322 (Part-2) : 1992, Non-Destructive Testing of Concrete – methods of test, Part 2, Rebound hammer.

4. RDSO Report No.BS-51 : Guidelines on use of Acoustic emission technique

5. RDSO Report No.BS-52: Guidelines on use of corrosion monitoring equipments

6. RDSO Report No.BS-53: Guidelines on use of ultrasonic instruments for monitoring of concrete structures.

7. RDSO Report No.BS-50: Guidelines on use of Profometer.

8. RDSO Report No.BS-42: Guidelines on use of Microcovermeter

9. "Handbook on Non Destructive Testing of Concrete" (second edition) by V.M. Malhotra and N.J. Carino

10. "Non destructive testing" by Louis Cartz.


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