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Guidebook on non-destructive testing of concrete structures

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FOREWORD

The International Atomic Energy Agency (IAEA) has been active in the promotion of non-destructive testing (NDT) technology for many years. NDT is an important component of a number of IAEA regional projects successfully executed or currently being executed. These are the Regional Co-operative Arrangements for the Promotion of Nuclear Science and Technology in Latin America (ARCAL), the Regional Co-operative Agreement for Research, Development and Training Related to Nuclear Science and Technology for Asia and the Pacific (RCA), the African Regional Co-operative Agreement for Research, Development and Training Related to Nuclear Science and Technology (AFRA) and, more recently, the NDT Regional Project in West Asia. Under these regional projects many regional and national training courses are conducted. Since the IAEA relies heavily on experts from Member States to conduct training courses, it is necessary to have agreed syllabi, training guidelines and training material not only to guide the experts but also to provide some consistency between courses and resultant uniformity in the training provided.

The syllabi for training courses that cover the conventional NDT methods are available in IAEA-TECDOC-628. This TECDOC covers the conventional methods of liquid penetrant testing, magnetic particle testing, eddy current testing, radiographic testing, ultrasonic testing, visual inspection and leak testing.

Based on these syllabi, training course notes have been produced to cover Industrial Radiography (IAEA Training Course Series No. 3) and Ultrasonic Testing of Materials at Level 2 (IAEA Training Course Series No. 10).

These training course notes deal predominantly with the NDT of metallic materials. While NDT of metallic materials is a very important application, NDT is being used increasingly for the inspection of concrete structures. Training Course Series Nos. 3 and 10 cover the inspection of concrete using the relevant NDT method; however, coverage is brief and does not present the whole range of NDT methods used for the NDT of concrete. Concrete has become a very common construction material in most IAEA Member States and problems have occurred because of faulty construction practice. A need was therefore identified for a guidebook on the NDT of concrete. The first IAEA Training Course on the NDT of Concrete and other Non-Metallic Materials was held in 1987 in Japan, at the Japanese Society for Non-Destructive Inspection. Subsequent courses/workshops were held in Thailand and Singapore. In 1998, AFRA national co-ordinators prepared a draft syllabus on the NDT of Concrete. This syllabus was circulated for comment to national co-ordinators in other IAEA projects. R.S. Gilmour (Australia) compiled the first draft of the training material, which was circulated to the national NDT co-ordinators for the NDT subproject in different RCA countries. IAEA experts discussed the amendments made to this draft at a Meeting on the NDT of concrete in the Malaysian Institute for Nuclear Technology (MINT), Malaysia in September 1999.

During the compilation of this manuscript, guidance and support were provided by Abd Nassir Ibrahim from Malaysia and G. Singh from India. The IAEA officer responsible for this publication was A.A. Khan of the Division of Physical and Chemical Sciences.

EDITORIAL NOTE

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1. GENERAL KNOWLEDGE

1.1. INTRODUCTION

1.1.1. Importance and need of non-destructive testing

It is often necessary to test concrete structures after the concrete has hardened to determine whether the structure is suitable for its designed use. Ideally such testing should be done without damaging the concrete. The tests available for testing concrete range from the completely non-destructive, where there is no damage to the concrete, through those where the concrete surface is slightly damaged, to partially destructive tests, such as core tests and pullout and pull off tests, where the surface has to be repaired after the test. The range of properties that can be assessed using non-destructive tests and partially destructive tests is quite large and includes such fundamental parameters as density, elastic modulus and strength as well as surface hardness and surface absorption, and reinforcement location, size and distance from the surface. In some cases it is also possible to check the quality of workmanship and structural integrity by the ability to detect voids, cracking and delamination.

Non-destructive testing can be applied to both old and new structures. For new structures, the principal applications are likely to be for quality control or the resolution of doubts about the quality of materials or construction. The testing of existing structures is usually related to an assessment of structural integrity or adequacy. In either case, if destructive testing alone is used, for instance, by removing cores for compression testing, the cost of coring and testing may only allow a relatively small number of tests to be carried out on a large structure which may be misleading. Non-destructive testing can be used in those situations as a preliminary to subsequent coring.

Typical situations where non-destructive testing may be useful are, as follows:

— quality control of pre-cast units or construction *in situ*

— removing uncertainties about the acceptability of the material supplied owing to apparent non-compliance with specification

— confirming or negating doubt concerning the workmanship involved in batching, mixing, placing, compacting or curing of concrete

— monitoring of strength development in relation to formwork removal, cessation of curing, prestressing, load application or similar purpose

— location and determination of the extent of cracks, voids, honeycombing and similar defects within a concrete structure

— determining the concrete uniformity, possibly preliminary to core cutting, load testing or other more expensive or disruptive tests

— determining the position, quantity or condition of reinforcement

— increasing the confidence level of a smaller number of destructive tests

— determining the extent of concrete variability in order to help in the selection of sample locations representative of the quality to be assessed

— confirming or locating suspected deterioration of concrete resulting from such factors as overloading, fatigue, external or internal chemical attack or change, fire, explosion, environmental effects

— assessing the potential durability of the concrete

— monitoring long term changes in concrete properties

— providing information for any proposed change of use of a structure for insurance or for change of ownership.

1.1.2. Basic methods for NDT of concrete structures

The following methods, with some typical applications, have been used for the NDT of concrete:

— Visual inspection, which is an essential precursor to any intended non-destructive test. An experienced civil or structural engineer may be able to establish the possible cause(s) of damage to a concrete structure and hence identify which of the various NDT methods available could be most useful for any further investigation of the problem.

— Half-cell electrical potential method, used to detect the corrosion potential of reinforcing bars in concrete.

— Schmidt/rebound hammer test, used to evaluate the surface hardness of concrete.

— Carbonation depth measurement test, used to determine whether moisture has reached the depth of the reinforcing bars and hence corrosion may be occurring.

— Permeability test, used to measure the flow of water through the concrete.

— Penetration resistance or Windsor probe test, used to measure the surface hardness and hence the strength of the surface and near surface layers of the concrete.

— Covermeter testing, used to measure the distance of steel reinforcing bars beneath the surface of the concrete and also possibly to measure the diameter of the reinforcing bars.

— Radiographic testing, used to detect voids in the concrete and the position of stressing ducts.

— Ultrasonic pulse velocity testing, mainly used to measure the sound velocity of the concrete and hence the compressive strength of the concrete.

— Sonic methods using an instrumented hammer providing both sonic echo and transmission methods.

- Tomographic modelling, which uses the data from ultrasonic transmission tests in two or more directions to detect voids in concrete.
- Impact echo testing, used to detect voids, delamination and other anomalies in concrete.
- Ground penetrating radar or impulse radar testing, used to detect the position of reinforcing bars or stressing ducts.
- Infrared thermography, used to detect voids, delamination and other anomalies in concrete and also detect water entry points in buildings.

1.1.3. Qualification and certification

The qualification and certification of NDT personnel for the inspection of concrete is not commonly covered by the qualification and certification schemes presently established in most countries. Usually such schemes are based on the requirements of the International Standards Organization (ISO) 9712 "The qualification and certification of NDT Personnel" and cover the use of methods such as ultrasonics, radiography, eddy current testing and surface methods tests to inspect essentially homogeneous materials such as metals. The growing interest in the use of NDT for the inspection of concrete may result in a demand for certification in the future.

1.2. BASIC MANUFACTURING PROCESSES AND DEFECTS OF CONCRETE STRUCTURES

1.2.1. Types of concrete structures

Concrete is a mixture of stone and sand held together by a hardened paste of cement and water. When the ingredients are thoroughly mixed they make a plastic mass which can be cast or moulded into a predetermined size and shape. When the cement paste hardens the concrete becomes very hard like a rock. It has great durability and has the ability to carry high loads especially in compression. Since it is initially plastic it can be used in various types of construction; however the forms used to produce the final shape can not be removed until the concrete has developed enough strength by hardening. Where tensile stresses are imposed on the concrete, it must be reinforced with steel.

1.2.1.1. Reinforced concrete

Reinforced concrete is a combination of concrete and steel. Alone concrete is very strong in compression but very weak in tension. Since concrete bonds firmly to steel reinforcement the combination acts as one material which offers high compressive strength, high tensile strength and high shear strength. Reinforcement in concrete also helps to control cracking such as shrinkage and surface cracking. There are two main types of reinforcement: deformed bars (i.e. with grooves) and mesh sheets, such as rectangular mesh, square mesh and trench mesh. The position of reinforcement is always shown on drawings. Steel reinforcement must be securely fixed in the right position. To ensure that the correct concrete cover over the reinforcement is being achieved, plastic bar chairs or concrete blocks should be used at the specified distance from the forms. Timber, bricks or stones should not be used. Reinforcement may be bent, hooked or lapped to suit design requirements and improve the bond between the steel and the concrete. The reinforcement must be clean and free from grease, dirt or flaky rust. It is necessary to have enough room to place and compact the concrete around the steel. Congested reinforcement will make compaction using internal vibrators difficult and may result in voids. Reinforced concrete is used for concrete slabs, decks, concrete pavements, columns, walls, concrete bridges, retaining walls etc.

1.2.1.2. Prestressed concrete

The basic principle of prestressed concrete is that superimposing compressive stresses eliminate tensile stresses in the concrete. This involves the installation of high tensile strength steel as reinforcement, stretching the steel by applying a pre-stressing force, and holding the tension. The pre-stressing force in the steel wire or strand is transferred to the concrete, placing the concrete under compression. Pre-stressing of beams is done either by pre-tensioning or post tensioning.

In pre-tensioning, the high strength steel wires or strands are tensioned against a fixed external anchorage before the concrete is poured. The concrete is then poured into the forms around the steel to develop bond: After the concrete has hardened, the pre-stress is transferred to the concrete by releasing the steel wires from the anchorage.

In post tensioning, after the concrete has hardened, the high strength steel tendons are passed through ducts cast into the concrete and then tensioned. The tension is then transferred to the concrete putting it under compression. To protect the steel tendons from corrosion, the ducts are filled with grout. The location of pre-stressing tendons is sometimes required, as assurance that the ducts have been properly grouted.

One of the common defects in post tensioned concrete bridges is the lack of grout in the post tensioning ducts. A lack of grout may allow the ingress of water and possibly the initiation of corrosion. As the failure mode of these bridges is “brittle” it is crucial to identify the ungrouted sections.

Possible NDT investigation procedures include:

— Radiography - this method requires a people exclusion zone to be maintained during testing for safety reasons.

— Ground penetrating radar to locate the tendon ducts followed by careful drilling to check whether the duct is fully grouted or a void exists. An endoscope can be used to view any voids.

1.2.2. Composition of concrete

The main ingredients of concrete are:

— cement
— coarse aggregate (i.e. screenings, gravel, etc.)
— fine aggregate (i.e. sand)
— chemical admixtures (if necessary)
— water.

Acceptable concretes usually have proportions within the following ranges (by volume)

—	cement	-	7% to 17%
—	water	-	15% to 20%
—	aggregate (coarse and fine)	-	78% to 63%
—	paste (water + cement)	-	22% to 37%.

It should be noted that the aggregates in the concrete mix constitute by far the bulk of the mass. The properties of the concrete produced depend upon the amount and type of materials used, and the way they are mixed, handled, compacted, finished and cured.

1.2.2.1. Cement

There are many types of cement available for making concrete. Some countries have a national standard, which prescribes the requirement for any cement manufactured or used in that country. e.g. Australian Standard AS3972-1991 "Portland and Blended Cements". Each type of cement produced has a different chemical composition and fineness and gives different properties to a concrete.

1.2.2.2. Portland cement

There are four main types:

(1) General purpose cement - the most common and the one used for the majority of structures
(2) Type HE - high early strengths cement - gains strength quickly
(3) Type LH - low Heat Cement - produces less heat than GP and Type HE cement. It gains strength more slowly
(4) Type SR - sulphate resisting cement - cement that resists sulphate attack

1.2.2.3. Blended cements

There are also two types of Blended Cements. These are mixtures of Portland cement, and either fly ash (FA) or blast furnace slag (SA).

Blended cement generally has slower rate of strength gain, and less heat is generated during curing. However, with adequate curing, impermeable and durable concrete with higher strength than that of normal cement can be achieved.

1.2.2.4. Other cements

Other cements include white and coloured cements, which are used for decorative finishes.

1.2.2.5. Mixing water

Water, which is necessary for hydration, must be clean and fresh and not contain any impurities since these may affect the concrete properties. It is generally accepted that water, which is fit for drinking, is suitable for making concrete. Seawater should not be used in making concrete, particularly reinforced concrete, as it will corrode the steel reinforcement. Bore water must be analysed first. Industrial, waste or brackish water should not be used. Sugars and detergents have a retarding effect on the setting properties of concrete and, therefore, should not be added to a concrete mix.

1.2.2.6. *Aggregates and their properties*

Coarse aggregates are stones that are more than 5 mm in diameter and are either crushed rock generally from quarries, or gravels excavated from pits or dredged from river beds. Fine aggregates consist of fine and coarse sands or crushed rock finer than 5 mm. The aggregates used should cover a range of sizes. They should be clean and free of any contaminating substances, which may adversely affect the setting time, strength or durability of the concrete, or corrode the steel reinforcement. Aggregates should not contain:

- weak substances such as pieces of wood, humus or coal
- dirt, clay dust or silt coatings. These reduce the bond between aggregates and the cement paste
- water soluble salts such as sulphates or chlorides.

Aggregates should be strong, hard and durable in order to develop the full strength of the cement paste and maximum wear resistance of the concrete. Crumbly or flaky rocks like sandstone, slates or shales should not be used since they lack strength. Chert aggregates should not be used to avoid alkali-silica reaction.

The shape of the aggregate particles is also important since the shape affects both the workability and strength of the concrete. Smooth rounded aggregates produce workable and easy to handle concrete. Angular materials tend to give a stronger concrete but reduce workability. Flaky and elongated materials promote segregation and reduce workability. They also require more sand and cement. The use of these aggregates should be limited.

There are certain types of aggregates that can react with cement in the presence of moisture. Such reactions result in expansive compounds, which crack and deteriorate the concrete. Suspect aggregates should not be used.

1.2.2.7. *Chemical admixtures*

Chemical (usually in liquid form) can be added to concrete to change its properties. They usually affect the time it takes for concrete to harden and the workability of freshly mixed concrete. The most common types of chemical admixtures are:

(1) Set-accelerating admixtures, which speed up concrete setting
(2) Set-retarding admixtures, which slow down concrete setting
(3) Water reducing admixtures (or plasticizers), which help to improve the workability of concrete
(4) High range water reducing admixtures (super plasticizers) – these help improve workability of concrete and its ability to flow into congested areas of steel reinforcement
(5) Air-entraining admixtures – these put air bubbles into the concrete and make the concrete more workable and cohesive. They also reduce segregation. They are very useful in cold weather where they improve durability.

Admixtures should be used in a controlled manner as part of the overall concrete mix design. Misuse can be detrimental to concrete's properties.

1.2.3. Process of concrete manufacture

The process of concrete manufacture is simply:

Aggregates + Cement + Water + Chemical Admixtures = Concrete

However, the place of manufacture can either be at a construction site as a small batch produced in a portable concrete mixer or at a large batching plant at the construction site or transported by concrete mixing truck from a concrete plant some distance from the construction site. In the latter case the concrete is called ready mix concrete.

If ready mix concrete is being ordered from a concrete plant the manufacturer needs to know the

— intended use of it (i.e. kerb, slab, etc.)
— amount required in cubic meters
— strength required (i.e. Megapascals, MPa)
— slump in mm
— maximum size aggregate (i.e. 14 mm, 20 mm, etc.)
— method of placement (i.e. pump, off the chute, etc.), and any admixtures required.

$$\frac{\text{Force}}{\text{Area}} = \frac{\text{Newtons}}{\text{square millimeter s}} = \frac{\text{N}}{\text{mm}^2} = \text{MPa} \qquad (1)$$

Typical strengths range from 15 MPa to 50 MPa. For instance, for simple driveways, footpaths and other domestic work, typical strengths are in the order of 15 MPa to 25 MPa. The structural concrete required for bridge deck slabs should be about 32 MPa, columns and superstructures 40 or 50 MPa and concrete pavements about 30 MPa. Typical slumps range from 40 mm to 80 mm for normal concrete. If super plasticizers are used, the slump can be as high as 160 to 200 mm. The typical mix proportions of cement, sand, aggregate and water-to-cement which produce concrete of 15 MPa, 20 MPa and 25 MPa strength is given in Table 1.1.

TABLE 1.1. TYPICAL MIX PROPORTIONS FOR 20 MM MIX

	Mix Proportions			
Strength	Cement	Sand	Aggregate	Water/Cement Ratio
15 MPa	1	2.5	4	0.60
20 MPa	1	2	3	0.55
25 MPa	1	2	2.5	0.50

The concrete mix design used must take into account the required properties of the concrete in the plastic state, the method of placement and the in-service conditions of the concrete (i.e. traffic load, exposure conditions, chemicals, etc. However, the first factors to be considered are the desired compressive strength and slump since these are usually used to specify the concrete required. The proportions of each material in the mixture affect the properties of the final concrete, as follows:

— As the cement content increases both strength and durability increase.

— As the water content increases the concrete becomes weaker; hence, there should just be enough water to make the mix workable.

— As the water/cement ratio increases, strength and durability decrease.

— As the fine aggregate increases the mix becomes sticky and, after compaction, the top few millimeters of concrete become very sandy.

— As the coarse aggregate increases the mix becomes bony and some of the stones can protrude from the surface after compaction.

When concrete is placed in the formwork after thorough mixing, care must be taken not to damage or move the formwork or the reinforcing steel. Also care must be taken to ensure that the concrete does not segregate. For instance the concrete should not be dropped from heights greater than 2.0 meters. The formwork is filled by starting to place the concrete from the corners of the formwork, and from the lowest level if the surface is sloping. Place each load of concrete into the face of the previous plastic concrete, not away from it. Deposit the concrete in horizontal layers and compact before the next layer is placed. Do not place the concrete if the air temperature is below 5°C or above 35°C and never spread concrete with a poker vibrator as segregation will occur. At all times avoid delays. The concrete is then compacted by vibrating the concrete to force the air out and fill all the voids. Concrete is compacted to make it dense, strong and durable. Both external and internal vibration can be used. During external vibration a mechanical screed is used to compact flat slabs. Two workers pull the screed along the top of the forms and external vibrators are attached to the formwork. For internal vibration a poker vibrator is placed in the concrete while the concrete is still in the plastic state. It is kept vertical and taken out very slowly. This is to avoid making holes in the plastic concrete. There are different sizes of poker vibrators. To prevent cold joints the poker vibrator should be long enough to reach the previous layer of concrete. The internal vibrator should not be vibrated at any point for more than 15 seconds. Excessive vibration should be avoided. The formwork should not be touched with the poker. Do not rest the poker vibrator on the reinforcement. Do not move the concrete with the poker vibrator. Use a shovel if the concrete has to be moved.

The required appearance of the concrete is obtained by levelling and smoothing. Levelling and smoothing are done by screeding, floating or trowelling. Initial finishing takes place right after placing and vibrating. The concrete is screeded (with a screed board) to the level of the formwork and bullfloated if necessary and left to set. As the concrete sets, bleed water comes to the surface. Final finishing can only begin when this bleed water dries up, and the concrete can support finishers with only a slight indentation (about 5 mm). Any area with free surface water should not be finished since if the finishing is too early a weak surface and laitance will be produced. Cement should not be used to dry up surface water since this will produce a weak surface and cause cracking. Brooming, colouring or patterned finishes can be applied. Final finishing involves floating, trowelling, edging and jointing. Floating is done using a wooden hand float or power float. Floating helps to smooth irregularities, embed large aggregate and close minor cracks, which can occur as the surface dries out. Hand floats produce a rougher texture. Steel trowelling is done after floating is finished. It provides a smooth, dense and hard surface, which is also durable and easy to clean. This kind of surface is slippery when wet. The surface should be trowelled at least twice. Trowelling can be done by hand or power trowel.

Slab edges are finished with a special edge tool. This gives a neater and stronger edge. Joints are preplanned and cut into concrete during finishing. Redo edges and joints after trowelling to maintain uniformity and fine lines. During or after placing and finishing it may be necessary to protect concrete from the weather.

After the concrete has been finished it must be cured. Curing is the process whereby the concrete is kept moist to maximize the concrete's strength and durability by maintaining the hydration reaction as long as possible to produce more cement products.

There are two types of curing methods.

(1) Methods that supply more moisture to the concrete, e.g. ponding, sprinkling and wet coverings (i.e. hessian or sand). This prevents the concrete from crazing or cracking due to drying.
(2) Methods that stop the loss of moisture by sealing the surface, e.g. leaving the forms in place, covering with plastic sheets or using spray-on compounds.

Concrete should be cured for as long as practicable since concrete becomes stronger and more durable with longer curing. It is preferable to cure concrete for at least seven days.

Both hot and cold temperatures can cause problems to concrete, particularly in its plastic state and early curing period. Adverse weather conditions also include dry, windy, low humidity or frost conditions. The main problems with hot weather are associated with cracking since the concrete stiffens quickly (loses its workability) and it is more difficult to place and finish. The shrinkage of the concrete also increases, increasing the tendency for cracking of the concrete surface (i.e. plastic shrinkage cracking). It also increases the danger of cold joints forming. In cold weather concrete takes much longer to set, gain strength and finish. Below freezing point the water in the concrete turns to ice which expands and can cause cracking of concrete.

1.2.4. Properties of concrete and their control

1.2.4.1. Plastic-state concrete

The two most important properties of plastic state concrete are workability and cohesiveness.

Workability describes the ease with which concrete is mixed, handled, placed, compacted and finished. Concrete, which is stiff or dry, may be difficult to mix, handle, place, compact and finish. Concrete, which is runny or wet, may be easy to place but more difficult to handle and properly compact to a dense material. There are many factors, which affect workability:

— The mix becomes harsher and less workable if the amount of cement is reduced provided the amount of aggregate remains the same.

— The mix becomes more workable if the amount of cement is increased provided the amount of aggregate remains the same.

However, an excessive amount of cement produces a very sticky and unworkable mix.

If the aggregate grading, size and shape are considered:

— Well-graded aggregates with different particle sizes (i.e. from large - about 20 mm, to small - about 14 to 10 mm) produce a more workable concrete. Also, well graded aggregates that are smooth, round and as large as possible improve workability.

— Rough, angular aggregates produce less workable concrete.

— Chemical admixtures increase the workability of concrete by lubricating and dispersing the cement particles.

— Never make concrete more workable by just adding water. Increasing the water content without an increase in cement content lowers the strength and durability of concrete.

— To make a more workable mix, add more cement (paste), use well graded aggregates and chemical admixtures.

Cohesiveness measures how well the concrete holds together. Factors affecting cohesiveness, are:

— A mix that has too much water will not be cohesive and may separate and bleed. A dry mix can crumble, with the coarse aggregate segregating from the cement paste and sand.

— A well graded aggregate gives a more cohesive mix. Less fine aggregate (sand) gives a bony mix, which tends to segregate. Excess fine aggregate makes the concrete cohesive, but sticky and difficult to work and place.

1.2.4.2. Hardened concrete

The two most important properties of hardened concrete are durability and strength.

DURABILITY is described as the ability of concrete to resist wear and tear and other in-service conditions without breaking up. Concrete durability increases with strength. Durable concrete is dense and watertight. Durability is very important to protect steel in reinforced concrete.

COMPRESSIVE STRENGTH is a measure of concrete strength in the hardened state. Concrete is very strong in compression. It is not strong in tension because it has a low tensile strength.

Durability and strength increase with lower water content, higher cement content, higher densities, extended moist curing and correct type of cement. Therefore, if water-to-cement ratio is altered by raising water content, the concrete will be less durable and weaker. Proper compaction will also give higher densities and improve strength and durability. Curing time is also important. The longer the concrete is cured and kept damp, the stronger and more impermeable and durable it will be.

A lower cement content means weaker and less durable concrete. Different types of cement may gain strength quickly or slowly. They also have different resistance to aggressive conditions.

1.2.5. Discontinuities and defects in concrete structures

The most common defects that occur in concrete, their causes and some of the ways they can be avoided are, as follows.

1.2.5.1. Cracking of concrete

Cracking affects the appearance of concrete. In some cases it affects its structural adequacy and durability. In reinforced concrete cracking allows easier access to air and moisture which can cause steel to rust and eventually weaken the concrete. Cracks can occur at two stages.

(a) Before concrete hardens

Movement of concrete causes these cracks to occur before the concrete has set. They include:

— plastic shrinkage cracks
— plastic settlement cracks
— cracks caused by movement of the formwork.

If they are detected before the concrete sets they can be retrowelled or refloated. If they are only detected when the concrete hardens they should be filled with epoxy resin.

Plastic shrinkage cracks

This type of crack is encountered in hot and windy conditions. They appear as straight lines either parallel or pattern. They are similar to crazing but in a larger scale. To avoid this type of cracking concrete should be compacted and finished as fast as possible on hot days and the forms should be dampened to protect the concrete from heat and wind.

Plastic settlement cracks

This type appears while concrete is still plastic and tends to follow the lines of reinforcement.

Formwork movement cracks

These cracks may occur during placement and compaction due to movement of a weak formwork. It is essential to ensure that the formwork is strong and well propped and braced.

(b) After concrete hardens

Cracks, which occur in the concrete when it has hardened, may be due to drying shrinkage, settlement, structural cracks, etc. They may require structural repair such as high pressure epoxy injection or other means.

1.2.5.2. Spalling

This occurs when concrete edges or other surfaces chip or break. Spalling can be repaired by breaking out to sound and dense concrete then wetting and refilling the area with a cement material that is then compacted, finished and cured. Since this is a visual defect, non-destructive testing is not an applicable repair technique.

1.2.5.3. Honeycombing

This is when too much coarse aggregate appears on the surface with some cavities underneath. It occurs as a result of poor compaction or if a bony mix is used with not enough sand. If it only occurs on the surface it can be reprofiled with a render (thin layer of sand/cement mortar) or a proprietary cement product. If cavities exist below the surface, it is more appropriate to break out to sound and dense concrete and repair as per spalling above.

1.2.5.4. Dusting

This is a surface defect that appears as fine powder on the concrete surface and comes off when brushed. It is caused by finishing the concrete before bleed water has dried out, as well as by inadequate curing. It is repaired by applying a chemical floor hardener or bonded topping.

1.2.5.5. Crazing

This type of cracking resembles a map pattern. The cracks only extend through the surface layer. It is caused by minor surface shrinkage as a result of the drying conditions. It is avoided by finishing and curing as soon as possible. These cracks do not cause any subsequent deterioration of the concrete. If appearance is a problem a surface coating of paint can be applied to cover the cracks.

1.2.5.6. Rain damage

Surface pitted or eroded concrete can occur as a result of heavy rain. It is avoided by covering newly placed concrete with plastic sheeting when it rains. If there is rain damage and the concrete has not hardened it can be reworked and refinished.

1.2.5.7. Efflorescence

This is a white crystalline deposit, which appears soon after completion. It is removed by dry brushing and flushing with clean water. Efflorescence has no effect on the structural performance of the concrete.

1.2.5.8. Blistering

Blisters occur when the fresh concrete surface is sealed by trowelling trapping air or bleed water under the surface. It is avoided by delaying trowelling as long as possible and covering to prevent evaporation.

1.2.5.9. Corrosion of reinforcing bars

Corrosion occurs when the concrete surface cracks allowing water entry, or if water enters the concrete by diffusion during carbonation. The increase in diameter of the reinforcing bars caused by the formation of iron oxide (rust) can cause the concrete above the affected bars to spall off.

1.2.6. Situations where NDT is an option to consider for investigation of *in situ* concrete

- to investigate the homogeneity of concrete mixing
- lack of grout in post tensioning ducts
- to determine the density and strength of concrete in a structure
- to determine the location of reinforcing bars and the cover over the bars
- to determine the number and size/diameter of reinforcing bars
- to determine the extent of defects such as corrosion
- to determine the location of in-built wiring, piping, ducting, etc.
- to determine whether internal defects such as voids, cracks, delaminations, honeycombing, lack of bonding with reinforcing bars, etc. exist in concrete
- to determine if there is a bond between epoxy bonded steel plates and concrete members.

1.3. TESTING OF CONCRETE

1.3.1. Quality control tests

Two simple tests are used to control the quality of concrete: SLUMP TEST is used when the concrete is in the plastic state. COMPRESSION TEST is used when concrete is in the hardened state. Both tests are used for the quality control of concrete during manufacture. The compression test can also be used to test a structure, which has been in service for some time by drilling a core from the structure and testing it in compression.

1.3.1.1. Slump tests

Slump test assesses the consistency or workability of concrete. e.g. Australian Standard 1012, Part 3. The acceptance of a load of concrete may depend on the results of a slump test.

The first step in testing is to take a test sample from the batch of concrete within 20 minutes of the concrete arriving on site. Normally a visual inspection is also undertaken prior to placing, to estimate the slump and ensure consistency of the concrete. The sample is taken in one of two ways:

- Sampling after 0.2 m^3 of the load has been poured (the most common method), or
- Sampling from three places in the load, at equal intervals and equal portions, during the discharge.

1.3.1.2. Method of slump test

The tools required to carry out the test are: standard slump cone (100 mm × 200 mm × 300 mm), small scoop, bullet-nosed tamping rod (600 mm × 16 mm), ruler, float and slump plate (500 mm × 500 mm). The test is performed by (a) cleaning and moistening the cone, and (b) placing it on the flat slump plate. Fill the cone one-third full with concrete and rod the layer exactly 25 times making sure that the whole area is rodded uniformly. Rodding means pushing a steel rod in and out of the concrete to compact it into a slump cone or a cylinder mould. Always rod in a definite pattern, working from outside into the middle. The cone must be held firmly by standing on the foot lugs while the concrete is being added during rodding.

After rodding the first layer fill the cone with a second layer until two-thirds full and rod this layer uniformly 25 times just into the top of the first layer. Then fill the cone until it slightly overflows and rod this top layer 25 times uniformly just into the top of the second layer. The excess concrete is removed from the top with a straight edge so that the cone is exactly filled and the spilled concrete removed from around the bottom of the cone. The cone is then lifted straight up very slowly. Without disturbing the concrete further turn the cone upside down and place the rod across the up-turned cone. Measure the distance from the rod to the top of the slumped concrete. If the top of the slump is irregular, do not measure the high point or the low point. Try to get the average. If the slump is too high or too low compared to the specification, another must be taken. If this fails the remainder of the batch should be rejected.

1.3.1.3. Compression test

Compression test determines the strength of concrete under standard conditions. Concrete cylinders or concrete cubes are used for the compression test depending on the national standard or contract requirements.

The methods used to conduct compression tests on concrete are given in the appropriate national standard, e.g. in Australia concrete test cylinders are prepared and tested in accordance with the Australian Standard 1012, Part 8, “Making and curing concrete test specimens”.

1.3.1.4. Compression test method

The concrete test samples, whether cylinders or cubes, are made on site and tested in a laboratory with a compression test machine. Moulding the test sample should be completed within 20 minutes of obtaining the sample. The compressive strength of the test samples determines the acceptability of the concrete represented.

Assuming test cylinders have to be tested, the tools required are: a) three-cylinder moulds (100 mm × 200 mm or 150 mm × 300 mm), b) small scoop, c) bullet-nosed tamping rod (600 mm × 16 mm), d) steel float, e) steel slump plate (500 mm × 500 mm), and f) rubber mallet.

First clean the cylinder mould and slump plate and coat the inside of the mould and plate with a thin film of mineral oil to prevent adhesion of the concrete. For the 100 mm × 200 mm cylinders the mould is then filled to one-half and the concrete compacted by rodding with the tamping rod 25 times. The strokes should be uniformly distributed over the cross-sectional area. The mould is then overfilled and compacted by rodding 25 times into the top of the first layer. If after compaction the top is not completely filled, add more concrete and work into the concrete surface. Each mould is tapped all around with a rubber mallet to remove air bubbles and assist compaction of the concrete. If the 150 mm × 300 mm cylinder mould are used the concrete is compacted in three equal layers instead of two. Then (a) the top of the concrete is leveled off with the tamping rod and any concrete around the mould is cleaned, (b) the surface of the concrete is smoothed with a wooden float, (c) the cylinders are capped, (d) the moulds identified with a code number and left in a cool dry place to set undisturbed for at least 24 h. The mould is then removed and the concrete cylinder marked and sent to the laboratory where it is cured for a specified period prior to testing in compression. All moulds are cleaned and oiled after use to prevent rusting. The curing period depends on the specification, although seven days and 28 days are commonly used.

The capping carried out on the test specimen before testing is to make the top surface of the specimen as smooth and plain as possible. If the specimen surface, which is in contact with the platen of the compression test machine, is rough and not plain, stress concentrations are introduced and the apparent strength of the concrete is greatly reduced. Suitable capping materials are aluminous cement, high strength dental plaster and a molten sulphur mixture. However, other capping materials have been used. The main requirement is that the capping compound should not be weaker than the concrete, or appreciably stronger.

1.3.2. Partial destructive tests

1.3.2.1. Pullout test

There are two options for the pullout test:

- DANISH LOK TEST which requires that the head be cast into the concrete at the time of construction. This test gives a good indication of near surface compressive strength.
- Building Research Establishment, UK (BRE) PULLOUT involves drilling a hole and inserting a “fixing” which is pulled out. The advantage of this test is that it does not require a head to be cast into the concrete during construction. The disadvantage is that the test really measures tensile strength and is then calibrated to compressive strength.

The pullout test is a test that falls in the transition area between a destructive test and a non-destructive test. It is destructive in the sense that a relatively large volume of the concrete is damaged but non-destructive because the damaged can be repaired.

The pullout test measures the force required to pull an embedded metal insert with an enlarged head from a concrete specimen or a structure. Fig. 1.1 illustrates the configuration of a Danish Lok pullout test. The insert is pulled by a loading ram seated on a bearing ring that is concentric with the insert shaft. The bearing ring transmits the reaction force to the concrete. Frustum geometry is controlled by the inner diameter of the bearing ring (D), the diameter of the insert head (d), and the embedment depth (h). The apex angle (2α) of the idealized frustum is given by:

$$2\alpha = 2\tan^{-1}\left(\frac{D-d}{2h}\right) \qquad (2)$$

The pullout test is widely used during construction to estimate the in-place strength of concrete to help decide whether critical activities such as form removal, application of post tensioning, or termination of cold weather protection can proceed. Since the compressive strength is usually required to evaluate structural safety, the ultimate pullout load measured during the in-place test is converted to an equivalent compressive strength by means of a previously established correlation relationship.

As the insert is pulled out, a conical shaped fragment of concrete is extracted from the concrete mass. The idealized shape of the extracted conic frustum is shown in Fig. 1.1.

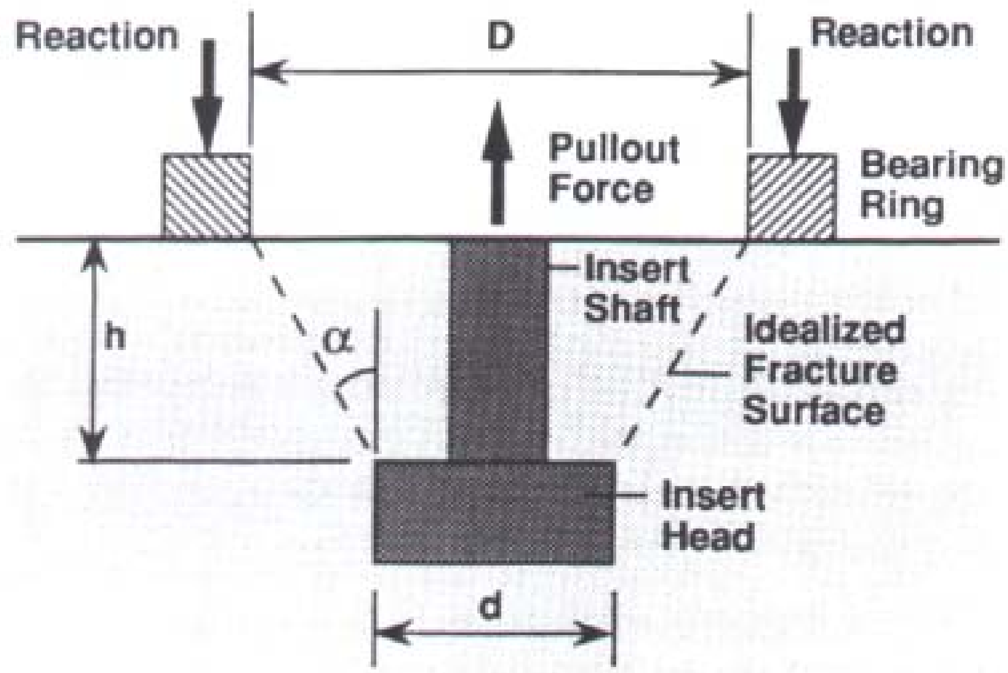

FIG. 1.1. Schematic of the pullout test.

Unlike some other tests that used to estimate the in-place strength of concrete, the pullout test subjects the concrete to a slowly applied load and measures an actual strength property of the concrete. However, the concrete is subjected to a complex three dimensional state of stress, and the pullout strength is not likely to be related simply to uniaxial strength properties. Nevertheless, by use of correlation curves the pullout test can be used to make reliable estimates of in-place strength. An important step in implementing the method is choosing the locations and number of pullout tests in a given placement of concrete. The inserts should be located in the most critical portions of the structure and there should be a sufficient number of tests to provide statistically significant results. Additional inserts are recommended in the event that testing begins too soon, and the concrete has not attained the required strength. The use of maturity meters along with the pullout tests is encouraged to assist in selecting the correct testing times and in interpreting possible low strength results.

The BRE pullout test was developed to permit testing in an existing construction by drilling a hole and inserting some type of expansion anchor. The results of these tests are difficult to interpret if a correlation curve does not exist for the concrete used in the construction.

1.3.2.2. Pulloff test

This test involves attaching a plate to the concrete using epoxy resin and, after curing has taken place, measuring the force required to pull the plate off. This test scars the concrete but gives a measure of the near surface tensile strength which can be converted to the compressive strength provided a correlation exists between the compressive strength and tensile strength for the concrete mix being investigated.

1.3.2.3. Core test

In most structural investigations or diagnoses extraction of core samples is unavoidable and often essential. Cores are usually extracted by drilling using a diamond tipped core cutter cooled with water. Broken samples, for example, due to popping, spalling and delamination, are also commonly retrieved for further analysis as these samples may provide additional evidence as to the cause of distress. The selection of the locations for extraction of core samples is made after non-destructive testing which can give guidance on the most suitable sampling areas.

For instance, a covermeter can be used to ensure there are no reinforcing bars where the core is to be taken; or the ultrasonic pulse velocity test can be used to establish the areas of maximum and minimum pulse velocity that could indicate the highest and lowest compressive strength areas in the structure.

Moreover, using non-destructive tests, the number of cores that need to be taken can be reduced or minimized. This is often an advantage since coring is frequently viewed as being destructive. Also the cost of extracting cores is quite high and the damage to the concrete is severe.

The extracted cores can be subjected to a series of tests and serve multiple functions such as:

- confirming the findings of the non-destructive test
- identifying the presence of deleterious matter in the concrete
- ascertaining the strength of the concrete for design purposes
- predicting the potential durability of the concrete
- confirming the mix composition of the concrete for dispute resolution
- determining specific properties of the concrete not attainable by non-destructive methods such as intrinsic permeability.

1.3.3. Other tests

1.3.3.1. Tensile tests

Tensile tests are mainly carried out on the reinforcing bars and stressing tendons used in concrete construction.

The application of a tensile test to concrete is not normally required because concrete is not designed to resist direct tension. However, knowledge of tensile strength is of value in estimating the load under which cracking will develop. The absence of cracking is of considerable importance in maintaining the continuity of a concrete structure and in many cases in the prevention of reinforcement corrosion. Cracking problems occur, for instance, when high-tensile steel reinforcement is used, or when diagonal tension arising from shearing stresses develops, but the most frequent case of cracking is due to restrained shrinkage and temperature gradients. An appreciation of the tensile strength of concrete helps in understanding the behaviour of reinforced concrete even though the actual design calculations do not in many cases explicitly take the tensile strength into account.

Since the direct application of a pure tension force that is free from eccentricity is difficult and further complicated by secondary stresses induced by the grips or by embedded studs, it is preferable to measure the tensile strength of concrete by subjecting a plain concrete beam to flexure. This is in fact the only standard tension test.

1.3.3.2. Flexure test

In a flexure test on a beam the theoretical maximum tensile stress reached in the bottom fibre of the test beam is known as the modulus of rupture. The qualification “theoretical” refers to the assumption in the calculation of the modulus of rupture that stress is proportional to the distance from the neutral axis of the beam while the shape of the actual stress block under loads nearing failure is known to be not triangular but parabolic. The modulus of rupture thus overestimates the tensile strength of concrete and gives a higher value than would be obtained in a direct tension test on a briquette or a bobbin made of the same concrete. Nevertheless, the test is very useful, especially in relation to the design of road slabs and airfield runways because the flexure tension there is a critical factor.

The value of the modulus of rupture depends on the dimensions of the beam and, above all, on the arrangement of loading. Two systems could be used: a) central point load, which gives a triangular bending moment distribution so that the maximum stress occurs at one section of the beam only, and b) symmetrical two point load that produces a constant bending moment between the load points. With the latter arrangement, a part of the bottom surface of the beam — usually one-third of the span — is subjected to the maximum stress, and the critical crack may start at any section not strong enough to resist this stress. On the other hand, with a central point load, failure will generally occur only when the strength of the fibre immediately under the load point is exhausted.

This statement is not strictly correct, for a fibre subjected to a stress lower than the maximum acting on the beam may also be weak enough to fail. However, it can be seen that the probability of a weak element (of any specified strength) being subjected to the critical stress is considerably greater under two point loading than when a central load acts. Since concrete consists of elements of varying strength, it is to be expected that two-point loading will yield a lower value of the modulus of rupture than one point loading. The difference can be gauged from Fig. 1.2.

There are four possible reasons why the modulus of rupture test yields a higher value of strength than a direct tensile test made on the same concrete. The first one is related to the assumption of the shape of the stress block mentioned earlier. The second one is that accidental eccentricity in a direct tensile test results in a lower apparent strength of the concrete. The third is offered by an argument similar to that justifying the influence of the loading arrangement on the value of the modulus of rupture: under direct tension the entire volume of the specimen is subjected to maximum stress so that the probability of a weak element occurring is high. Fourthly, in the flexure test, the maximum fibre stress reached may be higher than in direct tension because the propagation of a crack is blocked by less stressed material nearer to the neutral axis. Thus available energy is below that what is necessary for the formation of new crack surfaces. These four reasons for the difference between the modulus of rupture and the direct tensile strength are not all of equal importance.

Fig. 1.3 shows a typical relation between the direct tensile strength and the modulus of rupture, but actual values may vary depending on the properties of the mix.

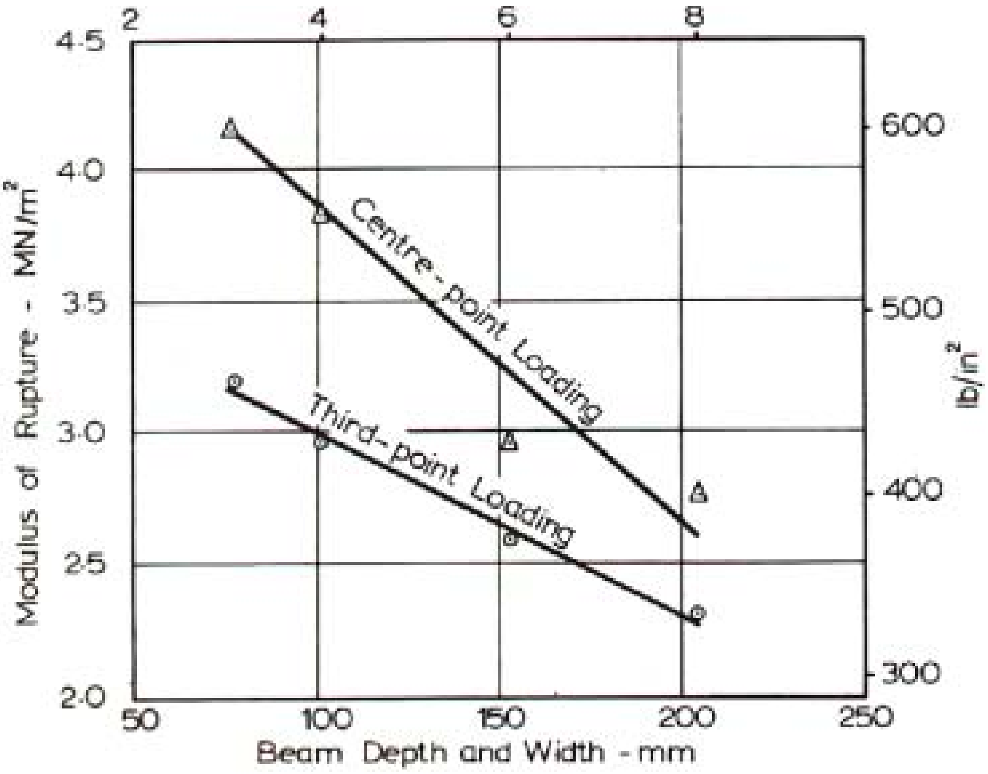

FIG. 1.2. Modulus of rupture of beams of different sizes subjected to centre-point and third-point loading.

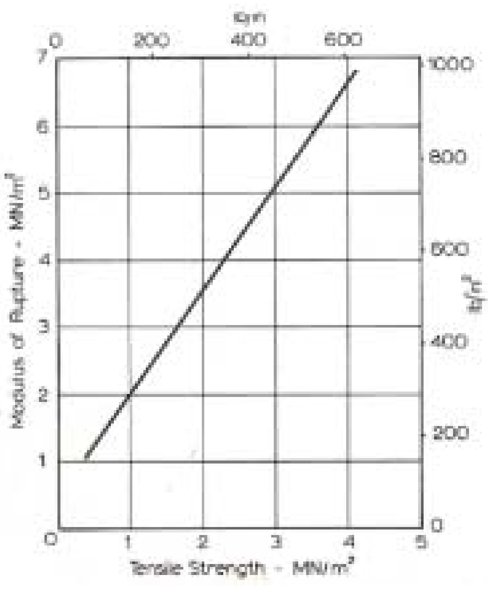

FIG. 1.3. Relation between modulus of rupture and strength in direct tension.

If fracture occurs within the central one-third of the beam, the modulus of rupture is calculated on the basis of ordinary elastic theory.

$$\text{Modulus of rupture} = \frac{PL}{bd^2} \tag{3}$$

where

P is the maximum total load on the beam,
L is the span,
b is the width of the beam,
d is the depth of the beam.

If, however, fracture occurs outside the load points, say, at a distance 'a' from the near support, 'a' being measured along the centre line of the tension surface of the beam, then the modulus of rupture is given by $3Pa/bd^2$. This means that the maximum stress at the critical section, and not the maximum stress on the beam, is considered in the calculations.

Beams are normally tested on their side in relation to the as-cast position but, provided the concrete is unsegregated, the position of the beam as tested relative to the as-cast position does not affect the modulus of rupture.

1.3.3.3. Splitting Test

An indirect method of applying tension in the form of splitting was suggested by Fernando Carneiro, a Brazilian, and the test is often referred to as the Brazilian test, although it was also developed independently in Japan. In this test a concrete cylinder, of the type used for compression tests, is placed with its axis horizontal between the platens of a testing machine, and the load is increased until failure by splitting along the vertical diameter takes place.

If the load is applied along the generatrix then an element on the vertical diameter of the cylinder is subjected to a vertical compressive stress (vts) of

$$\text{vts} = \frac{2P}{\pi LD}\left(\frac{D^2}{r(D-r)} - 1\right) \tag{4}$$

and a horizontal tensile stress (hts) of

$$\text{hts} = \frac{2P}{\pi LD} \tag{5}$$

where

P is the compressive load on the cylinder,
L is the length of the cylinder, D is the diameter of the cylinder,
r(D-r) is the distance of the element from the two loads, respectively.

However, immediately under the load a high compressive stress would be induced, and in practice narrow strips of packing material, such as plywood, are interposed between the cylinder and the platens. These strips are usually 3 mm thick, and it is convenient to make their width equal to 1/12 of the diameter of the cylinder; ASTM Standard C 496-71 prescribes a width of 25 mm. Under such circumstances the horizontal stress on a section containing the vertical diameter is as shown in Fig. 1.4.

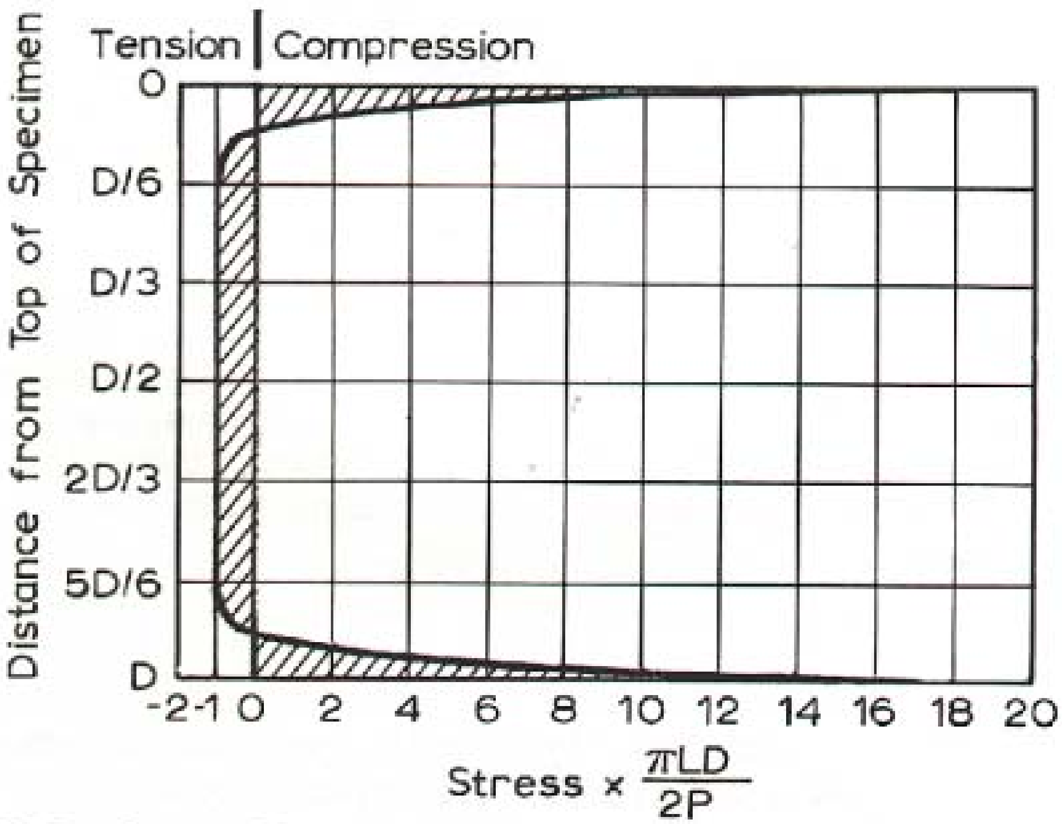

FIG. 1.4. Distribution of horizontal stress in a cylinder loaded over a width equal to 1/12 of its diameter.

Stress is expressed in terms of 2P/πLD. It can be seen that a high horizontal compressive stress exists in the vicinity of the loads. However, as this is accompanied by a vertical stress of comparable magnitude, producing a state of biaxial stress, failure in compression does not take place. Results of splitting tests on different concretes are shown in Fig. 1.5.

1.3.3.4. Chemical analysis

The chemical composition of concrete as determined from extracted core samples is usually essential in making an informed assessment of the concrete condition, its potential durability and suitability for continued use. Through such analysis, certain causes leading to

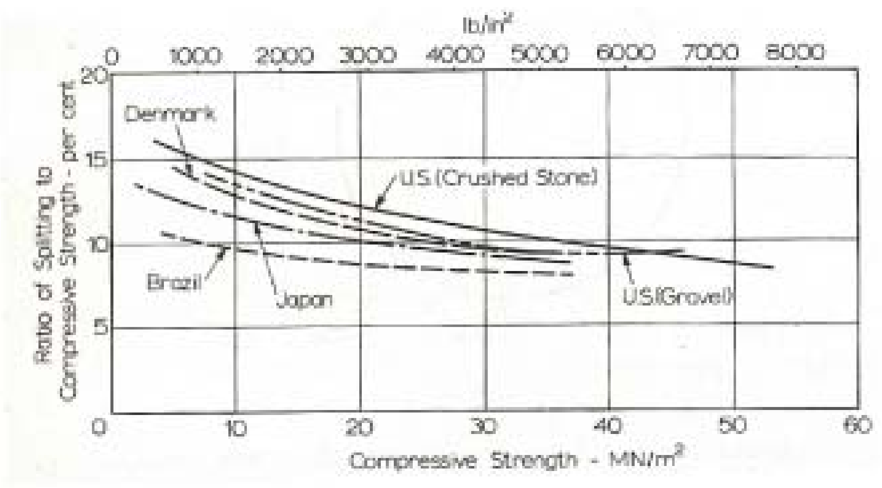

FIG. 1.5. Tensile splitting strength of cylinders of varying compressive strength.

degradation of the structure may also be established. These include, for instance, chloride content, sulphate content and pH value. The analyses are often requested for evaluation for compliance against the specification, in particular, the mix composition as compared to the design. Findings obtained from the analyses can then be used, for instance, to formulate a repair programme and resolve disputes.

Typical information obtainable and usually required include the following:

— cement content
— original water content and water cement ratio
— aggregate grading
— chloride content
— sulphate content.

The methods of determining the above are covered in British Standard 1881: Part 124.

Sometimes an analysis of the type and amount of admixtures is requested. Such an analysis is much more complicated and difficult, as it requires special techniques and instruments. Examples include gas liquid chromatography, high pressure liquid chromatography, Fourier transformed infrared spectroscopy and thin layer chromatography. Other allied techniques to identify various characteristics of the hardened concrete include X ray fluorescence spectroscopy, differential thermal methods and X ray diffraction.

As in all attempts to break down the composition of a complex material, there are bound to be uncertainties. Ideally, the individual constituents used in concrete should be made available for analysis. However, practically, this is rarely possible usually because these ingredients are no longer available at the time of testing. This is especially the case for old structures. In such situations, assumptions have to be made with regard, for example, to the types of cement and aggregates used and typical composition of the cement mixed.

Adequate sampling is probably one of the primary factors in the uncertainties of the analysis particularly when considering the heterogeneity of concrete in large structures. The assumptions made in the analysis in the absence of available raw materials would also contribute to errors. Subsequent secondary reaction or degradation of the concrete in service such as carbonation, leaching, chemical attack and alkali silica reaction can also induce inaccuracies in the analysis.

A guide to the test procedures and interpretation of results is given in the Concrete Society Technical Report No. 32.

1.3.3.5. Microscopical examination

The various microscopical methods, which may be applied to the study of hardened concrete, are derived from the science of petrography. Microscopical methods have been used in research since the chemistry and mineralogy of cements were first investigated but, over the last twenty years, petrography has increasingly become part of the routine procedure for the analysis of many practical concrete problems.

Petrography involves the use of a variety of microscopical techniques including stereo microscopy, polarizing microscopy, fluorescent microscopy, metallurgical microscopy and even electron microscopy for sub-micro examination. Quantitative analysis is made by using a point counter or an image analyser and reference thin sections are also required. The examinations are carried out on hand bulk specimens, ground sections, polished sections, thin sections usually with dye impregnation and small fragments depending on the technique employed. However, it does not lend itself to a rigid standardized approach as the complex nature of many of the problems to which it is applied calls for a high degree of flexibility. Usually at the onset of an examination, the most appropriate techniques will only become apparent once evidence is gathered from initial analysis. Therefore, ASTM C856-95 only offers guidance on the examination rather than a standard procedure. Determination of entrained air voids, which include parameters such as distribution and size, is covered in ASTM C457.

Petrography has enormous applications, as follows:

— identification of ingredients used such as types of cement, aggregates and cement replacement materials

— estimation of original mix proportion for example cement content, aggregates content and water cement ratio

— determination of air void content which include entrained air and entrapped air

— investigation of chemical and durability performance for instance:
  - chemical attack
  - alkali silica reaction, aggregate or cement paste shrinkage
  - frost attack
  - carbonation
  - leaching
  - detection of unsound contaminants
  - fire damage.

The purpose of the examination is similar to that outlined for chemical analysis but it covers a much wider scope of uses. This method can be used to ascertain the degree of curing or hydration of the cement, degree of compaction, homogeneity of the paste matrix, degree of cement dispersion and how well the concrete has been mixed. It also has other added advantages over the conventional chemical analysis. It allows physical evidence of damage or deterioration to be visually documented such as alkali silica reaction, sulphate attack, acid attack and delayed ettringite formation. In fire damaged concrete the method also permits assessment of the degree of degradation and an estimation can be made of the temperature to which the concrete has been exposed at various depths from the surface of exposure.

Since the examination basically involves microscopical means, the volume of concrete examined can be rather small and may not represent the condition of the bulk of the concrete especially in large structures. Selection of an appropriate number of specimens and locations is thus essential. Advice from the petrographer on such selection will improve the confidence level of the analysis. Very often, the petrographer will personally select the location for extraction of samples for examination. Such highly specialized examination should only be made and interpreted by a trained and experienced petrographer. Often, the findings of petrographic examination need to be complemented by chemical analysis.

1.3.3.6. Moisture measurements in concrete

There are a number of techniques available for measuring the moisture content in concrete.

— electrical conductivity measurement
— capacitance meter
— microwave meter
— radar
— neutron gauge
— gamma backscatter.

(a) Electrical conductivity measurement

The electrical conductivity is measured between two electrodes attached to the surface and it is often used as a quantitative measure of the moisture content of the concrete test object. However, there are serious limitations to this technique.

Conductivity depends highly on the dissolved salt content. The higher the salt content the higher the conductivity for the same moisture content.

The technique measures only surface or near surface values. There is little influence of the deeper layers.

Absolute values of the moisture content should not be taken. Several readings in a mesh can help identify areas with a potential problem, but must be verified with some other test method.

(b) Capacitance meters

These gauges are based on the principle of an electrical capacitor where capacitance depends on the dielectric properties of the medium between plates. The capacitor typically consists of a centre electrode and a symmetrical ring. A variety of other designs exist. The dielectric constant describes the ability of the medium to amplify an applied electrical field. Since water has such a high dielectric constant ($\simeq$81), it increases the dielectric constant of dry concrete ($\simeq$4.5) to values up to 15 for saturated concrete. Capacitance meters are widely used but some problems exist:

— The values measured depend largely on the conductivity (salt content).

— There is a strong influence of the coupling to the surface. The device must be fully in contact with the test surface. Rough surfaces may result in wrong values.

— Absolute values must be taken with care and verified with other methods (like Darr drying).

— The penetration depth of the electric field is not well defined.

— Generally capacitance meters are used for qualitative comparison on the same test object.

(c) Microwave moisture meters

Water has absorption bands in the range around 10 GHz. Microwaves with frequencies in that range are attenuated by the water in addition to the scattering process in the medium. There are a number of microwave gauges available, but most of them are installed in the production process of goods other than concrete.

In general microwave measurements also depend on the conductivity of the medium. Absorption measurements require two sided access or the installation of bore holes.

(d) Moisture measurements with radar

Ground penetrating radar (GPR) uses pulsed microwaves to locate interfaces of dielectric materials. The distance 'D' of a reflector from the antennae is calculated by:

$$D = (½v)/t_r \qquad (6)$$

where

v is velocity of microwave pulse,
t_r is time of arrival of reflection,
D is the distance between the reflector and the surface.
If velocity is known, distance can be calculated. On the other hand, if distance is known velocity can be measured. The velocity of an electromagnetic wave in a dielectric material is given by:

$$v = c/\sqrt{\varepsilon_r} \qquad (7)$$

Using these relations the dielectric constant of concrete can be measured, provided the travel length of the radar pulse is known. The relation between the dialectic constant and the moisture content of a given concrete must be established through calibration measurements.

1.4. COMPARISON OF NDT METHODS

The summary of NDT methods applicable for concrete is given in Table 1.2.

TABLE 1.2. COMPARISON BETWEEN NDT METHODS OF TESTING CONCRETE

Method	Acoustic emission	Concrete resistance meter
Principle	When a material is loaded part of it may be loaded beyond its elastic limit. Kinetic energy is released. This is known as acoustic emission They are inaudible but can be detected by sensors attached to the surface of a test object.	The resistivity of concrete is related to its moisture content and to a lesser degree to chloride content. Resistivity measurement gives an indication of the rate of corrosion, which may occur if oxygen-moisture or chloride-oxygen-moisture is present at the reinforcement. Resistivity is measured by inserting electrodes into small holes on the surface and passing an alternating current through them. The difference in potential is then measured.
Main applications	Continuous monitoring of structure during service life to detect impending failure and monitor performance of structure during proof testing. This method has also been used in recent years to study the initiation and growth of cracks in concrete under stress.	It is used for measuring the ability of the concrete to conduct the corrosion current. The higher the resistance the slower the corrosion process can proceed. The device can also be used to measure moisture contents and to map moisture migration patterns.
User expertise	Extensive knowledge is required to plan the test and to interpret results.	Low operative expertise is sufficient
Advantages	It monitors response of existing structure to applied load. It is capable of detecting onset of failure and locating source of possible failure. Since acoustical signals come from defects throughout the structures a few transducers are enough to detect and locate defects over large areas.	The equipment is inexpensive, simple to operate and many measurements can be rapidly made. It is very useful when used in conjunction with other methods of testing, e.g. half-cell potential.
Limitations	The equipment costs are high. The method requires means of loading the structure and complex electronic equipment. As the method is not yet fully developed it is still regarded as a laboratory method at present.	It is not reliable at high moisture contents. It needs calibration and precise results are not usually obtained. The electrodes require good contact and nearby bars can affect readings.

Text cont. on page 32

Table 1.2. (cont.)

Method	Covermeter	Electrical half-cell potential measurements
Principle	The basic principle is that the presence of steel affects magnetic field. An electromagnetic search probe is swept over the surface of the concrete under test. The presence of reinforcement within the range of the instrument is shown by movement of the indicator needle. When the probe is moved until the deflection of the needle is at a maximum, the bar in question is then parallel to the alignment of the probe and directly beneath it. The needle indicates the cover on the appropriate scale for the diameter of the reinforcing bar.	Essentially, electrical potential of steel reinforcement is measured relative to a reference electrode (half-cell). This enables potential contour maps to be plotted. The electrode potential of steel in concrete indicates the probability of corrosion.
Main applications	It is used for determining the presence, location and depth of rebars in concrete and masonry components. Advanced versions of covermeter can also indicate bar diameter when cover is known. It is moderately easy to operate. However, some training or experience is required to interpret the results.	The half-cell provides a relatively quick method of assessing reinforcement corrosion over a wide area without the need for wholesale removal of the concrete cover. Quantitative measurements are made so that a structure can be monitored over a period of time and any deterioration can be noted.
User expertise	It is portable and rugged equipment and gives reliable results if the concrete is lightly reinforced.	Strict adherence to standard procedures is very important to obtain meaningful results. Some experience is required. The user must be able to recognize problems in interpreting results.
Advantages	The presence of closely spaced reinforcing bar, laps, transverse steel, metal tie, wires or aggregates with magnetic properties can give misleading results. The meter has several scales for different bar sizes, therefore the bar diameter must be known if a true indication of cover is to be obtained.	It is portable equipment. Field measurements can be readily made and results can be plotted in the form of equipotential contour diagram, which can indicate likely areas of corrosion. It appears to give reliable information.
Limitations	The maximum range of the instrument for practical purposes is about 100 mm. It does not give indication of the quality of concrete cover or the degree of protection afforded to the reinforcement.	The main limitation is that it does not provide information on rate of corrosion. It also requires access to reinforcing bars to make electrical contact.

Table 1.2 (cont.)

Method	Fibre scope (endoscope)	Gamma radiography
Principle	It consists of a bundle of flexible optical fibres with lens and illuminating systems. It is inserted into pre-drilled boreholes of an element under investigation to examine its condition.	A radioactive isotope directs a beam at a member and an X ray photographic plate is held against the back face. Gamma radiation attenuates when passing through a building component. The density and thickness of the materials of the building component will determine the degree of attenuation. Photographic film records are usually made, which could be analyzed.
Main applications	It can be used to check condition of materials in cavities, concealed piping, electrical wiring in cavity walls, honeycombing in reinforced masonry construction or detect voids along grouted stressed tendons.	This technique is quite established for examination of steel members. It can be used for locating internal cracks, voids and variations in density of materials, grouting of post-tensioned construction as well as locating the position and condition of reinforcing steel in concrete.
User expertise	Interpretative visual skill is required. Hence experience and training are essential for correct results.	It must be operated by trained and licensed personnel.
Advantages	It affords direct visual inspection of otherwise unaccessible parts of an element.	It can be used for field measurements, simple to operate, relatively inexpensive compared to X ray radiography and is applicable to a variety of materials.
Limitations	It is semi destructive in that probe holes usually must be drilled and must connect to a cavity.	In many cases in engineering structures, the method is unusable because it is difficult to place the photographic films in a suitable position. There are also the problems of health and safety both for the operatives and those in the vicinity as it requires long radiation exposure time. Common site-radiography source such as Ir-192 can only be used for penetration depths of 200 mm in concrete. Areas must be isolated from public.

Table 1.2 (cont.)

Method	Ground penetrating radar	Neutron moisture gauge
Principle	Radio frequency waves (0.5 to 2GHz) from radar transmitter are directed into the material. The waves propagate through the material until a boundary of different electrical characteristic is encountered. Then part of the incident energy is reflected and the remainder travels across the boundary at a new velocity. The reflected (echo) wave is picked up by a receiver. The transducer is drawn over a surface and forms a continuous profile of the material condition below. The equipment consists of a radar console, a graphic scanning recorder and a combined transmitting and receiving transducer.	It works on the principle that hydrogen retards the energy of neutrons. Water in the concrete contains hydrogen molecules, which scatter the neutrons. The backscatter is measured. Hence the greater the back scatter the more moisture there is in the concrete. Instruments are generally calibrated based on semi-infinite volume and uniform moisture content.
Main applications	It is capable of detecting a number of parameters in reinforced concrete structures: ▪ the location of reinforcement ▪ the depth of cover ▪ the location of voids ▪ the location of cracks ▪ *in situ* density ▪ moisture content variations.	It can be used to measure moisture content of concrete, soil and bituminous materials and to map moisture migration patterns in masonry walls. Their application to concrete testing is very recent and still in the exploratory stage.
User expertise	User must have good knowledge of wave propagation behaviour in materials in order to meaningfully collect and interpret results. Training and experience are required.	It must be operated by trained and licensed personnel.
Advantages	It can be used to survey large areas rapidly for locating reinforcement, voids and cracks.	The instrument is portable and moisture measurements can be made rapidly.
Limitations	Results must be correlated to test results on samples obtained. Any features screened by steel reinforcement will not be recorded. With increasing depth, low level signals from small targets are harder to detect due to signal attenuation. It is expensive to use and uneconomical for surveying small areas.	A minimum thickness of surface layer is required for backscatter to be measured. It measures only the moisture content of surface layer (50 mm). It emits radiation. Results are inaccurate because hydrogen atoms of building materials are measured in addition to those of water. Its use in concrete is limited and requires calibration in order to calculate density or moisture content.

Table 1.2 (cont.)

Method	Pullout devices (a semi-destructive test)	Rebound hammer
Principle	The test involves drilling a hole in which a standard threaded or wedge anchor is placed. This is then pulled until the concrete raptures. With the help of calibration charts the maximum force gives an indication of the strength of concrete. Pullout devices can be inserted during casting of concrete.	It consists essentially of a metal plunger, one end of which is held against the concrete surface while the free end is struck by a spring-loaded mass which rebounds to a point on a graduated scale. The point is indicated by an index rider. The amount of rebound increases with increase in concrete strength for a particular concrete mix.
Main applications	It provides an estimation of the compressive and tensile strengths of hardened concrete.	It measures the surface hardness of concrete and provides an estimation of surface compressive strength, uniformity and quality of concrete.
User expertise	User expertise is low and can be used in the field.	User expertise is low and can be readily operated by field personnel.
Advantages	In-place strength of concrete can be measured quickly and appears to give good prediction of concrete strength.	It gives accurate assessment of the strength of the surface layer of material. The entire structure can be tested in its 'as-built' condition.
Limitations	Pullout devices must be preplanned and inserted during the construction stage, or inserted in hole drilled in hardened concrete. A cone of concrete may be pulled out, necessitating minor repairs. It can only test a limited depth of material. As it is a surface method, and in reinforced concrete could only be used to assess the concrete cover quality.	It can be very costly and time consuming as instrumentation is required to measure response. It requires careful planning and can damage structure. The member must be isolated from the rest of the structure prior to the test.

Table 1.2 (cont.)

Method	Penetration probe (Windsor probe)	Ultrasonic pulse velocity
Principle	Basically, the test consists of firing a standard probe into the concrete with a standard cartridge. The extent of the penetration is measured and is related to the concrete strength The strength results are based on predetermined correlation between the type of aggregates used in the concrete and the penetration depth.	Voltage pulses are generated and transformed into wave bursts of mechanical energy by the transmitting transducer (which must be coupled to the specimen surface through a suitable medium). A receiving transducer is coupled to the specimen at a known distance to measure the interval between the transmission and reception of a pulse. There are three practical arrangements for measuring pulse velocity, namely direct, diagonal and surface techniques. The direct approach provides the greatest sensitivity and is therefore superior to the other arrangements.
Main applications	It can be used for estimating compressive strength, uniformity and quality of concrete. It can also be used for estimating the same properties of mortars.	Determination of the variability and quality of concrete by measuring pulse velocity. Using transmission method, the extent of such defects such as voids, honeycombing, cracks and segregation may be determined. This technique is also useful when examining fire damaged concrete.
User expertise	Expertise required is low. Execution is fast and can be operated by field personnel.	Low level is required to make measurements. However, expertise is needed to interpret the results.
Advantages	The equipment is easy to use and does not require surface preparation prior to testing. It is good for determining *in situ* quality of concrete. The results are not subject to surface conditions, moisture content or ambient temperature.	Excellent for determining the quality and uniformity of concrete. It can rapidly survey large areas and thick members. Path lengths of 10m to 15m can be inspected with suitable equipment.
Limitations	It requires minimum edge distance and member thickness. It slightly damages small area. Calibration by manufacturers does not give precise prediction of strength for concrete older than 5 years and where surface is affected by carbonation or cracking. Calibration based on cover is necessary for improved evaluation.	Proper surface preparation is required. The work is very time consuming as it takes only point measurements. Skill is required in the analysis of results as moisture variations and presence of metal reinforcement can affect results. The interpretation of ultrasonic test results based on published graphs and tables can be misleading. It is therefore necessary that correlation with the concrete be inspected is carried out. It works on single homogenous materials.

Table 1.2 (cont.)

Method	Ultrasonic pulse echo	Thermography
Principle	Pulsed compressional waves are induced in materials and those reflected back are detected by a hand-held accelerometer. This is connected to a signal processor that integrates the signal and displays it on an oscilloscope.	An infrared scanning camera is used to detect variations in infrared radiation output of a surface. Thermal gradients arise because of difference in surface temperature between sound and unsound concrete. Hence delaminations in concrete surfaces can be detected. The temperature gradients are displayed on a TV screen in the form of colour thermal contours.
Main applications	This method has been recently developed for quality control and integrity pile testing. It can detect the type and location of defects or inconsistency of the pile.	It can be used for detecting delamination, heat loss and moisture movement through concrete elements especially for flat surfaces.
User expertise	High level of expertise is required to interpret results.	User expertise is not high but interpretation of results requires understanding of thermal behaviour and patterns.
Advantages	It is portable, simple and cheap to use. Photographic records can be made. Internal discontinuities can be located and their sizes estimated.	It is portable and permanent records can be made. Testing can be done without direct access to surface and large areas can be rapidly inspected using infrared cameras.
Limitations	It cannot determine net cross section of piles or its bearing capacity. Interpretation of results can be difficult and calibrating standards are required. It also requires smooth surfaces for the probe.	It is an expensive technique. Reference standards are needed and a heat source to produce thermal gradient in the test specimen may also be required. It is very sensitive to thermal interference from other heat sources. Moisture on the surfaces can also mask temperature differences.

1.5. QUALITY CONTROL

1.5.1. The need for quality and quality control

In any manufacturing, fabrication or production process, the quality of the structure or component produced (or service provided) is a key factor in the long term economic and engineering success of that process. Increasing awareness of the importance of quality in every area of technology has resulted from sensitivity to the growing pressure of international competition, more discriminating demands from the marketplace and stricter consumer protection and product liability legislation. Part of this awareness is that consistent quality requires much more than product testing. The need to identify and correct inadequacies well before the final product is ready for shipping or hand over has become an economic priority in many industries. Quality control is required because of changing buyer-producer relationships and major marketplace demands for quality.

The social and economic demands for effective use of materials and production processes to turn out higher technology based products assure the need for quality assurance. Similarly the changing work practices in factories and offices and the need to compete in international markets require total quality control of all products and services.

Because the human factor is of great importance in the quality control operation, special attention must be paid to the personnel in the organization. They need to be educated to the benefits of quality control, they need to feel involved in the quality control process and they must be able to communicate with other personnel on quality control. This allows them to develop a quality control spirit and improve morale necessary to the success of any quality control programme.

Quality circles have been developed in many factories to oversee the quality of products. These involve staff representatives at all levels that meet for short periods of time, e.g. an hour every week to discuss the quality control of their product and any changes necessary.

Quality control has its roots in the guilds of the Middle Ages where quality was assured by long periods of training. This training instilled in workers pride for the workmanship in their product. Specialization of jobs, as industry grew, meant workers no longer made the entire product. This resulted in a decline in workmanship and alienation of the work force. As products became more complicated it became necessary to inspect them after manufacture. In the 1920s statistics were applied, initially at Bell Laboratories in the USA, in the development of acceptance sampling as a substitute for 100% inspection. General acceptance of the techniques occurred during World War II when the early military standards containing quality-control clauses were developed. Subsequently quality control institutes and standards associations were formed. The institutes promoted the use of quality control techniques for production and service through publications, conferences and training. Standards associations have promoted the development of universal standards, which may be adopted as part of the quality control process.

1.5.2. Basic definitions related to quality assurance

1.5.2.1. Quality

The “Quality” of an industrial product does not mean the best or excellence. It is defined as the fitness of the product to do the job required of it by the user. It may also be said to be the ability of the product to meet the design specifications that are usually set, keeping in view the intended purpose of the product. As stated earlier it would be better to set or define an optimum quality level for a product rather than trying to make it of best possible quality. This approach will unnecessarily make the product more expensive, which may not be acceptable to the customer.

In a generalized way, the typical characteristics of an industrial product which help to define and fix its specifications and quality are chemical composition, metallurgical structure, shape and design, physical properties of strength and toughness, appearance, environmental properties, i.e. response to service conditions, and presence or otherwise of internal defects. These requirements should be met within the specified tolerances. The cost, of course, is an important component. The ability of an organization to meet quality criteria in production of goods or services will ultimately bear on its profitability and survivability. If it cannot produce goods to the customer's requirements, it cannot compete except under very abnormal and short term circumstances. However, if the customer's requirements are impossible to meet, or difficult to meet within the financial constraints imposed, the solution may very well be to redefine the requirement. Insistence on an unnecessarily high performance requirement may be completely impractical. In every industry in every corner of the world, striving for quality has become a popular activity, the success of which depends on the organization and its level of commitment. It should be recognized that quality is not an accident, rather, it should be planned. Quality of a product cannot be inspected after it is made. Instead, the inspection criteria are only to verify that quality criteria are being achieved. The complexity of management of quality within an organization depends on the complexity of the product and the process as well as the performance criterion. Once a customer's requirement is accepted, quality is the producer's responsibility.

1.5.2.2. Quality control

Quality control can be defined as the controls applied at each manufacturing stage to consistently produce a quality product. It can also be expressed as the application of the operational techniques and activities, which sustain the quality of a product or service to satisfy given needs. The concept of total quality control is defined as a system for defining, controlling and integrating all company activities that enable economic production of goods or services that will give full customer satisfaction. The word “control” represents a management tool with four basic steps, namely, setting quality standards, checking conformance with the standards, acting when the standards are not met, and assessing the need for changes in the standards.

In brief the objective of quality control is to provide the customer with the best product at minimum cost. Improvements in product design, consistency in manufacture, reduction in costs and improved employee morale can achieve this objective. The factors affecting product quality can be divided into two major groups. The first factor is technological, which includes machines, materials and processes. The second factor is human, which includes operators, foremen and other personnel. The latter is often more important.

1.5.2.3. Quality assurance

As the name suggests quality assurance is the taking of all those planned and systematic technical and administrative actions necessary to assure the item is being produced to optimum quality level and that it will, with adequate confidence, perform satisfactorily in service.

Quality assurance is aimed at doing things right the first time and involves a continuing evaluation of the adequacy and effectiveness of the overall quality control programme with a view to initiate corrective measures where necessary. For a specific product or service this involves verification audits and evaluation of quality factors that affect the production or use of the product or service.

Quality assurance is quality control of the quality control system.

1.5.2.4. Examination and testing

Examination and testing are those quality control functions carried out during the fabrication of an industrial product by persons employed by the manufacturer. Testing may also be defined as the physical performance of operations (tests) to determine quantitative measures of certain properties. Most non-destructive testing is performed under this heading.

1.5.2.5. Inspection

Inspections are the quality control functions carried out during fabrication of an industrial product by an authorized inspector. They include measuring, examining, testing, gauging or otherwise comparing the findings with applicable requirements. An authorized inspector is one who is properly qualified and has the authority to verify to his satisfaction that all examinations specified in the construction code of an industrial product have been made in accordance with requirements of the construction code. The manufacturer of the product does not employ him.

1.5.2.6. Procedure

In non-destructive testing, a procedure is an orderly sequence of rules or instructions, which describe in detail where, how and in which sequence an NDT method should be applied to a production.

1.5.2.7. Technique

A technique is a specific way of utilizing a particular non-destructive testing method. Each technique is identified by at least one particular important variable from another technique within the method (Example: RT method-X ray/gamma ray Techniques)

1.5.2.8. Report

A report of a non-destructive examination or testing is a document that includes all the necessary information required in order to:

(1) take decisions on the acceptance of the defects by the examination
(2) facilitate repairs of unacceptable defects
(3) permit the examination or testing to be repeated.

1.5.2.9. Records

Records are documents that will give, at any time in the future, the following information about a non-destructive testing examination: a) the procedure used to carry out the examination, b) the data recording and data analyzing techniques used, and c) the results of the examination.

1.5.3. Responsibility for quality

The functional departments responsible for quality are listed in Fig. 1.6. Quality is not the responsibility of any one person or department. It is everyone's job. It includes the assembly line worker, typist, purchasing officer and managing director.

The responsibility for quality begins when the marketing department determines the customer quality requirements and continues through to the satisfied customer.

As can be seen from Fig. 1.6 the responsibility for quality is delegated to all departments. Each has the authority to make quality decisions. The figure also shows the ideal place for an effective quality control department. In this location it is independent, reporting directly to upper level management.

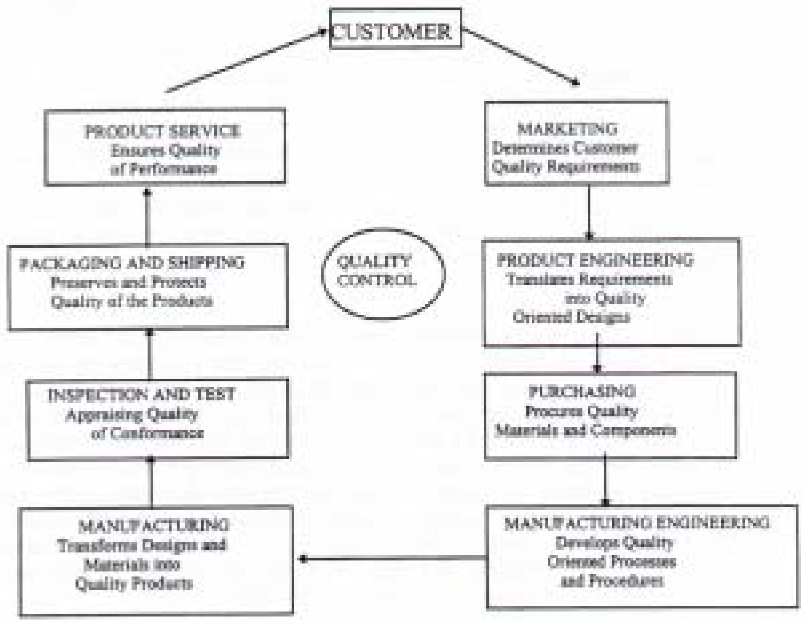

FIG. 1.6. Departments responsible for quality.

1.5.3.1. Inspection and test department

The inspection and test department has the responsibility to appraise the quality of purchased and manufactured items and to report the results. These results can be given to the appropriate departments so that corrective action can be taken when necessary. In order to perform inspection and testing, accurate equipment is necessary which must be maintained and regularly calibrated.

It is necessary to continually monitor the performance of inspectors. Some defects are more difficult to find and require more patience. Inspectors vary in ability and the defect level affects the number of defects reported. Samples with known defects should be used to evaluate and improve the inspectors' performance. The reliability of inspection can usually be quantified and is most often affected by the operator and not the possible defects in the component presented for inspection.

Education (training) is the most effective way of improving reliability.

1.5.3.2. Quality control department

The quality control department does not have direct responsibility for quality. It assists or supports the other departments as they carry out their responsibilities. The relationship between other departments and the quality control department is similar to a line-staff organizational relationship.

Quality control department appraises current quality, determines quality problem areas and assists in correction or minimization of these problem areas. The overall objective is improvement of the product quality in co-operation with responsible departments.

1.5.4. Quality control applications in concrete construction

The quality control of a concrete construction can involve many requirements, such as determining the:

- mechanical properties of the reinforcement to be used
- dimensions of the reinforcement
- location of the reinforcement in the construction before concrete is poured
- location of pre-stressing ducts
- properties of the cement used in the concrete
- properties of the concrete mix designed for use in the structure
- control of the aggregates and sand going into the concrete
- control of water additions
- mixing of the concrete
- transport of the concrete to the construction site
- slump of the concrete
- pouring of the concrete
- vibration/compaction of the concrete
- preparation of areas where different concrete pours are done
- control of compression test samples
- control of formwork removal.

In fact one could even begin with the preliminary work before construction begins that may involve geotechnical investigations necessary to assess foundation conditions, etc. The quality control department, at all times, needs to compare the results of inspections and/or tests with the requirements defined in the contract documents and with the relevant construction standards called up in the contract.

NDT methods are not generally used for this construction quality control. However, some methods are used to control formwork removal by allowing an estimation to be made of the compressive strength of the concrete based on previously established correlation curves.

The NDT methods discussed in this Guidebook are primarily used to measure some characteristics of a concrete construction when it is in the in-service situation. These NDT methods have often become necessary as a means of resolving legal and contractual requirements affecting a structure, or of assessing the integrity of a structure.

Factors that contribute to the reliable application of several non-destructive inspection methods are considered later.

1.5.4.1. Quality of inspection

As with all production processes, many quality considerations must be applied to the control of non-destructive inspection processes to ensure the information being supplied is accurate, timely and relevant. One of the greatest problems of non-destructive inspection is misapplication, which usually means the wrong information is supplied. Thus non-destructive inspection sometimes has only limited usefulness as a production or technical tool. Also, only when the capability of a non-destructive process is known in quantitative terms can the inspection results be considered a measure of true product quality. Unlike the application of NDT methods to materials such as metals, the application to concrete quality control is in its infancy.

Successful application of non-destructive methods to inspection of concrete requires that the:

- test system and procedure be suited to both inspection objectives and types of flaws to be detected
- operators have sufficient training and experience
- standard for acceptance appropriately defines the undesirable characteristics of a non-conforming material or structure.

If any of these pre-requisites is not met, there is a potential for error in meeting quality objectives. For instance, with inappropriate equipment or with a poorly trained operator, gross errors are possible in detecting and characterizing flaws.

This is of particular concern if it means failures to detect flaws that could seriously impair service performance. With inadequate calibration standards, flaws having little or no bearing on the structure's performance may be deemed serious, or significant flaws may be deemed unimportant.

It is essential that the NDT method chosen for a particular evaluation of a concrete construction is capable of locating the type of flaw or anomaly, which has to be found. This requires an understanding of concrete construction methods. For instance cracking in a concrete bridge beam may have little structural significance compared to cracking in a steel bridge girder. On the other hand, the presence of rust staining on a steel bridge girder is of aesthetic concern only unless there is a gross reduction in the steel section due to corrosion. Whereas the presence of rust staining and associated cracking on a concrete bridge girder could mean that the reinforcing steel was being badly corroded due to water entry and the girder may be losing its ability to carry the design load.

1.5.4.2. Human factors

Education of all levels of personnel engaged in non-destructive inspection, including formal training, and certification in accordance with government, technical society or industry standards, is probably the greatest single factor affecting the quality of non-destructive inspection. All methods of non-destructive inspection are highly dependent on operators for obtaining and interpreting data.

Inadequate education of personnel jeopardizes the reliability of inspection. Ideally, inspection should be performed by personnel who are trained to the national equivalent of ISO

9712 Level 2 in the particular method being used and supervisory personnel should have skill equivalent to ISO 9712 Level 3. At the moment however no national certifying body has established a certification process specifically to cover the inspection of concrete structures.

1.5.4.3. Acceptance limits

Since the use of NDT to inspect concrete structures is a relatively new field, accept-reject criteria setting, critical to the application of NDT to metallic components is lacking. Acceptance limits are usually selected arbitrarily and not by reference to recognized manufacturing standards. Whenever acceptance levels are specified, it is essential that the ability of the NDT method to work to those acceptance levels be demonstrated. If possible this is best done by constructing a mock up containing whatever anomaly has to be detected. An acceptance limit that is too strict increases cost, but one that is too lax can contribute to failure to meet service requirements.

1.5.4.4. Inspection standards

Inspection standards should be established so that decisions to accept/rework or scrap parts are based on the probable effect that a given defect will have on the service life or product safety. Once such standards are established non-destructive inspection can characterize flaws in terms of a real effect rather than on an arbitrary basis that may impose useless or redundant quality requirements.

Most non-destructive inspection methods rely on a reference standard to define acceptance limits or to estimate defect sizes. However there often is no recognized universal standard that can be used on diverse products or to satisfy varying inspection requirements of individual users. Under normal circumstances, there needs to be agreement in advance between the NDT organization carrying out an NDT inspection and the organization responsible for the concrete construction as to the design of the reference standard and to the procedure for using it.

1.5.4.5. Effect of manufacturing operations

It is difficult to define the best point in a sequence of construction operations where inspection should be performed since the timing of NDT application depends very much on the type of structure being constructed. However, waiting for construction to be completed before applying NDT is often not desirable in terms of quality of inspection or overall economy of construction. In many instances it is easier, more reliable and more economical to perform limited NDT at each of several points in the construction sequence rather than performing all inspections at the end of the sequence.

In general, the principles listed below should be followed when choosing the point of inspection:

- Perform intermediate inspection following each operation or series of operations that have a significant probability of introducing serious flaws.

- Perform intermediate inspection when the part shape affords easiest access to the region to be examined.

— Limit the extent of non-destructive inspection to the detection of flaws in size, type and location that will significantly affect subsequent service performance.

— Use different inspection methods to detect different types of flaws or material characteristic particularly when no single method yields an optimal balance between inspection cost and sensitivity to parameter being measured.

— Perform final non-destructive inspection only to detect those flaws that could have been introduced after the last previous inspection or to serve as a check (audit) of intermediate inspection.

1.5.5. Quality management system

All organizations function by having employees follow defined processes to achieve outcomes perceived as the organization's objectives. The degree to which processes are documented varies greatly from organization to organization as does the degree to which employees of the organization work to meet the organization's primary goals. The management of an organization can be aided by following the requirements of the ISO 9000 series of standards. Currently there are three main standards in these series: ISO 9001, ISO 9002 and ISO 9003 — all issued in 1994. These are to be replaced by ISO 9001: 2000, scheduled for release in the second part of 2000. The application of this International Standard can be used by an organization to demonstrate its capability to meet customer requirements for products and/or services, and for assessment of that capability by internal and external parties. The standard does not cover the technical requirements for an organization it merely provides guidelines on a method of managing the various interlinked functions of an organization. The standard is based on the concept that any activity or operation, which receives inputs and converts them to outputs, can be considered as a process. Almost all product and/or service activities and operations are processes. Fig. 1.7 is a conceptual presentation of the generic quality management system requirements of a 'process model'.

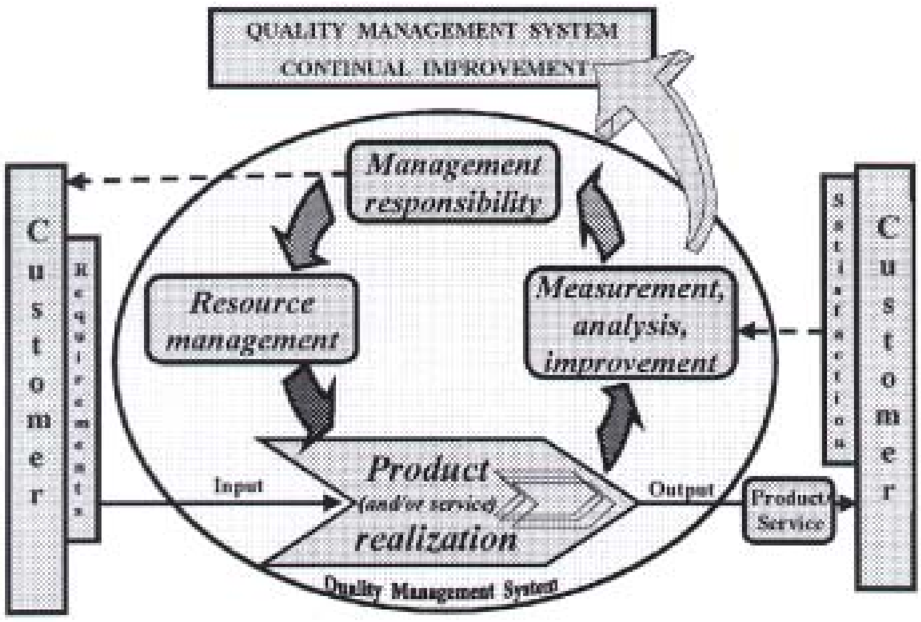

FIG. 1.7. Quality management process model.

1.5.5.1. Quality management system requirements

An organization needs to prepare procedures that describe the processes required to implement the quality management system. The range and extent of the system procedures depends upon such factors as the size and type of the organization, complexity and interaction of the processes, methods used and skills and training of personnel involved in performing the work. The procedures are typically at three levels:

(1) System level procedures that describe the activities required to implement the quality management system
(2) Process level procedures that describe the sequence and interactive nature of the processes necessary to ensure the conformity of the product and/or service
(3) Work instructions or operation level procedures that describe the operating practice and control of process activities.

1.5.5.2. Management responsibility

To implement a successful management system, top management needs to:

— create and maintain an awareness in the organization of the importance of fulfilling customer requirements
— establish quality policy and measurable quality objectives which lead to continual improvement and customer satisfaction
— establish a quality management system
— perform management reviews
— ensure availability of trained personnel and resources
— ensure customer needs and expectations are determined and converted into requirements with the aim of achieving customer confidence
— ensure customer requirements are fully understood and met
— ensure the organization identifies and has access to the legal requirements that are applicable to quality aspects of its products and/or services.

1.5.5.3. Quality manual

A requirement of the quality management system is for the organization to prepare a quality manual containing descriptions of the elements of the quality management system and their interaction, system level and process level procedures, or a reference to them if they are stand-alone documents. Often the quality manual also contains the organization structure and an overview of personnel responsibilities.

1.5.5.4. System level procedures

These procedures need to cover:

— document and data control including approval, review and distribution
— control of records including their identification, storage, retrieval, protection, retention time, and disposition.
— the management review process to ensure the system's continuing suitability, adequacy and effectiveness. The review needs to evaluate the need for changes to the organization's quality management system, including policy and objectives. The periodic

review process needs to consider current performance and improvement opportunities caused by:

- the result of audits
- customer feedback
- process performance and product conformance analyses
- status of preventive and corrective actions
- follow-up actions from earlier management reviews
- changing circumstances.
- human resource management.

1.5.5.5. Process level procedures and instructions

Processes that are necessary to manufacture, fabricate or construct the required product or provide the necessary service need to be determined, planned, documented and implemented so that the processes are carried out under controlled conditions and produce outputs that meet customer requirements. The organization needs to:

- determine and implement arrangements for measurement, monitoring and follow-up actions to ensure processes continue to operate to achieve planned results and outputs;

- ensure the availability of the information and data necessary to support the effective operation and monitoring of the processes;

- maintain as quality records the results of process control measures, to provide evidence of effective operation and monitoring of the processes;

- establish a process for identifying customer requirements and to ensure that no changes are necessary before supplying a product or service;

- capture customer complaints and any organizational actions relating to nonconforming product and/or service customer responses relating to performance of product and/or service;

- plan and control design and/or development of the product and/or service;

- control its purchasing processes to ensure purchased product and/or service conforms to the organization's requirements. This requires the evaluation and selection of suppliers based on their ability to supply product and/or service in accordance with the organization's requirements;

- ensure the availability and use of suitable measuring and monitoring equipment;

- ensure the implementation of suitable monitoring or verification activities suitable methods for release and delivery and/or installation of product and/or service;

- make arrangements for identifying the status of a product and/or service with respect to required measurement and verification activities and, where applicable, needs to identify the product and/or service by suitable means throughout all processes.

- where traceability is a requirement, control and record the unique identification of product and/or service;

- if at any time the organization has under its control any property of a customer, exercise care so that the property is identified, verified, stored and maintained; any customer property that is lost, damaged or otherwise found to be unsuitable for use needs to be recorded and reported to the customer;

- ensure that during internal processing and final delivery of product and/or service to the intended destination that the identification, packaging, storage, preservation, and handling do not affect conformity with product and/or service requirements; it is important that product release and/or service delivery does not proceed until all the specified activities have been satisfactorily completed and the related documentation is available and authorized;

- determine any production and/or service processes where the resulting output is not able to be readily or economically verified by subsequent monitoring, inspection and/or testing; these processes need to be validated to demonstrate their effectiveness and acceptability;

- control, calibrate and maintain those measuring and monitoring devices used to demonstrate conformance of product and/or service to specified requirements.

1.5.5.6. Measurement, analysis and improvement

An important requirement of the quality management system is that the organization implementing the system needs to define, plan and implement measurement, monitoring, analysis and improvement processes to ensure that the quality management system, processes and products and/or services conform to requirements. The type, location, timing and frequency of measurements and the requirements for records need to be defined and the effectiveness of measures implemented need to be periodically evaluated. This may require the use of appropriate statistical tools. Areas needing to be measured and monitored include:

- customer satisfaction or dissatisfaction
- internal audit of all processes including the identification of opportunities for improvement
- process monitoring to demonstrate the continuing ability of the process to satisfy its intended purpose
- monitoring and control of non-conformity to prevent unintended use or delivery
- review and disposition of non-conformities
- analysis of data for improvement
- use of corrective action to reduce or eliminate the causes of non-conformity in order to prevent recurrence
- establishment of preventive action to establish a process for eliminating the causes of potential non-conformities to prevent occurrence.

2. VISUAL INSPECTION

2.1. INTRODUCTION

Visual testing is probably the most important of all non-destructive tests. It can often provide valuable information to the well trained eye. Visual features may be related to workmanship, structural serviceability, and material deterioration and it is particularly important that the engineer is able to differentiate between the various signs of distress which may be encountered. These include for instance, cracks, pop-outs, spalling, disintegration, colour change, weathering, staining, surface blemishes and lack of uniformity. Extensive information can be gathered from visual inspection to give a preliminary indication of the condition of the structure and allow formulation of a subsequent testing programme. The visual inspection however should not be confined only to the structure being investigated. It should also include neighbouring structures, the surrounding environment and the climatic condition. This is probably the most difficult aspect of the whole structural investigation or any diagnostic works since what appears obvious to one may not be so to another. The importance and benefits of a visual survey should not be underrated. Often the omission of what appears to be insignificant evidence can lead to a wrong conclusion being made. The advantage of a trained eye is best described by Sherlock Holmes when he wrote: *"I see no more than you but I have trained myself to notice what I see."*

2.2. TOOLS AND EQUIPMENT FOR VISUAL INSPECTION

An engineer carrying out a visual survey should be well equipped with tools to facilitate the inspection. These involve a host of common accessories such as measuring tapes or rulers, markers, thermometers, anemometers and others. Binoculars, telescopes, borescopes and endoscopes or the more expensive fibre scopes may be useful where access is difficult. A crack width microscope or a crack width gauge is useful, while a magnifying glass or portable microscope is handy for close up examination. A good camera with the necessary zoom and micro lenses and other accessories, such as polarized filters, facilitates pictorial documentation of defects, and a portable colour chart is helpful in identifying variation in the colour of the concrete. A complete set of relevant drawings showing plan views, elevations and typical structural details allows recording of observations to be made.

2.3. GENERAL PROCEDURE OF VISUAL INSPECTION

Before any visual test can be made, the engineer must peruse all relevant structural drawings, plans and elevations to become familiar with the structure. Available documents must also be examined and these include technical specification, past reports of tests or inspection made, construction records, details of materials used, methods and dates of construction, etc.

The survey should be carried out systematically and cover the defects present, the current and past use of the structure, the condition of adjacent structures and environmental condition. All defects must be identified, the degree classified, similar to those used for fire damaged concrete and, where possible, the causes identified. The distribution and extent of defects need to be clearly recognized. For example whether the defects are random or appear in a specific pattern and whether the defect is confined to certain locations of members or is present all over the structure. Visual comparison of similar members is particularly valuable as a preliminary to testing to determine the extent of the problems in such cases. A study of similar structures or other structures in the local area constructed with similar materials can also be helpful in providing ‘case study’ evidence, particularly if those other structures vary in age from the one under investigation. There is a need to identify associated or accompanying defects, especially which particular defect predominates.

Segregation or excessive bleeding at shutter joints may reflect problems with the concrete mix, as might plastic shrinkage cracking, whereas honeycombing may be an indication of a low standard of construction workmanship. Lack of structural adequacy may show itself by excessive deflection or flexural cracking and this may frequently be the reason for an *in situ* assessment of a structure. Long term creep defections, thermal movements or structural movements may cause distortion of doorframes, cracking of windows, or cracking of a structure or its finishes.

Material deterioration is often indicated by surface cracking and spalling of the concrete and examination of crack patterns may provide a preliminary indication of the cause. Systematic crack mapping is a valuable diagnostic exercise when determining the causes and progression of deterioration. Observation of concrete surface texture and colour variations may be a useful guide to uniformity. Colour change is a widely recognized indicator of the extent of fire damage.

Visual inspection is not confined to the surface but may also include examination of bearings, expansion joints, drainage channels and similar features of a structure. Any misuse of the structure can be identified when compared to the original designed purpose of the structure.

An assessment may also need to be made of the particular environmental conditions to which each part of the structure has been exposed. In particular the wetting and drying frequency and temperature variation that an element is subjected to should be recorded because these factors influence various mechanisms of deterioration in concrete. For example, in marine structures it is important to identify the splash zone. Settlement of surrounding soil or geotechnical failures need to be recorded. Account must also be taken of climatic and other external environmental factors at the location, since factors such as freeze thaw conditions may be of considerable importance when assessing the causes of deterioration.

A careful and detailed record of all observations should be made as the inspection proceeds. Drawings can be marked, coloured or shaded to indicate the local severity of each feature. Defects that commonly need recording include:

- cracking which can vary widely in nature and style depending on the causative mechanism
- surface pitting and spalling
- surface staining
- differential movements or displacements
- variation in algal or vegetative growths
- surface voids
- honeycombing
- bleed marks
- constructional and lift joints
- exudation of efflorescence.

Classification of the degree of damage or condition requires experience and engineering judgement but guides are available. Where a large structure is to be examined it may be appropriate to produce a plan or a series of plans indicating 'climate exposure severity' to overlay the engineer's plans of the structure.

2.4. APPLICATIONS OF VISUAL INSPECTION

For existing structures, presence of some feature requiring further investigation is generally indicated by visual inspection, and it must be considered the single most important component of routine maintenance. It will also provide the basis for judgements relating to access and safety requirements when selecting test methods and test locations.

As mentioned earlier, a visual inspection provides an initial indication of the condition of the concrete to allow the formulation of a subsequent testing programme. It is also through such inspections that proper documentation of defects and features in the concrete structure can be effected. With a trained eye, visual inspection can reveal substantial information regarding the structure such as the construction methods, weathering, chemical attack, mechanical damage, physical deterioration, abuse, construction deficiencies or faults and many others.

Section 16 describes a possible procedure for investigating a structure.

2.5. SKETCHES OF TYPICAL DEFECTS FOUND BY VISUAL INSPECTION

Although experience is the best trainer, the following Figs. 2.1–2.20 are sketches of typical defects found in concrete structures.

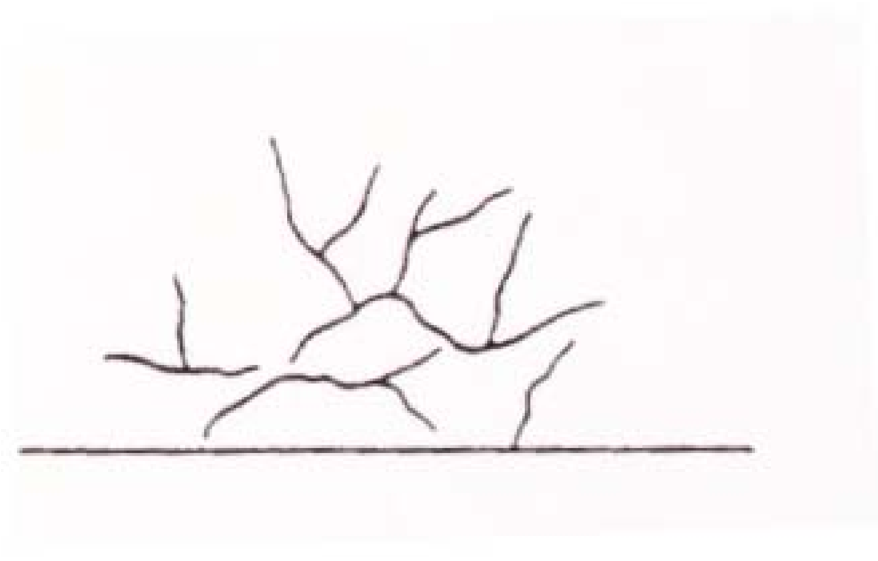

FIG. 2.1. Sketch of surface appearance when concrete has been mixed for too long or the time of transport has been too long.

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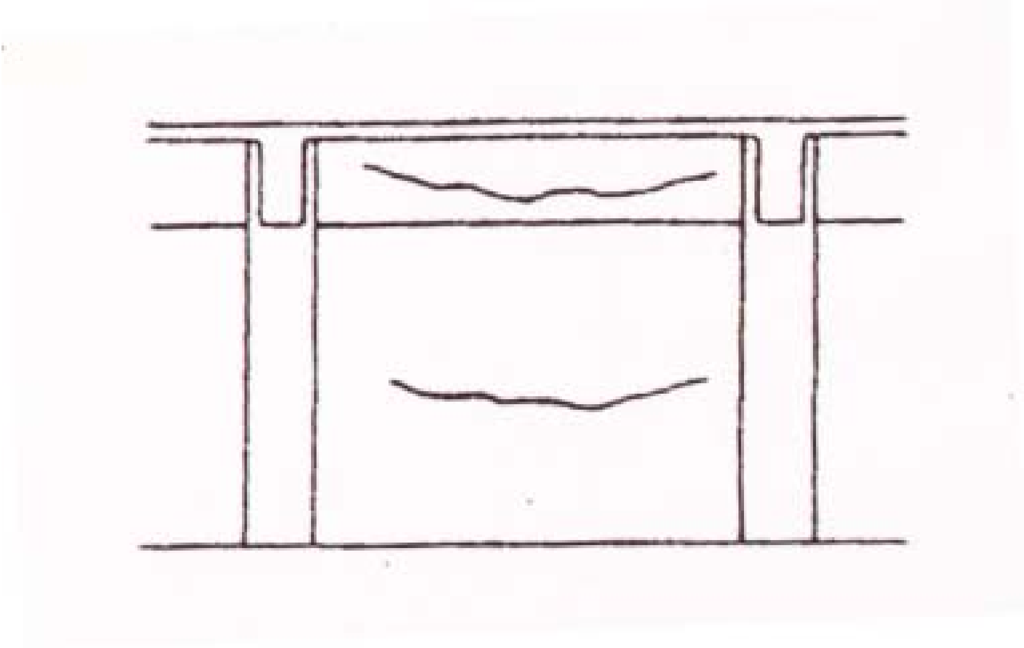

FIG. 2.2. Sketch of crack due to concrete settling.

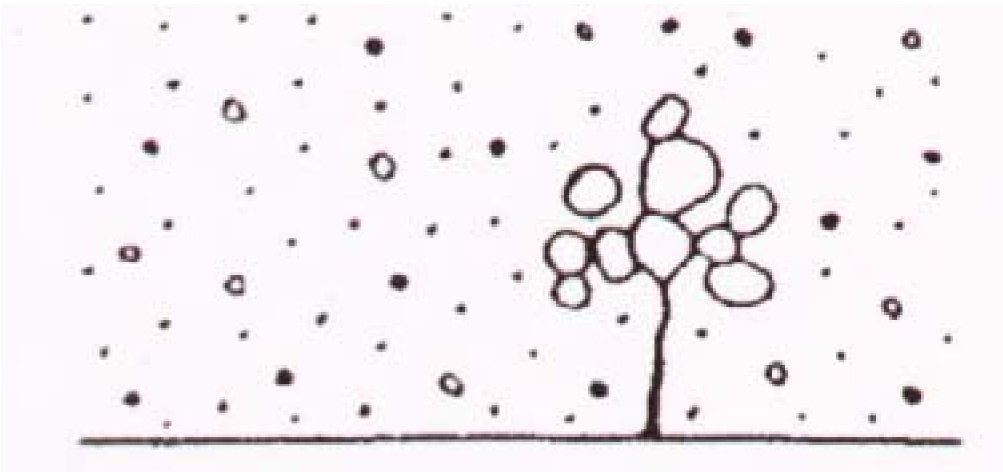

FIG. 2.3. Sketch of exposed aggregate.

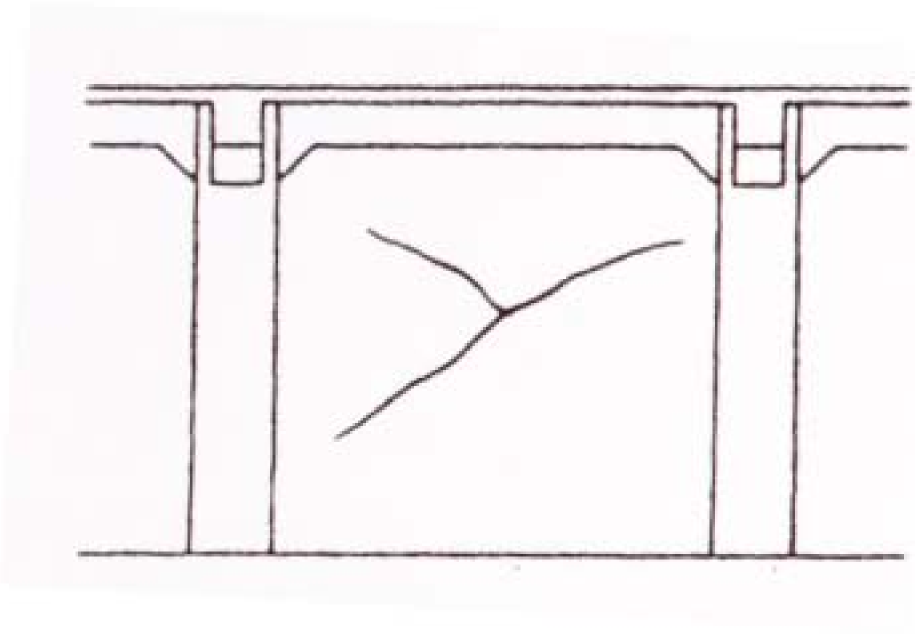

FIG. 2.4. Unsuitable process at construction joint.

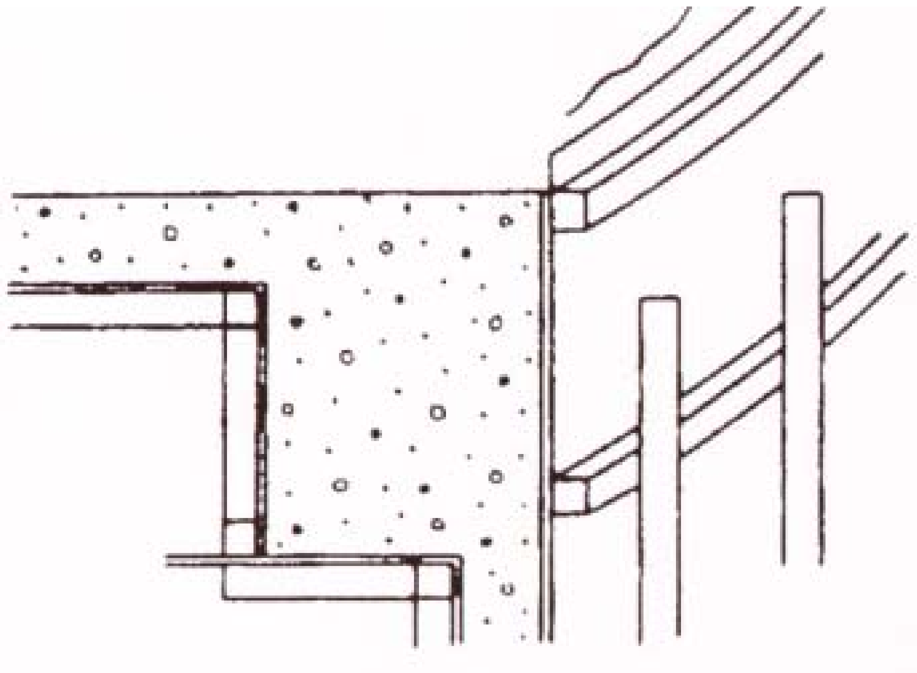

FIG. 2.5. Sketch of cracking due to bowing of formwork.

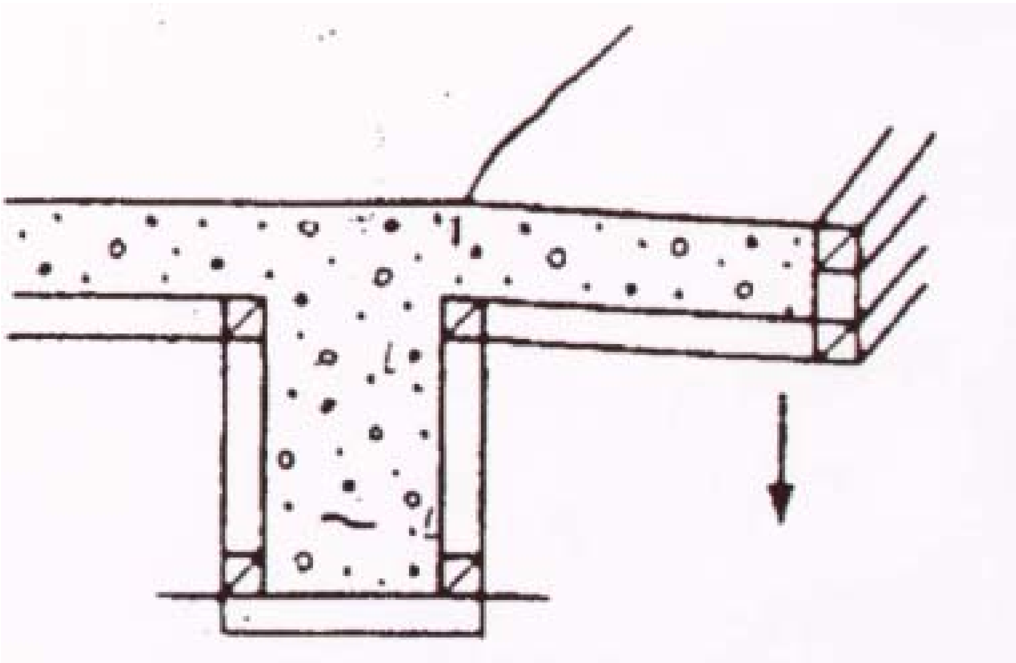

FIG. 2.6. Sketch of cracking due to sinking of timbering.

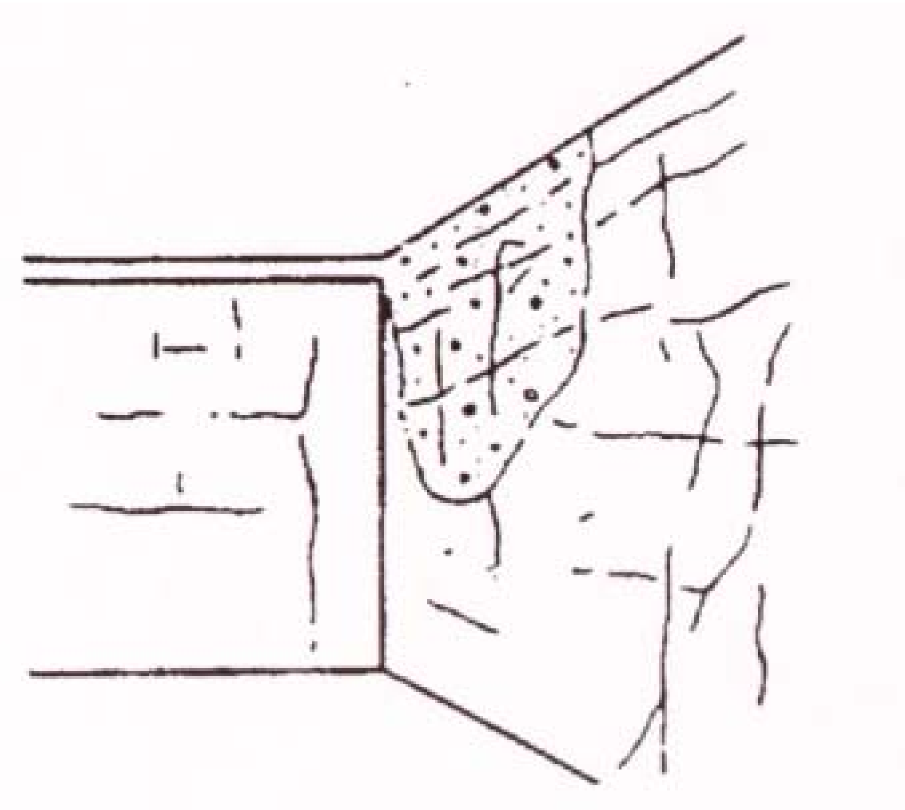

FIG. 2.7. Sketch of severe rusting of reinforcing bars due to chemical action.

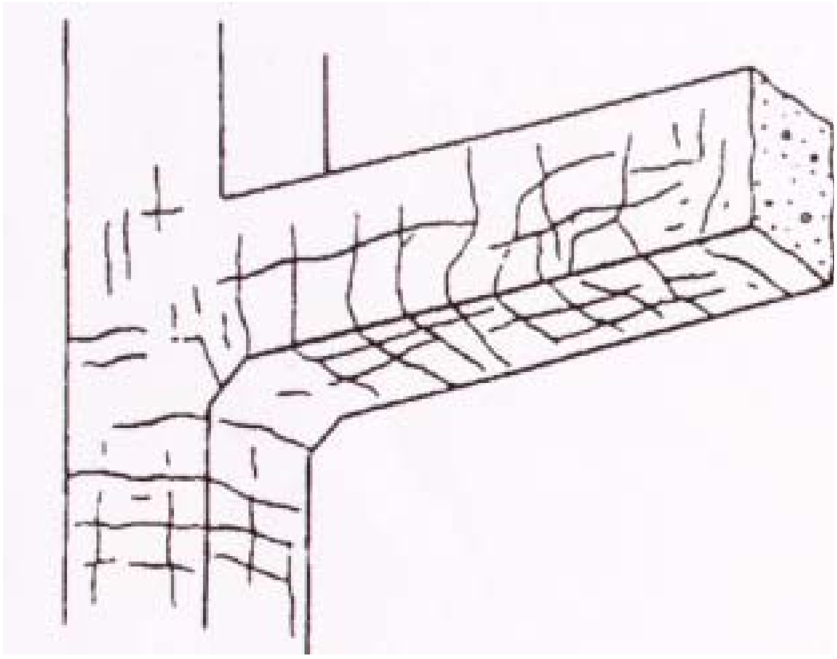

FIG. 2.8. Sketch of effect of fire on concrete.

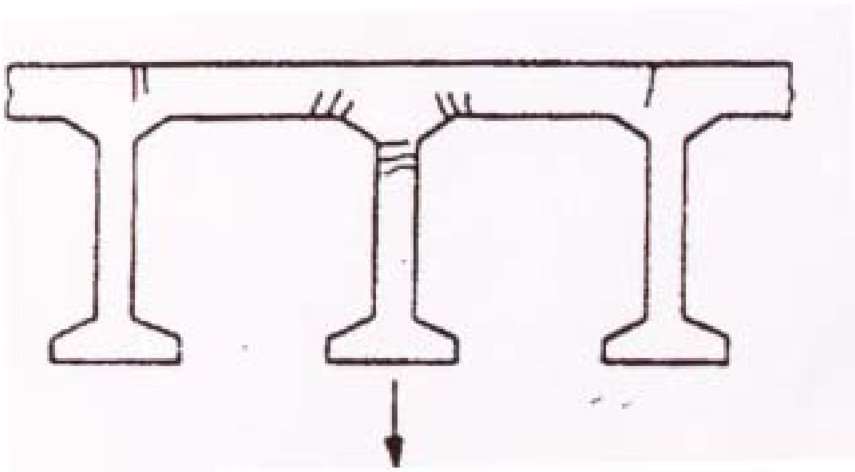

FIG. 2.9. Cracks due to differential settlement of central column.

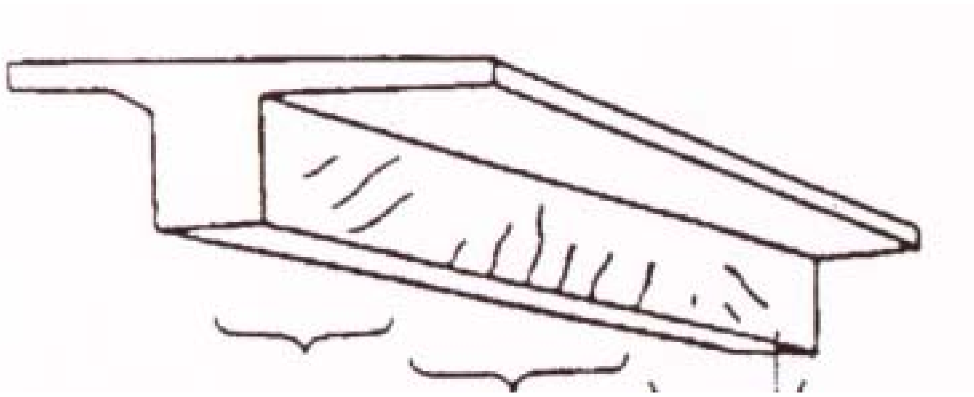

FIG. 2.10. Cracks due to bending and shear stresses.

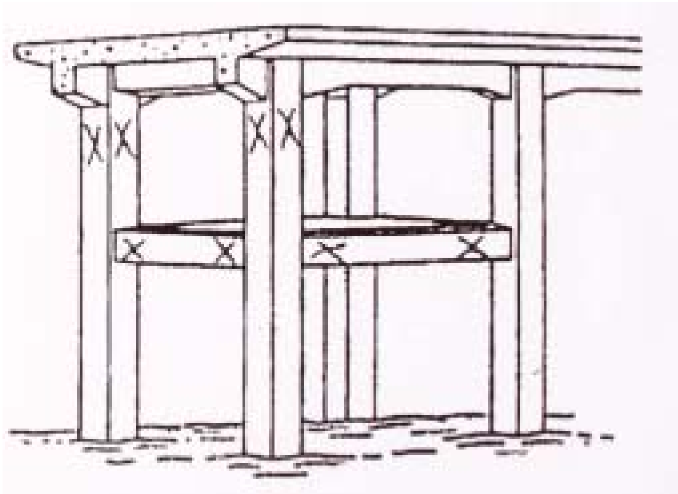

FIG. 2.11. Cracking in columns and beams due to an earthquake.

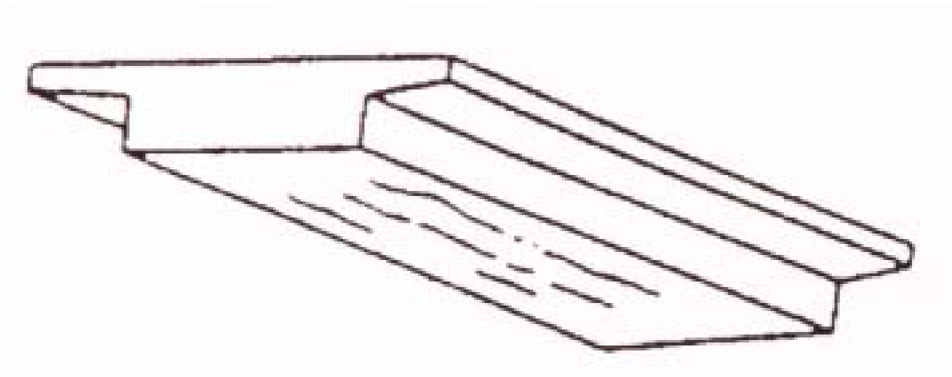

FIG. 2.12. Cracks due to insufficient reinforcing bars.

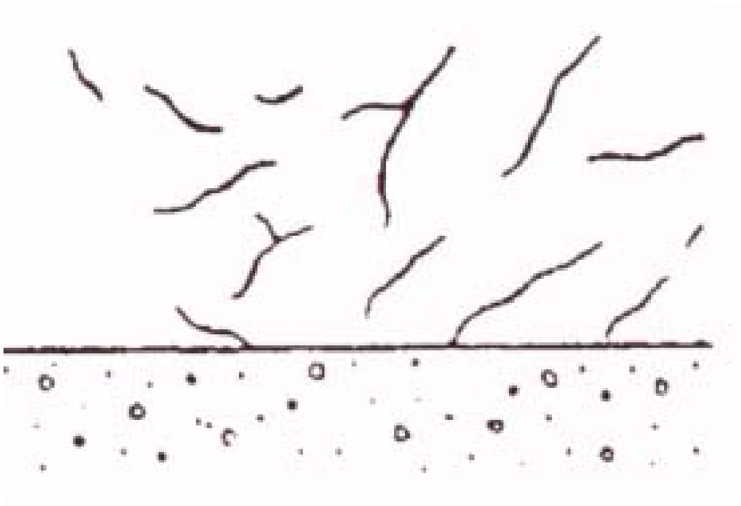

FIG. 2.13. Cracks due to abnormal set of cement.

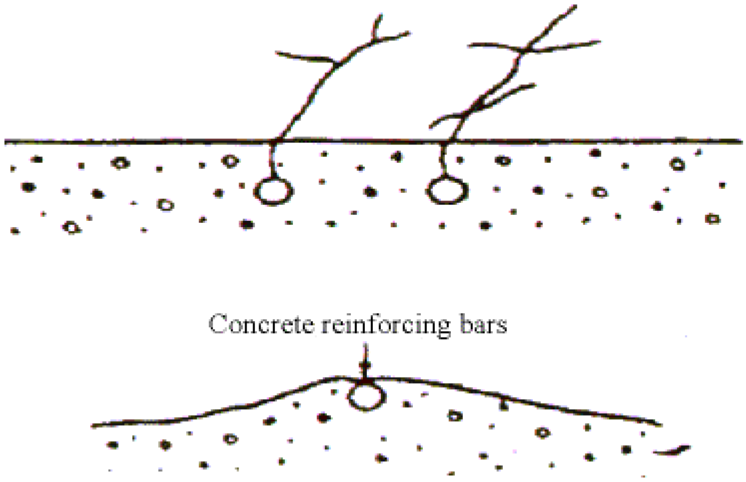

FIG. 2.14. Sinking of concrete.

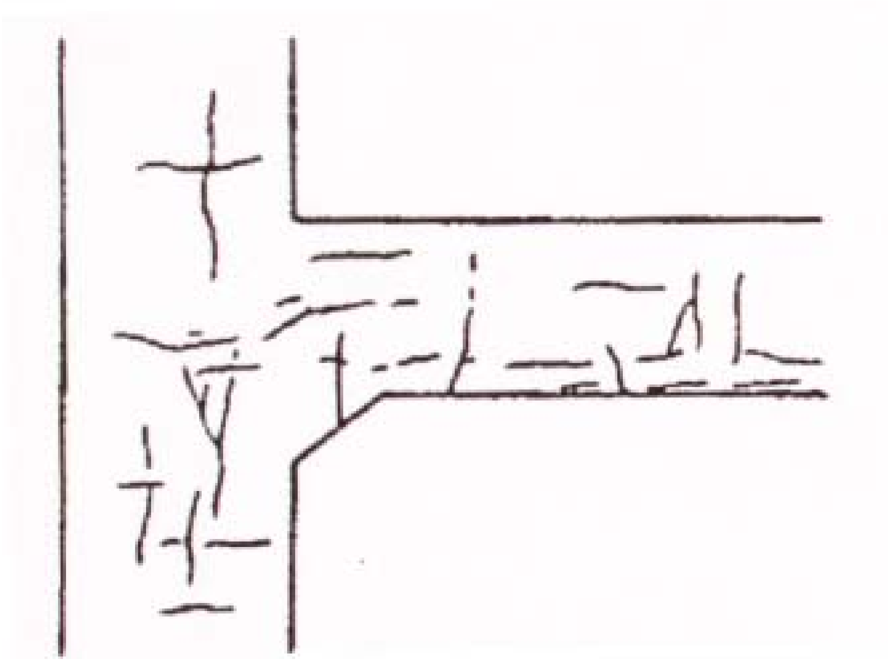

FIG. 2.15. Rusting of reinforcing bars.

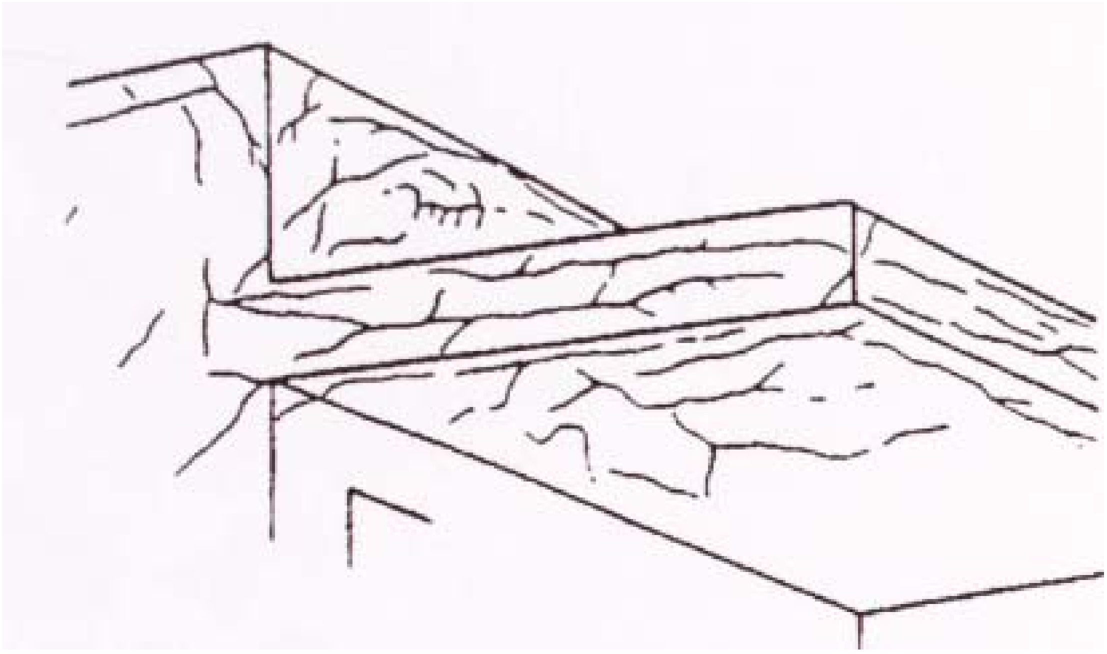

FIG. 2.16. Effect of heating and freezing cycles

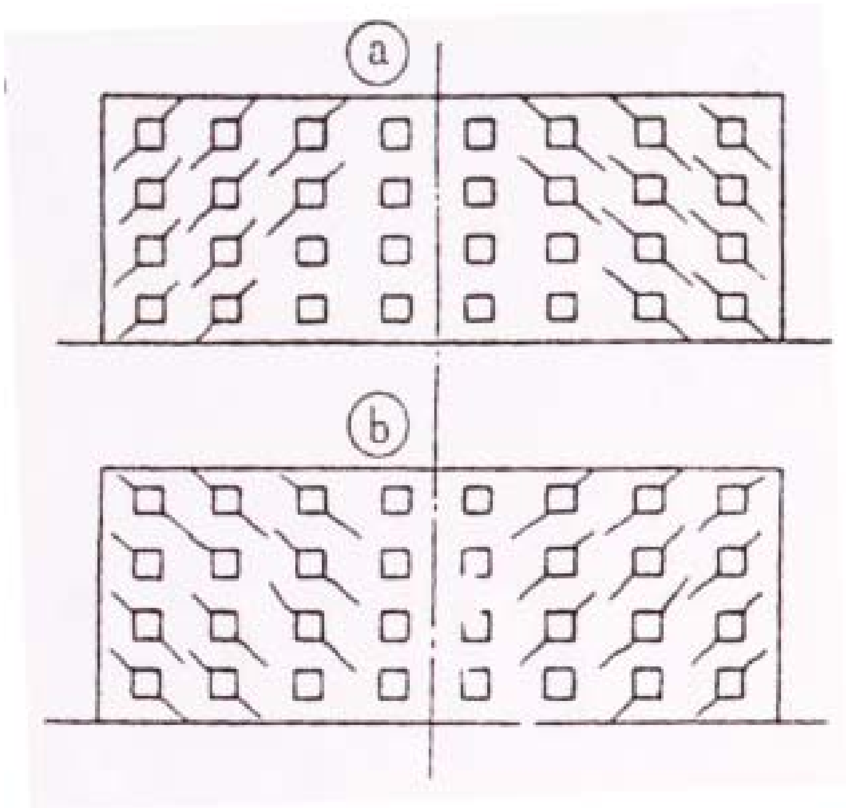

FIG. 2.17. Effect of changing ground conditions:
a) low temperature or b) dryness.

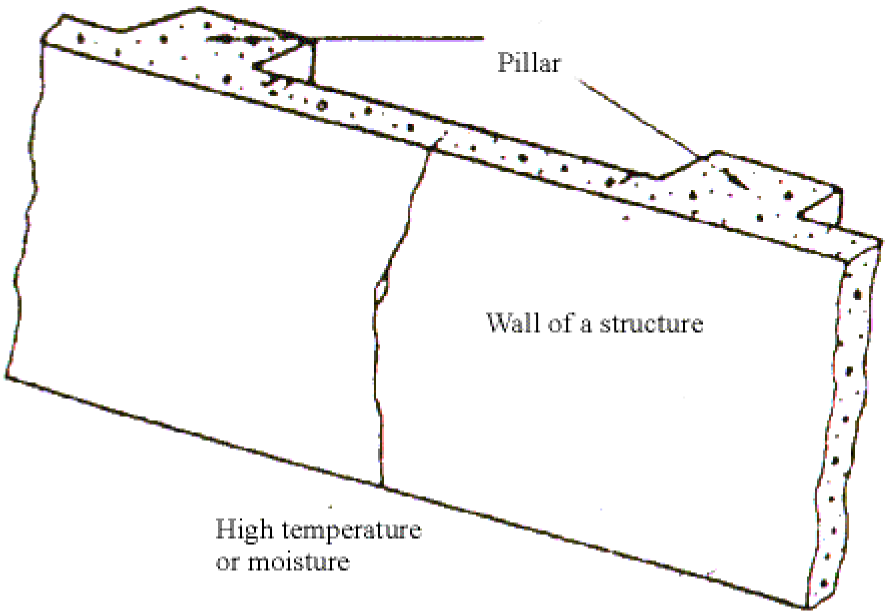

FIG. 2.18. Effect of atmospheric conditions.

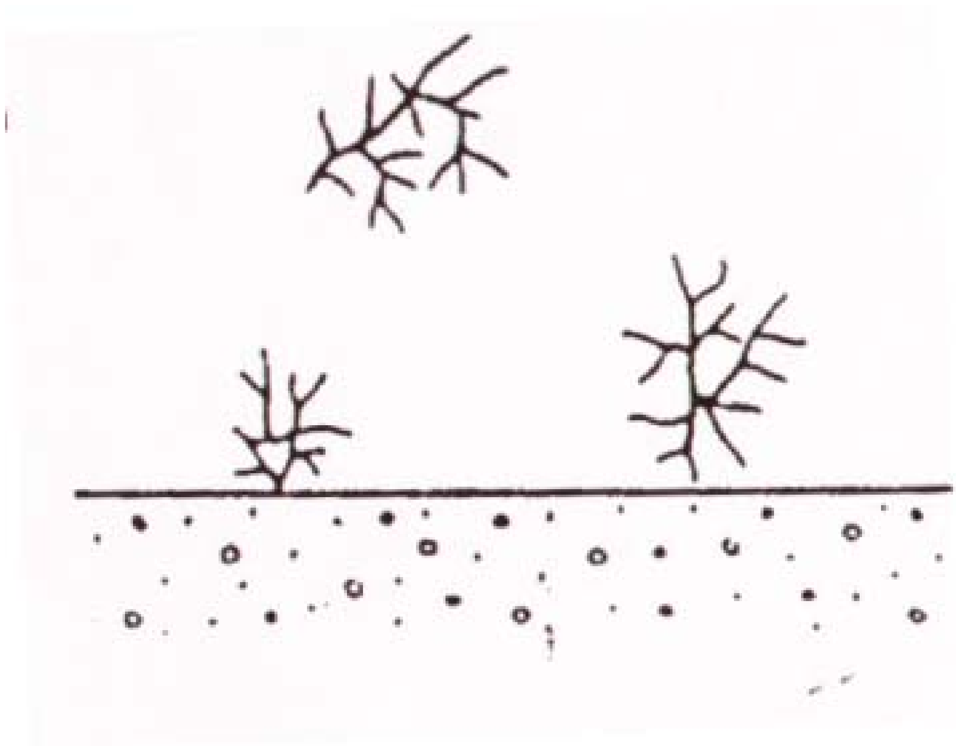

FIG. 2.19. Non-uniformity of admixture.

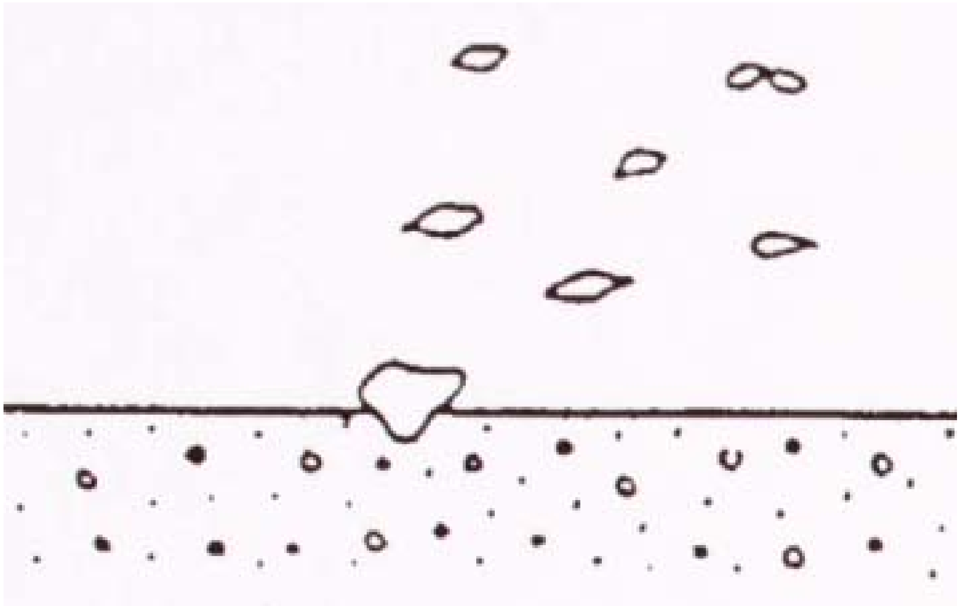

FIG. 2.20. Pop-out due to reactive aggregate and high humidity.

3. HALF-CELL ELECTRICAL POTENTIAL METHOD

3.1. FUNDAMENTAL PRINCIPLE

The method of half-cell potential measurements normally involves measuring the potential of an embedded reinforcing bar relative to a reference half-cell placed on the concrete surface. The half-cell is usually a copper/copper sulphate or silver/silver chloride cell but other combinations are used. The concrete functions as an electrolyte and the risk of corrosion of the reinforcement in the immediate region of the test location may be related empirically to the measured potential difference. In some circumstances, useful measurements can be obtained between two half-cells on the concrete surface. ASTM C876 - 91 gives a Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete.

3.2. EQUIPMENT FOR HALF-CELL ELECTRICAL POTENTIAL METHOD

The testing apparatus consists of the following (Fig. 3.1):

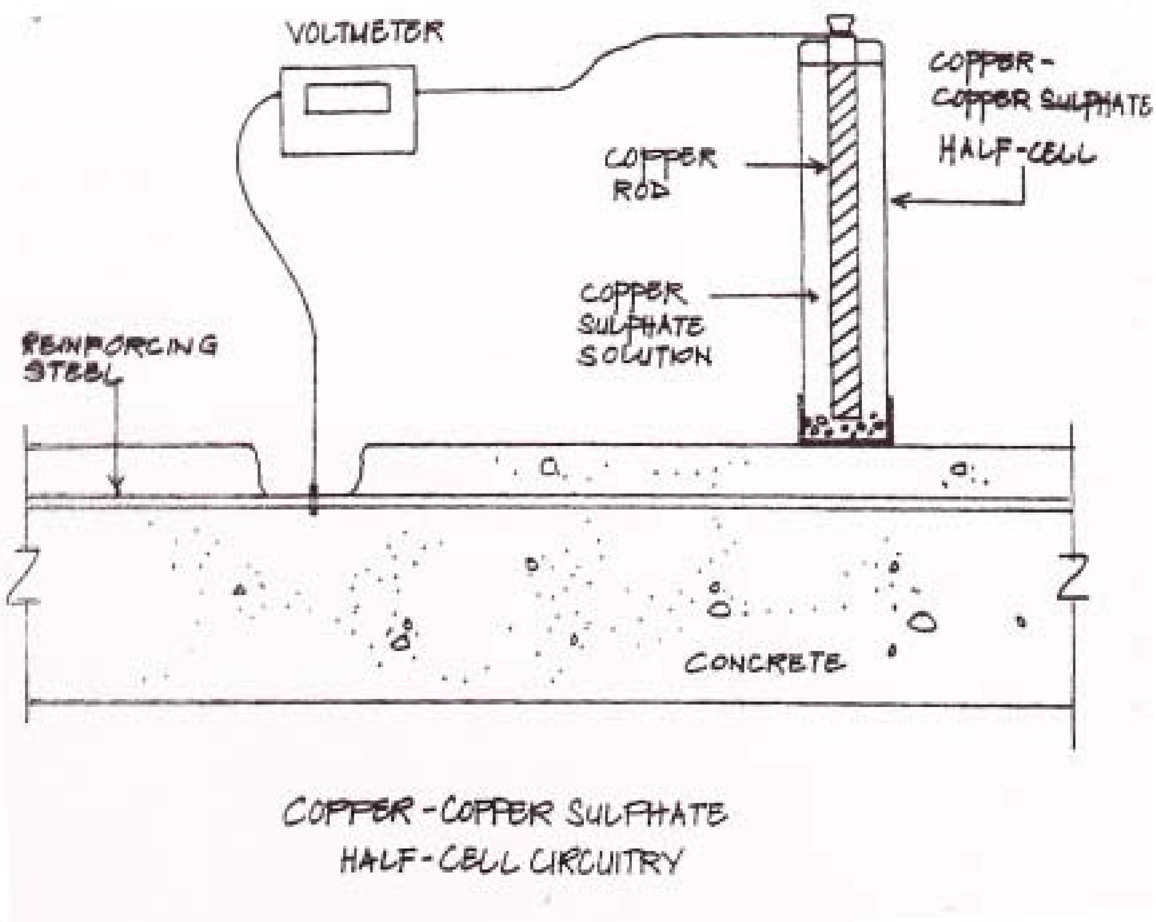

FIG. 3.1. A copper-copper sulphate half-cell.

Half-cell: The cell consists of a rigid tube or container composed of dielectric material that is non-reactive with copper or copper sulphate, a porous wooden or plastic plug that remains wet by capillary action, and a copper rod that is immersed within the tube in a saturated solution of copper sulphate. The solution is prepared using reagent grade copper sulphate dissolved to saturation in a distilled or deionized water.

The rigid tube should have an inside diameter of not less than 25 mm; the diameter of the porous tube should not be less than 13 mm; the diameter of the immersed copper rod should not be less than 6 mm and its length should be at least 50 mm.

Present criteria based on the half-cell reaction of $Cu \rightarrow Cu^{++} + 2e$ indicate that the potential of the saturated copper-copper sulphate half-cell as referenced to the hydrogen electrode is -0.316 V at 72°F (22.2°C). The cell has a temperature coefficient of about 0.0005V more negative per °F for the temperature range from 32 to 120°F (0 to 49°C).

Electrical junction device: An electrical junction device is used to provide a low electrical resistance liquid bridge between the surface of the concrete and the half-cell. It consists of a sponge or several sponges pre-wetted with a low electrical resistance contact solution. The sponge can be folded around and attached to the tip of the half-cell so that it provides electrical continuity between the porous plug and the concrete member.

Electrical contact solution: In order to standardize the potential drop through the concrete portion of the circuit, an electrical contact solution is used to wet the electrical junction device. One solution, which is used, is a mixture of 95 mL of wetting agent or a liquid household detergent thoroughly mixed with 19 L of potable water. At temperatures less than 10°C approximately 15% by volume of either isopropyl or denatured alcohol must be added to prevent clouding of the electrical contact solution, since clouding may inhibit penetration of water into the concrete to be tested.

Voltmeter: The voltmeter should be battery operated and have ± 3% end of scale accuracy at the voltage ranges in use. The input impedance should be not less than 10 MW when operated at a full scale of 100 mV. The divisions on the scale used should be such that a potential of 0.02 V or less can be read without interpolation.

Electrical lead wires: The electrical lead wire should be such that its electrical resistance for the length used does not disturb the electrical circuit by more than 0.0001 V. This has been accomplished by using no more than a total of 150 m of at least AWG No. 24 wire. The wire should be suitably coated with direct burial type of insulation.

3.3. GENERAL PROCEDURE FOR HALF-CELL ELECTRICAL POTENTIAL METHOD

Measurements are made in either a grid or random pattern. The spacing between measurements is generally chosen such that adjacent readings are less than 150 mV with the minimum spacing so that there is at least 100 mV between readings. An area with greater than 150 mV indicates an area of high corrosion activity. A direct electrical connection is made to the reinforcing steel with a compression clamp or by brazing or welding a protruding rod. To get a low electrical resistance connection, the rod should be scraped or brushed before connecting it to the reinforcing bar. It may be necessary to drill into the concrete to expose a reinforcing bar. The bar is connected to the positive terminal of the voltmeter. One end of the lead wire is connected to the half-cell and the other end to the negative terminal of the voltmeter. Under some circumstances the concrete surface has to be pre-wetted with a wetting agent. This is necessary if the half-cell reading fluctuates with time when it is placed in contact with the concrete. If fluctuation occurs either the whole concrete surface is made wet with the wetting agent or only the spots where the half-cell is to be placed. The electrical half-cell potentials are recorded to the nearest 0.01 V correcting for temperature if the temperature is outside the range 22.2 ± 5.5°C.

Measurements can be presented either with a equipotential contour map which provides a graphical delineation of areas in the member where corrosion activity may be occurring or with a cumulative frequency diagram which provides an indication of the magnitude of affected area of the concrete member.

Equipotential contour map: On a suitably scaled plan view of the member the locations of the half-cell potential values are plotted and contours of equal potential drawn through the points of equal or interpolated equal values. The maximum contour interval should be 0.10 V. An example is shown in Fig. 3.2.

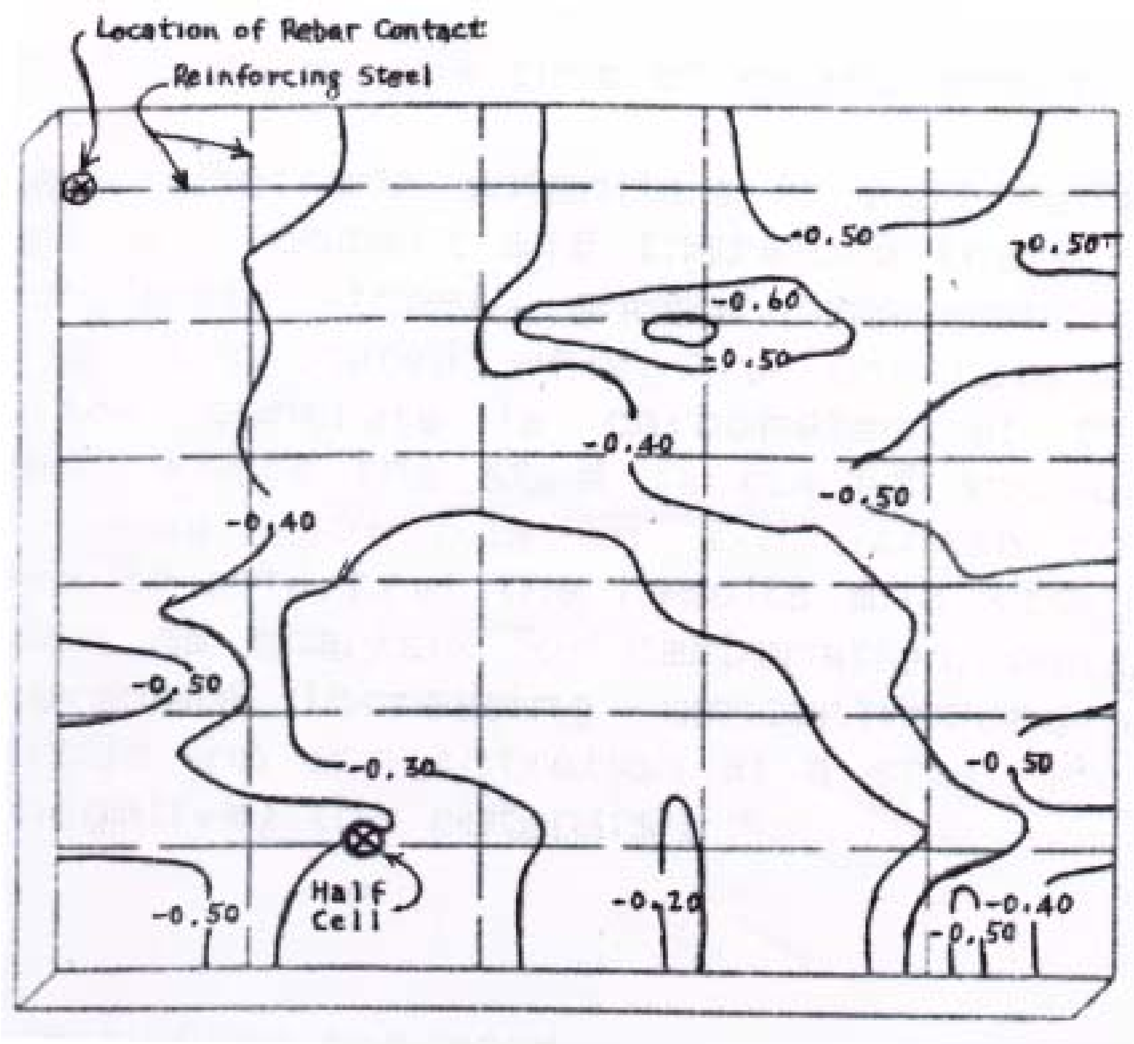

FIG. 3.2. Equipotential contour map.

Cumulative frequency distribution: The distribution of the measured half-cell potentials for the concrete member are plotted on normal probability paper by arranging and consecutively numbering all the half-cell potentials in a ranking from least negative potential to greatest negative potential. The plotting position of each numbered half-cell potential is determined by using the following equation.

$$f_x = \frac{\mathrm{r}}{\sum n+1} x100 \tag{8}$$

where

f_x plotting position of total observations for the observed value, %

r rank of individual half-cell potential,

$\sum n$ total number of observations.

The ordinate of the probability paper should be labeled “Half-cell potential (millivolts, CSE)” where CSE is the designation for copper-copper sulphate electrode. The abscissa is labeled “Cumulative frequency (%)”. Two horizontal parallel lines are then drawn intersecting the –200mv and –350mv values on the ordinate across the chart, respectively. After the half-cell potentials are plotted, a line is drawn through the values. The potential risks of corrosion based on potential difference readings are shown in Table 3.1.

TABLE 3.1. RISK OF CORROSION AGAINST THE POTENTIAL DIFFERENCE READINGS

Potential difference levels (mv)	Chance of re-bar being corroded
less than –500	visible evidence of corrosion
-350 to -500	95%
-200 to -350	50%
More than -200	5%

However, half-cell electrode potentials in part reflect the chemistry of the electrode environment and therefore there are factors which can complicate these simple assumptions. For example, interpretation is complicated when concrete is saturated with water, where the concrete is carbonated at the depth of the reinforcing steel, where the steel is coated and under many other conditions. In those situations an experienced corrosion engineer may be required to interpret the results and additional testing may be required such as analysis for carbonation, metallic coatings and halides. For example, increasing concentrations of chloride can reduce the ferrous ion concentration at a steel anode thus lowering (making more negative) the potential.

3.4. APPLICATIONS OF HALF-CELL ELECTRICAL POTENTIAL TESTING METHOD

This technique is most likely to be used for assessment of the durability of reinforced concrete members where reinforcement corrosion is suspected. Reported uses include the location of areas of high reinforcement corrosion risk in marine structures, bridge decks and abutments. Used in conjunction with other tests, it has been found helpful when investigating concrete contaminated by salts.

3.5. RANGE AND LIMITATIONS OF HALF-CELL ELECTRICAL POTENTIAL INSPECTION METHOD

The method has the advantage of being simple with equipment also simple. This allows an almost non-destructive survey to be made to produce isopotential contour maps of the surface of the concrete member. Zones of varying degrees of corrosion risk may be identified from these maps.

The limitation of the method is that the method cannot indicate the actual corrosion rate. It may require to drill a small hole to enable electrical contact with the reinforcement in the member under examination, and surface preparation may also be required. It is important to recognize that the use and interpretation of the results obtained from the test require an experienced operator who will be aware of other limitations such as the effect of protective or decorative coatings applied to the concrete.

4. SCHMIDT REBOUND HAMMER TEST

4.1. FUNDAMENTAL PRINCIPLE

The Schmidt rebound hammer is principally a surface hardness tester. It works on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges. There is little apparent theoretical relationship between the strength of concrete and the rebound number of the hammer. However, within limits, empirical correlations have been established between strength properties and the rebound number. Further, Kolek has attempted to establish a correlation between the hammer rebound number and the hardness as measured by the Brinell method.

4.2. EQUIPMENT FOR SCHMIDT/REBOUND HAMMER TEST

The Schmidt rebound hammer is shown in Fig. 4.1. The hammer weighs about 1.8 kg and is suitable for use both in a laboratory and in the field. A schematic cutaway view of the rebound hammer is shown in Fig. 4.2. The main components include the outer body, the plunger, the hammer mass, and the main spring. Other features include a latching mechanism that locks the hammer mass to the plunger rod and a sliding rider to measure the rebound of the hammer mass. The rebound distance is measured on an arbitrary scale marked from 10 to 100. The rebound distance is recorded as a “rebound number” corresponding to the position of the rider on the scale.

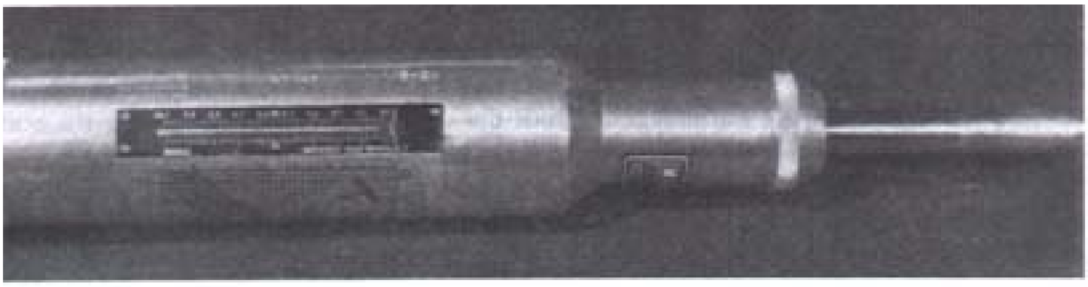

FIG. 4.1. Schmidt rebound hammer.

4.3. GENERAL PROCEDURE FOR SCHMIDT REBOUND HAMMER TEST

The method of using the hammer is explained using Fig. 4.2. With the hammer pushed hard against the concrete, the body is allowed to move away from the concrete until the latch connects the hammer mass to the plunger, Fig. 4.2a.

The plunger is then held perpendicular to the concrete surface and the body pushed towards the concrete, Fig. 4.2b. This movement extends the spring holding the mass to the body. When the maximum extension of the spring is reached, the latch releases and the mass is pulled towards the surface by the spring, Fig. 4.2c.The mass hits the shoulder of the plunger rod and rebounds because the rod is pushed hard against the concrete, Fig. 4.2d. During rebound the slide indicator travels with the hammer mass and stops at the maximum distance the mass reaches after rebounding. A button on the side of the body is pushed to lock the plunger into the retracted position and the rebound number is read from a scale on the body.

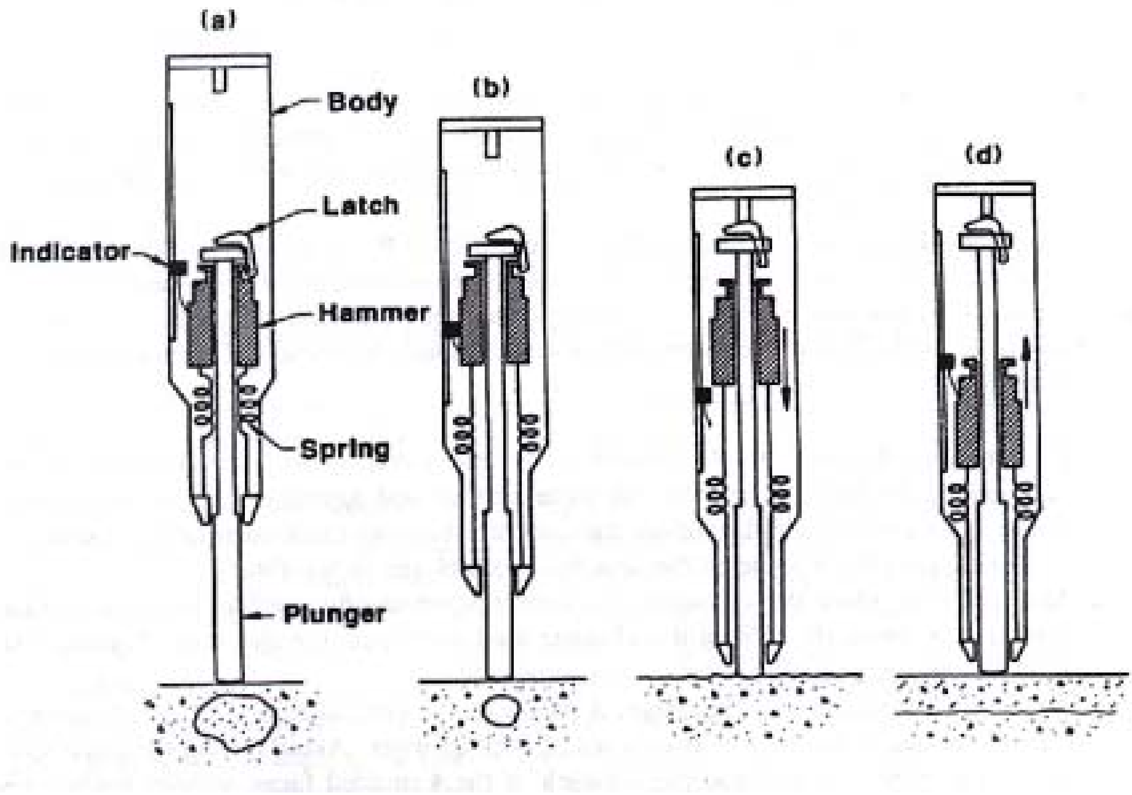

FIG. 4.2. A cutaway schematic view of the Schmidt rebound hammer.

4.4. APPLICATIONS OF SCHMIDT REBOUND HAMMER TEST

The hammer can be used in the horizontal, vertically overhead or vertically downward positions as well as at any intermediate angle, provided the hammer is perpendicular to the surface under test. The position of the mass relative to the vertical, however, affects the rebound number due to the action of gravity on the mass in the hammer. Thus the rebound number of a floor would be expected to be smaller than that of a soffit and inclined and vertical surfaces would yield intermediate results. Although a high rebound number represents concrete with a higher compressive strength than concrete with a low rebound number, the test is only useful if a correlation can be developed between the rebound number and concrete made with the same coarse aggregate as that being tested. Too much reliance should not be placed on the calibration curve supplied with the hammer since the manufacturer develops this curve using standard cube specimens and the mix used could be very different from the one being tested.

A typical correlation procedure is, as follows:

(1) Prepare a number of 150 mm × 300 mm cylinders (or 150 mm^3 cube specimens) covering the strength range to be encountered on the job site. Use the same cement and aggregates as are to be used on the job. Cure the cylinders under standard moist-curing room conditions, keeping the curing period the same as the specified control age in the field.
(2) After capping, place the cylinders in a compression-testing machine under an initial load of approximately 15% of the ultimate load to restrain the specimen. Ensure that cylinders are in a saturated surface-dry condition.

(3) Make 15 hammer rebound readings, 5 on each of 3 vertical lines 120° apart, against the side surface in the middle two thirds of each cylinder. Avoid testing the same spot twice. For cubes, take 5 readings on each of the 4 molded faces without testing the same spot twice.
(4) Average the readings and call this the rebound number for the cylinder under test. Repeat this procedure for all the cylinders.
(5) Test the cylinders to failure in compression and plot the rebound numbers against the compressive strengths on a graph.
(6) Fit a curve or a line by the method of least squares.

A typical curve established by Zoldners for limestone aggregate concrete is shown in Fig. 4.3. This curve was based on tests performed during 28 days using different concrete mixtures.

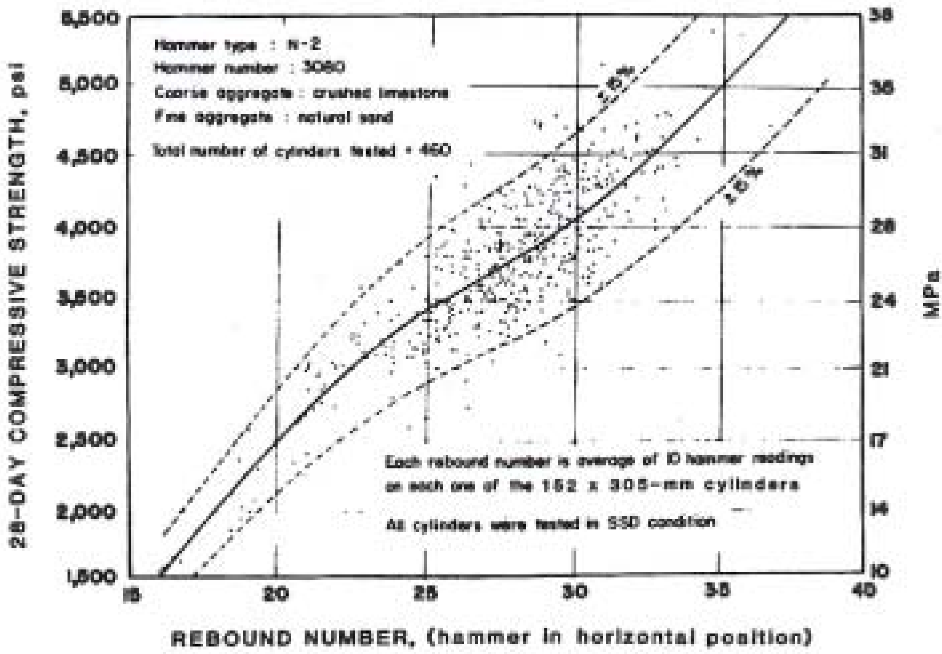

FIG. 4.3. Relationship between 28 day compressive strength and rebound number for limestone aggregate concrete obtained with Type N-2 hammer.

Fig. 4.4 shows further three calibration curves obtained by research workers compared to the curve supplied with the hammer identified as "Schmidt". It is important to note that some of the curves deviate considerably from the curve supplied with the hammer.

4.5. RANGE AND LIMITATIONS OF SCHMIDT REBOUND HAMMER TEST

Although the rebound hammer does provide a quick, inexpensive method of checking the uniformity of concrete, it has some serious limitations. The results are affected by:

1. Smoothness of the test surface

Hammer has to be used against a smooth surface, preferably a formed one. Open textured concrete cannot therefore be tested. If the surface is rough, e.g. a trowelled surface, it should be rubbed smooth with a carborundum stone.

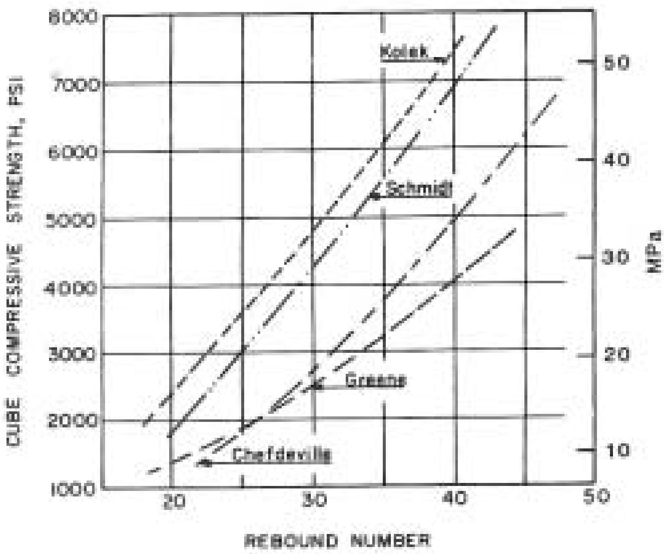

FIG. 4.4. Correlation curves produced by different researchers. (Greene curve used Type N hammer; others used Type N-2).

2. Size, shape and rigidity of the specimen

If the concrete does not form part of a large mass any movement caused by the impact of the hammer will result in a reduction in the rebound number. In such cases the member has to be rigidly held or backed up by a heavy mass.

3. Age of the specimen

For equal strengths, higher rebound numbers are obtained with a 7 day old concrete than with a 28 day old. Therefore, when old concrete is to be tested in a structure a direct correlation is necessary between the rebound numbers and compressive strengths of cores taken from the structure. Rebound testing should not be carried out on low strength concrete at early ages or when the concrete strength is less than 7 MPa since the concrete surface could be damaged by the hammer.

4. Surface and internal moisture conditions of concrete

The rebound numbers are lower for well-cured air dried specimens than for the same specimens tested after being soaked in water and tested in the saturated surface dried conditions. Therefore, whenever the actual moisture condition of the field concrete or specimen is unknown, the surface should be pre-saturated for several hours before testing. A correlation curve for tests performed on saturated surface dried specimens should then be used to estimate the compressive strength.

5. Type of coarse aggregate

Even though the same aggregate type is used in the concrete mix, the correlation curves can be different if the source of the aggregate is different. An example is shown in Fig. 4.5 where correlation curves for four different sources of gravel are plotted.

Fig. 4.6 shows the considerable difference that can occur between correlation curves developed for different aggregate types.

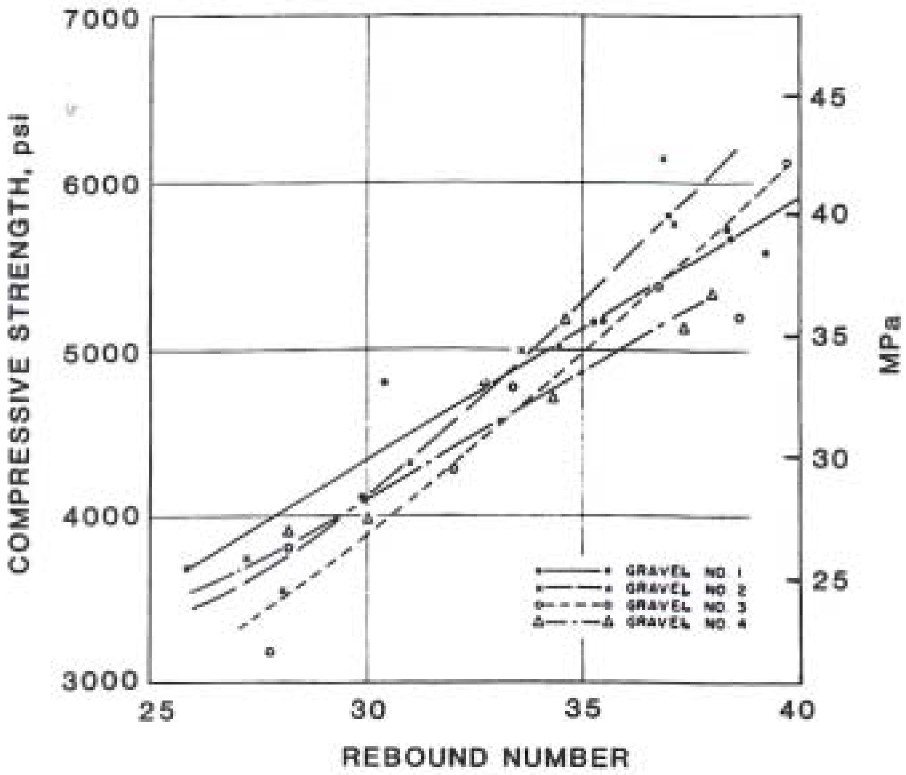

FIG. 4.5. Effect of gravel from different sources on correlation curves.

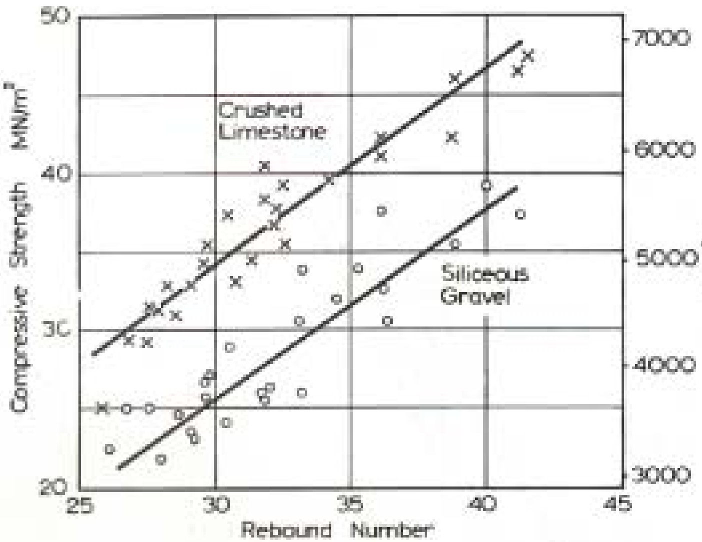

FIG. 4.6. Comparison between correlation curves for crushed limestone and siliceous.

6. Type of cement

High alumina cement can have a compressive strength 100% higher than the strength estimated using a correlation curve based on ordinary Portland cement. Also, super sulphated cement concrete can have strength 50% lower than ordinary Portland cement.

7. Carbonation of the concrete surface

In older concrete the carbonation depth can be several millimeters thick and, in extreme cases, up to 20 mm thick. In such cases the rebound numbers can be up to 50% higher than those obtained on an uncarbonated concrete surface.

5. CARBONATION DEPTH MEASUREMENT TEST

5.1. FUNDAMENTAL PRINCIPLE

Carbonation of concrete occurs when the carbon dioxide, in the atmosphere in the presence of moisture, reacts with hydrated cement minerals to produce carbonates, e.g. calcium carbonate. The carbonation process is also called depassivation. Carbonation penetrates below the exposed surface of concrete extremely slowly. The time required for carbonation can be estimated knowing the concrete grade and using the following equation:

$$t = \left(\frac{d}{k}\right)^2 \tag{9}$$

where

t is the time for carbonation,
d is the concrete cover,
k is the permeability.

Typical permeability values are shown in Table 5.1.

TABLE 5.1. PERMEABILITY VALUES VERSUS CONCRETE GRADE

Concrete Grade	Permeability
15	17
20	10
25	6
30	5
35	4
40	3.5

The significance of carbonation is that the usual protection of the reinforcing steel generally present in concrete due to the alkaline conditions caused by hydrated cement paste is neutralized by carbonation. Thus, if the entire concrete cover over the reinforcing steel is carbonated, corrosion of the steel would occur if moisture and oxygen could reach the steel.

5.2. EQUIPMENT FOR CARBONATION DEPTH MEASUREMENT TEST

If there is a need to physically measure the extent of carbonation it can be determined easily by spraying a freshly exposed surface of the concrete with a 1% phenolphthalein solution. The calcium hydroxide is coloured pink while the carbonated portion is uncoloured.

5.3. GENERAL PROCEDURE FOR CARBONATION DEPTH MEASUREMENT TEST

The 1% phenolthalein solution is made by dissolving 1gm of phenolthalein in 90 cc of ethanol. The solution is then made up to 100 cc by adding distilled water. On freshly extracted cores the core is sprayed with phenolphthalein solution, the depth of the uncoloured layer (the carbonated layer) from the external surface is measured to the nearest mm at 4 or 8 positions, and the average taken. If the test is to be done in a drilled hole, the dust is first removed from the hole using an air brush and again the depth of the uncoloured layer measured at 4 or 8 positions and the average taken. If the concrete still retains its alkaline characteristic the colour of the concrete will change to purple. If carbonation has taken place the pH will have changed to 7 (i.e. neutral condition) and there will be no colour change.

Another formula, which can be used to estimate the depth of carbonation, utilizes the age of the building, the water-to-cement ratio and a constant, which varies depending on the surface coating on the concrete.

$$y = \frac{7.2}{R^2(4.6x - 1.76)^2} C^2 \tag{10}$$

where
y is age of building in years,
x is water-to-cement ratio,
C is carbonation depth,
R is a constant (R= αβ).

R varies depending on the surface coating on the concrete (β) and whether the concrete has been in external or internal service (α). This formula is contained in the Japanese Construction Ministry publication “Engineering for improving the durability of reinforced concrete structures.” α is 1.7 for indoor concrete and 1.0 for outdoor concrete. β values are shown in Table 5.2.

TABLE 5.2. VALUES OF ß

Finished condition	Indoor	Outdoor
no layer	1.7	1.0
plaster	0.79	
mortar + plaster	0.41	
mortar	0.29	0.28
mortar + paint	0.15	
tiles	0.21	0.07
paint	0.57	0.8

The carbonation depth is therefore given by:

$$C = \frac{y^{1/2} R(4.6x - 1.76)}{\sqrt{7.2}} \tag{11}$$

5.4. RANGE AND LIMITATIONS OF CARBONATION DEPTH MEASUREMENT TEST

The phenolphthalein test is a simple and cheap method of determining the depth of carbonation in concrete and provides information on the risk of reinforcement corrosion taking place. The only limitation is the minor amount of damage done to the concrete surface by drilling or coring.

6. PERMEABILITY TEST

6.1. FUNDAMENTAL PRINCIPLE

Permeability of concrete is important when dealing with durability of concrete particularly in concrete used for water retaining structures or watertight sub-structures. Structures exposed to harsh environmental conditions also require low porosity as well as permeability. Such adverse elements can result in degradation of reinforced concrete, for example, corrosion of steel leading to an increase in the volume of the steel, cracking and eventual spalling of the concrete. Permeability tests measure the ease with which liquids, ions and gases can penetrate into the concrete. *In situ* tests are available for assessing the ease with which water, gas and deleterious matter such as chloride ions can penetrate into the concrete.

6.2. GENERAL PROCEDURE FOR PERMEABILITY TEST

A comprehensive review of the wide range of test methods is given in the Concrete Society Technical Report No. 31. Two of the most widely established methods are the initial surface absorption test (ISAT) and the modified Figg air permeability test. The former measures the ease of water penetration into the surface layer of the concrete while the latter can be used to determine the rate of water as well as air penetration into the surface layer of the concrete which is also called the covercrete. Another newly developed technique uses a modification of the laboratory test to determine chloride ion permeability. All the site tests emphasize the measurement of permeability of the outer layer of concrete as this layer is viewed as most important for the durability of concrete.

6.3. EQUIPMENT FOR PERMEABILITY TEST

6.3.1. Initial surface absorption test

Details of the ISAT is given in BS 1881:Part 5 which measures the surface water absorption. In this method, a cup with a minimum surface area of 5000 mm^2 is sealed to the concrete surface and filled with water. The rate at which water is absorbed into the concrete under a pressure head of 200 mm is measured by movement along a capillary tube attached to the cup. When water comes into contact with dry concrete it is absorbed by capillary action initially at a high rate but at a decreasing rate as the water filled length of the capillary increases. This is the basis of initial surface absorption, which is defined as the rate of water flow into concrete per unit area at a stated interval from the start of test at a constant applied head at room temperature.

6.3.2. Modified Figg permeability test

The modified Figg permeability test can be used to determine the air or water permeability of the surface layer of the concrete. In both the air and water permeability test a hole of 10 mm diameter is drilled 40 mm deep normal to the concrete surface. A plug is inserted into this hole to form an airtight cavity in the concrete. In the air permeability test, the pressure in the cavity is reduced to –55 kPa using a hand operated vacuum pump and the pump is isolated. The time for the air to permeate through the concrete to increase the cavity pressure to –50 kPa is noted and taken as the measure of the air permeability of the concrete. Water permeability is measured at a head of 100 mm with a very fine canula passing through a hypodermic needle to touch the base of the cavity. A two-way connector is used to connect this to a syringe and to a horizontal capillary tube set 100 mm above the base of the cavity. Water is injected through the syringe to replace all the air and after one minute the syringe is isolated with a water meniscus in a suitable position. The time for the meniscus to move 50 mm is taken as a measure of the water permeability of the concrete.

6.3.3. *In situ* rapid chloride ion permeability test

This method was originally designed for laboratory application but has been modified for *in situ* use. The procedure for the laboratory test is given in AASHTO T277 and ASTM C1202. The technique is based on the principle that charged ions, such as chloride (Cl^-), will accelerate in an electric field towards the pole of opposite charge. The ions will reach terminal velocity when the frictional resistance of the surrounding media reaches equilibrium with the accelerating force. This is the basis of "electrophoresis", which is utilized in many chemical and biological studies.

A DC power supply is used to apply a constant voltage between the copper screen and the steel reinforcement. The total current flowing between the mesh and the reinforcing bar over a period of six hours is then measured. The total electric charge (in coulombs) is computed and can be related to the chloride ion permeability of the concrete.

6.4. APPLICATIONS OF PERMEABILITY TEST

The methods described do not measure permeability directly but produce a 'permeability index', which is related closely to the method of measurement. In general, the test method used should be selected as appropriate for the permeation mechanism relevant to the performance requirements of the concrete being studied. Various permeation mechanisms exist depending on the permeation medium, which include absorption and capillary effects, pressure differential permeability and ionic and gas diffusion.

Most of these methods measure the permeability or porosity of the surface layer of concrete and not the intrinsic permeability of the core of the concrete. The covercrete has been known to significantly affect the concrete durability since deterioration such as carbonation and leaching starts from the concrete surface. This layer thus provides the first defense against any degradation.

Guidelines on the different categories of permeability are given in Table 6.1.

6.5. RANGE AND LIMITATIONS OF PERMEABILITY TEST

For the ISAT, tests on oven dried specimens give reasonably consistent results but in other cases results are less reliable. This may prove to be a problem with *in situ* concrete. Particular difficulties have also been encountered with *in situ* use in achieving a watertight fixing. The test has been found to be very sensitive to changes in quality and to correlate with observed weathering behaviour. The main application is as a quality control test for precast units but application to durability assessment of *in situ* concrete is growing.

The main difficulty in the modified permeability test is to achieve an air or watertight plug.

The electrical properties of concrete and the presence of stray electric fields affect the rapid chloride permeability test results. Some concrete mixes that contain conductive materials, e.g. some blended cements, in particular, slag cement, may produce high chloride ion permeability though such concrete is known to be very impermeable and dense. The test is also affected by increases in temperature during measurements. However, reasonably good correlation has been obtained between this technique and the traditional 90 day ponding test (AASHTO T259) in the laboratory.

TABLE 6.1. GUIDELINES ON DIFFERENT CATEGORIES OF PERMEABILITY

Test Methods		Units	Concrete permeability/absorption		
			Low	Average	High
ISAT	10 min	mL/m2/s	<0.25	0.25-0.50	>0.5
	30 min		<0.17	0.17-0.35	>0.35
	1 hr		<0.10	0.10-0.20	>0.20
	2 hr		<0.07	0.07-0.15	>0.15
Figg water absorption 50 mm (dry concrete)		s	>200	100-200	<100
Modified Figg air permeability index		s	>300	100-300	<100

7. PENETRATION RESISTANCE OR WINDSOR PROBE TEST

7.1. FUNDAMENTAL PRINCIPLE

The Windsor probe, like the rebound hammer, is a hardness tester, and its inventors' claim that the penetration of the probe reflects the precise compressive strength in a localized area is not strictly true. However, the probe penetration does relate to some property of the concrete below the surface, and, within limits, it has been possible to develop empirical correlations between strength properties and the penetration of the probe.

7.2. EQUIPMENT FOR WINDSOR PROBE TEST

The Windsor probe consists of a powder-actuated gun or driver, hardened alloy steel probes, loaded cartridges, a depth gauge for measuring the penetration of probes, and other related equipment. As the device looks like a firearm it may be necessary to obtain official approval for its use in some countries. The probes have a tip diameter of 6.3 mm, a length of 79.5 mm, and a conical point. Probes of 7.9 mm diameter are also available for the testing of concrete made with lightweight aggregates. The rear of the probe is threaded and screws into a probe driving head, which is 12.7 mm in diameter and fits snugly into the bore of the driver. The probe is driven into the concrete by the firing of a precision powder charge that develops energy of 79.5 m kg. For the testing of relatively low strength concrete, the power level can be reduced by pushing the driver head further into the barrel.

7.3. GENERAL PROCEDURE FOR WINDSOR PROBE TEST

The area to be tested must have a brush finish or a smooth surface. To test structures with coarse finishes, the surface first must be ground smooth in the area of the test. Briefly, the powder-actuated driver is used to drive a probe into the concrete. If flat surfaces are to be tested a suitable locating template to provide 178 mm equilateral triangular pattern is used, and three probes are driven into the concrete, one at each corner. A depth gauge measures the exposed lengths of the individual probes. The manufacturer also supplies a mechanical averaging device for measuring the average exposed length of the three probes fired in a triangular pattern. The mechanical averaging device consists of two triangular plates. The reference plate with three legs slips over the three probes and rests on the surface of the concrete. The other triangular plate rests against the tops of the three probes. The distance between the two plates, giving the mechanical average of exposed lengths of the three probes, is measured by a depth gauge inserted through a hole in the centre of the top plate. 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 has published tables relating the exposed length of the probe with the compressive strength of the concrete. For each exposed length value, different values for compressive strength are given, depending on the hardness of the aggregate as measured by the Mohs' scale of hardness. The tables provided by the manufacturer are based on empirical relationships established in his laboratory. However, investigations carried out by Gaynor, Arni, Mallotra, and several others indicate that the manufacturer's tables do not always give satisfactory results. Sometimes they considerably overestimate the actual strength and in other instances they underestimate the strength.

It is, therefore, imperative for each user of the probe to correlate probe test results with the type of concrete being used. Although the penetration resistance technique has been standardized the standard does not provide a procedure for developing a correlation. A practical procedure for developing such a relationship is outlined below.

(1) Prepare a number of 150 mm × 300 mm cylinders, or 150 mm^3 cubes, and companion 600 mm × 600 mm × 200 mm concrete slabs covering a strength range that is to be encountered on a job site. Use the same cement and the same type and size of aggregates as those to be used on the job. Cure the specimens under standard moist curing conditions, keeping the curing period the same as the specified control age in the field.
(2) Test three specimens in compression at the age specified, using standard testing procedure. Then fire three probes into the top surface of the slab at least 150 mm apart and at least 150 mm in from the edges. If any of the three probes fails to properly penetrate the slab, remove it and fire another. Make sure that at least three valid probe results are available. Measure the exposed probe lengths and average the three results.
(3) Repeat the above procedure for all test specimens.
(4) Plot the exposed probe length against the compressive strength, and fit a curve or line by the method of least squares. The 95% confidence limits for individual results may also be drawn on the graph. These limits will describe the interval within which the probability of a test result falling is 95%.

A typical correlation curve is shown in Fig. 7.1, together with the 95% confidence limits for individual values. The correlation published by several investigators for concrete made with limestone gravel, chert, and traprock aggregates are shown in Fig. 7.2. Note that different relationships have been obtained for concrete with aggregates having similar Mohs' hardness numbers.

7.4. APPLICATIONS OF WINDSOR PROBE TEST

7.4.1. Formwork removal

The Windsor probe test has been used to estimate the early age strength of concrete in order to determine when formwork can be removed. The simplicity of the test is its greatest attraction. The depth of penetration of the probe, based on previously established criteria, allows a decision to be made on the time when the formwork can be stripped.

7.4.2. As a substitute for core testing

If the standard cylinder compression tests do not reach the specified values or the quality of the concrete is being questioned because of inadequate placing methods or curing problems, it may be necessary to establish the *in situ* compressive strength of the concrete. This need may also arise if an older structure is being investigated and an estimate of the compressive strength is required. In all those situations the usual option is to take a drill core sample since the specification will generally require a compressive strength to be achieved. It is claimed, however, that the Windsor probe test is superior to taking a core. With a core test, if ASTM C42 –87 is applied, the area from which the cores are taken needs to be soaked for 40 h before the sample is drilled. Also the sample often has to be transported to a testing

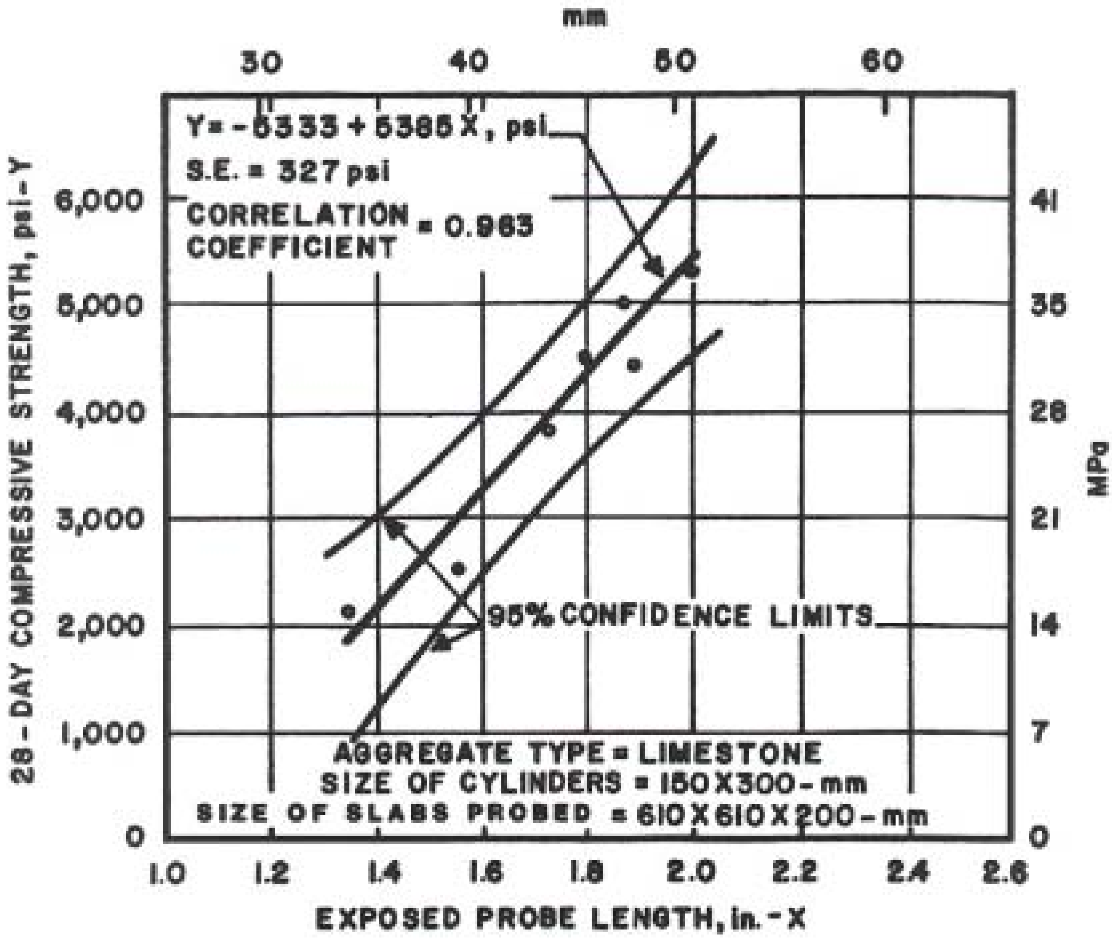

FIG. 7.1. Relationship between exposed probe length and 28 day compressive strength of concrete.

laboratory which may be some distance from the structure being tested and can result in an appreciable delay before the test result is known. Swamy and Al-Hamedreport that the Windsor probe estimated the wet cube strength to be better than small diameter cores for ages up to 28 days. For older concrete the cores estimated the strength better than the probe.

7.5. ADVANTAGES AND LIMITATIONS OF WINDSOR PROBE TEST

The advantages are:

— The test is relatively quick and the result is achieved immediately provided an appropriate correlation curve is available.

— The probe is simple to operate, requires little maintenance except cleaning the barrel and is not sensitive to operator technique.

— Access is only needed to one surface.

— The correlation with concrete strength is affected by a relatively small number of variables.

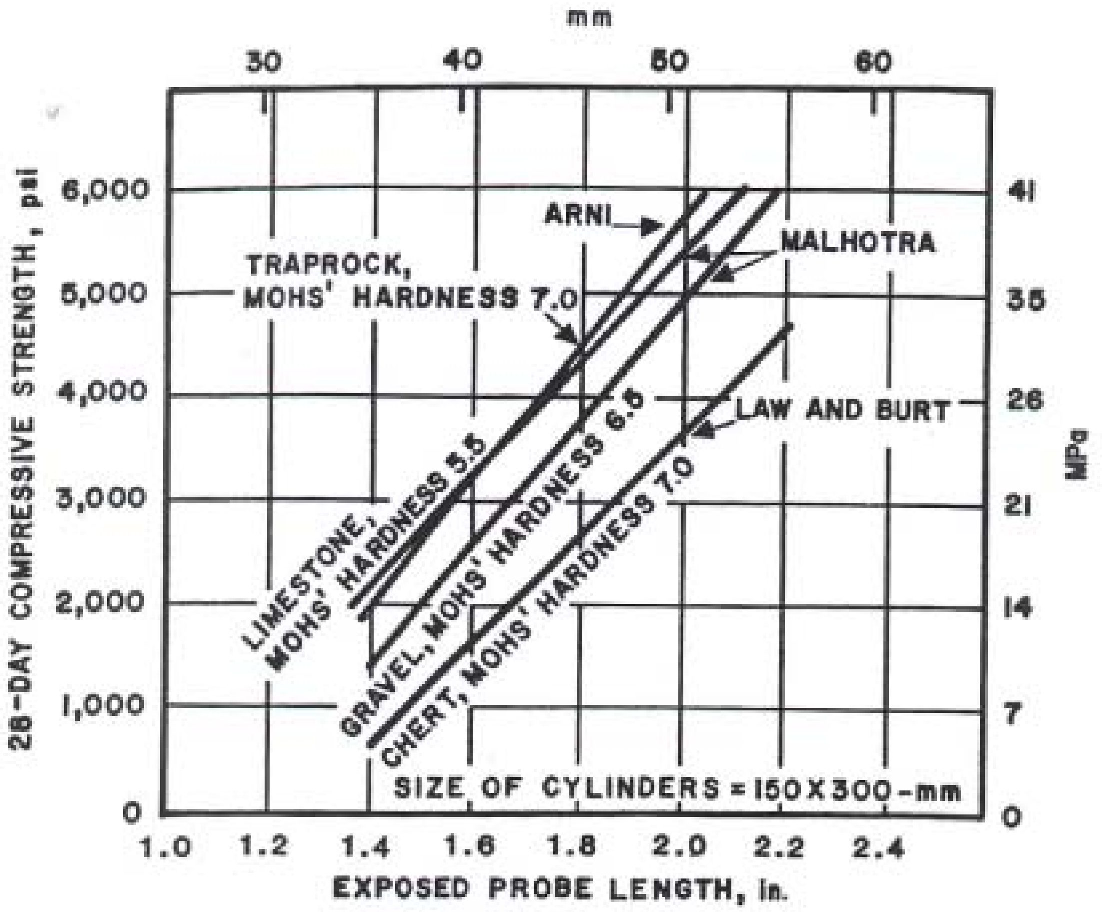

FIG. 7.2. Relation between exposed probe length and compressive strength for different coarse aggregates.

— The test result is likely to represent the concrete at a depth of from 25 mm to 75 mm from the surface rather than just the property of the surface layer as in the Schmidt rebound test.

The limitations are:

— The minimum acceptable distance from a test location to any edges of the concrete member or between two test locations is of the order of 150 mm to 200 mm.

— The minimum thickness of the member, which can be tested, is about three times the expected depth of probe penetration.

— The distance from reinforcement can also have an effect on the depth of probe penetration especially when the distance is less than about 100 mm.

— The test is limited to <40 Mpa and if two different powder levels are used in an investigation to accommodate a larger range of concrete strengths, the correlation procedure becomes complicated.

— The test leaves an 8 mm hole in the concrete where the probe penetrated and, in older concrete, the area around the point of penetration is heavily fractured.

— On an exposed face the probes have to be removed and the damaged area repaired.

8. RESISTIVITY MEASUREMENT

8.1. FUNDAMENTAL PRINCIPLES

There are many techniques used to assess the corrosion risk or activity of steel in concrete. The most commonly used is the half cell potential measurement that determines the risk of corrosion activity. Whilst the half cell potential measurement is effective in locating regions of corrosion activity, it provides no indication of the rate of corrosion. However, a low resistance path between anodic and cathodic sites would normally be associated with a high rate of corrosion than a high resistance path. Such resistivity measurements determine the current levels flowing between anodic and cathodic portions, or the concrete conductivity over the test area, and are usually used in conjunction with the half-cell potential technique. This is an electrolytic process as a consequence of ionic movement in the aqueous pore solution of the concrete matrix. An alternative technique to estimate the rate of corrosion, which is becoming increasingly popular, is the linear polarization resistance.

8.2. EQUIPMENT

Although other commercial devices like the less accurate two probe system are also available, the Wenner four probe technique is generally adopted for resistivity measurement of *in situ* concrete. The technique was first used by geologists to investigate soil strata. The technique can be used to determine resistivities quickly and with little or no damage to the concrete structures under study, Fig. 8.1.

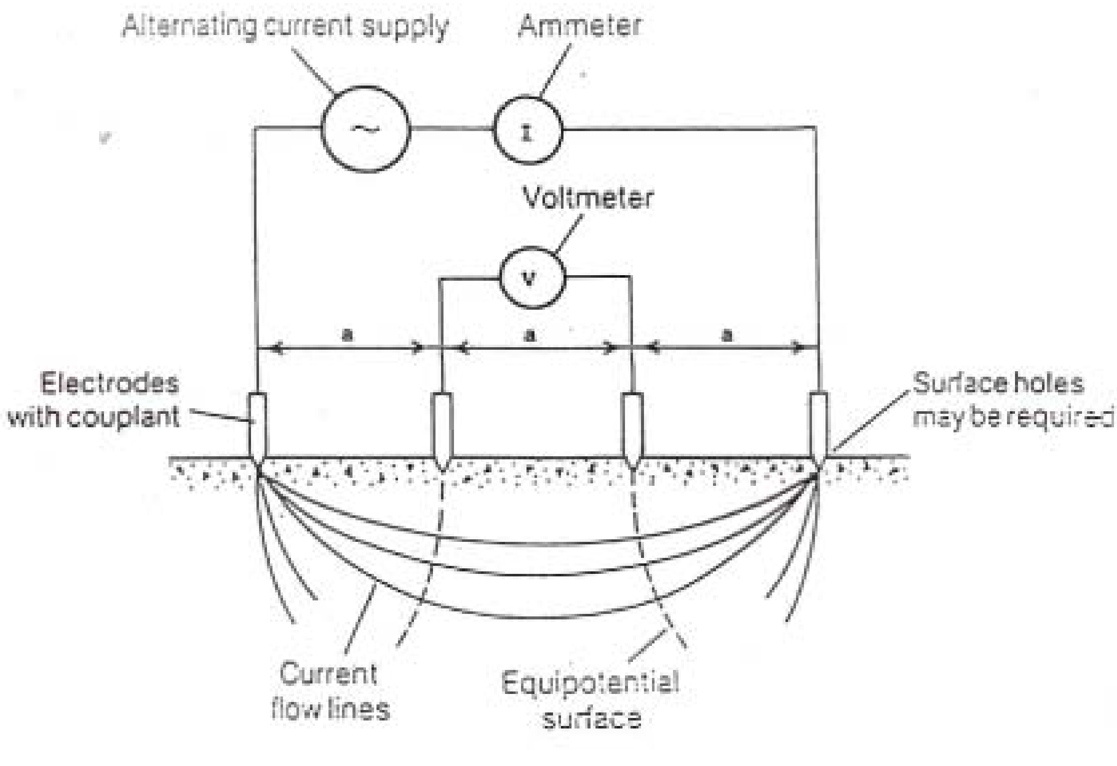

FIG. 8.1. Schematic of Wenner 4 probe resistivity meter.

The equipment consists of four electrodes (two outer current probes and two inner voltage probes) which are placed in a straight line on or just below the concrete surface at equal spacings. A low frequency alternating electrical current is passed between the two outer electrodes whilst the voltage drop between the inner electrodes is measured. The apparent resistivity (ρ) in "ohm-cm" may be expressed as:

$$\rho = 2\pi aV/I \tag{12}$$

where
V is voltage drop,
I is applied current,
a is electrode spacing.

The calculation assumes the concrete to be homogeneous and the inhomogeneity caused by the reinforcement network must be allowed for by properly placing the probes to minimize its effect.

8.3. GENERAL PROCEDURE

Resistivity measurement is a fast, simple and cheap *in situ* non-destructive method to obtain information related to the corrosion hazard of embedded reinforcement.

The spacing of the four probes determines the regions of concrete being measured. It is generally accepted that for practical purposes, the depth of the concrete zone affecting the measurement will be equal to the electrode spacing. If the spacing is too small, the presence or absence of individual aggregate particles, usually having a very high resistivity, will lead to a high degree of scatter in the measurement. Using a larger spacing may lead to inaccuracies due to the current field being constricted by the edges of the structure being studied. In addition, increased error can also be caused by the influence of the embedded steel when larger spacings are employed. A spacing of 50 mm is commonly adopted, gives a very small degree of scatter and allows concrete sections in excess of 200 mm thick to be measured with acceptable accuracy.

The efficiency of surface coupling is also important. In order to establish satisfactory electrical contact between the probes and the concrete, limited damage to the concrete surface sometimes can not be avoided. In some commercial devices, wetting or conductive gel is applied when the probes are pushed against the concrete surface to get better contact. Prewetting of the surface before measurement is also advised. Small shallow holes may also be drilled into the concrete which are filled with a conductive gel. The probes are then dipped into each hole. However, this procedure is not practical for site use.

8.4. APPLICATIONS

The ability of corrosion currents to flow through the concrete can be assessed in terms of the electrolytic resistivity of the material. This resistivity can determine the rate of corrosion once reinforcement is no longer passive. The presence of ions such as chloride will also have an effect. At high resistivity, the rate of corrosion can be very low even if the steel is not passive. For example, reinforcement in carbonated concrete in an internal environment may not cause cracking or spalling due to the very low corrosion currents flowing.

The electrical resistivity of concrete is known to be influenced by many factors including moisture, salt content, temperature, water/cement ratio and mix proportions. In particular, the variations of moisture condition have a major influence on *in situ* test readings. Fortunately, in practice, the moisture content of external concrete does not vary sufficiently to significantly affect the results. Nevertheless, precautions need to be taken when comparing results of saturated concrete, e.g. those exposed to sea water or measurements taken after rain showers, with those obtained on protected concrete surfaces. Another important influence is the ambient temperature. Concrete has electrolytic properties; hence, resistivity will increase as temperature decreases. This is particularly critical when measurements are taken during the different seasons, with markedly higher readings during the winter period than the summer period.

The principle application of this measurement is for the assessment of the corrosion rate and it is used in conjunction with other corrosion tests such as the half-cell potential measurement or linear polarization measurement methods. There are no generally accepted rules relating resistivity to corrosion rate. However, a commonly used guide has been suggested for the interpretation of measurements of the likelihood of significant corrosion for non-saturated concrete where the steel is activated, see Table 8.1.

TABLE 8.1. GUIDE FOR THE INTERPRETATION OF THE MEASUREMENTS DURING CORROSION ASSESSMENT

Resistivity (ohm cm)	Likely Corrosion Rate
Less than 5,000	Very high
5,000 – 10,000	High
10,000 – 20,000	Low / Moderate
Greater than 20,000	Negligible

In practice, it is necessary to calibrate the technique, either through exposing the steel to assess its condition, or by correlating the resistivity values with data collected with other techniques. For instance, the values given in Table 8.1 apply when the half-cell potential measurement shows that corrosion is possible.

9. ELECTROMAGNETIC METHODS OF TESTING CONCRETE

9.1. FUNDAMENTAL PRINCIPLES

The physical principle involved can either be by utilizing eddy current effects or magnetic induction effects.

With covermeters using eddy current effects, currents in a search coil set up eddy currents in the reinforcement which in turn cause a change in the measured impedance of the search coil. Instruments working on this principle operate at frequencies above 1 kHz and are thus sensitive to the presence of any conducting metal in the vicinity of the search head.

With covermeters using magnetic induction, a multi coil search head is used with a lower operating frequency than the eddy current type of device (typically below 90 Hz). The principle used is similar to that of a transformer, in that one or two coils (the primary coils) carry the driving current while one or two further coils (the secondary coils) pick up the voltage transferred via the magnetic circuit formed by the search head and embedded reinforcing bar. Such instruments are less sensitive to non-magnetic materials than those using the eddy current principle. When there is a change to the amount of ferromagnetic material under the search head e.g by the presence of reinforcing bar or other metal object, there is an increase in the field strength. This results in an increase in the voltage detected by the secondary coil, which can be displayed after amplification by a meter.

In both types of instruments both the orientation and the proximity of the metal to the search head affect the meter reading. It is therefore possible to locate reinforcing bars and determine their orientation. The cover to a bar may also be determined if a suitable calibration can be obtained for the particular size of bar and the materials under investigation. Most instruments have a procedure to allow an estimate to be made of both bar size and distance from the probe to the bar when neither is known.

9.2. EQUIPMENT FOR ELECTROMAGNETIC INSPECTION

A number of suitable battery or mains operated covermeters exist. They comprise a search head, meter and interconnecting cable. The concrete surface is scanned, with the search head kept in contact with it while the meter indicates, by analogue or digital means, the proximity of reinforcement.

9.3. GENERAL PROCEDURE FOR ELECTROMAGNETIC TESTING

Calibration of the covermeter

Regular checks on the covermeter should be carried out to establish the accuracy of the instrument. A basic calibration method is given in BS4408 part1 involving a cube of concrete of given proportions with reinforcing bars at specified distances from the surface. If different search heads are to be used with the same meter, calibration checks should be carried out for each head.

9.4. APPLICATIONS OF ELECTROMAGNETIC TESTING METHOD

Electromagnetic covermeters can be used for:

(a) quality control to ensure correct location and cover to reinforcing bars after concrete placement
(b) investigation of concrete members for which records are not available or need to be checked
(c) location of reinforcement as a preliminary to some other form of testing in which reinforcement should be avoided or its nature taken into account, e.g. extraction of cores, ultrasonic pulse velocity measurements or near to surface methods
(d) location of buried ferromagnetic objects other than reinforcement, e.g. water pipes, steel joists, lighting conduits.

9.5. RANGE AND LIMITATIONS OF ELECTROMAGNETIC TESTING METHOD

The search head is traversed systematically across the concrete and, where reinforcement is located, rotated until a position of maximum disturbance of the electromagnetic field is indicated by a meter or by an audible signal. In such a position, under ideal conditions, the indicated cover to the nearest piece of reinforcement may be read if the bar size is known. Further, the axis of the reinforcement will then lie in the plane containing the centre line through the poles of the search head. Where reinforcement is not too congested, it is possible to map out all bars within the area under examination, which lie sufficiently close to the surface. It may also be possible to determine the position of laps. If the bar size is known, the cover can be measured. If the cover is known, the bar size can be estimated. In some equipment it is claimed to be possible to determine the concrete cover over the first layer of reinforcement and then, by using a spacer, estimate the size of the reinforcement used in that layer. It cannot be too strongly emphasized that for maximum accuracy, interference from other reinforcement or magnetic material has to be avoided.

If the concrete cover is in the range of 0 to 20 mm, which is less than the normal operating range of some instruments, the following procedure can be used. A 20 mm thick spacer of material inert to the covermeter, such as wood or plastic, is placed between the search head and the concrete surface. An apparent cover for the particular bar size is read directly from the meter and the indicated cover obtained by subtracting 20 mm from this value. This method is only suitable where the concrete surface is flat and smooth.

The limitations of the method are:

— It is very slow and labour intensive.

— The results are affected by the presence of more than one reinforcing bar in the test area, by laps, by second layers, by metal tie wires and by bar supports.

— For maximum accuracy it has to be calibrated for the concrete used in the structure to eliminate the influence of iron content of the aggregate and cement used.

— The method is unsuitable in the case of closely packed bar assemblies.

— The accuracy is reduced if rough or undulating surfaces are present, e.g. exposed aggregate finishes. The effect on the indicated cover will be similar in magnitude to the surface irregularities within the area of the search head.

— The claims made for the accuracy of reinforcing bar size estimation range from ±2% to ±15%. Care must therefore be taken in reporting a reinforcing bar size which differs from a design requirement without seeking confirmation by exposing the bar. A bar size may only be able to be reliably estimated to within one bar size of the actual bar size.

— Calibrated meter scales are generally valid for a particular grade of reinforcing steel. The effect of different types of steel on the readings obtained is generally small but, in special cases, such as high tensile prestressing bars, it may include errors as high as ±5% or more. Where such materials are present, the covermeter should be calibrated for the reinforcing steel used by constructing a calibration curve.

— For accurate measurement of cover and size, the bar has to be both straight and parallel to the concrete surface.

— Where significant corrosion to reinforcement has occurred, in particular, scaling and migration of corrosion products, misleading indicated cover readings are likely to be obtained.

— Interference effects will occur in the neighbourhood of metallic structures of significant size, such as window fixings, scaffolding and steel pipes, especially when they are immediately behind the search head. The degree of influence will depend on the particular covermeter used but all are affected by either stray magnetic fields or electric fields or both. In such cases reliable use of the instrument may be severely restricted.

9.6. WORK OR SITE CALIBRATION

A site calibration of the covermeter can be carried out by drilling a series of test holes to the bars at positions representing different covers as found by the covermeter. Care should be taken not to damage the reinforcement. The distance between each bar and the concrete surface is then measured at each point using a depth gauge. A calibration curve can then be constructed comparing the actual depth with that given by the covermeter. In situ covers are then calculated using the reference scale readings and this calibration curve.

It may be possible to take advantage of projecting reinforcement to check the performance of the covermeter.

10. RADIOGRAPHIC TESTING

10.1. FUNDAMENTAL PRINCIPLES

The intensity of a beam of X rays or gamma rays suffers a loss of intensity while passing through a material. This phenomenon is due to the absorption or scattering of the X or gamma rays by the object being exposed. The amount of radiation lost depends on the quality of radiation, the density of the material and the thickness traversed. The beam of radiation, which emerges from the material, is usually used to expose a radiation sensitive film so that different intensities of radiation are revealed as different densities on the film.

The relationship between the intensity of photons incident and transmitted is:

$$I = I_0 e^{-\mu x} \tag{13}$$

where

I is transmitted photon intensity,
I_0 is incident photon intensity,
μ is attenuation coefficient,
x is thickness of object.

Fig. 10.1 illustrates this relationship. The specimen absorbs radiation but where it is thin or, where there is a void, less absorption takes place. Since more radiation passes through the specimen in the thin or void areas, the corresponding areas of the film are darker.

10.2. EQUIPMENT FOR RADIOGRAPHIC TESTING METHOD

10.2.1. X ray equipment

Three basic requirements must be met to produce X rays, namely, (a) source of electrons as a heated filament, (b) means of directing and accelerating the electrons as a high voltage supply, and (c) target which the electrons can bombard, normally in the form of heavy metal target. These requirements are fulfilled in an X ray tube (Fig. 10.2), consisting of a glass envelope in which two electrodes are fitted, a cathode and an anode. The cathode serves as a source of electrons. Applying a high voltage across the cathode and the anode first accelerates the electrons, and then stopping them suddenly with a solid target fitted in the anode. Stopping the fast moving electrons results in the generation of X rays.

The important operational requirements of an X ray generator are:

— the X ray tube must be powered by a stable electrical supply. Power variations in the filament and the high voltage circuit alter the spectrum and intensity of the generated X ray.

— the target anode and its connecting support structure must be cooled and be designed to facilitate heat dissipation. A large rotating anode, which spreads the heat produced over a larger area of the anode, is often used to extend the serviceable life of the anode and provide a stable emission of spectra.

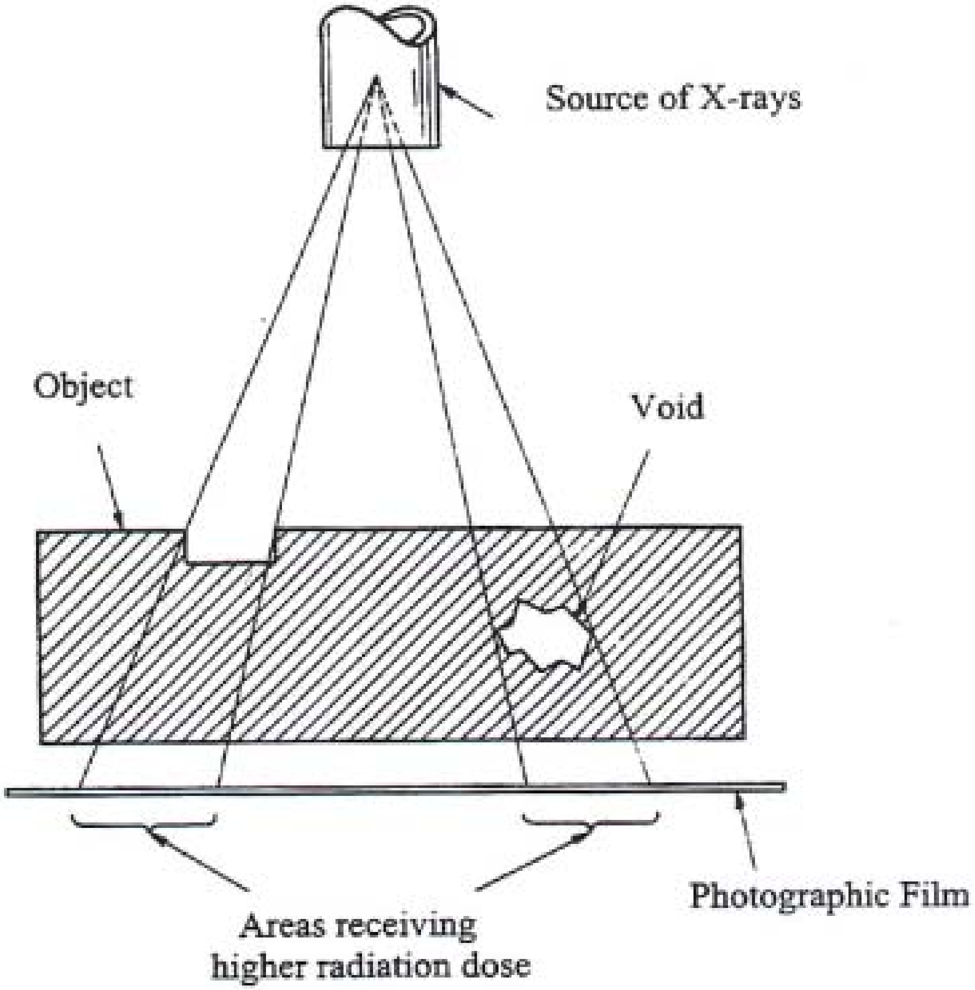

FIG. 10.1. Principle of radiography.

— the electron beam emitted from the cathode and the X ray beam emitted from the anode must be focussed so that a narrow, high intensity beam of X rays is produced.

10.2.2. Gamma ray sources

A radioactive isotope source produces radiation by electron or nuclear energy transitions usually of a single energy or a few discrete energies. Isotope sources emit photons continuously and do not require electrical power. The characteristics of gamma ray sources are:

— HALF-LIFE, which is the period of time required for the intensity of the radiation emitted to fall to one half of its initial value.

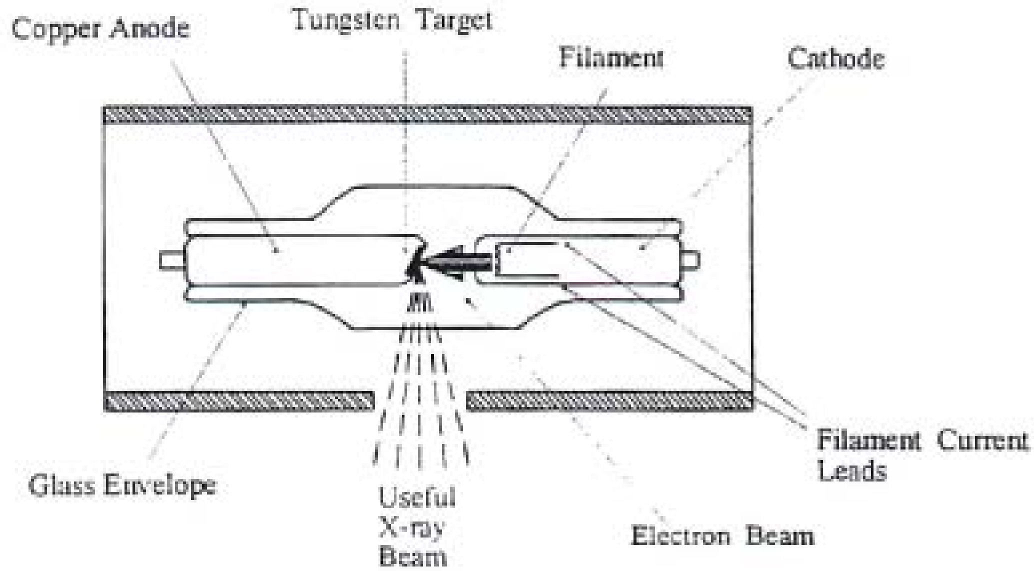

FIG. 10.2. A typical X ray tube.

— ACTIVITY, which is given by the number of atoms of the substance that disintegrate in a given time. This is measured in becquerels (Bq). Becquerel is the "quantity of any radioactive substance in which the number of disintegrations is 1 per second" (1 Bq = 1 d.p.s.).

$$1 \text{ curie} = 3.7 \times 10^{10} \text{ Bq}$$

— Röntgen hour meter (RHM) per curie is the radiation intensity of the source. It is also called the "output" of the source. Some common isotopes used for radiography in decreasing order of photon energy are given in Table 10.1.

TABLE 10.1. COMMON ISOTOPES USED IN RADIOGRAPHY

Source	Half-life	Energy	RHM per Ci
Co_{60}	5.3 years	1.332 MeV 1.173 MeV	1.3 R
Cs_{137}	30.3 years	0.662 MeV	0.33 R
Ir_{192}	74 days	0.317MeV 0.468 MeV	0.55 R

— Half value layer (HVL) is that thickness of a given material that reduces to half the intensity of radiation of a given energy passing through it.

$$T_{1/2} = \ln\frac{2}{\mu'} = \frac{0.693}{\mu'} \quad (14)$$

where

$T_{1/2}$ is the half value layer,

μ' is the linear absorption coefficient of material,

The HVL thickness of some materials for common radioactive sources used in radiography are given in Table 10.2.

TABLE 10.2. HALF VALUE LAYER THICKNESS OF MATERIALS

Material	Half value layer mm			
	Co_{60}	Cs_{137}	Ir_{192}	Ra_{226}
lead	12.45	6.35	4.83	14.22
copper	21.08	16.51	14.48	22.10
iron	22.10	17.27	15.49	23.11
tin	24.13	18.29	16.76	24.89
zinc	26.57	20.57	18.54	27.43
titanium	45.72	35.31	33.02	47.24
aluminium or concrete	68.58	55.34	48.26	73.66

10.2.3. Comparison of X ray sources and gamma ray sources

The decision on which source should be used in a particular situation depends on the application, specification requirements and economic constraints. The main advantage of an X ray source is that they can produce a regulated beam of X rays of greater intensity, in the order of 10^4 times greater than an isotope source. They can also be turned on and off, which may represent a significant safety margin. The disadvantages are that they produce polychromatic X rays, are expensive and cumbersome to move around. Isotope sources on the other hand generate monochromatic radiation. They are relatively cheap and are dimensionally small facilitating portability. The main disadvantage is that isotopes have a limited life as they are subject to an exponentially diminishing activity. They normally produce low intensity fluxes and cannot be turned off which creates additional radiation shielding problems even when not in use.

10.3. GENERAL PROCEDURE FOR RADIOGRAPHIC TESTING METHOD

Unlike most metallic materials, concrete is a non-homogeneous material, a composite with low density matrix, a mixture of cement, sand, aggregate and water, and high density reinforcement made up of steel bars or tendons. Radiography can therefore be used to locate the position of reinforcement bar in reinforced concrete and also estimates can be made of bar diameter and depth below the surface. It can reveal the presence of voids, cracks and foreign materials, the presence or absence of grouting in post tensioned construction and variations in the density of the concrete.

The main limitations of radiography are that because of the thick sections, which have to be radiographed, high-energy radiation is often needed. If X ray equipment is to be used, it can be very heavy, and therefore difficult and time consuming to set up in the field. Because the focus to film distance may have to be long, the exposure time is also long so that the cost of radiography can be high.

If radiographic sources are to be used, the approximate maximum thickness, which can be radiographed in a reasonable amount of time with the sources currently available, is 500 mm using Co-60. The presence of scattered radiation also means that, to achieve a high quality image, a system of shielding the film has to be developed. This can mean collimation, masking, filters, back shielding and special intensifying screens.

The interpretation of concrete radiographs can also be difficult since there is no standardized terminology for imperfections and no standardized acceptance criteria. The complex shape of many concrete structures can also lead to problems and test documentation and reporting can be complex.

"BS1881 Part 205:1986 Testing Concrete - Recommendations for the Radiography of Concrete" does not lay down standards of radiographic image quality, which need therefore to be agreed between parties before an investigation can begin, or give any criteria for the severity of imperfections. The recommendations are based on good practice and are designed to provide methods applicable to the investigation of concrete where radiography is suitable.

10.4. EXTRACTS FROM BS1881 PART 205

10.4.1. Radiation sources

Gamma (γ) ray sources are usually used for concrete thickness up to about 500 mm. Above 500 mm the use of high energy X rays is more appropriate (Table 10.3).

TABLE 10.3. SOURCES OF RADIATION FOR DIFFERENT CONCRETE THICKNESSES

Source	Approximate concrete thickness	
	Minimum	Maximum
Co-60	125 mm	500 mm
Ir-192	25 mm	250 mm
Linac, `18 MeV X rays	500 mm	1600 mm

10.4.2. Type of film

The film should generally be of the medium speed or fast direct-type X ray for use with or without lead screens.

10.4.3. Lead intensifying screens

The thickness of lead foil screens used should be in accordance with Table 10.4.

TABLE 10.4. THICKNESS OF LEAD INTENSIFYING SCREENS USED WITH GAMMA RAYS AND X RAYS FROM LINAC (8 MeV)

Source	Thickness of front screen	Thickness of back screen
Gamma rays	0.10 to 0.25 mm	0.12 to 1.20 mm
Linac (8 MeV)	0.5 to 1.0 mm	0.5 to 1.0 mm

10.4.4. Cassettes

The film and intensifying screens should be enclosed in a flat, rigid, metal or plastic light-tight cassette having sufficient compression to ensure adequate film-screen contact.

10.4.5. Calculation of geometric unsharpness

Geometric unsharpness, U_g (Fig. 10.3) is dependent on the size of the source, object-to-film distance, and source-to-film distance in a manner given by the following equation:

$$U_g = \frac{s.d}{sfd - d} \qquad (15)$$

where

s is the size of the source,
d is the object-to-film distance,
sfd is the source-to-film distance.

Where d is unknown, the thickness, T, of the sample can be used.

In the radiography of concrete, the geometric unsharpness can vary from bar to bar because the angle of the beam from the source relative to each bar is different (Fig. 10.4). This makes estimation of reinforcing bar diameter more difficult.

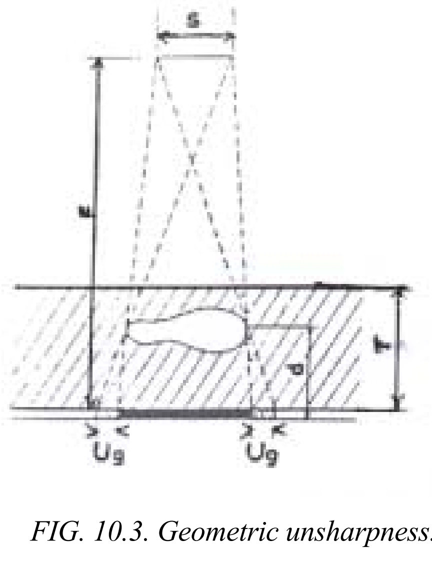

FIG. 10.3. Geometric unsharpness.

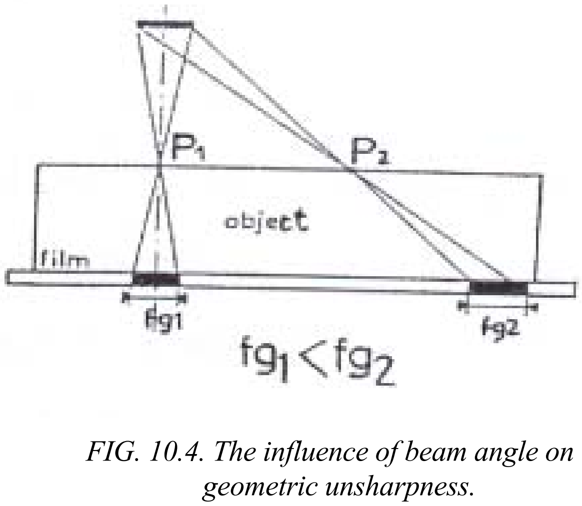

FIG. 10.4. The influence of beam angle on geometric unsharpness.

10.4.6. Calculation of source-to-film distance

The source-to-film distance (sfd) is calculated so that the geometric unsharpness is less than 0.75 mm. An increase of sfd reduces the U_g, and the minimum sfd to obtain the maximum U_g is determined by the following equation:

$$(sfd)_{\min} = \frac{(S-0.75)d}{0.75} \qquad (16)$$

where

sfd is the source-to-film distance,
S is the source diameter,
d is the object-to-film distance.

If the object-to-film distance is unknown, then d can be taken as half the concrete thickness, T, hence, d = T/2 = concrete thickness/2. Then, equation 16 becomes:

$$(sfd)_{\min} = \frac{(S-0.75)T}{1.5} \qquad (17)$$

Table 10.5 gives the value of sfd_{min} for various sizes of source that would produce a radiograph with an acceptable Ug.

TABLE 10.5. MINIMUM SOURCE-TO-FILM DISTANCES FOR VARIOUS SIZES OF SOURCES

Source diameter(mm)	2	3	4	6
Minimum distance (mm)	1.83 × T	2.5 × T	3.17 × T	4.5 × T

For a 3 mm minimum source size the minimum SFD_{min} that would give an acceptable U_g for various thicknesses, T, are as shown in Table 10.6.

TABLE 10.6. SFD_{min} TO GIVE ACCEPTABLE U_g

T (mm)	F (mm)
70	175
80	200
90	225
10	250
110	275

10.4.7. Calculation of exposure time

In the image of the concrete examined, film density corresponding to sound material should not be less than 1.5 or greater than 3: these values include fog density, which should not be greater than 0.3. Exposure time is therefore calculated by the following equation:

$$t_{\exp} = \frac{F.F.\,x\,SFD^2\,x\,\dfrac{T}{HVL}\,x\,60}{A\,x\,RHM\,x\,(100)^2} \qquad (18)$$

where

t_{exp} is exposure time (minute),
F.F. is film factor,
SFD is source-to-film distance,
T is thickness of concrete,
HVL is half value layer,
A is source activity (Ci),
RHM is output of the source.

The exposure time calculations for the radiography of concrete ranging from 70 to 110 mm in thickness and for a 700 mm SFD using a 100 Ci Ir-192 source are given in Table 10.7.

TABLE 10.7. EXPOSURE TIME CALCULATIONS FOR CONCRETE USING A 100 CI Ir-192 SOURCE

Source	Activity (Ci)	Thickness (mm)	SFD (mm)	t_{exp} minutes Film D7	Film D4
Ir-192	100	70	700	1.09	3.88
		80		1.24	4.44
		90		1.39	4.97
		100		1.55	5.53
		110		1.71	6.09

10.4.8. Alignment of the beam

The beam of radiation should be directed to the middle of the section under examination and should be normal to the material surface at that point. The only exception would be where it is known that a specific geometric shape would be best revealed by a different alignment of the beam, e.g. examination of multiple grouting ducts the images of which would otherwise overlap.

10.4.9. Overlap of film

Where it is necessary to overlap film, the overlap should not be less than 10% of the film length or width, and film marking should demonstrate continuity of examination on both front and rear faces of the concrete.

10.4.10. Image quality

The required image quality in terms of the definition and contrast of the image depends upon the purpose of the investigation and should be the subject of mutual agreement between the contracting parties. The assessment can be based either on the use of trial exposures to provide an acceptable image quality or on the use of image quality indicators of a mutually acceptable type.

Based on the DIN series of wire diameters, Table 10.8 is suggested.

TABLE 10.8. SUGGESTED DIN WIRE DIAMETERS FOR VARIOUS CONCRETE THICKNESSES

Concrete thickness and IQI on film side	Concrete thickness and IQI on source side	No. of wires	Diameter (mm)	IQI DIN
	30 cm	1	3.2	1-7
		2	2.5	1-7
	20 cm	3	2.0	1-7
		4	1.6	1-7
		5	1.25	1-7
		6	1.00	1-7 6-12
40 cm		7	0.8	1-7 6-12
	10 cm	8	0.63	6-12
30 cm		9	0.50	6-12
20 cm		10	0.40	6-12 10-16
		11	0.32	6-12 10-16
10 cm		12	0.25	6-12 10-16
		13	0.20	6-12 10-16
		14	0.16	10-16
		15	0.125	10-16
		16	0.100	10-16

10.5. RADIATION PROTECTION IN INDUSTRIAL RADIOGRAPHY

10.5.1. Introduction

Ionizing radiation can be very hazardous to humans and steps must be taken to minimize the risks. This section provides a brief summary of some of the principles of radiation protection associated with the use of sources of radiation. In order to concentrate on the important principles, a certain fundamental level of knowledge of radiation physics has been assumed (e.g. there is no explanation of radiation units as this information is available in many of the IAEA publications).

The essential requirements for protection from ionizing radiation are specified in the "International Basic Safety Standards for Protection against Ionizing Radiation and for the Safety of Radiation Sources" (the BSS) [1]. The standards state that the prime responsibility for radiation protection and safety lies with the licensee, registrant or employer. The Safety Report "Radiation Protection and Safety in Industrial Radiography" [2] describes current practices for the safe control and operation of radiography equipment and facilities, and

Practice Specific Regulations "Radiation Safety in Industrial Radiography" are currently being developed by the IAEA.

10.5.2. Principles of radiation protection

The principles of radiation protection are:

(a) No practice involving exposures to radiation should be adopted unless it produces sufficient benefit to the exposed individuals or to society to offset the radiation detriment that it causes [The justification of practices principle];
(b) In relation to any particular source, the magnitude of individual doses, the number of people exposed and the likelihood of incurring exposure where these are not certain to be received shall be kept as low as reasonably achievable (ALARA), economic and social factors taken into account [The optimization of protection principle]; and
(c) The exposure of individuals resulting from the combination of all the relevant practices should be subject to dose limits. These are aimed at ensuring that no individual is exposed to radiation risks that are judged to be unacceptable in normal circumstances [Dose limitation principle].

It should be emphasized that the most important aspect of radiation protection, assuming that the practice is justified, is to keep radiation doses as low as reasonably achievable.

Occupational dose limits are chosen to ensure that the risk to radiation workers are no greater than the occupational limit in other industries generally considered safe. The BSS specifies that doses to individuals from occupational exposure should not exceed:

(a) an effective dose of 20 mSv per year averaged over five consecutive years;
(b) an effective dose of 50 mSv in any single year;
(c) an equivalent dose to the lens of the eye of 150 mSv in a year; and
(d) an equivalent dose to the extremities (hands and feet) or the skin of 500 mSv in a year.

Dose limits are also specified in the BSS for apprentices (16-18 years of age) and for members of the public.

10.5.3. Administrative requirements

10.5.3.1. Authorization

In order to control the use of radiation sources and to ensure that the operating organization meets the requirements of the BSS, the legal person for any radiation source, unless the source is exempted, will need to apply for a license or registration from the national regulatory authority. Therefore prior to purchasing an industrial radiography source, the user will need to submit an application to the regulatory authority. After ensuring that the proposed use of the radiation source, including working rules, training of staff, storage, etc. meets regulatory requirements, permission for use of the source may be issued with certain terms and conditions. All new industrial radiography equipment should be manufactured and classified according to ISO 3999 [3] or an equivalent standard. Sealed sources are to be in compliance with the requirements of ISO 2919 [4].

10.5.3.2. Inspection and enforcement

The regulatory authority may inspect the registrant/licensee to check their provisions for radiation safety and to physically inspect the premises. They may also inspect on-site radiography to ensure that regulatory requirements are met. Enforcement action may be taken against the operating organization if the level of protection and safety are considered unacceptable.

10.5.4. Management requirements

An effective management structure is required to ensure that a high standard of radiation protection and safety is maintained and that the BSS requirements are met. Written policies should demonstrate management's commitment to safety and the responsibilities of each individual need to be identified. Some organizations need to consult a qualified expert for advice on specific areas of radiation protection. The scope and role of experts is to be clearly defined. Quality assurance programmes are needed to ensure that radiation protection and safety measures within the organization continue to be effective.

10.5.4.1. Safety culture

Management is to foster and maintain a safety culture within their organization. A questioning and learning attitude towards protection and safety should be promoted, and complacency discouraged. Policies and procedures need to be established to ensure that safety and protection have the highest priority. The roles and responsibilities of individuals need to be clearly defined and training provided where this is needed.

10.5.4.2. Local rules and supervision

Employees are to follow the procedures specified in local rules to ensure an adequate level of protection and safety is maintained during normal daily work. A radiation protection programme is to be developed and at least one radiation protection officer (RPO) for overseeing its implementation should be appointed. The RPO is to have the authority to ensure that operating procedures and local rules are followed. The radiography equipment is to be operated by designated radiographers, who have received appropriate training. An example of classroom based training and on the job training in radiation protection for industrial radiographers is presented in Annex III and Annex IV respectively on the IAEA Safety Report "Training in Radiation Protection and the Safe Use of Radiation Sources" [5]. All other personnel, such as assistant radiographers, drivers and storemen who are occupationally exposed to radiation are also to receive training in radiation protection.

10.5.4.3. Quality assurance

Assurance that the radiation protection and safety requirements are being satisfied should be achieved through formal quality control mechanisms and procedures for reviewing and assessing effectiveness of protection and safety measures. Systematic audits and reviews should detect and result in correction of systems that do not function.

10.5.5. Practical protection

The practical elements of radiation protection are: time, distance, shielding and prevention of access. The shorter the time that a person spends near a radiation source, the lower the radiation dose to that individual. Radiation levels decrease rapidly with increasing distance and it is therefore important to never directly handle radiation sources. Specially designed tools with long handles must always be used if a source is to be manipulated. To prevent high radiation fields around a source, they can be surrounded by an adequate thickness of suitable shielding material. In many cases, it is not possible to fully shield the source. It is necessary to prevent access to areas of high radiation fields by building shielded enclosures, interlock systems and by designation of controlled areas.

10.5.5.1. Design and use of shielded enclosures

Experience shows that in general, industrial radiography is most safely carried out in a shielded enclosure. A shielded enclosure is an enclosed space engineered to provide adequate shielding from ionizing radiation to persons in the vicinity. The general design principles are similar for all enclosures, although different characteristics are incorporated, depending on whether the enclosure is to be suitable for X ray, accelerator or gamma radiation equipment. Designs of shielded enclosures requires guidance in terms of anticipated doses, dose rates and exposure times. Designs are based on the ALARA principle, and on any additional dose constraints that may have been specified by the regulatory authority. Design considerations for these installations include: shielding considerations; personnel access door interlocks; fixed radiation monitors; warning signs and symbols; and emergency stops.

10.5.5.2. Site radiography procedures

Most industrial radiography is performed on-site and is influenced by site specific conditions. It needs to be done in an area where specific protection measures and safety provisions are in place i.e. in an area designated as a controlled area. The boundary of the controlled area is set at a dose rate contour which is appropriate under the prevailing circumstances and specific exposure times and is authorized by the regulatory authority.

The boundary of the controlled area has to be demarcated: when reasonable practicable, this has to be done by physical means. Notices are to be displayed at the controlled area boundary. The notices bear the international trefoil symbol, warnings and appropriate instructions in the local language. Visible or audible signals or both are used when a radiographic source is exposed or an X ray machine is energized. Before the start of radiographic work, the area is to be cleared of all people except for authorized personnel. Shielding reduces the size of the controlled area and the radiation doses received by radiographers. Shielding in the form of collimators is designed so that the radiation beam is primarily in the direction necessary for radiography.

Exposure devices on-site have to be secured against unauthorized removal or theft when not under direct surveillance. The devices are to be in a locked area for overnight or temporary storage, e.g. during work breaks.

10.5.6. Radiation monitoring

Workers who may receive significant occupational doses (as defined by the regulatory authority) have to wear appropriate personal dosimeters. They have to be provided and processed by a laboratory or company authorized by the regulatory authority. In addition, a direct reading dosimeter and an audible or alarming ratemeter need to be used by a radiographer when working with ionizing radiation. Such devices are not a substitute for survey meters.

Survey meters are the single most important item of radiation safety related equipment. They are to have characteristics (e.g. energy response, dose rate response capability) required for industrial radiography applications. All monitors are to be routinely calibrated (normally annually) by a qualified expert. Persons carrying out monitoring should be trained, follow approved procedures and keep appropriate records of the radiation levels measured. Monitoring is carried out for several reasons, for example:

- to determine the extent of the controlled area for on-site radiography;
- to determine that radiation source has returned to shielded container;

— when approaching exposure devices since there is the possibility that the radiographic source being stuck in its exposed position or the X-ray exposure control may have failed;

— to check the shielding around a source storage facility;

— to check the radiation levels around a transport container to ensure that it is safe to transport.

10.5.7. Storage, movement and transport of radiographic sources and exposure devices

Storage facilities are to be designed to restrict exposure, keep radiographic sources, exposure containers and control sources secure against theft or damage, and prevent any unauthorized person from carrying out any actions that would be dangerous to themselves or the public. Clear warning notices are to be displayed at the storage facilities. The facilities are to be kept locked, and physical inventory checks are to be made periodically to confirm the location of radiographic sources.

The transportation of radioactive materials is to satisfy the requirements published by the IAEA [6], or the local regulations. The sources are only to be moved around work sites in appropriate containers such as transport packages which are locked correctly, and the keys of which are removed.

10.5.8. Maintenance and leak testing

There should be a programme for testing and maintenance of equipment, including periodic testing of interlocks for shielded enclosures, and examination of safety critical components of radiation sources. The maintenance programme should include routine checks on exposure devices and ancillary equipment, such as control cables, guide tubes and exposure heads. Persons carrying out the maintenance programme need to be aware of the radiation hazards and to be appropriately trained.

When a new radiation source is purchased, it should be provided with a certificate confirming that it is free from contamination. Periodic re-checks are to be carried out by an appropriately trained and qualified person to ensure that the structure of the source remains intact. The frequency of leak testing is recommended by the source manufacturer or supplier, and is normally specified by the regulatory authority.

10.5.9. Emergency response planning

Experience has shown that events involving gamma and X ray exposure devices have the potential for significant radiation exposure to workers and the general public [2,7]. The most serious exposures have occurred when a worker remains next to, or physically handles, an unshielded gamma source assembly, when the gamma source assembly is mishandled or when it is in the possession of a member of the public. The dose rates are high enough to cause localized overexposure in a matter of seconds or minutes, and can result in severe injury and even death.

Users are responsible for the preparation of emergency response plans. Emergency planning and preparedness has four major components: assessment of hazards; acquisition of emergency equipment; development of written procedures; and training to deal with emergency situations, including training in handling of emergency equipment and in following written procedures. Advice and guidance on developing and implementing emergency plans are provided in Safety Series No. 91 [8].

10.5.10. Source disposal

The disposal of used radioactive sealed sources is a major concern that needs to be considered. All such sources must be disposed of safely, in compliance with national regulations and this should be done promptly without extended periods of storage. In many cases, the sources can be returned to the original supplier but the question of eventual disposal and costs is a matter that should be taken into consideration when initially purchasing the radiation source.

10.6. APPLICATIONS OF RADIOGRAPHIC TESTING METHOD

An advantage of radiography over other NDT methods is that it is possible to determine the depth of a flaw or reinforcing bar or tendon in concrete by a shift of the flaw's shadow on the film when the X ray tube or gamma source is moved. The three methods used are based on parallax and on taking two exposures made with different positions of the X ray tube or source.

10.6.1. Measurement of reinforcing bar depth or flaw depth — rigid formula method

Fig. 10.5 is a schematic diagram showing the rigid formula parallax method, which is described by the relationship:

$$\frac{D}{T - D} = \frac{B}{A}$$

or (19)

$$D = \frac{BT}{A + B}$$

where

B is the image shift of the bar,
A is the source shift between exposures,
T is the source to film distance,
D is the distance of the bar above the film or image plane.

Then,

$$H = D - K = \frac{BT}{A + B - K} \quad (20)$$

where

H is the distance of the flaw from the bottom of the plate,
K is the distance from the film plane to the bottom of the plate.

By measuring or knowing the first three parameters, the fourth parameter can be calculated. With this method no markers are necessary. However, the part thickness, source-to-film distance and source shift must be accurately known. In addition the image of the bar must be present on a double exposed radiograph. Normally this radiograph is made by (1) calculating the necessary exposure time, (2) making one part of the radiograph with one-half of this exposure time, (3) moving the source parallel to (and a specified distance along) the film plane, and then (4) making the second half of the exposure.

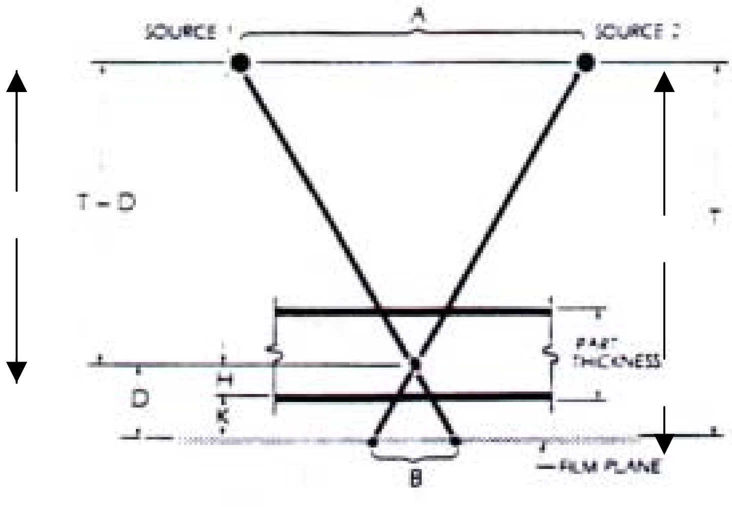

FIG. 10.5. Rigid formula method.

The rigid parallax method can be used when the film is placed in intimate contact with the bottom of the part (i.e. K would be 0) and when there are no limitations on the height of the source above the film plane. It is important to have sufficiently large source-to-film versus top-of-object-to-film ratios when utilizing this method.

10.6.2. Measurement of reinforcing bar depth or flaw depth — single marker approximate method

When the part thickness and bar or flaw height are small relative to the source to film distance, the relationship between D and B approaches linearity and the height of the flaw above the film plane becomes approximately proportional to its parallax. A proportional relationship offers certain advantages in that an artificial flaw or marker can be placed on the source side of the object as shown in Fig. 10.6. The height of the bar or flaw can be estimated or calculated by comparing the shift of its radiographic image with that of the marker. For example, if the single marker shift is twice the shift of the bar image, this indicates that the bar is approximately at the centre of the thickness. This parallax method eliminates the need for detailed measurement of the part thickness, source to film distance and the source shift as required by the rigid method. With source to film distance at least ten times greater than the part thickness, maximum errors of the order of three percent (of the part thickness) can be expected. This is based on the premise that the film is in intimate contact with the part being radiographed. If the film is not in intimate contact with the part, the error will be increased because the proportional ratio is based on the bar height above the film plane.

10.6.3. Measurement of reinforcing bar depth or flaw depth — double marker approximate method

This method can be used when the film cannot be placed in intimate contact with the object or the image of the flaw is not present on a double exposed radiograph (Fig. 10.7). If both markers are thin, their thickness is neglected and it is assumed that they represent the top and bottom of the object. By measuring the parallax or image shift of each marker as well as that of the bar, the relative position of the bar between the two surfaces of the test object can be obtained by linear interpolation.

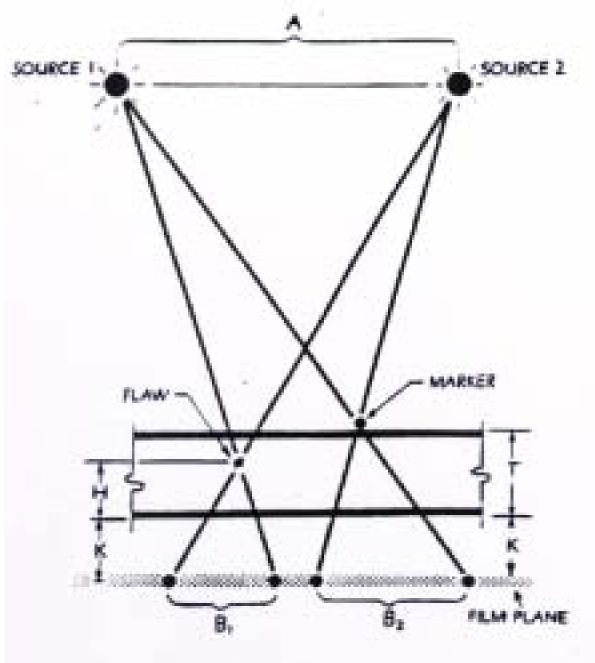

FIG. 10.6. Single marker approximate method.

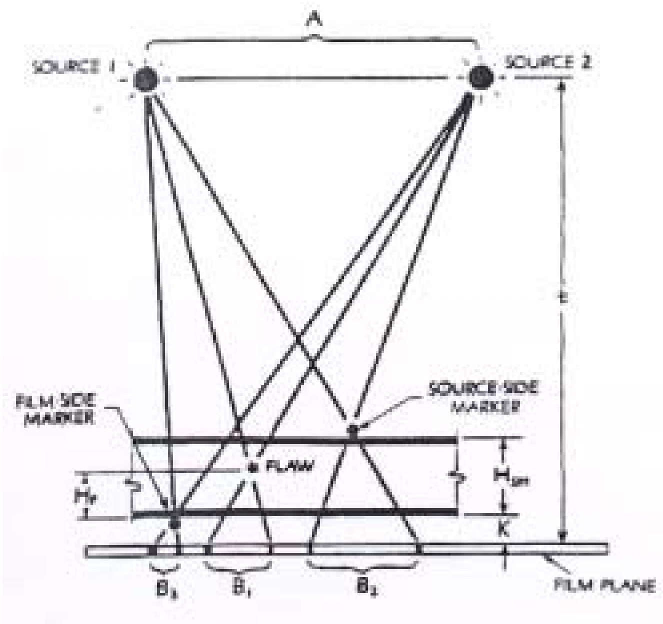

FIG. 10.7. Double marker approximate method.

$B^1 - B^2 \approx \Delta B_f$

and

$B^2 - B^3 \approx \Delta B_{sm}$

and (21)

$$\frac{H_f}{H_{sm}} \approx \frac{\left(B^1 - B^3\right)}{\left(B^2 - B^3\right)} \approx \frac{\Delta B_f}{\Delta B_{sm}}$$

where

H_f is the height of the flaw above the film side marker,
H_{sm} is the distance between the source side marker and the film side marker.

Listed in Table 10.11 are the various parallax formulas, triangulation measurement requirements, and general areas of application for the double marker, single marker and rigid formula parallax methods.

TABLE 10.11. TRIANGULATION MEASUREMENT REQUIREMENTS

Formula	Flaw & marker shifts(B)	Source-to-film distance (T)	Source shift (A)	Film separation (K)	General application rules
Rigid	yes	yes	yes	yes	1. For relatively short source to film distances or where marker placement is difficult. 2. Where part thickness is unknown or difficult to measure.
Approximate formula: source side marker	yes	no	no	yes	1. Also requires the D_2 (part thickness) plus (K) be known. 2. For relatively long distances. 3. For situations where film side marker placement is difficult.
Approximate formula: source side and film side markers	yes	no	no	no	1. Also requires that H_{sm} (part thickness) be known. 2. Most accurate approximate formula. 3. Best for long source to film distances. 4. Simplifies data retrieval.

10.7. RADIOGRAPHIC APPLICATION TO POST TENSIONED CONCRETE BRIDGES

This technique is the most successful for investigating voids in metallic grouted post tensioned ducts in concrete bridge beams; however, the power needed means that areas several hundred metres from the site may have to be cleared to eliminate the possibility of accidental exposure to radiation. This may render the technique inapplicable in urban areas. Also, because of considerable thickness of concrete structures compared to metal structures, radiographic exposure time can be long. The French, however, have pioneered the use of the "Skorpion" to test bridges, and with considerable success in rural areas.

REFERENCES TO SECTION 10

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11. ULTRASONIC TESTING

11.1. PULSE VELOCITY TEST

11.1.1. Fundamental principle

A pulse of longitudinal vibrations is produced by an electro-acoustical transducer, which is held in contact with one surface of the concrete under test. When the pulse generated is transmitted into the concrete from the transducer using a liquid coupling material such as grease or cellulose paste, it undergoes multiple reflections at the boundaries of the different material phases within the concrete. A complex system of stress waves develops, which include both longitudinal and shear waves, and propagates through the concrete. The first waves to reach the receiving transducer are the longitudinal waves, which are converted into an electrical signal by a second transducer. Electronic timing circuits enable the transit time T of the pulse to be measured.

Longitudinal pulse velocity (in km/s or m/s) is given by:

$$v = \frac{L}{T} \tag{22}$$

where

v is the longitudinal pulse velocity,
L is the path length,
T is the time taken by the pulse to traverse that length.

11.1.2. Equipment for pulse velocity test

The equipment consists essentially of an electrical pulse generator, a pair of transducers, an amplifier and an electronic timing device for measuring the time interval between the initiation of a pulse generated at the transmitting transducer and its arrival at the receiving transducer. Two forms of electronic timing apparatus and display are available, one of which uses a cathode ray tube on which the received pulse is displayed in relation to a suitable time scale, the other uses an interval timer with a direct reading digital display.

The equipment should have the following characteristics. It should be capable of measuring transit time over path lengths ranging from about 100 mm to the maximum thickness to be inspected to an accuracy of ±1%. Generally the transducers used should be in the range of 20 to 150 kHz although frequencies as low as 10 kHz may be used for very long concrete path lengths and as high as 1 MHz for mortars and grouts or for short path lengths. High frequency pulses have a well defined onset but, as they pass through the concrete, become attenuated more rapidly than pulses of lower frequency. It is therefore preferable to use high frequency transducers for short path lengths and low frequency transducers for long path lengths. Transducers with a frequency of 50 kHz to 60 kHz are suitable for most common applications.

11.1.3. Applications

Measurement of the velocity of ultrasonic pulses of longitudinal vibrations passing through concrete may be used for the following applications:

- determination of the uniformity of concrete in and between members
- measurement of changes occurring with time in the properties of concrete
- correlation of pulse velocity and strength as a measure of concrete quality.
- determination of the modulus of elasticity and dynamic Poisson's ratio of the concrete.

The velocity of an ultrasonic pulse is influenced by those properties of concrete which determine its elastic stiffness and mechanical strength. The variations obtained in a set of pulse velocity measurements made along different paths in a structure reflect a corresponding variation in the state of the concrete. When a region of low compaction, voids or damaged material is present in the concrete under test, a corresponding reduction in the calculated pulse velocity occurs and this enables the approximate extent of the imperfections to be determined. As concrete matures or deteriorates, the changes, which occur with time in its structure, are reflected in either an increase or a decrease, respectively, in the pulse velocity. This enables changes to be monitored by making tests at appropriate intervals of time.

Pulse velocity measurements made on concrete structures may be used for quality control purposes. In comparison with mechanical tests on control samples such as cubes or cylinders, pulse velocity measurements have the advantage that they relate directly to the concrete in the structure rather than to samples, which may not be always truly representative of the concrete *in situ*.

Ideally, pulse velocity should be related to the results of tests on structural components and, if a correlation can be established with the strength or other required properties of these components, it is desirable to make use of it. Such correlations can often be readily established directly for pre-cast units and can also be found for *in situ* work.

Empirical relationships may be established between the pulse velocity and both the dynamic and static elastic moduli and the strength of concrete. The latter relationship is influenced by a number of factors including the type of cement, cement content, admixtures, type and size of the aggregate, curing conditions and age of concrete. Caution should be exercised when attempting to express the results of pulse velocity tests in terms of strengths or elastic properties, especially at strengths exceeding 60 MPa.

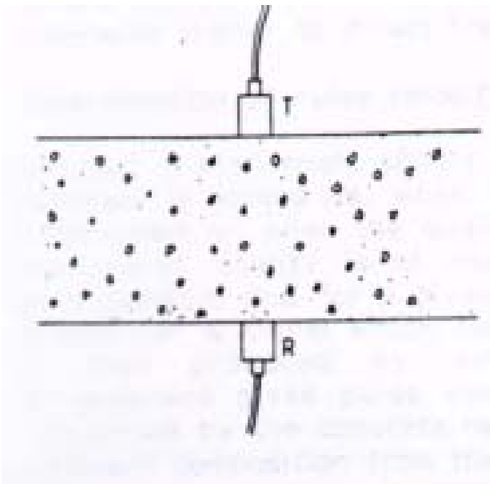
11.1.4. Determination of pulse velocity

11.1.4.1. Transducer arrangement

The receiving transducer detects the arrival of that component of the pulse, which arrives earliest. This is generally the leading edge of the longitudinal vibration. Although the direction in which the maximum energy is propagated is at right angles to the face of the transmitting transducer, it is possible to detect pulses, which have travelled through the concrete in some other direction. It is possible, therefore, to make measurements of pulse velocity by placing the two transducers on either:

- opposite faces (direct transmission)
- adjacent faces (semi-direct transmission): or
- the same face (indirect or surface transmission).

These three arrangements are shown in Figs. 11.1(a), 11.1(b) and 11.1(c).



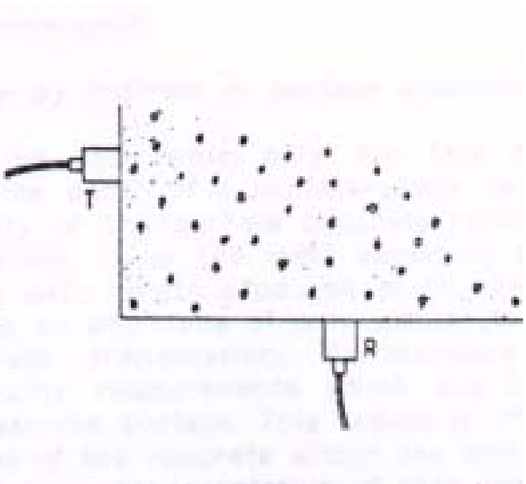


FIG. 11.1(a): Direct transmission. *FIG. 11.1(b): Semi-direct transmission.*

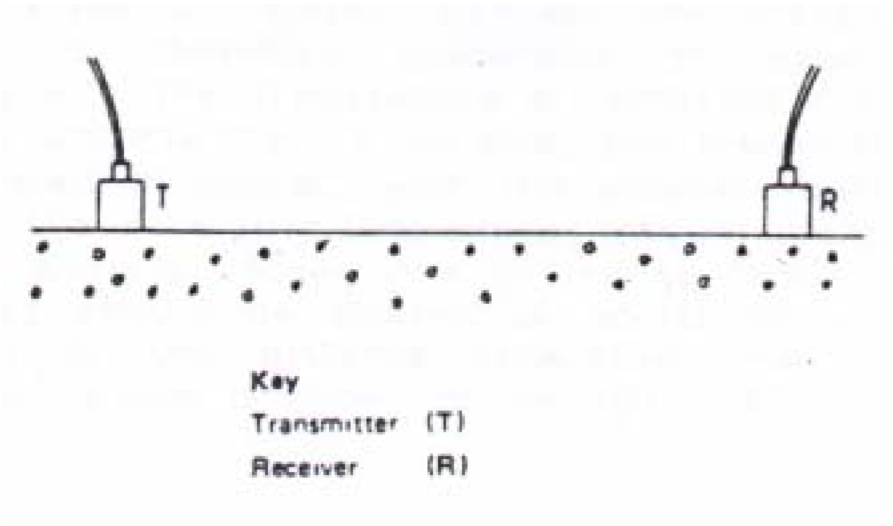


Figure 11.1(c): Indirect or surface transmission.

Fig. 11.1(a) shows the transducers directly opposite to each other on opposite faces of the concrete. However, it is sometimes necessary to place the transducers on opposite faces but not directly opposite each other. Such an arrangement is regarded as semi-direct transmission, Fig 11.1(b).

11.1.4.2. Determination of pulse velocity by direct transmission

Where possible the direct transmission arrangement should be used since the transfer of energy between transducers is at its maximum and the accuracy of velocity determination is therefore governed principally by the accuracy of the path length measurement. The couplant used should be spread as thinly as possible to avoid any end effects resulting from the different velocities in couplant and concrete.

11.1.4.3. Determination of pulse velocity by semi-direct transmission

The semi-direct transmission arrangement has a sensitivity intermediate between those of the other two arrangements and, although there may be some reduction in the accuracy of measurement of the path length, it is generally found to be sufficiently accurate to take this as the distance measured from centre to centre of the transducer faces. This arrangement is otherwise similar to direct transmission.

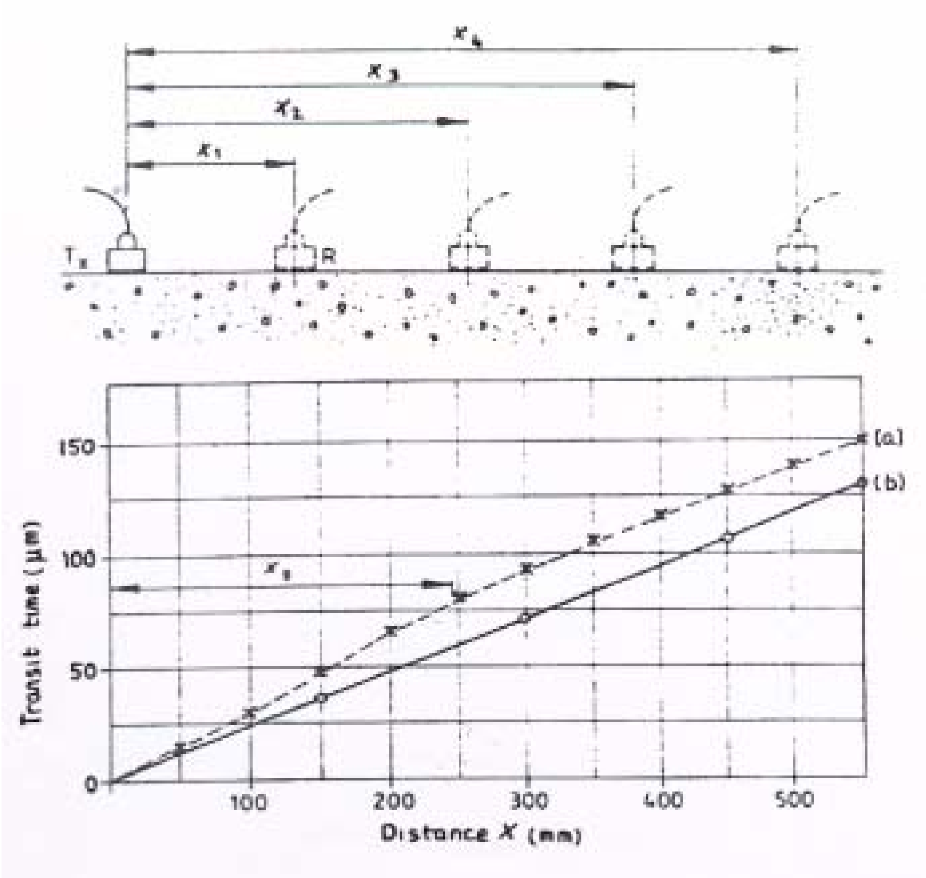
11.1.4.4. Determination of pulse velocity by indirect or surface transmission

Indirect transmission should be used when only one face of the concrete is accessible, when the depth of a surface crack is to be determined or when the quality of the surface concrete relative to the overall quality is of interest. It is the least sensitive of the arrangements and, for a given path length, produces at the receiving transducer a signal which has an amplitude of only about 2% or 3% of that produced by direct transmission. Furthermore, this arrangement gives pulse velocity measurements which are usually influenced by the concrete near the surface. This region is often of different composition from that of the concrete within the body of a unit and the test results may be unrepresentative of that 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 largely on the quality of the concrete under test. Where practicable site measurements should be made to determine this difference. With indirect transmission there is some uncertainty regarding the exact length of the transmission path because of the significant size of the areas of contact between the transducers and the concrete. It is therefore preferable to make a series of measurements with the transducers at different distances apart 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 transducer should be placed 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. An example of such a plot is shown as line (b) in Figure 11.2. The slope of the best 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 a velocity measured in such an instance is unreliable.

11.1.4.5. Coupling the transducer onto the concrete

To ensure that the ultrasonic pulses generated at the transmitting transducers pass into the concrete and are then detected by the receiving transducer, it is essential that there is adequate acoustical coupling between the concrete and the face of each transducer. For many concrete surfaces, the finish is sufficiently smooth to ensure good acoustical contact by the use of a coupling medium and by pressing the transducer against the concrete surface. Typical couplants are petroleum jelly, grease, soft soap and kaolin/glycerol paste. It is important that only a very thin layer of coupling medium separates the surface of the concrete from its contacting transducer. For this reason, repeated readings of the transit time should be made until a minimum value is obtained so as to allow the layer of the couplant to become thinly spread.

Where possible, the transducers should be in contact with the concrete surfaces, which have been cast against formwork or a mold. Surfaces formed by other means, e.g. trowelling, may have properties differing from those of the main body of material. If it is necessary to work on such a surface, measurements should be made over a longer path length than would



(a) Results for concrete with the top 50 mm of inferior quality
(b) Results for homogeneous concrete.

FIG. 11.2. Pulse velocity determination by indirect (surface) transmission.

normally be used. A minimum path length of 150 mm is recommended for direct transmission involving one unmolded surface and a minimum of 400 mm for indirect transmission along one unmolded surface.

When the concrete surface is very rough and uneven, the area of the surface where the transducer is to be applied should be smoothed and leveled. Alternately, a smoothing medium such as quick setting epoxy resin or plaster may be used, but good adhesion between the concrete surface and the smoothing medium has to be ensured so that the pulse propagates correctly into the concrete under test. It is important to ensure that the layer of smoothing medium is as thin as possible. If it is necessary to make a significant build up then the pulse velocity of the smoothing medium has to be taken into account.

11.1.5. Factors influencing pulse velocity measurements

11.1.5.1. Moisture content

The moisture content has two effects on the pulse velocity, one chemical the other physical. These effects are important in the production of correlations for the estimation of concrete strength. Between a properly cured standard cube and a structural element made from the same concrete, there may be a significant pulse velocity difference. Much of the difference is accounted for by the effect of different curing conditions on the hydration of the cement while some of the difference is due to the presence of free water in the voids. It is important that these effects are carefully considered when estimating strength.

11.1.5.2. Temperature of the concrete

Variations of the concrete temperature between 10°C and 30°C have been found to cause no significant change without the occurrence of corresponding changes in the strength or elastic properties. Corrections to pulse velocity measurements should be made only for temperatures outside this range as given in Table 11.1.

TABLE 11.1. EFFECT OF TEMPERATURE ON PULSE TRANSMISSION

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

11.1.5.3. Path length

The path length over which the pulse velocity is measured should be long enough not to be significantly influenced by the heterogeneous nature of the concrete. It is recommended that, except for the conditions stated in 11.1.4.5, the minimum path length should be 100 mm for concrete where nominal maximum size of aggregate is 20 mm or less and 150 mm for concrete where nominal maximum size of aggregate is between 20 mm and 40 mm. The pulse velocity is not generally influenced by changes in path length, although the electronic timing apparatus may indicate a tendency for velocity to reduce slightly with increasing path length. This is because the higher frequency components of the pulse are attenuated more than the lower frequency components and the shape of the onset of the pulse becomes more rounded with increased distance traveled. Thus, the apparent reduction of pulse velocity arises from the difficulty of defining exactly the onset of the pulse and this depends on the particular method used for its definition. This apparent reduction in velocity is usually small and well within the tolerance of time measurement accuracy for the equipment.

11.1.5.4. Shape and size of specimen

The velocity of short pulses of vibration is independent of the size and shape of the specimen in which they travel, 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 11.2 gives the relationship between the pulse velocity in the concrete, the transducer frequency and the minimum permissible lateral dimension of the specimen. If the minimum lateral dimension is less than the wavelength or if the indirect transmission arrangement is used, the mode of propagation changes and therefore the measured velocity will be different. This is particularly important in cases where concrete elements of significantly different sizes are being compared.

TABLE 11.2. EFFECT OF SPECIMEN DIMENSIONS ON PULSE TRANSMISSION

Transducer frequency	Pulse velocity in concrete (km/s)		
	v_c	v_c	v_c
	Minimum permissible lateral specimen dimension		
KHz	mm	mm	mm
24	146	167	188
54	65	74	83
82	43	49	55
150	23	27	30

11.1.5.5. Effect of reinforcing bars

The pulse velocity measured in reinforced concrete in the vicinity of reinforcing bars is usually higher than in plain concrete of the same composition. This is because the pulse velocity in steel may be up to twice the velocity in plain concrete and, under 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 on the proximity of the measurements to the reinforcing bar, the diameter and number of bars and their orientation with respect to the propagation path. The frequency of the pulse and surface conditions of the bar may both also affect the degree to which the steel influences the velocity measurements. Corrections to measured values to allow for reinforcement will reduce the accuracy of estimated pulse velocity in the concrete so that, wherever possible, measurements should be made in such a way that steel does not lie in or close to the direct path between the transducers.

11.1.5.6. Determination of concrete uniformity

Heterogeneities in the concrete within or between members cause variations in pulse velocity, which in turn are related to variations in quality. Measurements of pulse velocity provide a means of studying the homogeneity and for this purpose a system of measuring points which covers uniformly the appropriate volume of concrete in the structure has to be chosen.

The number of individual test points depends upon the size of the structure, accuracy required and variability of the concrete. In a large unit of fairly uniform concrete, testing on a 1m grid is usually adequate but, on small units or variable concrete, a finer grid may be necessary. It should be noted that, in cases where the path length is the same throughout the survey, the measured time might be used to assess the concrete uniformity without the need to convert it to velocity. This technique is particularly suitable for surveys where all the measurements are made by indirect measurements.

It is possible to express homogeneity in the form of a statistical parameter such as the standard deviation or coefficient of variation of the pulse velocity measurements made over a grid. However, such parameters can only be properly used to compare variations in concrete units of broadly similar dimensions.

Variations in pulse velocity are influenced by the magnitude of the path length because this determines the effective size of the concrete sample, which is under examination during each measurement. The importance of variations should be judged in relation to the effect which they can be expected to have on the required performance of the structural member being tested. This generally means that the tolerance allowed for quality distribution within members should be related either to the stress distribution within them under critical working load conditions or to exposure conditions.

11.1.6. Detection of defects

The use of the ultrasonic pulse velocity technique to detect and define the extent of internal defects should be restricted to well-qualified personnel with previous experience in the interpretation of survey results. Attention is drawn to the potential risk of drawing conclusions from single results.

When an ultrasonic pulse travelling through concrete meets a concrete-air interface there is negligible transmission of energy across this interface. Thus any air filled void lying immediately between transducers will obstruct the direct ultrasonic beam when the projected length of the void is greater than the width of the transducers and the wavelength of sound used. When this happens the first pulse to arrive at the receiving transducer will have been diffracted around the periphery of the void and the transit time will be longer than in similar concrete with no void.

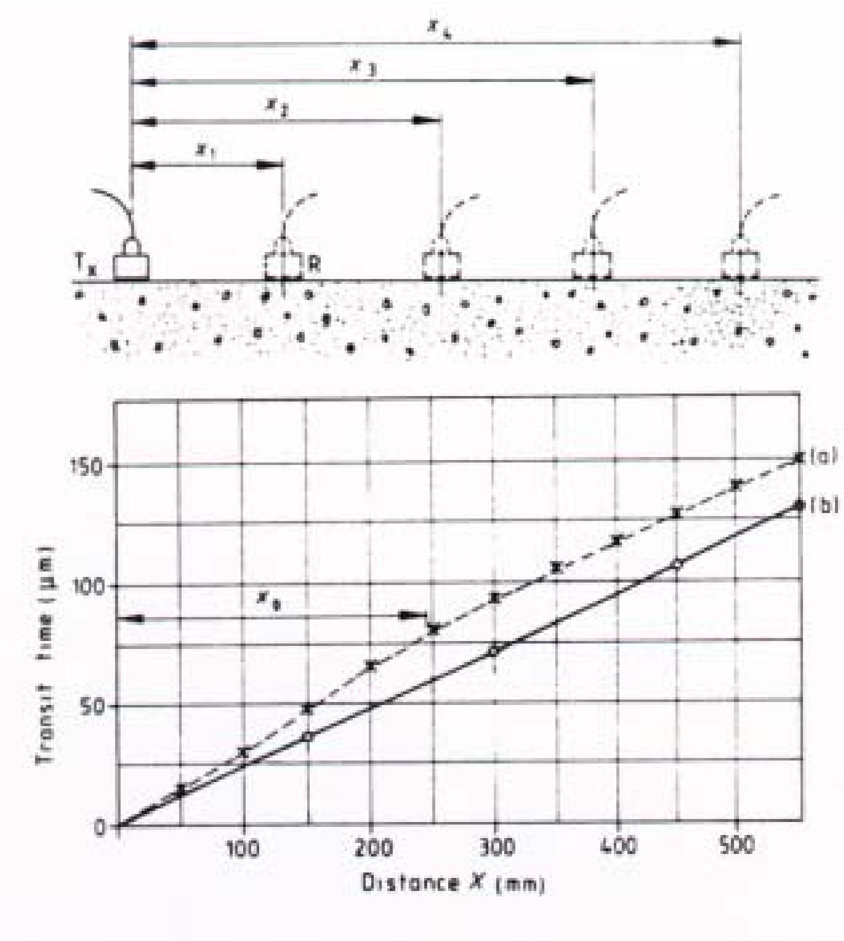
It is possible to make use of this effect for locating flaws, voids or other defects greater than about 100 mm in diameter or depth. Relatively small defects have little or no effect on transmission times but equally are probably of minor engineering importance. Plotting contours of equal velocity often gives significant information regarding the quality of a concrete unit. The method used to detect a void is to draw a grid on the concrete with its points of intersection spaced to correspond to the size of void that would significantly affect the concrete performance. A survey of measurements at the grid points enables a large cavity to be investigated by measuring the transit times of pulses passing between the transducers when they are placed so that the cavity lies in the direct path between them. The size of such cavities may be estimated by assuming that the pulses pass along the shortest path between the transducers and around the cavity. Such estimates are valid only when the concrete around the cavity is uniformly dense and the pulse velocity can be measured in that concrete.

The method is not very successful when applied to structures with cracks because the cracked faces are usually sufficiently in contact with each other to allow the pulse energy to pass unimpeded across the crack. This can happen in cracked vertical bearing piles where there is also sufficient compression to hold the faces close together. If the concrete is surrounded by water such that the crack is filled with water, the crack is undetectable since ultrasonic energy can travel through a liquid.

11.1.6.1. Estimating the thickness of a layer of inferior quality concrete

If concrete is suspected of having a surface layer of poor quality because of poor manufacture, or damage by fire, frost or sulphate attack, the thickness of the layer can be estimated from ultrasonic measurements of transit times along the surface.

The technique used is to place the transmitting transducer on the surface and the receiving transducer a distance "x_1" from the transmitting transducer. The transit time is measured and then measured again at distances of "x_2", "x_3", etc. The transit times are plotted against distance as in Fig. 11.3 in which x is 50 mm. At the shorter distance of separation of



(a) Results for concrete with the top 50mm of inferior quality
(b) Results for homogeneous concrete.

FIG. 11.3. Plot of transit time versus distance.

the transducers, the pulse travels through the surface layer and the slope of the experimental line gives the pulse velocity in this surface layer. Beyond a certain distance of separation the first pulse to arrive has passed along the surface of the underlying higher quality concrete and the slope of these experimental points gives the velocity in that concrete.

The distance x_o at which the change of slope occurs together with the measured pulse velocities in the two different layers of concrete, enables an estimate of the thickness t (in mm) of the surface layer to be made using the equation below.

$$t = \frac{x_o}{2}\sqrt{\frac{(v_s - v_d)}{(v_s + v_d)}} \tag{23}$$

where

v_d is the pulse velocity in the damaged concrete (in km/s),
v_s is the pulse velocity in the underlying sound concrete (in km/s),
x_o is the distance from the transmitter at which the slope changes (in mm).

The method is applicable to extensive surface areas in which the inferior concrete forms a distinct layer of fairly uniform thickness. Localized areas of damaged or honeycombed concrete are more difficult to test but it is possible to derive an approximate thickness of such localized poor quality material if both direct transmission and surface propagation measurements are made.

11.1.6.2. Determination of changes in concrete properties

Pulse velocity measurements are particularly useful to follow the hardening process, especially during the first 36 h. Here, rapid changes in pulse velocity are associated with physiochemical changes in the cement paste structure, and it is necessary to make measurements at intervals of 1 h or 2 h if these changes are to be followed closely. As the concrete hardens these intervals may be lengthened to 1 day or more after the initial period of 36 h has elapsed.

Measurements of changes in pulse velocity are usually indicative of changes in strength and have the advantage that they can be made over progressive periods of time on the same test piece throughout the investigation. Since the quality of concrete is usually specified in terms of strength; 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 factors including age, curing conditions, moisture condition, mix proportions, type of aggregate and type of cement. If an estimate of strength is required it is necessary to establish a correlation between strength and velocity for the particular type of concrete under investigation. This correlation has to be established experimentally by testing a sufficient number of specimens to cover the range of strengths expected and to provide statistical reliability. The confidence that can be ascribed to the results will depend on the number of samples tested. It is possible to establish a correlation between ultrasonic pulse velocity and strength either as measured in accordance with compressive strength tests or by carrying out tests on a complete structure or unit. The reliability of the correlation will depend on the extent to which the correlation specimens represent the structure to be investigated. The most appropriate correlation will be obtained from tests in which the pulse velocity and strength are measured on a complete structure or unit. It is sometimes more convenient to prepare a correlation using tests on molded specimens. It should be noted that experience has shown that a correlation based on molded specimens generally gives a lower estimate of strength than would be obtained by cutting and testing cores.

11.1.6.3. Examples of relationships between pulse velocity and compressive strength

Some figures suggested by Whitehurst for concrete with a density of approximately 2400kg/m^3 are given in Table 11.3. According to Jones, however, the lower limit for good quality concrete is between 4.1 and 4.7 km/s. Fig. 11.4, based on Jones' results, illustrates this effect. Despite this relationship between pulse velocity and compressive strength, ultrasonic pulse velocity measurements are not usually used as a means of quality control on construction sites. Unfortunately there is no satisfactory correlation between the variability of the compression test samples, be they cubes or cylinders, and the variability of the pulse velocity measurements.

TABLE 11.3. CLASSIFICATION OF THE QUALITY OF CONCRETE ON THE BASIS OF PULSE VELOCITY

Longitudinal pulse velocity		Quality of concrete
km/s.10^3	ft/s	
>4.5	>15	excellent
3.5-4.5	12-15	good
3.0-3.5	10-12	doubtful
2.0-3.0	7-10	poor
<2.0	<7	very poor

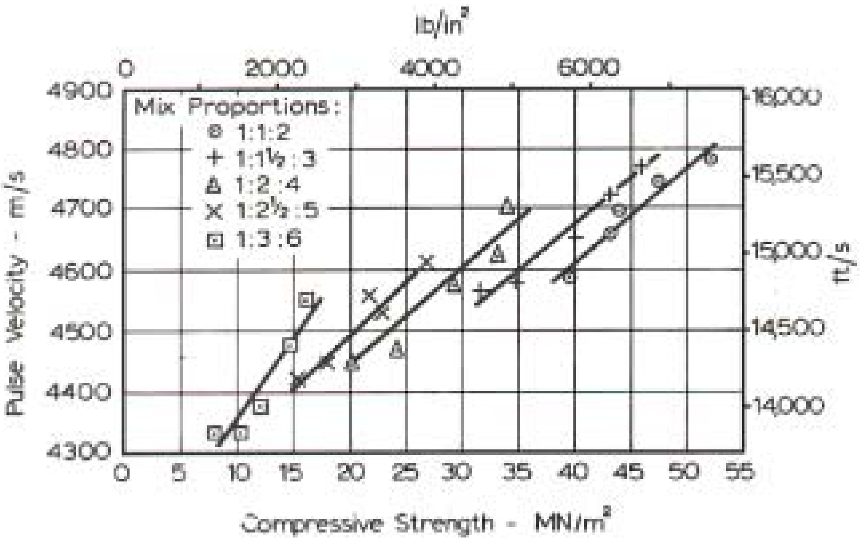


FIG. 11.4. Relation between ultrasonic pulse velocity and compressive strength for concretes of different mix proportions.

11.1.6.4. Determination of the modulus of elasticity and dynamic Poisson's ratio

The relationship between these elastic constants and the velocity of an ultrasonic pulse travelling in an isotropic elastic medium of infinite dimensions is given below:

$$E_d = \frac{\rho v^2 (1+\upsilon)(1-2\upsilon)}{(1-\upsilon)} \qquad (24)$$

where

E_d is the dynamic elastic modulus (in MN/m^2),
υ is the dynamic Poisson's ratio,
ρ is the density (in kg/m^3),
v is the pulse velocity (in km/s).

If the values of υ and ρ are known, it is possible to use above equation (24) to determine the value of E_d in concrete samples for a wide range of shapes or sizes. This is because the pulse velocity is not significantly affected by the dimensions of the test specimen, except when one or more of the lateral dimensions is small relative to the wavelength of the pulse. Similarly υ could be determined if the values of ρ and E_d are known.

11.1.7. Developments in ultrasonic tomography

Tomographic modelling using fuzzy logic enables an NDT user to produce a 2-D cross-section of the structure based upon the longitudinal velocity calculated from an ultrasonic system — typically using 50 kHz transducers on concrete.

The work has produced successful and simple 2-D images on concrete beams of 1-2 m thickness (Martin and Forde, 1999).

Tomographic surveys were carried out on a beam, Fig. 11.5, at positions 0.4 m, 0.8 m and 0.9 m from the front of the beam. The surveys at position 0.4 m and 0.8 m were undertaken to aid interpretation of the impact-echo results. A further test was carried out at 0.9 m in order that a comparison could be made to that undertaken at the section 0.8 m from the front.

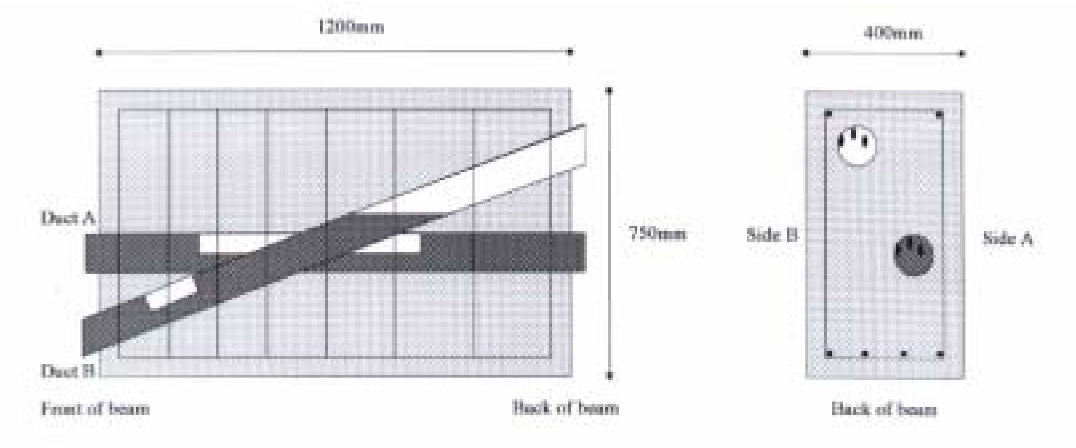


FIG. 11.5. Beam with tendon ducts.

The tomographic investigation grid was set up on the sides and top surface of the beam with test points at a spacing of 100mm. The results for the velocity contour plot from section 0.4m is shown in Fig. 11.6. There is a clear reduction in velocity at the position of duct A. This, taken with the impact-echo results, indicates that duct A is voided at this point. No clear indication of voiding in duct B is seen. An area of low velocity is shown in the lower right hand corner. However, this low velocity section is largely confined to the outer layer of pixels. The number of ray paths in these areas is small and unreliable results are to be expected.

Fig. 11.7 shows a velocity plot at section 0.8m. Impact-echo results indicated possible voiding in both ducts, with Duct B being more likely to be voided and poor concrete in the lower section of the beam. These results are generally confirmed by the tomographic survey. A low velocity area can be seen in the lower part of the beam corresponding to poor concrete. The calculated velocity at the position of duct B (slightly above the centre and to the right) is reduced indicating voiding in this duct. The velocity calculated at the position of duct A seems similar to that of the surrounding concrete but lower velocity areas are clearly visible below this. Voiding in the concrete beneath the duct would give this tomographic response and could also give rise to the reflections measured using the impact-echo techniques.

The study concluded that

— field trials have shown that time of flight tomography potentially provides a highly successful method of investigating post-tensioned concrete beams

— the method is somewhat time consuming and so should be used in conjunction with a simpler testing method, e.g. sonic impact-echo, which identifies areas of interest

— the smaller the ducts to be investigated, the smaller the required distances between testing stations; this therefore significantly increases the testing time

— array systems could be developed which would greatly reduce the testing time.

11.2. ULTRASOUND PULSE ECHO

With Ultrasound Pulse Echo testing it is possible to detect internal features in concrete with one sided access to the structure. The principle is based on the measurement of the time interval between transmitting an ultrasonic impulse into the structure and receiving an echo. The distance to an inner reflector can be determined, if the velocity of sound is known through the simple equation $d=v_L/2T$ (T: transit time, v_L: Velocity of the longitudinal wave, d: distance).

The frequency of the ultrasound must be as low as 50 kHz because of the scattering of the sound waves by the aggregates and air pores. Ultrasound is highly attenuated in concrete so it is impossible in most cases to get a direct reading of the echo. There have been recent advances in the utilization of the ultrasonic pulse echo method for concrete structure testing in the situations that follow.

11.2.1. Thickness measurement of concrete slabs with one sided access

The phase shift superposition technique can be applied to determine the thickness of a concrete slab. The test is performed in a bistatic arrangment by measuring several A-scans (20+) with varying distances between the transmitting and the receiving transducer. The velocity of sound must be known, e.g. by testing it on a core.

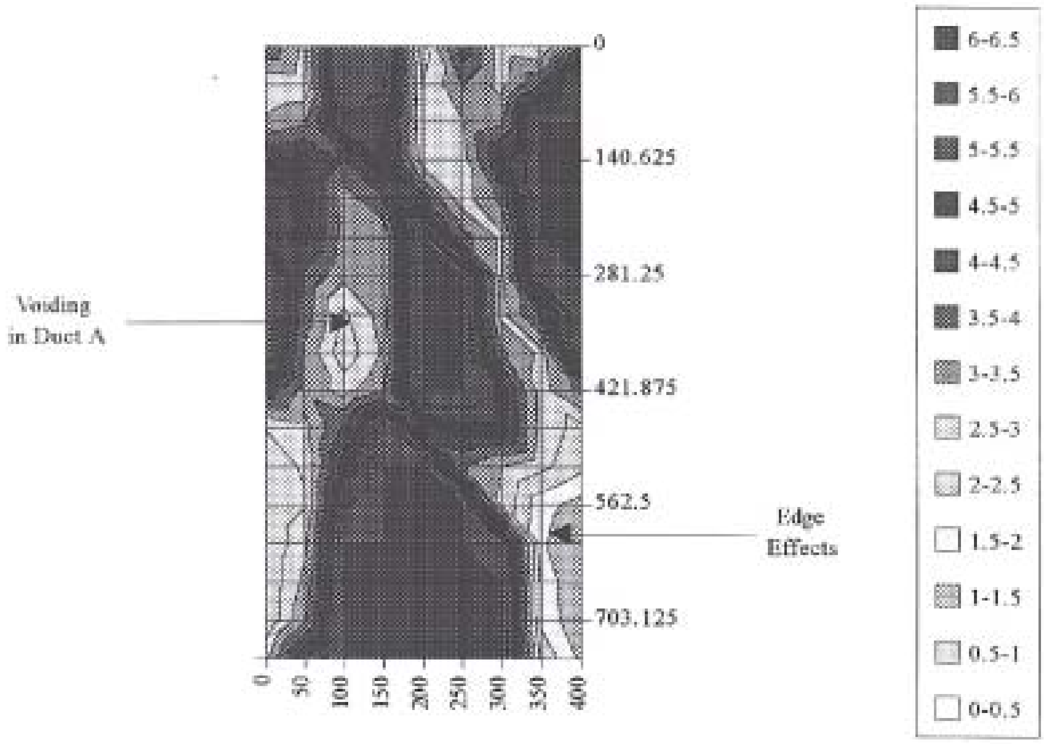


FIG. 11.6. Tomographic survey- Position 0.4 m from front end.

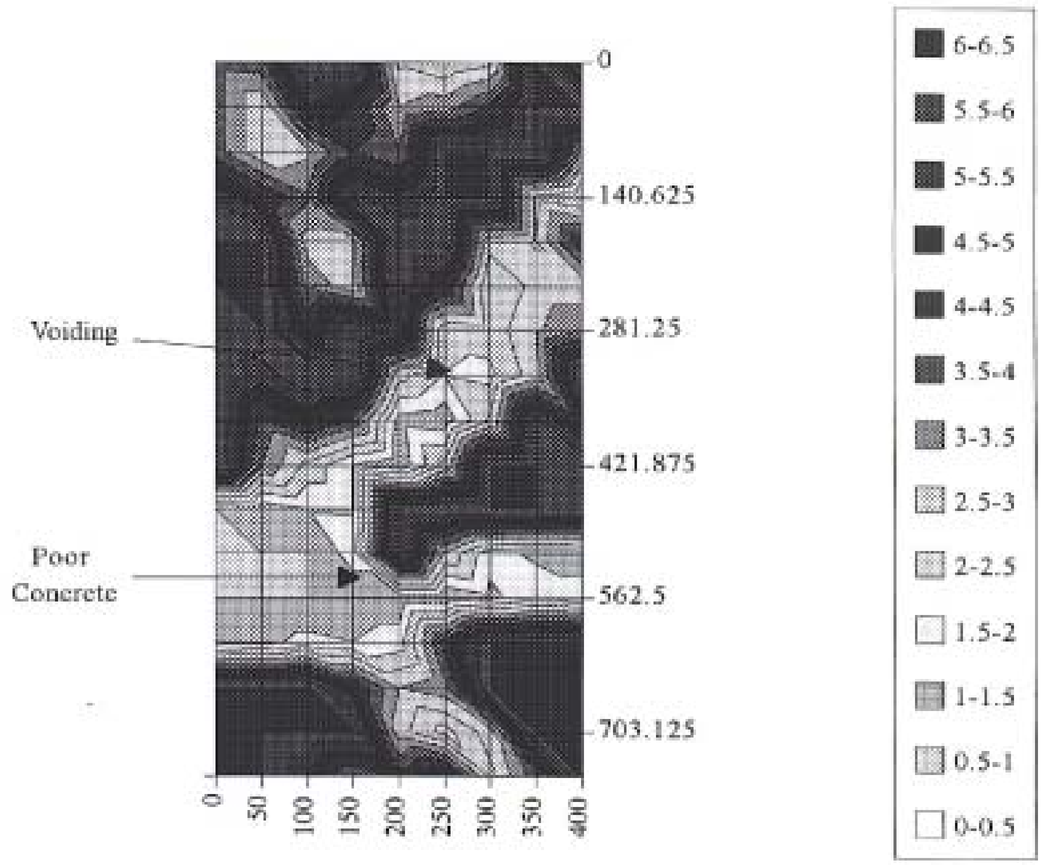


FIG. 11.7. Tomographic survey – Position 0.8 m from front end.

For a given assumed thickness of the concrete slab, the phase shift for the backface echo in each A-scan is calculated and the A-scans time is shifted accordingly and averaged. If true thickness is the same, the backface reflection is amplified and any scattering is reduced because it appears statistical. By calculating the synthetic time shifted average over the range of the expected thickness of the specimen, the maximum of the synthetic echo is at the true thickness of the specimen, Fig. 11.8.

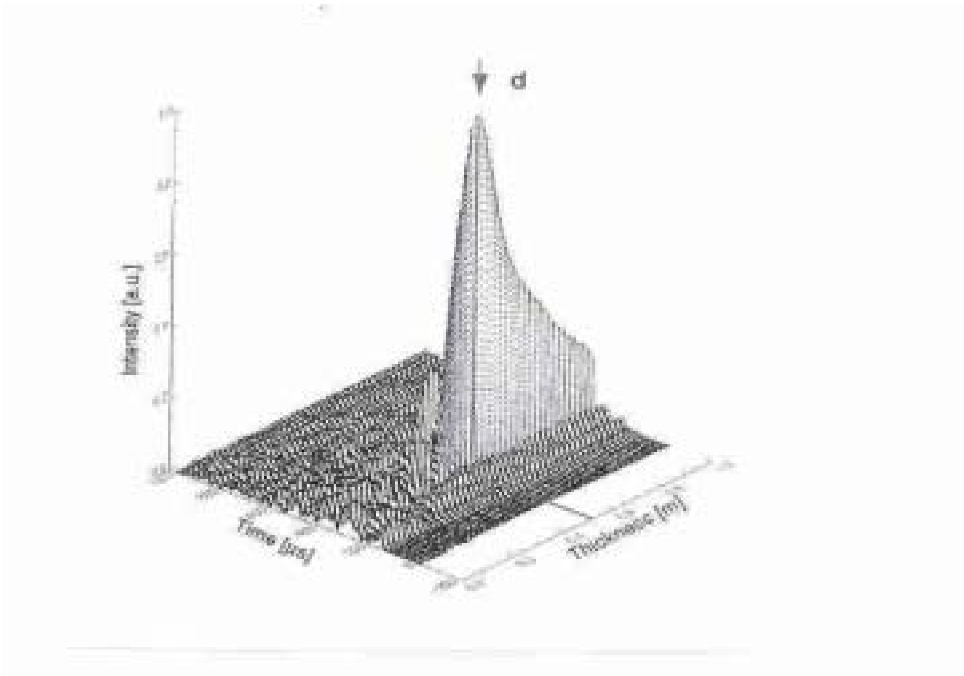


FIG. 11.8. Maximum of the synthetic echo is at the true thickness of the specimen.

Using an array of approximately 10 transducers, which can act as transmitter and receiver, a large number of measurements can be done in a very short time and the calculation performed. Again it should be pointed out that the thickness measurement can only be accurate if the velocity of sound in the test object is known.

11.2.2. Post-tensioned duct inspection

Ultrasound pulse echo has been successfully used to locate voids inside ducts. These defects can be a serious safety hazard and therefore it is important to have testing methods available.

The investigation of ducts with a thick concrete cover (>10 cm) requires time consuming and complicated data acquisition and analysis. Many readings (>1000) have to be taken on the surface of the object above the area to be inspected. All these A-scans are then put into a 3D SAFT (Synthetic Aperture Focusing Technique) analysis, which produces a 3D image of the inner reflectors. By plotting projections and slices through this 3D array, the position of the duct and possible voids can be identified.

To reduce the resources needed it has been proven advantageous to first locate the duct with radar and then pre-select suspected positions of the duct.

This method is not widely available but is the only method, which is able to reliably locate ungrouted areas in ducts that are covered by more than 10 cm in concrete.

11.3. IMPACT-ECHO/RESONANCE FREQUENCY/STRESS WAVE TEST

A number of non-destructive test methods rely on the effect a structure has on the propagation of stress waves. The most common techniques are pulse-echo, impact-echo, impulse–response and spectral analysis of surface waves. The methods differ in the way that the stress waves are generated and on the signal processing techniques that are used.

11.3.1. Fundamental principles

This is an effective method of locating large voids or delaminations in plate like structures, e.g. pavements or bridge decks, where the defect is parallel to the test surface. A mechanical impact produces stress waves of 1 to 60 kHz. The wavelengths of from 50 mm to 2000 mm propagate as if in a homogeneous elastic medium.

The mechanical impact on the surface generates compression, shear and surface waves. Internal interfaces or external boundaries reflect the compression and shear waves. When the waves return to the surface where the impact was generated, they can be used to generate displacements in a transducer and subsequently a display on a digital oscilloscope. The resulting voltage-time signal is digitized and transformed, in a computer, to amplitude vs. frequency plot. The dominant frequencies appear as peaks on the frequency spectrum. The dominant frequency is not necessarily the thickness signal. Using each of the frequencies identified as peaks on the frequency spectrum, the distances to the reflecting surfaces can be calculated from

$$d = \frac{V}{2f} \qquad (25)$$

where

d is distance,
f is dominant frequency,
V is velocity of compression waves in the test material.

If the receiver is placed close to the impact point the reflected signals may not be seen because the transducer is still ringing due to the impact. The type of impact used has a significant influence on the success of the test. The shorter the contact time, the higher the range of frequencies contained in the pulse. An estimate of the maximum frequency excited is the inverse of the contact time:

$$f_{max} = 1/\ t_{D}, \qquad (26)$$

where

t_D is the contact time,
f_{max} is the maximum frequency.

Sansalone and Street gave an estimate of the maximum frequency for a steel ball bearing of diameter D:

$$f_{max}\,(KHz) = 291/D\ (mm) \qquad (27)$$

Thus the contact time determines the depth of the defect that can be detected by impact-echo testing. As the contact time decreases, the frequency increases and the depth of defect, which can be detected, decreases. Also short duration impacts are needed to detect defects close to the surface.

11.3.2. Equipment for impact-echo testing

Examples of the equipment used for impact-echo testing are the systems developed by Impact-Echo Instruments as illustrated in Fig. 11.9. There are two systems offered.

Type A Test System comprising a Data Acquisition System, one cylindrical hand held transducer unit, 200 replacement lead disks for the transducer, Ten spherical impactors 3 mm to 19 mm in diameter (used to vary the contact time), one 3.7 m cable and one 7.6 m cable.

Type B Test System comprising a Data Acquisition System, two cylindrical hand-held transducer units, 200 replacement lead disks for the transducer, ten spherical impactors 3 mm to 19 mm in diameter, one 3.7 m cable, one 7.6 m cable and a spacer bar to use with the two transducers.

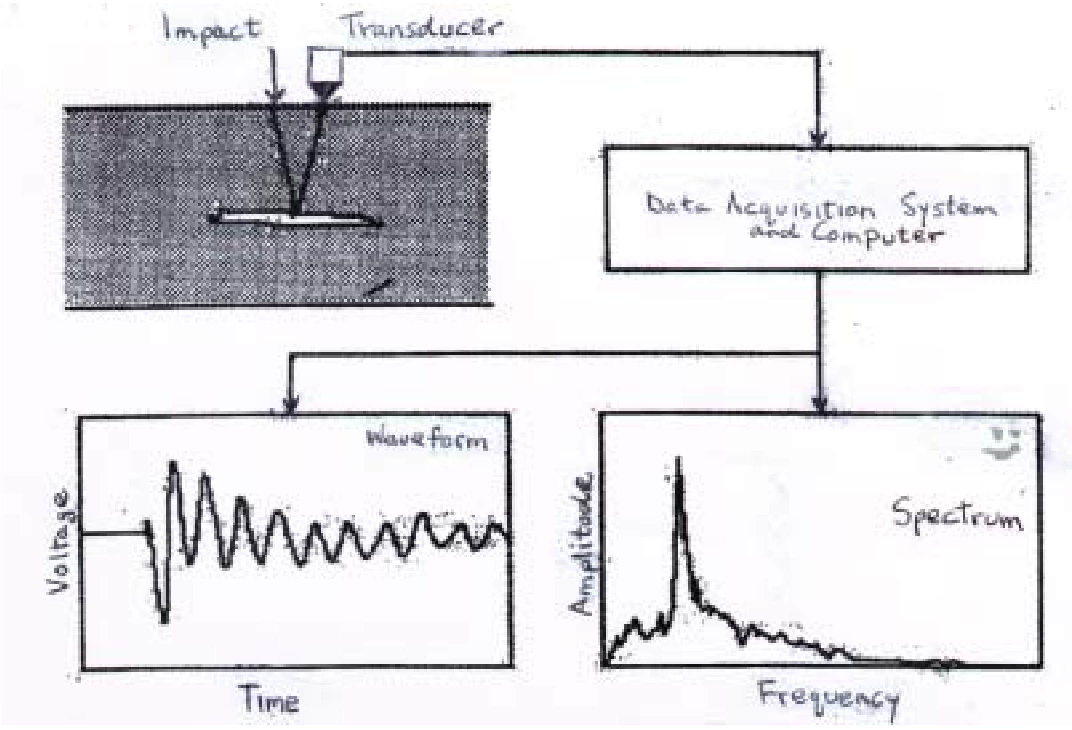


FIG. 11.9. Schematic diagram showing how impact-echo works.

11.3.3. General procedure for impact-echo testing

Using the Impact–Echo Instruments System A, the technique used is to vary the diameter of the impactor until a clear dominant frequency is obtained. Typically the diameter of the impactor has to increase as the thickness of the material being tested increases to obtain reflections from the rear surface of the material being tested.

11.3.4. Applications of and examples of the use of the impact-echo testing method

The investigation of cracking in the deck of a reinforced concrete railway bridge due to alkali-aggregate reaction resulted in horizontal cracking being detected at mid depth over the entire span. The cracking was verified by taking cores. The bridge was subsequently demolished.

Another use has been in measuring the thickness of concrete pavements. The accuracy of the thickness measurement was found to vary depending on the sub base on which concrete is laid. For example the uncertainty of the thickness measurement was within 1% for a concrete pavement on lean concrete sub-base, 2% for pavement on an asphalt sub-base and 3% for pavement on an aggregate sub-base.

Voids have also been located in grouted tendon ducts of a post-tensioned highway bridge. Areas of full or partial voids were found in 3 of 14 girders tested.

Delaminations have also been found in 200 mm thick concrete bridge deck with a 100 mm asphalt overlay. Extensive areas of delamination were detected at the top layer of the reinforcing steel. The delamination was confirmed by taking cores. The deck was subsequently repaired and a new asphalt overlay laid.

Cracking has also been detected in the beams and columns of parking garage. Cracks were identified at flange to web intersections at certain T beam configurations and the extent of cracking was determined in columns.

11.3.5. Range and limitations of impact-echo testing method

In generic terms the impact-echo method is a commercial development of the well-known frequency response function method (Frf) and the theory of vibration testing of piles. Further reading may be obtained in Ewens(1984) and Davis and Dunn (1974).

The user should beware of the claimed accuracy of detecting defects or thickness in terms of an absolute measurement. It is better to think in terms of a multiple of the wavelength:

$$\text{Velocity} = \text{frequency} \times \text{wavelength} \tag{28}$$

$$V = f\lambda$$

where

λ is wavelength.

For impact test work, recent research has shown that the “near field” detection capability of impact-echo (Martin, Hardy, Usmani and Forde, 1998) is:

$$\text{minimum depth of detectable target} = \lambda/2$$

Many test houses will deliberately or otherwise use the null hypothesis:

“If a defect is not identified – then none exists.”

In order to determine λ, one could assume the velocity through the good concrete to be:

$$\text{Velocity} = 4{,}000 \text{ m/s}$$

(Poorer or younger (<28 days) concrete might have a velocity equal to 3,500 m/s), thus:

$$\lambda = 4000/\text{frequency metres}$$

When using impact-echo equipment, one would select the excitation frequency by turning a dial in order that the appropriate size of spherical hammer is chosen. For example, if a 10 KHz excitation frequency hammer is chosen, the near field minimum depth resolution would be

$$\lambda/2 = 4000/10\text{KHz} \times 2 = 4/20 = 0.2 \text{ metres}$$

It is argued by Sansalone, et al. that when one cannot detect the shallow "target", the "anomaly" can be detected by observing the apparent depth to the base of a slab or depth to a backwall. This depth will appear to increase when a defect occurs. This method of interpretation must be used with some caution.

A check needs to be undertaken on actual impact frequency achieved as the surface of the concrete may crumble. If the surface crumbles, even a little, on impact:

— contact time increases
— lower frequency of excitation is achieved
— longer wavelength signal is generated
— lower "near field" resolution is achieved.

Good practice would be to take multiple impact-echo readings and discard the first two readings. This assumes that the third and subsequent readings are good.

The size of the test object plays an important role in the results obtained. Geometrical effects due to limited size are the cause of signals, which can be misleading. It is therefore necessary to perform the impact-echo test at several points on the surface to identify possible geometrical effects.

11.4. RELATIVE AMPLITUDE METHOD

11.4.1. Fundamental principles

The ultrasonic pulse velocity method is the most widely used ultrasonic non-destructive method for assessing concrete quality. It has been used to estimate concrete strength for over 40 years. However, the relation between strength and pulse velocity is in general not reliable enough for practical purpose. The relative amplitude method is alternative method for estimating strength. The relative amplitude method is basically an attenuation method, which measures the ratio of the wave amplitudes.

Ultrasonic waves are attenuated as they pass through the materials. Basically attenuation is caused by beam divergence (distance effect), absorption (heat dissipation) and scattering. Only scattering is affected by the characteristics of the materials they pass through. The degree of inhomogeneity and frequency of the transducer affect it. Attenuation caused by scattering is given by:

$$\alpha_s \quad \propto \quad \begin{cases} 1/D & \text{for diffusion range} \quad \lambda \le D \\ Df^2 & \text{for stochastic range} \quad \lambda \approx D \\ D^3 f^4 & \text{for Rayleigh range} \quad \lambda >> D \end{cases} \tag{29}$$

where

f is the wave frequency,
λ is the wavelength,
D is the average inhomogeneity; in concrete, D may be the void or aggregate size.

The main factor that influences strength of brittle material is porosity. Several models to relate strength to porosity have been proposed, but the most common one is the exponential model:

$$K \quad = \quad K_o \mathrm{e}^{-kP} \tag{30}$$

where

K_o is the strength at zero porosity,
P is the fractional porosity,
k a constant that depends on the system being studied.

For λ >> D, it is found that the concrete strength is related exponentially with the wave attenuation.

11.4.2. Equipment for relative amplitude method

The equipment consists of, as a minimum, a conventional ultrasonic pulse velocity meter (e.g. PUNDIT or James V-meter) and a two channel oscilloscope. Transducers of 50 mm diameter and 54 kHz may be used for generating and receiving ultrasonic waves through samples.

11.4.3. General procedure for relative amplitude method

The best technique is by direct transmission and the semi-direct technique is also possible. However, the surface technique is not possible since the amplitude of the pressure and the torsion waves are difficult to determine.

PROCEDURE:

— Adjust the test range on the oscilloscope (µs/div.) so that the received signal is on the CRT. The time base range should cover at least two times the pressure wave arrival time.

— Measure the amplitude of the pressure wave (i.e. the first amplitude), say A_p.

— Measure the amplitude of the combination of pressure and torsional wave, say A_{ps}. This amplitude should be just after the arrival of the torsional wave. It is about 2 times the pressure wave arrival, Fig. 11.6.

— Measure the attenuation of the ultrasonic waves.

The so-called relative amplitude β is given by:

$$\beta \quad = \quad 20 \quad \log\left(\frac{A_{ps}}{A_p}\right) \qquad (31)$$

where

A_p is the amplitude of pressure wave
A_{ps} is the wave amplitude after torsion wave arrival.

Fig. 11.10 shows how A_p and A_{ps} are determined from the oscilloscope display.

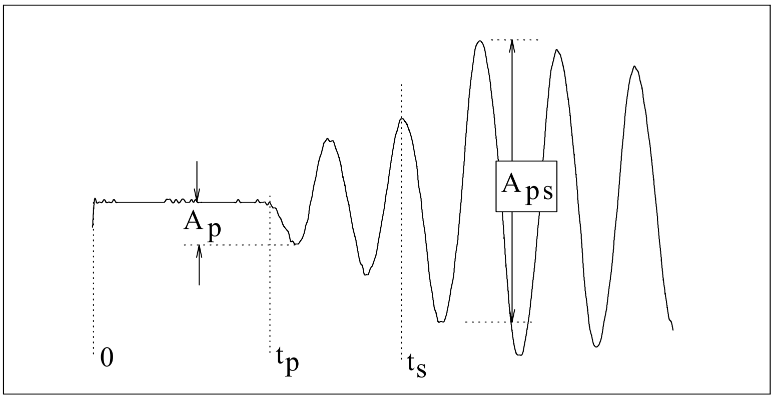


FIG. 11.10. Typical oscilloscope display defining the transit time and wave amplitude.

A_p is the pressure wave amplitude, A_{ps} the amplitude after the arrival of the torsional wave, t_p the transit time of the pressure wave and t_s the transit time of the torsional wave.

11.4.4. Applications of relative amplitude method

The sole application of the method is in strength estimation. The general relationship between strength, K and relative amplitude, β is an exponential form. For a specific condition, the empirical relationship is, as follows:

$$K \quad = \quad e^{5.2115-0.1444\beta} \qquad (32)$$

Above equation is applicable to concrete with a moisture content of 3-4%, an age of 90 days, made from crushed granite aggregate with a maximum size of 20 mm, cured by immersion in water for 28 days, and measured by the direct technique at 150 mm beam path distance without reinforcement bars. The relative amplitude decreases as the strength is increased. The factors, which influence the relationship, are shown in Figure 11.11 and can be described, as follows:

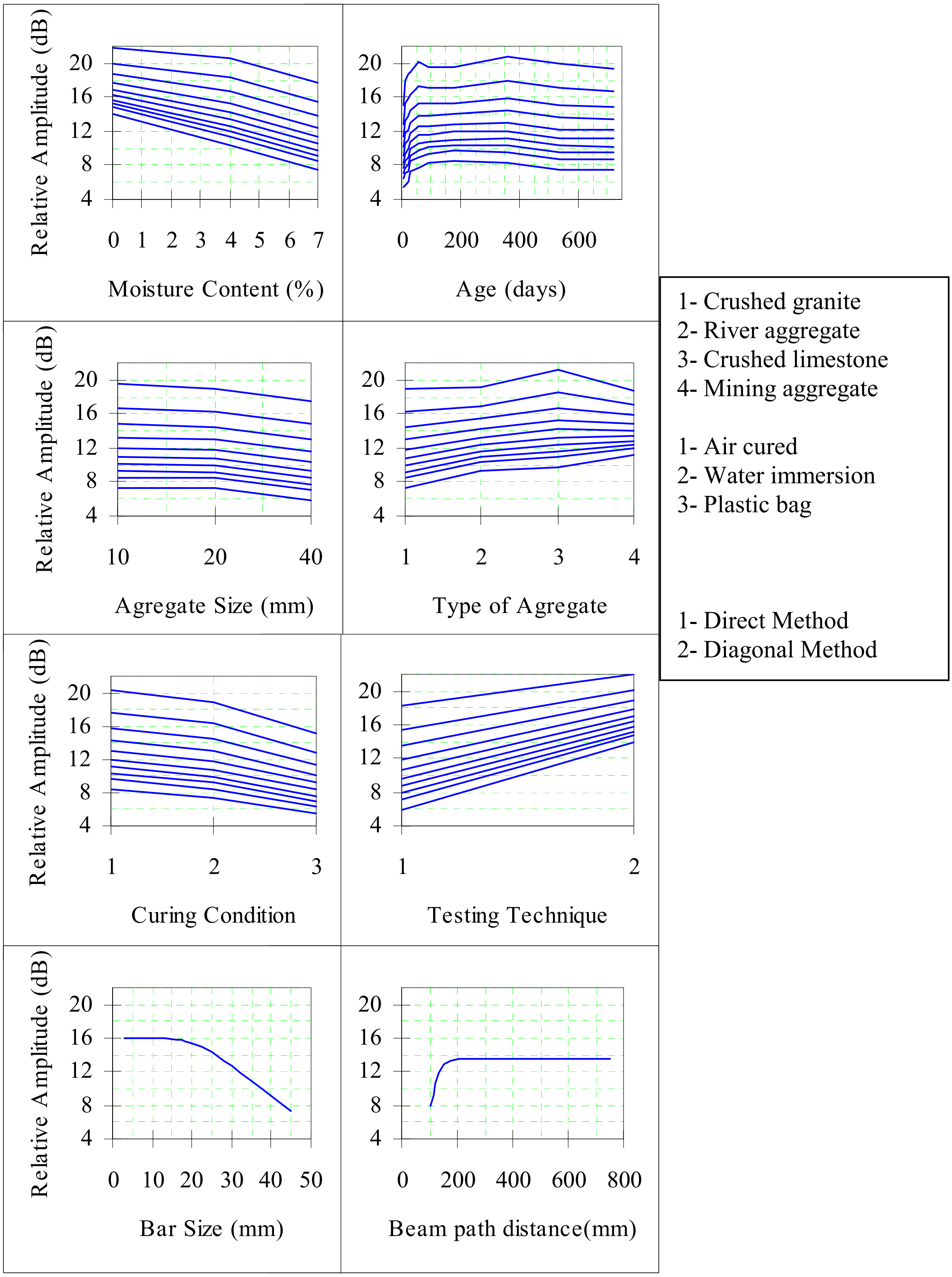


FIG. 11.11. Corrections for β.

MOISTURE CONTENT

Wet samples have lower β than dry samples for the same strength. β varies by ~2 dB between normal wet and dry samples but it increases by ≥3 dB if the sample is oven dried. The strength of the oven dried sample increases by 15% as compared to the wet sample and β increases by 50%. Observing β, the drier the sample is, the poorer the quality. The reverse effect is shown on the strength.

CONCRETE AGE

It can observed that β is constant for samples of age 90 days and above, but for samples of age 7 to 90 days, β varies by 4~5 dB.

AGGREGATE TYPE AND SIZE

Granite aggregate gives the highest β as compared to the other types of aggregate for the same strength. There is a small change in β for aggregate 10 mm and 20mm, but β reduces by ~2 dB with aggregate of 40 mm.

CURING CONDITION

Concrete cured in air (top surface of the cube not covered) gives the highest β as compared to the cubes immersed in water or covered with plastic bag. The variation is higher for low strength than the high strength concrete.

TESTING TECHNIQUE

Surface technique is not possible for the relative amplitude method. This is due to difficulties in locating the position of the torsion wave arrival on the waveform and might be confused with the surface wave. Similar to the pulse velocity method, the most sensitive technique of the transducer arrangement is also by direct technique. Slight misalignment between the transmitter and the receiver (as long as it does not exceed 20^o) does not significantly change β. With the diagonal technique, the angle between transmitter and receiver is 45^o.

BEAM PATH DISTANCE AND REINFORCEMENT BAR SIZE

There is no significant change in the relative amplitude for the beam path distance ≥150 mm and the bar size of ≤20 mm. The curve for the relative amplitude versus the beam path length and bar size is plotted only for one strength since it is expected that it will be the same slope for other strengths as they are not properties of concrete.

11.4.5. Range and limitations of relative amplitude method

Determination of the amplitude of A_{ps} may be difficult in some concrete. Surface technique is not possible. For a specific condition the error of estimation is ~4 N/mm^2 at 95% confidence level and the correlation coefficient is about 0.8 ~ 0.95. Without knowing the history of the sample the estimation of strength is subjected to large error. The main advantage of the relative amplitude over pulse velocity method is that the relative amplitude do not require parameter distance. Low value of β indicates high quality of concrete.

11.5. VELOCITY VERSUS REBOUND NUMBER CURVES

11.5.1. Introduction

For some investigations it is convenient to use both the ultrasonic pulse velocity test and the rebound (Schmidt) hammer test in association with the concrete compressive strength to establish the concrete quality. If the ingredients of the concrete mix and their proportions are known (e.g. cement, aggregate and admixtures, a relationship can be established between the compressive strength of the concrete, rebound number and ultrasonic pulse velocity. The SONREB method developed largely by the RILEM Technical Committees 7 NDT and 43 CND can be represented by the nomogram in Fig. 11.12.

Using a series of correction coefficients developed for the specific concrete grade and type being investigated and, knowing the pulse velocity and the rebound number, a more accurate prediction can be made of the compressive strength of concrete. Facaoaru has used the following coefficients:

Cc - coefficient of influence of cement type
Cd - coefficient of influence of cement content
Ca - coefficient of influence of petrological aggregate type
Cg - coefficient of influence of aggregate fine fraction (less than 0.1 mm)
Co - coefficient of influence of maximum size of aggregate.

The accuracy of the estimated strength (the range comprising 90% of all the results) is considered to be:

- 10% to 14% when the correlation relationship is developed with known strength values of cast specimens or cores and when the composition is known
- 15% to 20% when only the composition is known.

11.5.2. Procedure for drawing velocity-rebound number curves

In an ordinary concrete, between 60% and 70% of the absolute volume is taken up by aggregate and the rest by cement paste, consisting of hydrated and unhydrated cement grains, chemically bound and free water, and entrained (small voids) or entrapped (larger voids) air. Subject to availability in a particular country or region, part of the cementitious material may be ground granulated blast furnace slag, fly ash, silica fume, or some other pozzolanic or reactive siliceous material. The paste may also contain chemical admixtures. The strength characteristics of a given cement paste, subjected to the influence of a particular environment, will be a function of time and the strength characteristics of a given aggregate, for all practical purposes, can be considered time independent and a function of its petrological type only.

Thus, even in concrete of suspect quality and unknown composition, there are two variables which can be identified with a reasonable degree of accuracy, namely, petrological type of the aggregate and approximate age of the concrete. Removing some of the matrix in an out-of-sight part of a structural member can identify the aggregate. Coarse aggregate is particularly easy to identify in this way and, in the majority of commercial grade concrete, coarse aggregate content is significantly higher than the fine aggregate content.

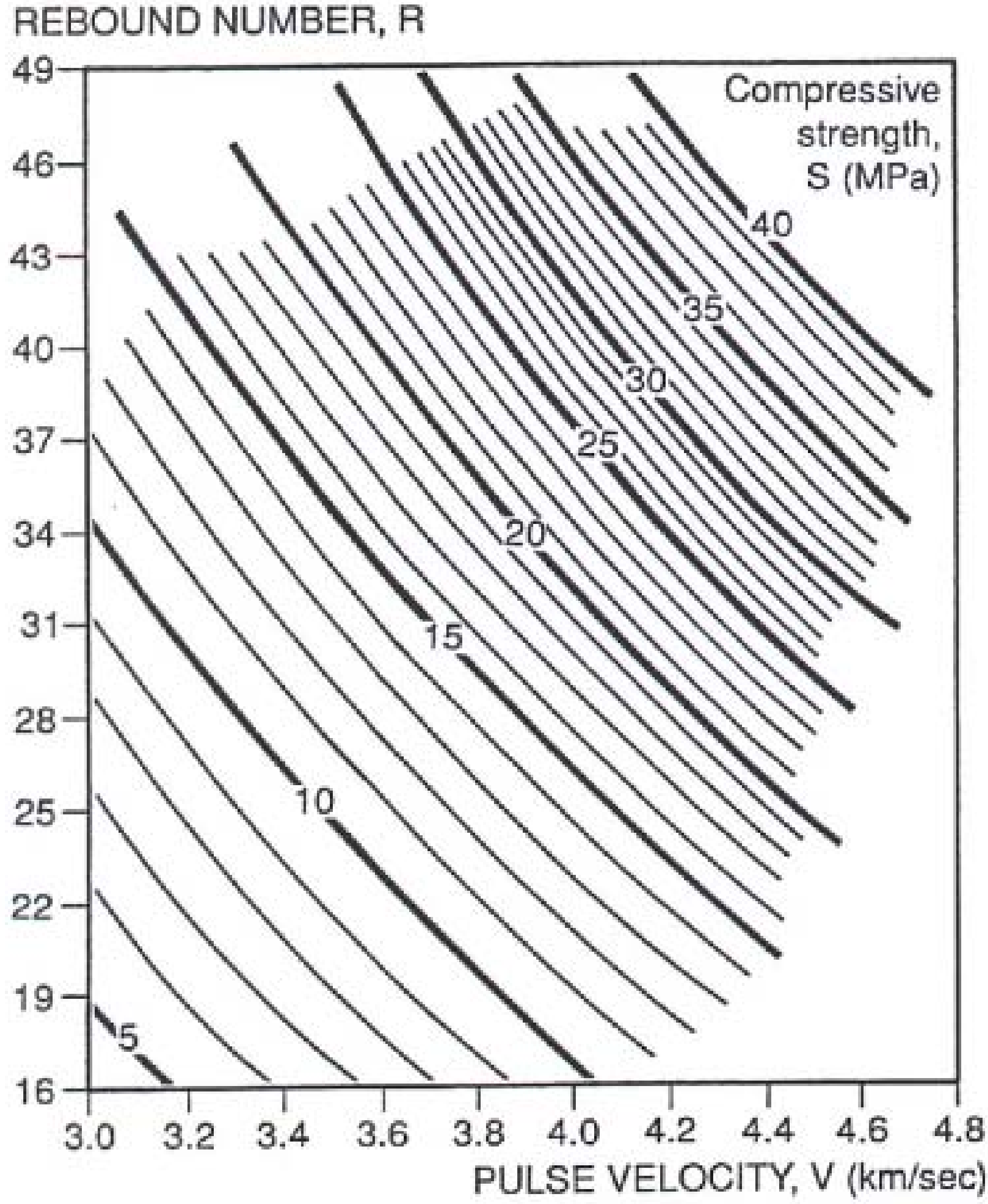


FIG. 11.12. ISO-strength curves for reference concrete in SONREB method.

Establishment of a series of specific correlations between the combination of rebound hammer number (R) and ultrasonic pulse velocity (V) and the compressive strength (S) of concretes, each containing a particular aggregate type and being of a particular age group, was completed in the early 1970s by Samarin and Smorchevsky. Because the transit time of an ultrasonic pulse through concrete consists of the sum of transit times through aggregate and paste, the identification of aggregate type and time dependent properties of cement paste eliminates two major uncontrollable variables of the general correlation. The accuracy of the estimated compressive strength can thus be measurably improved. Yet another factor, which can improve the accuracy of prediction, particularly over a wide range of concrete strength levels, is the provision for non-linearity of some functions.

Work by Samarin has shown that, for Australian concretes, the relationship between rebound hammer number (R) and the compressive strength (S) is nearly linear. A curve fitting analysis indicated that a fourth order function gives the best correlation between ultrasonic pulse velocity (V) and the compressive strength (S) for the same concrete. Thus the general equation for the rebound hammer correlation relationship is

$$S = a_0 + a_1 R \tag{32}$$

where

R is the rebound hammer number,
S is the compressive strength,

and a_o and a_1 are constants.

The general equation for the pulse velocity correlation relationship is of the form:

$$S = b_0 + b_1 V^4 \tag{33}$$

where

S is the compressive strength,
V is the ultrasonic pulse velocity,

and b_0 and b_1 are constants.

It is worth mentioning that the universally accepted empirical relationship between the elastic modulus of concrete and the compressive strength of concrete is of the following general form:

$$E = AS^{0.5} \tag{34}$$

where

E is the elastic modulus of concrete,
S is the compressive strength of concrete,

and A is a constant, depending on concrete density, statistical evaluation of strength and the selected system of measures. At the same time, the theory of propagation of stress waves through an elastic medium states that for a compression wave the following functional relationship is valid:

$$E = BV^2 \tag{35}$$

where

E is the elastic modulus of concrete,
V is the ultrasonic pulse velocity,

and B is a constant, depending on density and Poisson's ratio.

In the work of Samarin and Meynink, it was considered convenient to divide concrete into three age groups, namely:

— 7 days and younger
— over 7 days, but less than 3 months
— 3 months and older.

It is known, for example, as reported by Elvery and Ibrahim, that the sensitivity of ultrasonic pulse velocity to concrete strength is very high in the first few days, but after about 5 to 7 days (depending on curing conditions) the results become considerably less reliable. Most of the concrete, which is identified as being suspect, is subsequently tested *in situ* at the age of between 1 and 3 months. The majority of the laboratory test data for which the correlations have been developed also fall into this period.

To develop a correlation relationship for each age group and aggregate type, concretes having a wide range of strength grades and mix composition are cast into standard laboratory (cylindrical) specimens. Compaction, curing, capping, etc. are carried out strictly according to the requirements of the relevant standards. Just prior to a compression strength test, each specimen is placed in a horizontal rig and the transit time of an ultrasonic pulse through the length of a capped cylinder is recorded. The pulse velocity is calculated as the ratio of the length over the transit time. The specimen is then placed in a compression machine and a load of approximately 1.4 MPa (200 psi) is applied, while 15 rebound hammer readings are taken around the circumference of the cylinder. A similar technique can be used with a cube or a prismatic specimen. The specimen is then tested in unconfined compression, using a test method, which complies with the relevant standard requirements.

When the multiple correlation relationship for each aggregate type and each age group is developed, the results, as compared with correlations between compressive strength and ultrasonic pulse velocity alone, or between compressive strength and rebound hammer reading alone, indicate:

(1) An increase in the multiple correlation coefficient above the correlation coefficient for rebound number alone for pulse velocity alone (Note that correlation coefficients for rebound number and strength are generally higher (better) than those for pulse velocity and strength).
(2) A decrease in the standard error of estimate for a multiple correlation relationship compared with relationships between rebound number and strength alone and between pulse velocity and strength alone.

The degree of improvement due to the combined technique depends on a number of factors. Of these, the most significant (in the order of importance) appear to be:

— Grouping concrete for a particular multiple regression analysis according to the petrological type of the coarse aggregate.

— Use of least-squares curve fitting to establish the correct form of the relation between concrete strength and each independent variable separately. (For Australian concretes, the correlation between compressive strength and rebound hardness is very near linear, and pulse velocity has to be raised to the fourth power in order to produce near optimum curve-fitting in its functional relationship to strength).

— In establishing the multiple correlation relationship, a reasonably wide range of strength grades of concrete (say, from 20 to 50 Mpa) — all manufactured using identical coarse aggregate — should be used.

A typical multiple correlation relationship for Australian concrete in the form of a nomogram is shown in Fig. 11.13.

11.5.3. Accuracy of measurement of concrete properties using velocity rebound number curves

A variety of non-linear and linear multiple correlation equations have been considered by different research workers and the results are presented in Table 11.4.

Factors that affect reliability, sensitivity and reproducibility of the results obtained by the combined technique are:

- increase in the moisture content increases the pulse velocity but reduces the rebound hammer number (Bellander)
- both the pulse velocity and compression strength are reduced by the effect of fire but not by the same amount (Chung and Law)
- presence of reinforcing steel affects the pulse velocity (Chung)
- in concrete with superplasticizer the estimate strength tends to be conservative, (Samarin and Thomas)
- rebound hammer readings are affected by finishing treatments and curing regimes, e.g. by abrasion resistant surface treatments, but not by liquid surface treatments.

TABLE 11.4. VARIOUS MULTIPLE REGRESSION CORRELATIONS SUGGESTED BY DIFFERENT RESEARCHERS TO ESTIMATE THE COMPRESSIVE STRENGTH OF CONCRETE

Researchers	Equation
McLeod	$S = k_0 + k_1 R + k_2 V$
Di Maio, et al	
Tanigawa,et al	
Knaze and Beno	$S = a_0 + a_1 R + a_2 R^2$
	$S = b_0 + b_1 V + b_2 V^2$
Bellander	$S = k_0 + k_1 R^3 + k_2 V$
Weibinga	$\log_e S = k_0 + k_1 R + k_2 V$
Shah	
Tanigawa	
Schickert	$S = k_0 R^n V^m$
Samarin, et al	$S = k_0 + k_1 R + k_2 V^4$
Tanigawa et al	$S = V\left(a_0 + a_1 R + a_2 R^2 + a_3 R^3\right)$

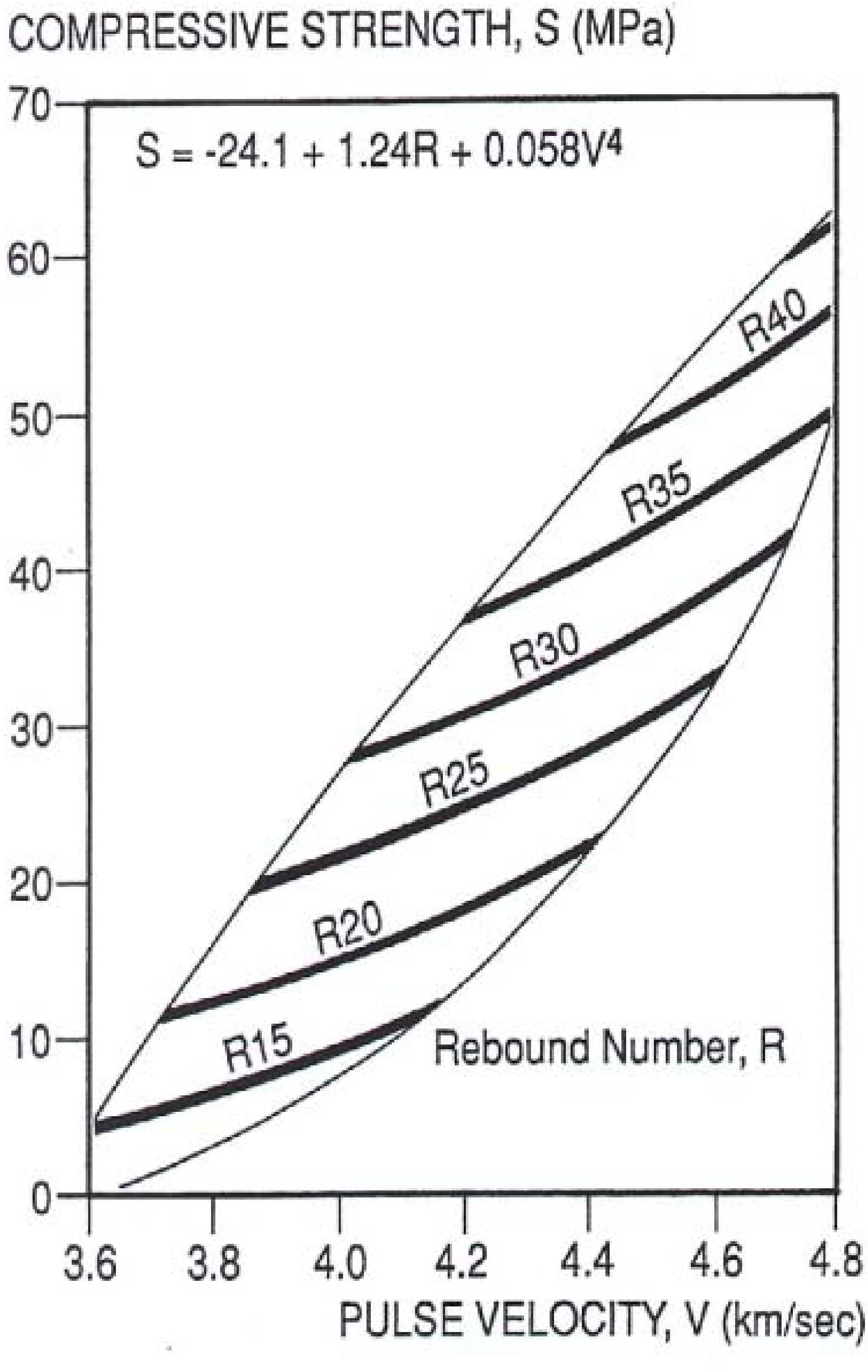


FIG. 11.13 Nomogram for concrete of a particular aggregate type and age.

12. INFRARED THERMOGRAPHY

12.1. FUNDAMENTAL PRINCIPLES

According to the fundamental Law of Planck all objects above absolute zero emit infrared radiation. This radiation only becomes visible to the human eye when the temperature is above about 500^{o}C. Infrared monitoring equipment has been developed which can detect infrared emission and visualize it as a visible image. The sensitive range of the detector lies between 2 and 14 microns. The 2-5.6 micron range is generally used to visualize temperature between 40^{o}C and 2000^{o}C and the 8-14 micron range is used for temperature between -20^{o}C and ambient temperatures.

The thermograms taken with an infrared camera measure the temperature distribution at the surface of the object at the time of the test. It is important to take into consideration that this temperature distribution is the result of a dynamic process. Taking a thermogram of this object at an earlier or later time may result in a very different temperature distribution. This is especially true when the object has been heated or cooled.

The detectability of any internal structure such as voids, delaminations or layer thicknesses depends on the physical properties (heat capacity, heat conductivity, density, emissivity) of the materials of the test object. Naturally any interior ’structure’ has an effect on the temperature distribution on the surface. If the temperature changes on the surface there is a delay before the effect of this change occurs below where a defect such as a void occurs. The longer the time delay before the temperature changes, the greater the depth of the defect below the surface. Generally anything deeper than 10 cm will only show after a long period of time (>1 hr) after the temperature change has occurred.

Since the infrared system measures surface temperatures only, the temperatures measured are influenced by three factors: (1) subsurface configuration, (2) surface condition; and (3) environment. As an NDT technique for inspecting concrete, the effect of the subsurface configuration is usually most interesting. All the information revealed by the infrared system relies on the principle that heat cannot be stopped from flowing from warmer to cooler areas, it can only be slowed down by the insulating effects of the material through which it is flowing. Various types of construction materials have different insulating abilities or thermal conductivities. In addition, differing types of concrete defects have different thermal conductivity values. For example, an air void has a lower thermal conductivity compared with the surrounding concrete. Hence the surface of a section of concrete containing an air void could be expected to have a slightly different temperature from a section of concrete without an air void.

There are three ways of transferring thermal energy from a warmer to a cooler region: (1) conduction; (2) convection; and (3) radiation. Sound concrete should have the least resistance to conduction of heat, and the convection effects should be negligible. The surface appearance, as revealed by the infrared system, should show a uniform temperature over the whole surface examined. However, poor quality concrete contains anomalies such as voids and low density areas which decrease the thermal conductivity of the concrete by reducing the energy conduction properties without substantially increasing the convection effects.

In order to have heat energy flow, there must be a heat source. Since concrete testing can involve large areas, the heat source should be both low cost and able to give the concrete surface an even distribution of heat. The sun fulfils both these requirements. Allowing the sun to warm the surface of the concrete areas under test will normally supply the required energy. During night-time hours, the process may be reversed with the warm ground acting as the heat source. For concrete areas not accessible to sunlight, an alternative is to use the heat storage ability of the earth to draw heat from the concrete under test. The important point is that in order to use infrared thermography, heat must be flowing through the concrete. It does not matter in which direction it flows.

The second important factor to consider when using infrared thermography to measure temperature differentials due to anomalies is the surface condition of the test area. The surface condition has a profound effect upon the ability of the surface to transfer energy by radiation. This ability of a material to radiate energy is measured by the emissivity of the material, which is defined as the ability of the material to radiate energy compared with a perfect blackbody radiator. A blackbody is a hypothetical radiation source, which radiates the maximum energy theoretically possible at a given temperature. The emissivity of a blackbody equals 1.0. The emissivity of a material is strictly a surface property. The emissivity value is higher for rough surfaces and lower for smooth surfaces. For example, rough concrete may have an emissivity of 0.95 while shiny metal may have an emissivity of only 0.05. In practical terms, this means that when using thermographic methods to scan large areas of concrete, the engineer must be aware of differing surface textures caused by such things as broom textured spots, rubber tire tracks, oil spots, or loose sand and dirt on the surface.

The final factor affecting temperature measurement of a concrete surface is the environmental system that surrounds that surface. Some of the factors that affect surface temperature measurements are:

SOLAR RADIATION: testing should be performed 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.

CLOUD COVER: clouds will reflect infrared radiation, thereby slowing the heat transfer process to the sky. Therefore, night-time testing should be performed during times of little or no cloud cover in order to allow the most efficient transfer of energy out of the concrete.

AMBIENT TEMPERATURE: This should have a negligible effect on the accuracy of the testing since one important consideration is the rapid heating or cooling of the concrete surface. This parameter will affect the length of time (i.e. the window) during which high contrast temperature measurements can be made. It is also important to consider if water is present. Testing while ground temperatures are less that $0^{o}C$ should be avoided since ice can form, thereby filling subsurface voids.

WIND SPEED: High gusts of wind have a definite cooling effect and reduce surface temperatures. Measurements should be taken at wind speeds of less than 15 mph (25 km/h).

SURFACE MOISTURE: Moisture tends to disperse the surface heat and mask the temperature differences and thus the subsurface anomalies. Tests should not be performed while the concrete surface is covered with standing water or snow.

Once the proper conditions are established for examination, a relatively large area should be selected for calibration purposes. This should encompass both good and bad concrete areas (i.e. areas with voids, delaminations, cracks, or powdery concrete). Each type of anomaly will display a unique temperature pattern depending on the conditions present. If, for example, the examination is performed at night, most anomalies will be between 0.1° and 5°C cooler than the surrounding solid concrete depending on configuration. A daylight survey will show reversed results, i.e. damaged areas will be warmer than the surrounding sound concrete.

12.2. EQUIPMENT FOR INFRARED THERMOGRAPHIC METHOD

In principle, in order to test concrete for subsurface anomalies, all that is really needed is a sensitive contact thermometer. However, even for a small test area, thousands of readings would have to be made simultaneously in order to outline the anomaly precisely. Since this is not practical, high resolution infrared thermographic cameras are used to inspect large areas of concrete efficiently and quickly. This type of equipment allows large areas to be covered and the resulting data can be displayed as pictures with areas of differing temperatures designated by differing grey tones in a black and white image or by various colours on a colour image. A wide variety of auxiliary equipment can be used to facilitate data recording and interpretation.

There are two types of infrared cameras available:

(1) Focal Plane Array (FPA) cameras – where there are a large number of active elements (256x256 or larger). Cooling is done by Stirling engines in a few minutes so the system can be used independent of liquid nitrogen supply. The newer cameras use newer sensor materials such as PtSi. Uncooled infrared cameras based on the bolometer principle are available with sensitive arrays but have not reached the sensitivity of the cooled detectors. For transient experiments frame rates (the number of frames taken per second) up to 60 Hz are standard, higher rates are available from special manufacturers. High quality data can be stored by writing direct digital storage up to 16 bit resolution, avoiding the degradation of data by digital/analogue conversion and storage of the images on video tapes in video format.

(2) Single active element scanner cameras where the image is mechanically scanned by a single detector.

A complete infrared camera and analysis system can be divided into four main subsystems. The first is the infrared camera, which normally can be used with interchangeable lenses. It is similar in appearance to a portable video camera. The camera's optical system is transparent either to short wave infrared radiation with wavelengths in the range of 3 to 5.6 μm or to medium wave infrared radiation with wavelengths in the range of 8 to 12 μm. Typically the infrared camera's highly sensitive detector is cooled by liquid nitrogen to a temperature of $-196^{o}C$ and can detect temperature variations as small as 0.1°C. Alternate methods of cooling the infrared detectors are available which use either compressed gases or electric cooling. These last two cooling methods may not give the same resolution since they cannot bring the detector temperatures as low as liquid nitrogen. In addition, compressed gas cylinders may present safety problems during storage or handling.

The second major component of the infrared scanning system is a real time microprocessor coupled to a display monitor. With this component, cooler items being scanned are normally represented by darker grey tones, while warmer areas are represented by lighter grey tones. In order to make the images easier to understand for those unfamiliar with interpreting grey-tone images they might be transferred into false colour images. This transformation assigns different colours (8.16.or 256) to the temperature range displayed. The colour palette used for this transformation can be created as one wishes. It is important to remember that the colour assigned to a temperature has no physical meaning.

The third major component of the infrared scanning system is the data acquisition and analysis equipment. It is composed of an analogue to digital converter, a computer with a high resolution colour monitor, and data storage and analysis software. The computer allows the transfer of instrumentation videotape or live images of infrared scenes to single frame computer images. The images can then be stored individually and later retrieved for enhancement and individual analysis. The use of the computer allows the engineer in-charge of testing to set specific analysis standards based upon destructive sample tests, such as cores, and apply them uniformly to the entire area of concrete. Standard, off-the-shelf image analysis programs may be used, or custom written software may be developed.

The fourth major component consists of various types of image recording and retrieving devices. These are used to record both visual and thermal images. They may be composed of instrumentation video tape recorders, still frame film cameras with both instant and 35 mm or larger formats, or computer printed images.

All of the above equipment may be carried into the field or parts of it may be left in the laboratory for additional use. If all of the equipment is transported to the field to allow simultaneous data acquisition and analysis, it is prudent to use an automotive van to set up and transport the equipment. This van should include power supplies for the equipment, either batteries and inverter, or a small gasoline driven generator. The van should also include a method to elevate the scanner head and accompanying video camera to allow scanning of the widest area possible depending on the system optics used.

Several manufacturers produce infrared thermographic equipment. Each manufacturer's equipment has its own strengths and weaknesses. These variations are in a constant state of change as each manufacturer alters and improves his equipment. Therefore, equipment comparisons should be made before purchase. Recently the three main manufacturers of thermography equipment — AGEMA, Inframetrics and FLIR — have merged.

12.3. GENERAL PROCEDURE FOR INFRARED THERMOGRAPHIC METHOD

In order to perform an infrared thermographic inspection, a temperature gradient and thus a flow of heat must be established in the structure. The first example deals with the simplest and most widespread situation. Assume that it is desired to test an open concrete bridge deck surface. The day preceding the inspection should be dry with plenty of sunshine. The inspection may begin two to three hours after either sunrise or sunset, both times being of rapid heat transfer.

The deck should be cleaned of all debris. Traffic control should be established to prevent accidents and to prevent traffic vehicles from stopping or standing on the pavement to be tested. It will be assumed that the infrared scanner be mounted on a mobile van along with other peripheral equipment, such as recorders for data storage and a computer for assistance in data analysis. The scanner head and either a regular film-type camera or a standard video camera should be aligned to view the same sections to be tested.

The next step is to locate a section of concrete deck and establish, by coring, that it is sound concrete. Scan the reference area and set the equipment controls so that an adequate temperature image is viewed and recorded.

Next, locate a section of concrete deck known to be defective by containing a void, delamination, or powdery material. Scan this reference area and again make sure that the equipment settings allow viewing of both the sound and defective reference areas in the same image with the widest contrast possible. These settings will normally produce a sensitivity scale such that full scale represents no more than 5°.

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.

As has been mentioned, when tests are performed during daylight hours, the defective concrete areas will appear warmer, while during tests performed after dark, defective areas will appear cooler.

Once the controls are set and traffic control is in place, the van may move forward as rapidly as images can be collected, normally 1 to 10 miles (1.6 to 16 km) per hour. If it is desired to mark the pavement, white or metallic paint may be used to outline the defective deck areas. At other times, a videotape may be used to document the defective areas, or a scale drawing may be drawn with reference to bridge deck reference points. Production rates of up to 130 m^2/day have been attained.

During long testing sessions, re-inspection of the reference areas should be performed approximately every 2 h, with more calibration retests scheduled during the early and later periods of the session when the testing “window” may be opening or closing.

For inside areas where the sun cannot be used for its heating effect, it may be possible to use the same techniques except for using the ground as a heat sink. The equipment should be set up in a similar fashion as that described above, except that the infrared scanner's sensitivity will have to be increased. This may be accomplished by setting the full scale so it represents 2°C and/or using computer enhancement techniques to bring out detail and to improve image contrast.

Once data are collected and analysed, the results should be plotted on scale drawings of the area inspected. Defective areas should be clearly marked so that any trend can be observed.

Computer enhancements can have varying effects on the accuracy and efficiency of the inspection systems. Image contrast enhancements can improve the accuracy of the analysis by bringing out fine details, while automatic plotting and area analysis software can improve the efficiency in preparing the finished report.

A WORD OF CAUTION: when inspecting areas where shadows occur, such as bridges with superstructures or pavements near buildings, it is preferable to perform the inspection after sunset since during daylight hours the shadows move and can result in confusing test results.

12.4. SOME APPLICATIONS OF THE INFRARED THERMOGRAPHIC METHOD

In order to illustrate some different applications of infrared thermographic testing, some applications are reviewed, namely: (1) bridge deck concrete (2) airport taxiway concrete (3) garage deck concrete (4) defective cladding on buildings (5) water ingress through flat roofs or external wall systems (6) energy loss in buildings.

(1) Bridge deck concrete

An example of this application occurred with an inspection of the bridge deck on the Dr. Martin Luther King Bridge, St. Louis. Because the weather conditions in St. Louis can change quickly, going from clear sky to fog and rain in a matter of minutes, it was decided to perform the field inspection of the bridge deck within an eight hour period or less. The infrared equipment was mounted on a mobile lift platform, which could travel up to ¾ miles per hour (about 1.2 km/h) to allow sufficient height to view all four lanes of the bridge in a single pass. The data, both infrared and visual, were recorded on both instrumentation videotape and 35 mm film formats. Due to the traffic restrictions, the bridge could not be fully closed during inspection. Therefore, the survey was performed on a weekend night between 10:00 p.m. and

2:00 a.m. Before, during, and after the inspection, reference areas were scanned to determine equipment settings, which would give the greatest contrast on the infrared imager. The main sensitivity of the equipment was set at 5° for full scale.

After inspection, a simple technique was used to confirm the infrared data and interpretations. Three separate areas were chosen from the void, delamination, and anomaly drawings developed from the infrared data. Then an 8 in. (200 mm) nail was driven into the pavement and its penetration was determined under a standard blow delivered by the supervising engineer. The locations where the nail penetrated deeply correlated exactly with the locations of anomalies found using the infrared scanner.

(2) Airport taxiway concrete

Over 3,125 slabs of reinforced concrete on the taxiway of one of the busiest airports in America, Lambert St. Louis International Airport, were inspected over a total of seven nights, during two of which no surveying could be done because of rain. Because air traffic flow could not be interrupted on the taxiways, inspection was performed from 11:00 p.m. to 5:00 a.m. when traffic was at it lowest. In order to move the infrared equipment rapidly and move out of the path of approaching planes quickly, a van was used to carry all the infrared testing equipment along with associated surveying tools, such as power supplies, drawing equipment, and various recording devices. The van was custom designed to allow the scanner head and visual camera to be raised to a 4.8-m height during scanning runs to allow the surveying of an 8.0-m wide × 8.0-m long slab in a single view. Production rates, which included the scanning operation, storage of images on computer discs and videotape, occasional 35-mm photographs and all analyses, allowed the inspection of up to 50000 m^2/night.

Before beginning the inspection, reference and calibration areas of both sound concrete and concrete with subsurface voids and delaminations were found. These areas were rescanned at regular intervals during the inspection to ensure that equipment settings allowed for accurate data collection. The scanning information was fed continuously into a computer and a colour monitor was used to assist in the location of anomalies. To speed up data interpretation, thermal data presented on the monitor were divided into three categories represented by three colours: green for solid concrete, yellow for concrete areas with minor temperature deviations most likely caused by minor surface deterioration, and red for concrete areas with serious subsurface cracks/voids. A computer was also used to determine the area on each slab that appeared in the above colours. These data were used to designate each individual slab for no corrective action, spot repairs, or major replacement.

(3) Garage deck concrete

An inspection was undertaken of the garage concrete and adjacent roadway concrete at the Lambert St. Louis International Airport. The same techniques as described above were used, but particular attention was paid to the expansion joint areas between concrete slabs. The deteriorated areas were confirmed and rehabilitated the following year.

(4) Defective cladding on buildings

Suspected defective cladding on buildings can be an expensive problem to resolve by conventional methods. These methods often involve erecting scaffolding to provide access to the whole face of the building or a gondola suspended from the roof for high rise buildings. The test technique used is to tap the tiles with a small hammer and correlating well bonded tiles against poorly bonded tiles by the sound produced. Suspect tiles can be marked for later removal and replacement. A cheaper option, at least for a preliminary appraisal of the building, is to use infrared thermography to scan the whole face of the building either from a building opposite or from another vantage point. An infrared photograph can be produced showing areas of temperature difference. There are two possible periods when testing can be carried out – either during the heating up or the cooling down of the building surface. Of the two the cooling cycle is usually preferred. This means that testing is done 2-3 h after sunset. The areas with defective tiles are shown as being cooler than the areas with tiles attached correctly.

(5) Water ingress through flat roofs or external wall systems

Thermography has been used to reveal the point where water is entering a building either through the flat roof or an external wall. The reasons for water entry are usually:

- badly made and fitted fittings and frames
- badly fitted air conditioner or extractor fan installations
- defects in water proof cladding
- voids, honeycombing, cracks, etc. in the concrete.

A technique that can be used to determine the point of water entry in a flat roof involves: (a) removal of all protective tiling and bedding compounds from the roof, (b) cleaning the waterproofing membrane of all debris, and (c) soaking the membrane with water for 24 h. The roof area is then drained, dried and allowed to absorb solar heat. When the roof is inspected at night the water entry areas are revealed as being warmer than the dry areas. Generating a temperature differential between inside and outside the building checks water entry in a window area. The window area is then scanned from the outside to locate the point of air leakage. The assumption is that the air leakage point will be the point of water entry

(6) Energy loss in buildings

The same technique is used to check where heat energy or cooling energy is being lost in a building.

12.5. ADVANTAGES AND LIMITATIONS OF INFRARED THERMOGRAPHY

Thermographic testing techniques for determining concrete subsurface voids, delaminations, and other anomalies have advantages over destructive tests like coring and other NDT techniques such as radioactive/nuclear, electrical/magnetic, acoustic and radar techniques.

The obvious advantage of infrared thermographic analysis over the destructive testing methods is that major concrete areas need not be destroyed during testing. Only small calibration corings are used. This results in major savings in time, labour, equipment, traffic control, and scheduling problems. In addition, when aesthetics is important, no disfiguring occurs on the concrete to be tested. Rapid set up and take down are also advantages when vandalism is possible. Finally, no concrete dust and debris are generated that could cause environmental problems.

Other advantages are that infrared thermographic equipment is safe as it emits no radiation. It only records thermal radiation, which is naturally emitted from the concrete, as well as from all other objects. It is similar in function to an ordinary thermometer, only much more efficient.

The final and main advantage of infrared thermography is that it is an area testing technique, while the other NDT methods are mostly either point or line testing methods. Thus, infrared thermography is capable of forming a two dimensional image of the test surface showing the extent of subsurface anomalies.

The other methods including radioactive/nuclear, electrical/magnetic, acoustic, and radar are all point tests. They depend upon a signal propagating downward through the concrete at a discrete point. This gives an indication of the concrete condition at that point. If an area is to be tested, then multiple readings must be taken.

Radar however has the advantage over the other point testing techniques in that the sensor may be mounted on a vehicle and moved in a straight line over the test area. This improves efficiency somewhat, but if an area is wide, many line passes have to be made.

There is one major disadvantage to infrared thermographic testing. At this stage of development, the depth or thickness of a void cannot be determined, although its outer dimensions are evident. It cannot be determined if a subsurface void is near the surface or farther down at the level of the reinforcing bars. Techniques such as radar or stress wave propagation methods can determine the depth of the void, but again these methods cannot determine the other dimensions in a single measurement.

In most testing instances, the thickness of the anomaly is not nearly as important as its other dimensions. But in those instances where information on a specific anomaly thickness or depth is needed, it is recommended that infrared thermography be used to survey the large areas for problems. Once specific problem locations are established, radar can be used to spot check the anomaly for its depth and thickness. This combined technique would give the best combination of accuracy, efficiency, economy, and safety.

13. GROUND PENETRATING RADAR

13.1. FUNDAMENTAL PRINCIPLE

Ground penetrating radar (GPR) is a non-destructive technique with a wide range of potential applications in the testing of concrete. It is gaining acceptance as a useful and rapid technique for non-destructive detection of delaminations and the types of defects, which can occur in bare or overlaid reinforced concrete decks. It also shows potential for other applications such as measurement of the thickness of concrete members and void detection. The advantages and limitations of GPR in these applications are also discussed. GPR has been put to a variety of uses, such as:

- determining the thickness and structure of glaciers
- archeological investigations
- the location of buried objects eg bodies
- locating ice in permafrost
- finding sewer lines and buried cables
- measuring the thickness of sea ice
- profiling the bottom of lakes and rivers
- examining the subsurface of the moon
- detecting buried containerized hazardous waste
- measuring scouring around bridge foundations.

Because of the nature of the microwave pulses that are employed by the radar systems used and since the applications are no longer limited to the probing of subsurface geological features, GPR is often called impulse radar or just radar, particularly when applied to inspection of concrete structures.

GPR is the electromagnetic analogue of sonic and ultrasonic pulse echo methods. It is based on the propagation of electromagnetic energy through materials of different dielectric constants. The greater the difference between dielectric constants at an interface between two materials, the greater the amount of electromagnetic energy reflected at the interface.

The smaller the difference the smaller the amount reflected and conversely the more energy which continues to propagate in the second material. In this sense dielectric constant difference in the propagation of electromagnetic energy is analogous to impedance difference in the propagation of sonic and ultrasonic energy.

13.1.1. Behaviour of a microwave beam at the interface of two different materials

Consider the behaviour of a beam of eletromagnetic (EM) energy as it strikes an interface, or boundary, between two materials of different dielectric constants, Fig. 13.1. A portion of the energy is reflected, and the remainder penetrates through the interface into the second material. The intensity of the reflected energy, AR, is related to the intensity of the incident energy, AI, by the following relationship:

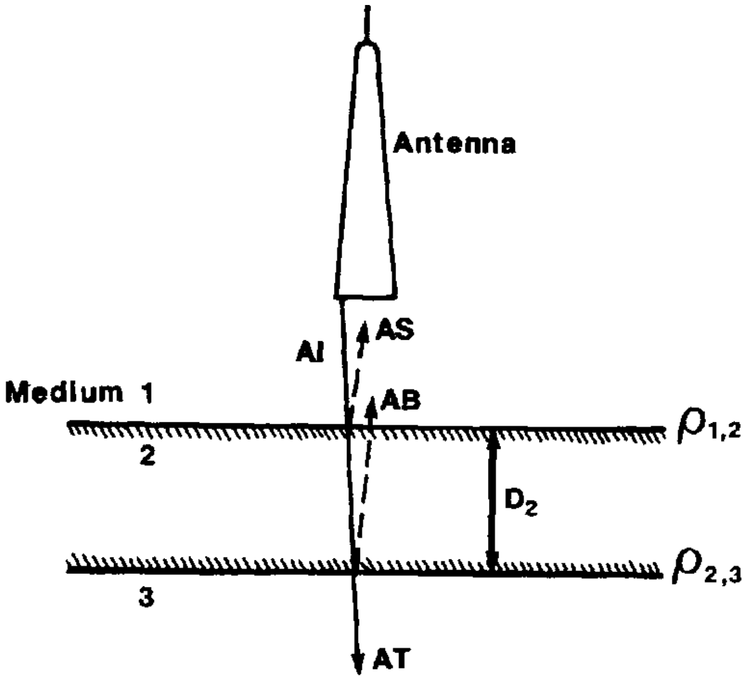


FIG. 13.1. Propagation of EM energy through dielectric boundaries.

$$\mu_{1,2} = \frac{AR}{AI} = \frac{\eta_2 - \eta_1}{\eta_2 + \eta_1} \tag{36}$$

where

$\mu_{1,2}$ is the reflection coefficient at the interface,

η_1/η_2 is the wave impedance of materials 1 and 2, respectively, in ohms.

For any non-metallic material, such as concrete or soil, the wave impedance is given by

$$\eta = \sqrt{\frac{\mu_0}{\varepsilon}} \tag{37}$$

where

μ_0 is the magnetic permeability of air, which is $4\pi \times 10^{-7}$ henry/meter,

ε is the dielectric constant of the material in farad/meter.

Metals are perfect reflectors of EM waves, because the wave impedance for any metal is zero. Since the wave impedance of air, η_0 is

$$\eta_0 = \sqrt{\frac{\mu_0}{\varepsilon_0}} \tag{38}$$

And, if we define the relative dielectric constant, ε_r, of a material as

$$\varepsilon_r = \frac{\varepsilon}{\varepsilon_0} \tag{39}$$

where

ε_0 is the dielectric constant of free space, which is 8.85 × 10^{-12} Farad/meter. Then, we may rewrite Equation 37 as

$$\eta = \frac{\eta_0}{\sqrt{\varepsilon_r}} \tag{40}$$

and Equation 36 as

$$\mu_{1,2} = \frac{\left(\sqrt{\varepsilon_{r1}} - \sqrt{\varepsilon_{r2}}\right)}{\left(\sqrt{\varepsilon_{r1}} + \sqrt{\varepsilon_{r2}}\right)} \tag{41}$$

where

ε_{r1} and ε_{r2} are the relative dielectric constants of materials 1 and 2, respectively.

Equation 41 indicates that when a beam of microwave energy strikes the interface between two materials, the amount of reflection ($\mu_{1,2}$) is dictated by the values of the relative dielectric constants of the two materials. If material 2 has a larger relative dielectric constant than material 1, $\mu_{1,2}$ would have a negative value — that is, with the absolute value indicating the relative strength of reflected energy, and the negative sign indicating that the polarity of the reflected energy is the opposite of what is arbitrarily set for the incident energy.

After penetrating the interface and entering into material 2, the wave propagates through material 2 with a speed, given by:

$$V_2 = \frac{C}{\sqrt{\varepsilon_{r2}}} \tag{42}$$

where

V_2 is the speed,
C is the propagation speed of EM waves through air, which is equivalent to the speed of light, or 0.3 m/ns.

As the wave propagates through material 2, its energy is attenuated, as follows:

$$A = 12.863 \times 10^{-8}\ f\ \sqrt{\varepsilon_{r2}}\left(\sqrt{1+\tan^2\delta} - 1\right)^{1/2} \tag{43}$$

where

A is attenuation, in decibel/meter*
f is wave frequency, in Hz,
and the loss tangent (or dissipation factor) is related to σ, the electrical conductivity (in mho/meter) of the material by:

$$\tan \delta = 1.80 \times 10^{10} \frac{\sigma}{f\varepsilon_{r2}} \tag{44}$$

When the remaining microwave energy reaches another interface, a portion will be reflected back through material 2 as given by Equation 41. The resulting two way transit time (t_2) of the microwave energy through material 2 can be expressed as:

$$t_2 = \frac{2D_2}{V_2} = \frac{2D_2\sqrt{\varepsilon_{r2}}}{C} \tag{45}$$

where

D_2 is the thickness of material 2.

13.2. EQUIPMENT FOR THE GPR TECHNIQUE

The physical phenomenon recorded by a GPR system is the reflection of low-level electromagnetic energy at frequencies ranging from 20 MHz to 2 GHz although some systems use frequencies as high as 8 GHz. The short-pulse radar systems used in applications related to the inspection of concrete operate by transmitting a single pulse of three half sine waves. A basic radar system consists of a control unit, a monostatic antenna (i.e. a box with two antenna units in it that are used for transmitting and receiving), a VDU or oscillographic recorder, and a power converter for DC operation (see Fig. 13.2). Typically, the control unit has been replaced by a computer system into which the electrical signal is fed using high speed A/D conversion. Raw data is stored on digital media such as digital tape or hard drives for post processing.

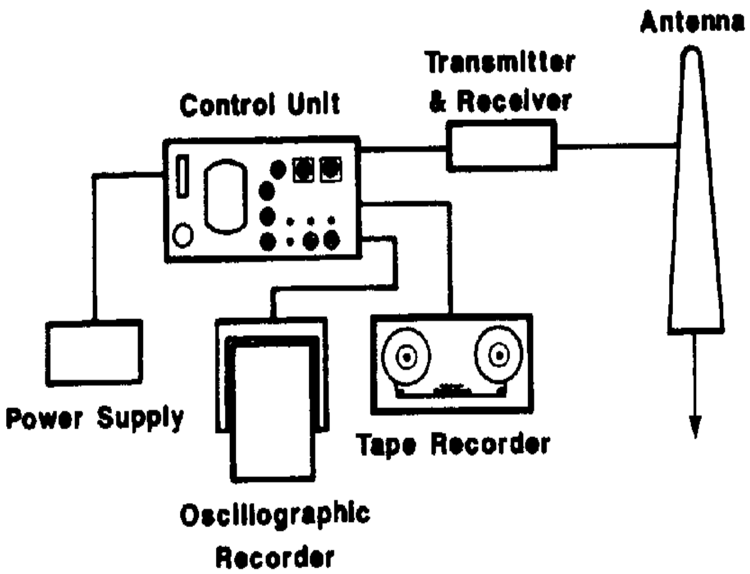


FIG. 13.2. Components of a typical GPR system.

Antenna units are available for a variety of frequencies and are the most important factor for the performance of a radar system. Those used for concrete inspection have frequencies ranging from 500 to 1500 MHz. They are dipole antennas (bow tie) with a linear polarization (the E-vector is directed perpendicular to the long axis of the box).

The radar signal is typically amplified with a time varying function. More highly attenuated signals that are received later after the transmission get higher amplification than early signals. This is a very useful feature and helps in reading the radargrams.

Radargrams are visualizations of radar signals collected over time. The amplitude is plotted as a colour-coded bar along the Y axis, with X axis is the time axis and being advanced by trigger signals from a wheel attached to the antenna box. If the wheel is not moved and the antenna is moved with constant speed, the radargram represents the signal pattern measured along this line.

For the interpretation of a radargram, it is necessary to know that each radar antenna has an opening angle (typically 30° in the direction of the polarization and 45° in the other direction). The bar plotted at a certain position in the radargram represents the signal being received from the whole space within this opening cone of the antenna. This effect causes a point reflector to appear as a hyperbola in the radargram. The lateral position of the object is at the highest point of the hyperbola.

In the inspection of concrete, it is desirable to use a radar antenna with relatively high resolution to locate as small a discontinuity as possible. A typical short pulse width would be 1 ns or preferably less. With such antennas the thinnest layers of concrete that can be distinguished or accurately measured, if it is assumed that concrete has a dielectric constant of 6, would be 3.1 to 6.1 cm, depending on the radar system. An antenna with a centre frequency of 900 MHz has a characteristic pulse width of 1.1 ns. At 0.3 m away from an object, it provides coverage of approximately 0.30 m × 0.38 m. Comparison of the two high frequency radar antennas on the detectability of ducts below reinforcement is shown in Fig. 13.3.

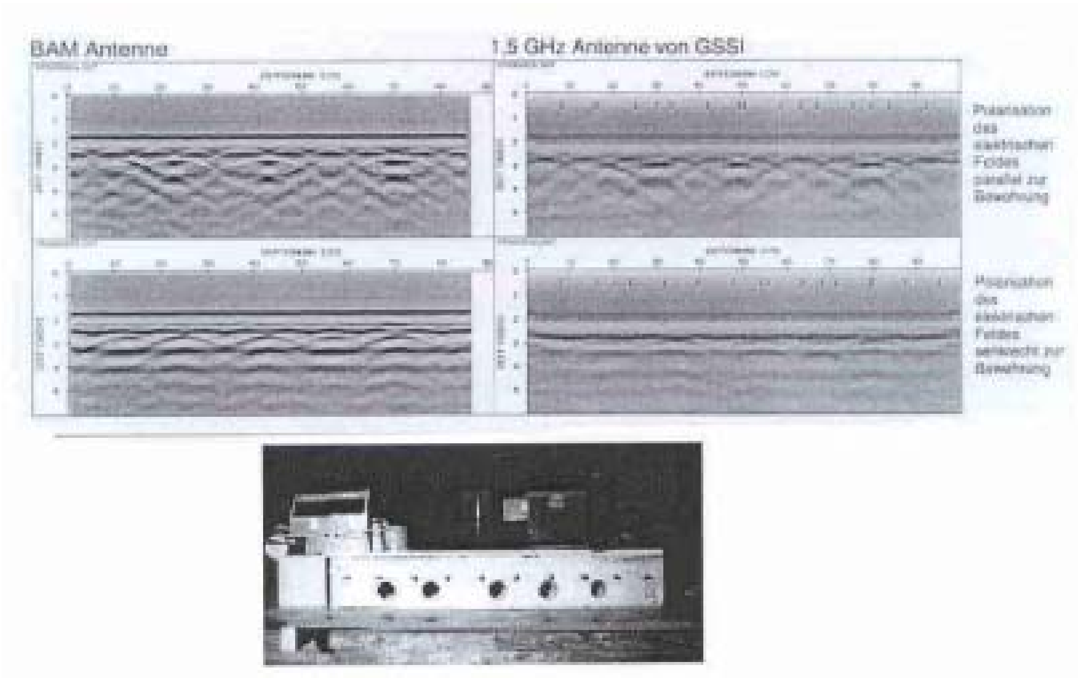


FIG. 13.3. Comparison of two high frequency radar antennas (left 1.8 GHz, right 1.5 GHz) showing the effect of polarization on the detectablity of ducts below reinforcement.

It is quite difficult to estimate a radar system's depth of penetration before inspection is actually done, especially the penetration into reinforced concrete, since penetration is affected by the concrete's moisture content, conductivity, and amount and type of reinforcement. However, for dry and unreinforced concrete the penetration can be as much as 0.6 m although up to 1 m has been claimed by some systems.

In operation, a circuit within the radar control unit generates a trigger pulse signal at a rate of 50 kHz, i.e. a pulse at every 20 μ sec. Each trigger pulse, in turn, causes a solid-state impulse generator in the transmitting antenna to produce pulse with a very fast rise time, which is then electrically discharged from the antenna in the transducer as a short burst of electromagnetic energy. The resulting pulse is then radiated into the material being examined. As the radiated pulses travel through the material, different reflections will occur at interfaces that represent changing dielectric properties. Each reflected electromagnetic pulse arrives back at the receiving antenna at a different time that is governed by the depth of the corresponding reflecting interface and the dielectric constant of the intervening material, see Equation 45.

13.3. APPLICATION OF GPR TECHNIQUES

The most common mode of operation of GPR is the common-offset mode where the receiver and transmitter antennas are maintained at a fixed separation distance and moved along a line to produce a profile. Since energy does not only propagate downwards, reflections are received from objects off to the side. Therefore, the reflected image from a buried object may be laterally larger than the actual buried object.

GPR has some unusual characteristics. The transmitted pulse is very short which allows for accurate measurement of distances from the antennas on the surface to the sub-surface targets. However, antennas having narrow beams cannot in practice be built for such radars. Thus it is often difficult to localize the target in terms of direction. One can only say that the target most probably lies somewhere on the surface of the segment of a sphere of radius r, whose cone angle is 120 degrees. Likewise individual radar echoes seen at the radar receiver display cannot be easily processed. They must be manually scaled and interpreted.

A rebar, as with any metallic object, reflects the radar wave totally. The density of the rebar layout (diameter of the rods and spacing between the individual rebars) will determine whether enough energy is transmitted through the rebar layer. Typically, a high frequency antenna (900 MHz or higher) can penetrate a layer of rebar mesh (12 mm diameter and 15 cm spacing) allowing the detection of objects below the mesh. Exploiting the polarization feature of the radar waves can help to improve the signals and the detection ability, e.g. a duct otherwise masked by the rebars. It is necessary to align the antenna perpendicular to the rebars to minimize any interference with the transmitted signal. If the mesh has a narrow spacing, one rebar is aligned just below the joint between the transmitting and receiving antenna in the antenna box. This will require some trial and error.

13.4. ACCURACY AND INTERPRETATION OF GPR

There are a number of factors to be taken into account when interpreting radar data and signals:

- hyperbolic shapes typically represent a point reflector
- the diameter of cylindrical objects ranging from rebars to metallic oil drums cannot be determined from radargrams
- radar wave velocity reduces when travelling through wet concrete
- radar waves are more rapidly attenuated when travelling through wet concrete

— radar waves cannot penetrate conductors such as: metals, clays, salt water, e.g. sea water
— radar antennas cannot identify objects in the near field which are closer to the surface than λ/3, where

$$\text{Velocity}(v) = \text{frequency}(f) \times \text{wavelength}(\lambda)$$

Therefore, $\lambda = v/f$.

Typical rear field resolutions for different antenna centre frequencies and dielectric constants are given

TABLE 13.1. TYPICAL NEAR FIELD RESOLUTIONS FOR DIFFERENT ANTENNA CENTRE FREQUENCIES AT DIFFERENT DIELECTRIC CONSTANTS

Antenna centre frequency MHz	Near field resolution (λ/3) cm		
	$\varepsilon = 6$ dry concrete	$\varepsilon = 9$	$\varepsilon = 12$ wet concrete
500	25	20	18
900	13.5	11	10
1000	12	10	9
1500	8	7	6

13.5. ADVANTAGES AND LIMITATIONS OF GPR TECHNIQUES

13.5.1. Advantages

Radar has some advantages that are very desirable in the inspection of bridge decks and other concrete structures. Unlike infrared thermography, which has been found to be relatively effective provided proper ambient conditions exist during inspection, radar is free of such restriction.

Since radar yields information on the structural profile across the depth of the object being tested, it is at present the only commercially available non-destructive method for the inspection of concrete bridge decks that have asphalt overlays, which constitute a significant portion of bridges. Under favourable conditions, radar can also detect localized loss of bond between the overlay and the concrete, in addition to detecting delaminations in the concrete.

Since it is unnecessary for the antenna to be in contact with a bridge deck, the disruption to traffic during radar inspection is minimal.

13.5.2. Limitations

Radar surveys on several bridge decks, which were performed prior to the removal of existing overlays, and the results of chain dragging conducted after the removal of the overlays found that, on the average, radar detected only 80% of the concrete delamination present. Radar falsely identified 8% of the area tested as delaminated. Hence, radar occasionally gave an indication of delamination in what is actually sound concrete. Radar may also fail to locate small delaminated areas, especially those that are only 0.3 m wide or less, because the strong reflections from the rebars tend to mask reflections from delaminations.

The inaccuracy is partly due to the inherent limitation of current radar antennas to resolve consecutive reflections arriving at time intervals shorter than the characteristic pulse width of the antenna. Therefore, depending on the dielectric constant of the concrete, the reflection from a small delamination at the level of rebars that are relatively close to the surface may not be able to be resolved because of the reflected signal from the surface.

Another cause of inaccuracy is the lack of a complete understanding of the relationship between radar signatures and the various types of defects that are encountered in concrete structures and how these signatures are affected by the condition in the structure, especially moisture content. A recent EU funded BRITE-EURAM project on the application of radar for the building and construction industries was dedicated to these problems. The results can be obtained from the EU, the participating partners or from publications (SF&R 99 Session on this EU project).

To maximize the depth of penetration of GPR a low frequency radar is desired. However low frequencies mean long wavelengths so that the radar's ability to localize and resolve targets is reduced. Modern computers with colour palettes can now be used to partially overcome this problem since a range of colours can be produced resulting in far better contrast, making interpretation easier. Signal enhancement and processing eliminate much of the potential subjective error from the analysis and may filter out swamping signals such as those from steel to get at the underlying information.

GPR as an NDT technique for the inspection of concrete is still reliant on the experience of the operator. The technique is still developing and there is no doubt that with the great advances made in recent times in data processing that the method will continue to develop. However, the data gathered suffers from several problems, including poor signal to noise ratios, reverberations of the radar waves and mispositioning of objects in the subsurface layers. GPR data are in many respects similar to the data produced by the seismic reflection technique used in the oil and gas exploration industry and, since the oil and gas industry has invested billions of dollars in the development of acquisition and processing methods for the seismic technique, there is tremendous potential to apply those techniques to GPR data with little or no modification. Such dedicated data analysis programmes are emerging for most GPR systems and are able to make a dedicated analysis of multidimensional GPR data (e.g. REFLEX by Sandmeier from Kalsruhe, Germany).

The amount of reinforcing steel may make the inspection of lower layers of the concrete very difficult. It should also be pointed out that radar waves can never penetrate through metals, so testing the interior of metal ducts, e.g. for ungrouted areas, is NOT possible with radar.

Any radar site should be well documented by photographs or notes. Radar waves can be reflected from metallic objects such as cars or telephone lines and give spurious data. Good documentation helps identify such effects.

The conversion from time to a depth scale can only be as accurate as the velocity of light is known for this material (hence the dielectric constant). A linear transformation is only correct for homogeneous objects. In multilayer test objects, the speed of light can be different for each layer. No data analysis program is yet able to handle this problem.

The propagation velocity can be determined non-destructively by the common-mid-point-method, also known as the walk-away test. Two sets of antenna are needed to perform this test. The antennas are placed atop a reflector in the test object, which give a clear signal and moved symmetrically away from the common midpoint. The distance between the antennas is then plotted versus the delay time of the reflected signal. The slope of the function for longer distances is the propagation velocity of the radar waves in this specimen. The velocity can also be determined from the opening of the hyperbola when moving an antenna across an object.

13.6. SAFETY ADVICE

Radar uses pulsed microwaves, which interact with water. Although the emitted energy is much too low to cause any harm to humans, it is advisable not to point the antenna at humans when operating the system. Also, interference with mobile telephones may occur when antennas are not coupled to the surface of the concrete or the ground.

13.7. EXAMPLES OF INSPECTION OF STRUCTURES

13.7.1. Detection of underground utilities

There are many kinds of public utilities such as water pipes, gas pipes, communication ducts, sewerage boxes, etc. which lie beneath footpaths and roadways and which have to be detected from time to time to carry out maintenance work. The traditional method to locate the service is to dig a trench in the area indicated by drawings. However if drawings do not exist or are inaccurate there can be considerable time wasted and additional cost in digging to locate the service. GPR can be used to locate such services. 225, 450 and 900 MHz antennas have been used depending on the size of service to be found and the depth below the surface.

Some examples of the reflection patterns from different services have been given by Seong-Ho Bae, Hag-Soo Kim and Woon-Sang Yoon. These reflection patterns are shown in Figs. 13.4 to 13.5. In some box shaped targets diffraction patterns can be observed from the corners of the upper part of the box. However when the ground materials are highly non-homogeneous, the reflection patterns are very complex and the diffraction patterns are not clear.

In Fig. 13.4(d) the flat reflection signals, shown with a black arrow are from the upper part of the concrete duct and the hyperbolic signals, shown with a white arrow, are from PVC pipes containing communication cables inside the duct.

Patterns vary considerably. Fig. 13.4(a) is the reflection from steel water pipes, (b) is a concrete pipe filled with water and shows multiple reflection patterns. Fig. 13.4 (c) and (d) show reflection images of a sewage box and a communication cable duct, respectively.

13.7.2. Inspection of tunnel lining

Another use made of GPR is in the inspection of tunnel lining. The application can be either during construction to detect voids behind the lining or during the maintenance and repair of linings. One of the advantages of GPR in this situation is that the results are not influenced at any great degree by the presence on the surface of lining applied for lighting and ventilation.

GPR can provide information on:

- concrete lining thickness and the variation in thickness
- location of defective zones in the concrete lining, e.g. cavities, water leakage, fractured zones
- location of steel supports (rock bolts, steel ribs) and their omission

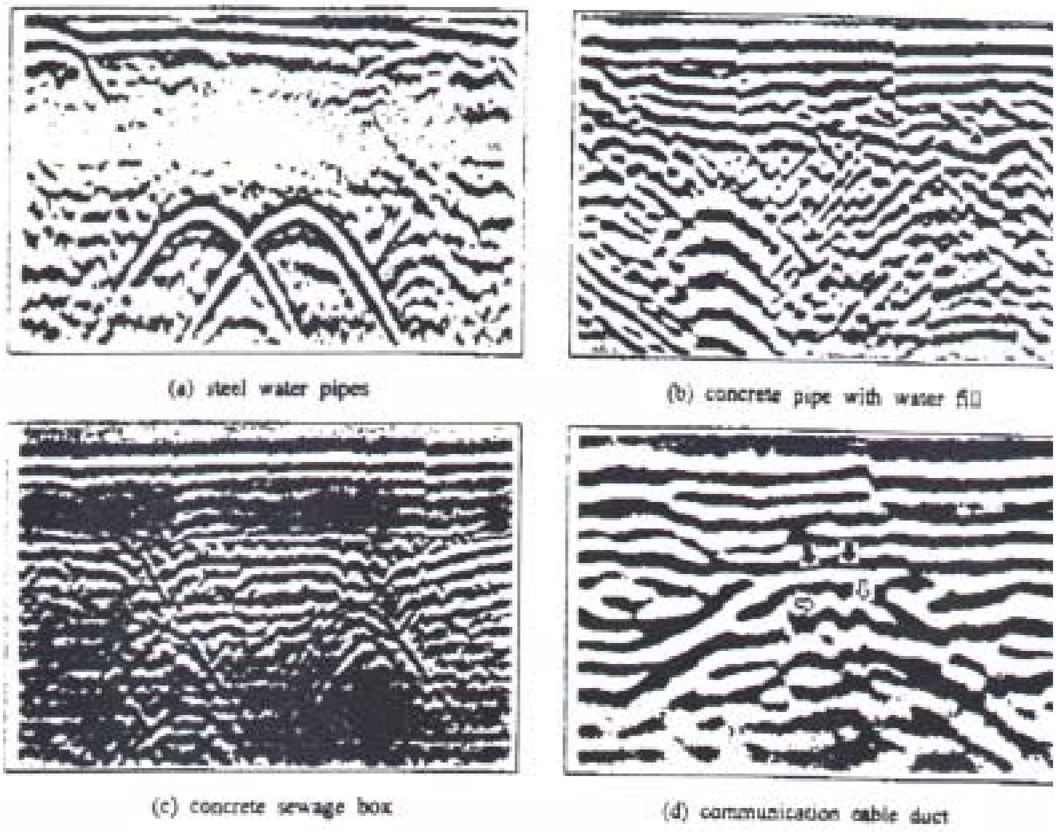


FIG. 13.4. Reflection images from buried services.

- depth, spacing and omission of reinforcing steel
- interface between the concrete lining and rock mass
- homogeneity of the concrete.

Suitable antennas for tunnel lining inspection range from 450 MHz to 900 MHz. Depending on the dimension of the targets, inspection depth and the wave parameters in the concrete material, the 450 MHz and 900 MHz antennas can be used selectively or together.

Fig. 13.5(a) is a radar section from a highway tunnel using a 450 MHz antenna. It shows clearly the interface between shotcrete and rock and the rock bolts inserted into the rock to improve the bearing capacity of the rock mass. The interface between the shotcrete and the plain concrete lining does not show up clearly since both materials have similar characteristics.

In Fig. 13.5(b) the survey was performed with a 900 MHz antenna and the spatial sampling interval was 2 cm.

In practice, it is difficult to detect small and irregularly shaped cavities inside or behind the lining. It is likewise difficult to distinguish the reflection signals from a cavity from the other signals reflected from the various objects which can be inside the lining, such as steel supports, rebar, moist patches, gravel, etc.

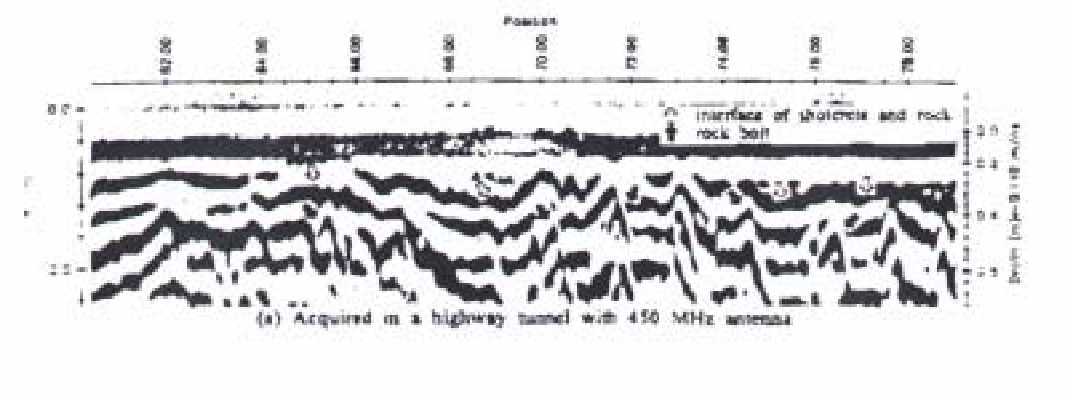


FIG. 13.5(a). The inspection of concrete tunnel lining using GPR.

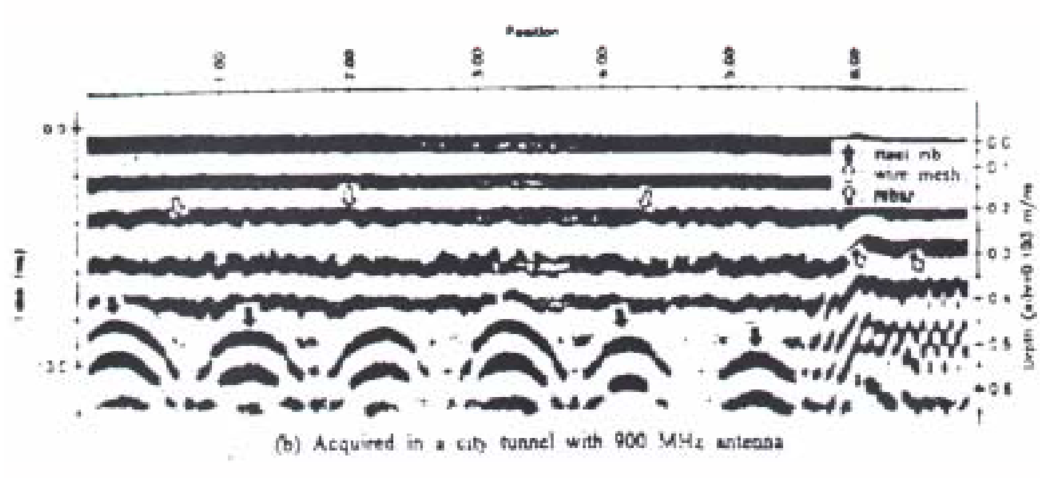


FIG. 13.5(b) Presence of a steel ribs, wire mesh and reinforcing bars in a section of a tunnel.

13.7.3. Detection of delamination in concrete bridge decks

A major problem with the reinforced concrete bridge decks located in coastal areas, or areas where de-icing salts are used on roadways during winter, is the premature deterioration of concrete. The intrusion of chloride from these salts into the concrete causes the embedded reinforcement to corrode, which eventually causes concrete to crack or delaminate. Marine structures are also subjected to this type of deterioration.

The simplest method to detect delamination in concrete involves sounding the concrete with a hammer or heavy chains, which produces a characteristic hollow sound when delamination is present. Although the method is effective, it is adversely affected by the presence of traffic noise. Since sounding is a contact method, its use requires closure of traffic lanes, which is often costly and undesirable. Because of the need for an alternative to the sounding method, the use of non-contact and non-destructive methods such as infrared thermography and radar have been studied.

13.7.3.1. Principle of radar operation on concrete bridge decks

When a beam of microwave energy is directed at a reinforced concrete slab (Fig. 13.6), a portion of the energy is reflected from the surface of the concrete and the remaining energy penetrates this interface.

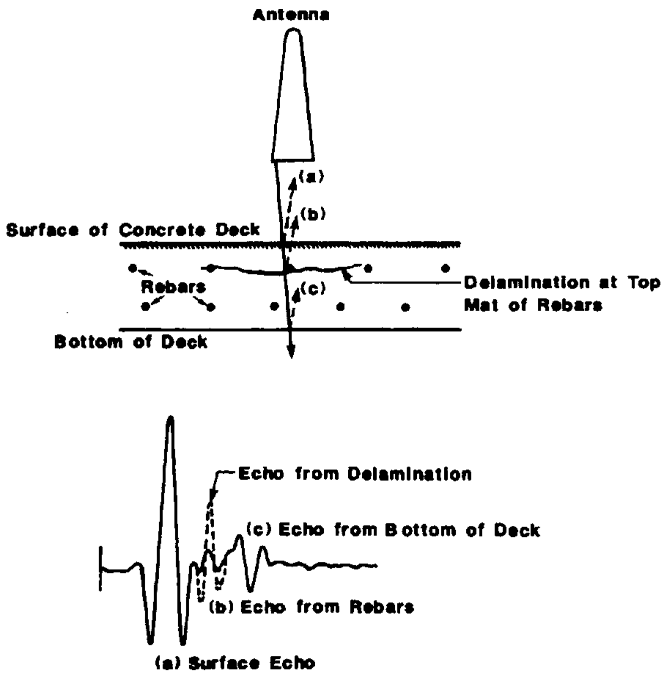


FIG. 13.6. Radar echoes from the cross-section of a reinforced concrete deck.

The surface reflection has a negative polarity since the dielectric constant of concrete, which has been reported to range from 6 when dry to about 12 when saturated, is considerably higher than that of air, which is 1. It must be noted that the actual *in situ* relative dielectric constant of concrete and most materials will vary because it is affected to varying degrees by not only its water content but also by its conductivity, mineral composition, etc. As the remaining microwave energy propagates into the concrete, a portion of the beam will be completely reflected and scattered as it strikes the top mat of reinforcement. This reflection will also have a negative polarity, since the dielectric constant of metal is infinite compared with that of the surrounding concrete. The remaining energy will continue deeper into the concrete slab until a portion of it strikes the second mat of reinforcement and the same reflection and scattering processes occur. Eventually some portion of the original beam of microwave energy will reach the bottom of the concrete slab and some of it will be reflected at the concrete air/interface to give a positive reflection signal. The remainder will penetrate through this interface and be lost from the receiving antenna. When the concrete slab is delaminated, usually at the level of the top mat of reinforcement, there is an additional reflection from the deteriorated section. This additional reflection, usually of negative polarity, serves as an indicator of the presence of a delamination in the concrete slab.

The presence of a delamination causes additional reflection of the incident energy.

13.7.3.2. Test principles

Two types of antenna are typically used for bridge deck evaluation:

(1) Air coupled (0.2-0.5 m above the ground) horn antenna designed to operate at 1GHz or 2.5GHz. The higher the frequency the better the resolution, but the lower the depth penetration. Thus a 2.5GHz antenna may give the required resolution of say 3 cm but be restricted to a penetration of 35 cm. That may mean that the bottom of the deck may not be identified.

(2) Bow tie antenna is normally close coupled to (i.e in contact with) the ground. Very little energy would be transmitted into the bridge deck if a bow tie antenna is held at more than $\lambda/10$ above the ground.

Research and practice in mid to late 1990's in Europe and the USA indicated that good results could be obtained with the 1.5 GHz bow tie antenna close coupled to the road surface of the bridge deck.

One main reason for choosing air coupled horn antennas was to enable high speed radar scans to be made of highways and bridge decks at, say 50, km/h. By contrast, ground coupled bow tie antenna surveys would be undertaken at 10-15 km/h, requiring lane closures at considerable expense and disruption to traffic. However, the radar systems for high speed measurement using horn antennas typically cost double than that of a normal radar system. Also, the resolution may not be as high as the ground coupled 1.5 GHz bowtie antenna.

In the inspection of bridge decks, in which a relatively large concrete area has to be inspected, the antenna is mounted on the front or the rear of an inspection vehicle, which is also instrumented with a horizontal distance-measuring device. If a single antenna radar system is used the vehicle has to make several passes over each traffic lane from one end of a deck to the other at a selected speed, usually in the range from 8 to 16 km/h. During each pass, the antenna scans a different area in the lane. The stream of radar signals is recorded continuously with an instrumentation tape recorder. With a two-antenna or a multiple antenna radar system, a single pass may be made over each lane. It must be emphasized that when a lane is scanned with a two antenna radar system in a single pass only, a significant portion of the lane will be missed. This procedure creates continuous recordings of the reflections from the entire depth of the deck along the paths of the antenna. These recordings are played back at a later time for signal interpretation.

Full recording of a lane (2.5 m wide) in high traffic (65 mph) was the goal of a radar system developed at Lawrence Livermoore National Laboratory and the Federal Highway Administration (FHWA) in the USA. The system, named High Speed Electromagnetic Roadway Mapping and Evaluation System (HERMES), Fig. 13.7 collects data simultaneously from 64 wide band air coupled horn antennas mounted on a trailer that is towed by a truck. The design specification includes a resolution of 3 cm and a penetration depth of 35 cm. The system is presently under test.

A scaled down version of the FHWA HERMES is the Precision Electromagnetic Roadway Evaluation System (PERES), Fig. 13.8, which has only one pair of horn antennas that can be moved along a line that is 2 m long. By moving the system large areas can be tested. The software allows one to plot depth slices of the tested area, giving a visual representation of the internal features as indicated in Fig. 13.9.

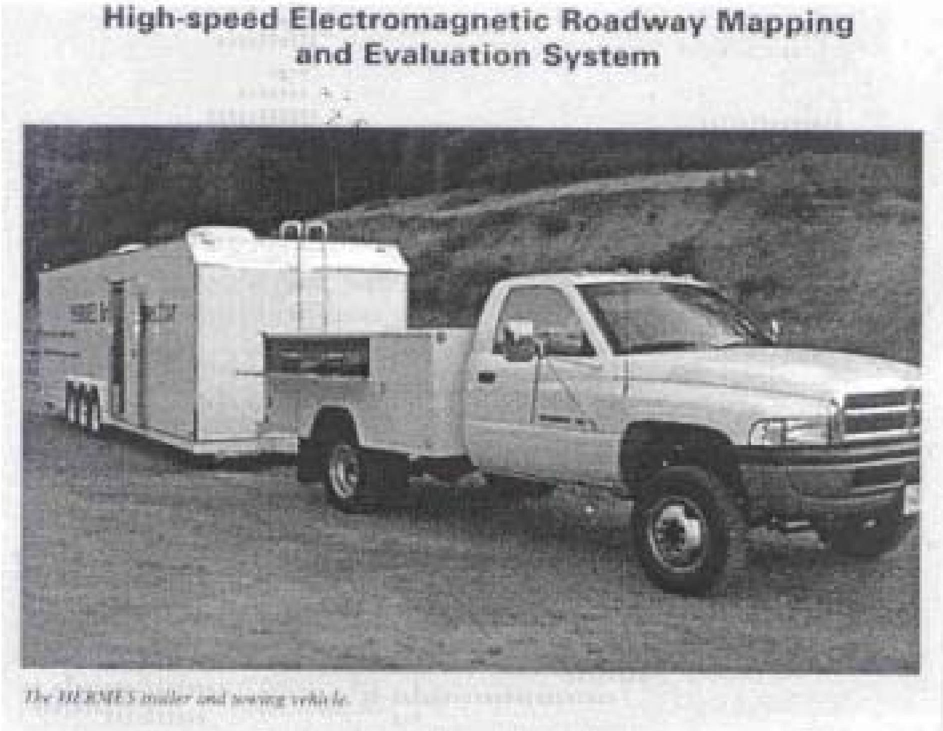


FIG. 13.7. HERMES system for bridge deck inspection.

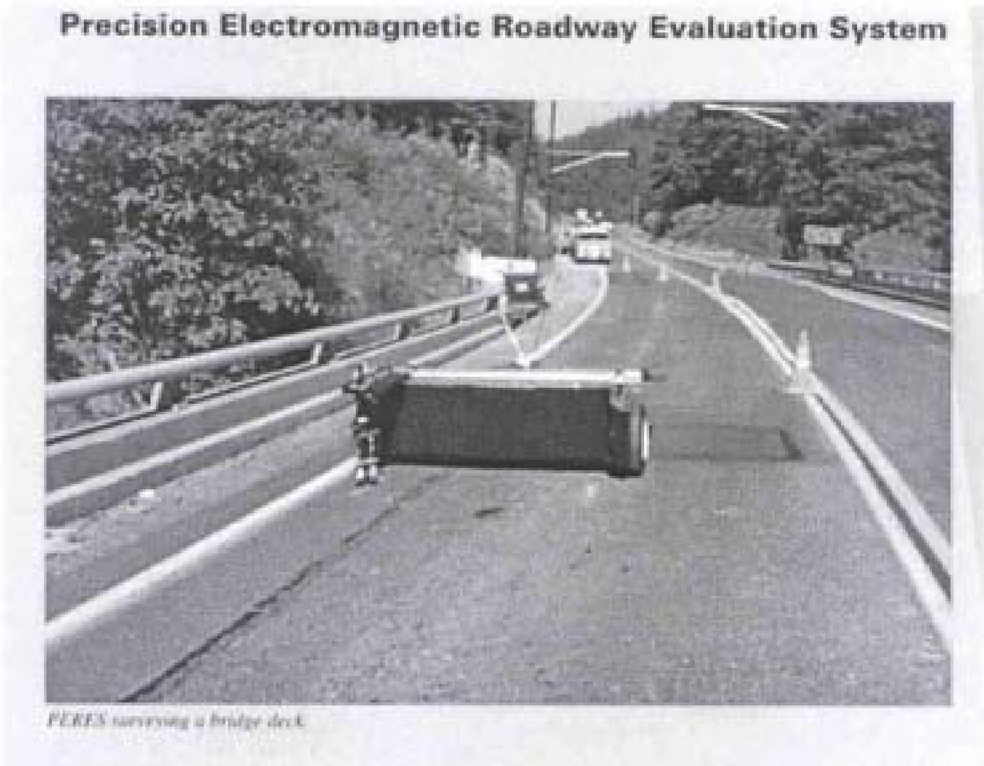


FIG. 13.8. PERES scanning radar system for bridge deck inspection.

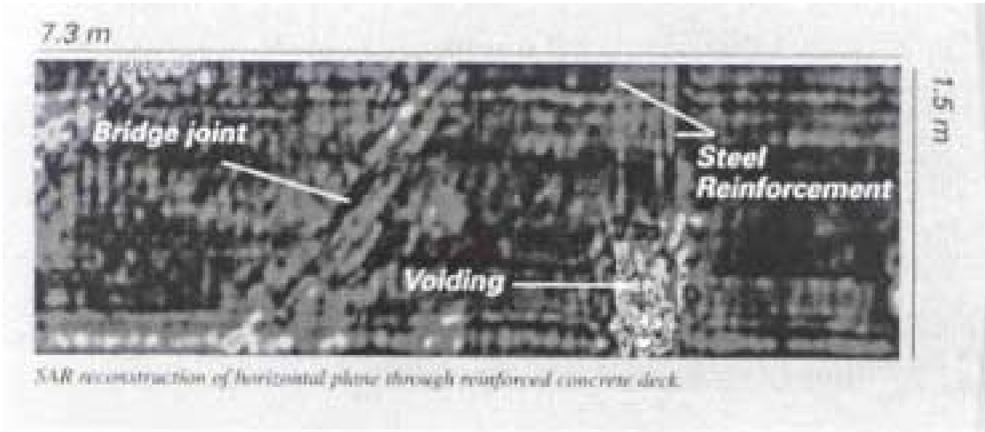


FIG. 13.9. Synthetic aperture reconstruction of horizontal plane through bridge deck using PERES system.

14. RADIOISOTOPE GAUGES

14.1. THICKNESS AND DENSITY GAUGES

14.1.1. Fundamental principles

The use of radioisotopes for the non-destructive testing of concrete is based on directing the gamma radiation from a radioisotope against or through the fresh or hardened concrete. When a radiation source and a detector are placed on the same or opposite sides of a concrete sample, a portion of radiation from the source passes through the concrete and reaches the detector where it produces a series of electrical pulses. When these pulses are counted the resulting count or count rate is a measure of the dimensions or physical characteristics, e.g. density of the concrete. Although this radiometry method has not been commonly used on concrete, the increasing use of radioisotopes to measure the compaction of asphalt or bituminous concrete and the soil-aggregate mixtures used in road construction means that the method may be more commonly used in the future. The method has been used, for instance, for density determinations on roller compacted and bridge deck concrete.

The interaction of gamma rays with concrete can be characterized as penetration with attenuation – that is, if a beam of gamma rays strikes a sample of concrete, (a) some of the radiation will pass through the sample, (b) a portion will be removed from the beam by absorption, and (c) another portion will be removed by being scattered out of the beam (when gamma rays scatter, they lose energy and change direction). If the rays are traveling in a narrow beam, the intensity I of the beam decreases exponentially according to the relationship:

$$I = I_0 e^{-\mu x} \tag{46}$$

where

I_0 is the intensity of the incident beam,
x is the distance from the surface where the beam strikes,
μ is the linear absorption coefficient.

For the gamma ray energies common in nuclear instruments used to test concrete, the absorption coefficient includes contributions from a scattering reaction called Compton scattering, and an absorption reaction called photoelectric absorption. In Compton scattering, a gamma ray loses energy and is deflected into a new direction by collision with a free electron. In photoelectric absorption, a gamma ray is completely absorbed by an atom, which then emits a previously bound electron. The relative contributions of Compton scattering and photoelectric absorption are a function of the energy of the incident gamma rays. In concrete, Compton scattering is the dominant process for gamma ray energies in the range from 60 keV to 15 MeV, while photoelectric absorption dominates below 60 keV.

The amount of Compton scattering, which occurs at a given gamma ray energy, is a function of the density of the sample being irradiated. The amount of photoelectric absorption that occurs is chiefly a function of the chemical composition of the sample; it increases as the fourth power of the atomic number of elements present.

The detectors for the radiometry techniques absorb a portion of the radiation and turn it into electrical pulses or currents, which can be counted or analyzed. In Geiger-Muller tubes, for example, gamma rays ionize some of the gas in the tube. When the amount of ionization is then multiplied by a high voltage applied across the tube, it produces an electrical pulse, which indicates radiation has interacted in the tube. Geiger-Muller tubes are used widely in nuclear density gauges.

14.1.2. General procedure for thickness and density gauges

All gamma radiometry systems are composed of (a) a radioisotope source of gamma rays, (b) the object (concrete) being examined, and (c) a radiation detector and counter. Measurements are made in either of two modes, direct transmission (Fig. 14.1.) or backscatter (Fig. 14.2).

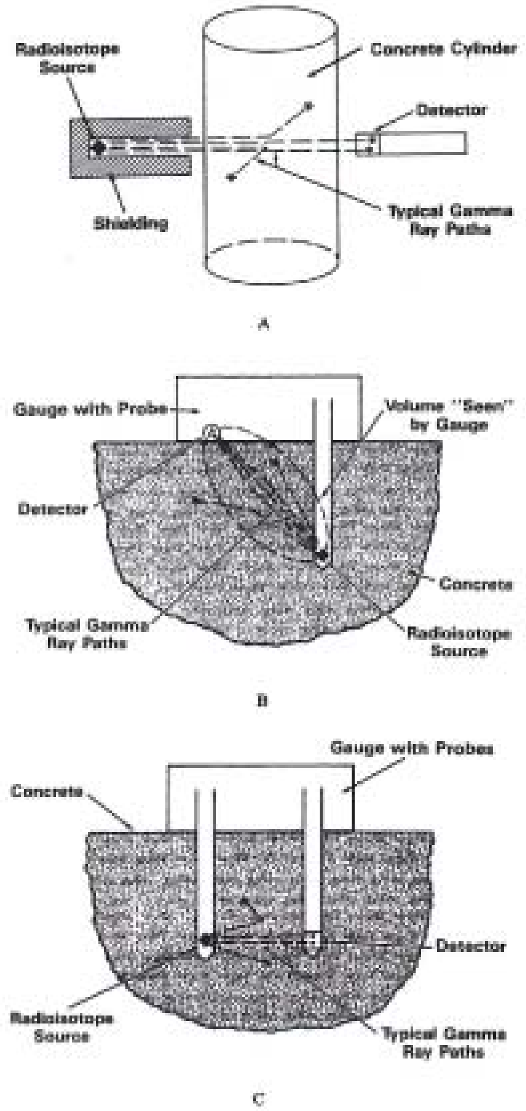
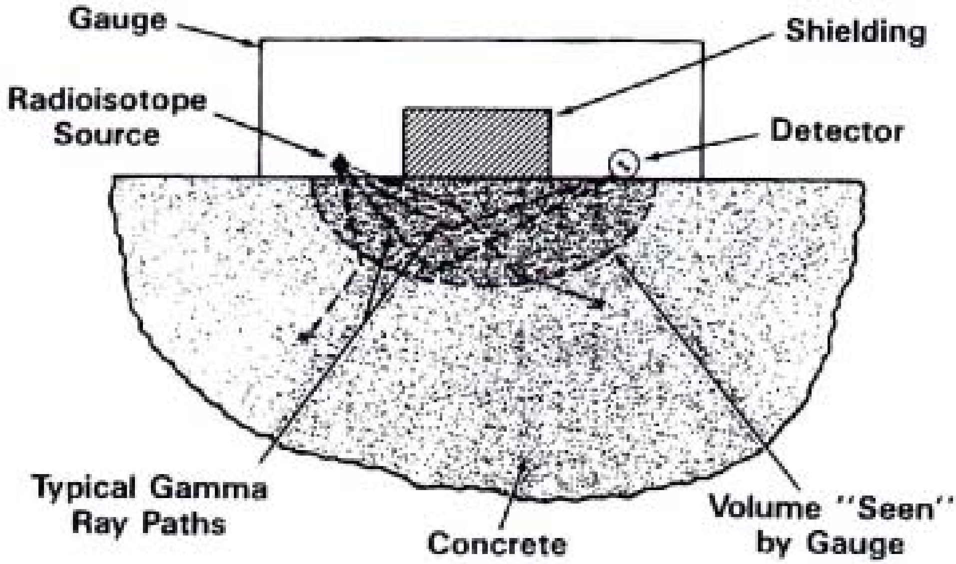
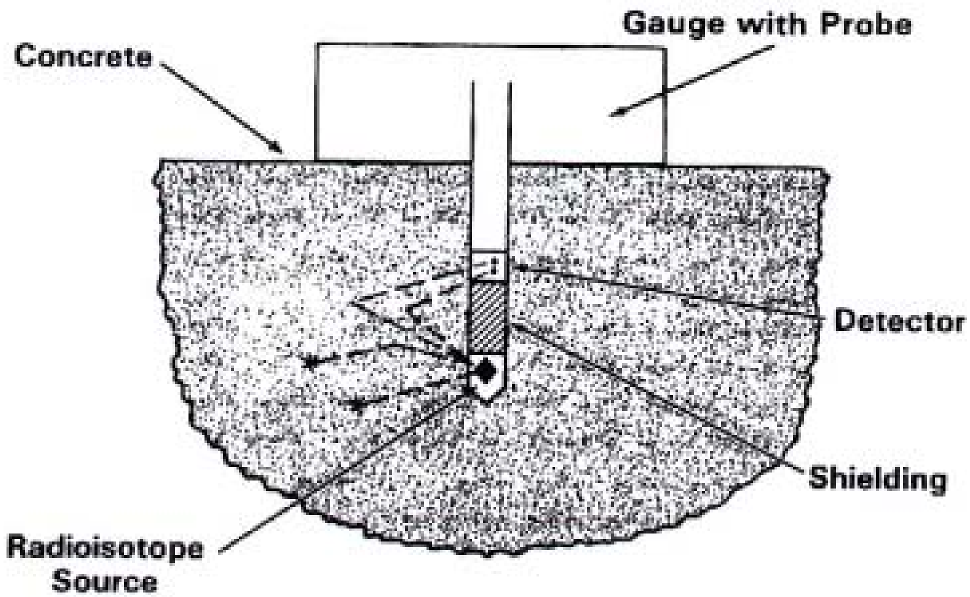


FIG. 14.1. Direct transmission (A) source and detector external to concrete, (B) source internal, detector external, and (C) source and detector both internal.



A



B

FIG. 14.2. Backscatter (A) source and detector both external to concrete, (B) both in probe internal to concrete.

In direct transmission, the specimen, or at least a portion of it, is positioned between the source and the detector. The source and detector may be both external to the concrete sample (Fig. 14.1A); e.g. in making density scans on cores or thickness determinations on pavements. The source may be inside the concrete and the detector outside (Fig. 14.1B), e.g. in determining the density of a newly placed pavement or bridge deck. Or the source and detector may both be inside the concrete (Fig. 14.1C), e.g. in determining the density of a particular stratum in a newly placed pavement or in a hardened cast concrete pile.

In direct transmission, the gamma rays of interest are those that travel in a straight (or nearly straight) line from the source to the detector. Gamma rays that are scattered through sharp angles, or are scattered more than once, generally do not reach the detector. The fraction of the originally emitted radiation that reaches the detector is primarily a function of the density of the concrete, and of the shortest distance between the source and the detector through the concrete, as shown in Equation 46. Typical gamma ray paths are shown in Fig. 14.1. The actual volume of the concrete through which gamma rays reach the detector, i.e. the volume which contributes to the measurement being made, is usually ellipsoidal in shape (see Fig. 14.1B), with one end of the volume at the source and the other at the detector.

Sources typically used in direct transmission devices allow measurements to be made through 50 to 300 mm of concrete.

In backscatter measurement, the source and the detector are next to each other, although separated by radiation shielding. No portion of the concrete sample lies on a direct path between the source and detector. The source and detector may both be external to the concrete (Fig. 14.2A), e. g. in determining the density of a newly placed pavement or bridge deck from the top surface of the concrete. Or they may both be in a probe which is inserted in the concrete (Fig. 14.2B), e.g. in borehole in a cast pile.

In backscatter, only gamma rays that have been scattered one or more times within the concrete can reach the detector. Shielding prevents radiation from traveling directly from the source to the detector. Examples of gamma ray paths are shown in Fig. 14.2A. Each time a gamma ray is scattered it changes direction and loses some of its energy. As its energy decreases, the gamma ray becomes increasingly susceptible to photoelectric absorption. Consequently, backscatter measurements are more sensitive to the chemical composition of the concrete sample than are direct transmission measurements in which unscattered gamma rays form the bulk of the detected radiation.

Backscatter measurements made from the surface are usually easier to perform than direct transmission measurements, which require access to the interior or opposite side of the concrete. However, backscatter has another shortcoming besides sensitivity to chemical composition: the concrete closest to the source and detector contributes more to radiation count than does the material farther away.

14.1.3. Equipment for thickness and density gauges

For typical, commercially available backscatter density gauges, the top 25 mm of a concrete sample yields 50 to 70% of the density reading, the top 50 mm yield 80 to 95%, and there is almost no contribution from below 75 mm. The source, detector, and shielding arrangement can be modified to somewhat increase the depth to which a backscatter gauge will be sensitive. A gauge has been developed for mounting on the back of a slip form paver for continuous density monitoring. Slightly over 70% of the device's reading comes from the top 50 mm of concrete and about 5% comes from below 90 mm. Gauges with minimal depth sensitivity may be desirable for applications such as measuring density of a thin [25 to 50 mm] overlay on a bridge deck. Backscatter measurement has a third disadvantage: its sensitivity to surface roughness; however, this is rarely a concern for measurements on concrete. Gamma radiometry systems for monitoring density generally use ^{137}Cs (662 keV) sources, but ^{226}Ra (a wide range of gamma ray energies, which can be treated as equivalent, on the average, to a 750 keV emission) and ^{60}Co (1.173 and 1.332 MeV) are employed in some. These sources are among the few that have the right combination of long half-life and sufficiently high initial gamma ray energy for density measurements. The half-life of ^{137}Cs, for example, is 30 years.

Most commercially available density gauges employ gas filled Geiger-Muller (G-M) tubes as gamma ray detectors because of their ruggedness and reliability. Some prototype devices have employed sodium iodide scintillation crystals as detectors. The crystals are more efficient capturers of gamma rays than G-M tubes. They also can energy discriminate among the gamma rays they capture, a feature which can be used to minimize chemical composition effects in backscatter mode operation. However, the crystals are temperature and shock sensitive and, unless carefully packaged, they are less suitable for field applications than the G-M detectors.

Portable gauges for gamma radiometry density determinations are widely available. A typical gauge is able to make both direct transmission and backscatter measurements, as shown in Figs. 14.1B and 14.2A, respectively. The gamma ray source, usually 8 to 10 mCi of ^{137}Cs, is located at the tip of a retractable (into the gauge case) stainless steel rod. The movable source rod allows direct transmission measurements to be made at depths up to 200 or 300 mm, or backscatter measurements when the rod is retracted into the gauge case. The typical gauge would have one or two G-M tubes inside the gauge case about 250 mm from the source rod. With the source rod inserted 150 mm deep into the concrete, the direct transmission source-to-detector distance would be about 280 mm.

Detailed procedures for both direct transmission and backscatter measurements are given in ASTM Standard Test Method C 1040. Density measurements require establishment of calibration curves (count rate vs. sample density) prior to conducting a test on a concrete sample. Calibration curves are created using fixed density blocks, typically of granite, limestone, aluminium, and/or magnesium. Method C 1040 encourages users to adjust the calibration curves for local materials by preparing fresh concrete samples in fixed volume containers (the containers must be at least 450 mm × 450 mm × 150 mm for backscatter measurements). The nuclear gauge readings on the concrete in such a container are compared with the density established gravimetrically, i.e. from the weight and volume of the sample, and the calibration curve is shifted accordingly.

In-place tests on concrete are straightforward. For direct transmission measurement, the most common configuration is that shown in Fig. 14.1B; the gauge is seated with the source rod inserted into a hole that has been formed by a steel auger or pin. For a backscatter measurement, the most common configuration is shown in Fig. 14.2A, with the gauge seated on the fresh or hardened concrete at the test location. Care must be taken to ensure reinforcing steel is not present in the volume “seen” by the gauge. Reinforcing steel can produce a misleadingly high reading on the gauge display. Counts are accumulated, typically over a 1 or 4 minute period, and the density is determined from the calibration curve or read directly off a gauge in which the calibration curve has been internally programmed.

Tests with other gamma radiometry configurations (Figs. 14.1A, 14.1C, and 14.2B) employ the same types of sources and detectors. Various shielding designs are used around both sources and detectors in order to collimate the gamma rays into a beam and focus it into a specific area of a sample. The two-probe direct transmission technique (Fig. 14.2C) needs additional development but has considerable potential for monitoring consolidation at particular depths, e.g. below the reinforcing steel in reinforced concrete pavements.

Iddings and Melancon used a commercially available gauge with a 5 mCi ^{137}Cs source and a 25 mm diameter × 25 mm sodium iodide scintillation crystal, respectively, in two probes that were separated by 300 mm of concrete. They reported the effective vertical layer thickness for density measurements with this system to be about 25 mm.

14.1.4. Applications of thickness and density gauges

The development of gamma radiometry techniques did not really begin until radioisotope sources became widely available with the advent of nuclear reactors after World War 2. Malhotra reported Smith and Whiffin as the first users of gamma radiometry on concrete in 1952. They made direct transmission measurements using a ^{60}Co source inserted in a hole in a concrete block and a Geiger-Muller tube detector external to the block. The apparatus allowed measurements of variations in density with depth in order to evaluate the effectiveness of an experimental surface vibrating machine.

In his 1976 survey, Malhotra reported gamma radiometry had been used for measuring the *in situ* density of structural concrete members, the thickness of concrete slabs, and the density variations in drilled cores from concrete road slabs. With the possible exception of its application in Eastern Europe for monitoring density in precast concrete units, radiometry was still an experimental non-destructive testing tool for concrete at that time. Density monitoring applications increased in the highway industry after a 1972 report by Clear and Hay showed the importance of consolidation in increasing the resistance of concrete to penetration by chloride ions. A number of U.S. state and Canadian province highway agencies began using commercially available nuclear gauges to evaluate the density achieved in bridge deck overlays, particularly overlays employing low slump, low water-cement ratio mixes. In 1979, the American Association of State Highway and Transportation Officials (AASHTO) adopted a standard method, T 271, for the Density of Plastic and Hardened Portland Cement Concrete in Place by Nuclear Methods, and, in 1984, the American Society for Testing and Materials (ASTM) followed with a slightly different version, Test Method C1040. Most recently, Whiting, et al. showed the strong influence of consolidation on several critical properties of concrete including strength, bond to reinforcing steel, and resistance to chloride ion penetration. They also evaluated several existing nuclear (gamma radiometric) gauges and strongly recommended their use for monitoring consolidation during construction. They pointed out the value of density monitoring in evaluating the quality of concrete construction itself, rather than just the quality of the materials being delivered to the job site. Currently no procedures are in standard use to measure the in-place quality of concrete immediately after placement; that quality is not assessed until measurements such as strength, penetration resistance, and/or smoothness can be made after the concrete has hardened.

Gamma radiometry is also being used extensively for monitoring the density of roller compacted concrete. Densification is critical to strength development in these mixtures of cement (and pozzolans), aggregates and a minimal amount of water. After placement the concrete is compacted by rollers, much the same as asphalt concrete pavements. Commerica1ly available nuclear gauges have become standard tools for insuring the concrete is adequately compacted.

Gamma radiometry has found limited application in composition determinations on PCC. When radioisotope sources emit low energy (below 60 keV) gamma rays, photoelectric absorption is the predominant attenuation mechanism, rather than Compton scattering. Since the absorption per atom increases as the fourth power of the atomic number Z, it is most sensitive to the highest Z element present in a sample. Noting that calcium in portland cement is the highest Z element present in significant quantities in PCC (in mixtures containing non-calcareous aggregates), Berry used ^{241}Am (60 keV gammas) in a prototype backscatter device for measuring the cement content of fresh concrete. Mitchell refined the technique, and the resulting instrument was reported to measure cement contents to within ± 8% (at a 95% confidence level) for siliceous aggregate mixtures and to within ±11% for calcareous aggregate mixtures. Because of the sensitivity of photoelectric absorption to Z, the cement content procedure required calibration on a series of mixtures of different cement contents for a given aggregate source. This sensitivity to aggregate composition remains a barrier to further application of the technique.

A short lived but interesting application of gamma radiometry is in pavement thickness determinations. As Equation 46 shows gamma ray absorption is a function of the thickness of a specimen. Therefore, a source could be placed beneath a PCC pavement, and, if a detector is positioned directly over the source, the count recorded by the detector would be a function of the pavement thickness. Researchers placed thumbtack-shaped ^{46}Sc sources on a pavement sub-base before a PCC pavement was placed. The sources were difficult to locate after the concrete was placed, however, and the technique was abandoned albeit with a recommendation that it deserved further research.

Tayabji and Whiting reported on two field tests using commercially available nuclear density gauges on PCC pavement. A typical data collection effort is shown in Fig. 14.3, where measurements are being taken during slipform paving operations from a platform that was part of a dowel inserter. Readings were taken in the direct transmission mode, with a 10 mCi ^{137}Cs source inserted 8 in. (200 mm) into the pavement. The technician made a 15 sec. count at every third stop of the inserter unit, i.e. at about 42 ft (13-m) intervals.

Measurements were made at eight locations in each of eight lots. The average consolidation in the eight lots ranged from 98.9 to 100.2% of the rodded unit weight, with standard deviations within the lots ranging from 0.5 to 1.3%.

Density monitoring is critical on RCC projects since high density is needed to develop adequate flexural strength. On a pavement project such as the one shown in the figure, the concrete behind the paver typically has a density of 95% of the laboratory maximum, but will reach 98% after additional roller compaction.

Ozyildirim cautions that static gauges are not suitable for exact determinations of degree of consolidation in the field, because variations in component proportions or air content within acceptable ranges can cause variations in the maximum density attainable.

Fig. 14.4 is a photograph of the consolidation monitoring device (CMD) in use over a newly placed, conventional PCC pavement. The CMD is a non-contact backscatter density gauge, shown here with the source/detector unit mounted on a track on the back of a slip form paver. The unit rides back and forth transversely while the paver moves forward, thus monitoring the density of a significant portion of the pavement. The CMD uses a 500 mCi ^{137}Cs source and a 1-3/4 in. dia. × 4 in. long (45 × 102 mm) sodium iodide scintillation crystal. Results indicate that the device is capable of duplicating core density measurements within a ± 2-1/4 lbs/ft^3 (± 36 kg/m^3) range at a 95% confidence level The CMD appears to be effective for tasks such as establishing proper vibrator operation, alerting vibrator malfunctions, and detecting significant changes in mixture composition, i.e. too little or too much air entrainment.

A

FIG. 14.3. Static nuclear density gauges in use during construction of conventional PCC pavement (Photo courtesy of FWHA).

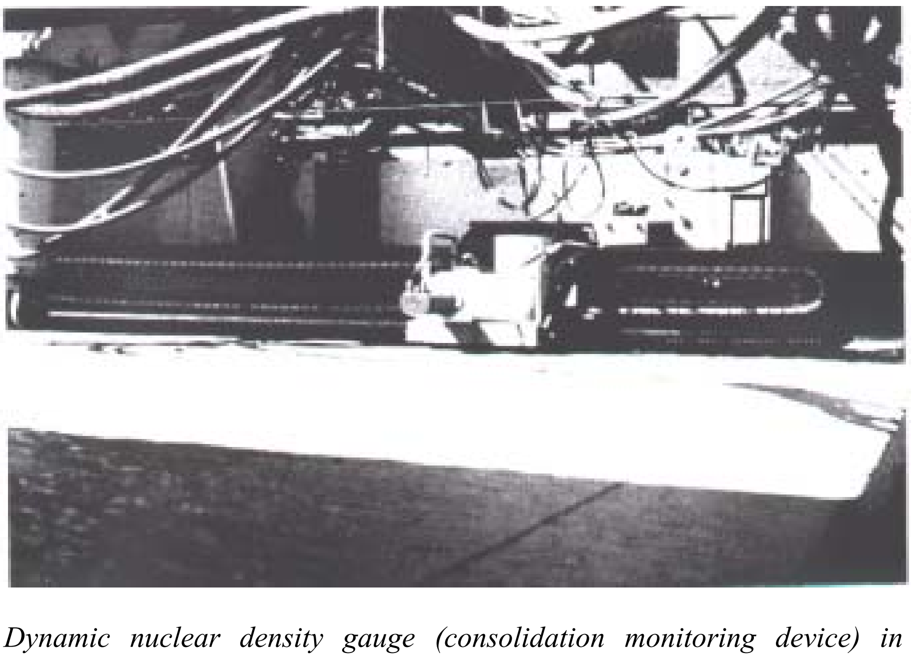


FIG. 14.4. Dynamic nuclear density gauge (consolidation monitoring device) in use during construction of conventional PCC pavement (Photo courtesy of FWHA).

14.1.5. Advantages and limitations of thickness and density gauges

Gamma radiometry offers engineers a means for rapidly assessing the density and, therefore, the potential quality of concrete immediately after placement. Direct transmission gamma radiometry has been used for density measurements on hardened concrete, but its speed, accuracy, and need for internal access make it most suitable for quality control measurements before newly placed concrete undergoes setting. Backscatter gamma radiometry is limited by its inability to respond to portions of the concrete much below the surface, but it can be used over both fresh and hardened concrete and can be used, in non-contact devices, to continuously monitor density over large areas. Gamma radiometry techniques have gained some acceptance in density monitoring of bridge deck concrete and fairly widespread acceptance for density monitoring of roller-compacted concrete pavement and structures. Summary of the advantages and limitations of backscatter and direct transmission gamma radiometry techniques is given in Table 14.1.

TABLE 14.1. ADVANTAGES AND LIMITATIONS VARIOUS GAMMA RADIOMETRY TECHNIQUES

Technique	Advantages	Limitations
Gamma radiometry for density	Technology well developed; rapid, simple, rugged and portable equipment; moderate initial cost; minimal operator skill	Requires license to operate; requires radiation safety program
Backscatter mode	Suitable for fresh or hardened concrete; can scan large volumes of concrete continuously	Limited depth sensitivity; sensitive to concrete's chemical composition and surface roughness
Direct transmission mode	Very accurate; suitable primarily for fresh concrete; low chemical sensitivity	Requires access to inside or opposite side of concrete

14.2. MOISTURE GAUGES

14.2.1. Fundamental principles

Moisture gauges consist of a source of neutron radiation, which irradiates the material under test. As a result of radiation, gamma rays are created and detected. The result is a series of counts, which are a measure of the composition of the concrete. The sources used to generate the neutrons produce fast neutrons, which are scattered by the various elements in the material under test losing energy and changing direction after every collision. Neutron radiometric procedures usually employ a source/detector configuration similar to that used in gamma backscatter probes, as in Fig. 14.2B. The probe might contain a 100 mCi fast neutron source (^{241}Am/Be) and a gas filled BF_3 or ^{3}He detector. Because the detector is almost totally insensitive to fast neutrons, no shielding is employed between it and the source. The response of a neutron radiometry gauge arises from a much larger volume of concrete than does that of a gamma backscatter gauge. For example, a neutron radiometry probe completely surrounded by concrete with a water content of 250lb/yd^3 (150 kg/m^3) will effectively be seeing the concrete up to 14 in. (350 ml) away; a gamma backscatter probe in the same concrete will be seeing the concrete no more than 4 in. (100 mm) away. Hydrogen atoms are the most effective scatterers of the neutrons and collisions with hydrogen atoms rapidly change neutrons from fast to slow. Neutrons with energies greater than 10 keV are described as “fast”, between 0.5 eV and 10 keV, as “epithermal”, and less than 0.5 eV as “slow”. A measurement of the number of slow neutrons present, therefore, serves as an indicator of how much hydrogen is present in a sample. Since the only hydrogen present in concrete typically is in water molecules, slow neutron detection can be used as a measure of water content in concrete.

Neutrons do not ionize the gas in a gas filled tube directly, but are absorbed by boron trifluoride or ^{3}He in a tube. The latter gases emit secondary radiation that ionizes the gas in the tube and produces electrical pulses. Gas filled neutron detectors are widely used in moisture gauges in agriculture and civil engineering applications.

14.2.2. Applications of moisture gauges

Although neutron radiometry is widely used in highway construction (on soils and asphalt concrete), in well logging, and in roofing rehabilitation, it is rarely used in testing concrete. Bhargava in 1969 used a neutron moisture gauge to measure the water content at three locations (top, middle, and bottom) of 6 in. × 12 in. × 5 ft. (150 mm × 305 mm × 1.5 m) mortar and concrete columns. In another study Lepper and Rodgers used commercially available neutron moisture gauges with the probe placed at the centre of 0.5 ft^3 and 1 ft^3 (14 and 28 dm^3) volumes (unit weight measures) of fresh PCC. They found that the gauges could establish water content to within ± 3 to 6% of the actual value at a 95% confidence level; the accuracy depended on the gauge model and the sample volume.

15. OTHER METHODS OF NDT

15.1. ACOUSTIC EMISSION

Acoustic emissions are microseismic activities originating from within the test specimen when subjected to an external load. Acoustic emissions are caused by local disturbances such as microcracking, dislocation movement, intergranular friction, etc. An acoustic signal travels to a number of piezoelectric transducers, which convert the acoustic signals (mechanical waveforms) to electric signals. A digital oscilloscope captures the electric signals. The time of arrival of the signal at each transducer depends on the distance of the transducer from the AE source. The source, frequency and amplitude of the AE events have been used to quantify the nature of microfracture in various materials. AE sources are determined by calculating the difference in time taken for the wave to arrive at the different transducers. The velocity of the waves in the specimen is determined using the ultrasonic pulse velocity method.

15.2. COMPUTER TOMOGRAPHY

Computer tomography (CT), also called computerized radioactive tomography, is the reconstruction of a cross-sectional image of an object from its projections. In other words, it is a coherent superposition of projections obtained using a scanner to reconstruct a pictorial representation of the object. Mathematical formulation of CT was performed by Radon in 1917, and was first used in medicine as a diagnostic tool after the invention of the X ray computed tomographic scanner by Hounsfield in 1972. A schematic representation of the basic parallel beam is shown in Fig. 15.1.

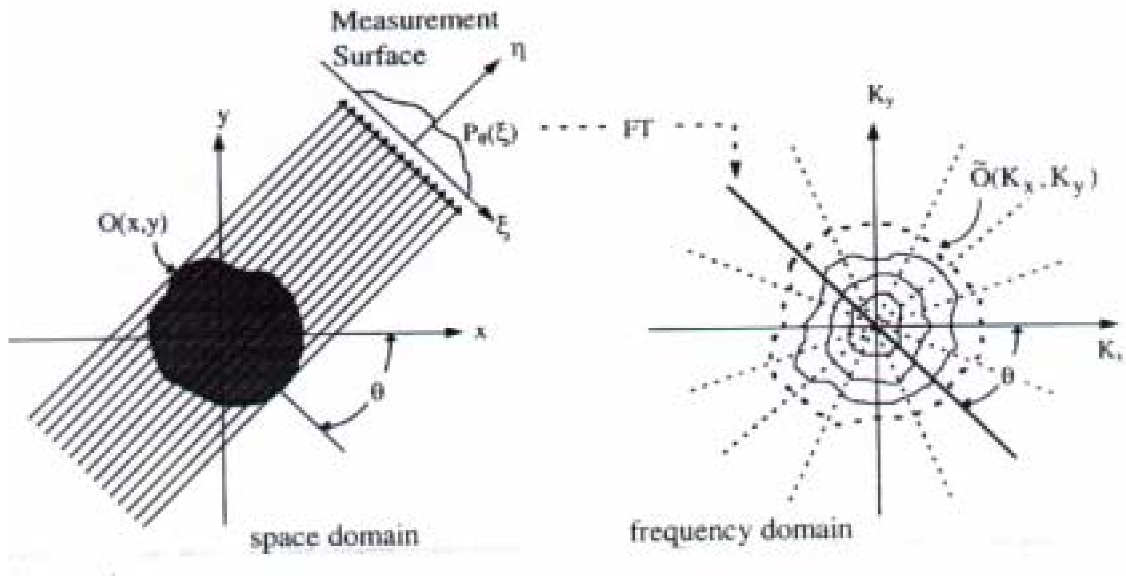


FIG. 15.1 Basic parallel beam computerized tomographic image reconstruction.

Fourier slice theorem states that the Fourier transform of a parallel projection of an image at an angle θ shown as $P_\theta(\xi)$ in the figure gives the two-dimensional spatial Fourier transform of the object $\check{O}(K_x, K_y.)$ along a line at an angle θ with the K_x axis. This line is shown as a solid line in the frequency domain. By rotating the source and detector 360° around the object, the Fourier space is filled, as shown by the dashed lines.

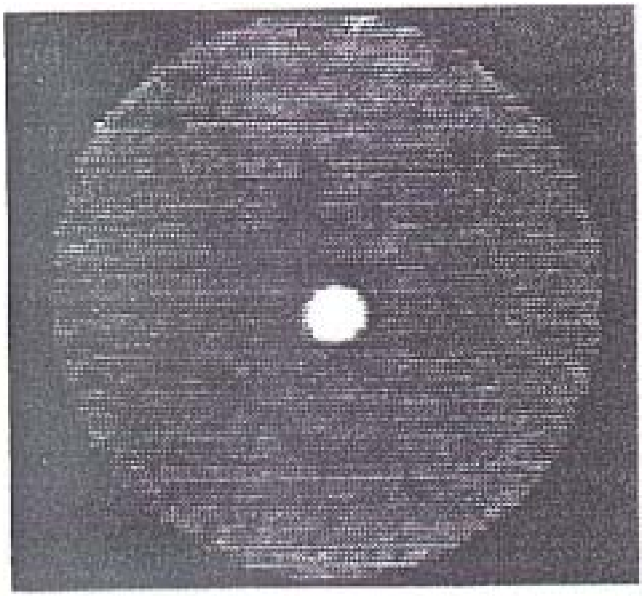


FIG. 15.2 (b). Computerized tomographic imaging of a concrete cylinder with a rebar at the centre.

At this point, the object can be reconstructed by a simple two dimensional inverse Fourier transform. The filtered back projection algorithm performs a better way of object reconstruction. This algorithm reconstructs the final image by first filtering each projection in the frequency domain, and then adding together the two dimensional inverse Fourier transform of each weighted projection. Similar reconstruction techniques exist for different sources, e.g. point sources that generate fan shaped beams.

In 1980, Morgan et al. developed a CT system that used an isotopic source to generate photon beams, and tested 6 inch diameter concrete cylinders to determine the density variations inside the cylinders, locate the reinforcement and voids, and determine their sizes. Image reconstruction was made using 100 projections obtained by rotating the source 360° around the cylinders. The exposure time for each projection was 40 min due to low source intensity. The system was able to identify the density within 1%. Results of two concrete cylinder specimens are shown in Figs. 15.2 (a) and (b). In Fig. 15 2(a) the reconstructed image of a concrete cylinder with a 3/8 inch diameter bar is shown. As seen from the figure, the rebar and the voids in the cylinder are accurately shown. Figure 15 (b) shows the image of a cylinder loaded to failure. The failure plane is clearly identified in the image.

A more recent application of CT to concrete is reported by Martz et al. They developed an X ray CT system to quantitatively inspect small concrete samples for density variations with a spatial resolution of about 2 mm. Fig. 15.3 shows an image of a 20 cm diameter hollow cylinder with a 4.4 cm central hole reconstructed from 45 projections at 4^{o} intervals over 180^{o}.

On the right of the cylinder image is a one dimensional attenuation profile extracted along a diagonal white line indicated on the image.

Computerized tomography is capable of producing highly accurate images of millimetre or sub-millimetre resolution. However, application of computerized tomography to concrete is generally limited to laboratory studies since the scanners are expensive, measurements take a long time and are limited to small sizes, and accessibility to both sides of the object is required. Image reconstruction from limited views has been the subject of several studies. However, such reconstruction still requires accessibility to both sides. Further research is needed in this area before the technique can be applied in the field.

FIG. 15.2 (a). Computerized tomography T of a cylinder loaded to failure.

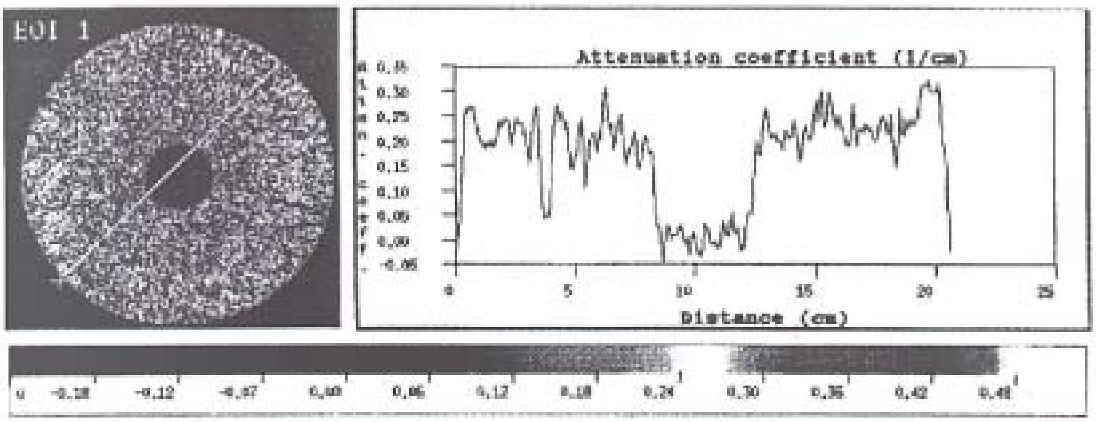


FIG. 15.3. Tomographic image of a concrete cylinder with a hole at the centre and one dimensional profile of attenuation coefficient along the white line.

15.3. STRAIN SENSING

Strain sensing is commonly used to monitor the reaction of a structure or structural elements when subjected to load, e.g. during a full scale load test. It is also employed to monitor the behaviour or rate of deterioration of structures in service, like crack growth. In mass concrete, such as dams and mat footing, strain gauges are utilized in conjunction with temperature sensors to monitor the development of stress arising from temperature (often encapsulated in a precast mortar ‘dog bone’) and, if adequately protected, is considered suitable for long term tests. However, it is not appropriate for dynamic tests because of its slow response time. Particular care is necessary to avoid magnetic influences, and a stabilized electrical supply is recommended for the recording devices.

15.3.1. Mechanical gauges

Among the various mechanical methods, the most commonly used equipment consists of a spring lever system coupled to a sensitive dial or digital gauge to magnify surface movements of the concrete. Alternatively, a system of mirrors and beam of light reflected onto a fixed scale or an electrical transducer may be used. A popular type, which is demountable, is known as a Demec gauge. The basic principle of this gauge is to measure the distance between two fixed points. Reference points are set using predrilled metal studs, which have been fixed onto the surface of the concrete. This method is widely used in monitoring cracks or joint movements. A simpler but much less precise method involves the use of two plastic or glass strips that are allowed to slide over one another. The top strip is transparent and both are imprinted with grids that allow the measurement of the degree of movement and direction. One such commercially available system is the Avongard Crack Monitor.

15.3.2. Electrical resistance gauges

The most common electrical resistance gauge is of the metal or alloy type in the form of a flat grid of wires, or etch-cut copper-nickel foil, mounted between thin plastic sheets. This gauge is stuck to the test surface, and strain is measured by changes in the electrical resistance resulting from stretching and compression of the gauge. The mounting and protection of gauges are critical, and the surface must be totally clean of dirt, grease and moisture as well as laitance and loose material, which may inhibit proper adhesion of the gauges. Care must also be taken in applying the adhesive to ensure that the adhesive is applied in a thin layer, free from bubbles and is cured completely particularly in cold weather. The relationship between strain and resistance will change with temperature, and gauges may be self-compensating or incorporate a thermocouple. Alternatively, a dummy gauge may be used. Gauges must be sited away from draughts and direct sunlight although changes in ambient temperature will not normally affect readings over a short time period. Humidity will affect gauges so they must be waterproofed if they are to be subjected to changes in humidity, to be embedded in concrete or to be in use for a long period. Suitably encapsulated gauges can be cast into the concrete. The use of these gauges requires considerable care, skill and experience if reliable results are to be obtained. Their fatigue life is low and this, together with the long term instability of gauge and adhesive, limit their suitability for long term tests. The strain capacity will also generally be small unless special 'post yield' gauges that have high strain limit are used.

15.3.3. Acoustic gauges

Acoustic methods are based on the principle that the resonant frequency of a taut wire will vary with changes in tension. A tension wire is sealed into a protective tube and fixed to the concrete. The wire is excited by an electric pulse passed through an electromagnet situated close to the centre of the wire. The same magnet can then be used to detect and transmit the vibration to a frequency measuring device. This type of gauge may be cast into the concrete (often encapsulated in a precast mortar 'dog bone') and, if adequately protected, is considered suitable for long term tests. However, it is not appropriate for dynamic tests because of its slow response time. Particular care is necessary to avoid magnetic influences, and a stabilized electrical supply is recommended for the recording devices.

15.3.4. Electrical displacement transducers

In electrical displacement transducers, the movement of an armature is measured by its effect on the properties of a circuit in the main body of the transducer fed by a high frequency AC supply. Different principles are involved using changes in resistance, capacitance, inductance etc. Each type of transducer requires compatible electronic equipment to provide the necessary input and convert the output to a reading. The body of the gauge is fixed to the concrete of a reference frame, so that the armature bears upon a metal plate fixed to the concrete. In this way, lateral or diagonal as well as longitudinal strains can be measured. An adjusting mechanism allows zeroing of the equipment. These gauges are particularly sensitive over small lengths, but much expensive, complicated electrical equipment is needed to operate them and to interpret their output.

15.3.5. Other types of strain measuring devices

Besides the above four basic categories of strain sensing techniques, there are several other methods that have found uses in concrete. They include the photoelastic method, piezo-electric gauges and fibre optics.

The photoelastic method involves a transparent thin photoelastic coating with a reflective (mirror) backing stuck to the concrete face. Polarized light is directed at this surface and fringe patterns show the strain configuration at the concrete surface under subsequent loading. It is very difficult to obtain precise value of strain from this method and requires calibration of the photoelastic coating. This method can prove useful in examining strain distributions or concentrations at localized critical points of a member.

In piezo electric gauges, the electrical energy generated by small movements of a transducer crystal coupled to the concrete surface is measured and related to strain. This is particularly suitable if small, rapid strain changes are to be recorded, since the change generated is very short lived. These gauges are more likely to find applications in the laboratory rather than on site.

Recently, fibre optic strain sensors have attracted more attention than any other method for strain measurement owing to its inherent advantages of small size, high accuracy, fast response, resistance to corrosion, broad range of measurement and complete immunity against electromagnetic and radio frequency interference. In addition, due to material inhomogeneities, the exact location of cracks or high strain in a concrete structure cannot be predicted. Conventional point sensors, such as those described previously (except for the photoelastic method), which measure strain at a local point, can easily miss a crack. The use of integrated sensors that measure displacement between two points separated by relatively long distance cannot distinguish between the harmless case of many fine cracks and the undesirable situation of one widely open crack. These limitations can, however, be overcome by using the fibre optic technique. One commonly used sensor is based on interferometry principles. A light signal is launched at one end of an optical fibre of a given length, passes through the cavity and reflects back to a readout system. A strain variation results in a change of cavity length thus a change of reflected light path. The change of cavity length can be determined instantaneously by the analysis with the special readout unit. Thus strain in the axial direction of the sensor can be obtained immediately. Either embedded or surface type sensors can be used to continuously monitor various structural parameters such as strain, crack formation, stress, etc.

15.3.6. Choice of strain measuring method

The non-homogeneous nature of concrete generally excludes small gauge lengths unless very small aggregates are involved. A gauge length of less than four times the size of the largest aggregate particle is likely to be affected by local variations in the mix. For most practical *in situ* testing mechanical gauges are the most suitable, unless there is a particular need for remote sensing in which case vibrating wires may be more useful and more accurate. Electrical resistance gauges can be useful if reinforcement strains are to be monitored or for examining pre-cracking behaviour in the laboratory. Such gauges are usually associated with smaller gauge lengths offering greater accuracy than mechanical methods. However, these gauges are not reusable. Mechanical gauges, on the other hand, have the advantage of not being damaged by crack formation across the gauge length. For dynamic tests, electrical resistance or transducer gauges will be most suitable, although operation of the latter will generally be more complicated and expensive.

In all the electrical based methods, precautions are advised to avoid stray electromagnetic fields. Photoelastic and piezo-electric methods have their own specialist applications as outlined.

Numerous research and development works are currently carried out to realize the best utilization of fibre-optic sensors. They have proved to be particularly useful for monitoring large civil structures such as bridges, dams, tunnels and composite structures or those structures in high electromagnetic or radio frequency interference environments as well as for normal steel and concrete buildings.

Instrumentation for short term static test instrumentation will generally be simple. For long term monitoring or dynamic tests, measurement devices must be carefully selected. The effects of ageing and temperature on the performance of the gauges have to be considered. Certain gauges, by themselves, may be liable to physical damage or deterioration. Humidity and electrical instability of gauges must also be weighed. Some adhesives used in mounting gauges are subject to creep and chemical deterioration. Selection must also take into account the anticipated strain levels and orientation, as well as the gauge length and accuracy required.

It should be borne in mind that a strain gauge measures the strain in the gauge length only. Care is necessary in locating the gauges in or on the concrete. The effects of local strain concentrations and gradients should also be considered.

For measuring strains in different directions at the same location, electrical resistance gauges are available in the form of rosettes and can be used as single two way devices or single three way devices, depending upon whether or not the principal strain axes are known. Similar arrangements can be contrived with other gauges.

15.4. CORROSION RATE MEASUREMENT

Various techniques are available to estimate the corrosion rate of steel in concrete. These include linear polarization resistance (LPR) and AC impedance. LPR has been widely used in the laboratory and is now being applied in structures. The LPR equipment is now commercially available for non-destructive testing of concrete.

The basic principle of LPR is to measure the corrosion current which gives an indication of how quickly a known area of steel is corroding. The amount of steel loss during the corrosion process can be measured to a certain degree of accuracy by means of the measurement of the electrical current generated by anodic reaction and consumed by the cathodic reaction. There is a direct relationship between the measured corrosion current and the mass of steel consumed by Faraday's Law. Corrosion current can be derived indirectly through half-cell potential measurement through the following expression:

$$I_{corr} = B / R_p \qquad (47)$$

where

I_{corr} iss the change in current ($\mu A\ cm^{-2}$),
B is a constant relating to the electrochemical characteristics of steel in concrete,
R_p is the polarization resistance expressed as R_p = (change in potential)/(applied current).

It deserves emphasis that this linear relationship is only valid when the potential change is kept less than 20mV.

In practice, an electrical current is applied to steel and its effect on the half-cell potential is monitored. There are two ways to measure it: either galvanostatic (steady applied current) or potentiostatic (one or more fixed target potential). Alternatively, the obtained R_p can be used to calculate corrosion rate by substituting in the following equation:

$$x = (11 \times 10^6 B) / (R_p A) \qquad (48)$$

where

x is corrosion rate ($\mu m\ yr^{-1}$),
A is surface area of steel measured.

One commercially available system with the function of defining the surface area of steel (A) has been developed. It consists of a half-cell to measure potential, an auxiliary electrode to pass the current, and a guard ring around the electrode to confine the area of impressed current. The interpretation is as shown in Table 15.1 (Broomfield *et al*., 1993, 1994): for corrosion monitoring in existing structures, half-cell and auxiliary electrodes can be installed for exposure in a part of the structure. Alternatively, measurement can be performed on an isolated reinforcement bar with a well-defined surface area in a controlled environment. LP measurement is very useful in finding the true condition of corrosion in a structure although it is slow compared with the half-cell potential test. However, various factors may affect the corrosion rate measurement. Changes in temperature have a direct influence on the corrosion rate as the heat energy will affect the oxidation reaction as well as concrete resistivity and relative humidity (RH). RH level in the pores can affect the access of oxygen.

TABLE 15.1. CORRELATION BETWEEN CHANGE IN CURRENT WITH THE CORROSION CONDITION

I_{corr} (μA cm^{-2})	Corrosion condition
< 0.1	Passive
0.1 < I_{corr} < 0.5	Low to moderate corrosion
0.5 < I_{corr} < 1.0	Moderate to high corrosion
> 1.0	High corrosion rate

To minimize temperature effects, measurements can be carried out at different times within a day and a year. It has also been proven that pitting may complicate the readings of corrosion rate measurement since the linear polarization devices cannot differentiate between pitting and general corrosion. Another important factor influencing measurement is the calculated surface area of steel under survey. Although the commercial device using guard ring system is available, it is reasonably accurate only at high corrosion rate. In most cases, assumptions about the surface area of steel have to be made and may lead to significant errors.

16. METHODS OF SURVEY

16.1. INTRODUCTION AND FUNDAMENTAL PRINCIPLES

Civil engineers or building surveyors may be asked to carry out an inspection of a concrete structure to assess its condition. The request may be caused by doubts about the safety of the structure, because of damage to the structure or age of the structure. On other occasions there may be a proposal to carry out alterations to the structure, for instance by making new doorways or windows in a building, or extending the building. In such situations it is necessary to tackle the inspection of the structure in a planned manner. Typical procedures are given in:

"Engineering for Improving the Durability of Reinforced Concrete Structure" prepared by the Engineering Survey Section, Ministry of Construction Secretariat, Foundation Engineering Research Centre of Land Development, Published by Gihodou Shuppan, Japan, and

"Procedure for Old and Damaged Buildings" Document No: II/NDT/BB01, Malaysian Institute for Nuclear Technology Research.

It is usual in such inspections to begin by gathering as much information about the structure as possible and then by visually inspecting the structure. This is followed if considered necessary, by further investigation of any areas of severe deterioration. This investigation focuses on the extent of carbonation of the structure, the extent of corrosion of the reinforcement and theoretical remaining concrete life. The final investigation, again if considered necessary, examines in greater detail the extent of any cracking in the structure, the compressive strength of the concrete, extent of corrosion, etc. Finally an assessment can be made of the condition of the building and whether repair is an option.

16.2. METHODS AND INSPECTION TECHNIQUE REQUIRED

16.2.1. First survey (regular inspection)

It is important to find out as much information as possible about the structure. A typical list of information, which should be gathered is, as follows:

- date of survey
- name and address of building
- building's use
- date of construction
- no.of floors
- area of each floor
- type of construction
- span between beams
- kind of foundation
- designer
- building supervisor
- builder
- name of maintenance personnel
- environmental conditions (tropical, temperate etc}
- presence of vibration
- presence of chemicals

— presence of air conditioning
— distance from sea
— prevailing wind direction
— side of building closest to the sea
— average wind speed
— history of building use
— rxtensions or rebuilding carried out
— any repairs necessary
— any accident
— type of concrete used(cement, sand, aggregate, use of admixture
— design strength
— fabrication method.
—

Armed with this information the building can then be visually inspected. This may require the use of binoculars to view more inaccessible parts of the building. The visual inspection needs to concentrate on those areas that are most likely to show damage, namely column, beam and floor areas, and particularly those areas where tension occurs. This is the corner areas if the floor is inspected from above and the centre of the floor if the floor is inspected from below. Any cracks identified during this process are recorded. At the end of this assessment one can give a rough estimate of the condition of the building. If cracks, lifting, exfoliation, deteriorated surface and water leaking are found it may be necessary to carry out a second more detailed survey. This decision is usually made by grading the degree of deterioration. The degree of deterioration criteria given is shown in Table 16.1.

16.2.2. Second survey (specific/particular inspection)

A second survey is carried out if the degree of deterioration reaches Grade III. This survey determines the depth of carbonation of the concrete, extent of corrosion of the reinforcing bars, extent of any cracking, severity of water leakage, any deterioration of concrete strength, identification of any areas of excessive deflection and the identification of any areas of surface deterioration.

16.2.2.1. Carbonation test

Using the carbonation test described in Chapter 5 the depth of carbonation is determined at 4 or 8 points and classified, see Table 16.2, where D is distance from surface to first layer of reinforcing bars and MCD is the measured carbonation depth.

As shown in Section 5 theoretical carbonation depth (TCD) can be calculated, as follows:

$$\text{TCD}= y^{1/2}\,\frac{R(4.6x-1.76)}{\sqrt{7.2}} \tag{49}$$

A classification of the theoretical depth of carbonation is shown in Table 16.3. The degree of deterioration due to carbonation can then be classified as shown in Table 16.4.

TABLE 16.1. CRITERIA FOR ASSESSMENT OF DEGREE OF DETERIORATION

Kind of deterioration	Unit for classification	Grade I	Grade II	Grade III
Cracks along main bars	No. of 1 m crack lengths per $100m^2$	0	1-2	3 and over
Cracks along supplementary bars	No. of 1 m crack lengths per $100m^2$	0-2	3-4	5 and over
Cracks around openings	Number of cracks for 10 openings	0-2	3-4	5 and over
Mesh cracks	Area of meshed cracks as a %	less than 5%	5-10%	10% and over
Other cracks	No. of 1m crack lengths per $100m^2$	0-4	5-9	10% and over
Lifting	(Area lifted/area of side)%	Less than 1%	1-3%	3% and over
Exfoliation - only on finished layer	(Exfoliated area/area of side) %	0%	0-1%	1% and over
No explosion of bars	Number per 100^2	0	0	1% and over
Explosion of bars	Number per 100^2	0	0	1% and over
Deteriorated surface				
• stain on surface	no. per 100^2	0	less than 2	2 and over
• efflorescence	no. per 100^2	0		4 and over
• pop out	no. per 100^2	0		1 and over
• weakened surface	% of weakened	less than 1%		3% and over
• other stain	% of weakened	less than 1%		5% and over
Water leaking		no	no	yes
Abnormal structural movement or deflection		no	no	yes

TABLE 16.2. CLASSIFICATION TABLE FOR MEASURED CARBONATION DEPTH

Classification of carbonation	Outdoors or contact with soil	Indoors
A1	MCD<0.5D	MCD<0.7D
A2	0.5D<MCD>D	0.7D<MCD<D + 20mm
A3	D<MCD	D+20mm<MCD

TABLE 16.3. CLASSIFICATION OF THE THEORETICAL DEPTH OF CARBONATION FOR THE PURPOSE OF A SECOND SURVEY

Classification	Depth of carbonation
B1	MCD<0.5TCD
B2	0.5TCD≤MCD<1.5TCD
B3	MCD≥1.5TCD

TABLE 16.4. CLASSIFICATION OF DEGREE OF DETERIORATION DUE TO CARBONATION

Degree of deterioration	Classification
I-Minor	A1 and B1,A2 and B1, A1 and B2
II-Mild	A1 and B3,A2 and B2
III-Severe	A2 and B3, A3 and B1, A3 and B2, A3 and B3

It is also possible to calculate the remaining concrete life knowing the depth of carbonation and the age of the building since:

$$C = A\sqrt{t} \tag{50}$$

where

C is the measured depth of carbonation,
t is the current age of the building in years,
A is a constant with the units "rate of carbonation per year".

Thus,

$$A = \frac{C}{\sqrt{t}} \tag{51}$$

The following is an example of such a calculation.

Year of building construction	1965
Date of carbonation test	1995
Thus age of concrete	30 years
Carbonation depth measured indoor	2.4 cm (no finish coating)
Design cover over reinforcement	4.0 cm
Measured minimum cover over concrete	2.5 cm

$$A = 2.4/\sqrt{30} = 0.438 \tag{52}$$

Since corrosion will occur when minimum depth of cover + 2cm is reached,

$t = C^2/A^2$

$$= \frac{(2.5+2)^2}{0.438^2} = 105 \text{ years} \quad (53)$$

Thus, remaining life of the concrete = 105 – 30
= 75 years

16.2.2.2. Corrosion of reinforcing bars

An assessment of the extent of corrosion of the reinforcing bars is carried out by selecting a number of representative areas to survey. The intent of the survey is to determine the amount of concrete cover over the bar at the positions selected, establishing the type, diameter and direction of the reinforcing bar, assessing the condition of the reinforcing bar and checking the depth of carbonation and surface condition. The corrosion of the bars can be classified as in Table 16.5.

TABLE 16.5. CLASSIFICATION OF CORROSION OF THE BARS

Classification	Points	Condition of bar
I	0	Surface with mill scale, slightly rusted without stain on concrete
II	1	Bar has begun to rust, mill scale has begun to flake and there is some pitting
III	3	Whole bar surface has rusted and flaked
IV	6	There is a reduction of bar area.

The assessment of bar deterioration is given by:

$$\alpha = \frac{\sum_{i=1}^{i=4} \alpha_i n_i}{\sum_{i=1}^{i=4} n_i} \quad (54)$$

where

α is point allocated,
n is number of bars.

For instance, if 20 bars are investigated and they are assessed, as follows:

10 grade I bars = 10 × 0= 0

5 grade II bars = 5x1=5

3 grade III bars = 3 × 3 = 9

2 grade IV bars = 2 × 6 = 12

Then,

$$\alpha = \frac{(0x10)+(1x5)+(3x3)+(6x2)}{20} = 1.3 \quad (55)$$

Table 16.6 shows a method of assessing the degree of deterioration based on the points calculated.

TABLE 16.6. METHOD OF ASSESSING DEGREE OF DETERIORATION BASED ON THE POINTS CALCULATED

Degree of deterioration	Points calculated
I (good)	$0 \leq \alpha < 1$
II (slight)	$1 \leq \alpha < 3$
III (medium)	$3 \leq \alpha < 4.5$
IV (serious)	$4.5 \leq \alpha < 6$

Table 16.7 gives guidance on whether the corrosion of the bars needs to be repaired and whether a third survey is required.

TABLE 16.7. GUIDANCE TO DETERMINE THE NEED TO REPAIR THE BARS OR CONDUCT A THIRD SURVEY

Degree of deterioration	Needs repair	Third survey required
I (good)	no	no
II (slight)	no	if necessary
III (medium)	yes	if necessary
IV (serious)	yes	if necessary

16.2.2.3. Assessment of cracks

If cracks have been classified as Grade III during the first survey, a second survey is required to determine the pattern, width and depth of the cracks and their cause. A hammer is used to sound the area around the crack. Dense concrete will produce a different sound from concrete containing a void underneath. If the crack is only in the coating on top of the concrete no further assessment is required. However, if the crack runs into the concrete and is not just a surface crack, the crack width is measured. If necessary, evidence of the location of the crack can be recorded on a photograph. The grade of the crack can be assessed using Table 16.8.

TABLE 16.8. GUIDE TO ASSESS GRADE OF CRACK

Crack severity	Crack width in mm Outdoor crack	Crack width in mm Indoor crack
I	<0.05	<0.2
II	0.05~0.5	0.2~1.0
III	>0.5	>1.0

The need to repair the crack or the requirement to proceed to a third survey depends upon the degree of its severity as indicated in Table 16.9.

TABLE 16.9. GUIDE TO DETERMINE NEED TO REPAIR CRACK AND CONDUCT THIRD SURVEY

Degree of deterioration	Characteristics of crack	Need to repair	Need for third survey
I	crack not growing	no	no
	crack growing		yes
II	crack not growing	yes	no
	crack growing		yes
III	crack not growing	yes	no
	crack growing		yes

16.2.2.4. Assessment of evidence of water leakage

The second survey of water leakage requires the surface coating to be removed and the area containing the watermark to be measured. The area is then inspected both on a wet day and a fine day to establish the cause of the watermark. The degree of deterioration due to water leakages is classified as sown in Table 16.10. Furthermore, the decision whether repair is required or a third survey is necessary depends on the degree of deterioration as indicated in Table 16.11.

16.2.2.5. Deterioration of concrete strength

NDT is used during the second survey to assess the deterioration of concrete strength. The areas surveyed are those assessed during the first survey as being suspect. For comparison some sound areas are also selected. Three NDT techniques are used:

— rebound hammer test
— pulse velocity measurement
— pullout test.

REBOUND HAMMER TEST

The surface layer on the area to be checked is first removed then the concrete is smoothed using a carborundum whetstone. About 20 points are marked in the test area with a minimum of 25 mm spacing. No point should be marked any closer than 30 mm from a corner and the test area should be greater than 100 mm × 100 mm. The average of all 20 test points is taken and any reading greater than ± 20% of the average is discarded. The test is repeated until all 20 tests are less than ± 20% of the average.

PULSE VELOCITY MEASUREMENT

Pulse velocity is measured as described in Section 11.1.

TABLE 16.10. CLASSIFICATION OF DETERIORATION DUE TO WATER LEAKAGE

Degree of deterioration	Outdoor area (under cover)	Inside room (water used in area)	Inside room (no water used in area)	Exposed outdoor area (no scaffolding requested)
I (good)	-	-	Mark dried	-
II (slight)	Mark dried	Mark dried	-	Mark dried
III (medium)	Mark wet	Mark wet	Mark wet	Mark wet
IV (serious)	Water leaking	Water leaking	Water leaking	Water leaking

TABLE 16.11. GUIDE TO DETERMINE IF WATER LEAKAGE AREAS NEED TO BE REPAIRED AND THIRD SURVEY REQUIRED

Degree of deterioration	Need for repair				
	Outdoor area (under cover)	Inside room (water used in area)	Inside room (no water used in area)	Exposed outdoor area (no scaffolding requested)	Need for third survey
I (good)	-	-	no	-	no
II (slight)	no	no	-	yes	no
III (medium)	yes	yes	yes	yes	yes
IV (serious)	yes	yes	yes	yes	yes

PULLOUT TEST PROCEDURE

Drill the area to be tested using a 15 mm diameter drill and 35mm depth. Install the plug. The pullout strength and compressive strength of the concrete is estimated using the following formula:

$$F_P = \frac{P}{A}(kg / cm^2) \tag{58}$$

where,

F_p is pullout strength,
P is proof stress of pullout,
A is side area of hole.
The compressive strength F_c of concrete = 97 F_p – 205.

Concrete can be graded based on its compressive strength as shown in Table 16.12.

TABLE 16.12. GRADE OF CONCRETE BASED ON ITS COMPRESSIVE STRENGTH

Degree of deterioration	% of the design strength
I (no deterioration)	100 and over
II (deteriorated)	75-100
III (severe deterioration)	Less than 75%

Assessment on whether repair is necessary and whether a third survey is necessary is shown in Table 16.13.

TABLE 16.13. GUIDE TO ASSESS THE NEED FOR REPAIR AND A THIRD SURVEY

Degree of deterioration	Need for repair	Need for third survey
I	No	Depends on necessity
II	Yes	Yes
III	Yes	Yes

16.2.2.6. Assessment of a large deflection

To assess an apparent large deflection it is necessary to quantify the deflection. This is done by measuring the bend in a beam or floor by determining the difference in level between either end of beam (or edge of the floor) and centre of the beam (or floor). This can be done with a surveying instrument (or a string line) and a measuring tape, or a straight edge and a measuring tape. The length of span of the beam or floor slab is also measured. If cracking is present in the beam or slab, the width of crack is measured with a suitable scale. The length of crack is also measured and recorded. If possible the time when cracking was first noticed should be obtained from people familiar with the building. This should be combined with questions about the building's loading history to determine whether the building has ever been overloaded. Also, the building's current loading should be estimated to assess whether it exceeds the design parameters.

The result of the deflection survey can be classified as shown in Table 16.14.

TABLE 16.14. DEGREE OF DETERIORATION DUE TO DEFLECTION AND CRACK

Degree of deterioration	Deflection/span	Width (mm) and total length of crack (m)
I (good)	Less than 1/300	<0.5mm and <6m
II (slight)	1/300 to less than 1/200	<1.5mm and <15m
III (medium)	1/200 to less than 1/100	<3mm and <20m
IV (serious)	1/100 and over	>3mm and >20m

In using Table 16.14 it is important to note that if a beam is being assessed, the width of the crack is the most important aspect and, if a slab is being assessed, the total length of crack is the most important aspect. In a situation where there is a difference in the assessed degree of deterioration between the rating with deflection/span and either the crack width or the crack length, the deflection/span ratio is the more important parameter. Further course of action depends upon the severity of defect as graded in Table 16.15.

16.2.2.7. Assessment of surface deterioration

The second survey of surface deterioration is carried out by selecting a few representative areas of each kind of deterioration present. These include:

— efflorescence
— stains (water and rust)
— lifting, separation and exfoliation
— rub off
— pop out
— weakness including disintegration.

By visually assessing the selected areas, a record is made of the area, depth and degree of deterioration. A classification of the deterioration is shown in Table 16.17, whereas the need to repair or for a third survey depends on its grade as indicated in Table 16.18.

TABLE 16.15. GUIDE TO DETERMINE NEED FOR REPAIR AND A THIRD SURVEY

Degree of deterioration	Need for repair	Need for third survey
I (good)	no	no
II (slight)	yes	no
III (medium)	yes	depends on necessity
IV (serious)	yes	yes

TABLE 16.17. CLASSIFICATION OF DETERIORATION

Degree of deterioration	Description
I (good)	Deterioration is noticeable but it is only a small area and there is no danger of the area falling.
II (medium)	Area of deterioration is large, however, in only some areas is the depth of penetration up to 20 mm.
III (serious)	Loss of cross-sectional area is large with the depth of deterioration reaching the reinforcing bars. Rate of progress of deterioration is estimated to be fast.

TABLE 16.18. GUIDE TO DETERMINE NEED FOR REPAIR OR THIRD SURVEY

Grade	Assessment of future progress of deterioration	Need for repair	Need for third survey
I	Dormant – will not progress further	No, except if required to improve appearance	No
I	Will progress further	Yes	Depends on necessity
II	- do -	Yes	- do -
III	- do -	Yes	- do -

16.2.3. Third survey

A third survey is necessary if the concrete condition is very severe.

16.2.3.1. Corrosion of bars

If it is decided after the second survey that a third survey of the corrosion of reinforcing bars is necessary a selection is made of areas of normal concrete as well as defective areas containing cracks, construction joints, cold joints, honeycombing, rust staining and exfoliation. Ten positions are selected for each condition or four to six bars in an area of about 500 mm × 500 mm are surveyed.

As for the second survey the following are assessed using the same tables for corroded bar: classification of deterioration, presumption of cause, remaining life and need for repair. In addition, salt content of the concrete is measured.

16.2.3.2. Cracking

Areas are selected containing large growing cracks discovered during the second survey stage. Investigation involved the following:

- width and growing status of the cracks are checked at intervals of 6 months to 1 year
- degree of corrosion of the bars in the cracked area is established using the same tables as for the second survey
- extent of carbonation is determined
- depth of the crack is established using an ultrasonic technique or by extracting a core from the cracked area.
-

A core is extracted from an area where Schmidt Hammer tests have been carried out to establish the compressive strength of the concrete. The core is analysed to establish as much information as possible about the concrete used, e.g. aggregate type, mix proportions, degree of compaction, etc. A proof load test is applied on the floor slab and the width of the crack monitored during the test.

The structure is checked for differential settlement by checking the floor, windows etc.

16.2.3.3. Water leakage

If the second survey into water leakage showed that a third survey was necessary or if it was not possible to reach the area where water leakage is occurring without scaffolding, the third survey is carried out. This may require scaffolding to be erected if previous access was not possible. Essentially the third survey requires the design drawings to be checked for the possible source of the water. From a practical point of view it may be possible to detect the source of the water by colouring the water in the possible sources. The concrete in the area of the leakage can also be removed to determine if the reinforcing is being corroded. Once the reason for the leakage is determined the concrete can be repaired.

16.2.3.4. Large deflection

In this case the third survey is carried out to either

- determine the percentage of residual deflection, or
- determine the ratio of measured to calculated frequency of vibration of the member.

If the percentage of residual deflection is to be measured, the deflection of the member in the existing condition is first measured. The member is then put under tension by loading with for instance a known load. When the load is removed the deflection is again measured and the residual additional deflection established. A core needs to be taken from the member before the load test to determine the compressive strength of the concrete.

If the ratio of measured to calculated frequency of vibration of the member is to be established, free vibration wave is measured by generating a shock wave and using a vibrometer to determine frequency. Knowing the dynamic force applied and the dimensions of the member the theoretical frequency of vibration can be calculated.

The result of those investigations can be assessed using Table 16.19. Whether the results justify repair or not can be judged using Table 16.20.

TABLE 16.19. GUIDE TO ASSESS THE RESULT OF INVESTIGATIONS

Degree of deterioration	Ratio = frequency measured/frequency calculated	% of residual deflection
I	0.90 and over	Less than 15
II	0.75 and over	15 and over
III	Less than 0.75	Ditto

TABLE 16.20. GUIDE TO DETERMINE IF RESULTS JUSTIFY REPAIR

Degree of deterioration	Need for structural analysis	Need for repair
I	no	no
II	yes	yes
III	yes	yes

16.2.3.5. Surface deterioration

If the need for a third survey has been established, additional information is required

— On the weakness of the surface of the concrete
— On any deterioration of the compressive strength of the concrete
— On the depth of carbonation of the concrete
— On the depth of deterioration.
—

Assessment of the deterioration after further investigation can be obtained using the table below.

TABLE 16.21. GUIDE TO ASSESS DETERIORATION AFTER FURTHER INVESTIGATION

Degree of deterioration	Description
I (good)	Deterioration is noticeable but it is only a small area and the depth is 10 mm or less.
II (medium)	Area of deterioration is large, however, depth of penetration up to 20mm is true only in some areas.
III (serious)	Loss of cross-sectional area is large with the depth of deterioration reaching the reinforcing bars or else the depth of penetration is 20 mm or less. However, the reason for deterioration is unknown or the progress of deterioration estimated to be fast and is progressing quickly.

16.3. CONCLUSION

One of the most important parameters that determine the safety of a building is its strength. In all cases, if the investigation finds the strength of concrete is less than the design strength, the result needs to be presented to the engineer in charge (civil engineer/structural engineer) who must make a decision based on the results presented as well as other considerations.

17. CASE STUDIES

17.1. RADAR TESTS ON CONCRETE BRIDGES

In order to achieve an overview of the most popular commercial radar systems in the market, measurements of laboratory models and pre-stressed bridges were done. Following are reports on the field tests carried out on bridges in Frankfurt and Oslo. The results or radargrams presented are given in alphabetical order by manufacture's name and do not imply a scoring.

17.1.1. Field measurements, Frankfurt, Germany

Installation of suspended scaffolding was required to recondition a pre-stressed bridge over the river Main. In order to avoid damage to the transverse tendons by the drilling work, HOCHTIEF located the transverse tendon ducts using radar. Comparison measurements were also made with the ERA, GSSI and MALA commercial radar systems using a 1 GHz antenna. On the underside of the bridge, the same ~4 m long profile was recorded with each system. The GSSI system was operated by HOCHTIEF, the ERA system by the German ERA distributor and the MALA system by an employee of MALA. The results are shown in Fig. 17.1 using the visualization software REFLEX (Sandmeier, 1996).

In Fig. 17.1 all radargrams are plotted with the same time scale of 8 ns, but they differ in the data acquisition mode applied. The GSSI and MALA measurements were triggered by a survey wheel, while the ERA measurements were speed controlled. Fig. 17.1 radargrams show a transition zone where the lateral distance of the tendon ducts changes from 25 cm at the beginning to 50 cm at the end of the profile.

If polarization is parallel to the tendon ducts, reflection of the mesh reinforcement (12 mm diameter and 4 cm concrete cover) dominates the radargram. With all systems the position of the reinforcement is easily detectable by the reflection hyperbolas between 1 ns and 2 ns. With perpendicular polarization the disturbing influence of the reinforcement is weaker and reflection hyperbolas of the tendon ducts more obvious (32 mm diameter and 15 cm concrete cover). Clear reflections of the tendon ducts are visible with each system especially at the end of the profile.

17.1.2. Field measurements, Oslo, Norway

On the Ranafoss bridge near Oslo, comparison measurements with four different radar systems were performed; these were the systems of GSSI and MALA using the 1 GHz antenna, of Sensors & Software using the 1.2 GHz antenna, and a frequency step system of the project partner NGI (Kong & Westerdahl, 1998). In order to compare the different radar systems under equal conditions, profile lines were marked at four defined positions on the pavement. The aim was to detect the longitudinal tendons, which had a diameter of 90 mm and a varying concrete cover from 9 cm up to 25 cm. The covering mesh reinforcement had a bar diameter of 20 mm and a mesh size of 12 cm.

Each system was operated by a project partner, who chose independently the best suited range, filter and gain settings. In Fig. 17.2 the radargrams recorded at position III are shown, using the original settings of each system. According to the blueprint, five or six tendon ducts should be visible in the second half of the radargram. The profiles start at the bridge edge and end at the car barrier. According to the technical drawings, five or six tendon ducts, depending on the recorded profile length, should be visible in the second half of the radargram. Clear reflection hyperbolas of the tendon ducts are not visible in any radargram. However, the structure of the radargram changes in the second half.

The interference of reinforcement reflections with the reflections of the tendon ducts has resulted in the radargrams being almost impossible to interpret in Fig. 17.2.

This interference, however, strongly depends on the position of the antenna in relation to the reinforcement. For instance, moving the antenna along a reinforcement bar has a different result from moving the antenna just between two bars. Furthermore, in any real structure the position of the reinforcement in relation to the tendon ducts varies. Many parallel profile lines have to be performed in order to achieve a more interpretable radargram. Fig. 17.3 demonstrates this successful strategy: It shows nine radargrams after a background removal, recorded with 10 cm lateral spacing. The quality of these radargrams varies greatly. Only in some radargrams are the reflection hyperbolas of the six tendon ducts visible in the right half of the radargram.

17.1.3. Conclusion

The presented field measurements on two pre-stressed bridges show that, to successfully locate tendon ducts, the most important factor is the experience of the measuring person. As measurements on laboratory models show, all the tested commercial radar systems perform similarly in detecting tendon ducts under a reinforcement mesh. Consequently, for successful tendon duct location in the field, the know-how of finding the best profile line, choosing the best range, gain and filter settings and the ability to interpret the data are more important than the system itself. The presented forward simulation program is a fast tool to obtain an impression of the expected radargram. It also helps to avoid interpretation errors due to signal interference.

17.2. BUILDING CASE STUDY: QUASI NON-DESTRUCTIVE STRUCTURAL CONDITION ASSESSMENT OF SELF PRE-STRESSED REINFORCED CONCRETE

17.2.1. Trusses at Berlin Tempelhof Airport

Fig. 17.4 shows a view of the spatial arrangement of the reinforced concrete trusses, which were examined in the airport hall at Berlin Tempelhof Airport.

The building is a three bayed structure, consisting of two side bays and a central bay. The central bay is spanned by reinforced concrete trusses, which are to be examined. The trusses have a support width of 32.5 m and a height of 4 m. Above the entire hall length of 108 m are 15 trusses arranged in six different types.

The position of reinforcement in such trusses is shown in Fig. 17.5. The tension reinforcement in the diagonal and lower girder members is very solidly emplaced, having a maximal bar diameter of 70 mm. The quality of the steel is St 52.

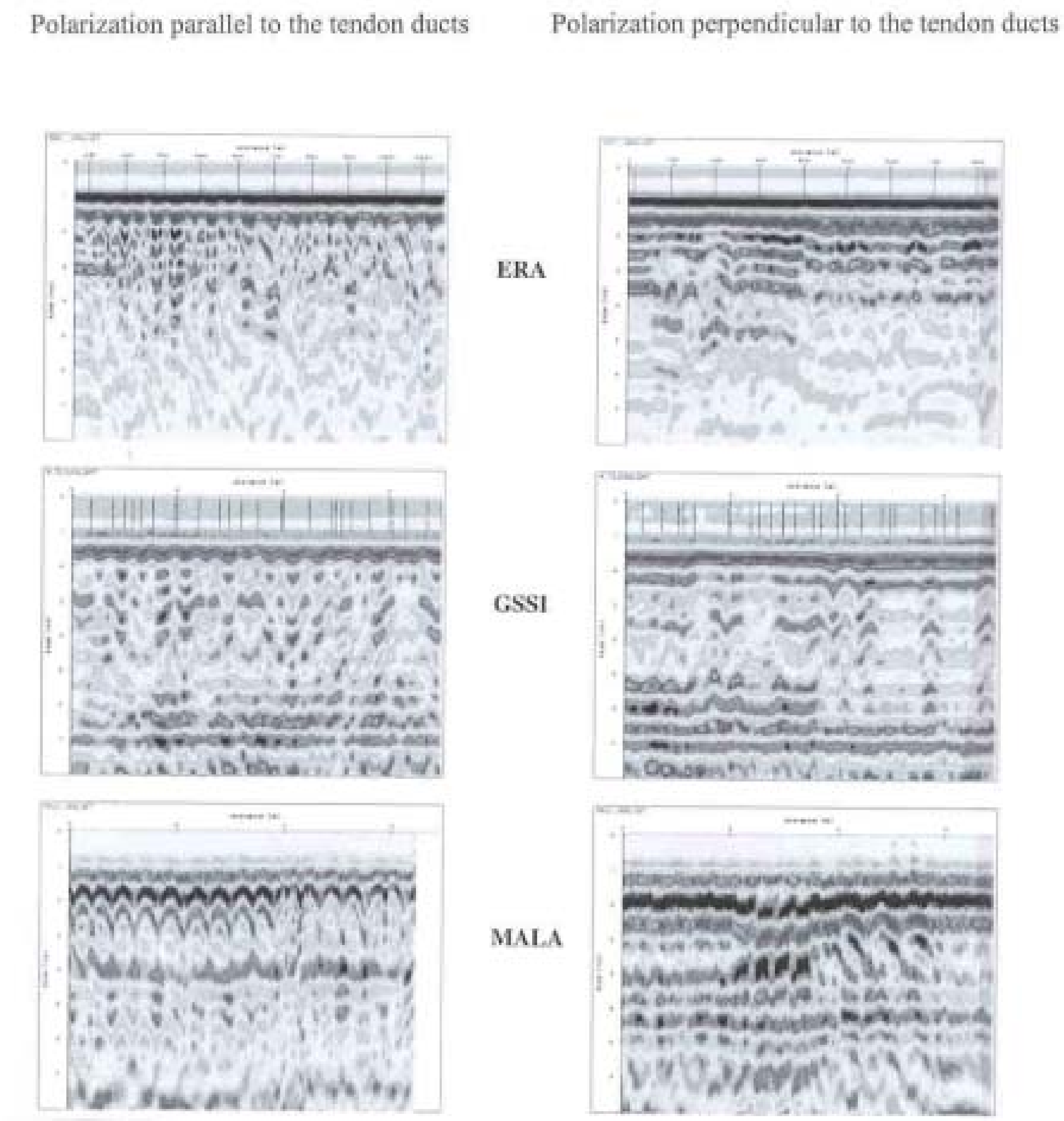


FIG. 17.1. Radargrams recorded on the underside of the bridge with the radar systems of ERA, GSSI and MALA using the 1 GHz antenna.

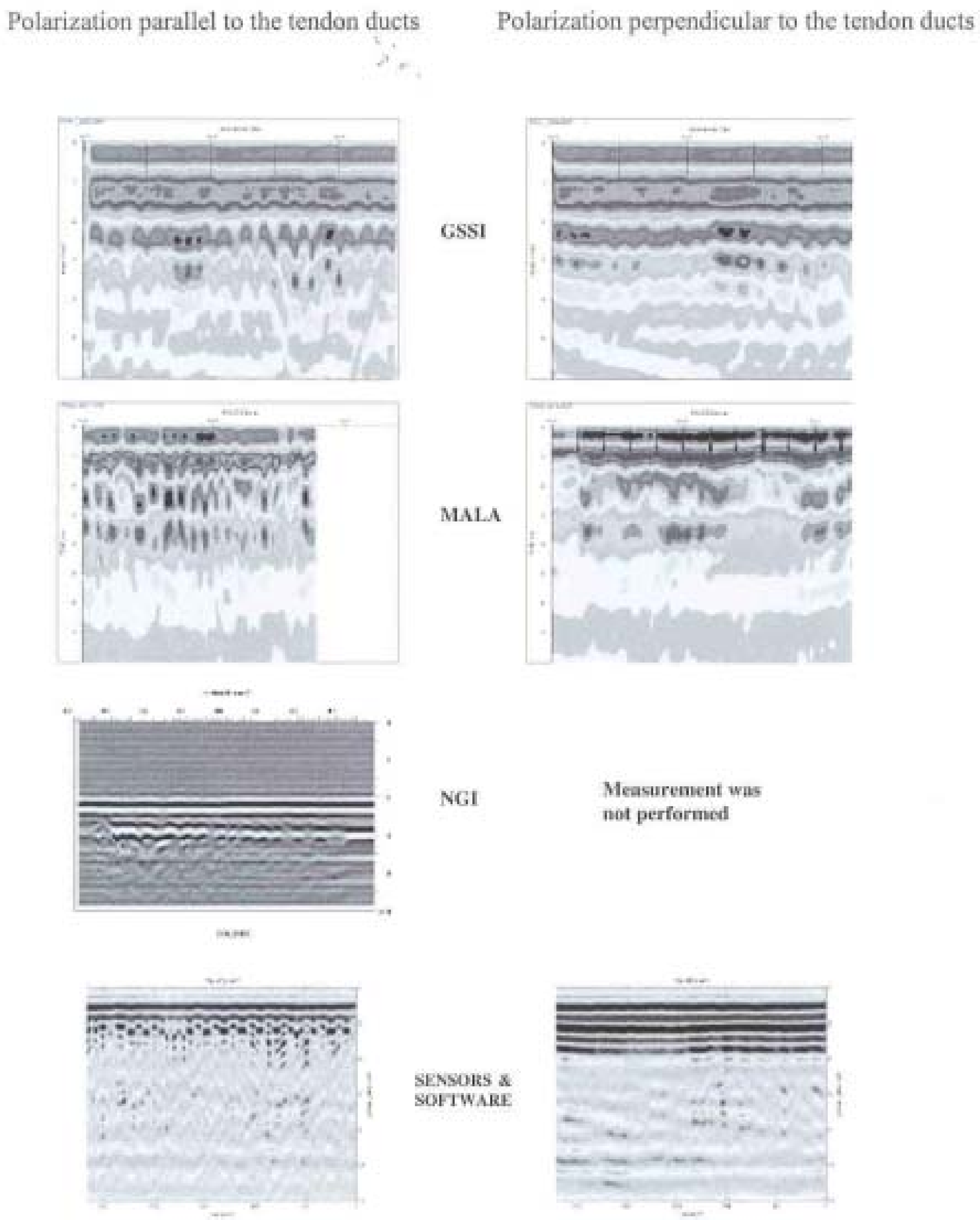


FIG. 17.2. Radargrams, recorded on Ranafoss bridge along the same profile line using different radar systems.

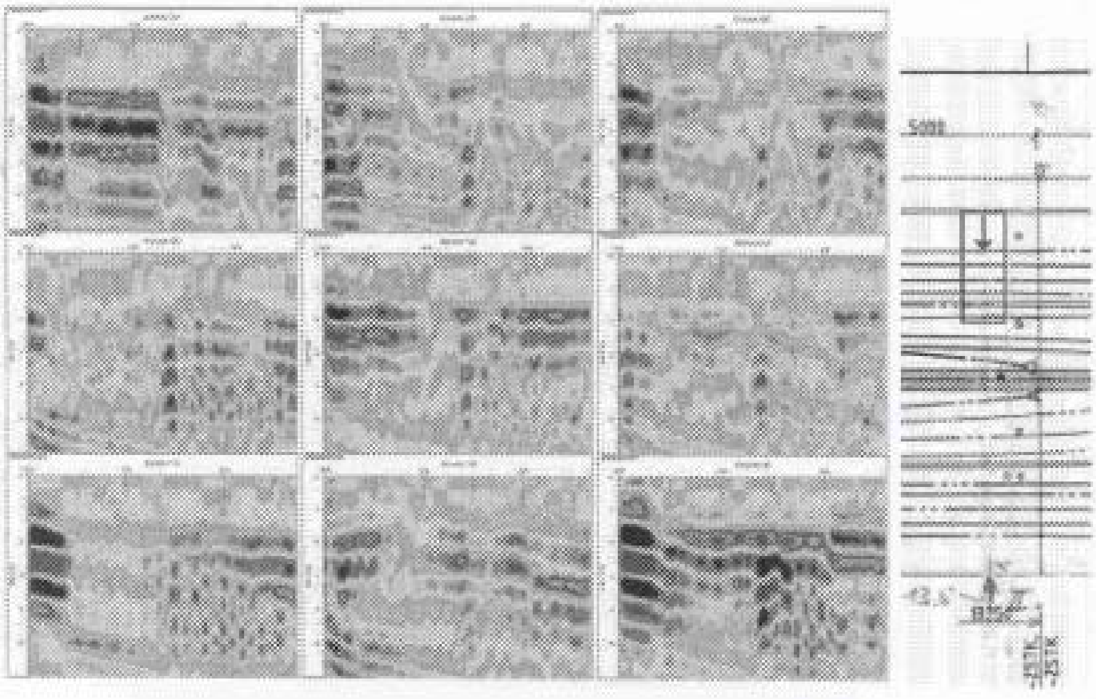


FIG. 17.3. Parallel recorded radargrams with 10 cm lateral spacing after performing a background removal. The appearance of the reflection hyperbolas of the six tendon ducts in the second half of the radargram (see blueprint) strongly depends on the chosen profile line.

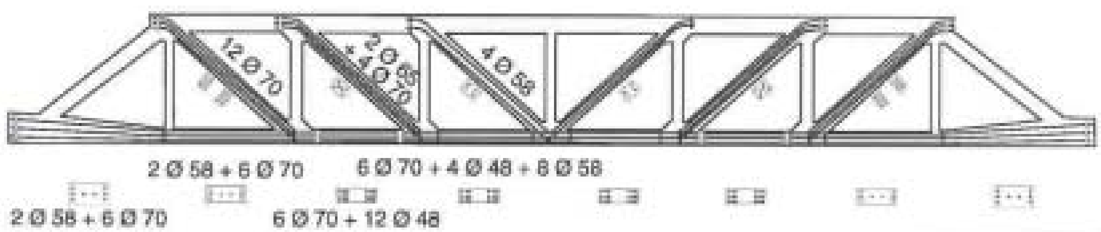


FIG. 17.4. Truss with reinforcement position.

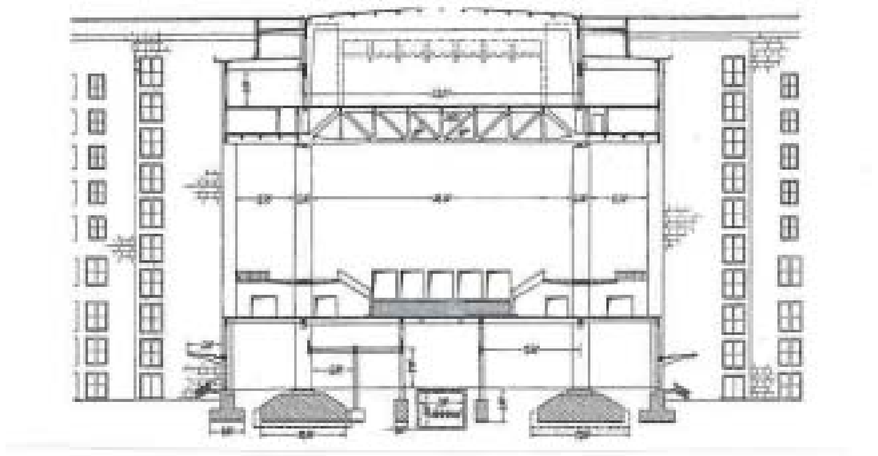


FIG. 17.5. Airport hall at Berlin Tempelhof Airport.

Problems occurred in the fabrication of such trusses (Finsterwalder, 1938), (Finsterwalder 1, 1938), (Schleusner, 1938). For example a major problem during truss fabrication was at the construction site, regarding the fabrication quality of the pressure welding bond in the large diameter rods. The reason for the present investigation was the planned employment of significantly heavier loads in the existing trusses. BAM was assigned in this context to develop a test programme for a non-destructive structural condition assessment and to partially conduct examinations on two selected trusses.

17.2.1.1. TEST PROGRAMME

The developed test programme includes examinations of the entire construction as well as on selected construction elements (Fig. 17.6). Additionally, mechanical characteristics on comparative steel samples were obtained to assess the steel and welding seam quality.

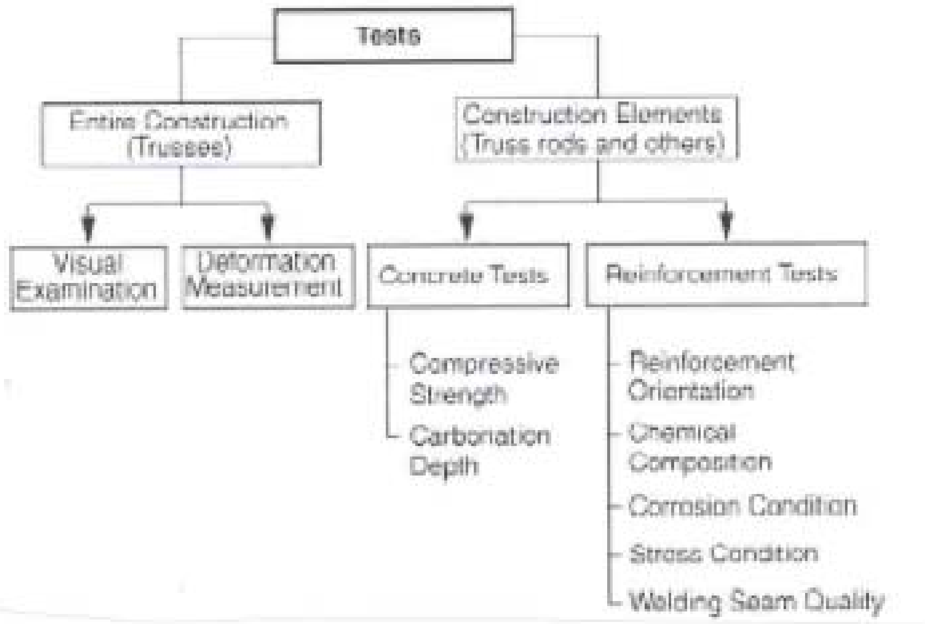


FIG. 17.6. Test programme.

17.2.1.2. VISUAL EXAMINATION AND DEFORMATION ANALYSIS

The tests on the entire construction allow a qualitative estimate of the load capacity of the trusses. During the visual examination on both trusses, no serious damage was detectable. The deformation-analysis measurement in the truss joints of the lower chord revealed no bowing due to fabrication inaccuracies.

17.2.1.3. CONCRETE TESTS

COMPRESSIVE STRENGTH

The structure and core samples were tested using the transmission mode ultrasonic method (DGZfP, 1993), (Schickert, Krause, & Wiggenhauser, 1991). For calibration the core samples were additionally subjected to a controlled load compressive test according to DIN 1048 Teil 2, 1991. The tests were limited to selected vertical rods and lower and upper girder members.

The ultrasonic velocity measured on the structure showed only slight variations and suggests a high concrete homogeneity. A comparison of the ultrasonic results with the average concrete compressive strength reveals that the vertical rods and concrete slabs in the area of the upper girder members display a relatively low ultrasonic velocity with a high concrete grade. This is indicative of a low dynamic modulus of elasticity caused by a high proportion of cement to gravel. The existing concrete grade strength is often greater than required. A threshold ultrasonic pulse velocity value of 3.5 km/s was obtained for the concrete.

CARBONATION DEPTH

The carbonation depth in the truss was determined using the phenolphthalein test to estimate the degree of corrosion protection. The carbonation depth was found to be only 1 mm in the vertical rods and the concrete slabs despite the 60 years service life. At the lower girder rods, however, this reached a considerably higher value of 26 mm due to the lower concrete grade of the tension members. This was nevertheless regarded as non-critical because of the larger concrete covering.

17.2.2. Reinforcement tests

17.2.2.1. Reinforcement orientation

The reinforcement orientation serves to determine the number and position of reinforcement steel and, on the other hand, it permits conclusions to be made about the load capacity of the truss by means of a comparison between the required and actual values. The test method primarily employed here was the electromagnetic alternating field technique (DGZfP, 1990), which is based upon the transformer principle. The radar method (Maierhofer & Funk, 1995) was also tried. This is based upon the reflection of short electromagnetic impulses by interfaces in the concrete due to the greatly different dielectric characteristics which occur particularly in the transition from cement to steel.

The following conclusions are drawn as a result of the examination of both trusses:

- The electromagnetic alternating field technique is better suited than the radar method to finding the position of reinforcing bars.
- The number and location of girder steel are statistically correct.

— Intermediate members were successfully located to allow the exposure of the welding joints.

17.2.2.2. Chemical composition

The determination of the chemical composition of the rod steel serves for the evaluation of the steel type to estimate weldability, stability, and corrosion resistance. As a test method, the spark-induced emission spectrum analysis (Brauner, Glaubitz & Kremer, 1980) was utilized in the structure and in the laboratory. This was performed on the exposed main reinforcement and crossbeams of a truss. The analysis revealed that the main reinforcement showed the following chemical properties:

— low C content (±0.2 M%), which is indicative of a good steel weldability
— high Mn content (ca. 1 M%), which results in a steel of fine grain size
— high Cu content (0.4 – 0.6 M%) and Cr content (ca. 0.4 M%), which confers high corrosion resistance
— high Si content (±0.4 M%), which is required for the oxygen binding of steel during the production process and consequently for its stabilization.

The chemical properties allow the conclusion (according to Dubbel, 1943 and Kuntscher, Kilger & Biegler, 1958) that the steel used for the reinforcing bars was a Mn-Cr-Cu grade St 52 steel which is in agreement with the steel specified. However, the crossbeams, should have been grade St 37 low alloy steel.

17.2.2.3. Corrosion condition

The determination of the corrosion state was restricted to a visual examination of the partially exposed main reinforcement on both trusses. No visual corrosion appearance with mass loss was observed in any area.

17.2.2.4. Stress condition

For the examination of the static load of the girder-rod reinforcement, the pre-stressing force was exemplarily measured on a steel rod and compared with the calculated value. As a test method, the static as well as the dynamic methods were used. Both methods required a 3 m length of the reinforcement steel to be exposed due to the large rod diameter of 70 mm. The exposed rod steel was then observed to be a stressed single field support having both ends embedded in concrete. Common to both methods is that the visual free length of rod steel LS is less than the effective free rod length LW. This is because the concrete embedment of the rod steel is not completely rigid in the peripheral area.

In the static test method (Weise, Porzig & Mayer, 1999; Mayer, 1978) the steel rod restrained on both ends in concrete is centrally loaded with a defined point load Pm, and then the deflection fm is determined in the beam centre. The ratio of the centrally located point load and the central deflection Pm/fm, which is also designated as elasticity constant cD, is thereby a measure of the pre-stressing force Kstat. Thus, at a constant point load and increasing deflection, a decreasing pre-stressing force can be concluded. To ascertain the statically effective free length LS, the deflections f1 and fr were additionally determined in the left and right peripheral areas of the exposed steel rod. The calculation algorithm for determining the pre-stressing force Kstat is shown in the following simplified flow diagram (Fig. 17.7).

With the dynamic test method (Weise, Porzig & Mayer, 1999), (Sato, Taniguti & Iwata, 1987) the single-field support having both ends under stress is stimulated into vibration with a rubber hammer. The initial eigen frequency of the oscillation v1 is a measure of the pre-stressing force K_{dyn}. This rises with increasing pre-stressing force. For the determination of the dynamically effective free length LS, another eigen frequency must certainly be measured. The functional correlation between the pre-stressing force K_{dyn} and measured values are here derived by solution of the differential equation of motion (Fig. 17.8).

With the static test method, an exemplarily examined rod steel here yielded an effective length *LW* of 2833 mm. This is 163 mm greater than the visual free span length *LS* of 2670 mm. Fig. 17.9 illustrates the determined functional interrelation between *cD* and $K_{stat.}$ Under consideration of measurement uncertainty, the pre-stressing force K_{stat} here is (271 ± 23) kN which corresponds to a strain of (69.4 ± 5.9) N/mm^2.

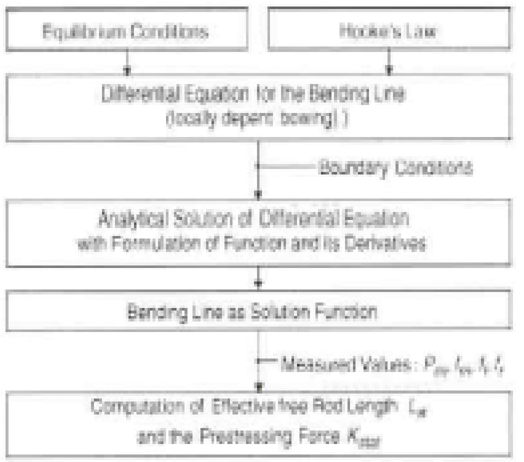


FIG. 17.7. Calculation procedure for developing the prestressing force K_{stat} *of the static test.*

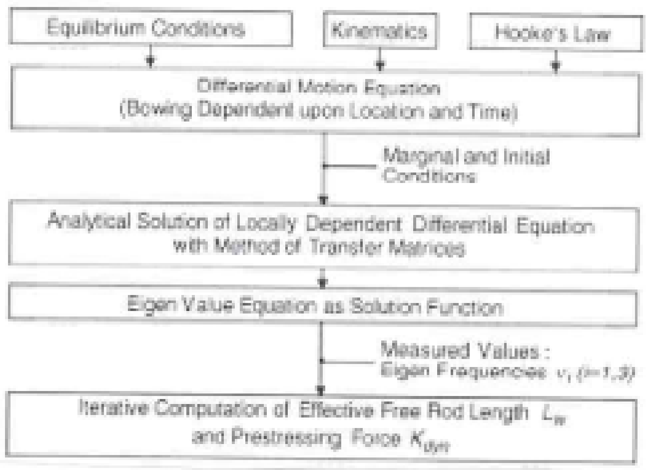


FIG. 17.8. Calculation for determining the prestressing force K_{dyn} *of the dynamic test.*

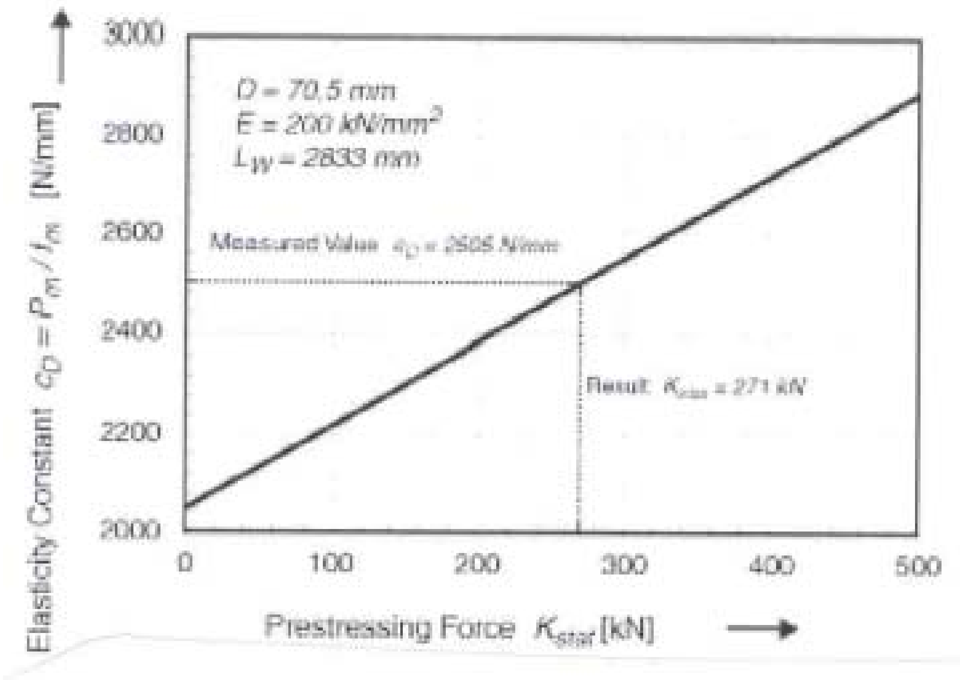


FIG. 17.9. Determining pre-stressing force K_{stat} *from force* Pm *and bowing under load fm.*

Based upon the measured eigen frequencies, the dynamic test method yields $v_1 = 43.35$ Hz and $v_3 = 217.20$ Hz plus the value D = 70.5 mm, E = 200 kN/mm^2 and _ = 7.85 g/cm^3, an effective length *Lw* of 2853 mm and a prestressing force of (75.1 ± 5.9) N/mm ^{2} . K_{dyn} = 293 kN with an uncertainty of ± 23 kN, corresponding to a stress σ of (75.1 ± 5.9) N/mm^2.

Despite both very different test methods, there is quite good agreement of the test results. Each single result still lies within the uncertainty area of the alternate test method. Coincidentally both tests result in the same measurement uncertainty. This was numerically determined by the theoretical relationships derived from the estimated uncertainties of the correlation parameters.

Under these conditions it is meaningful to give an average value. This is calculated as:

$$K = (282 \pm 16)\ \text{kN or } \sigma = (72.2 \pm 4.2)\ \text{N/mm}^2 \qquad (59)$$

The determined prestressing force therefore lies approximately 14% under the theoretical value of the static calculation of 328.5 kN. This permits the conclusion that the supporting capacity of the examined rod steel at the lower chord is not critical.

17.2.2.5. Welding seam quality

Based on the aforementioned problems with the fabrication of the pressure welding joints at the construction site between the large diameter rod steel, the welding seam quality deserves special attention. The selection of a suitable non-destructive test method for this is based upon (Merkblatt DVS 2922, 1991) and (DIN 8524 Teil 2, 1979). Following the exposure of the welding seams, the following non-destructive tests were conducted:

- visual examination
- magnetic powder test
- ultrasonic test
- radiographic inspection.
-

These exemplary tests were performed on five welding seams of two trusses.

VISUAL EXAMINATION

For a visual assessment of welding seam quality, the following criteria were applied:

- workability condition
- scoring
- welding seam elevation.
-

While all three welding seams of a truss displayed no visual objections, the outer appearance of both obviously unfinished welding seams in other trusses gives a reason for closer critical examination. Therefore, this raises the question whether the visual scoring can lead to a cross-sectional weakness of the rod steel. Following overwork of the elevations, no significant cross-sectional weaknesses were however discernible.

MAGNETIC POWDER TEST

This serves to locate near-surface proximity of welding seam defects perpendicular to magnetic flux (DIN 54130, 1974). In the present case, the welding seam surface was first smoothed. Then yoke magnetization followed with an AC hand magnet. As a test medium, magnetic powder with fluorescent substance and water as carrier was used. The test medium control ensued on an MTU controlling instrument. The test showed that all three welding seams of one truss revealed few indications while both welding seams of the other truss showed many indications. However, following the smoothing they all displayed no indications for a mandatory warning notice.

ULTRASONIC TEST

This test conducted in accordance with AD-Merkblatt HP 5/3 (1989) serves to locate surface flaws oriented perpendicularly to the rod axis. Because of the welding seam geometry, the test was performed with the angular test head in various testing techniques. In this way the mono, bistatic and transmission modes were applied. Equipment calibration was done on a specially constructed reference body. The recording limit for the welding seam flaws was set in accordance with DIN 54125 (1989) and DIN 54124 (1989). The tests revealed that the five examined welding seams for all test techniques showed no readings for a mandatory warning notice.

RADIOGRAPHIC INSPECTION

This serves to locate spatial volume flaws oriented in the beam path. In the present inspection, the tests were conducted in accordance with DIN 54111 Teil 1 (1988) in Test Class A. Iridium 192, having an activity of 1620 GBq, was used as the radiation source. The exposures were based upon the geometry of the rod steel in a dual film technique. The film-focus distance was 700 mm; exposure time was 60 min. For reduction of contrast loss due to scattered radiation, lead foil was partially placed upon the welding seam surface. During the inspection exclusively on one truss, the three examined welding seams displayed no findings for a mandatory warning notice.

TESTS ON COMPARABLE STEEL SAMPLES

It was not possible to have a sample of steel rods in the trusses; also, it would be desirable to obtain mechanical characteristic values to assess the steel and welding seam quality. Therefore, comparable steel samples were obtained on structures of the same age that have the same quality of steel and a similar welding junction. Three samples were made available from a reinforced concrete bridge in Aue, erected in 1937-38, and one sample of a tie rod from a similar bridge in Wiedenbruck. It can be concluded, based on the results of tensile tests on small and large samples (Weise, Porzig & Mayer, 1999), that rod steel possesses a sufficient component safety margin by compliance with the maximal allowable stresses of 160 N/mm 2 considered under the existing installation conditions of the trusses (static load, room temperature).

17.2.3. Summarized evaluation and outlook

The investigations performed on both trusses lead to the following conclusions:

- The test concept developed for quasi non-destructive analysis of structural integrity has proved to be reliable in practical operation. In further examinations of the trusses; however, the application of radar and radiography can be dispensed with, due to the limited available test data.
- The static test allows the determination of a stress condition in pre-stressed steel with sufficient accuracy, also with large cross-sections. The results of the static and dynamic tests show good agreement. The determined pre-stressing force is only 86% of the static value.
- All examined construction elements showed no defects of concrete or of the reinforcement.
- All five examined welding seams on both trusses possess sufficient load-bearing capacity.
- The additional examination of comparable steel samples show that the welding seam area of rod steel has a sufficiently high tensile strength, despite great lack of deformation as a consequence of thermally caused open-grain structure.

Based on the knowledge gained and in close co-ordination with truss planners, test engineers, and BAM, the Berlin Airport Association decided on the following stipulations for further action:

(1) The maximal permissible live loads in the rooms located above the trusses shall be limited depending upon existing static load capacity.
(2) The non-destructive tests on the entire construction shall be conducted for all trusses by:

- annual visual examinations
- deformation measurements (every 3 years)

Upon detection of significant changes in the preceding tests, previously proven non-destructive tests shall be carried out on selected construction elements.

17.3. RADAR INVESTIGATION OF A PRE-CAST POST-TENSIONED CONCRETE SEGMENTAL RAIL BRIDGE

This case study illustrates the inspection and assessment of the superstructure of a 160 m long post tensioned, segmental railway bridge in Manchester, UK to determine its load carrying capacity prior to a transfer of ownership of the structure for use in the Metrolink light rail system. The plan is shown in Figure 17.10 and the elevation in Figure. 17.11.

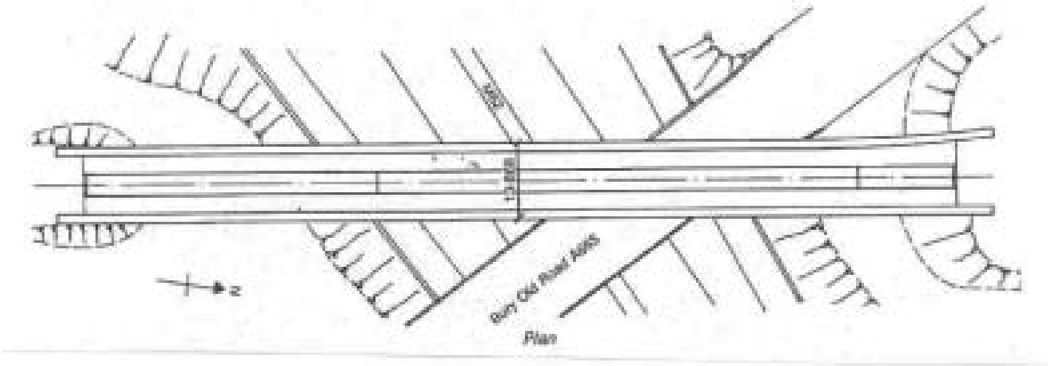


FIG. 17.10. Plan layout of bridge.

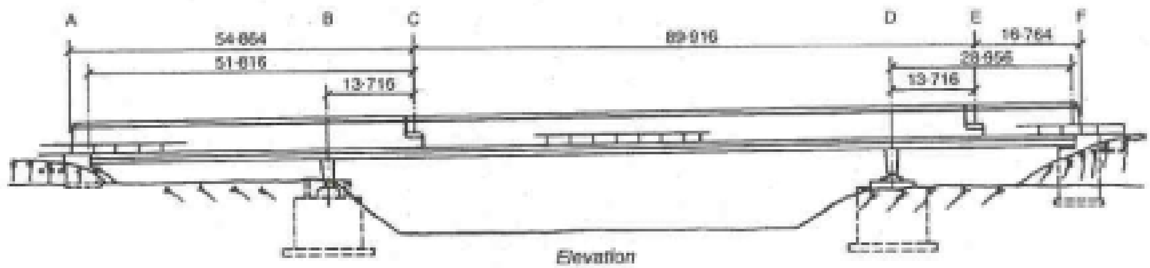


FIG. 17.11. Elevation of bridge.

Particular attention was given to the integrity of its post tensioned steel elements. Physical inspection, non-destructive radar testing and other exploratory methods were used to investigate for possible weaknesses in the bridge.

Since the sudden collapse of Ynys-y-Gwas Bridge in 1985 [1,2] there has been concern about the long term integrity of segmental, post tensioned concrete bridges which may be prone to "brittle" failure without warning. The corrosion protection of the post tensioned steel cables where they pass through joints between the segments was identified as a major factor affecting the long term durability and strength of this type of bridge. The identification of voids in grouted tendon ducts at vulnerable positions is recognized as an important step in the detection of such corrosion.

17.3.1. Description of bridge

17.3.1.1. General arrangement

Besses o' th' Barn Bridge is a 160 m long, 3 span, segmental, post tensioned concrete railway bridge built in 1969. The main span of 90m crosses over both the M62 motorway and A665 Bury to Prestwick Road.

The superstructure consists of a central hollow trapezoidal concrete box section 6.7 m high and 4 m wide. The majority of the south and central spans are constructed using 1.27 m long precast concrete trapezoidal box units, post tensioned together. This box section supports the *in situ* concrete transverse cantilever slabs at bottom flange level, which carry the rail tracks and ballast, Fig. 17.12.

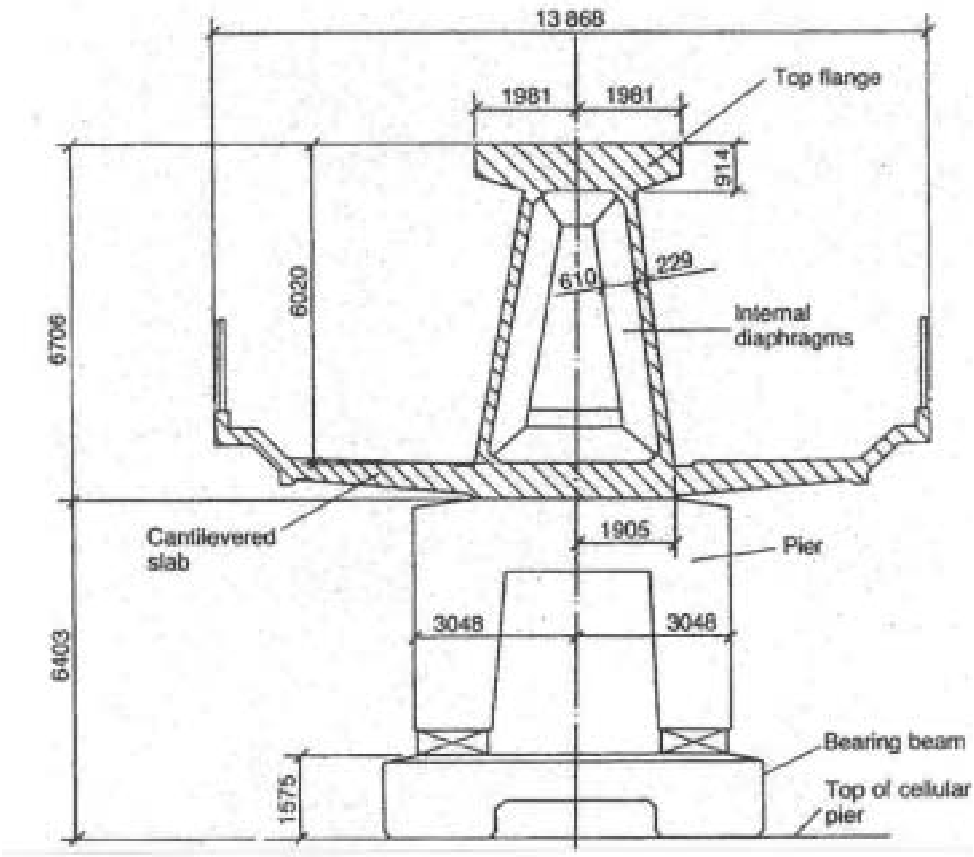


FIG. 17.12. Cross-section of Besses o' th' Barn Bridge.

The most northern section of superstructure is of *in situ* reinforced concrete construction. It contains two hollow cells, one of which was filled with railway ballast, thereby providing a counterbalance to loads in the main span. The bridge was designed to accommodate mining subsidence and consequently the deck is provided with half-joints in the centre and north spans. Each section of the bridge is supported statically at only three points, with freedom allowed for rotations or translations to take place.

The centre and south span sections are of post tensioned construction. These post tensioned sections have five types of pre-stressing, as shown in Fig. 17.12, as follows:

- Longitudinal tendons in grouted ducts within the top and bottom flanges.
- Longitudinal internal draped tendons located alongside the webs. These are deflected at internal diaphragm positions and are encased in *in situ* concrete.
- Longitudinal macalloy bars in the transverse cantilever slabs (in the central span only).
- Vertical macalloy bars in the 229 mm wide webs to enhance shear capacity.
- Transverse macalloy bars through the bottom flange to support the transverse cantilever slabs.

17.3.1.2. Segmental construction

The pre-cast segmental system of construction used for the south and centre span sections was an alternative method proposed by the contractor. Current thinking [3,4] suggests that such form of construction can lead to the entire structure's "brittle" failure without warning due to corrosion of tendons across a construction joint. The original design concept had been for *in situ* concrete construction. Details of the precast units are shown in Fig. 17.13.

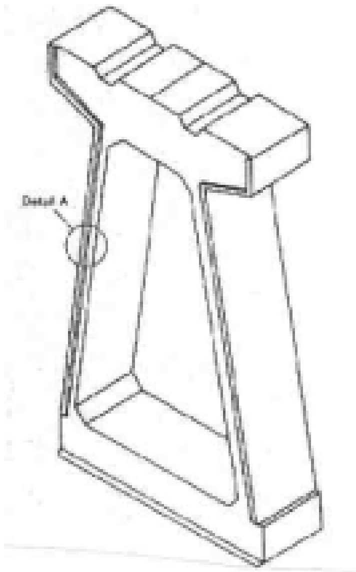


FIG. 17.13. Pre-cast unit details.

17.3.2. Inspection and assessment

17.3.2.1. Inspection

Inspection work was undertaken in a number of phases linked with the testing required for the structure. The initial inspections recorded a number of visible problems, including:

- defective waterproofing on the exposed surface of the top flange
- water trapped in the internal space of the hollow box with depths up to 300 mm
- various drainage problems at joints and abutments
- longitudinal cracking of the exposed soffit of the central span
- longitudinal cracking on sides of the top flange of the pre-stressed sections
- widespread spalling on some *in situ* concrete surfaces with exposed rusting reinforcement.

17.3.2.2. Assessment

The objectives of the assessment were:

— estimate load capacity using the RU type railway loading [BS 5400 Part 2]
— identify any structural deficiencies in the original design
— determine reasons for existing problems identified by the inspection.

17.3.2.3. Conclusion of inspection and assessment

Following inspection and analytical assessment, one major element of doubt still existed. This concerned the condition of the embedded prestressing wires, strands, cables or bars. For the purpose of structural analysis these elements had been assumed to be sound. However, due to the very high forces involved, a risk to the structure caused by corrosion to these primary elements, was identified.

The initial recommendations, which completed the first phase of assessment, comprised of:

— carrying out detailed material testing to determine the condition of hidden structural elements in particular the grouted post-tensioned steel cables
— conducting concrete durability tests
— undertaking repairs of defective waterproofing and surface defects in concrete.

17.3.3. Testing procedures

Testing was split into two main phases: the first is the subject of an earlier paper and involved extensive material testing; the second involved more detailed work on specific aspects identified by earlier tests and included investigation of possible corrosion of the grouted post-tensioned steel cables. This included the use of impulse radar.

17.3.3.1. Ground penetrating radar testing

During the first phase investigation at a joint between precast deck segments, the observation of a void in a post-tensioned cable duct gave rise to serious concern about corrosion and the integrity of the pre-stress. However the extent of this problem was extremely difficult to determine. The bridge contains 93 joints with an average of 24 cables passing through each joint, i.e. there were about 2200 positions where investigations could be carried out. A typical section through such a joint is shown in Fig. 17.14. The 24 draped tendons within the spine did not cause concern because these were protected by *in situ* concrete poured without joints after the cables had been stressed. It was clearly impractical to consider physically exposing all tendon/joint intersections. Various options were considered, with non-destructive methods being preferred for obvious reasons. It was concluded that radar might provide a means to look at large numbers of tendons and hence locate duct voids within a modest time scale.

The corrugated steel ducts around the tendons are discontinuous through the joints allowing the radar to detect the tendons and voids. The problem however was still highly complex due to the high density of other steel elements which could interfere with the radar signals and the fact that the area of interest was at most 102 mm wide and embedded between 150 mm and 800 mm deep in thick concrete slabs.

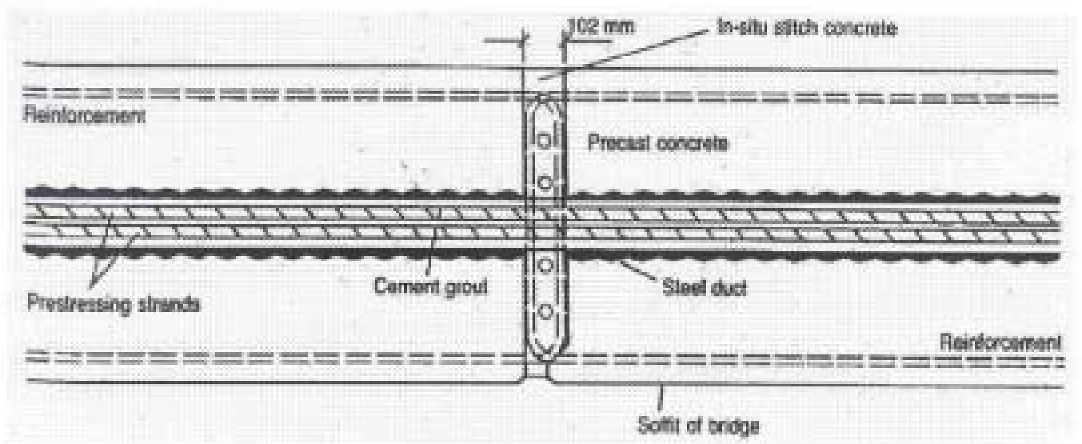


FIG. 17.14. Section through cast in situ joint.

17.3.3.2. Trial radar investigations

Three companies were invited to visit the bridge to consider conducting a trial investigation. The costs of these tests were to be reimbursed. One company did not proceed: they were concerned that the joints were too heavily congested with steel reinforcement. The other two were given two weeks to mobilize, test and report. Their results were then compared with physical explorations.

Observation holes were drilled vertically downwards into the ducts at 10 locations where voids were predicted and several others where ducts were predicted to be fully grouted. A rotary percussive 25 mm diameter hole was required to facilitate use of the chosen boroscope. The interpretation provided by The University of Edinburgh was sufficiently promising.

17.3.3.3. Main radar survey, boroscope verification of voids

After a radar survey of the total structure and analysis of results was completed, the University of Edinburgh submitted their interpretative report. A boroscope was used to investigate all predicted voids that were within range of drilling and, in more than 60% of cases, this gave a clear confirmation. In several other cases some evidence of honeycombing in the *in situ* stitch concrete above the duct was found.

17.3.4. Digital radar testing

The test method involved exciting the joints using radio frequency radar antenna. Three different radar frequencies were used: 1 GHz, 900 MHz, & 500 MHz. The highest frequency gives the highest resolution but has shallow depth penetration. The lowest frequency gives the greatest depth penetration but yields lower resolution.

The data was collected on a GSSI SIR System 10. This system involves digital radar pulsing and recording. The data from the antenna is transformed from an analogue signal to a digital signal using a 16 bit analogue-digital (A/D) convertor. This gives very high resolution for subsequent data processing. The data is displayed on site on a high resolution colour monitor. Following visual inspection it is then stored digitally on a 2.3 GB tape for subsequent analysis and signal processing. At particular locations along the traces, the trace was marked using a marker switch on the recording unit. An alternative approach would have been to use a marker switch on the antenna.

Signal processing was undertaken using GSSI RADAN 3 software. Techniques included changing the colour transform and changing the scales from linear to a skewed distribution in order to highlight certain features. Also, the colour transforms could be changed to highlight phase changes. In addition, to these colour transform facilities, sophisticated horizontal and vertical filtering procedures were available. A further facility from the RADAN 3 software was the ability to display the individual radar pulses as time domain wiggle plots. This is a particularly valuable feature when looking at individual records in the vicinity of the tendons.

17.3.4.1. Waveform image processing

Each waveform is analysed in terms of the magnitude of its deflection from the zero line. The minimum to maximum amplitude is generally split into 16 equal sections and each section is assigned a colour. This is known as a linear transform but others may be used. Waveforms are then placed adjacent to each other and plotted as depth, or two-way travel time, on the vertical axis against horizontal distance moved on the horizontal axis. The magnitude of deflection from the zero line is shown as a point of colour. Therefore a horizontal layer of homogeneous material will show up as a band of colour on the radar system output device. A complete profile is known as a line scan image. Alternatively the data can be displayed as a time domain plot or wiggle plot, with the data stretched out in the time axis the antenna traverses.

The key principle is that if the dielectric constant of the second layer is substantially higher than that of the first layer there will be a marked phase change.

17.3.5. Analysis of the problem

The size of the tendon ducts in the cast *in situ* area were assumed to be of the order of 75-80 mm diameter. Assuming that there are 12 tendons each of seven strands of wire and that each tendon is approximately 15mm diameter, then it can be assumed that the tendons will fill approximately half the cross-sectional area of the tendon duct. Thus, depending upon the location of the tendons they might completely fill the top half of the duct or the bottom half of the duct. Clearly there may be intermediate scenarios, but in many instances it will be one or the other. This has significant implications for the interpretation of the data. Various different tendon scenarios are shown in Fig. 17.15, together with the different dielectric constants for the different materials. A substantial reflection will only occur when there is a significant change in dielectric constant, as when an air void exists. Where there is an air void in the vicinity of a tendon this results in a double phase shift – the first example is given in Fig. 7.16 where a complex set of colour transforms are shown.

Although attractive on first inspection, this transform is difficult to interpret due to interference from the conventional reinforcement.

In view of this it was decided to apply horizontal filtering which would remove the conventional reinforcement appearing in multiple e-m pulses. Then a black and white colour transform was applied: black being one phase whilst white was the other phase (Fig. 17.17).

From Fig. 17.15, Case 1 represents the grouted tendon. The first phase shift where there is a negligible change in dielectric constant would not be detected by a radar wave.

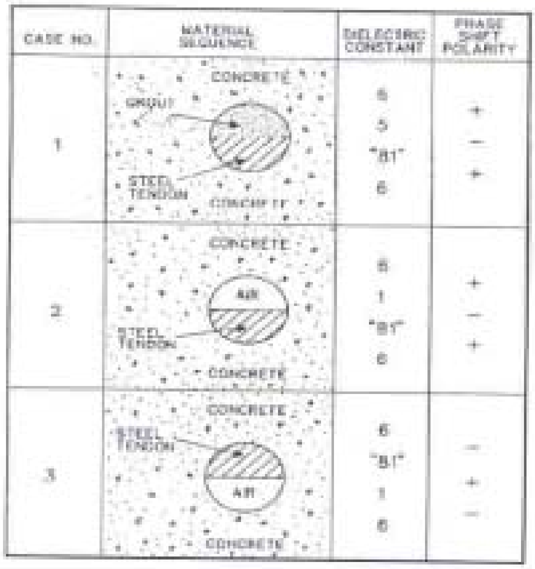


FIG. 17.15. Grouted and ungrouted tendon cases.

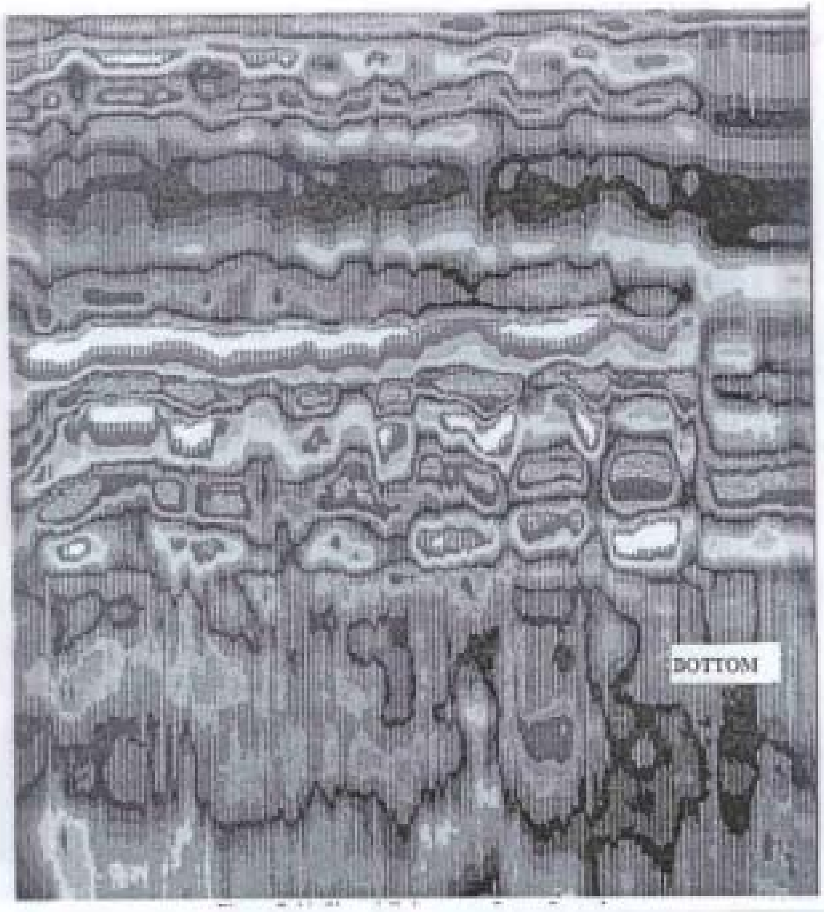


FIG. 17.16. Unfiltered colour transform of raw data.

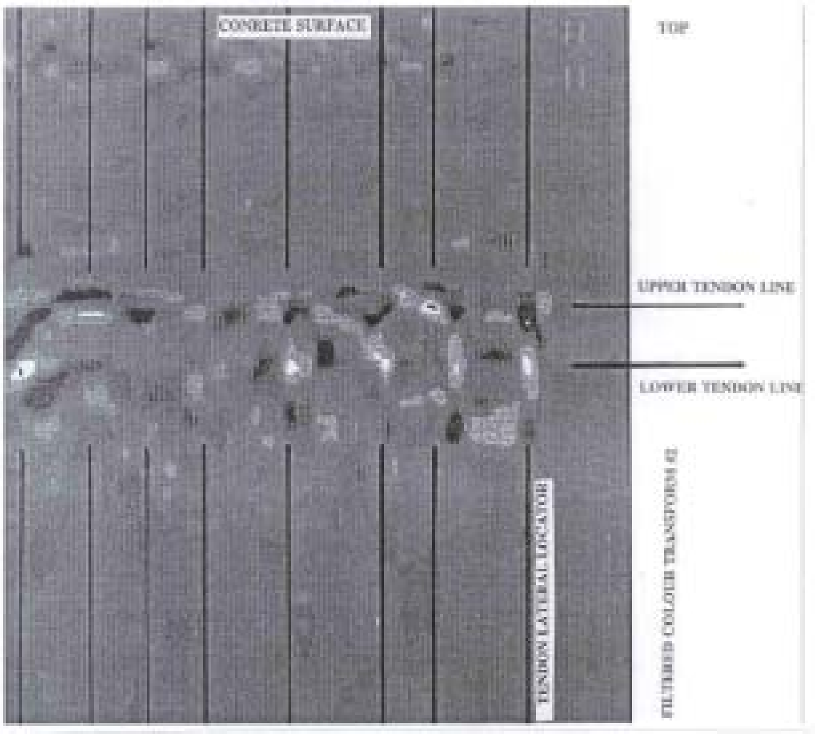


FIG. 17.17. Horizontally filtered radar line scan using a black and white transform.

Thus the grouted tendon case (1) would be seen as a single phase shift. However, in the instance of cases 2 and 3, the greater change in the dielectric constant would be exhibited by a double phase change. This may be observed in Figure 8 where the signal goes from the "background grey-white-black-white- background grey" or from the "background grey-black-white-black-background grey". Three examples of this double phase shift may be observed in Fig. 17.17.

17.3.5.1. Trial hole: investigation for corrosion

In order to estimate whether cables were corroded a larger trial hole was broken out at one location in the bottom flange to expose the largest of the voids — a position of relatively high corrosion risk where water had been standing inside the structure.

The steel tendon was found to be in good and shiny condition with no sign of corrosion.

17.3.6. UK perspective

Besses o' th' Barn Bridge is just one of approximately 500 post tensioned concrete bridges in the UK of which about 120 use a segmental construction method. It has presented a number of unusual problems not generally encountered by bridge assessment engineers. The bridge clearly demonstrates the need for a flexible approach to inspection, assessment and testing.

17.3.7. Conclusions

The use of GPR has contributed considerably to the level of confidence in the assessment of the Besses o' th' Barn rail bridge.

The radar investigations revealed extensive voiding within the post-tensioned cable ducts. However no sign of corrosion on the stressing wires had been found except for the very first investigation.

The future durability of the stressing wires, coated with only cement grout could not be predicted with confidence for further 50 years.

The bridge was not subject to the effects of de-icing salts, as in a road bridge, and this single fact may have preserved the general structure thus far. However, its future durability will need to be positively preserved.

The overall conclusions of the assessment and testing are that strengthening work should be conducted. The present condition of the bridge is considered acceptable for its age although work must be done to ensure that no further deterioration is allowed to take place. Further monitoring will be needed as part of a pre-determined inspection and maintenance programme.

17.4. FLYOVER

Spalling of concrete was reported in a 20 year old heavy traffic flyover from one of the girder beams. The flyover was constructed of reinforced concrete with precast post-tensioned girder beams and hammerheads. The flange of the girder beam was pre-tensioned. An initial visual survey showed signs of corrosion and possibly leaching. This was followed by a detailed investigation to determine the general condition of the structure. Several non-destructive testing methods were used as a qualitative survey and they included the ultrasonic pulse velocity and rebound hammer test. Corrosion activity was also assessed using a combination of the half-cell potential, resistivity and linear polarization techniques. The reinforcement bars and pre-tension steel were exposed to examine the condition and at the same time confirm the corrosion activity. Details of the reinforcement bars such as cover, locations and diameters were checked with an electromagnetic covermeter. Depth of carbonation was also determined using phenolthalein solution on freshly broken concrete and core samples. Holes were made to expose the post tension ducts to inspect whether the ducts were filled with grout in both the girder beams and hammerhead. Any sign of corrosion of the cables in the ducts were recorded during this inspection. Core samples were extracted from various locations selected based on the results of the non-destructive survey. The extracted core samples were tested to determine the compressive strength, porosity/permeability, chloride content, sulphate content and mix proportion. Petrographic examination was made on the cores to evaluate the condition of the concrete such as degree of deterioration, type of materials used potential durability and others.

Investigation revealed that the concrete was generally quite uniform and corrosion was found to be actively taking place, especially in the pre-tension steel. The cover of the concrete was lower than that specified during construction, however, the depths of carbonation were fortunately relatively low considering the age of the structure. A significant proportion of the length of the post tension ducting was found to be unfilled with grout but no sign of corrosion was observed in the pre-stress cables. No major degradation of the concrete was evident as the chloride content and sulphate content was low while the strength and permeability were within acceptable limits. On the other hand, the petrographic examination confirmed that isolated leaching had occurred resulting in pockets of high porosity and hence low durability.

A rehabilitation programme was subsequently designed to address the corrosion and isolated pockets of concrete degradation and to prolong the life of the structure. As this flyover is at the heart of several major expressways, the volume of vehicular traffic is very high and no major future repair works, which involves closing of any of the lanes, is allowed. The rehabilitation programme thus incorporates strengthening, protection and monitoring systems. Further tests were made to study the condition of the structure during the rehabilitation exercise. Ultrasonic pulse velocity and carbonation tests were conducted on the concrete in the hammerheads. On the girder beams, the post tension cables were inspected for ungrouted ducts non-destructively. GPR or impulse radar was used at first to locate the position of the ducts and then the impact-echo method was used to identify voids in the ducts. The impact-echo survey was only successful on the ducts along the web of the beam, whereas in the flange, the signals received could not be easily interpreted. The condition of the duct in the flange was subsequently inspected by drilling into the duct and then examined visually using a fibrescope. Due to the complex configuration of the structure and presence of multiple ducts, non-destructive inspection was not possible in the hammerheads. Fibrescope inspection was used instead. Core samples were also extracted from the hammerheads for determination of strength, porosity and chemical composition. Petrographic examination was made to evaluate the condition of the concrete.

A programme was introduced to monitor the long term performance of the structure in service. These included the use of fibre optics sensing, corrosion activity and vibration monitoring system. The fibre optics sensing system was utilized to monitor the strain development throughout the whole length of the strengthened beam. Monitoring of the corrosion activity was made using the half-cell potential technique. Results of these various sensors were remotely collected via modem and analysed using a finite element programme.

17.5. SCOUR OF RIVERBEDS AROUND BRIDGE PIERS

The scouring of riverbeds around the foundations of bridges with mid river piers has resulted in some bridge collapses during flood conditions. In field trials conducted by S.G. Millard, et al. using ground penetrating radar, examples of bridge scour were found. The best results were achieved by using 300 and 500 MHz antennae. Results from a significant scour hole located by the upstream nose of the pier of a trunk road bridge are shown in Fig. 17.18.

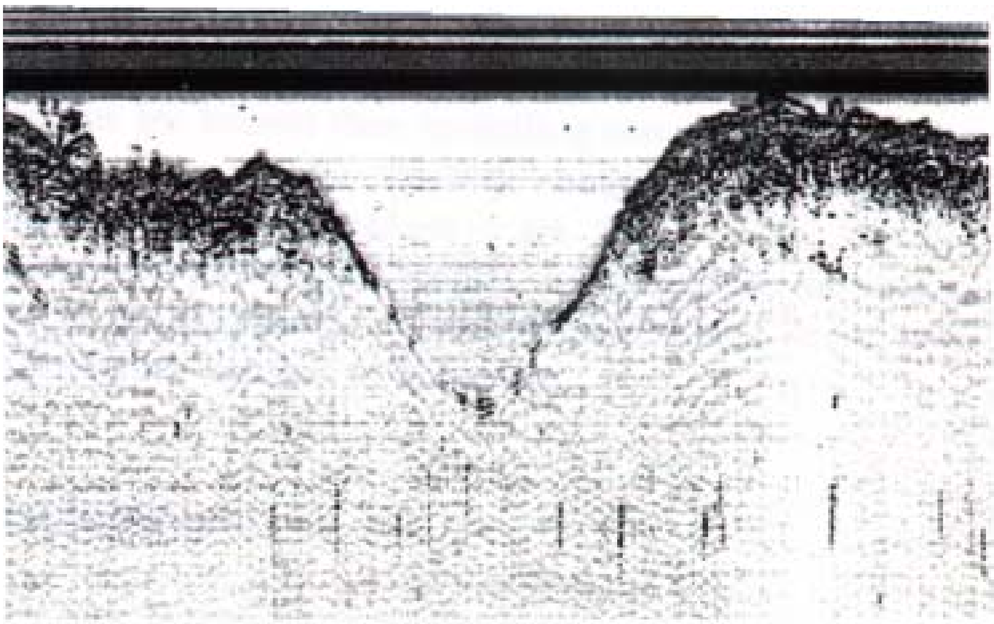


FIG. 17.18. Radar result from scour hole.

Laboratory tests carried out on a simplified scour hole geometry, with a range of bridge pier and infill parameters, show that radar can be used to evaluate the position, size and depth of a scour hole even when infilled with sediment provided the electrical properties of the river bed and the sediment are quite dissimilar. However, when the electrical properties of the infill material are close to those of the riverbed, any infilled hole could be difficult to detect.

17.6. ASSESSMENT OF SLUICES

Maierhofer, Krause and Wiggenhauser successfully applied radar and ultrasonic impulse echo for the assessment of two almost 100 year old sluices. The interior condition of the side walls was investigated with radar enabling the detection of detachments of the faced brickwork as well as the determination of moisture content. With ultrasonic impulse echo, working joints were found in the concrete slab of the sluice heads. The results of these investigations gave sufficient information for a reduction to be made to the number of cores taken for compression testing.

The investigation of the side walls was made with a commercially available short pulse radar system with bistatic antennae emitting with main frequencies of 500MHz and 900MHz. These antennae were moved directly on the surface of the side walls in horizontal scans. For each side wall of both sluices, two horizontal scans at different heights (above the water line and below the water line) were made which required the use of waterproof housings. Above-water-line scan revealed brickwork with a thickness of 0.8 m. From the known thickness of the wall and the measured travel time, permitivity of 9.8 for sluice 1 and 10.8 for sluice 2 was calculated. From calibration curves for solid brick produced by Hasted and Shah, moisture content of about 18 vol.% for sluice 1 and 20 vol.% for sluice 2 was calculated. Below-water-line scan revealed for sluice 2 that the brickwork was 0.2 m thick and was backed by concrete. The same scan on sluice 1 revealed that the brickwork had become detached from the concrete.

The ultrasonic pulse echo method was used to measure the thickness of the concrete base of one of the heads of the sluices. However, because of the inhomogeneity of the concrete the method was modified using the principle of synthetic aperture that is based on the time of flight corrected superposition of numerous A-scans with different positions of sending and receiving transducers. All measurements were made at the base of the sluice 2 m below the water level having a diver positioning the transducers. A template was used (0.5×1 m^2) with numbered openings for the transducers. One data set consisted of 80 A-scans measured at different distances d of the transmitting and receiving transducers (d = 0.3 … 0.9 m). A pair of broadband transducers with a centre frequency of 100 kHz was used. Integration of the pulse energy versus depth revealed maxima at depths of 0.55 m and 0.4 m. Cores taken in those areas showed a layer of non-compacted concrete at a depth of 0.55-0.57 m and some sand inclusions as well as steel reinforcement in the depth range from 0.27 to 0.31 m. Because of the presence of those two layers, it was not possible to determine the thickness of the concrete base.

18. CODES, STANDARDS, SPECIFICATIONS AND PROCEDURES

18.1. GENERAL CONSIDERATIONS

18.1.1. The need for standards

One of the consequences of the Industrial Revolution era was the eventual community realization that the developments taking place had to be subjected to some measure of control. For instance material properties had to be specified so that manufacturers could manufacture material with agreed minimum properties. Design engineers needed to know the minimum properties so that designs could be produced with an appropriate factor of safety. Insurance organizations needed to be sure that the risk of failure of a structure was as low as possible to minimize insurance claims. The end user of the technology, the general public, also needed to be sure that the technology was safe. In most countries national bodies were formed to provide the necessary control. For any specific topic the national body would convene a committee of representatives of all interested parties who meet to develop a draft for circulation to all interested parties. The committee formed to develop the document involves technical experts from the producers, the users and the general public usually represented by a regulatory body. The committee may also include representatives from technical societies and from universities. The resulting ‘standard’ developed by this consensus process is often referred to in contracts between organizations to control the quality of work. In this case the document is legally binding and deliberate non-compliance can result in legal penalties being applied.

Standards can play an important role in international co-operation when they are used in contracts and treaties for the supply of goods and services between one country and another. The growth in international trade has resulted in a growth in the need for International Standards, which can be an acceptable compromise between different national standards.

18.1.2. Different categories of standards

18.1.2.1. Standards

Standards are documents that govern and guide the various activities occurring during the production of an industrial product. Standards describe the technical requirements for a material, process, system or service. They also indicate as appropriate, the procedures, methods, equipment or tests to determine that the requirements have been met.

18.1.2.2. Codes and Specifications

Standards may also be referred to as Codes or Specifications. One of the best examples of a code is the ASME Boiler and Pressure Vessel code which is a set of documents that assure the safe design, construction and testing of boilers and pressure vessels.

18.1.2.3. Other types of national documents

To provide guidance on test methods, a national standards organization may develop Guidelines or Recommendations to assist users of a specific technology. These types of documents are not usually called up in contracts between organizations.

18.2. STANDARDIZATION ORGANIZATIONS AND SOME OF THE STANDARDS RELATING TO TESTING CONCRETE

18.2.1. American Society for Testing and Materials (ASTM)

(1) ASTM C 42-87, Standard Test Method (STM) for obtaining and testing Drilled Cores and Sawed Beams of Concrete, Annual Book of ASTM Standards, 1988, ASTM, Philadelphia, USA
(2) ASTM C85-66, "Cement content of hardened Portland cement concrete", ASTM, Philadelphia, USA
(3) ASTM C457-80, "Air void content in hardened concrete", ASTM, Philadelphia, USA
(4) ASTM C823-75, "Examining and sampling of hardened concrete in constructions"
(5) ASTM C779-76, "Abrasion resistance of horizontal concrete surfaces"
(6) ASTM C944-80, "Abrasion resistance of concrete or mortar surfaces by the rotating cutter method"
(7) ASTM C856-77, "Petrographic examination of hardened concrete"
(8) ASTM D4788-88 Standard Test Method for detecting Delamination in Bridge Decks using Infrared Thermography
(9) ASTM D6087-97 STM for Evaluating Asphalt covered Concrete Bridge Decks using Ground Penetrating Radar
(10) ASTM D4580-86 (1997) Standard Practice for measuring Delamination in Concrete Bridge Decks by Sounding
(11) ASTM D2950-91 (1997) STM for Density of Bituminous Concrete in place by Nuclear Methods
(12) ASTM C1383-98a STM for measuring P wave Speed and the Thickness of Concrete Plates using the Impact-Echo Method
(13) ASTM C1150-96 STM for the Break off Number of Concrete
(14) ASTM C1040-93 STM for Density of Unhardened and Hardened Concrete in place by Nuclear Methods
(15) ASTM C900-94 STM for Pullout Strength of Hardened Concrete
(16) ASTM C876-91 STM for Half-cell Potentials of Uncoated Reinforcing Steel in Concrete
(17) ASTM C805-97 STM for Rebound Number of Hardened Concrete
(18) ASTM C 803-82, STM for Penetration Resistance of Hardened Concrete
(19) ASTM C801-98 STM for Determining the Mechanical Properties of Hardened Concrete under Triaxial Load
(20) ASTM C597-97 STM for the Pulse Velocity through Concrete.

18.2.2. British Standards Institution (BSI)

(1) BS 1881: Part 102: 1983 - Method for Determination of Slump
(2) BS 1881 Part 5:1970 - Testing Concrete. Methods of testing hardened concrete for other than strength. Determination of dynamic modulus of elasticity by electromagnetic method
(3) BS 1881 Part 205:1970 - Testing Concrete. Recommendations for radiography of concrete
(4) BS 1881 Part 206:1986 - Testing Concrete. Recommendations for determination of strain in concrete. Advice on the use of mechanical, electrical resistance and vibrating wire gauges and electrical displacement gauges

(5) BS 1881 Part 202:1986 - Testing Concrete. Recommendations for surface hardness testing by rebound hammer
(6) BS 1881: Part 114: 1983 - Methods for Determination of Density of Hardened Concrete
(7) BS 1881: Part 116: 1983 - Method for Determination of Compressive Strength of Concrete Cubes
(8) BS 1881: Part 117: 1983 - Method for Determination of Tensile Splitting Strength
(9) BS 1881: Part 118: 1983 - Method for Determination of Flexural Strength
(10) BS 1881: Part 120: 1983 - Method for Determination of Compressive Strength of Concrete Cores
(11) BS 1881: Part 121: 1983 - Method for Determination of Static Modulus of Elasticity in Compression
(12) BS 1881: Part 122: 1983 - Method for Determination of Water Absorption
(13) BS 1881: Part 201: 1986 - Guide to the Use of Non-Destructive Methods of Test for Hardened Concrete
(14) BS 1881: Part 202: 1986 - Recommendations for Surface Hardness Testing by Rebound Hammer
(15) BS 1881: Part 203: 1986 - Measurement of the Velocity of Ultrasonic Pulses in Concrete
(16) BS 1881: Part 204: 1986 - Recommendations on the Use of Electromagnetic Covermeters
(17) BS 1881: Part 207 1992 - Recommendations for the Assessment of Concrete Strength by Near-to-Surface Tests
(18) BS 8110: Part 1: 1985 - Structural Use of Concrete: Code of Practice for Design and Construction
(19) BS 8110: part 2: 1985 - Structural Use of Concrete: Code of Practice for Special Circumstances
(20) BS 4408: pt. 4, "Non-destructive methods of test for concrete - surface hardness methods" British Standards Institution, London
(21) BS 1881: pt. 4, "Methods of testing concrete for strength", British Standards Institution, London
(22) BS 4408: pt. 5, "Non-destructive methods of test for concrete - Measurement of the velocity of ultrasonic pulses in concrete", British Standards Institution, London
(23) BS 4408: pt. 2, "Recommendations for non-destructive methods of test for concrete – strain gauges for concrete investigations", British Standards Institution, London, 1969,(83)
(24) BS 4408: pt. 1, "Non-destructive methods of test for concrete-electromagnetic cover measuring devices", British Standards Institution, London
(25) BS 4408: pt. 3, 1970 "Non-destructive methods of test for concrete-gamma radiography of concrete", British Standards Institution, London
(26) BS 1881: pt. 6, "Methods of testing concrete: analysis of hardened concrete", British Standards Institution, London
(27) BS4551, "Methods of testing mortars, screeds and plasters", British Standards Institution, London
(28) BS 812: pt. 1, "Methods for sampling and testing of mineral aggregates, sands and fillers", British Standards Institution, London.

18.2.3. German Standards Institution (Deutsches Institut für Normung) (DIN)

(1) TGL 21 100/01 Non-destructive testing of concrete buildings and structures – Guideline for the determination of the density with gamma rays (Zerstörungsfreie

Prüfung von Bauwerken und Bauteilen aus Beton - Richtlinie zur Bestimmung der Rohdichte mit Gammastrahlen)
(2) E DIN EN 12398 (July 1996) Testing of Concrete – Non-Destructive Testing – determination of the rebound number (Prüfung von Beton - Zerstörungsfreie Prüfung - Bestimmung der Rückprallzahl; German Version prEN 12398: 1996)
(3) E DIN EN 12399 (July 1996) Testing of concrete – Determination of the Pull-Out Force (Prüfung von Beton Bestimmung der Ausziehkraft; German Version prEN 12399: 1996).

18.2.4. International Organization for Standardization (ISO)

(1) ISO/CD 1920 Teil 7 - Testing Concrete: Non destructive tests of hardened concrete
(2) ISO/DIS 8045 Concrete, hardened - Determination of rebound number using rebound hammer
(3) ISO/DIS 8046 Concrete, hardened - Determination of pull-out strength.

18.2.5. Australian Standards International

(1) AS 1012.1-1993. Method of Testing Concrete - Sampling of fresh concrete
(2) AS 1012.2-1994. Method of Testing Concrete - Preparation of concrete mixes in the laboratory
(3) AS 1012.3.1-1998. Method of Testing Concrete - Determination of properties related to the consistency of concrete - Slump test
(4) AS 1012.3.2-1998. Method of Testing Concrete - Determination of properties related to the consistency of concrete- Compaction factor test
(5) AS 1012.3.3-1998. Method of Testing Concrete - Determination of properties related to the consistency of concrete - Vebe test
(6) AS 1012.3.4-1998. Method of Testing Concrete - Determination of properties related to the consistency of concrete- Compactibility index
(7) AS 1012.4.1-1999. Method of Testing Concrete - Determination of air content of freshly mixed concrete -Measuring reduction in concrete volume with increased air pressure
(8) AS 1012.4.2-1999. Method of Testing Concrete - Determination of air content of freshly mixed concrete-Measuring reduction in air pressure in chamber above concrete
(9) AS 1012.4.3-1999. Method of Testing Concrete - Determination of air content of freshly mixed concrete-Measuring air volume when concrete dispersed in water
(10) AS 1012.5-1999. Method of Testing Concrete - Determination of mass per unit volume of freshly mixed concrete
(11) AS 1012.6-1999. Method of Testing Concrete- Method for the determination of bleeding of concrete
(12) AS 1012.8-1986. Method of Testing Concrete - Method of making and curing concrete compression, indirect tensile and flexure test specimens in the laboratory and field
(13) AS 1012.9-1999. Method of Testing Concrete - Determination of compressive strength of concrete
(14) AS1012.10-1985 Method of Testing Concrete - Method for the determination of indirect tensile strength of concrete cylinders (Brazil or splitting test)
(15) AS 1012.11-1999. Method of Testing Concrete - Method for determination of flexure strength of concrete specimens
(16) AS 1012.12.1-1998. Method of Testing Concrete - Method for the determination of mass per unit volume of hardened concrete-Rapid measuring method

(17) AS 1012.12.2-1998. Method of Testing Concrete - Determination of mass per unit volume of hardened concrete-Water displacement method
(18) AS 1012.13-1992. Method of Testing Concrete - Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory
(19) AS 1012.14-1991. Method of Testing Concrete - Method for securing and testing cores from hardened concrete for compressive strength
(20) AS 1012.16-1996. Method of Testing Concrete - Determination of creep of concrete cylinders in compression
(21) AS 1012.17-1997. Method of Testing Concrete - Determination of the static chord modulus of elasticity and Poisson's ratio of concrete specimens
(22) AS 1012.18-1996. Method of Testing Concrete - Determination of setting time of fresh concrete, mortar and grout by penetration resistance
(23) AS 1012.19-1988- Method of Testing Concrete - Accelerated curing of concrete compression test specimens (laboratory or field) - Hot water and warm water methods
(24) AS 1012.20-1992- Method of Testing Concrete - Determination of chloride and sulfate in hardened concrete and concrete aggregates
(25) AS 1012.21-1999- Method of Testing Concrete - Determination of water absorption and apparent volume of permeable voids in hardened concrete.

18.2.6. American Concrete Institute

(1) ACI 228 Committee Report, In place methods for determination of strength of concrete, ACI J. Mater. September/October 1988
(2) ACI 207-79, Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions, ACI Manual of Concrete Practice, American Concrete Institute, Detroit, MI, 1983
(3) ACI Spec. Publ., SP-82, *In situ*/Non-destructive Testing of Concrete, American Concrete Institute, Detroit. 1984.

18.2.7. DGZfP Recommendations: (German Society for Non-destructive Testing)

(4) Recommendations for Radiographic Testing of Concrete and Post Tensioned concrete
(5) Recommendations for Reinforcement Detection and Concrete Cover Testing for Reinforced and Post-tensioned Concrete
(6) Recommendations for Electrochemical Potential Measurement for Corrosion Detection in Reinforced Concrete Structures
(7) Recommendations for the Ultrasonic Pulse Echo Method for Testing Building Materials and Structures
(8) Recommendations for Thermographic Investigations of Structures and Building Parts
(9) Recommendations for Visual Inspection and the Use of Endoscopes for Non-Destructive Testing in Civil Engineering
(10) Recommendations for the Use of Digital Image Processing for Non-Destructive Testing in Civil Engineering
(11) Recommendations for Seismic Methods for Subsurface Detection and Materials Parameters Determination of Soil.

18.2.8. Japanese Society for Non destructive Inspection

(1) NDIS 1401-1992 Methods of radiographic examination for concrete constructions
(2) NDIS 2416-1993 Method for measurement of sound velocity in concrete by ultrasonic pulse through transmission technique using reference blocks
(3) NDIS 3418-1993 Visual testing method of concrete structures
(4) NDIS 2421-2000 Recommended practice for *in situ* monitoring of concrete
(5) structures by acoustic emission (in course of preparation).

18.2.9. Japan Concrete Institute (JCI)

(1) The velocity measurement by UT
(2) The measurement of the depth of surface crack by UT
(3) The measurement of internal void and the position of reinforcement bar by UT
(4) AE method for concrete.

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