GUIDELINES ON REPAIR, STRENGTHENING AND REHABILITATION OF CONCRETE BRIDGES

(First Revision)

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GUIDELINES ON REPAIR, STRENGTHENING & REHABILITATION OF CONCRETE BRIDGES

CHAPTER 1

INTRODUCTION

1.1 Weakness signs and rapid deterioration of old bridge structures has become a serious problem in our country. The issue of maintaining the bridges has, therefore, become one of the most important challenges. Bridge engineers need a reliable way to assess structural integrity of bridges to maintain the continuous operation of the road network while ensuring the safety of the public. Strengthening and rehabilitation need expert knowledge and specialization. In order to guide bridge professionals, IRC first brought out IRC:SP:40 “Guidelines for Techniques for Strengthening and Rehabilitation of Bridges” in the year 1990. These guidelines covered common procedure for assessment of distress in bridges, selection of techniques and materials as also approach to remedial measures and formulation of suitable repair plans and document was widely adopted by professionals.

Since then lot of advancement have taken place in testing methodology, testing equipment, repair and strengthening techniques and materials and a need was felt to revise the document to keep pace with the latest advancement and international best practice. The task of revision of IRC:SP:40 was assigned to Bridge Maintenance and Rehabilitation Committee (B-8) during the tenure 2015-17. The initial draft was prepared by the Sub-group of B-8 Committee under Chairmanship of Shri P.Y. Manjure, comprising Dr. D.K. Kanhere, Shri R.K. Jaigopal and Dr. Harshavardhan Subbarao as members. After re-constitution of B-8 Committee in 2018, the earlier subgroup was requested to carry out further work of revision and induct new Members Dr. Samir Surlaker and Dr. Sanjay Wakchaure.

The document prepared by the Sub-group was deliberated in number of meetings by B-8 Committee and finally approved in its meeting held on 28th September, 2018 for placing before the Bridges, Specifications and Standards Committee (BSS) Commi ttee. The Condition Assessment Criteria has been introduced in this revision and all the results of visual inspection and detailed investigations are to be categorized into five condition states namely Excellent, Good, Fair, Poor and Critical.

The personnel of the Management, Maintenance and Rehabilitation Committee (B-8) for the tenure 2018-20 is as given below:

Patankar, V. L.	……..	Convenor
Banerjee, T.B.	……..	Co-Convenor
Wakchaure, Dr. Sanjay	……..	Member-Secretary

Members

Bagish, Dr. B. P.	Parameswaran, Dr. (Mrs.) L.
Banerjee, Col. S.	Puri, S.K.
Bhowmick, Alok	Sharma, R.S.

The Bridges, Specifications and Standard Committee approved this document in its meetings held at IRC headquarters on 24th October, 2018. Subsequently, the Executive Committee approved the document on 27th October, 2018 for placing it before the IRC Council. Thereafter guidelines were approved by the IRC Council in its 216th meeting held during 79th Annual Session at Nagpur on 22nd November, 2018 for publishing.

1.2 The deterioration of bridges is a world-wide phenomenon and the causes for this are also well known. The bridge condition and its performance may be affected due to inadequate design, poor quality construction, overloading, congestion of traffic on the bridge, inadequate maintenance, atmospheric effects, lack of knowledge about long term behavior, traffic, capacity augmentation, unforeseen events like abnormal floods, earthquake etc. The consequences faced are premature bridge deterioration, loss of traffic safety or reduction of structural strength necessitating imposition of load limitations and in some cases needing replacement resulting in heavy economic as also financial loss. These could be avoided if focused attention is given to conservation of existing bridges through regular monitoring and timely repairs on the basis of systematic investigation and adequate funds are devoted by Bridge Authorities on a regular basis based on assessed requirement for this purpose.

1.3 **Scope:** The scope of these guidelines is as under:

(i) Defining general process and approach for assessment of distresses, diagnosis of causes, proposing remedial measures and the corresponding methods and techniques for the engineering operations for different types of bridges constructed in the country.

(ii) Appraisal methods and conservation techniques for different types of bridge.

(iii) General guidelines on selection of techniques and materials.

(iv) Widening of existing bridges.

1.4 Definitions: Since the publication of earlier guidelines, there has been an evolution of many terms in respect of repair & rehabilitation all over the world. Many terms resemble one-another but are not identical. In order to have clarity, various terms are broadly defined in this guideline and are given below:

i. **Assessment:** Analysis with the aim to ascertain the condition and the capacity of an existing structure.

ii. **Damage:** Usually related to defects caused by external actions such as accidents, may also result from faulty design or workmanship. Frequently, used for defects resulting from deterioration processes.

iii. **Degradation:** Usually associated with wear and tear with age, also used to denote reduction in quality or strength.

iv. **Defect:** Slightly worse, more categorical and specific than deficiency.

v. **Deficiency:** Anything which found to be lacking or affecting the condition.

vi. **Demolition:** The process of dismantling and removal of existing structure, normally with the aim of total/partial renewal or replacement. May also be used in connection with repair work, e.g. demolition of deteriorated concrete.

vii. **Destruction:** Total damage to a structure caused by a deliberate action or by accidental or exceptional loads including earthquake.

viii. **Deterioration:** Reduction in condition and function of structure caused by "natural" causes like climate, environment.

ix. **Disintegration:** Severe deterioration resulting in breakup of components.

x. **Evaluation:** Often used as a synonym for assessment, but assessment is closely related with damaged and deteriorated structures.

xi. **Inspection:** Activity to ascertain the actual condition of the structure, including laboratory tests; but less exhaustive than investigation.

xii. **Investigation:** More in-depth and detailed than inspections, also includes analytical studies of strength and traffic, etc.

xiii. **Maintenance:** Activity meant either to prevent or rectify the effects of deterioration. May partly or wholly restore a specific component back to the original condition anticipated in the design. Maintenance is usually done periodically.

It comprises any activity and work including inspection necessary to enable a structure to continue to fulfill the intended function or to sustain its original or required form and appearance. It excludes any work leading to improvement of the structure whether by strengthening widening or by vertical realignment of the road surface, repairs of any damage caused by exceptional causes like landslides, earthquakes, cyclones, fire, etc. The maintenance operation starts with the opening of the bridge to traffic.

xiv. **Penetration:** Ingress of water inside the components.

xv. **Rebuild:** See Replacement.

xvi. **Reconstruction:** See Replacement.

xvii. **Rehabilitation:** Bringing the structure back to its original level of function including strength and durability.

xviii. **Remodeling:** Includes changes in function and performance of the structure due to change in usage or occupancy.

xix. **Removal:** Removing components from the structure.

xx. **Renewal:** Almost the same as replacement, but may include new materials/ techniques.

xxi. **Repairs:** Repair work is intended to prevent further damages to the members and restore functional performance. More routine and frequent activity than rehabilitation. Often repair includes a part of the structure do not imply bringing back the structure to its original level of durability and strength.

Repair activities shall generally cover minor damages to the components of the bridge. Such damages could be surface protection, hollows, honeycombs, cavities, loss of section, spalling, de-lamination and cracking of concrete etc. In steel bridges, it could be corrosion, breaking or rupturing etc. These activities would be similar to maintenance activities but on a larger scale. In short, repair activities shall consist of remedial action which would broadly include concrete restoration, cathodic control & protection, preserving or restoring passivity.

xxii. **Replacement:** "Worn out components" are replaced by new ones. May include improvements and strengthening, but does usually not include change in function.

xxiii. **Restoration:** Bring the structure back to the original condition not only with regard to function, but also to aesthetic or performance etc.

xxiv. **Retrofitting:** Particularly used in connection with either repair and rehabilitation related to earthquake damaged structures or as a means of preventing damage to structures situated in earthquake zone and also structurally deficient bridge elements. Includes not only rehabilitation but also strengthening and remodeling, i.e. structural intervention.

Retrofitting is improvement intended to restore a specific structural function or enhancement of bridge status to a higher performance level or modification of the structure to make it fit for additional load capacity. Retrofitting procedures include the addition of supplemental spanning designed to reduce seismic damage.

xxv. **Strengthening:** Increasing the strength of the structure by various means such as plate bonding, jacketing, Carbon Fiber Reinforced Plastic (CFRP) bonding, external pre-stressing, etc. It shall implyre-instating the designed strength/load bearing capacity of the structure as intended in the original design.

xxvi. **Survey:** Inspection of a large area or a large number of structures/components in order of get objective information.

CHAPTER 2

BASIC APPROACH

2.1 Introduction

The bridges must have a specified level of strength, safety and serviceability during their life under the anticipated traffic and environmental conditions. The process and approach methodology for assessment and remedial measures are elaborated in this chapter. The objective is to:

- Establish desired levels of maintenance service.
- Form the basis for cost analysis of maintenance program.
- Use men, material and money efficiently in an optimal manner.
- Establish Methodology to assess performance both with and without corrective action.
- Improve existing facilities wherever economically practicable through maintenance.

2.2 Factors Influencing Repair / Rehabilitation / Strengthening Strategy

While deciding the approach to the problem of distressed bridges, various factors are required to be investigated. Bridges are vital link on the highways and road network and are susceptible to deterioration. Needs of society for road network and consequently the usage of bridges do change continuously. These give rise to the need for repairs, retrofitting, rehabilitation and reconstruction of existing bridges. Besides, it necessitates establishing the requirements, needs and changes the present and future bridge stock may impose on the Bridge Authority. In this regard the following factors are necessary to be considered:

i. Traffic demands
ii. Environmental constraints
iii. Technical restrictions
iv. Social-Economic factors
v. Geographical conditions
vi. Strategic importance

2.2.1 *Traffic demands*

The traffic volumes and gross weight of vehicles have been observed to grow continuously. The axle loads have also had a strong upward trend during the recent decades. The tendency is to carry as much load as possible on a single vehicle in order to reduce the transportation cost. The reason for increasing axle loads is thus an economic. Authorities many a times, increases the legal axle load limits which needs to be considered for strengthening of a bridge. Another important factor to be considered in traffic loads is the size of exceptionally heavy vehicles which may cross the bridge. The growth in traffic volumes and axle weights have thus to be considered for determining the type of strengthening measures to be adopted.

2.2.2 *Environmental constraints*

As the awareness about environmental degradation increases in the public mind, there will be more demands from them for protection against environmental damages. After the traffic, environmental factors are accountable for the deterioration of bridges. Extreme climatic conditions such as, abnormal variations in wind speed, water flow & temperature and vicinity of foul gases, harmful chemicals etc. aggravate the deterioration mechanisms. The selection of treatment depends on the type and severity of the environment. The residual service life of existing concrete structures is largely determined by its deterioration over time. The deterioration rate of concrete structures depends not only on the construction processes employed and the composition of the materials used in the construction process, but also on the current as well as past environment. Meanwhile, global warming and climate change may alter this environment in the future, especially in the long term, causing more acceleration of deterioration processes and consequently affecting the safety and serviceability of existing concrete infrastructure.

2.2.3 *Technical restrictions*

The more complex the structure of bridges, the more difficult it is to strengthen or rehabilitate them. The higher cost related to repair and rehabilitation of such structures can sometimes make replacement more economical. The understanding of type of the bridge structure and its structural configuration is paramount in deciding the type of repair option to be implemented.

2.2.4 *Socio-economic factors*

The bridges of regions of developed or developing socio-economic activities and public demands shall be given priority over the other areas. Past experience shows that damage to bridge components can severely disrupt traffic flow and thus negatively impacting on the economy of the affected region. The extent of these impacts will depend not only on the nature and magnitude of distress on the bridge, but also on the mode of functional impairment of the highway system as a network resulting from physical damage of its components.

2.2.5 *Geographical conditions*

Many a times these conditions will influence the nature of rehabilitation that needs to be carried out. Treatment to be given to the bridges located in hills and landslide prone areas would differ from the measures adopted for bridges in coastal areas or bridges located in plains.

2.2.6 *Strategic importance*

Some bridges are strategically located such as those at sensitive areas near country's borders, those located in areas which are vulnerable to cyclones and storms etc. Remedial measures will therefore need to take into consideration the location and sensitivity of bridges.

The increased importance of strengthening and rehabilitation of bridges can be characterized by:

- Increased loads and traffic
- Increased technical challenges in areas not yet fully envisaged,
- Increased costs due to (i) limitations of the possibilities for replacement (ii) technical and practical difficulties in carrying out rehabilitation and (iii) diversion or absence of alternate route.
- Increased maintenance owing to the high cost of rehabilitation of the bridge.

2.3 Decision Making Approach

2.3.1 Decision making approach for maintenance and repairs shall be preventive rather than reactive. The safety and serviceability of both the users and the bridge shall be the prime objective and shall be the basis of decision. Bridge condition, deterioration rate, future developmental planning considerations, and remaining life of bridge shall influence policy decision. Fund availability and estimated costs of maintenance & repairs including for diversion, if any, shall also be kept under consideration.

The individual decision considerations will vary in accordance with the size and importance of bridge. For rehabilitation/strengthening of major bridges, elaborate analytical techniques for evaluation of various solutions, cost as well as performance effectiveness of alternate measures/ solutions including optimization shall be done. The life cycle cost, risk minimization, safety maximization etc. could be the factors of optimization. Rehabilitation and strengthening work of minor bridges may be carried out in accordance with inter-se priority and availability of funds. In all cases, however, it should be ensured that available funds are earmarked and allocated in accordance with the Authority's overall objectives and policies. The general policy must take into account the number of parameters already described in Para 2.2 above.

In principle, the decision ranging from no action, partial/complete immediate and/or differed maintenance, repairs, rehabilitation, strengthening to replacement of bridge could be reached through a cost-benefit, NPV, LCC and similar optimization analysis. All the activities of bridge maintenance and repairs shall be institutionalized.

2.3.2 The bridges may be divided into non-structural elements with relatively short-life spans such as wearing courses, water proofing, expansion joints, paint, railings, signage, drains, appurtenances etc. and structural elements with long-life spans such as bridge decks, abutments, column/piers, foundations etc. Such division between short-life and long-life bridge elements is necessary because of the stronger economic motivation to rehabilitate and strengthen long life elements than short life elements. A division between culverts, minor bridges, major or important/state of the art bridges is also warranted. A consideration of alternate materials and techniques available, their suitability and economy are very crucial while making a decision. Traffic considerations and a division between bridges on type of roads i.e. rural roads, state highways, national highways and expressways is also necessary. The general policy for rehabilitation/strengthening of bridges must therefore be based on the following factors:

Bridge Geometry: Length, Carriage Way Width, Waterway, Clearances; Environment, Load Carrying Capacity, Seismic vulnerability, Urgency of Maintenance Emergency, Future Demand, Strategic Importance, Location, Category of Road, Traffic Volume, Bridge Condition, Administrative & Political Considerations, Life of Elements.

As far as possible, the decision should be based on quantitative analysis.

2.4 Technical Approach for Rehabilitation/Strengthening

The best strategies can only be determined in the light of assessment of the present condition of the bridge through detailed investigation and analysis of the deterioration mechanism. Wherever possible, the root cause(s) of the deterioration should be evaluated and eliminated. Repair and strengthening operations should be mechanically and chemically compatible with the properties of the original or surrounding material and the basic structural concept. The various steps involved in working out a plan for rehabilitation/strengthening could be as below:

(a) Evaluation of the Structure from Past Records, Documented Data Base and Inspection

These have been given in IRC: SP-35.

(b) Distress Types, Location etc.

The deterioration of a structure can often be found visually through visible signs of damages. However, there may be hidden distresses which may surface after considerable period of time. Therefore visual inspection by an experienced engineer is a vital step in the chain of further follow-up action. A routine or principal inspection provides a detailed description of some of the damages where an assessment of the structure may become necessary.

The use of various testing methods may often become necessary to ascertain the observations of the visual inspection of the structure. Testing techniques and equipment should be capable to determine the extent and type of deterioration or damage and consequences of failure to the importance of structure. To the extent possible, non-destructive tests should be used. If necessary, the results of these testes may be supplemented and/or calibrated by sampling procedures in accordance with the small step principle, that is when deficiencies are discovered from a minimum number of specimens (a small sampling) the investigation may require a more detailed analysis. These procedures are standardized.

(c) Analysis of Causes of Damages/Defects and Distresses

The purpose of evaluation of a distressed structure is not only to determine the effect of damage on the structure's life expectancy/load carrying capacity, but also, and perhaps more importantly, determinate, to the possible extent, its cause so as to intelligently determine an effective retrofit. Before repair plan is implemented, the cause of the damage has to be removed or the repair measures have to be designed to accommodate the cause and protection against it in future. Otherwise the risk of repetition of damage will continue to exist.

(d) Evaluation of Results of Structural Assessment

Data resulting from the investigation of a damaged structure including monitoring of its distress form the basis for the decision as to what corrective action must be undertaken. This depends on type, extent and severity of the damage. The bridges may be affected by a combination of two or more deterioration mechanisms, which is complex to analyse and needs to be dealt with care.

The initial concern should be to find out whether or not there is a risk of failure to the damaged structure. If this risk is present, the first course of action must be to immediately provide an adequate auxiliary support mechanism and to reduce the loads to remove the risk. Where minor damage exists it should be investigated whether the damage is stable or will propagate with subsequent service loading. This is often a difficult and subjective assessment, made on the basis of visual examination until it can be verified with additional data and detailed calculations. The elements of time and/or the harshness of the environment becomes an important parameters when either the load carrying capacity is being diminished by deterioration (corrosion etc.) or when the load carrying capacity has to be increased due to increased traffic load or increase in legal axle load limits. Some evaluations will relate to whether or not an economically effective repair plan can limit or contain and thus can enhance the effective life of the structure. In some instances, an evaluation will be concerned with the degree of urgency required to implement a repair plan because of the advanced stage of damage.

The urgency of repairs, strengthening or replacement must be evaluated in a technical sense along with realistic cost estimates so that proper priorities can be established in budget planning. The question of urgency must also consider an estimation of remaining life expectancy. Although assumptions in a time dependent damage process are at best subjective, their evaluation will make an assessment of the degree of urgency less complex. Approximate estimations of life expectancy are valuable where damage is related to time dependent deterioration, e.g. corrosion of reinforcement. However, because of inherent uncertainties, the estimate will have to be presented in terms of upper and lower probability limits.

In some cases, it may be necessary to evaluate the lord carrying capacity of the bridge. Guidelines for this evaluation have been issued separately (IRC:SP:37). De-rating and/or regulated traffic options need also to be examined in these cases.

(e) Design of Repairs for Rehabilitation/Strengthening Works

The most important step in the design of repairs for rehabilitation and/or strengthening work is a careful assessment of the existing structure. The purpose of this assessment is to identify all defects and damages, diagnose their causes and to evaluate the present and likely future adequacy of the structure.

Generally the structural design for repairs shall conform to the relevant IRC guidelines. However, it must be recognized that the repairs for rehabilitation/strengthening is a special type of work and many a time accurate structural analysis may not be possible for the assessment of the existing strength as well as for the repairs for rehabilitation /strengthening. At the same, the design in some cases may have to account for effects of secondary stress and composite actions. When the structural system is complex for accurate analysis, specification more conservative than IRC specification may have to be adopted. On the other hand, in certain special cases construction difficulties and thus calculated risk may have to be considered. The designer of the rehabilitation/strengthening measures has, therefore, to be very judicious in his approach.

(f) Proposals and Estimation of Costs

The complexity and magnitude of the repair work will depend on whether

- Only the cause of damage has to be rectified or
- The structure must be restored to original condition or
- The structure needs to be upgraded for its load carrying capacity and/or for its geometry.

The degree of restoration will depend on whether or not the bridge is required to be restored to the original or greater load carrying capacity. If for technical and/or economic reasons, it is not feasible to achieve a complete restoration to original capacity and at the same time total replacement is not an acceptable option, a reduction of the applied live load becomes mandatory. (Ref.IRC:SP:37)

The Bridge Authority in reaching a decision as to the course of action will have to evaluate not only the technically feasible options available, but also the costs of each option, time of execution, political considerations (economic impact on communities served by the facility), life expectancy associated with various options available, any historical significance of the structure, any risks that may be involved with any changes in safety level or reduction in load carrying capacity etc. The rehabilitation and/or strengthening of major bridges is a complex task requiring many a time input from several specialists. The bridge engineer, therefore, has to consult the experts in different fields to work out the appropriate repair plan.

2.5 Technical Pre-Requisites for Intervention

2.5.1 Main objective of this activity is to introduce and present procedures for assessment of bridge structures with a view to arrive at proper remedial scheme for their maintenance, repair and rehabilitation. The assessment is a complex interaction between structural condition from visual inspection and construction records, information from in-situ and laboratory investigation and possible potential remedial actions. Such data can be gathered from investigation of the causes of distress and mechanism of deterioration.

Assessment of existing bridge structure shall generally comprise the following activities:

- Collecting historical information about performance and maintenance.
- Routine inspection usually visual followed by basic testing.
- Condition evaluation and planning detailed investigation if necessary.
- Detailed investigation and special testing of materials.
- Examination of deterioration phenomenon for assessment of durability, strength, safety and serviceability.
- For reinforcement and pre-stressing steel, checking the extent and severity of corrosion and residual pre-stressing forces.

2.5.2 Various steps involved in working out a plan for maintenance, repair and rehabilitation could be as described below:

(a) Preparation

Proper preparation and planning are basic requirements to be met in order to carry out professional assessment of distressed bridge. The objectives and concerns of the bridge authority shall be taken into consideration. An overview shall be established with typical reasons of deterioration or structural conditions. It shall also include program for various tests.

(b) Inspections

The visual inspection is the fundamental basis for providing a life time monitoring and understanding of the structure, site conditions and deterioration. The inspection should provide detail description of the type & location of damages where assessment of the structure is necessary. From the results of the visual inspection and basic testing Main mechanism of deterioration has to be established. Planning of the detailed investigation can be done if necessary on the basis of inspection.

(c) Investigations

The objective is to establish causes of deterioration, their degree and consequences on the present as well as future structural strength, safety, durability & serviceability. Use of various testing methods may become necessary to complement the results of visual inspection. Testing techniques and equipment shall be arrived at, depending upon the importance of the structure; location, type, extent, and severity of defects. In case of corrosion, tests to establish concrete and steel properties considering mechanical and durability aspects shall be carried out. Rate and quantum of corrosion shall be established. Besides, assessment of the global response of the structure on account of corrosion shall be done.

(d) Analysis of Cause of Distress

Information collected during inspection and investigation data of the bridge structure shall be analyzed to establish deterioration mechanism thereby the cause of damage, its effect on life expectancy and load carrying capacity. This would help in arriving at possible maintenance, repair and rehabilitation measures. An appropriate protection against the defect cause has to be provided to avoid repetition.

2.5.3 Analysis done as referred earlier provides understanding of the causes and degree of deterioration. It is then possible to take review of the design aspects and an assessment can be proposed. The review can cover aspects such as evolution of the current service conditions, remaining load carrying capacity and deterioration progress etc.

An initial concern shall be whether there is any risk of failure of the damaged structure. If it is there, the first action shall be to provide adequate support mechanism and to reduce loads. When there is minor damage, it shall be assessed whether it would aggravate due to service loads. The recalculation exercise (design review) can throw light on this aspect.

The evaluation shall bring out efficacy of repair plain in as much as it can contain damage and enhance life to the bridge. In some instances, an evaluation will be concerned with the degree of urgency required to carry out repairs due to advanced stage of damage. On the basis of measures evaluated for rehabilitation, cost of it shall be estimated and incorporated in the budgeting. In case of time dependent damage, estimation of life expectancy is important.

2.5.4 Generally the structural design for repairs shall conform to the relevant IRC codes. However, it must be recognized that the repairs for rehabilitation a special type of work and many a time accurate structural analysis may not be possible both for assessment of existing strength as well as for rehabilitation. At the same time, the design in some cases may have to account for effects of secondary stress and composite actions. Sometimes specification more conservative than the IRC may have to be adopted when structural system is more complex. In certain cases calculated risk will have to be taken by permitting higher stresses due to constructional difficulties. The measures finally to be adopted would be governed by needs, access, traffic closure atmospheric conditions, funds etc.

2.5.5 The scheme of maintenance, repair and rehabilitation of a bridge would depend upon the following factors:

- Removal of the cause of the damage.
- Requirements of rehabilitation of structure to the original condition.
- Requirements of up-gradation of structure to increase load carrying capacity.

There are alternate options to be considered in evaluation and restoration of the damaged structure. Therefore opinion of an expert bridge engineer in different related fields is must to work out the appropriate repair plan/intervention scheme.

CHAPTER 3

TYPES OF BRIDGES AND DISTRESSES NORMALLY OBSERVED

3.1 Introduction

The distresses normally observed in different types of road bridges are discussed in this chapter. The type of distress is related to the material used in design and construction of various components.

3.2 Types of Bridges

Various types of road bridges in India are broadly classified on the basis of materials used and their structural form as under:

3.2.1 *Type of Material:*

- i Masonry: Stone, Brick and Cement Concrete Block
- ii Concrete: Plain, Reinforced and Pre-stressed
- iii Steel
- iv Composite
- v Timber

3.2.2 *Type of Structural Form:*

- i Arches
- ii Slab
- iii Girder/Beam and Deck Slab
- iv Box Cell Bridges
- v Steel girders with concrete deck slab
- vi Reinforced concrete girders and deck slab, box girders - simply supported, continuous, balanced cantilevered with suspended spans etc.
- vii RCC rigid frame
- viii Truss/Open Web Bridges
- ix Pre-stressed concrete girders and deck slab, box girders - simply supported, continuous, balanced cantilevered with suspended spans etc.
- x Cable Stayed Bridges
- xi Suspension Bridges etc.
- xii Extradosed Bridges

3.3 Distresses Normally Observed

Distresses normally observed in these bridges are mentioned below:

3.3.1 *Arch bridges:*

- i Changes in profile of the arch (like flattening of arch, arch ring deformation etc.)
- ii Loosening of mortar: This could be considered as ageing effect.

iii Movement of the abutment or supporting pier: This is normally followed by the arch ring deformation, hog or a sag.

iv Longitudinal cracks: These could be due to varying subsidence along the length of the abutment or pier.

v Lateral and diagonal cracks indicate a dangerous state.

vi Cracks between the arch ring, spandrel or parapet wall.

vii Old cracks no longer widening and these probably occurred immediately after the bridge was built.

viii A vertical crack in the return wall: This is generally seen at locations where foundations on yielding soil are stepped.

ix Bulging of wall: This could be due to absence or malfunctioning of weep holes.

3.3.2 *Reinforced Cement Concrete (RCC)*

The most common distresses in R.C.C. bridges are as follows :

(a) Cracking

Cracks could be of Structural and/or Non-Structural in nature. The significance of cracks depends on type of the crack, length, width & depth, location, nature (orientation), type of the structure and whether the width and length increase with time and load. These cracks can be due to several reasons like:

i Plastic shrinkage and plastic settlement

ii Drying shrinkage

iii Early thermal movement cracks

iv Reactive aggregates

v Corrosion of reinforcement

vi Sulphate attack & Delayed Ettringite Formation (DEF)

vii Physical salt weathering

viii Frost damage

ix Uneven settlement

x Structural deficiency

xi Inadequate drainage

xii Vegetation growth and

xiii Constructional deficiencies etc.

Shrinkage cracks occur within first few hours after initial set due to excessive bleeding and rapid early drying and result into loss of bond to bars and exposure of reinforcement.

Drying shrinkage cracks occur in walls and slabs and take a few weeks to years for development due to loss of moisture. They create path of seepage and leakage.

Thermal contraction cracks occur within first few weeks in thick walls and slabs due to excessive heat generation. Thermal contraction cracks could result into exposure of reinforcement, seepage and leaking.

Alkali aggregate reaction can be a cause of cracks on account of internal bursting force caused by expansive reaction of certain aggregates in high alkali content situations while frost damage can occur at any age in porous concrete.

Cracks due to corrosion in reinforcement take several months or years before it leads to rapid deterioration of concrete. Corrosion induced cracks are located directly above or below the reinforcement and occur parallel to rebars. Rust stains may be visible and such cracks can indicate loss of load carrying capacity with time.

Cracks due to sulphate attack may take several years to develop mostly near or below ground level on account of sulphate salts in damp ground reacting with the hydrated cement constituents.

Physical salt weathering requires many months to many years for development of cracks in the inter tidal and splash zone or just near ground level in desert terrain, leading to deposition of salts and volume changes and final disintegration.

Settlement cracks due to movement should be recorded and the cause of the cracks should be investigated and ascertained. Such cracks can be critical and affect the load carrying capacity of the bridge.

Structural cracks may be occurred due to overstressing which in turn can be due to the overloads or due to under-designed members or due to deficiency in construction. These cracks must be evaluated depending on the location, size and apparent cause.

Cracks caused by chemical reaction, alkali silica reaction can lead to serious damages to the concrete and loss of strength and capacity.

(b) Delamination

Delaminations are separations along a plane parallel to the surface of the concrete. These can be caused by corrosion of reinforcement. Bridge decks and corners of concrete beams, caps and columns are particularly susceptible to delamination and delaminations ultimately can cause spalling of concrete.

(c) Spalling

Spalling of concrete is generally recognized to be a serious defect as it can cause local weakening, expose reinforcement, impair riding quality of deck and with time can cause structural failure. Spall is a depression caused by separation and removal of surface concrete. Major causes of spalling are corrosion of the reinforcement, overstresses, etc.

(d) Leaching

Leaching is the accumulation of salt and lime deposits white in colour on the concrete surface. These are noticed normally on the underside of concrete decks and along cracks on vertical faces of abutment walls, wing walls etc. These indicate porous or cracked concrete. Where salts (NaCl or Sulphates) are present, the migration of moisture associated with leaching may initiate severe early deterioration.

(e) Scaling

Scaling is the manifestation on the surface of loss of concrete in patches. If the process continues, coarse aggregates can get exposed and become loose and disintegrated and may eventually get dislodged. Kerbs and parapet walls are particularly susceptible to scaling.

(f) Stains

Most significant stains is that due to rust which indicates presence of corrosion. But absence of rust is not necessarily indicative of no corrosion.

(g) Voids/Honeycombing

If tapping with a hammer or rod produces a 'Hollow/Dead' sound, this is indication of poor quality concrete without proper compaction or delamination. Internal voids may not be detected by hammer rap survey. If tapping produces metallic sound then the concrete is sound and well compacted.

(h) Deformations

These are the effects of distress which may show in the form of deflection, spalling, delaminations, scaling, cracks etc. Swelling or expansion of concrete is usually an indication of presence of reactive materials. However, localised swelling may be due to excessive compressive stress in the concrete. Twisting of substructure or superstructure units may be evidence of a settlement of foundation problem.

(i) Excessive Deflections

This could be due to deficiency in the structural capacity of the superstructure or due to passage of abnormal loads. Time dependent stresses also can cause such deflections if the estimated values of creep are different from the actual values.

(j) Holes in Deck Slab

This could be due to local weaknesses in concrete or other causes.

The above mentioned distresses may also applicable to plain cement concrete bridge elements also.

3.3.3 *Pre-stressed concrete*

Most of the forms of distresses in pre-stressed concrete are similar to those in RCC. However, certain special features are as under:

(a) Cracking

Cracking in pre-stressed concrete is an indication of a potentially serious problem. Horizontal cracks near the ends of pre-stressed members may indicate a deficiency of reinforcing steel, to cater for bursting stresses. Vertical cracking in the lower portion of the member not near the support could be due to serious overstressing or loss of pre-stress. Vertical crack in the bottom of the unit and at the support may be a result of restricted movement in bearings. Vertical cracks in precast members above the neutral axis of a pre-stressed member can be due to mishandling during transportation or erection but these cracks close when dead load of the deck is applied. Transverse cracks on top of deck in cantilever bridges associated with vertical deflection at cantilever tip are result of prestress loss.

(b) Leaching

Leaching is also evidenced in pre-stressed bridges and the associated moisture movements will aggravate any corrosion risk. Particular attention needs to be paid to the concrete or mortar adjacent to joints in pre-stressed concrete e.g. box girders.

(c) Stains

Rust stains in pre-stressed concrete indicate corrosion of pre-stressing cables and should be considered a serious threat to structural integrity of the member. No rust stain does not necessarily mean there is no corrosion.

(d) Spalling

Spalling in pre-stressed concrete is a serious problem and can result in loss of pre-stress.

(e) Excessive deformations

In pre-stressed members, the abnormal deflections could also occur due to loss of pre-stress with time.

(f) Abnormal Vibrations

These could be due to slender members or combination of various reasons.

(g) Environmental Effects

The pre-stressing strands or wires are adversely affected due to hostile environment and presence of chlorides, sulphates and carbonates etc. in grout/concrete. Load carrying capacity of the member thereby gets reduced due to loss of pre-stress.

3.3.4 Steel

For various distresses in case of steel bridges and remedial measures reference may be made to IRC:SP:74 and IRC:SP:75.

3.3.4.1 Normally observed defects in steel bridges are

- i Corrosion
- ii Excessive vibrations
- iii Excessive deflections and deformations like buckling, kinking, warping and waving
- iv Fractures
- v Distresses in connections, and
- vi Fatigue cracking

3.3.4.2 Deterioration of steel

The causes of deterioration of steel are:

- i Deterioration of the protective paint systems: may be due to accumulation of debris, accumulation of moisture, flaking of the paint, cracks in the paint
- ii Corrosion
- iii Rust: It may be Light rust formation & pitting the paint surface, Moderate rust formation with scales or flakes; and Severe rust formation & stratified rust or rust scale with pitting of the metal surface.
- iv Electrolytic action - Other metals that are in contact with steel may cause corrosion similar to rust.

v Chemical or physical attack: It may be due to air and moisture, animal wastes, deicing agents, industrial fumes such as hydrogen sulphide, sea-water, and welds where the flux is not neutralized.

3.3.4.3 Abnormal deformations or movements

They may include:

i Abnormal vertical deflection

ii Abnormal horizontal deflection

iii Long-term deformation e.g. creep and sagging

iv Abnormal vibration due to traffic and/or wind

v Excessive noise due to traffic

vi Excessive wear due to traffic of members accommodating movements such as pins

vii Buckling, kinking, warping and waviness due to overloading members in compression

viii Bent or twisted members due to vehicular impact

3.3.4.4 Fracture and cracking

i Fracture: due to overloading, brittleness, stress corrosion and fatigue

ii Cracking: due to sudden change in the cross-section of members, in welds in adjacent metal because of stress fluctuations or stress concentrations.

3.3.4.5 Loose bolts and rivets

It may be due to Overloads, Mechanical loosening and Excessive vibration.

3.3.5 *Composite bridges*

In Composite bridges the distresses are normally the same as those for the masonry, concrete and/or steel bridges. However it is usually observed that the distresses like cracks are more common at the interface between two materials due to horizontal shear, the shear connectors being either absent or being of insufficient capacity.

3.3.6 *Timber bridges*

Some of the normally observed distresses are:

i. Cracking and splitting of members due to overload, ageing or under-designing of members.

ii. Abnormal deflections due to overloads or under designing or imperfect joints.

iii. Infestations decay etc., due to environmental aggressiveness.

iv. Loosening of joints due to lack of good workmanship.

3.3.7 *Cable stay bridges*

Nature of distresses for pre-stressed concrete girders and pylons would be similar to those referred in section 3.3.3 earlier. However for cable stay portion distresses generally observed are as follows:

- i Reduction of force in stay cable necessitating its augmentation or replacement of cable.
- ii Corrosion of cable/strands requiring their replacement.
- iii Disintegration of protective grout of the stay cable calling for re-grouting.
- iv Damage to stay pipes or external ducts providing outer protection to the cables needing replacement of such pipes and ducts.
- v Reduction in efficacy of corrosion protection system requiring its improvement & replenishment as necessary.
- vi Reduction in the efficacy of dampeners.

3.3.8 *Extradosed bridges*

Natures of distresses are similar to those referred in clause 3.3.7 for stay cable described above.

3.3.9 *Suspension bridges*

Distresses in Suspension bridges shall be as per the materials used and have not been enumerated separately.

3.3.10 *Miscellaneous*

Distresses in bearings and expansion joints manifest themselves in various forms and have been dealt with separately.

Although the normally observed distresses have been listed above, the bridge engineer should keep an open eye and reasoning attitude to observe any new type of unusual distress/behaviour during inspection or monitoring

CHAPTER 4

TESTING AND DIAGONSIS

4.1 Introduction

Most of the road bridges built in India are constructed in concrete. It is envisaged that concrete will continue to be the primary construction material for bridges in coming years also. Considerable investigations and research have been done in respect of concrete and more research is in progress. Based on research and field experience procedure for testing and diagnosis of concrete bridges has been updated and brought out in this chapter.

4.2 Investigation

A desk study for review of available drawings to identify vulnerable areas and components should be done before investigation. In case drawings are not available, concerned engineer should provide basic details.

Identification of human and technical resources for investigation shall be done by an expert engineer.

Investigation is to be carried out at three stages – namely a visual survey to assess the overall integrity of the bridge. A general survey including a limited amount of physical testing to the extent possible and finally a detailed investigation to determine the extent and severity of deterioration is to be carried out.

4.3 Visual Inspections

Visual inspection by an experienced bridge engineer who has previously handled similar situations is the most important basic and essential step. Generally the occurrence of distress becomes apparent much before the load carrying capacity is adversely affected. However, it is likely that in some pre-stressed concrete structures, manifestation of distress may not be very distinct and clear. Warning signals of distresses are indicated by spalling, rust stains, cracks, excessive deflections, abnormal vibrations, loss of camber, malfunctioning of bearings & joints, leakages and deformation of structural elements etc. During inspection, these signs should be verified and noted.

Arrangement of proper access to various components of the bridge is a pre-requisite prior to embarking on this task to ensure thorough inspection.

4.4 Testing Methods

A variety of semi-destructive and non-destructive testing methods are available for investigating different properties of concrete in addition to vital visual inspection described earlier. Tests are aimed at assessment of strength and other properties and to locate and obtain comparative results indicating weaker regions, cracks, delaminations and areas of lower integrity than the rest. It is essential to emphasize here, that it is not necessary to carry out all the tests in each case except the most relevant one. It is necessary to arrive at essential tests considering visual inspection report. As a matter of fact, in certain cases, engineering judgment would help to take decisions faster. Details of various tests and their suitability and availability are given in the **Table 4.1**. Different tests related to corrosion in RCC and PSC structure are mentioned under Note below the **Table 4.1**.

Table 4.1 Summary of Test Methods

Sl. No.	Name of the Test/Technique	Application and properties measured	Extent of damage to concrete surface	Type of Equipment used	Reference Codes
1	Pull-out Test (Hole drilled & insert placed in old concrete)	In-situ Compressive Strength	Minor	Hydro-Mechanical	1. E DIN EN 12399 2. ISO/DIS 8046 3. ASTM C 900
2	Penetration Resistance/Windsor Probe Test	In-situ Compressive Strength	Minor	Mechanical	1. ASTM C 803
3	Resistivity Measurements	Probability/Risk of Reinforcement Corrosion	Minor	Electrical	1. BS 1881
4	Half -Cell Potential Measurements	Rate of Reinforcement Corrosion	Nil	Electro-Chemical	1. ASTM C876 2. BS 1881-Part 201
5	Ultrasonic Pulse Velocity	Homogeneity and Quality	Nil	Electronic	1. IS13311 (Part I)
6	Acoustic Emission	Internal cracks, voids & other defects	Nil	Electronic	1. ISO/TC 135/ SC 9
7	Dynamic Response Technique	Pile Integrity & Voids	Nil	Electro Mechanical	1. IS 14893
8	Ground Penetrating Radar	Location of Voids, Thickness of Members, Delamination	Nil	Electro-Magnetic	1. ASTM D6432
9	Radiography	Location of Reinforcement, Voids, Cracks Grout defects in Pre-stressed cables	Nil	Electro-Magnetic	1. IS 2595
10	Carbonation Test	Depth of Carbonation	Minor	Chemical	1. AFPCAFREM 2. JIS A 1153 3. UNI 9944 4. ASTM C876
11	Crack Measurement	Monitoring of crack width & length	Nil	Mechanical	1. Using special Microscopes & Crack Gauges

Sl. No.	Name of the Test/Technique	Application and properties measured	Extent of damage to concrete surface	Type of Equipment used	Reference Codes
12	Trepanning Test	Measurement of Pre-stress	Minor	Mechanical	1. ASTM 837E
13	Endoscopic Test	Detection of voids in concrete and grout of prestressed cables	Minor	Electro Mechanical/ Optical	
14	Thermography	Detection of cracks, Delamination & voids	Nil	Electro Mechanical	1. ASTM D 4788 - 03
15	Petrography/ Petrographic testing is the use of microscopes to examine samples of rock or concrete to determine their mineralogical and chemical characteristics	Chemical or Alkali Concrete reaction	Minor	Laboratory Test	1. ASTM C 586-11 2. IS 2386 Part VIII
16	Impact Echo Testing	Voids & Delamination	Nil	Mechanical	1. ASTM C1383-98a
17	Schmidt/Rebound Hammer Test	Assessment of likely surface compressive Strength	Nil	Mechanical	1. IS13311 (Part 2)
18	Permeability Test	Water tightness & Porosity	Minor	Hydro-Mechanical	1. IS 3085
19	Cover Meter Testing	To check cover of reinforcement	Nil	Electro-Magnetic	1. BS 1881: Part 204
20	Chemical Analysis	Cement, Chloride, Sulphate & Water & pH values	Moderate	Laboratory Test	1. BIS 456 2. BIS 3025/R24 3. BIS 3025/32
21	Hammer Rap/ Heavy Chains	Voids & Delamination	Nil	Mechanical	1. ASTM D 4580 2.ASTM D 4580M
22	Core Test	In-situ compressive strength	Moderate	Mechanical	1. IS – 516
23	Linear Polarization Resistance (LPR)	Rate of Reinforcement Corrosion	Nil	Electro Mechanical	1. ASTM G 59

Sl. No.	Name of the Test/Technique	Application and properties measured	Extent of damage to concrete surface	Type of Equipment used	Reference Codes
24	Galvano-Static Pulse Method	Rate of Reinforcement Corrosion	Nil	Electronic	1. ASTM C 876

Note: Tests at Sl. No. 4, 5, 10, 15, 24 & 25 are related to investigation of risk/potential/rate of corrosion of reinforcement

The various testing methods are detailed in Appendix-I

4.5 Guidelines for Non-Destructive Methods

It is necessary to have detailed discussion between bridge authority and the group who is assigned the task of carrying out the tests to recognize the difficulties in the field to enable them to arrive at a realistic program.

Prior estimation of various design parameters with high and low assumptions as regards to the assessment of bridge condition is needed. This will facilitate the choice of best suited technology and fixing operating limits during the tests.

The various non-destructive and other evaluation methods have several limitations as there are various parameters influencing each test and many a time combination of different methods have to be used during investigations.

Use all necessary means so that cost remains optimum. For this purpose fix just adequate number of measurements points and use suitable alternate methods wherever available methods.

A general summary of capabilities of the various test methods to detect different forms of defects or deterioration is given in the **Table 4.2**.

Table 4.2 Capability of Defect Detection of NDT Tests

Sl. No.	Test Name	Cracking	Scaling	Corrosion	Wear and Abrasion	Voids	Strength
1	Pull-out Test (Hole Drilled & insert placed in old concrete)	N	N	N	N	N	Good
2	Penetration Resistance/ Windsor probe Test	N	N	N	N	N	Good
3	Resistivity Measurements	N	N	Good	N	N	N
4	Half-Cell potential Measurements	N	N	Good	N	N	N

Sl. No.	Test Name	Cracking	Scaling	Corrosion	Wear and Abrasion	Voids	Strength
5	Ultrasonic Pulse Velocity	Good	N	N	N	Good	N
6	Acoustic Emission Test	Good	N	N	N	Good	N
7	Dynamic Response Test	Good	N	N	N	Good	Good
8	Ground Penetrating Radar	Good	Good	N	Fair	Good	N
10	Radiography	Good	N	N	Fair	Good	N
11	Carbonation Test	N	N	Good	N	N	N
12	Crack Measurement	Good	N	N	N	N	N
13	Trepanning Test	N	N	N	N	N	Good
14	Endoscopic Test	Fair	N	Fair	N	Good	N
15	Thermography	Fair	Good	Poor	Fair	Good	N
16	Petrography	N	N	Poor	N	N	N
17	Impact-Echo	Fair	N	N	N	Fair	N
18	Schmidt/Rebound Hammer Test	N	N	N	N	N	Good
19	Permeability	N	N	N	N	Good	N
20	Cover Meter	N	N	N	N	N	N
21	Chemical Analysis (Sulphate & chloride Content)	N	N	Good	N	N	N
22	Hammer Rap/Heavy Chains	N	N	N	N	Fair	N
23	Linear Polarization Resistance (LPR)	N	N	Good	N	N	N
24	Galvano-Static Pulse Method	N	N	Good	N	N	N

LEGEND: G: Good, **F:** Fair, **P:** Poor, **N:** Not Suitable

4.6 Decision Matrix

Following decision matrix shall be followed for maintenance, repairs, rehabilitation and strengthening of existing structures. **Table 6.2** prescribes the condition of the component of bridge to be categorized in to five groups namely Excellent, Good, Fair, Poor and Critical as below:

Table 6.2 Condition States for Bridge Components

Sl. No.	Condition State	Condition	Extent & Severity of Distress	Type of maintenance
1	Excellent	Sound structural condition; component do not individually or as a whole impair the strength, stability, traffic safety, durability and serviceability of the structure.	Only constructional deficiencies may be present. Extent of deficiencies is nil or insignificant. Severity of deficiencies is very low.	No need of repair except routine maintenance.
2	Good	More than satisfactory structural condition; component do not individually or as a whole impair strength/stability: traffic safety, durability and/or serviceability of the structure might be impaired slightly in the long term.	Extent of deficiencies is minor; Severity of deficiencies is low.	Specialized maintenance and repairs may be needed at convenience.
3	Fair	Satisfactory structural condition; strength, stability and traffic safety of the components/ structure is assured however considerable reduction is possible in the long term; serviceability and durability of the affected component is reduced and durability of the structure might be impaired considerably in the long term.	Extent of deficiencies is major; Severity of deficiencies is medium.	Specialized maintenance and repairs needed soon.

Sl. No.	Condition State	Condition	Extent & Severity of Distress	Type of maintenance
4	Poor	Structurally deficient bridge; strength/stability/ traffic safety no longer assured; durability may be affected in short term; monitoring is required; restriction of use of the bridge may be needed.	Extent of deficiencies is large; Severity of deficiencies is high.	Rehabilitation/ replacement on program basis is needed; measures for reconstruction or warning signs for upholding traffic safety might be necessary in the short-term; detailed investigations and economic analysis is required for justification of funds.
5	Critical	Weak structural condition; partial failure or risk of total failure of the component or as a whole; durability of the structure is no longer ensured; immediate propping of the structure and closing may be required.	Extent of deficiencies is very large/ expensive; Severity of deficiencies is very high.	Repair/rehabilitation/ replacement is required immediately; design strength, expected serviceability and desired remaining service life of a component/bridge can no more be achieved with economic costs.

The above mentioned criterion is subjective and shall be quantified on the basis of various distress types and corresponding range of values along with the photographs.

4.7 Full Scale Load Testing of Bridges

The above techniques present possibility of making an assessment of the overall condition of the bridge or changes in the condition and of detecting faults. But it is often difficult to analyse the effects or defects or deterioration on the overall performance of the bridge or on the stresses in individual components. A full scale load test can, therefore, be useful. Load tests can be expensive and on larger bridges they require considerable planning, involve many people and demand the use of sophisticated equipment. Testing in remote locations can present additional difficulties. Particular care is needed for bridges with brittle failure modes. However, load testing can often be justified where the effect of defects and/ or deterioration on load capacity cannot be determined by analysis alone. However, a decision to carry out full scale load testing should not be undertaken without a serious thought. The procedure and acceptance criteria for load rating of bridges is given in IRC:SP:37 and for load testing IRC:SP:51. The instrumentation and equipment needed for load testing is also elaborated in the above mentioned guidelines. The following may also be considered with regard to load testing of bridges:

(a) Bridge testing is both an art and a science. In its simplest form, load testing involves measuring the response of the bridge to a known applied load. Considerable experience is required to know where to locate gauges and to determine load increments and the maximum load to be applied to prevent damage to the bridges. The load is applied, usually by vehicles although occasionally by dead load or through cables, to induce maximum effects. Where measurement of stress at given location under known load is all that is required, data processing may not be a major consideration. In other cases, the amount of data recorded is often extensive such that automatic data recording and analysis is highly desirable. It is also preferable that this be done on site as the test progresses so that deviations from anticipated behavior are known and the necessary changes in procedure or equipment can be made. Because concrete properties are required to define stress values (from strain measurements), samples must be taken from the structure.

(b) Jacks, duly calibrated, can be used to measure the reactions at the supports of the structure. This may be required for such purpose as assessing the impact of thermal gradients, or the re-distribution of stresses due to creep, settlement or faulty construction. In addition to the requirement for suitable jacking points the deck joints must be removed together with any other feature which might interfere with the free movement of the jack. Alternatively, for better accuracy and to obtain direct reading, appropriate and duly calibrated load cells, may be used to give support reactions within a range of 0.3 to 1.0 %.

CHAPTER 5

REPAIRS AND STRENGTHENING TECHNIQUES – GENERAL

5.1 Criteria for Selection

It is intended to cover in this chapter only the more important techniques and materials used for bridge repairs and strengthening. The maintenance techniques, described in IRC:SP:35 for inspection and maintenance of bridges, are not repeated. The criteria for selection of methods, materials and techniques for repairs and strengthening would be governed by the following factors:

(i) Bridge condition
(ii) Type of the distress
(iii) Causes of distresses
(iv) Location, Extent and Severity of distress
(v) Availability of materials, equipment and techniques
(vi) Efficacy of the materials and techniques in the preservation and/or enhancing the load carrying capacity of the structure
(vii) Importance of the bridge
(viii) Time available
(ix) Life Expectancy, future maintenance
(x) Aesthetic
(xi) Feasibility of traffic diversion
(xii) Structural system of the bridge superstructure
(xiii) Type of substructure and foundations
(xiv) Waterway condition
(xv) Type of environment in which bridge is located

5.2 Foundations

It is not possible to evolve a general method for repairs and/or strengthening of foundations (as they are generally out of sight).Each case has to be analysed individually and may require special investigations. Most repair works for foundations are in the category of protection and strengthening. Some examples are given below:

i) Scour and erosion protection
ii) Repair of washed away/damaged protection works
iii) Repair of foundations built on soft strata subjected to erosion
iv) Strengthening of foundations necessitated by additional load due to settlement of soft strata under the foundation

v) Remedying the effects of horizontal movements of abutments

vi) Strengthening of foundations due to widening of a waterway or road

vii) Extending existing foundations

viii) Repair of underwater/ submerged bridge components

ix) Repair of eroded/abraded concrete surface caused by high velocity of particles in the water

x) Impact due to boulders etc.

Foundation movements can substantially increase the loads and moments in some parts of the superstructure by redistribution. This must always be checked for. Given the wide range of materials and repairs techniques the choice of the most appropriate technique is difficult. Nature of distresses in concrete bridge structure together with important remedial measures in principle are given in **Table 5.1**. In addition to this, remedial measures are indicated according to component of the bridge and distressed observed and which are briefly, given in **Table No. 5.2**

TABLE 5.1 Distresses in Concrete and Principal Remedial Measures

Sl. No.	Nature of Distress	Cause of Distress	Principal Possible Measures for Remedy
1	• Cracking • Delamination • Spalling • Disintegration	**(A) Mechanical** Impact, Overloading, Movement, Settlement, Explosion, Vibration, Seismic	• **Crack Filling:** By Injection of suitable grouting and/or filler material after sealing them, Active cracks due to corrosion, ASR or foundation settlement etc. should not be injection grouted. • **Concrete Restoration:** By Hand applied mortar, Spraying of mortar (Guniting), Spraying of concrete (Shotcreting), Recasting of concrete, Replacement of old element by new one. • **Structural Strengthening:** By Addition of new reinforcement externally, Bonding of steel plates, Wrapping/ Bonding by Fiber Reinforced Plastic (FRP), Encasing the member partly or wholly either by concrete or by mortar or ultra-high performance concrete reinforced or non-reinforced, Addition of new reinforcement by drilling holes & embedding in concrete, Prestressing by external cables.

Sl. No.	Nature of Distress	Cause of Distress	Principal Possible Measures for Remedy
2		**(B) Chemical** Alkali Aggregate Reaction, Aggressive Agents(Acids, Salts (sulphates, phosphates, etc.), Biological Activities	• **Protection against Ingress:** By Sealing & Injection of cracks (for dormant / dead cracks only), of cracks, Impregnation of surface, Application of membranes, Surface coating, Providing external panels. • **Moisture Control:** By Coating of the exposed surface, Cladding over the surface, Impregnation of hydrophobic material. • **Increasing Resistance to Chemicals:** By Overlays, Coatings, Impregnation
3		**(C) Physical** • Shrinkage, • Erosion, • Wear and Tear, • Fire	• Protection against ingress • Moisture control • Structural strengthening • Increasing Physical Resistance: By Overlays, Coatings, Impregnation
4	Uniform Corrosion: Cracking & Delamination	Carbonation of Concrete	• Preserving or restoring passivity • Control of Anodic areas.
5	Pitting & Stress corrosion: Spalling	Corrosive contaminants: Sodium chloride, Calcium chloride & Others	• Cathodic control • Cathodic protection • Chloride of anodic areas • Restoring passivity
6	Scaling of steel: severe loss of surface area	Severe progress of corrosion due to above effects	Local replacement of steel reinforcement or part or full replacement of the member

Table 5.2 Remedial Measures for Various Components of Bridge

Sl. No.	Name of Component	Type of Distress	Remedial measures
Foundations			
1	Open Foundation	(a) Erosion of soil/ exposure of footing	(a_1) Protection byboulders, crates, concrete blocks etc. (a_2) Floor protection (a_3) Construction of spur / dykes
		(b) Undermining	(b_1) Filling undermined portion by non-erodible material and protection as per (a_1) above. (b_2) Grouting (b_3) Apron
		(c) Cracking and spalling	(c_1) Treatment of cracks and spalls Jacketing
		(d) Settlement	(d_1) Regaining stability by base treatment, strengthening by increasing size and modification of footing by re-building new footing around it.
2	Pile Foundation (Concrete piles) including caps	(a) Erosion of concrete above scour level and splash zone	(a_1) Rectification by high strength concrete (a_2) Steel lining
		(b) Settlement	(b_1) Stabilizing of soil (b_2) Review design of foundation & sub-structure and provide suitable measures including redesign & re-construction of necessary components (b_3) Modification of pile group by adding new pile/piles
		(c) Disintegration/cracking	(c_1) Removal of damaged concrete and Re-building of section by suitable concrete in the damaged portion

Sl. No.	Name of Component	Type of Distress	Remedial measures
3	Well Foundation	(a) Tilting & Shifting beyond the initial levels at the time of commissioning	(a_1) Review design of foundation & sub-structure and provide suitable measures including redesign & re-construction of necessary components
		(b) Cracking	(b_1) Treatment of cracks and spalls (b_2) Stitching (b_3) Filling the well with concrete to relieve load on steining
		(c) Excessive scour	(c_1) Garlanding with or without Peripheral Piling
		Sub-Structure	
4	Abutment, Pier Abutment caps and Pier cap	(a) Crushing, disintegration cracks, spalling, honey combing etc.	(a) As per **Table 5.1(1)**
		(b) Lateral deflection beyond design limit	(b) Improvement/control by stiffening
		Bearings	
5	Steel Rocker and Roller Bearings	• Tilting • Flat roller • Displacement • Breaking of lugs and keys • Sliding in transverse direction • Corrosion of steel • Pedestal damage	• Resetting/Re-alignment • Cleaning and greasing • Replacement
6	Elastomeric Bearings	• Damages including embrittlement of elastomer • Cracking and tearing • Displacement	Replacement
7	Pot Bearings	• PTFE/Elastomer and fixtures • Displacement • Failure of elastomeric piston/disc	Replacement
Note : Repairs for bearings can be used in isolation or combination			

Sl. No.	Name of Component	Type of Distress	Remedial measures
Super-Structure			
8	Girders Beams and Slabs	• Deflection • Cracking (dead / dormant), spalling and damage to Concrete • Displacement	Strengthening in shear and flexure by steel plates, FRP or external prestressing Treatment by grouting and/or filler material micro concrete and by FRP Realignment by lifting and correcting the position of superstructure
9	Deck system	Cracks (dead / dormant), spalling, peeling and damage to concrete	• Overlaying • Re-casting
10	Expansion Joints	Non-functioning of joints due to Clogging or wearing out and failure of anchoring system,	Replacement by new modern joints.
11	Appurtenances	Damages and non-functioning	Repairs & replacement as necessary
12	Super structure of balanced cantilever bridge	Sagging at the end of balanced cantilever	Construction of continuity girder
13	Approach slab	Settlement and damage of slab	Relaying of slab

Some of the repair works carried out for foundations is described below. IRC:89 shall be used for more guidance.

(1) **Erosion Problems**

Gabions, mattresses, rip-rap, boulders, stone/concrete block pitching, geo-textiles bags/tubes, grouting etc, can be used for erosion of bed as well as banks. For the details of materials, design etc. Handbook for flood protection, anti erosion and river training by Central Water Commission may be referred.

(2) **Protection against Scour**

Excessive scour is one of the major factors that may cause or lead to structural failure or distress of the foundation. The degree of damage depends on factors such as stream bed material, intensity of discharge, silt charge, obliquity of stream flow and shape of the structure. For deciding the extent and type of repair for scour, ascertaining the causative factors, such as change in the alignment of the stream, an inadequate waterway or the presence of debris, is of great help. Providing the most effective solution to a scour is challenging and may require model studies.

Spur dykes, jetties, deflectors and other measures may be constructed to deviate water away from a fill, bridge pier, or abutment. Only correctly designed and constructed training works are helpful in controlling scour which may vary from simple solution such as replacement of displaced material to complicated solutions like jacketing, sheet piling, strengthening the footing or training works.

Piers and abutments may be protected or repaired by placing sheet piling Sheet piling should be driven to a depth where non-erodible soil conditions or rock exist. The overhead clearance required or under substructures may be a major disadvantage in deciding sheet piling.

If supporting material has been removed from under a large area of the footing, consideration should be given to redesigning the foundation, including filling the void with concrete. In some cases, the footing may be extended by using sheet piling as forms for the extension and as stay in-place protection against the scour. In case of exposed supporting piles, particularly if they are short it may be necessary to drive supplemental piles that are as a part of the extended foundation.

Around piers it is common to provide what is known as 'garlanding technique'. In this, very heavy concrete blocks or stones of designed size and weight are placed around the pier foundations below the bed level by excavation. This technique can also be used during floods by dropping heavy stones or blocks from a distance on upstream side depending upon the velocity of water and weight of stones after trials. This would save excavation and special arrangements in Perennial River.

(3) Foundation on soft rock subject to erosion can be protected by reinforced concrete curtain walls enclosing the footing.

(4) The bearing capacity of the soil can be increased by injecting cement or chemical grout such that grout pressure does not exceed the overburden pressures.

(5) Rock or ground anchors are often used for erosion around abutments taking into account various factors likely to affect the bearing capacity and durability of the anchoring system.

(6) **Extension of Existing Foundations**

This would be necessary while widening an existing bridge. The foundation, substructure or superstructure of widened portion may be of same or the different material and structural configuration.

(7) **Liquefaction of Foundation Soil**

Some foundation failures could be the result of excessive soil movements especially due to liquefaction during earthquakes. There are two approaches to retrofitting that will mitigate these types of failures. This may be mitigate either by replacement/stabilization of soil or strengthening of the structure to withstand excessive relative displacements.

Possible methods for soil stabilization include: Lowering of ground water table, Consolidation of soil by vibro-floatation or sand compaction; Placement of permeable over-burden; grouting or chemical injection.

At a site subjected to excessive liquefaction, methods to strengthening of the structure as well as stabilization of the soil may be necessary.

(8) **Underwater Work**

While dealing with underwater work, it will be relevant to refer to underwater inspection also. Inspection of underwater portions of the structures is very difficult because of the harsher environment, poor visibility, deposition of marine organisms etc. To do an effective underwater inspection, it is necessary to deploy properly trained and equipped personnel. The quality of underwater inspection must be same as that of above water. Cleaning the marine growth from underwater portions of a bridge is always necessary. Visual inspection is a primary work of detecting underwater problems. In turbid waters, the inspector should use tactile examination to detect flaws, damage or deterioration. In some cases sophisticated techniques, ultra-sonic thickness gauges, computerized tomography or TV monitors may be used. After the initial identification of the damage, it may be necessary either to expose the member by means of a cofferdam and dewatering or by providing a small air-lock as described later for the purpose of detailed examination and carrying out repairs.

Underwater photographic techniques are also available wherein damages are detected by divers who can take photos, videos or scan the various components of affected areas, which can be displayed on a screen/monitor on the bridge deck.

Acoustic microscopic measurements or tomography can be used for identification of defects of underwater bridge components. But, Sonar procedures for mapping scour are useful.

For underwater repairs the surfaces of the pile or well or pier have to be cleaned of the dirt and other foreign materials and after removing the cracked and unsound concrete, the surface shall be ready for receiving new concrete. It is often useful to provide temporary cofferdam. Suitable priming coat by materials like moisture compatible may be helpful to ensure proper bonding. Placing of concrete under water can be carried out with the help of conventional underwater bucket/tremie concrete, pre-packed concrete, bagged concrete or pumped concrete depending on the site conditions.

The piles or wells which are substantially deteriorated by corrosion or other defects can be provided with integral jackets which may or may not be reinforced depending on the thickness of the jacket. Joint of the jacket at its ends have to be properly detailed out and treated with suitable adhesive. Grouting with quick setting cement or epoxy can also be carried out where necessary.

Methods used to perform underwater sealing and repairing of cracks by suitable grout injections are similar to methods used above water, except that for underwater use, grout must be water insensitive. Before application of surface sealer, cleaning is necessary. If oil or other contaminants are present in the cracks, and the sealer is used for restoring the strength of the cracked concrete pier or pile instead of simply blocking the free entry of water in the crack, bonding will be improved by mixing detergents or special chemicals with a water jet to clean the crack interiors. After all cracks are prepared and sealed and the nipples positioned, the low viscosity adhesive is injected under pressure into the cracked area.

Various methods used to prevent the corrosion of steel piling in sea water, include application of protective coatings, encasement of steel in concrete or a combination of these procedures. Cathodic protection can also be used.

5.3 Repairs to Masonry Structures

Existing masonry bridges are to be considered as historical landmarks and need preservation. Strengthening and widening maintaining the same appearance. Widening is usually not possible but strengthening can often be done. Strengthening of Masonry Bridges ensuring pleasant appearance is a delicate task and expertise is required. Some of the defects and remedial measures for stone or brick masonry arch bridges are given below.

(i) **Loss of Bond for the Crown/ Key Stone:** Flat jacks can be used for pushing the stone back to its original position. Generally, low pressure grouting is done to strengthen the old mortar. A suitable pre-bagged grouting material similar in mechanical properties to the existing jointing material should be used for effective bond.

(ii) **Longitudinal Cracks along the Direction of Traffic:** it is possible to rake mortar joints and refill with a polymer modified mortar similar in mechanical properties to the existing jointing material. Fine material grouting in the form of injection can be adopted for remedial measures. Grouting can also be done by using suitable polymer modified grouts depending upon size of cracks and mechanical properties expected. However it must be mentioned that the depth of penetration of grout is important, as usually it is not possible, to suspend traffic. If possible, the portion of earth fill could be removed to ensure that penetration is limited to masonry only. This can be achieved by controlled grouting and proper spacing of inlet for grouting. Fiber Reinforced Cementitious Matrix (FRCM) can also be used to strengthen the bridge by means of wrapping.

(iii) **Transverse Cracks:** Injection of, polymer or suitable grout as per size of cracks will provide a good bond between stones and brick masonry.

(iv) **Strengthening of Arch Rings:** The arch ring can be strengthened either by adding material to the intrados or to the extrados. The arch can be strengthened by constructing an RCC Arch below the masonry arch with suitable dowel connection. The intrados of the masonry arch can be used as formwork. For supporting the RCC Arch, necessary jacket can be provided to pier and foundation with suitable anchorage with old structure. In order to arrest lateral movement of arch, suitable reinforcement can be placed connecting rings of both sides of arch and anchored at ends.

(v) **Strengthening of Arch by helical reinforcement:** Using helical stainless steel ties and bars, which have good axial strength combined with sufficient lateral flexibility which allows the structure to move naturally and therefore minimize additional stresses. Details of helical reinforcement and its embedment at various locations of arch as given in **Fig. 5.1** below:

Fig. 5.1(a) A view of Heical Reinforcement

Fig. No. 5.1 (b) A Helical Reinforcement Provided in one of the Arch Bridges

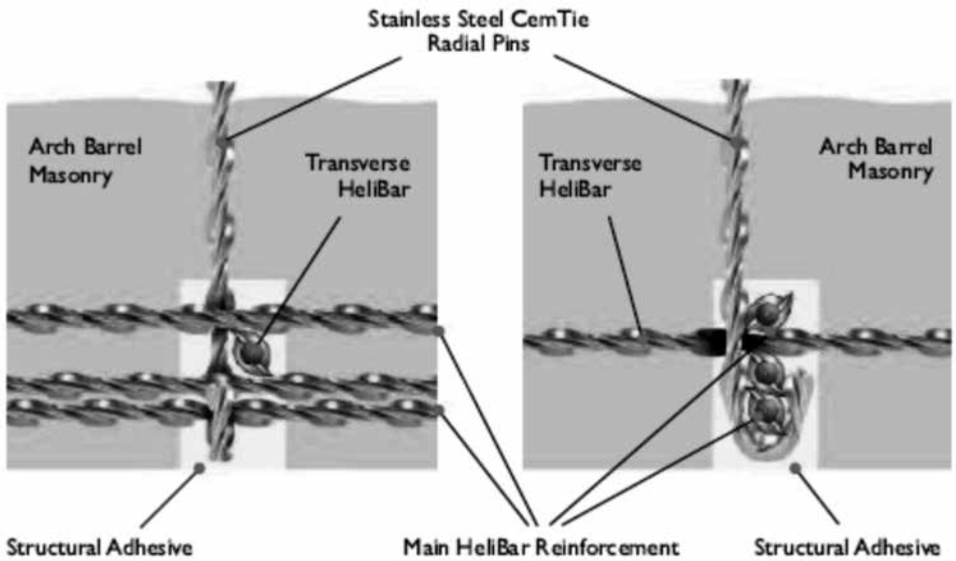

Fig. No. 5.1 (c) Provision of Helical Reinforcement for Arch

The helical reinforcement can be used for minimizing movement in spandrel walls, separation of brick arch rings, cracking in arch barrel, delaminated masonry, cracked piers, wing walls and abutments, cracked and unstable parapet walls.

A more effective, but at times an expensive, treatment is to remove the fill and cast the extra required thickness on the extrados of the arch. Usually, a full ring is cast but occasionally only the end quarters are strengthened to act as cantilevers and reduce the effective span of the arch. Normal concrete placing techniques are satisfactory. Replacement backfill may be with normal or lightweight concrete. The latter will reduce dead load on the foundations but may also reduce the factor of safety for stability of the substructure.

Another expedient which is satisfactory where the increase in load carrying capacity is relatively small, especially for small span bridges, is to cast slab at road level to act as an auxiliary deck. Strengthening of soffit of arch including arch elements such as spandrels can be done by the use of tie bar and jacketing with ultra-high performance fiber reinforced concrete or CFRP.

The fiber reinforced cementitious matrix is similar to carbon fiber fabrics and are embedded in a ductile cementitious matrix instead of epoxy, which has extremely high compressive strength. This material has many advantages over CFRP and they are:

- FRCM allows the application on wet surface, provides a high thermal resistance and similar fire resistance as concrete. Helps maintain ductility of structures strengthened.
- Suitable for enhancing flexural, shear and compressive strength of structural elements.
- Low thickness with a minimum increase of the cross-section on the structure. Keeps the geometry and appearance of the strengthened elements.
- Long lasting, it is highly resistant to abrasion, not affected by corrosion process, withstands marine environments and freeze-thaw cycles.

- Economical system with no maintenance. Very easy and quick to use, saving labour costs and reducing use of auxiliary tools.

5.4 Repairs to Concrete Structures

The majority of bridge structures will be either concrete, RCC or Pre-stressed Concrete, techniques for repairs to concrete structures are described in a separate Chapter 6.

5.5 Repairs to Composite Structures

Comparatively very few defects have been reported with well-designed and fabricated shear connectors in composite structures. Problems with concrete decks in composite structures are essentially of the same kind and order of magnitude as those found in concrete decks in regular structure. It is likely that some early structures are seriously inadequate with regard to shear connectors for the heavier design loads now specified. The same can be also said for the main load carrying structural steel components.

Difficulties may be encountered with deck replacement or even major deck rehabilitation and strengthening operations in those composite bridges in which residual relieving stresses have been introduced by sophisticated erection procedures combined with an elaborated casting sequences for the bridge deck. Such cases would be very few in this country. Prior to concreting deck slab, use of very high pressure water jetting say usually beyond 70 MPa, to remove the concrete around shear connectors is considered preferable to jack hammers so as to minimize damage.

5.6 Repairs to Steel Structures

5.6.1 *Deck Replacement*

Many of the old bridges, usually truss or arch bridges, having either warped steel plates with a bituminous surfacing or a concrete deck often get corroded due to insufficient waterproofing of steel plates.

Bridge decks can be replaced by new concrete decks or by new orthotropic steel decks, specially for reduction in weight or for widening. The use of light weight concrete is often preferred for reducing the loads. Bolting is the preferred method of connecting the new deck system to existing structural members.

Steel grid decking can either be left open or filled with concrete.

5.6.2 *Strengthening of Structural Members*

Strengthening usually involves more conventional techniques such as replacement of members, diaphragms or plate bonding either with bolts or welds. Strengthening of plate girders can be done by adding stiffness to flanges, webs and diaphragms or by external pre-stressing.

5.6.3 *Repair of Cracks*

Cracks can be due to any one or a combination of the following reasons:

i) High stress concentrations joints/location due to in appropriate detailing or loose connections may be avoided by champher

ii) Increased traffic loading

iii) An unexpected secondary loads

iv) Poor workmanship

v) Reduction in strength due to corrosion

Crack repair methods depend on the root cause of crack development/formation as well as its location, width etc. The structure and especially those components which influence the overall safety and stability of the structure should be analyzed.

5.6.4 *Action to be taken when a crack is detected or suspected in welded steel bridge girders*

Reference is invited to IRC:SP:104 for fabrication & erection; IRC:SP:74 for repair, rehabilitation of steel & steel composite bridges and IRC:SP:75 for retro-fitting of steel bridges.

i) Location, length, orientation and ends should be marked distinctly on the members as well as sketches to monitor crack propagation. Photographs and videos may be taken and appropriately labeled.

ii) If necessary, cracks should be examined in detail by NDT tests like magnifying glass, dye penetration, ultrasonic etc.

iii) If more identical details exist on the girder, they should also be inspected in detail.

iv) Cracks should be fully documented in the bridge inspection register and action initiated for its early repair.

v) If situation warrants, suitable speed restriction may be imposed.

vi) Significance of crack on the load carrying capacity of the girder should be studied and retrofitting or repair scheme should be planned.

vii) Repairs can be made either by replacement of cracked section, plate bonding, welding, strengthening etc.

5.6.5 *Underwater Welding*

Arc or gas welding is commonly used in underwater repairs. However this process is expensive.

5.6.6 *Use of Steel Arch superposition Scheme*

This can be used to strengthen old truss bridges. The strengthening scheme consists of superimposed arches, hangars and additional floor beams. The concept of combining a truss with an arch is by no means a new system. The idea is that a light arch can carry a significant load if properly supported laterally. In this case, the truss with its cross-beams provided the lateral support while the arch in combination with the hangars and additional floor beams provides the increased load carrying capacity. Additional floor beams and hangars are used for two reasons:

i) The more uniform the load distribution, the more efficient the arch will be in carrying the load.

ii) The floor systems of many old truss bridges get deteriorated and are sometimes under-designed and unreliable.

The scheme of strengthening by a steel arch superposition is illustrated in **Fig. 5.2**

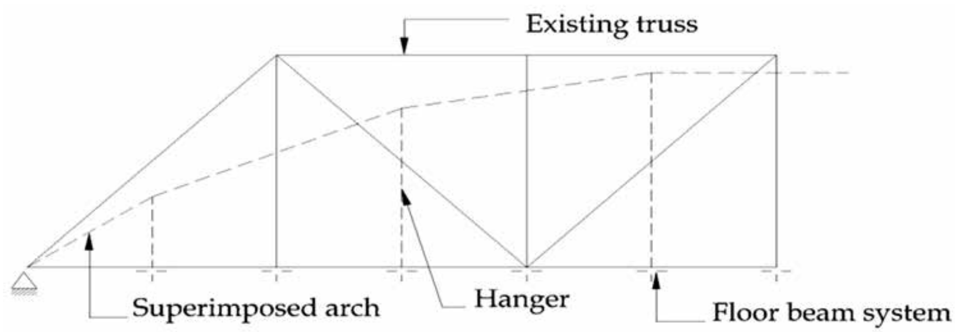

Fig. 5.2 Strengthening by a Steel Arch Superposition

The thrust of the arch can be resisted by one of the following means:

- The abutments, provided they are adequate and in good condition, or they can readily be repaired or strengthened.
- A reinforced lower chord.
- Superimposed cables or rods.
- Properly designed and detailed stingers or floor slab.

The arch superposition scheme can be considered as an overall strengthening measure. The load carrying capacity of the entire structure is upgraded, thus allowing the live load to be increased. There is no need for temporary shoring or jacking for the installations of the super positioned elements. Increase in dead load can be expected to be in order of approximately 15 % to 20 % percent. Slender arch contributes only modest amounts of additional stiffness to the truss.

5.6.7 Use of External Pre-stressing: Reference is invited to IRC:SP:67 & IRC:24.

5.6.8 *Excessive Vibrations*

These can be overcome by suitable structural alterations and increase damping for which dynamic behavior of structures have to be investigated.

5.7 Repairs to Timber Structures

Except for giving treatment to wood, there are no special techniques for repairs of timber structures. The distressed members could be either replaced or strengthened with steel plates.

CHAPTER 6

REPAIRS AND STRENGTHENING TECHNIQUES FOR CONCRETE BRIDGES

6.1 Repair Works

Repair works may be categorized as below:

(i) Repair of the concrete surface
(ii) Repair of cracks
(iii) Repair of corroded reinforcement
(iv) Repairs related to high tensile steel in Pre-stressed concrete (PSC)
(v) Repair of honeycombed concrete voids and holes
(vi) Repairs to damages/disintegrated/crushed components

6.1.1 *Preparation of the surface*

In all cases where of repaired, the condition of the existing concrete in the exposed area is of primary importance in the durability of the repair. The latter can be seriously compromised if there is a poor adhesion between the fresh concrete of the repair and the existing concrete surface. Therefore, it is important that the contact surface to be repaired is in hard and sound condition. In general, all foreign / loose / soft /damaged and fractured concrete materials are removed that might affect or other impair the repair and must be properly treated. The methods for preparation of surfaces can be categorised as below:

- Mechanical methods
- Hydraulic methods
- Thermal methods and
- Chemical methods

The choice of a suitable method depends on the type, location, extent and severity of distress as well as the thickness of the layer of the damage in the structure which has to be treated. The thermal and chemical methods are rarely used and are restricted to special circumstances and hence have not been described here.

i. Mechanical Methods

The Mechanical equipment is preferable, as it is more intensive, reliable and speedy. While choosing and applying mechanical methods, it must be ensured that the sound concrete and the reinforcement are not damaged. If necessary, trials should be carried out. The commonly used mechanical methods are jack hammers, milling, chipping, sand blasting, steam blasting and compressed air cleaning. During a mechanical surface treatment of concrete, dust will always occur, and therefore the surface must be free of dust before start of the repair work.

ii. Hydraulic Methods

Hydraulic methods such as use of water jetting or blasting can also be used and are considered preferable to jack hammers for damage prevention. A water jet with 10 to 40 MPa pressure at the jet shall remove loose particles, scaled concrete or vegetation coatings. This method is not applicable for roughening of solid concrete surface. High pressure water-jet the pressure 40 to

120 MPa at the jet is most efficient for removal of soft areas of the concrete surface. In hydro-jet method, with the jet pressure at 140 – 240 MPa a deep penetration into the concrete or even cutting grooves is possible. This method is essentially free of vibrations, but there will be deep penetration of moisture into the concrete and needs careful handling or else could be hazardous.

6.1.2 *Bonding agents*

(a) General

Bonding agents are recommended to improve the bond between old concrete and the repair concrete. There are two types of bonding mechanisms as below:

- Physical bond through adhesion and cohesion; and
- Chemical adhesion through reaction with the surface

In most cases, both types of bonding exist in combination. There are various types of bonding agents.

(b) Cement Paste/Slurry

The cement paste with a low water/cement ratio is used as bonding agent which is applied by brush on to the surface to be repaired.

(c) Cement Mortar

A cement mortar consisting of equal parts of cement and sand along with water of requisite viscosity may be used as bonding agent. It can, also be used for the repair itself wherein the coarse aggregate has been absent.

(d) Polymer Modified Cement Paste/Mortar

The polymer mixed into the cement paste or cement mortar via the mixing water can also be used as bonding agent for greater adhesion. The plasticizers with certain portions of solid substances such as Styrene Butadiene Rubber (SBR) polymer or acrylic resin may be added to the mix. The emulsions may also be used. The bonding effect depends on the type of polymer/additive being used. These additives are used to improve the workability and water retention capacity in addition to improve the bond strength.

(e) Resins

There are two basic types of bonding agents made of resins namely; emulsified agents and normal agents. The first case consists of a combination of a water emulsified epoxy, resin, a polyamide resin hardener and a filling material. The epoxy/resin and the hardener are initially mixed together before placement. If required, the mixture may be diluted with water. In the two component resin bonding agents, a pure resin-hardener-mixture is used, with or without fillers. Filler may be added in a suitable design ratio. Filling materials mixed with resins are used for following reasons.

- Filling materials prevent a deep penetration of resin into the old concrete.
- Filling materials prevent a penetration of resin into the new concrete
- Filling materials are less expensive than epoxy/resins and
- Resins with fillers can be placed in thicker layers.

6.1.3 *Repair of concrete surfaces*

6.1.3.1 Surface protection measures

A concrete surface exposed to climatic effects will change its structure and physical appearance with time. Therefore, the durability of a structural element cannot be assessed only from the physical appearance of its surface.

The composition of the external surface layers is comparatively different from the interior of the structural element, in particular the cement content is more towards the surface. The concrete surface, for desired aesthetic appearance, should not affect the durability of the structural element. If the concrete is to be protected from the external influences and enhancement of an already existing weathering is of concern, measures need to be taken to either minimize or stop this process. The following measures in the order of increasing surface protection can be used:

- Hydrophobation
- Impregnation
- Painting
- Sealers and
- Coatings

There is a difference in how protection is achieved from impregnation systems and sealers and/or coating systems. Protection is achieved in the impregnation system through a prevention of a capillary absorption of water by the concrete. Depending on the material used, this effect will be achieved by a hydrophobation of the pores at the surfaces or by a narrowing of the capillary ducts, which result from a film formation on these surfaces. Sealers or coatings lead to a closed thin film on the surface.

6.1.3.2 Materials and techniques of application

(a) The efficiency of an impregnation basically depends on the preparation of the surface and on the required depth of impregnation. Requirements for impregnation material are small molecular size and low viscosity. The absorption is accomplished via the capillary voids of the concrete. The proportion of capillary voids increases with increasing water/cement ratio. The impregnation liquid must be placed on the concrete surface in an amount to fill the voids. The application may be accomplished by means of a brush, lambskin roller or by spraying. Depending on the absorptive capacity of the surface, several repetitions may be necessary. For solvent containing impregnation systems, the concentration of the solution during the first application may require thinning to achieve a deeper penetration. Penetration depth is especially important where traffic wear is expected. Therefore, impregnation protection systems are only suitable where the concrete surface will not be removed by abrasion, damaged or locally affected by the formation of cracks.

While impregnation with resins may be successfully used on horizontal surfaces, hydrophobizing impregnations are not suitable for horizontal surfaces where water will stay on the surface. Therefore, the primary field of application of hydrophobizing impregnations is on vertical or sloped surfaces, where the water can flow off easily.

(b) **Sealers:** Is a treatment to concrete to reduce the surface porosity and strengthen the surface. These are also often referred to as surface primers or sealers.

- The pores and capillaries are partially or totally filled.

- Treatment leads usually to a discontinuous, thin film on the surface.
- Binders may be Low Viscosity, Filler Free compositions based on organic / inorganic polymers such as Acrylics, Epoxy, PU, PMMA or Silicone Resins.

(c) **Coatings:** Is a treatment to produce a continuous protective layer on the surface of the concrete. Thickness is typically of 0.1 to 5.0 mm. Particular applications may require higher thickness than 5 mm.

The materials for coatings to be used shall be organic / inorganic polymers such as Acrylics, Epoxy, PU, PMMA or Silicone Resins (individual or in combination) with or without fillers. These coatings may also be referred to in common terminology such as but not limited to: Anti-Carbonation Coatings, Poly-Urethane (PU) Coatings, Bridge Deck Waterproofing Systems, Coal-Tar Epoxy Systems, Epoxy or PU Coatings. Compared to sealers, coatings have an increased resistance to the diffusion of internal moisture and provide an additional protection against mechanical influence.

A differentiation should be made between thin and thick coatings. Thin coatings, will follow the contour of any unevenness of the surface while a thick coating will smooth out any unevenness of the surface. Coatings should also have the capability to bridge cracks. This requires a high elasticity of the coating material.

Some systems, however, are not sufficiently resistant to mechanical loads and need an additional protective layer. They may also be used as a membrane underneath asphalt overlays. A special mention needs to be made in terms of protective coatings. Piers above the tidal zone and the superstructures are best coated with Anti-Carbonation Acrylic, Elastic-Elastomeric coating. The system comprises water based (solvent free), conforming to code and specification, acrylic polymer modified with selected mineral fillers, applied over the prepared surface to form an anticarbonation elastic elastomeric protective membrane.

The coating should have anti carbonation and water vapor diffusion property and should be resistant to action of UV radiation. It should be waterproof and capable of bridging crazings and cracks. The coating protects exposed faces of the superstructure and the substructure of concrete bridges from the aggressive action of industrially polluted and marine/saline environment.

The specifications of the coating can be taken from IRC:SP:80 or from MORTH - Specifications for Road & Bridge Works.

Another specific coating system is one for protection of Bridge Decks. This system is also often referred to as Bridge Deck Waterproofing. These coatings can be based on either Epoxy, Polyurethane or Polyurea based Systems. These materials should have adequate crack bridging ability to withstand thermal stresses.

The coating systems shall have requisite certifications along with national and international reference with proven for performance.

6.1.3.3 Replacement of substantial depth of concrete section

If the deterioration has reached a level where a shallow surface repair may not be sufficient, a replacement of the affected concrete section should be considered. The choice of the repair material depends on volume to be replaced, the depth of the repair, the loading effects to be expected and the site conditions. In all cases, an appropriate pre-treatment of surface is required.

The various measures for damage repair may, in addition, require surface protection measures to provide for durability of repair. The following materials for replacement of a substantial depth loss of concrete surface should be considered:

- Polymer modified, cement-bond systems; and
- Cement mortar or concrete (normally concrete similar to original mix but with reduced water cement ratio or higher grade of concrete is best).
- Shotcrete (gunite): Shotcrete is suitable for the repair of surface damages, concrete replacement as well as for the strengthening of structural elements.

Pre-treatment of the surface is of prime importance while using shotcrete. Sand blasting has proved to be an efficient surface treatment procedure. However, environmental protection regulations should be verified before use. The surface should be sufficiently pre-moistened. No bonding agent is necessary because at the interface surface, mortar enrichment occurs as a result of aggregate rebound.

Shotcreting in multiple layers requires that the preceding layer achieves a sufficient degree of hardness. Minimum reinforcement may be required for thicknesses larger than 50 mm. The reinforcement should be fixed in position in such a manner that it remains stiff/firm and keeps its position during shotcreting operations and to ensure adequate cover in the finished works.

Curing may be accomplished by an evaporation protection, e.g. plastic sheet. If a freeze-thaw/ salt resistant concrete is required, air entrainment admixtures may have to be added to the concrete mix. Also, surface protection measures may become necessary. There are two basis shotcrete processes:

i) A dry mix process where most of the mixing water is added at the nozzle and the cement-sand mixture is carried by compressed air through the delivery hose to a special nozzle, and

ii) A wet mix process where all of the ingredients, including water, are mixed before entering the delivery hose.

Shotcrete suitable for normal construction requirements can be produced by either process. However, differences in cost of equipment, maintenance and operational features may make one or the other more feasible for a particular application.

Properly applied shotcrete is a structurally adequate and durable material and is capable of excellent bond with concrete, masonry, steel and some other materials. However, these favorable properties are contingent on proper planning, supervision, skill and continuous attention by the application crew.

In general, the in-place physical properties of sound shotcrete are comparable to those of conventional mortar or concrete having the same composition.

Special variants of shotcretes result from the addition of polymers, fibers or of synthetic resins. Steel, glass (boron-silicate-glass) and plastics are used for the fibers. The ratio of the fiber to cement will be larger in the initial mixture than in the rebound material. The last layer must not contain steel fibers. In the case of steel fibers, corrosion protections must be considered, unless the fibers are protected from corrosion.

6.1.4 *Removal and replacement of concrete*

This is considered necessary when the concrete is found to be delaminated by sounding with hammer or chloride ion content is critical or micro cracks are found on a chipped surface or concrete is carbonated upto to reinforcement. The removal of damaged concrete is usually done with electrically powered or compressed air ensuring that the reinforcement is not damaged. Flat chisel is normally used to minimize micro crack formation which can cause repair failures. For a complete removal of a structural element larger equipment such as sawing, cracking, thermal lancing and blasting may also be adopted. Special care needs to be taken while removing concrete in pre-stressed concrete structures. Hydro demolition is the latest method where water is sprayed on to the concrete in thin jets at a very high pressure and enables removing of concrete in a more efficient and precise manner without damaging reinforcement and in a better working environment.

The replacement of concrete in larger continuous areas should proceed in the same manner as during the construction of the concrete structure. However, certain features resulting from the combination of old and new concrete should be considered.

Placing concrete in the area to be repaired should be accomplished in such a manner as not to impede concrete flow and to avoid the entrapment of air, thus avoiding voids in the concrete. Therefore, the formwork must be sufficiently rigid and tightly fitted to the existing concrete in a manner to minimize leakage of cement paste. The surface of the existing concrete will require adequate preparation, careful cleaning and pre-moistening.

The replacement concrete should have final properties that match the existing concrete as closely as possible (strength, modulus of elasticity, creep co-efficient, etc.) To avoid temperature and shrinkage cracks, especially in the transition area, the type of cement, cement content and the water/cement ratio should be carefully evaluated.

The use of plasticizers is recommended. Re-compaction /re-vibration may be required to improve the contact to the old concrete; however, care should be exercised to avoid a re-tempering of concrete after an initial set. Trial repairs on non-critical structures are essential before the main work is undertaken.

For larger concrete volumes, minimizing the temperature difference between old and new concrete may require special procedure (cooling of new concrete and/or heating of old concrete). Type and duration of curing should be evaluated on a case by case basis.

6.1.5 *Repair of cracks and other defects*

6.1.5.1 General

Before deciding the most appropriate methods/material for repairing/sealing cracks a study should be made on the cause of the cracks ascertain and whether they are active or dormant. Crack activity (propagation or breathing) may be determined by periodic observations with gauges, optical crack gauges, gauges or tell tales.

6.1.5.2 Reasons for occurance of cracks

Cracks may occur due to creep, shrinkage, fatigue, fire, non-functioning of bearing, temperature gradient, chlorination and carbonation. It may also be caused by plastic settlement, plastic shrinkage, early thermal contraction, long-term drying shrinkage, crazing, corrosion of reinforcement and alkali-aggregate reaction. Repair of cracks becomes necessary when:

- Corrosion protection/cover is at risk allowing corrosive agents to reach the reinforcing steel.
- The strength, stability and safety of the structure is at risk.
- A restoration of the tensile strength of the member is structurally necessary, e.g. reduction of the stress range in pre-stressing elements through cracked coupling joints.
- Cracks in longitudinal direction of the reinforcement present a risk to bond strength/spalling; and
- A crack free appearance is required from aesthetic as well as lack of safety.

It is always desirable to attempt repairs to cracks at as early a stage as possible.

Basically, a crack resulting from one time load application and which has ceased to propagate can be repaired by pressure injection with epoxy/ resins such that stability is restored and any adverse influence on the life expectancy of the structure is eliminated or minimized.

6.1.5.3 Materials

The material used for crack repair must be such as to penetrate easily into the crack and also provide a durable adhesion to the crack surfaces. The adhesion strength the interface of the material and the crack surfaces should be such as not to allow infiltration of water and to resist all physical and chemical attacks. The following materials are used for crack injection:

- Epoxy/Resin (EP)
- Polyurethane Resin (PUR)
- Cement Slurry (CG)
- Micro fine Cement Suspension (CS)

Selection of Injection Systems is based on the crack being treated. A reckoner on the type of injection material to be used is given in below **Table 6.1**.

Table 6.1 Injection Material Systems Available and the condition of their Application

Sl. No.	Crack Void Property / Materials	Cement Based	Epoxy/ Resins	PU	Acrylic Gels	Coatings
1	Surface Crack	+	++	*	*	++
2	Deep Crack	*	++	++	*	*
3	Load Transfer	++	++	+	*	*
4	Water Stoppage	*	*	++	++	*
5	Moving	*	*	++	++	*
6	Non Moving	++	++	*	*	*
7	Water Bearing / Damp	+	+	++	+	*
8	Dry	++	++	++	*	*

Legend: +: Suitable; ++: Most Suitable; *: Not Suitable

The **Fig. 6.1** shows crack widths and material to be used to inject them.

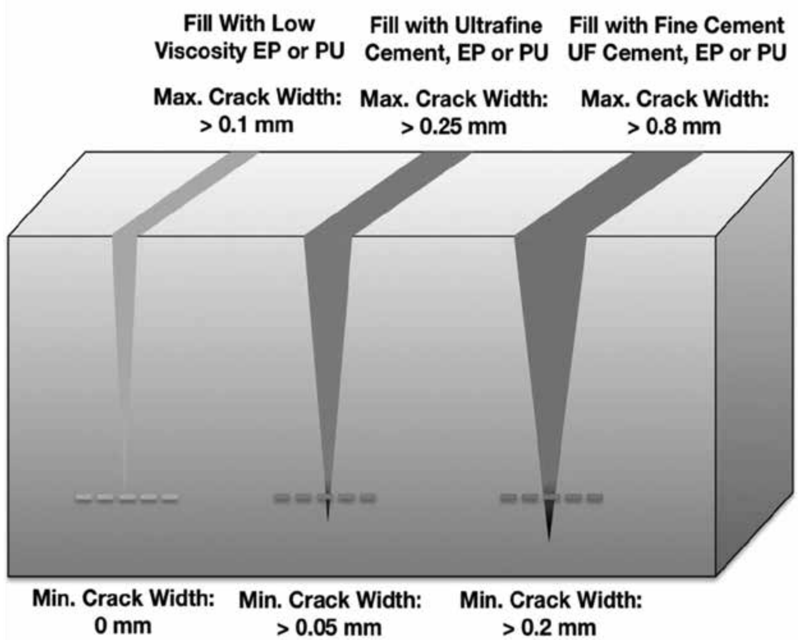

Fig. 6.1: Figure showing crack widths and materials that can be used to inject them

Although cement paste is relatively inexpensive, its use is limited to crack widths around 0.3 mm

Improvement of workability will be obtained if the cement suspension is introduced into the mix with high speed mixers.

6.1.5.4 Repair techniques

The repair techniques generally applicable for the various types of damages and cracking of concrete are as follows

i) Active Cracks: Caulking, jacketing, stitching, stressing, injection etc.

ii) Dormant Cracks: caulking, coatings, dry pack, grouting, jacketing, concrete replacement, pneumatically applied mortar, thin re-surfacing.

iii) Crazing: coatings, pneumatically applied mortar.

iv) Alkali Aggregate Reaction: concrete replacement, total replacement

v) Holes & Honeycombing: total replacement, combing pneumatically applied mortar, pre-packed concrete, replacement, injection.

vi) Cavitation: pneumatically applied mortar, concrete replacement, jacketing (may be inside/surface) injection grouting.

vii) Excessive Permeability: coatings, jacketing, pneumatically applied mortar, prepacked concrete, total replacement, grouting.

6.1.5.5 Injection process

As a rule, the following steps are necessary during injection:

– Drilling of the injection-holes and blowing out of the holes and cracks,

– Installation of packers,

- Tamping of surface in the area of the cracks to be injected,
- Mixing of the injection material and
- Re-injection and testing.

(i) Packer

Packers are auxiliary means by which the injection material is filled into the crack. Depending on the method of installation, they may be classified as an adhesive packer and drilling packer.

Adhesive packers are pasted into the crack. The hose to the injection device is connected to the nozzle of the adhesive packer. In the case of drilling packers, holes are drilled in the plane of the crack or may be inclined to the crack plane. The packer consists of a threaded metal tube which is encased in a rubber like sleeve and equipped with a nut. After insertion into the drill hole, the rubber sleeve is compressed by screwing down the nut. In this manner, the drill hole is sealed. A nipple, equipped with a ball valve to which the injection hose is attached, is screwed into the packer opening. The valve opens itself when subjected to the injection pressure. **Fig. 6.2** shows various types of packers in practice as below.

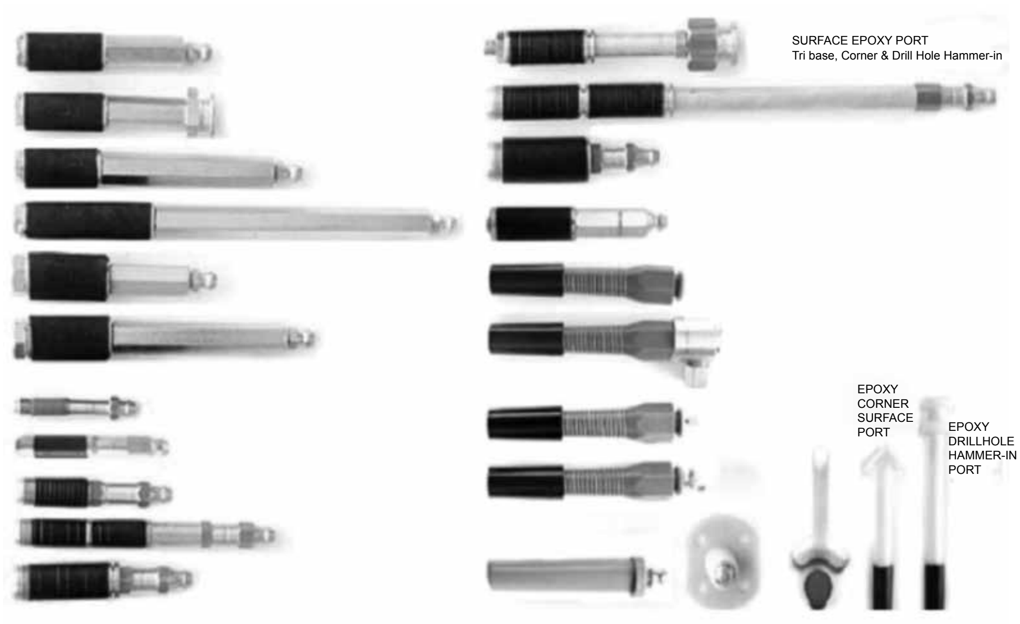

Fig. 6.2 Types of Injection Packers

(ii) Injection Equipments

Injection equipments are differentiated as one-component or two-component equipments. In the case of one-component equipment, the resin is mixed first and subsequently injected into the crack. Typical representative one-component equipments are a hand grease gun, treadle press, air-pressure tank, high-pressure tank and a hose pump. With these equipments, rather high pressures can be applied. However, the influence of the applied pressure on the packer, the tamping and the crack itself should be considered. The pot lift of the material is an important parameter in the application of one-component equipments. Therefore, the length of crack that can be injected is subject to the volume of material being used and its pot life.

In the case of two-component equipments, resin and hardener are separately transported to the mixing head by means of fully automatic dispensing equipment. Therefore, pot life is only of secondary importance. Errors in mixing two-component resins can have significant effect on the hardening of the resin. Therefore, the use of pre-packaged batches prepared by the manufacturer is recommended. Generally, in the case of two-component automatic dosing devices errors will not be discovered in sufficient time to apply corrective measures. **Fig. 6.3** below figure shows various types of Injection equipments in practice.

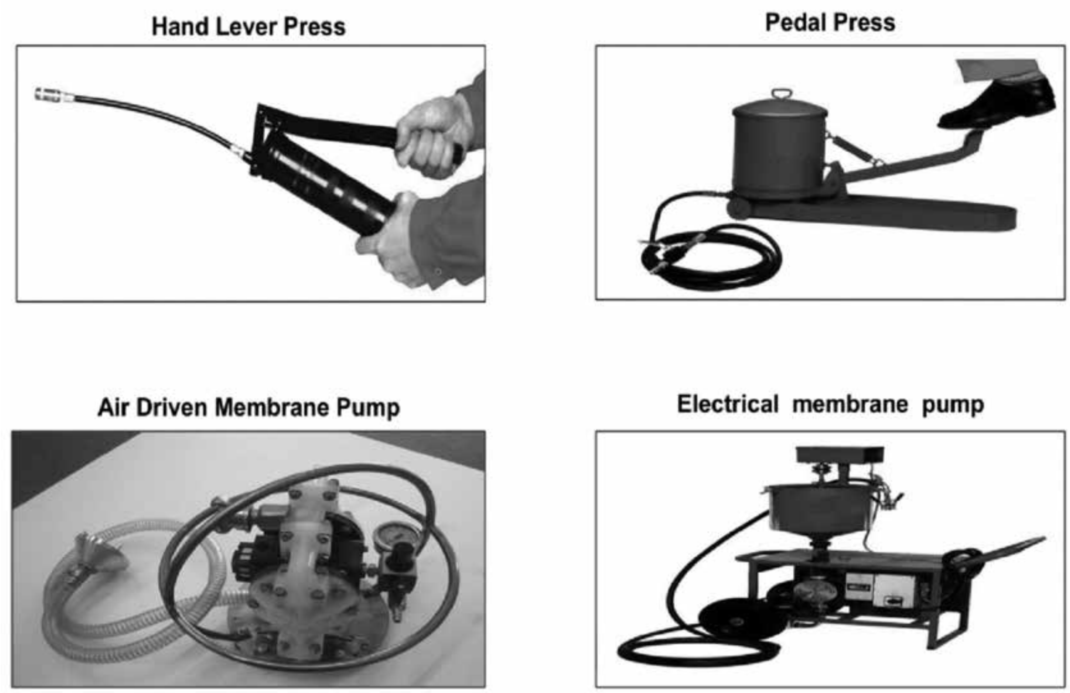

Fig. 6.3 Injection equipments

(iii) Injection

Injection pressure depends on various parameters such as tensile strength of concrete, depth and width of crack, viscosity of injection material. The penetration speed of the injection resin does not increase proportionately with increasing pressure. The viscosity of the resin strongly influences the rate of injection, especially for small crack widths and in the case of the crack depth.

The injection of a crack is completed when either the resin or hardener has been consumed from either of the containers or a back pressure has built in such a manner that no further material can be injected into the crack. The injection has deemed to have been completed when the packer refuses to take further material into it.

The injection of all types of materials shall be carried out by and experienced and trained applicator.

(iv) Testing: The usual testing methods are core drilling and ultrasonic testing.

(a) **Coring**

The success of an injection operation can be ascertained by a relatively simple procedure of removing cores taken through the crack plane. Because of the unavoidable damage to the structural element such evaluations should only be used in exceptional cases.

(b) **Ultrasonic Testing**

With ultrasonic measuring, the efficiently of the grouting operation can only be evaluated when the propagation of sound is oriented approximately normal to the crack surface.

6.1.5.6 Recommendations for Practical Implementations

Adequate materials, equipment and experience of the operating personnel are an essential requirement for the successful injection of cracks. Appropriate certification is necessary to determine the qualifications of operating personnel.

A system of quality control of the resin should be implemented to guarantee a consistent quality for each new application. These are the determination of the infrared composition (IR-Spectrum), the pot life, the viscosity, the density, the glass transition temperature as well as the development of the tensile strength during hardening and the hardened material. To avoid such expensive routine controls, some resin manufacturers have contracted with independent institutions to provide testing on a statistical sampling basis as a monitoring control. After successful testing, the resin batches are provided with a stamp of the testing institute as well as with information regarding durability. Stringent regulation for the use of resins in crack repair, especially where they must resists tensile stresses, is required to ensure the behavior of structural elements and to avoid additional damage that might be caused by the utilization of inadequate materials and procedures. Studies have shown that injection with epoxy/resins can be successfully accomplished.

A reduction of adhesive strength will occur when the concrete surface of a crack is excessively moist. There is also a risk of reduction in the quality of epoxy resins when they are used for the repair of structural elements at extreme temperatures. Current experience indicates that epoxy resins can be successfully utilized when the temperature of the structural element is not less than 8°C. Because of the lack of experience with other resin (e.g. PUR) the 8°C limit should be maintained.

For relatively hot structural elements, as compared to normal temperatures, a considerable reduction in the workability time of the resins may result. In these cases, the temperature of the structural element, in relation to its influence on the pot life should considerand pretesting may be appropriate.

In many cases, only one side of the structural element to be injected is economically accessible. Experience has shown that one-side injection of a through crack in a large structural element or deep cracks are not always uniformly filled.

An effective epoxy resin injection can still be accomplished when there is a cyclic width variation, as result of traffic loading, during injection and hardening, provided this variation does not exceed 0.05mm. Appropriate traffic limitations upto maximum of the first three days depending on temperature should be implemented if larger cyclic crack width variations are anticipated. In the case of large crack width, variations resulting from temperature injection should be co-ordinate such that hardening commences at maximum crack width opening. Thus, the filled crack will be subjected to a compressive stress, atleast for temperature variations. Experience indicates that there is no difference in behavior between an alkaline or carbonated concrete.

That deformability of the resin, as a rule, is not sufficient to close active cracks tightly and durably in case these movements cannot be stopped. Under these circumstances the feasibility of expanding the crack and forming a permanent expansion joint should be explored.

6.1.5.7 Other Crack Repair Methods

The other methods for repair of cracks are as below:

(i) Stitching

Stitching across the cracks by in-situ reinforced concrete is done either along the cracks or a series of bands around the members. Reinforcement is placed across the cracks in suitable grooves which are suitably packed with non shrink concrete. Alternatively, if geometry permits, bars grouted in holes could be used for stitching.

(ii) Jacketing

This involves fastening of external material over the concrete to provide the required performance characteristics and restoring the structural value. The jacketing materials are secured to concrete by means of bolts and shear connector, bolts adhesives or by bond with existing concrete. Plate bonding using steel or composite fibre laminates are commonly used. Cementitious jacketing with micro concrete or concrete mixed with fibres is also used.

6.1.6 *Pre-stressed concrete members*

For PSC members, the simple methods adopted are sealing and coating to fill out the cracks, grouting of cracks, repairing corrosion locations, and vacuum grouting using specially formulated resins to fill voids in the ducts. Some of the latest techniques include use of special chemical material formulations to satisfy the requirements like high tensile strength, special thermal properties etc. Some of the methods are common to those for RCC and the relevant details given earlier may be referred.

6.1.7 *Corrosion protection of steel reinforcement*

6.1.7.1 General

The embedment of reinforcing steel in concrete normally provides adequate corrosion protection due to the alkalinity of the concrete surrounding the steel. Because of its alkalinity, concrete forms a passivating film on the surface of the encapsulated steel as a result of the presence of a saturated lime solution in the cement gel. Moist concrete typically has a pH value in excess of 12 which maintains the passivating film. This film is however de-passivated when the pH level is reduced below a value of approximately 10 to 11, or when a sufficiently high chloride concentration of about 0.4% chloride by weight of cement is present.

In case the alkaline passivation film is destroyed or carbonation has reached the reinforcement or if moisture and oxygen are present, corrosion of the reinforcement will occur. In the absence of moisture (i.e., dry concrete) the corrosion process is inhibited, even if the concrete is carbonated, the alternate wetting and drying cycles increase corrosion.

When reinforcement corrodes to a certain extent, the surrounding concrete cover tends to expand and leads to crack and spall or split. The cracks are caused by internally bursting stresses developing in the concrete as a result of a net increase in volume by the formation of corrosion products. Cracking/Spalling of the concrete cover will then permit the entry of water and other corrosion accelerating agents and the rate of corrosion accelerates. Non-expansive black rust associated with severe pitting and rapid corrosion can occur in low oxygen wet high chloride conditions in salted bridge decks, substructure and marine structures.

6.1.7.2 Protection of reinforcing steel

(i) Preparation prior to protection

The decision on the necessity or removal of chloride contaminated concrete cover shall depend upon the amount of chloride content, availability of moisture and degree of carbonation. This decision requires a case by case evaluation. If the corrosion protection of the reinforcing steel requires removal, the reinforcement will have to be exposed completely.

The removal of rust from the uncovered reinforcing steel is generally accomplished by mechanical methods and hydro jetting or needle hammer and wire brushing. The removal of rust from the remote side of the bar is a difficult operation. A careful check and a repeated treatment of the individual bars is essential.

(ii) Restoration of the protection

This is similar to surface protection.

All exposed reinforcement cleaned to bare metal should be protected immediately after preparation by using suitable corrosion inhibiting active or barrier coatings. These are proprietary materials to be used to provide corrosion resistance to the cleaned reinforcement, prior to application of the polymer modified mortar/concrete system. The materials that can be used to provide corrosion protection include:

Active coatings for reinforcement: Arc coatings, which contain Portland cement or electrochemically active pigments, which may function as inhibitors or which may provide localized cathodic protection. Portland cement is considered to be an active pigment due to its high alkalinity. Typical product that can be used is a one-component polymer modified mineral based corrosion protection coat, which can be used for most repair applications.

Barrier coatings: Arc coatings, which isolate the reinforcement from pore water in the surrounding cementitious matrix. Typical product that can be used is a two component Epoxy Resin based Zinc Rich Primer and Coating Material for use in repairs subject to aggressive environmental and chemical attacks.

(iii) Preventive corrosion protection

In the case where concrete cover is thin it may be desirable to seal the surface with an epoxy/ resin and solvent containing acrylic resins to prevent carbonation or corrosion.

(iv) Cathodic protection

The Cathodic Protection (CP) technique has been adopted to protect steel pipe lines and tanks from corrosion. In recent years it has been applied experimentally for the protection of reinforcing steel in concrete.

Corrosion of steel in concrete proceeds by the formation of an electro-chemical cell with the concrete acting as coupling electrolyte, an anodic reaction occurs at some points on the steel surface and cathodic reactions consume the dissolve electrons on the remaining portion of the steel surface.

The presence of chloride ions will produce a local de-passivation. By means of an externally applied small Direct Current (DC) the electric potential between the steel and concrete is shifted to a non-critical level. Thus, the electrons impressed in the steel forces the steel to act as a cathode in the electro-chemical cell. The potential shift produced by the DC is critical to the cathodic protection. Because of the high resistivity of the electrolyte concrete a uniform distribution of the protection current throughout the structure is necessary. But difficulties in achieving this and high costs have prevented cathodic protection being widely used in bridge decks and superstructures. However, research continues.

It is accepted that there is still research required to be done before cathodic protection can be safely applied to pre-stressing steel.

6.1.8 *Pre-stressing steel protection*

6.1.8.1 General

This section deals only with the repairs to pre-stressed reinforcement. It should, however be noted that the repair of the concrete and the normal reinforcement of a pre-stressed concrete structure will also require attention. In most cases, the pre-stressing force is still active and the stresses transferred to the concrete must be carefully considered, especially when repairing concrete in the anchorage zones.

6.1.8.2 Repair of the corrosion protection system for bonded tendons

In the case of bonded tendons, the pre-stressing steel should be protected by the concrete cover and the cement grout in the ducts.

(i) Vacuum Procedure

Where the ducts are not completely filled with cement grout, subsequent grouting is necessary. This can be accomplished by vacuum grouting techniques. The advantage of this procedure is that the re-grouting of a duct requires only one drilled hole for each void. Such holes may be pre-existing in the form of the drilled holes used for tendon inspection or for obtaining samples for chloride content evaluation. Only a diameter adjustment may be required. A comparison between the assessed volume of the void and the amount of grout consumed will provide a control measure as to the success of the operation. Where discrepancies occur further additional borings will be required. A carful drilling procedure is required to avoid damaging of the pre-stressing steel. Special devices and techniques have been developed for this purpose, such as: slow drilling speed: special drill head, small impact force, drilling without flushing, sucking away of drill dust and automatic switch off when the drill bit reaches the duct. The repair must be accomplished as quickly as possible after opening of the duct to avoid corrosion.

After grouting, a pressure has to be applied to expel residual air from the voids. There is a risk that for large air cushions, setting water will be displaced towards defects and produce paths which will impair the corrosion protection. Therefore, mortar having slow setting characteristics should be used. Special cements are available for this purpose.

In special cases, surplus water in the duct can be evacuated. However, this process requires special equipment and knowledge.

(ii) Grouting of the Ducts with Special Resins

Where ducts filled with water cannot be drained through drilling or the vacuum process and drying is not possible, the water can be displaced by use of special resins with a long pot life and high specific weight.

6.1.8.3 Repair of corrosion protection systems for external tendons

The pre-stressing steel of external tendons is protected by a tight envelope of plastic pipe or painted steel pipe and the internal void of the pipe is filled with cement grout or suitable greases. If an inspection indicates deterioration of the protection system, measures must be taken for its re-establishment. Such measures may be re-painting of steel ducts and protective caps over the anchorages, replacing of plastic pipes, taping of local pipe damage, filling of voids inside the pipes etc.

Any materials used in the repair procedure must be compatible with the existing protection materials and with the pre-stressing steel. Some paintings, coating materials and special grouting mortars might contain substances that can produce stress corrosion and should, therefore not be used.

6.1.9 *Honeycombed concrete*

There are two methods of repairing i.e. either the porous parts of the concrete are replaced by sound, watertight concrete or the porous zones are injected with a sealing material. First, all the porous zones of the structure must be carefully removed. Then they are replaced by fully compacted concrete or mortar with a water/cement ratio not exceeding 0.4. This procedure cannot be used where there is a continuing inflow of water. In this case, sealing can be accomplished by injection or use of underwater epoxy/resins

6.1.10 *Repairs to damages/disintegrated/crushed components*

Depending on the extent and severity of damaged concrete may be rectified with the partial or full replacement either by higher grade of concrete, polymer modified or fiber reinforced concrete.

6.2 Strengthening of Concrete Structures

6.2.1 *General*

Strengthening of structural members can be achieved by:

- Replacing poor quality or defective material.
- Replacement of defective/distressed members.
- Providing additional load bearing material,
- Providing additional members and
- Re-distribution of the loading actions through imposed deformation of the structural system.

6.2.2 *Materials*

The new load bearing material will usually be:

- High quality/ grade concrete
- Modified concrete
- Fiber reinforced concrete
- Reinforcing steel bars
- Thin steel plates and straps,
- Post-tensioning tendons or

– Various combinations of these materials.
– Steel Plate bonding
– Fiber Reinforced Plastic bonding

The main problem associated in strengthening is to achieve compatibility and continuity in the structural behaviour between the original material/structure and the new material/repaired structure also:

i. The strengthening part of the structure participates only under live load and
ii. The strengthening part of the structure participates under live and dead load (or a part of it)

It may be noted that these strengthening measures improve the strength but not necessarily the durability of the original structure.

6.2.3 *Design aspects*

The strengthening of structures should be designed and constructed in accordance with related codes. If special codes for strengthening exist, they will of course be of assistance to the designer and contractors. However, this is seldom the case, and many problems in connection with strengthening are not dealt with in the codes. Typical problems of this kind are the transfer of shear forces between the old concrete and the new concrete applied for strengthening reinforcement and the post-tensioning of the existing structure which in some respects is different from the post-tensioning of a new structure etc.

6.2.4 *Interaction between new and old concrete*

Satisfactory interaction/ bonding between existing concrete and new concrete is required in case of strengthening and repairs. As a rule, the aim is to get the structural parts, composed of different concretes, to act as a single unit similar to that of homogeneously cast structural component. To achieve this, the joint between old concrete and the new concrete should have requisite bond strength and must be capable of transferring tensile and shear stresses without relative movements of such a magnitude that the static behaviour is significantly affected. Furthermore, the joint must be durable i.e., the composite structural component must not change its mode of action with time in existing/ changed environment.

While replacing large concrete volumes, the possibility of additional stresses as a result of heat of hydration has to be taken into account. Temperature differences can be limited by special measures, either, pre-heating of the old structural element and/or cooling of the fresh concrete.

Differences in creep and shrinkage properties between old and new structural elements will require careful evaluation. Cracks may develop as a result of potential increases in constraint forces. Therefore, it becomes necessary to correctly detail and anchor the reinforcement. To implement the strengthening measure, it will be necessary to employ suitable mortars or concretes with low creep and shrinkage properties as well as minimal development of hydration heat. At the same time, an effort should be made to match, as closely as possible, the strength and modulus of elasticity of the new material with that of old material. These requirements will be influenced to a large extent by the composition and treatment of the new material.

Vibrations due to traffic during hardening of the new concrete will have either negative influences or positive influences on its strength and its bond characteristics to the old concrete. This will depend on whether the vibrations are just sufficient to harden the concrete or too severe to disturb the components of hardened concrete and its bond. Traffic shall be permitted only after complete setting/hardening of fresh material. In case it is observed that vibrations do not have any negative influence then the traffic could be permitted in a controlled manner while the repairs are in progress. However, if the vibrations due to traffic have negative influence then stoppage of traffic or speed limits may have to be considered during the hardening phase as necessary. The critical phase may be 3 to 14 hours after making the concrete. The form work should be so detailed that no relative movement occurs between old and new concrete. The reinforcement has to be sufficiently fastened so as to keep relative displacement small.

6.2.5 *Strengthening of the reinforcement*

The strengthening of reinforcement can be achieved by:

- Replacing of corroded reinforcement;
- Providing additional reinforcement;
  * placed in the old cross section;
  * placed in an additional concrete layer;
- Pre-stressing and
- Bonded steel/ fiber plates.

6.2.5.1 Strengthening with reinforcing bars

In the simplest case, a strengthening of the concrete tension zone is possible by the addition of reinforcing steel. Reinforcement should be added after reducing locked up stresses to the extent possible and after the concrete cover has been removed or after recesses have been cut in the cover to accommodate the added reinforcement. Afterwards the concrete cover must be re-established. An effective anchoring of the ends of the reinforcing steel is required. This can either be done by providing sufficient anchorage length for the steel in the concrete, or by steel plates and bolts with anchoring discs.

The severely damaged reinforcing bars must be replaced. After unloading of the structure, the damaged sections of the corroded bar can be removed and the new reinforcing bar joined to the ends of the old ones by lapped splices, welding, or coupling devices. Transverse reinforcement is needed to assure a ductile behavior of the splice.

Staggering of lapped splices is recommended unless the c/c distance between the bars is greater than twelve times the diameter of the bars.

Lapped splices in a structural element can produce problems (congestion, interference with the proper compaction of concrete etc.) These difficulties may be overcome by the use of welded splices or couplers.

6.2.5.2 Strengthening by means of bonded steel/fiber plates

The strengthening of concrete structures can be achieved by means of bonded plates. A behavior of these systems needs to address following aspects:

Short-Term Behavior

The load carrying capacity of this type of strengthening depends on the strength of the reinforcement, the concrete and the adhesive. At yielding of the reinforcement the adhesive will fail. Utilization of high strength reinforcement is limited by the dimensions, concrete strength, etc.

Concrete strength has a large influence on the efficiency of the strengthening.

Theoretically, higher bond stress is to be expected from an increase in the elasticity of the reinforcing element and a decrease in the elasticity of the adhesive.

Geometrical influences are primarily the dimension of the reinforcing elements. Their length, thickness and widths are decisive. The length of these elements has an influence on the bond stress intensity, which decrease with length.

The bond stress will at the same time be influenced by the thickness. Therefore, glued on reinforcing elements behave differently from deformed bars which can be designed using the same permissible bond stress for all diameter. There is no proportionality between the width of the glued element and ultimate load, as an increase of width results in a reduction of bond strength. For a certain ratio of width to thickness the glued surface becomes a minimum.

With increasing width, there is a risk of defects in the adhesive. Therefore, the width of the reinforcing element should be limited to a maximum of 200 mm.

The thickness of the adhesive coat, within a range of 0.5 to 5 mm, has no significant influence on ultimate load. With increasing thickness of adhesive the slip between the reinforcing element and the concrete becomes greater. The concrete dimensions, according to previous test, do not appear to have any decisive effect. The surface condition of the steel is an important parameter. Suitable conditions can be achieved by sand blasting or by suitable method. Oil and grease should be removed by means of an organic solvent. As cleaned surfaces may corrode rapidly, a primer coating should be applied immediately. The primer serves as a corrosion protection and as an adhesive base for resin adhesive. It is a specially formulated solvent containing epoxy resin. Priming with zinc dust or hot-dip galvanizing is not suitable for glued on reinforcing elements.

The strengthening of concrete structures can be achieved by means of bonded plates.

Working principle of this system was plate-bonding technique, which was used. When the steel plate is bonded to concrete with epoxy adhesive the structural behaves will be composite structure. Three systems are commonly used:

i) Steel Plate Bonding
ii) GFRP/CFRP Bonding
iii) Fiber Reinforced Cementitious Matrix

Steel Plate Bonding

It is an efficient method of increasing flexural capacity in beams, when applied on tension side; the success depends upon the epoxy adhesive, which transfers load from concrete to steel. It is constructability that was reason for major success, as well as plate bonding does not increase the section. Disadvantages were cutting of steel plates to suit the geometry, its weight and problem of corrosion.

Fiber Reinforced Plastic Bonding

This is in fact an extension of the steel plate bonding technique with tremendous advantage of light weight, ease of cutting and mouldability to suit any element and high chemicals resistance.

Two systems commonly used are based on Glass Fiber Reinforced Plastic (GFRP), and Carbon Fiber Reinforced Plastic (CFRP)

FRP composites contain fibers of high tensile strength in polymer matrix of epoxy or vinyl ester. The rapid acceptance of this material is due to serviceability and ease of application without disturbing the structure. Fiber wrapping of columns provides passive confinement with increase of ductility and strength. Shear strengths are also increased. Wrapping gives excellent protection against explosions. FRP plates can be bonded with epoxies to increase flexural strengths. The limitation is the use of epoxy, which can change its characteristics during thermal variation and fire.

Fiber Reinforced Cementitious Matrix

This material system is similar to the CFRP Strengthening system, with only the matrix being a cementitious material, instead of an epoxy. This system addresses many of the shortcomings of the CFRP system. Some of these advantages include:

i) Fire resistant – Up to Class A like for the concrete and mortar.

ii) The system undergoes ductile failure (like steel reinforced concrete) and not brittle failure as in CFRP.

iii) The system is Moisture/humidity resistant, no minimum substrate moisture content requirements.

iv) The system is High temperature resistant unlike CFRP.

v) The performance of the composite is similar to CFRP System.

vi) Environmental friendly technology – Helps reduce the concrete and cement consumption.

vii) Design guidelines for the systems are available in ACI 549, similar to ACI440 for the FRP.

viii) The system is an easy technology, fast application – Once the concrete is repaired and restored to the original geometry, the FRCM system can be applied only after one day.

For the pre-treatment of the concrete surface, the procedures discussed earlier apply. Blasting with fine grained blasting materials has proved to be effective (minimum pull-off strength 1.5N/mm^2). Coarse grained blasting materials will achieve a deeper roughening of the concrete surface, which results in an increased consumption of adhesive, but not necessarily in an improvement of bond strength.

(i) Long-Term Behavior

The question of long term behavior is of particular importance for these materials, the properties of which are highly time-dependent, of considerable importance are: creep, ageing and having fatigue strength.

(ii) Creep

The creep of resin adhesives is considerably greater than for concrete. In accordance with the current state-of-the-art, it can be assumed that the creep deformation abates relatively quickly. The adhesives may have very different creep ratios. In thin adhesive layers upto 3 mm the influence of creep is restricted by the cohesion of the adhesive.

(iii) Ageing

Ageing is a change of properties resulting from mechanical, physical and chemical influence's; e.g., air humidity, radiation, heat, weathering and water over period of time. Ageing varies widely between various adhesives. For strengthening of metal elements, ageing reduces strength such that the long-term strength is only approximately 50% of the short-term strength. For strengthening of concrete, a more favorable relationship exists, as the adhesive coating will be loaded considerably less.

Epoxy resin/adhesive have a certain porosity, which will allow the penetration of water and other solutions. Exposure to water over long periods of time can cause resin/ adhesives to lose strength. Water sensitivity varies widely between various adhesives.

(iv) Fatigue Strength

The fatigue strength is approximately 50% of the short-term strength. This indicates that concrete with glued on reinforcement shows a more favorable behavior than those for glued metal structures, for which the dynamic strength for 10 millions load cycles is only 10% of the static strength. After a dynamic load has been applied, the static ultimate strength of the concrete structure with glued on reinforcement increases. This can be explained through a reduction of the bond stress speaks as a result of dynamic load.

(v) Behavior at Failure

The slip between the reinforcing element and concrete under tensile load has an approximately linear elastic behavior upto about half of the element and the adhesive layer. A further increase in the load leads to a progressive increase of the relative displacement. The elastic deformations results from the deformation of the adhesive layer. This slip starts at the loaded end of the reinforcing element and moves, with increasing load, to the centre of the element. In the plastic range, a slip deformation in the concrete sub-surface also occurs. The slip in the concrete develops some millimeters below the adhesive coat. A failure occurs suddenly, by abrupt elongation of the slip interface upto the end of the reinforcing element.

In correctly designed structures with bonded plate reinforcement, a ductile failure with yielding reinforcement can be attained. Bolting on plates to prevent pealing failures is now normally accepted in some countries. It should however, be kept in mind that strengthening with steel plates with epoxy is a very workmanship sensitive method and so operations have to be under expert guidance only.

6.2.6 *Strengthening with external pre-stressing*

6.2.6.1 General

In many cases, strengthening by means of external pre-stressing is a highly effective method. Both reinforced concrete and pre-stressed concrete structures can be strengthened by this method. The influence of the external pre-stressing on serviceability and ultimate limit states can be varied within wide limits by selecting different methods of introducing the tensioning force and using different profiles of the tendon.

6.2.6.2 Choice of system for external pre-stressing

For external pre-stressing, only post-tensioning systems have been developed. For normal applications in pre-stressed concrete, post-tensioning systems should comply with the requirements of the IRC codes for Acceptance and Applications of Post-Tensioning Systems. Both bonded and un-bonded tens can be used. If short pre-stressing elements are required, a post-tensioning system with minimal slipping in the anchorage (anchor set) should be chosen. Short pre-stressing elements can be sensitive to deviations due to construction tolerances (eccentricity, inclination and tolerance of the anchorage elements, the pre-stressing jacks, etc.)

The use of lower strength high ductility threaded bars for partial pre-stressed could be considered as a more robust, simple and durable approach to avoid the very high local anchorage loads and durability problems with pre-stressing tendons.

At deviation points (saddles) excessively small radii of curvature in the tendon should be avoided.

6.2.6.3 Special design consideration

The strengthening by means of external pre-stressing can normally be designed as an ordinary pre-stressed member. While calculating pre-stress losses, however, it should be noted that the effect of creep and shrinkage may generally be less than in normal design, due to the age of the old concrete. The stress in an unbounded tendon in the ultimate state will be only slightly larger than that after pre-stress losses.

6.2.6.4 Protection against corrosion and fire

The post-tensioning tendons should be protected against corrosion and fire to the same extent as in a newly built structure. The requirements for concrete cover are the same as for ordinary pre-stressed concrete structures.

6.2.6.5 Anchorages and deviators

Since the external pre-stressing tendons are not embedded in the structure in the conventional manner special attention must be given as to how the force is introduced. The space requirements of the anchorage and the pre-stressing device should be taken into consideration. When strengthening an existing structure, it is not generally possible to provide spalling or bursting reinforcement behind the anchorages in the same manner as for a pre-stressed concrete structure. Spalling can be prevented by means of transverse pre-stressing. This pre-stressing has the further function of creating contact pressure between new and original concrete, such that the necessary shear stresses can be transferred through the joint. To ensure full interaction between the tendons and the rest of the structure, the same method can be used along the entire beam. But the required shear stress is often so small that it can be dealt with by means of non-tensioned reinforcement. Another method could be to locate the anchorages in compressive zone and design the anchor plates for a suitably reduced bearing stress. There are several methods available for the attachment of supplementary pre-stressing as mentioned below:

i) Anchorage at girder ends

ii) Additional supports, either in concrete or steel, fixed to the web of the box girder. This method provides a good distribution for the force in the external tendons, but creates high stresses locally where the pre-stressing force is introduced. Because of the very short transverse dowels the fixation of the tendon supports or brackets can be a problem.

iii) Anchorages at existing diaphragms require extensive coring such that the tendon can pass through the diaphragm and be anchored at the backside. If the diaphragm does not have sufficient capacity to transmit the pre-stressing force, it may be necessary to provide a structural steel frame to transfer the longitudinal pre-stressing force.

iv) Deviators or deviation saddles

Where a draped profile is used, deviation saddles or deflectors have to be provided to achieve the profile. These devices can be either concrete or steel. They are attached to the existing webs or flanges by short pre-stressing bolts or other type of anchors. These short bolts or dowels are very sensitive to anchorage seating losses. A large radius of tendon curvature should be used.

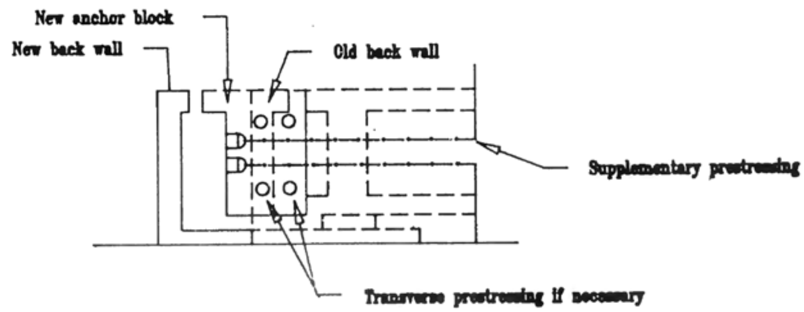

Fig. 6.1 Anchorage of supplementary prestressing elements at the end of the girder

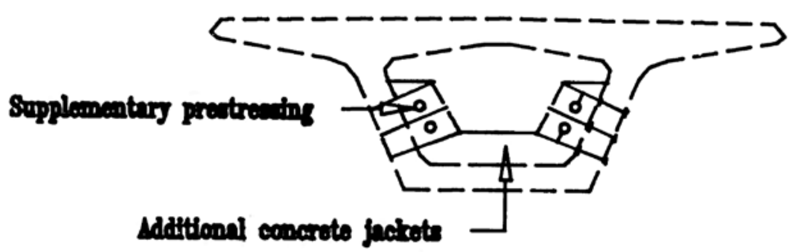

Fig. 6.2 Additional Supports

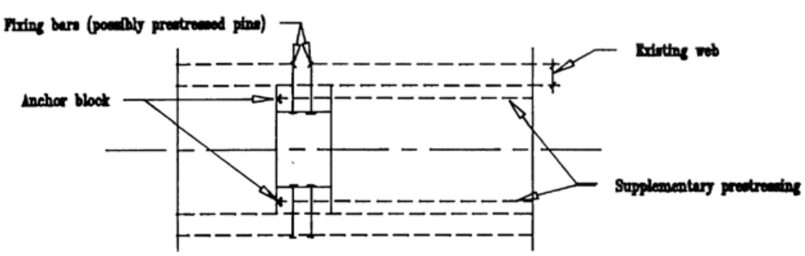

Fig. 6.3 Anchorage of supplementary prestressing elements with additional supports

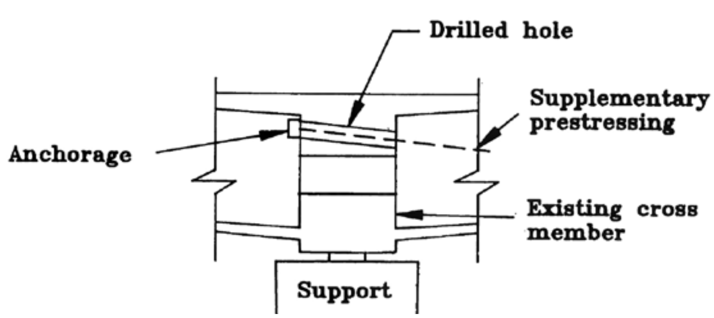

Fig. 6.4 Anchorage of supplementary prestressing at existing diaphragms

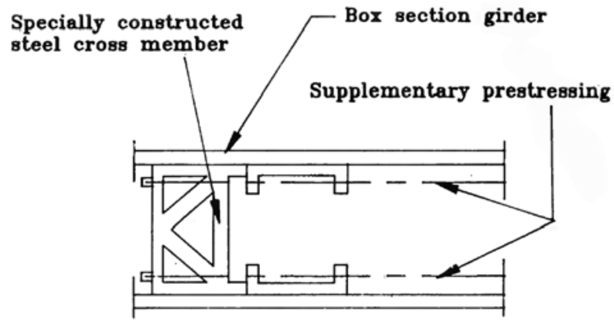

Fig. 6.5 Anchorages with auxiliary steel frames

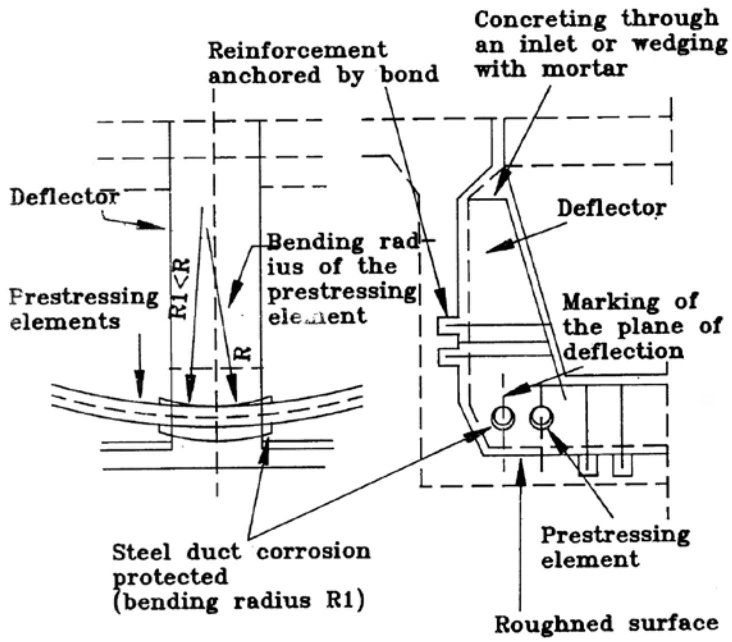

Fig. 6.6 Deflector for supplementary prestressing elements

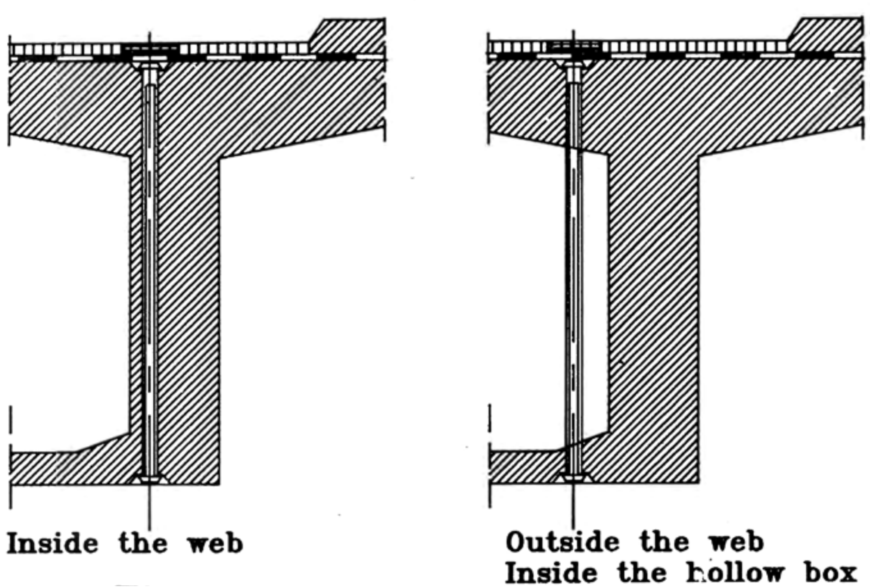

Fig. 6.7 Supplementary prestressing using straight tendons

6.2.7 *Strengthening with prefabricated reinforced concrete or precast concrete elements*

Strengthening is also possible by adding precast elements. This method will require a distressing (unloading) of the original section. The composite cross section of precast elements and the original concrete is then re-stressed (loaded). This provides an improved transmission of pre-stress force throughout the composite section. With time a re-distribution of permanent load will occur as a result of creep and shrinkage.

This strengthening method requires bond between the two structural elements at their interface. As a rule, a resin modified cement bond mortar layer is utilized.

For treatment of the contact surface of the structure, the same operations as described earlier are necessary.

In the fabrication of the precast elements, consideration should be given to the texture of the contact surface so as to provide increased bonding and shear characteristics at the interface. A sufficiently rough surface of the precast element can be obtained when the formwork of the contact surface is treated with a retarder. By early removal of the formwork and cleaning with water, a washed concrete surface can be achieved. A reduction of the largest grain size in the concrete on this surface is beneficial. Sufficient curing prevents micro-cracks between mortar and aggregates resulting from shrinkage. To roughen the surface, sand blasting is also suitable.

If no special measures are taken during fabrication, the contact surfaces of the precast elements have to be treated like that of the original structure.

6.2.8 *Strengthening by imposed deformation*

By means of imposed deformation, overstressed sections of a structure can be partially relieved. With this the load carrying capacity of the whole structure is improved. A self-equilibrated stress condition can be induced in the structure by relative displacement (raising and/or lowering) of the supports or by the introduction of new intermediate supports.

It is important to note that relieving some sections of the structure will increase action effects (bending moment, shear, torsion etc) in other sections. A strengthening of these sections may be required. Another important factor is time: relative settlement of the supports, shrinkage and creep of the old structure and the new supporting elements will influence the distribution of the actions- effects in the structure.

6.2.9 *Strengthening by other methods*

Concrete slabs or beams or columns (piers) can be strengthened by providing reinforced concrete jackets or overlays. Normally, thickness of new concrete layer should be less than about 1/3rd of the thickness of existing concrete. Proper attention to bonding and detailing of shear connections needs to be given.

Replacement of structural system or additions of new system to the existing structure are also sometimes adopted to rehabilitate or strengthen a structure. In such cases, the existing internal stresses in the members must be carefully analyzed.

Structural alterations could also be considered to reduce excessive vibrations.

CHAPTER 7
REPAIRS TO EXPANSION JOINTS AND BEARINGS

7.1 Introduction

Operational life of expansion joints, bearings, footpaths and railings is usually shorter than that of the bridge. The expansion joints, bearings, railings, parapets etc. need special attention for repairs and replacements or renewal. The capacity and efficiency of bearings may be a limiting factor in situations where it is required to increase the load carrying capacity of a bridge.

7.2 Expansion Joints

The expansion joints are not expected to last throughout the life of the bridge. It is, therefore, necessary that joints be replaced on a regular cycle. Based on the experience gained in India as well as in other countries, it may be reasonable to assume life of expansion joints around 20 years. Stipulations in IRC:SP:69 for replacement of joints shall be adhered to. The seals of the joints usually serve for 10 to 12 years with normal maintenance and may need replacement over a period of time. Some of the factors which would influence replacement are as under:

- Damage to seals such as tearing, loosening, cracking or displacement.
- Damage to other components
- Clogging of joints.
- Loosening of anchor plates.
- Damage to concrete near the joints leading to their malfunctioning.
- Leakage of water through the joints.
- Theft of the joints.

Early bridge stock was provided with joints such as angle and plate, copper or asphaltic or finger type joints. Many of these joints may need replacement for better functioning of the /bridge deck if not function properly or deteriorated. Replacement is warranted with elastomeric or modular joints.

It is vital to keep the joints watertight in order to prevent the ill-effects of moisture including corrosion on the beam ends, bearing shelves and substructure. Leaks must not be tolerated. It is not uncommon for joints to be watertight under the roadway but allow water to leak through at the kerb. Any replacement of joint shall be watertight and it shall be complete and full width of deck which includes carriageway, kerb, footpath and central verge.

Where water tight joints cannot be provided, adequate means of draining the water passing through the joints shall be provided. As far as possible, the water should be kept out of contact with the concrete and the bearings. This is sometimes difficult to realise. If these measures fail, regular maintenance of the bearings and pedestals will at least prevent water from damaging the concrete. Joints may be filled with a sealant or filler. The filler material must be designed to ensure water tightness. Debris may prevent joint movement if the filler fails and may damage the joint sides or joint material, it may spall the sides of jointed slabs or cause over-stress in other bridge elements. Debris also tends to retain moisture and hence contribute to the deterioration of adjacent bridge component.

Damage to finger type joints can manifest in the form of bent, cracked, corroded, broken, closure of gaps and jamming. Reasons can be due to traffic or movement of bituminous wearing course, poor alignment and loose anchorages. The fingers may also cave in or project up due to unacceptable deformation of the deck or differential settlement of foundation. Cracking and spalling of the pavement or deck in the area adjacent to the joint may cause subsequent failure of the joint by loosening the joint side support material. Joints can also get closed in balanced cantilever type bridges due to excessive deflection of cantilever or excessive hogging of main span.

Movement of abutment must also be considered when inspecting joints. Such movement may either increase or decrease the joint opening or may even close the joint opening completely, preventing free expansion of the bridge.

All damaged joints should be replaced. The sealant filler shall be replaced periodically. Cracked concrete in the zone of anchoring the joints shall be replaced. Periodic cleaning and removal of debris is a must.

Table 5.2 may be referred to for defects and possible replacement.

7.3 Bearings:

Bearings are vital elements of the bridge and need regular inspection and maintenance. Bearings may be affected/distressed due to:

- Manufacture, defective materials
- Inadequate design
- Inadequate or improper installation
- Negligent maintenance

The type of defect may be one or more of the following:

- Corrosion
- Defective seal in neoprene/ pot bearings
- Broken guides
- Cracked or broken rollers, plates
- Cracks, splits or tears in neoprene material,
- Accumulation of dirt/ debris at the bearing point
- Failure of anchorage system
- Movement or creep of parts out of place
- Partial contact of bearing plates
- Excessive tilt or even shift

The bearings in distress may need replacement or resetting under the following circumstances:

- Excessive tilting or collapse of cut rollers
- Displacement of rollers
- Shearing off of lugs
- Shearing off of rag bolts
- Corrosion of rollers and plates

- Seizure of bearing plates
- Tears, cracks or splits in neoprene material
- Breaking of guide plates
- Improper installation of bearing of any type
- Displacement of PTFE in pot bearings
- Damage to seals of pot bearings
- Damage to confined elastomeric pad of pot bearings

Appropriate corrective action shall be taken after detailed investigation of the defects. Repair or replacement of bearings requires traffic restrictions or even temporary suspension of traffic.

Excessive tilts in bearings like segmental bearings should be corrected in time. This can be done by lifting the superstructure, shifting the bottom or top plates and lowering the superstructure. The cracked or excessively deformed elastomeric bearings shall be replaced. This requires lifting of the superstructure. Lifting is normally done with hydraulic/special jacks. Where superstructure is very heavy, cranes may have to be used. In all events of lifting, checking the design of superstructure for stresses induced due to lifting is obligatory. These specialized activities shall only be undertaken by specialist agencies. **Table 5.2** may be referred to for defects and possible replacement.

CHAPTER 8
HYDRAULIC ASPECTS

8.1 This chapter has been introduced mainly to focus attention of the bridge engineers on this major cause of frequent damages to bridges. Fairly good knowledge about hydrology and geotechnical situation is now available. Nevertheless solutions to hydraulic problems cannot be generalized. Unique solutions on case to case basis will have to be evolved. Some of the most common hydraulic deficiencies which could occur are as under:

(a) Actual discharge in excess of that assumed in the design

(b) Substantial increase in velocity of the river/ stream from that for which it was designed.

(c) Increase in scour depth from the one adopted in design of foundations resulting in settlement of one or more foundations of the bridge.

(d) Damage to the piers of the bridge due to the impact of floating debris brought by the stream during floods.

(e) Oblique flow of the stream under the bridges, the angle of obliquity being more than that assumed in the design.

(f) Meandering of the river and flow concentration in few spans may adversely affect stability of bridge.

(g) If actual waterway provided is inadequate, river may overflow the banks or may cut across the approach road behind the abutments.

(h) Location of the bridge is on curvilinear portion of the river. In high floods, abutment and penultimate pier on convex portion may get adversely affected.

The causes for occurrence of these and other similar deficiencies have to be explored, examined and thereafter suitable remedial/ rehabilitation measures may be adopted so as to ensure safety and serviceability of the bridge structure.

Some of the remedial measures adopted commonly for hydraulic problems are given below:

8.2 A bridge structure can substantially get damaged by floods. There are cases where rehabilitation of bridges becomes necessary on account of changes in the hydraulic parameters as manifest during flood conditions. The damages can be caused due to (i) abnormal floods, (ii) normal floods, if the design of the bridge does not cater adequately to the normal design floods and/ or (iii) as in a few cases, due to man-made changes in catchments of watercourse, e.g. the flood levels may exceed the original design levels substantially back water effect of a storage constructed downstream requiring raising of the bridge superstructure.

8.3 Floods can damage the bridge structure as well as the approaches and the protective measures. The bridge engineer is advised to refer to IRC:89 “Guidelines for Design and Construction of River Training and Control Works for Road Bridges”

8.4 If either due to inadequacy of original hydraulic design or due to the requirements of traffic as in a submersible bridge, the bridge level has to be raised, the same can be done by raising the superstructure with the help of jacks and extending the sub-structure in suitable stages by successfully resting them on the precast concrete pads which can then be embedded in the raised height of the piers. Where, however, the bridge deck is not to be raised but the bridge has to be protected from floods higher than the design floods, then designing the bridge as a submersible one and strengthening the same may have to be explored. At the same time suitable corrective measures may have to be adopted for decking and approaches, like provision of air-vents between the girders, protection of embankment, strengthening of piers by jacketing etc.

8.5 When the velocity and consequentially calculated scour in the stream is expected to increase and the substructure is found to be unsafe under such conditions, a solution of paving the bed with suitable aprons upstream and downstream can be considered to prevent the scour around the piers and the piers may also be strengthened by jacketing.

8.6 If flood damages to the bridge and approaches are of frequent nature then after careful investigations it may be necessary to extend the length of the bridge to provide adequate waterway. If the floods attack one side of the bridge, then additional spans could be provided on the affected side. Sometimes such situation can be handled by adequately designed spurs or groynes. In some cases, the returns beyond the abutments get damaged and may require replacement by returns on deeper foundations. Where a pier gets damaged beyond repairs, the span lengths could be changed either by locating a pier in between or if possible, by doubling the spans with suitable strengthening of the remaining substructure.

8.7 Bed protection can get damaged due to excessive turbulent floods or disturbance of stone protection. Surfaces of concrete or masonry can get eroded by high velocity of stream and sometimes cavitation can occur.

8.8 The bridge hydraulics is a highly specialized subject and so the treatment of the damages must be designed and carried out in consultation with a specialist. Use of hydraulic model studies for specific problems is also of considerable help in arriving at a proper solution.

CHAPTER 9
MONITORING

9.1 Necessity

Once rehabilitation and strengthening of the bridge is completed, it is essential that the structure is inspected periodically and its condition monitored regularly. This would help in assessing the performance of the work done. It would also bring out any further distresses that may occur. The various methods of monitoring the bridge structures are given in the succeeding paragraphs.

9.2 Methods of Monitoring

During the distressed stage of a bridge and after the distressed bridge has been repaired, rehabilitated or strengthened, it is necessary to carefully monitor its behavior for a certain period of time to ascertain its performance and the efficacy of the measures adopted. The monitoring would involve carrying out certain laboratory and field tests as well as condition surveys and measurements to detect strains, movements, changes in reaction and deformations.

9.2.1 *Inspections*

The first and the foremost requirements is to carry out routine inspections at more intervals than the normal structures, say immediately after distress is noticed and on completion of the remedial measures and during the use of the structure at the frequency of 6 months to one year depending upon the type of bridge and nature of distress thereafter for a period of 3 or 5 years. Use of mobile inspection units should be made wherever needed. The techniques of underwater inspection described earlier may also be adopted.

9.2.2 *Changes in behaviour*

The usual methods adopted for monitoring the behaviour of a structure are:

a) Observing deflections by periodically taking levels. The movements of bridges can be measured at the joints using slide gauges for maximum /minimum movements and reference pins for routine check.

b) Visual observations for cracks, deflections, overall integrity, profile, functioning of bearings and hinges, corrosion stains. Particular note must be made of the cracking pattern, their width and length and whether cracks can be due to plastic shrinkage, settlements, structural deficiency, reactive aggregates, and corrosion. Signs of delamination, spalling, hollow or dead sound when tapped with hammer, honeycombing` and expansion of concrete should also be observed and levels of inspections have to be specified depending on individual cases.

c) The change in the width of the cracks with the passage of time needs to be observed through tell-tales and gauges to know whether the cracks are active or passive.

d) Plumb bobs are used to measure deviation from verticality for vertical members: Special tilt meters or inclinometers also could be used; (NB: Datum readings at the time of construction are essential).

e) Opening of joints, particularly near the hinges, expansion joints etc. need to be observed.

f) Redistribution of support reactions may also be measured in some cases.

9.2.3 *Corrosion monitoring*

The use of permanent electrodes for accurate measurement of the corrosion potential of steel in concrete is also made. Use of current density or rebar probes and the use of corrosion rate monitoring probes can be made to meet the particular requirements. Careful selection of permanent monitoring equipment is required. The locations should be minimum and should be at the area of most active corrosion rate.

Relatively thin steel wires are embedded in the structure near the reinforcement with permanent electrical connections to the tell-tales so that electrical resistance can be measured. Corrosion of tell-tales would cause an increase in the electrical resistance. Certain devices can be permanently embedded in the concrete for facility of later measurements of the extent and the rate of corrosion in future years.

9.2.4 *Strain measurement*

The measurement of strains at critical sections or joints is another method of monitoring the behaviour of critical bridge elements. Electronic strain gauges are fixed at predetermined points.

9.2.5 *Use of lasers*

Application of lasers in structural monitoring is finding increasing use in developed countries. In its simplest form the system consists of threading a laser beam through series of apertures in the plates fixed along the length of the beam, say along the soffit a girder or soffits or series of adjoining girders, (**Fig 9.1**). Similarly, a laser beam can also be directed vertically along the bearings or a column. The beam after passing through a series of apertures in plates thus fixed along this path reaches the light sensitive receivers at the farthest end. The failure of the beam in reaching the receiver requires further investigation because it could be due to some structural deformations of the members supporting the plates or due to some other reasons. A system of series of such laser beams can be provided in a structure and arrangement made to sound an alarm in case of blockage of light of any laser beam.

Further refinement of the system could be made by attaching detectors to the structure along the path of the laser beam whereby any movement of the structure at the location of each detector would be continuously tracked by the latter relative to the laser beam and the actual overall behaviors of the structure at each detector location can be measured, recorded and analyses with the help of computers controlling timing and the operational sequence of the various detectors. Readings to the accuracy of even 0.1 mm are possible and continuous& constant 24 hours-a-day monitoring of a structure for its integrity and soundness is possible.

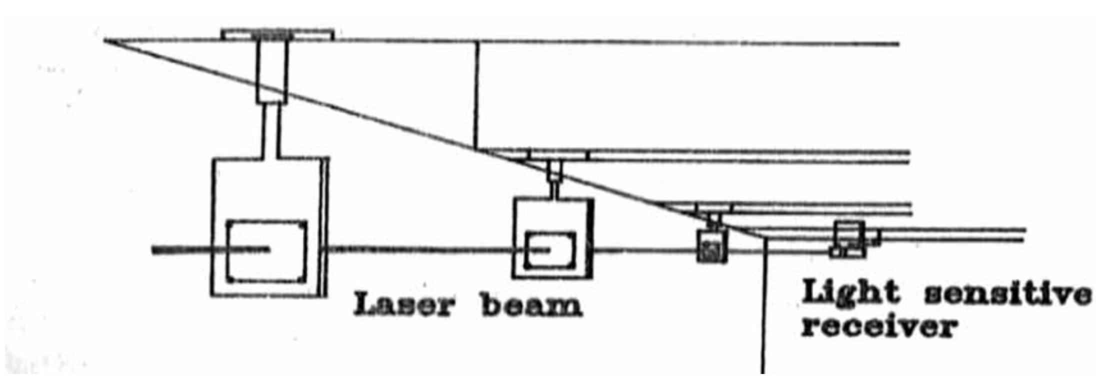

Fig. 9.1 Laser Monitoring of Deck Girders

Measurements for vibration characteristics of a structure can also be adopted in some cases to monitor the continued structural integrity and strength in the long run. However, a specialist's guidance should always be obtained.

9.3 Instrumentation

Instrumentation has to be provided for proper monitoring of long spans well as state of the art bridges to study their behaviour during their service life. The measurement may include concrete strain at critical points, temperature effects, defections, movement of hinges, etc.

9.4 Training

Monitoring of distressed bridges as well as rehabilitated bridges requires a great deal of skill and specialization. The engineers maintaining and inspecting such bridges therefore will need to be trained for such jobs.

9.5 Data Management

Monitoring also requires setting up a data bank as a reference frame. This should be initiated at the time of construction.

Testing, measurements and analysis of data are important to monitor the condition of the bridges. The sampling frequency of testing, therefore, has to be decided with the help of an expert. In the beginning, an extensive random sampling could be adopted to study the variability of results. Later, after studying the variability of the results limited target sampling may be decided. Both in selecting the sample size and the interpretation of the results, expert guidance is absolutely essential.

Monitoring shall also be augmented by continuous research and development. The details of the areas of research and development are enclosed as **Appendix-II**

CHAPTER 10
MISCELLANEOUS ASPECTS

10.1 There are certain important aspects of rehabilitation and strengthening of bridges involving measures other than technical which deserve due attention at various stages by concerned authorities. These are:

- Effective control of traffic and traffic restrictions, both in terms of speed as well as load, till the repairs are completed.
- It is advisable to put up warning and cautionary boards about the speed of vehicles and weight restriction on the vehicles if applicable to any bridge after repairs/strengthening on both ends of the bridge.
- Contingency plan for diversion of traffic and other necessary actions in case of any mishap.
- Providing proper information to public through publicity and press and countering any ill-founded rumours.
- Sometimes public-interest litigations also crop up which have to be handled properly.
- Safety precautions.
- Last but not the least, the morale of the engineers in charge of the work has to be maintained since they would be carrying out such repairs at a great risk to their own safety.

10.2 After completion of every job of rehabilitation of a bridge, an engineer must prepare a document to enable drawing lessons for the future so as to improve the bridge technology. Provision must be made in structures at the design stage itself for the possibility of future interventions like maintenance, repair, rehabilitation, strengthening and replacement of some components etc. A number of fruitful lessons can be learnt from the adverse experiences on the bridges and the consequent improvement of bridge technology.

10.3 Maintenance, Repair and Rehabilitation of State of the Art bridges like suspension, cable stayed, extradosed bridges etc. shall be carried out on the basis of specialized literature and expert opinion on the basis of advanced data collection method/techniques supplemented detailed quantitative analysis.

10.4 The new materials and techniques shall be used for Maintenance, Repair, Rehabilitation and strengthening with utmost care and due diligence with respect to the specifications, method of application and quality control promoter in addition to the respective IRC codes/guidelines.

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APPENDIX-I

TESTING METHODS

The various testing methods for assessment of the bridges are described below. Although brief procedure is also provided it for background information and for detailed procedure corresponding latest code as mentioned in **Table 4.1** shall be referred.

1. CARBONATION TEST

Use to determine the depth of the carbonated layer on the surface of hardened concrete by means of an indicator.

The Carbonation of concrete on the surface results in loss of alkaline protection of the cover over the steel against corrosion. Carbonation of concrete occurs when the carbon dioxide, in the atmosphere in the presence of moisture, reacts with hydrated cement compounds. The carbonation process is also called depassivation. When an indicator is sprayed on the surface which is freshly exposed and the indicator turns the colour, it indicates the presence of carbonates and hence carbonation.

The depth of carbonation is measured by spraying on the freshly broken surface of concrete with 0.10% solution of phenolphthalein.

The 1% phenolphthalein solution is made by dissolving 1gm of phenolphthalein 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. The concrete undergoes, a colour change (from pink to colourless) when PH value is below 10. If the concrete still retains its alkaline characteristic, the colour of the concrete will change to purple/ pink. If carbonation has taken place the pH will have changed to 7 (i.e. neutral condition) and there will be no colour change.

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 in 70 % ethyl alcohol. No specific equipment is required.

The measurement of carbonation depth using the phenolphthalein solution can be carried out by spraying the indicator on the split surface of the concrete cylinder or core dust sample. The solution became a pink colour in the un-carbonated concrete and can be differentiated from the carbonated concrete, giving a distinct boundary marking the carbonation front. A carbonation depth up to an accuracy of 5 mm can be identified with the naked eye.

It directly gives the effective cover available to the reinforcement before the start of corrosion. Factors affecting the test results are:

i. Humidity- Higher humidity results in greater carbonation process. Ideally, 50-70% lower humidity means not enough water available for carbonation.

ii. Temperature - The carbonation reaction rate increases with temperature.

iii. Low Rain Precipitation- When the concrete surface is wetted by rain, the diffusion of CO_2 is momentarily blocked as the pores get filled with water. The diffusion of CO_2 will continue once the concrete starts drying. Carbonation is then expected to be more severe in those regions with low precipitation and particularly on those parts of the structures that remain sheltered from the rain.

iv. High Concentration of CO_2 - The normal concentration of CO_2 in the atmosphere is approximately 0.03%. However, in certain areas as in bridges with heavy traffic, or close to industrial areas, the level of CO_2 contamination may reach and even exceed 0.1 % concentration.

The method is simple, fast and accurate and no equipment is necessary. The contour maps of the surface of the concrete member having varying carbonation depths can be prepared to get some insight to the durability aspects of the structure.

The tests results do not assist in determination of concrete durability directly. Minor surface damage shall occur and if not treated it may be the source of reduced cover to the reinforcement.

This technique is mostly used for assessment of the surface carbonation of reinforced concrete members to determine the possibility of corrosion and hence establish the durability of the section.

There may exist a partially carbonated zone, where the pH value is not easily detected by phenolphthalein indicator. The carbon dioxide could react at the depths greater than those indicated by the indicator. The test shall not be considered as a tool for estimating the structural properties of the reinforced concrete member.

2. SULPHATE CONTENT

To determine the sulphate content through the concrete samples.

The concrete attached by sulphate has a characteristic white appearance.

The quantity of sulphate is estimated by the precipitation of barium-sulphate and sulphate confined by identification of Calcium Sulpho Aluminate. Weigh into a 400 ml beaker 5 ± 0.005 g of the analytical or sieved sample. Disperse with 50 ml of water and add 10 ml of concentrated hydrochloric acid. Add 50 ml of hot water, cover the beaker and boil the solution gently for 5 min to 10 min. Filter through a medium ash less filter paper. Wash the residue thoroughly with hot dilute hydrochloric acid (1+49). Add three drops of the methyl red indicator and heat the filtrate to boiling. Just neutralize to yellow with the dilute ammonium hydroxide solution. Immediately add 1 ml of concentrated hydrochloric acid and then add drop wise to the boiling solution 10 ml of the barium chloride solution. If excess ammonium hydroxide was added, 1 ml of concentrated hydrochloric acid may not be sufficient to obtain the required acid solution and the barium sulphate precipitate will then be contaminated. In this case the test shall be repeated. Boil gently for 5 min, keep the solution at just below boiling for 30 min and allow to stand at room temperature for 12 hrs. to 24 hrs. Filter through a slow ashless filter paper and wash free from chlorides with hot water. Transfer the paper and contents to a weighed silica or platinum crucible and burn off the paper without flaming. Ignite the precipitate at 800 ºC to 900 ºC until constant mass is achieved.

The sulphate content G, expressed as SO_3, as a percentage of the cement to the nearest 0.1 % (m/m) from the expression

$$G=\frac{L}{M_d}*34.3*\frac{100}{C^1}$$

Where,

Md is the mass of the sample used (in g);

C^1 is the cement content of the sample used (in %);

L is the mass of ignited barium sulphate (in g).

Factors affecting the test results are:

i. Loss of material, particularly dust, during the crushing and grinding operations is should be minimize for better results.

ii. Exposure to Atmospheric carbon dioxide.

The samples from other NDT tests like core cutting, pull out can also be used.

3. CHLORIDE CONTENT

To determine the Chloride content in the concrete.

Salts of thiocyanates and thiosulphates are increasingly used as accelerating admixtures in place of chlorides. These compounds can interfere with the determination of chloride content.

The Chloride content in concrete is measured in laboratory by Mohr's method using potassium Chromate as indicator in a neutral medium or by using Volhard's volumetric titration method in acidic medium.

Weigh into a stoppered 500 ml conical flask 5 ± 0.005 g of the analytical or aggregate separated sample. Disperse with 50 ml of water and add 10 ml of nitric acid. Add 50 ml of hot water, boil for 4 min to 5 min and keep warm for 10 min to 15 min. Cool to room temperature and add a measured excess of the silver nitrate standard solution. Add 2 ml to 3 ml of 3.5.5 - trimethylhexanol stopper the flask and shake vigorously to coagulate the precipitate. Add 1 ml of iron I II indicator solution and titrate with the thiocyanate solution to the first permanent red colour.

Chloride content will be determined, the presence of acid soluble chlorides in concrete beyond the permissible limits is considered as a corrosion hazard in concrete structures.

The chloride iron content J as a percentage of the cement to the nearest 0.01 % (m/m) from the expression:

$$J=\langle V_5-\frac{mV_6}{0.1}\rangle\frac{0.3545}{M_c}*\frac{100}{C_1}$$

Where,

M_c is the mass of sample used (in g);

V_5 is the volume of 0.1 M silver nitrate solution added (in mL);

V_6 is the volume of thiocyanate solution used (in mL);

m is the molarity of the thiocyanate solution (in mol/L);

C^1 is the cement content of the sample used (in %).

Factors affecting the test results are:

i. Loss of material, particularly dust, during the crushing and grinding operations is should be minimize for better results.

ii. Exposure to Atmospheric carbon dioxide

The samples from other NDT tests like core cutting, pull out can also be used.

Rapid tests for in-situ chloride determination are being developed.

This test should be done under the expert guidance and proper sampling procedures need to be evolved due to high variability of laboratory test results.

4. SCHMIDT/REBOUND HAMMER TEST

It is used to estimate the compressive strength of concrete in an existing structure and as a quality control measure of a new construction.

It works on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges.

The equipment used for the test is called Schmidt Rebound Hammer which is shown in the **Fig. A1**. The main components of the equipment include the outer body, the plunger, the hammer mass, and the main spring. While testing a concrete surface, the plunger is held perpendicular to the concrete surface and the body is pushed towards the concrete. This movement extends the spring holding the mass to the body. When the maximum extension of the spring is reached, a latch located in the equipment releases, and the mass is pulled towards the surface by the spring. The mass hits the shoulder of the plunger rod and rebounds because the rod is pushed hard against the concrete. During rebound the slide indicator travels with the hammer mass and stops at the maximum distance the mass reaches after rebounding. It is essential to calibrate the equipment by testing the same on an anvil supplied with the equipment.

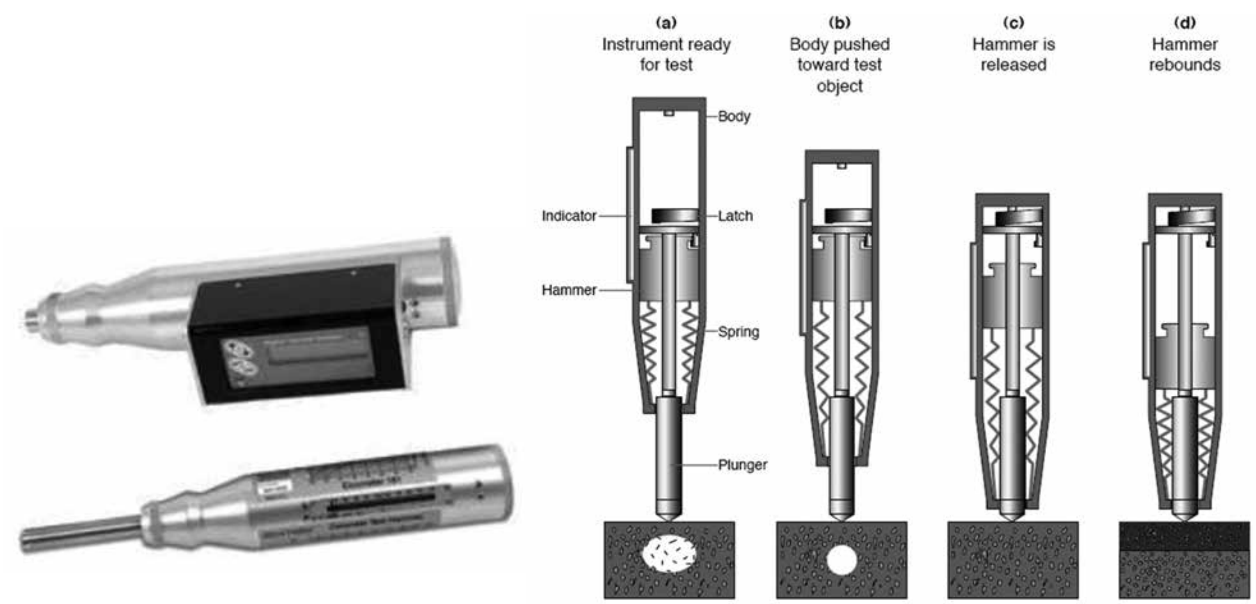


Fig. A1 Schmidt Rebound Hammer & Working Principle

Rebound hammer test data are recorded in terms of rebound numbers. Generally, up to 10 readings are taken on a square grid from a test location and the average of readings is calculated. It indicates the average surface hardness of concrete at a given location.

Each hammer is furnished with correlation curves developed by the manufacturer using standard cube specimens to convert the average rebound number into the compressive strength. However, the use of such curves is not recommended because the material and testing conditions may not be similar to those encountered in the field / laboratory.

Typically the correlation between the rebound number and the strength of the concrete is carried out as given below:

For an existing structure:

a) A minimum of 3-5 concrete cores preferably of size 150 mm Ø x 300 mm or 100 mm Ø x 200 mm are extracted from different locations of the structure
b) The cores are conditions as specified in IS 516. After capping, cores are placed in a compression-testing machine under an initial load of approximately 15% of the ultimate load to restrain the specimen.
c) About 15 hammer rebound readings are taken, 5 on each of 3 vertical lines 120° apart, against the side surface in the middle two thirds of each core. Testing the same spot twice shall be avoided, and average is calculated.
d) The cores are then tested in a UTM to determine the concrete compressive strength.
e) The core compressive strength is converted into the cube compressive strength by multiplying the core strength by 1.25, as suggested in IS: 516
f) The rebound numbers are plotted against the compressive strengths on a graph
g) A correlation is then developed between the average rebound numbers and the core compressive strength by fitting a curve or a line by the method of least squares.

This correlation may be used to estimate the compressive strength of concrete in the structure at different locations from average rebound hammer.

For quality control purposes of under-construction structure:

a) About 30 concrete cubes of 150 mm size are made using the same mix proportions to be employed for the construction.
b) The specimen are cured under standard moist-curing room conditions, keeping the curing period the same as the specified control age in the field.
c) The cubes are placed in a compression-testing machine under an initial load of approximately 15% of the ultimate load to restrain the specimen.
d) About 16 hammer rebound readings are taken, 4 on each of sides, avoiding testing the same spot twice, and average is calculated.
e) The rebound numbers are plotted against the compressive strengths on a graph.

f) A correlation is then developed between the average rebound numbers and the cube compressive strength by fitting a curve or a line by the method of least squares.

This correlation may be used to estimate the compressive strength of concrete in the structure at different locations, from average rebound hammer, at the specified age, to monitor the quality in the construction.

Factors affecting the test results are

Rebound number may be affected by (i) Smoothness of the test surface, (ii) Size, shape and rigidity of the specimen, (iii) Age of the specimen, (iv) Surface and internal moisture conditions of concrete, (v) Type of coarse aggregate, (vi) Type of Cement (vii) Carbonation of the concrete surface etc. Carbonated surfaces generally give higher rebound number readings, upto about 50% for a carbonation depth of 20 mm, (viii) the angle of striking the concrete and (ix) type of mold.

This allows survey to produce contour maps of the surface of the concrete member having varying rebound numbers.

The method is simple, fast and accurate and the instrument used is very handy.

The method gives only the surface hardness but does not give direct estimation of compressive strength. A correlation curve, specific to the concrete in the structure, is required to be developed to correlate the rebound number to compressive concrete strength.

Coefficients of variation for compressive strength for a wide variety of specimens average at 19 % and exceeded 30% for some groups of specimens, and the same can be narrowed down considerably by developing a proper correlation curve for the hammer, which allows for various variables discussed earlier. The depth of influence of the test extends up to about 20 mm from surface. The probable accuracy of estimation of concrete strength in a structure is ±25%, however for properly cast, cured concrete specimen, and tested under laboratory conditions by a properly calibrated hammer, an accuracy of ±15 and ±20% can be achieved.

Commercial rebound hammers with analog or digital display of the readings are available. Some of the recent equipments display the averagebound number.

5. COVER METER

Cover Meters are used to detect steel reinforcement bars and estimate concrete cover over rebar and diameter of the steel reinforcement bar in concrete structures.

The method works on electro-magnetic principles. An electromagnetic field is generated by the search head of the meter. When a reinforcing bar or other magnetic metal object lies within this field, the lines of force become distorted. The disturbance caused by the presence of the metal in turn produces a local change in field strength as detected by the search head and indicated by the meter.

Cover meters operating by monitoring two different effects of electromagnetism are available; eddy current effect or magnetic induction effect. Eddy current method works at higher frequencies (> 1 kHz) and is sensitive to any conducting material in the vicinity of the search head, while the magnetic induction method used low frequency current (below 90 Hz) and is less sensitive to non-magnetic metal. In both the methods, the proximity and orientation of the metal to the search head affect the meter reading.

Portable battery operated commercial cover meters (also called pachometers) are widely available. Cover meters capable of locating stainless steel bars are also available.

An application of typical cover meter is shown in the **Fig. A2**.

Fig. A2 Cover Meter in Use

It is possible to obtain the location of reinforcing steel bar, determine the depth of concrete cover and estimate the barsize using the cover meters.

Cover-meters can measure the cover with an accuracy of 5 mm up to a depth of about 75 mm.

Factors affecting the test results are: Measurements may be affected due to (i) Potential loss of accuracy, (ii) Type, orientation of steel, multiple bar, closely spaced bars and tie wire, (iii) Magnetic properties of aggregates & surface finish, (iv) Temperature and (v) Corrosion of reinforcement.

The technique is the least complicated and least expensive of all the NDT techniques. Under the most favourable site conditions, indicated cover can be measured to an accuracy or + 1 to 4 mm approaching that obtainable in the laboratory when the bar size is known.

The various limitations are:(i) It is a slow and labour intensive method. (ii) The level of accuracy may vary with congested reinforcement and severity of corrosion (iii) The equipment needs to be calibrated before use (iv) The equipment may not work if non-magnetic reinforcement such as galvanised bars or FRP bars are used in the structure.

6. GROUND PENETRATING RADAR

A high frequency pulsed radar can be used to detect any metallic or non-metallic objects, deterioration like delamination and the other types of defects, which can occur in bare or overlaid reinforced concrete decks. It can also measures thickness of components.

The Ground Penetrating Radar (GPR) is shown in the **Fig. A3**. It 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. Schematic view of Ground Penetrating Radar Technique is shown in **Fig. A4**.

A permanent record can be stored on magnetic tape and the unit is normally mounted on a vehicle and the data are collected as the vehicle moves slowly across the deck.

GPR can be used for locating the reinforcement, embedded cables and ducts in a bridge structure, and to determine the thickness of different layers of the structure. It is also used for profiling the bottom of lakes and rivers, and measuring scouring around bridge foundations etc.

Fig. A3 Ground Penetrating Radar (GPR) in use

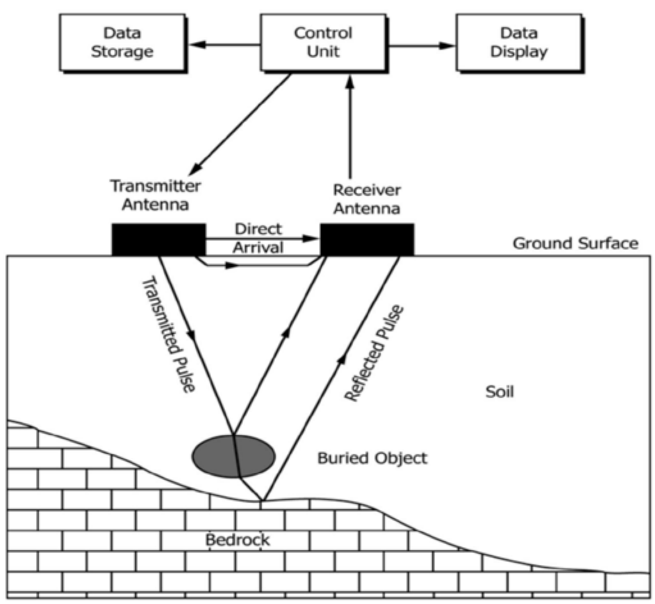


Fig. A4 Schematic view of Ground Penetrating Radar Technique

The test results may be affected due to metallic objects such as cars, telephone line and clustering of rebars below scanned surface could result in total reflection of incidence wave resulting in restriction in depth of analysis.

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.

GPR is still reliant on the experience of the operator. The radar waves can never penetrate through metals, so testing the interior of metal ducts, e.g. for un-grouted areas is not possible.

Ground radar surveys are used on bridge structures for assisting load assessments and as a follow-up to routine inspections. A GPR survey is able to locate internal defects for the accurate targeting of intrusive investigations. On masonry arch structures, locate defects and identify backing on the haunches of the arch.

A large number of commercial equipments available, hand held and tailor made to reach all locations and detect any structural discrepancies.

Radar may 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 cover reflections from delamination.

8. RADIOGRAPHY

Radiography techniques are applied on the pre-stressing cables to detect defects in the cable and to examine the quality of grouts within the ducts. Radiography is also used to locate the position of reinforcement bar in reinforced concrete, estimates can be made of bar diameter & depth below the surface, to determine presence of voids, cracks and foreign materials, and variations in the density of the concrete.

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 Principle of Radiography is shown in **Fig. A5**.

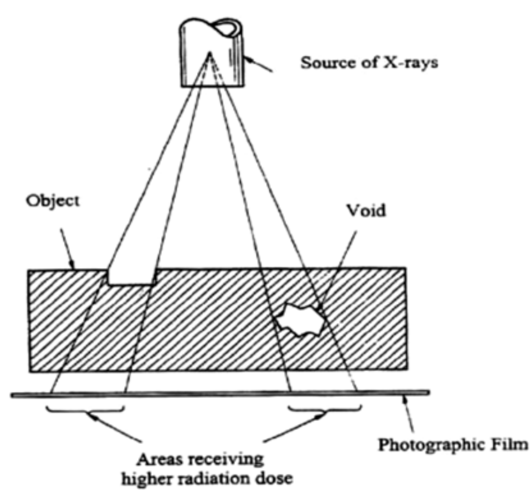


Fig. A5 Principle of Radiography

Most of the applications involve transmission of wave energy rather than reflection or refraction methods. The emerging radiation is detected by photographic emulsion or by a radiation detector. The former is called radiography and the latter radiometry. In short the procedure is as below:

X ray tube used to produce X-rays, which is the basic requirements, namely (i) source of electrons as a heated filament, (ii) means of directing and accelerating the electrons as a high voltage supply, and (iii) target which the electrons can bombard, normally in the form of heavy metal target.

The Tube (**Fig. A6**) 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 back-scatter techniques is based on intensity of X-rays can be used to detect the voids in grouts and testing strands or wires which are broken or are out of position.

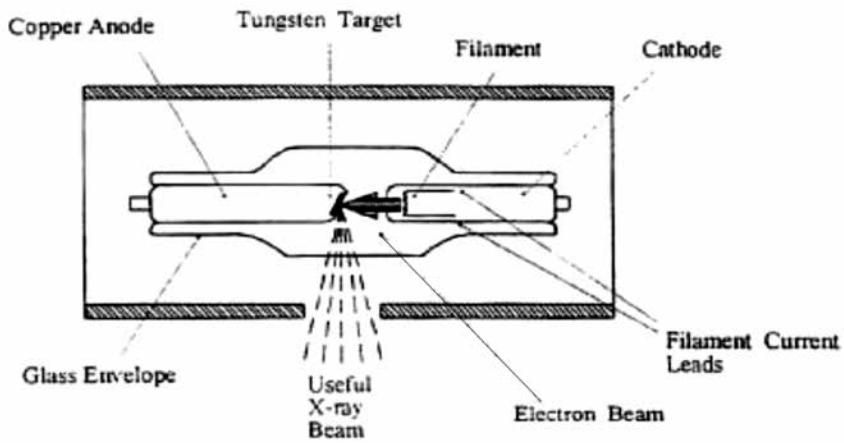


Fig. A6 A typical X-ray tube

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 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.

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

$$\mathbf{I = I_0 e(-\mu x)}$$

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

Factors affecting the test results are: The penetrating ability of portable X-ray units is limited and is used for members less than 300 mm thick.

Major disadvantage is the health risk associated with the radiation. High degree of skill and experience is required for exposure and interpretation. The equipment is relatively expensive. The process generally slow and require two-sided access to the component.

Radiography techniques can be used for (i) detecting both surface & internal discontinuities, (ii) detecting significant variations in component, (iii) inspecting hidden areas. Permanent test record is obtained and good portability especially for gamma-ray sources.

Small amounts of corrosion will not be detected and the technique is suitable only for isolated cables without any other obstructions in the path of the wave. The main limitation of radiography is that for thick sections, high-energy is required.

9. INFRA-RED THERMOGRAPHY

Infra-red thermography is used for the determination of delamination in bridge decks and columns exposed directly to sun.

The method works on the principle that discontinuity within the concrete, such as delamination, interrupts the heat transfer through concrete. Radiation is incident on the surface and that is reflected or absorbed. Absorbed radiation is for the reflected from the sub-surface defect leading to local heating. If this radiation falls on a concrete surface, then if the delaminations or the void was not there, then lot of radiation will simply get absorbed into the concrete. Whereas, if the void or a delamination present within the concrete surface the radiation gets reflected back to the concrete surface and a patch which has higher temperature than the neighbouring patches obtained. This difference in temperature observed on a concrete surface is used for detecting defects within the concrete surface.

In short the procedure is as below:

The equipment can be truck mounted permitting a lane width to be scanned by a single pass. The differences in surface temperature are measured by sensitive infrared detection system which consist of infrared signal, control unit and display screen. The images get recorded on the photographic plates or video tapes. A typical Infrared Thermography camera is shown in **Fig. A7**.

Fig. A7 Infrared Thermography Camera

Infrared thermography provides a two dimensional image i.e. Thermographs of the test surface showing the extent of subsurface anomalies like presence of voids, delaminations and occurrence of spalling.

Presence of Voids and occurrence of Spalling can be evaluated with the help of Thermographs.

Factors affecting the test results are: Thermographs may be affected by subsurface configuration, surface condition & environment.

The 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. The concrete need not be destroyed during testing.

The disadvantage of thermography is that while a positive result is valid, a negative result may not be always reliable because it relates to results under conditions prevailing at the time of tests. Nevertheless, the method itself has a considerable promise as a rapid screening tool for determining whether a more detailed investigation is required. The presence of voids or spalling can be found out but not the depth.

This technique is mostly used for assessment of the cracks, anomalies and spalling on the concrete surface.

Higher resolution cameras with continuous plots of bridge deck are possible to do a very fast analysis of cracks, anomalies and spalling in the deck.

The range of camera's available are varied and unless a very high resolution/pixelate camera is used, the results may not be accurate and subject to judgment.

10. ULTRA SONIC PULSE VELOCITY TEST

It is used for assessment of uniformity/integrity of concrete in structures. It can also be used to measure changes occurring with time in the properties of concrete, correlation of pulse velocity and strength as a measure of concrete quality.

The quality of concrete can be assessed by passing through concrete the ultra-sonic pulse and measuring its velocity. Measured values may be affected by surface texture, moisture content, temperature, specimen size, reinforcement and stress. Measured Time taken by wave to pass through the particular medium is empirically correlated to the uniformity or integrity of the concrete section. Calculation of ultrasonic pulse velocity taking into account time passing through testing base and length of the same as follows:

$$T = L / V$$

In short the procedure is as under:

i. This test is performed by using a transducer & receiver which are calibrated & applied with oil/grease/sticky material on concrete surface where integrity or homogeneity is to be measured. This placement can be done in three different types i.e. Place the transducer and Receiver on the Opposite face (Direct) or Adjacent face (Semi Direct) or on same face (Indirect).

ii. When the pulse generated is transmitted into the concrete from the transducer, the first waves to reach the receiving transducer are the longitudinal waves, which are converted into an electrical signal by transducer. Electronic timing circuits enable the transit time T of the pulse to be measured. The methods of propagating ultrasonic pulses are given in **Fig. A8** below:

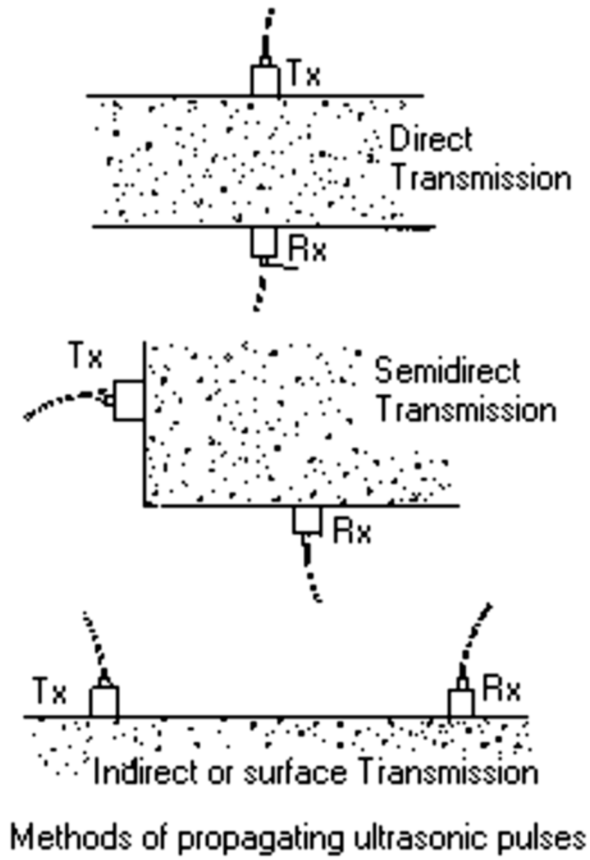


Fig. A8 Methods of Propagating Ultrasonic Pulses

The time taken by the signal in order to travel and reach the receiver is the result that will be showed on the UPV which is Velocity.

Evaluation of concrete is done according to the criteria given in IS 13311(Pt. 1).

Based on the test result Quality of Concrete can be categorized based on velocity is as follows: (i) V >4.5 km/sec-Excellent, (ii) V = 3.5 to 4.5km/sec –Good, (iii) V = 3.0 to 3.5 km/sec – Medium and (iv) V <3.0 km/sec-Doubtful

The velocity may be affected due to (i) Surface texture, (ii) Moisture content, (iii) Temperature of the concrete, (iv) Path length, (v) Reinforcing bars and (vi) Shape & size of the section, water cement ratio, hydration, curing, aggregate size of the concrete.

The advantage of this test is that it relates directly to the quality and homogeneity of concrete in the structure rather than to samples, which may not be always truly representative of the concrete in situ. UPV Testing has the possible solution to judge the porosity, density, strength of concrete section provided proper correlations are established. Generalised correlations are not suitable to define strength of the concrete sample.

This technique is mostly used for assessment of the integrity, homogeneity and uniformity of the concrete members where there is any ambiguity on the concrete integrity.

Co-relation can are more reliable if both test element and compressive strength test specimens are of similar size, consolidated to similar density, cured under similar conditions and has same age of concrete. The limitation of the method is that it cannot indicate the actual concrete strength.

11. ENDOSCOPIC TEST

It is useful in detecting voids in the grouts, concrete, corrosion in steel etc. The test can be performed for locating the tendons, evaluating the video images, and taking notes of the findings and making decisions regarding to the need for further inspection. This test if required, should preferably be done in association with radiography.

Endoscopy consists of usually flexible viewing tubes which can be inserted into holes drilled in the bridge components or into a cable duct of the pre-stressed concrete. A light is provided by optical fibres from external source. The endoscopes are available with attachments for a camera or a TV Monitor and are used for detailed examination of parts of the bridge structure which cannot otherwise be assessed.

A typical inspection team consisted of an inspector that operated the endoscope in the drilled hole, and an inspector controlling the video recording equipment **(Fig. A9)**.

Fig. A9 An Endoscope and View

Endoscopies consists of inserting a rigid or flexible viewing tube into holes drilled into concrete bridge components or cable ducts and view them with light provided by optical glass fibres from an external source.

Areas where tendons contain voids and other flaws can be found out and a written log documenting the inspection including the depth and length of voids, conditions of the strands if visible may be made.

The test results may be affected due to Humid Environment.

Endoscopy surveys provide a fast, cost effective means of identifying problematic/in-accessible areas where bear minimum destruction is allowed for inspection. The endoscope inspection is a reliable method for assessing the conditions of the post-tensioning tendons. The method is invasive but the damage induced to the tendons and deck is minimal.

It is not possible to assess the conditions of the strands where the voids with water present. Flexible endoscope lens is a very delicate and sensitive piece of equipment.

It is also useful for detail examination of other part of the bridge structure, which could not otherwise be assessed.

Endoscopes are available as attachments for a camera or a TV monitor.

This method can be used to validate and corroborate the findings of more economical NDT procedures.

12. ACOUSTIC EMISSION TEST

Test is used to check detect locate and characterise damages faults in structures and its components.

The stresses acting on the material produce the local deformation, which is breakdown of the material at specific places. This material breakdown produces acoustic emission i.e. an elastic wave that travels outward from the source, moving through the body until it arrives at a remote sensor. Acoustic emissions can be detected in frequency ranges under 1 kHz, and have been reported at frequencies up to 100 MHz, but most of the released energy is within the 1 kHz to 1 MHz range. Rapid stress-releasing events generate a spectrum of stress waves starting at 0 Hz, and typically falling off at several MHz. Acoustic Emission Testing process is given in **Fig. A10** below:

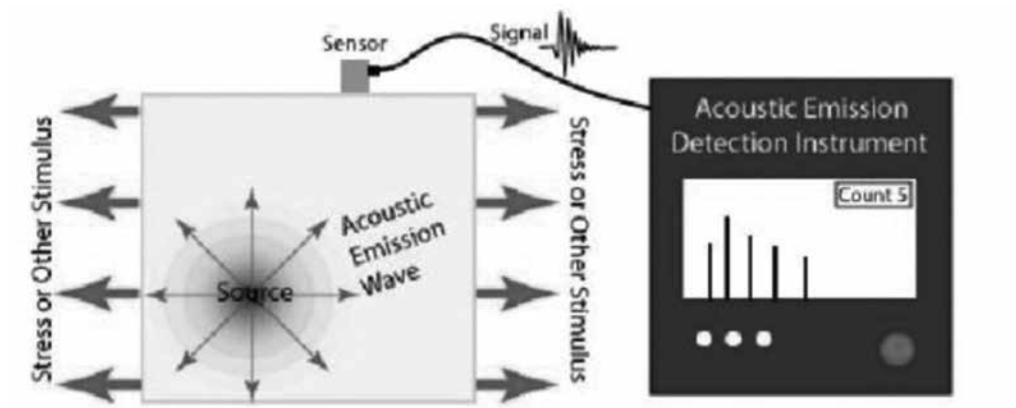


Fig. A10 Acoustic Emission Testing Process

The Acoustic Emission is based on the detection and conversion of high frequency elastic waves to electrical signals. This is accomplished by directly coupling piezoelectric transducers on the surface of the structure under test and loading the structure. Sensors are coupled to the structure by means of a fluid couplant and are secured with tape, adhesive bonds or magnetic hold downs. The output from each piezoelectric sensor (during structure loading) is amplified through a low-noise preamplifier, filtered to remove any extraneous noise and further processed by suitable electronic equipment.

AE sources are determined by calculating the difference in time taken for the wave to arrive at the different transducers.

The results may be affected due to (i) Geometric spreading, (ii) Scattering at structural boundaries and (iii) Absorption

Compared with ultrasonic, radiographic or the other NDT methods, the source of information is derived by creating some effect in or on the material by external application of energy or compounds. AE relies on energy that is initiated within the component or material under test. Acoustic Emission can be used at all stages of testing which include: pre-service (Proof) testing, in service (Re-qualification) testing, on-line monitoring of components and system, leak detection and location, and in-process weld monitoring.

Other advantages of the method are: Material anisotropy is good, Less Geometry sensitive, Less Intrusive, Global monitoring, Rea-time evaluation, Remote scanning and Performance price ratio

Acoustic Emission can be subjected to extraneous noise. For successful applications, signal discrimination and noise reduction are crucial. This test is best performed if the loading history of a structure is known.

The instrumentation of Acoustic Emission must provide some measure of the total quantity of detected emission for correlation with time and/or load. Acoustic Emission is stress unique and each loading is different. The structure under test will attenuate the acoustic stress wave.

13. CORE TEST

Core testing may be used for determination of the compressive strength of concrete.

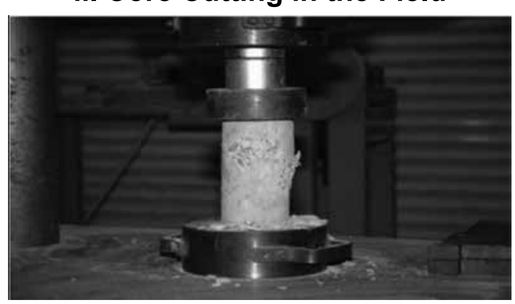
The basic principle for obtaining strength of core is the ratio of Maximum load applied to the Cross-Sectional Area.

The necessary equipments are given in **Fig. A11** which consist of: (i) Grinding equipment (ii) Steel collar, (iii) Steel plate (this is required before capping when the cores are stored in water at 20 ± 2ºC) and (iv) Compression testing machine.

i. Core Cutting Equipment

ii. Core Cutting in the Field

iii. Different Types of Drills



iv. Compressive Strength Machine

Fig. A11 Coring Equipments and Testing

The standard procedure is briefly given below:

i) Test the core in compression not less than two days after end preparation and immersing in water.

ii) Placing the core in the testing machine. Wipe the bearing surfaces of the testing machine and of any auxiliary platens clean and remove any water, loose sand or other material from the ends of the core.

iii) Placing the core in the testing machine. Wipe the bearing surfaces of the testing machine and of any auxiliary platens clean and remove any water, loose sand or other material from the ends of the core.

iv) Record the maximum load. Normal failures are reasonably symmetrical. Note any unusual failures and the appearance of the concrete.

Compressive strength of each core by dividing the maximum load by the cross-sectional area, calculated from the average diameter.

1. For cores free of reinforcement. Calculate the estimated in-situ cube strength from the equation.

   Estimated in-situ cube strength = $(D/(1.5+(1/\phi)))$x measured compressive strength of core

Where,

D is 2.5 for cores drilled horizontally (for precast units perpendicular to height when cast); or 2.3 for cores drilled vertically (for precast units parallel to height when cast).

A is the length (after end preparation) /diameter ratio

2. For cores with reinforcement perpendicular to the core axes.

(a) For cores containing a single bar:

$$\mathbf{1.0+1.5(\phi_r d/\phi_c l)}$$

(b) For specimens containing two bars no further apart than the diameter of the larger bar

$$\mathbf{1.0+1.5(\sum\phi_r d/\phi_c l)}$$

Where,

ϕ_r is the diameter of the reinforcement, ϕ_c is the diameter of specimen, d is the distance of axis of bar from nearer end of specimen, **l** is the length of the specimen after end preparation by grinding or capping.

The presence of reinforcement, cracked or loose caps in cores may affect the strength.

Along with the compressive strength it can also find out the presence of voids & size of voids, honeycombing and cracks. This test method is more reliable while comparing other partially destructive test and visual inspection to assess concrete uniformity (aggregate distribution & compaction). Core samples can be used for petrography or chemical analysis.

Core test is partially destructive, the damage will occur to the structure. On manually controlled machines as failures approached the load-indicator pointer will begin to slow down.

Other 'partially-destructive' techniques like penetration resistance, pull-out, pull-off, internal fracture & break-off methods for assessing strength of surface concrete are generally results in less damage & give instant results.

14. GALVANO-STATIC PULSE TRANSIENT METHOD

Method is used to determine the instantaneous corrosion rate of reinforcing steel in concrete.

The basic principle is the same as that of LPR method (galvano-static perturbation), however this is a transient polarization technique working in the time domain. The small anodic perturbation current results in change of reinforcement potential which is recorded by a reference electrode (usually located at the centre of the counter electrode) as a function of polarization time. The resistance offered by the system to passage of current comprises of an ohmic resistance R_Ω (resistance from concrete between the surface electrode and the steel bar, the double layer capacitance (C_{dl}) at the surface of the steel, and the charge transfer resistance (R_{ct}). It is the latter from which the rate of corrosion is determined. Analysis of the transient response from the Galvano-static Pulse Transient method (**Fig. A12**) allows the resistance and capacitance components of the system to be resolved, enabling an accurate corrosion rate to be determined.

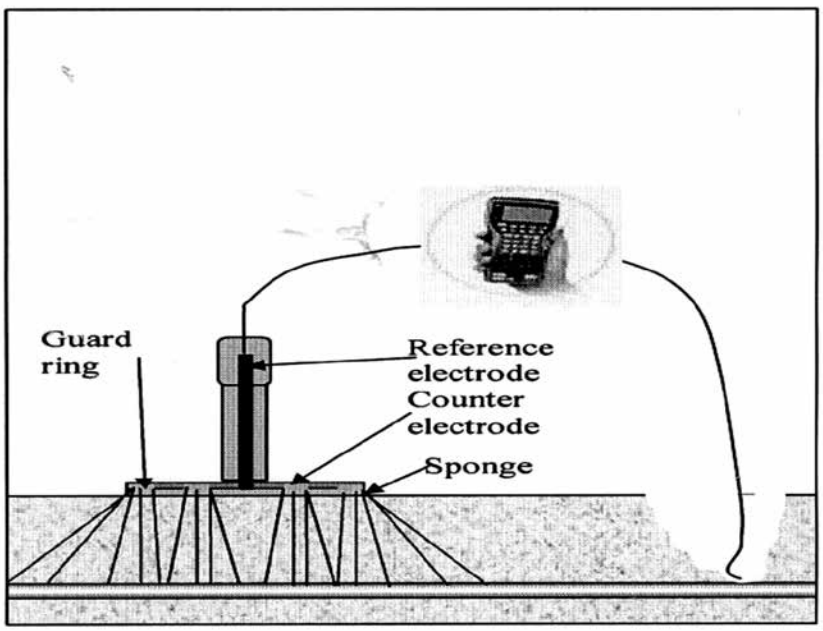


Fig. A12 Galvano-Static Pulse Transient Method

A short time anodic current pulse is impressed to reinforcement galvano-statically (galvano-static polarization), in anodic direction compared to its free corrosion potential, from a counter electrode placed on concrete surface together with a reference electrode. The applied current is normally in the range of 5 to 400 μA and the typical pulse duration is up to 10 seconds. The reinforcement is polarized in anodic direction compared to its free corrosion potential. The resulting change of the electrochemical potential of the reinforcement is recorded by a reference electrode (usually in the centre of the counter electrode) as a function of polarization time as shown in **Fig. A13**.

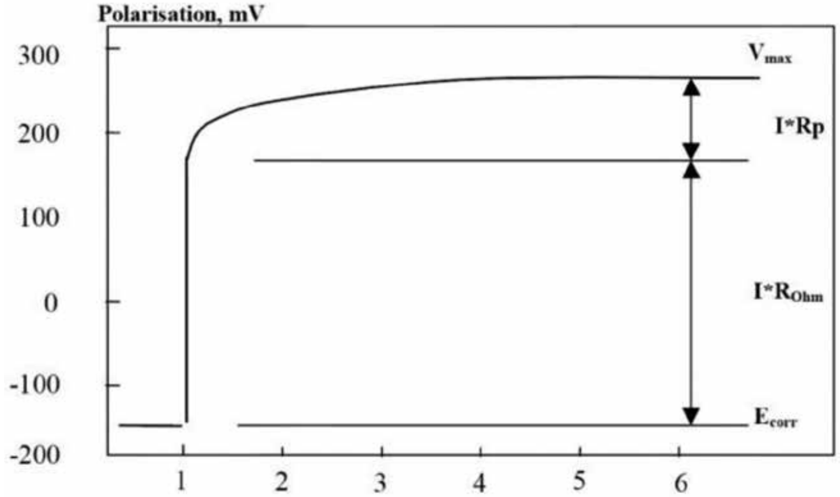


Fig. A13 A typical potential–time curve from Galvanostatic pulse Technique

The following figures (**Fig. A14a**) show typical transient plots for a non-corroding steel (passive steel) and a corroding steel (active steel).

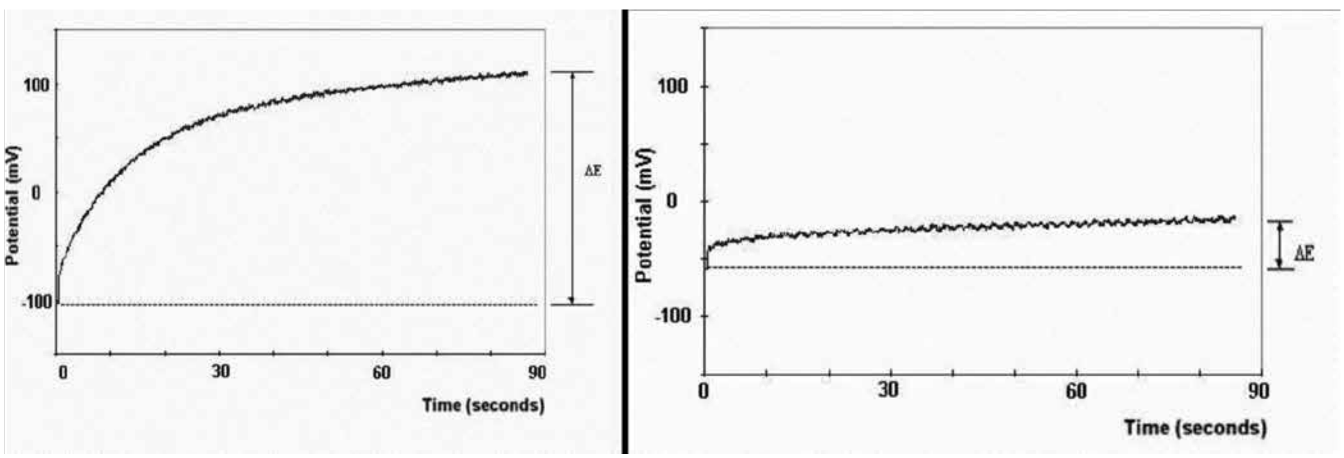


Fig. A14a Potential Transient curves (L): Passive steel (R): Corroding steel

The data obtained is analysed with the help of a Randels electric circuit and through mathematical formulations, the Rct (i.e. R_p) is calculated. The I_{corr} and i_{corr} are then calculated as detailed in LPR test method. Commercially available equipments directly display the icorr when the diameter of the bar is fed into the software.

Corrosion current density is obtained from which the corrosion rate can be estimated.

Corrosion rate measurements are affected by various factors such the temperature and humidity, a lack of correct electrical contact between the equipment and the reinforcement, a lack of correct electrolytic contact between auxiliary sensor and concrete surface and the existence of stray currents.

This test is very quick and information about ohmic resistance (R_{Ω}), double layer capacitance (C_{dl}), and the charge transfer resistance (R_{ct}) can be obtained.

The limitations given with LPR technique also apply to this technique.One specific difficulty with the galvanostatic pulse transient technique is that the response to the pulse has to have stabilized to give an accurate value for Vmax. Curtailing the measurements before an equilibrium value for Vmax has been attained may also lead to errors in the evaluation of R_p and C_{dl}.

Method can be used toassess the present corrosion condition of the reinforcement, that is, to discriminate between corroding and non-corroding (passivated) zones. It may also be used to evaluate the effectiveness of a repair work.

15. HALF -CELL POTENTIAL MEASUREMENTS

These are used to determine the corrosion activity of the reinforcing steel in the concrete and evaluating susceptibility of rebar corrosion in the given environment concrete matrix.

The electrode (half-cell, **Fig. A14b**) potential of reinforcement embedded in concrete provides a measure of the corrosion risk and indicates whether an electro-chemical reaction is in progress on the electrode surface. The electrical potential difference between the reinforcing steel and a reference electrode (copper/copper sulphate half-cell or copper calomel electrode or silver chloride electrode) is measured. Measured potential difference between the reference electrode and the steel rebar is empirically related to the corrosion risk. The measurements are interpreted as per ASTM C 876. Commercial equipment based on the above method is available.

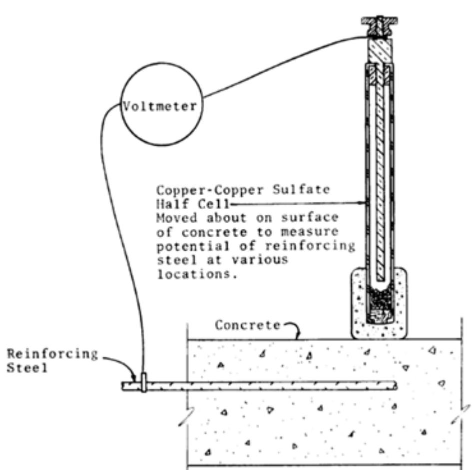
In short the procedure is as below:

i. Spacing Between Measurements-While there is no pre-defined minimum spacing between measurements on the surface of the concrete member, it is of little value to take two measurements from virtually the same point.

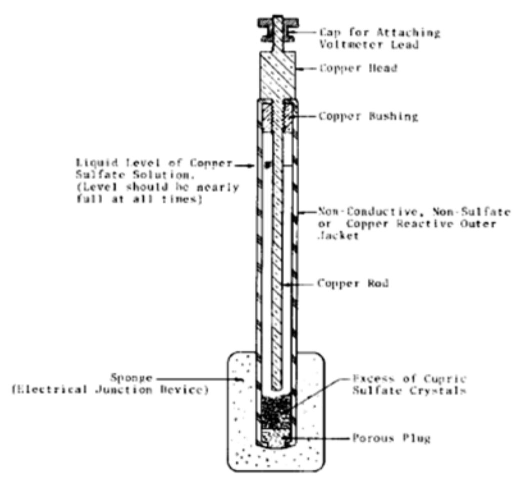
ii. Make a direct electrical connection to the reinforcing steel by means of a compression-type ground clamp, or by brazing or welding a protruding rod.

iii. Electrically connect one end of the lead wire to the half-cell and the other end of this same lead wire to the negative (ground) terminal of the voltmeter.

iv. Conditioning of concrete by Pre-Wetting of the Concrete Surface-This is necessary if the half-cell reading fluctuates with time when it is placed in contact with the concrete.



i. Copper-CopperSulphate Half Cell Circuitry



ii. Sectional View of a Copper-Copper sulphate Half Cell

Fig. A14b Half Cell Measurement

Test measurements may be presented by one or both of two methods.

i. The first an equi-potential contour map, provides a graphical delineation of areas in the member where corrosion activity may be occurring (**Fig. A15**).

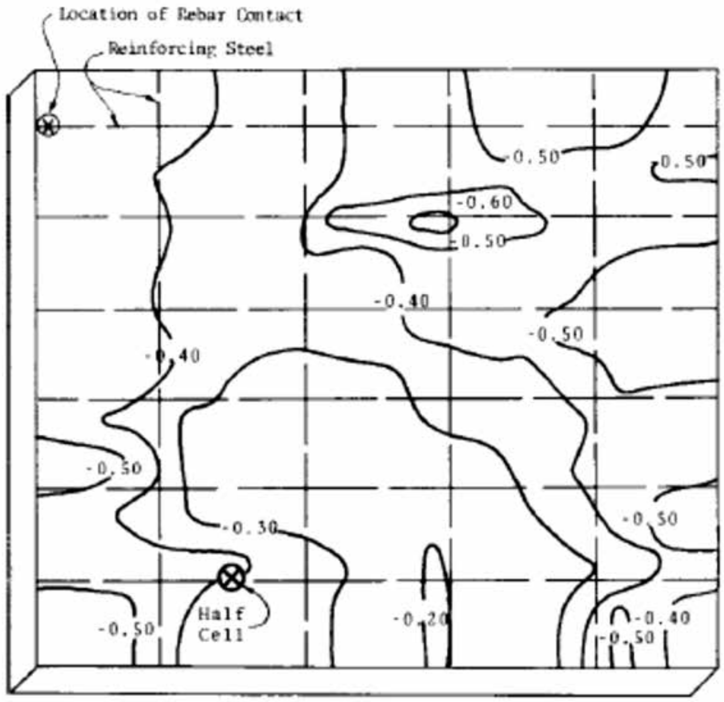


Fig. A14 Equi-potential Contour Map

The second method, the cumulative frequency diagram, provides an comparison of affected area of the concrete member based on the magnitude of potential range

The potential risks of corrosion based on potential difference readings are shown the below **Table A.T1**. Potential difference levels are reference electrode specific and the potential difference may be affected due to temperature and moisture content.

Table No. A T1

Potential difference levels (mv) w.r.t $Cu/CuSO_4$ Electrode	Probability of corrosion occurring in re-bar
More than 500	95%
350 to 500	90%
200 to 350	Uncertain condition
Less than 200	10 to 5%

The method is simple. It allows an almost non-destructive survey to be made to produce iso-potential contour maps of the surface of the concrete member. Zones of varying degrees of corrosion risk may be identified from these maps.

The half-cell potential measurement does not give any information regarding the rate of corrosion. It is often necessary to use other data such as chloride contents, depth of carbonation, delamination survey findings, rate of corrosion results, and environmental exposure conditions, in addition to corrosion potential measurements, to formulate conclusions concerning corrosion activity of embedded steel and its probable effect on the service life of a structure.

Method is mostly to be used for assessment of the durability of reinforced concrete members where reinforcement corrosion is suspected.

The difference between two half-cell readings taken at the same location with the same cell should not exceed 10 mV when the cell is disconnected and reconnected. The difference between two half-cell readings taken at the same location with two different cells should not exceed 20 mV. The results obtained by the use of this test method shall not be considered as a means for estimating the structural properties of the steel or of the reinforced concrete member.

16. HAMMER RAP/HEAVY CHAINS

Hammer/Chains can be used to locate delaminations in concrete bridge decks by sounding technique. The test is carried out as per ASTM D 4580-03.

Chain drag and hammer sounding are generally categorized as crude vibrational modal tests. The operator drags chains or strikes a hammer on the deck, listening to the resulting sound. A clear ringing sound represents a sound deck while a mute/hollow/dull sound represents a delaminated area. The hollow sound is a result of the flexural oscillations of the delaminated section of the deck, creating a drum-like effect. Flexural oscillation of a delaminated area is typically in a 1 to 3 kHz range i.e. audible range to humans. The perceived delaminated areas are outlined on the deck surface.

(a) Chain Drag – Chain dragging is carried out using a custom-made tool consisting of one or more steel chains attached to a handle so that the operator can drag or swing the chains as shown in **Fig. A15**.

Fig. A15: Typical Chains

(b) Hammer Rapping- Tapping the bridge deck surface with a steel rod or hammer is adopted in place of the chain drag particularly onsmall areas and vertical or overhead structural elements (**Fig. A16**).

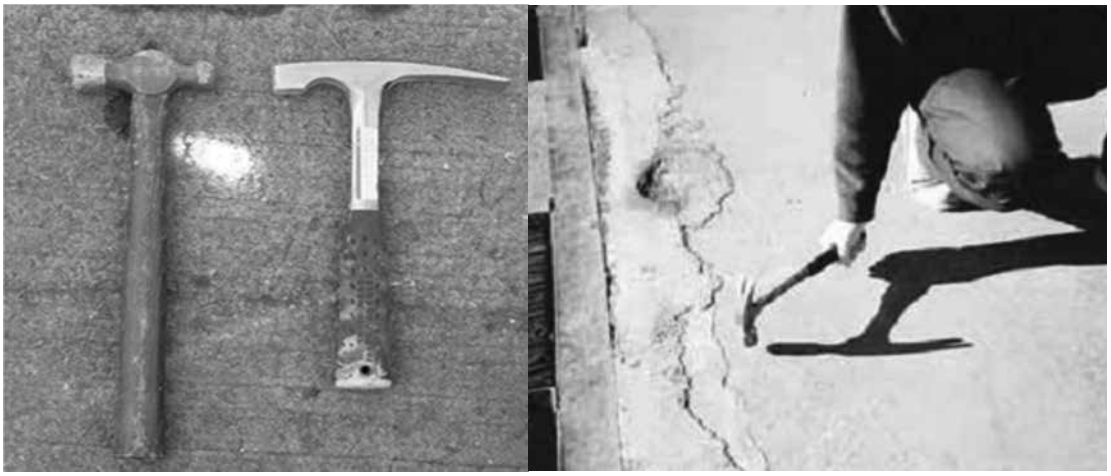
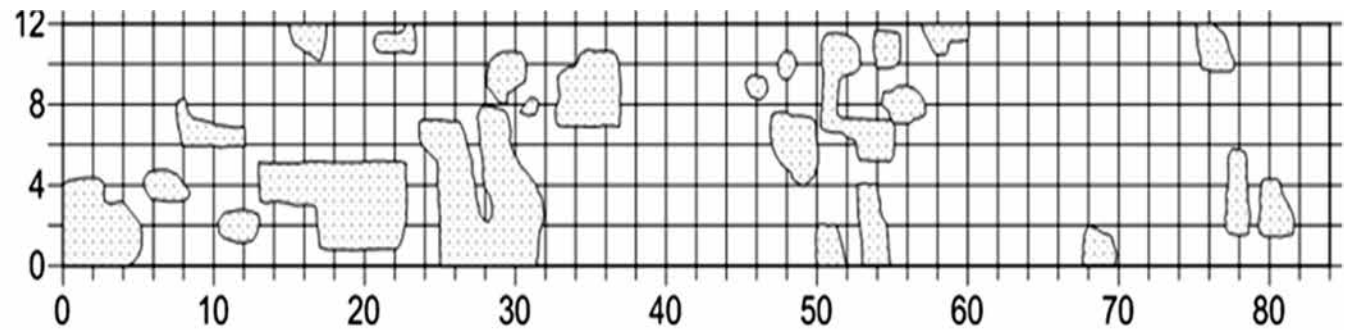


Fig. A16: Typical Hammers

In addition, electromechanical devices or rotary percussion devices may also be used. Marked areas can be mapped on a grid or placed in drawing/mapping software for output as shown in **Fig. A17**. However, no processing of acquired data is needed.



Longitudinal Distance (m)

Fig. A17: Typical Contour Map

Solid concrete will produce a ringing sound, while concrete that is spalled, delaminated, or contains voids will produce a flat or hollow sound.

The variable field conditions such as traffic noise, vibration, moisture content of the concrete may affect the results.

The method is best suited for slab surfaces where large areas can be tested in a reasonable amount of time.

This test method is not applicable on concrete decks overlaid with the bituminous concrete or on frozen concrete.

The devices with automatic recorders or automatic marking on the delaminated areas are available.

The chain drag technique is less accurate than simple hammer or rod soundings. Although a club hammer will locate laminations accurately at depth where the cover to the steel is greater than about 40-50 mm, beyond 100 mm concrete sounding can be ineffective in picking up deep laminations.

17. IMPACT-ECHO

This method is primarily used to determine the thickness of concrete slabs, pavements, bridge decks, walls, or other plate-like structures, locate poor consolidation, voiding and honeycombing in reinforced concrete, detect areas of delamination in concrete, detect de-bonding of an overlay, detect degree of grouting in post-tensioning ducts in plain, RCC and pre-stressed concrete elements.

The impact-echo technique is based on generating a short-duration, low frequency, stress pulse by mechanical impact on a concrete surface. The pulse propagates into the member along spherical wave fronts as P and S waves. These waves are reflected by the boundaries of, internal interfaces or flaws in, or the opposite face of, the concrete member, and are received by receiver or transducer. The time of receipt, or the frequency of the received wave is used to estimate the depth of the member, or locate the flaws.

The equipment for the test method consists of an impact source, a receiver and a data acquisition system with appropriate software. Impact sources with shorter-duration impacts (20 to 60 ms), such as small steel spheres and spring-loaded spherically tipped impactors are commonly used for detecting flaws within slab and wall structures ranging from 0.15 to 1 m thick. However steel spheres are convenient impact sources because they produce well-defined pulses (approximately a half-cycle sine curves), and the contact time is proportional to the diameter of the sphere.

Conically tipped, piezoelectric displacement transducers are used as the receiver as such transducers respond to surface displacement over a broad frequency range.

The data acquisition system may be a digital waveform analyzer, a portable computer with data acquisition hardware and software, or a dedicated impact-echo instruments.

The velocity of the p-wave generated by the impactor is determined first by impacting on the concrete surface where its thickness is already known (**Fig. A18** & **A19**). The velocity is calculated by dividing the thickness of the member and the time of the receipt of the pulse. The impactor is then struck at different locations on the surface of the concrete member. The depth of the delamination, flaw or honeycombing, from the surface is then determined from the time of receipt of the pulse and the velocity of the p-wave. Note that it would take lesser time for the wave to travel the delamination / flaw / honey combing in the member and return to surface after reflection.

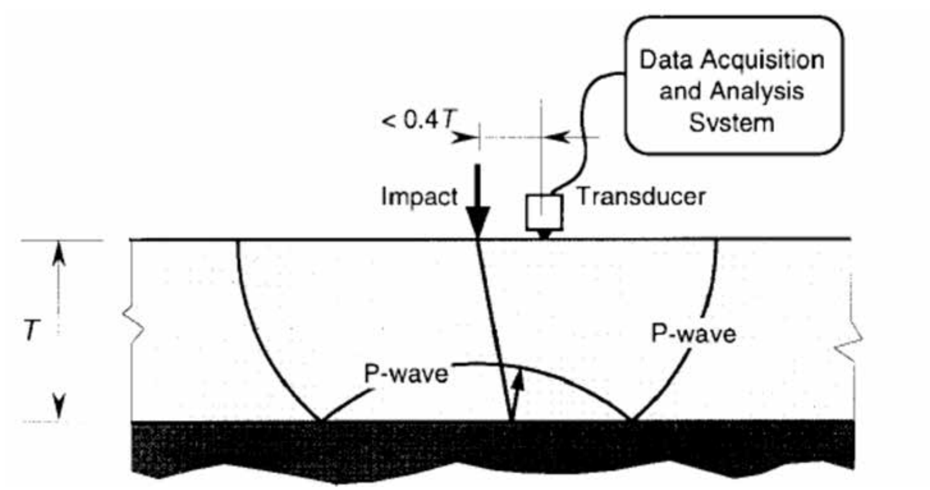


Fig. A18 Schematic of the working principle of Impact-Echo Method

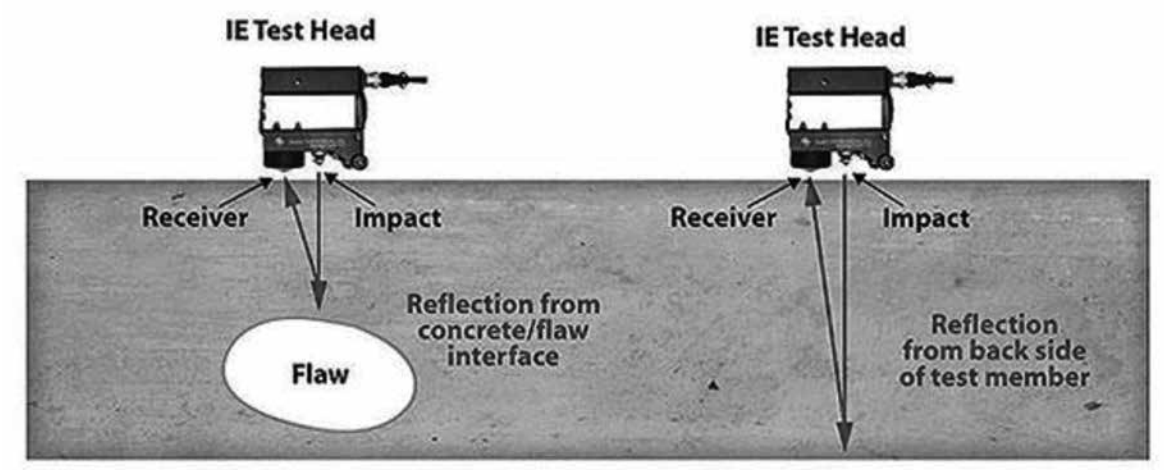


Fig. A19 Illustration of the test method on a concrete surface

The test results may be affected due to:

i. The velocity of the p-wave depends on the quality of concrete. Hence the velocity should be determined for the specific concrete member.

ii. Contact between the transducer tip and the concrete.

The results are obtained in the form the thickness of the concrete members such as deck slabs, beams, walls etc can be determined and homogeneity of the piles.

The presence of delaminations or large horizontal cracks, and voids can be determined. The method is not influenced by traffic noise or low frequency structural vibrations set up by normal movement of traffic across a structure. The another advantage of this method is that the test results are not influenced by weather conditions, or by moisture or chloride in concrete

The contact time (the duration of the Impact) determines the size of the defect that can be detected by impact-echo testing. As the contact time decreases and the pulse contains higher-frequency (shorter-wavelength) components, smaller defects can be detected. In addition, short-duration impacts are needed to accurately locate shallow defects. Contact with the concrete is necessary to perform the test, hence the bituminous wearing course on a bridge deck need to be opened at the test location.

Impact-echo is more reliable when relatively small areas are delaminated so as to indicate early signs of deterioration.

Commercial equipment on impact echo is available to determine the thickness of the concrete bridge deck slabs, plates, pavements etc. Wireless echo system is also available from some manufacturers for use of inspection team with limited man-power.

18. LINEAR POLARIZATION RESISTANCE (LPR) TECHNIQUE

LPR is used to determine the instantaneous corrosion rate measurement of reinforcing steel in concrete.

The method works on the principle that the polarization resistance of a corroding system is inversely proportional to its corrosion rate. An induced change (perturbation) in the potential or current of a corroding system is called polarization. The polarization resistance is determined either by applying a small voltage (10- 20 mV) and measuring the resulting change in current at the end of a selected time period, usually between 30 seconds to 5 minutes (potentiostatic perturbation), or by applying a small current (up to about 20 mA) and monitoring the resulting change in potential (galvanostatic perturbation). It has been shown that the applied voltage and the change in current (or vice versa) are linearly related within this small perturbation, and the proportionality constant is called linear polarization resistance (LPR).

LPR technique has been shown to be an effective electrochemical method for measuring rate of corrosion of steel reinforcement in concrete structures.

The polarization resistance, R_p, of the steel is calculated from the equation

$$R_p = \Delta E / \Delta I$$

Where, Δ**E** is applied voltage, Δ**I** is the change in (step) current when the metal is polarized (about 20-50 mV).

The corrosion rate, I_{corr}, is then calculated as follows:

$$I_{corr} = B/Rp$$

where, **B** is the Stern–Geary constant, a value of 25 mV has been adopted for active (corroding) steel and 50 mV for passive (non-corroding) steel.

The corrosion current density, icorr, can be calculated if the surface area, A, of Steel is known

$$i_{corr} = I_{corr} / A$$

Portable commercial equipment is available for use at site and in the laboratory (**Fig. A20**)

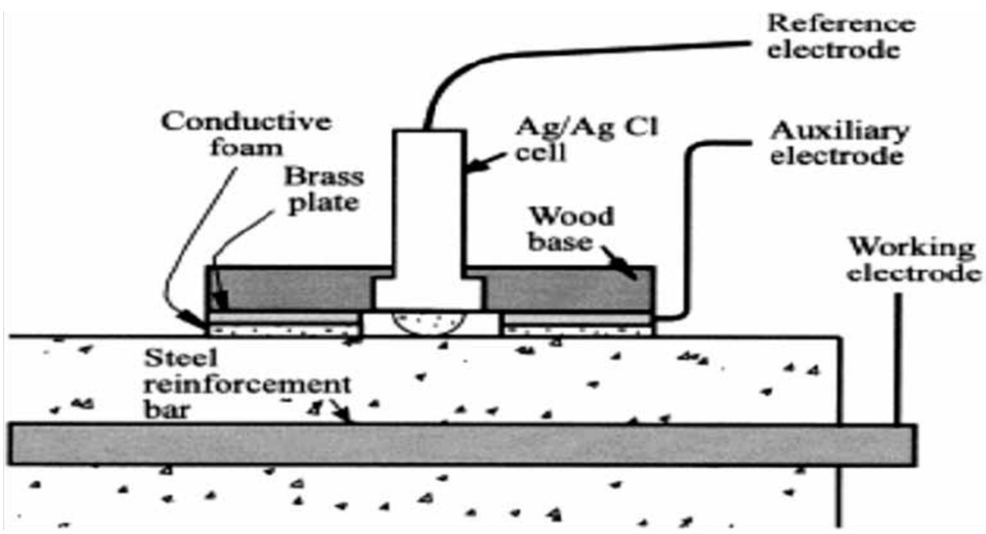


Fig. A20 Schematic of the test set up for Linear Polarization Resistance measurement

In the commercially available equipments, galvanostatic polarization is often employed. In the conventional test procedure, a current perturbation to a corroding steel reinforcement is applied through an auxiliary electrode placed on the concrete surface (**Fig. A20**). In the process, the surface area of steel that is lying directly beneath the auxiliary electrode is assumed to be polarized. However, the current flowing from the auxiliary electrode remains unconfined and spreads laterally over an unknown, larger area of steel. Therefore, to confine the applied current, a guard ring electrode is used which prevents the perturbation current from the auxiliary electrode spreading beyond a known area. In order to select an appropriate level for the confinement current two sensor electrodes are placed between the inner and outer auxiliary electrodes. The potential difference between these sensor electrodes is monitored and a confinement current selected to maintain this potential difference throughout the LPR measurement.

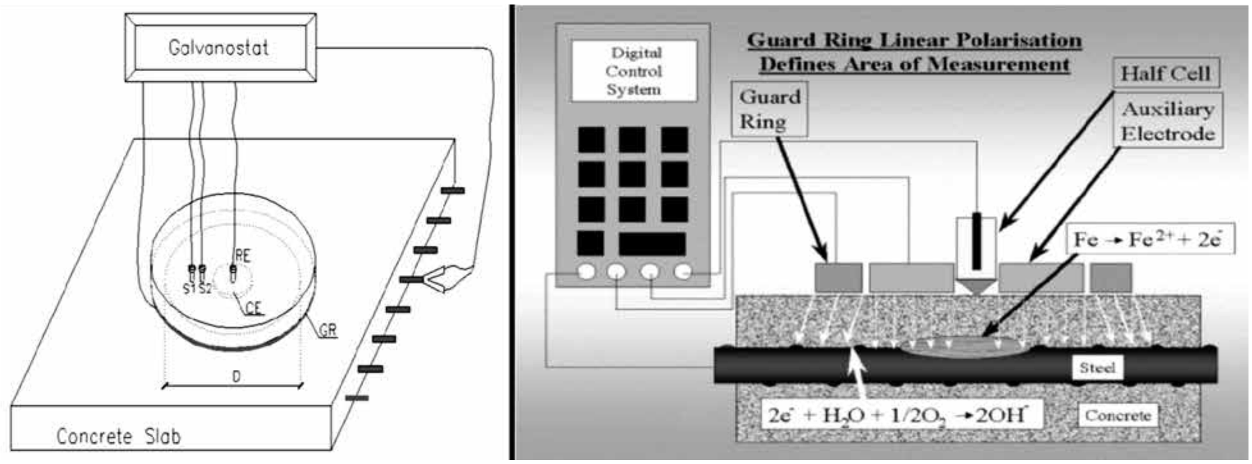


Fig. A21 Guard ring test set-up

The equipment directly displays the corrosion current density i_{corr}, when the diameter of the bar being tested is fed into the equipment. The actual loss of steel is calculated using Faraday's laws as per ASTM G 102.

The corrosion potential measurement may be affected due to (i) lack of correct electrical contact between the equipment and the reinforcement, (ii) A lack of correct electrolytic contact between auxiliary sensor and concrete surface, (iii) The existence of stray currents, (iv) temperature and humidity and (v) Flow of current from the auxiliary electrode is unconfined and can spread laterally over an unknown larger area of steel, which can affect the results.

It is very suitable for on-site condition assessment of steel reinforcement w.r.t corrosion in concrete structures. The instantaneous corrosion rate of steel can be obtained. The method can be applied regardless of the thickness of concrete cover and the rebar size or detailing.

Limitations are: (i)This method assesses only the instantaneous condition of the steel reinforcement, but cannot predict its future condition. Concrete surface condition w.r.t moisture, has a significant role in measurement (ii) On very dry concrete surfaces having concrete resistivity, ρ> 1000 Ωm, measurements are difficult to obtain or the method gives misleading results. (iii) Difficulty in selecting a suitable equilibration time at which to monitor the current response to the applied potential shift. This is because the resistance is made up from a number of different mechanisms occurring within the concrete which include those interfacial resistances at the surface of the bar which are directly related to corrosion and those resistances associated with the bulk concrete and, to calculate the corrosion rate accurately it is necessary to select the measurement time required to evaluate those processes directly associated with corrosion but to exclude those processes associated with the bulk concrete.

Method can be used to assess the present corrosion condition of the reinforcement, that is, to discriminate between corroding and non-corroding (passivated) zones. It may also be used to evaluate the effectiveness of a repair work.

19. PENETRATION RESISTANCE/WINDSOR PROBE TEST

Probes can be used to determine the resistance to penetration of hardened concrete by either a steel probe or pin.

Probe penetration relates 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.

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. 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 test can also be performed with a pin.

Two different methods available for resistance test i.e. with Probes or with Pins. 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. The area to be tested must have a brush finish or a smooth surface as it affects the result.

The evaluation of test can be done only after a co-relation is established with a sample; practical procedure for developing such a relationship is outlined below.

i. Prepare a number of 150 mm × 300 mm cylinders, or 150 mm^3 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.
ii. 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.
iii. Repeat the above procedure for all test specimens.
iv. 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%.

The probe is simple to operate, requires little maintenance except cleaning the barrel and is not sensitive to operator technique. The test is relatively quick and the result is achieved immediately provided an appropriate correlation curve is available. Access for measurement is only needed to one surface. The correlation with concrete strength is affected by a relatively small number of variables. Pullout test can be used advantageously for inaccessible areas and section having dense rebar placement.

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 the rebar affects the reliability of test. 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 levels are used in an investigation to accommodate a larger range of concrete strengths, the correlation procedure becomes complicated. This where co-relation can Such relationships are more reliable if both pullout test specimens and compressive strength test specimens are of similar size, consolidated to similar density, and cured under similar conditions and same age of concrete.

20. PERMEABILITY TEST

This test is used to determine the permeability of concrete in-situ, or concrete specimens either cast in the laboratory or obtained by extracting cores from existing structures.

Permeability is a property that governs the rate of flow of fluid into a porous solid. In case of concrete, it can be used to characterize its transport properties that link to its durability. Depending on the mechanism of movement of fluids in to concrete, the transport processes are classified into three categories, namely,

i. Absorption - movement of fluid due to capillary forces created inside the capillary forces.

ii. Permeation - movement of fluid due to action of pressure.

iii. Diffusion/migration - movement of fluid due to concentration gradient, with or without an applied electrical field.

It may be noted that the above mechanisms may be acting on concrete structures together or separately on different occasions, depending on exposure conditions. Test methods, suitable for in-situ testing or for testing on specimen extracted from the structure, or on laboratory cast specimen for evaluation of different concrete mixes, have been developed to assess the permeability of concrete to fluids (air, water or chlorides) based on the above mechanisms. Generally, the air permeability test is carried out first and then the water permeability test is done on the same specimen. However, not all methods have been adopted as standard test procedures. The following **Table A T2** lists most of the test methods available as standards / developed for the three transport mechanisms.

Table A T2 Test Methods for Permeability

Sl. No.	Transport mechanism	Test method	Test for concrete permeability to
1	Absorption	1. BS 1881 Pt. 208 – Initial Surface Absorption Test	Water
		2. IS 516-Pt. 2 – Initial Surface Absorption Test (under preparation)	Water
		3. Autoclam method	Water
		4. RILEM CPC 11.2	Water
		5. ASTM C 1585	Water
		6. Water absorptivity test (South Africa Test)	Water

Sl. No.	Transport mechanism	Test method	Test for concrete permeability to
2	Permeation	1. IS 3085	Water
		2. Autoclam method	Air and Water
		3. Torrent method	Water
		4. Figg's method	Air and Water
		5. Scholin and Hilsdorf method	Air
		6. Cembureau method	Water
		7. Oxygen Permeability Index Test	Oxygen
		8. German Water Permeation (GWT) Test	Water
		9. BS EN 12390-2009	Water
3	Diffusion / Migration	1. NT Build 443 (Non-steady state chloride diffusion)	Chlorides
		2. NT Build 492 (non-steady state chloride migration)	Chlorides
		3. ASTM C 1202-Electrical indication of chloride migration	Chlorides
		4. PERMIT Ion Migration Test	Chlorides
		5. ASTM C 1556-Chloride profiling-Apparent Chloride Diffusion	Chlorides

As it is beyond the scope of this document to discuss details of the above test methods, specialist literature may be referred for the same.

Air permeability

Determination of air permeability of concrete is useful to assess its state of compaction, presence of bleed voids and channels, and the degree of interconnectedness of the pore structure. It is determined either in terms of time over which a pressure applied to a concrete surface of known surface area decreases by a certain value, or increase in pressure over a concrete surface of known surface area on which vacuum was created previously, or in terms of suitable units. Good correlation between Oxygen Permeability Index (OPI) values recorded at 28 days and carbonation depths after natural exposure were reported.

Water Permeability

Water permeability of concrete is measured as a flow of water under a constant or decreasing pressure gradient through concrete. This property is of great importance when an assessment of hydro structures, reservoirs or any other civil engineering structure such as sub-structures in bridges etc. that are in direct contact with water is carried out. Structures exposed to harsh environmental conditions also require low porosity as well as permeability.

Instruments for testing water permeability on-site usually consist of a reservoir filled with water, which is connected to the concrete surface, a syringe or vacuum pump for introducing pressure into the reservoir and a transducer for monitoring pressure. Most of the laboratory methods consist of subjecting a mortar or concrete specimen of known dimensions, contained in a specially designed cell, to a known hydrostatic pressure from one side, measuring the quantity of water percolating through it during a given interval of time and computing the coefficient of permeability. The test permits measurement of the water entering the specimen as well as that leaving it.

Fig. A22 below show some of the air/ water permeability test apparatus often employed at site and / or in the laboratory.

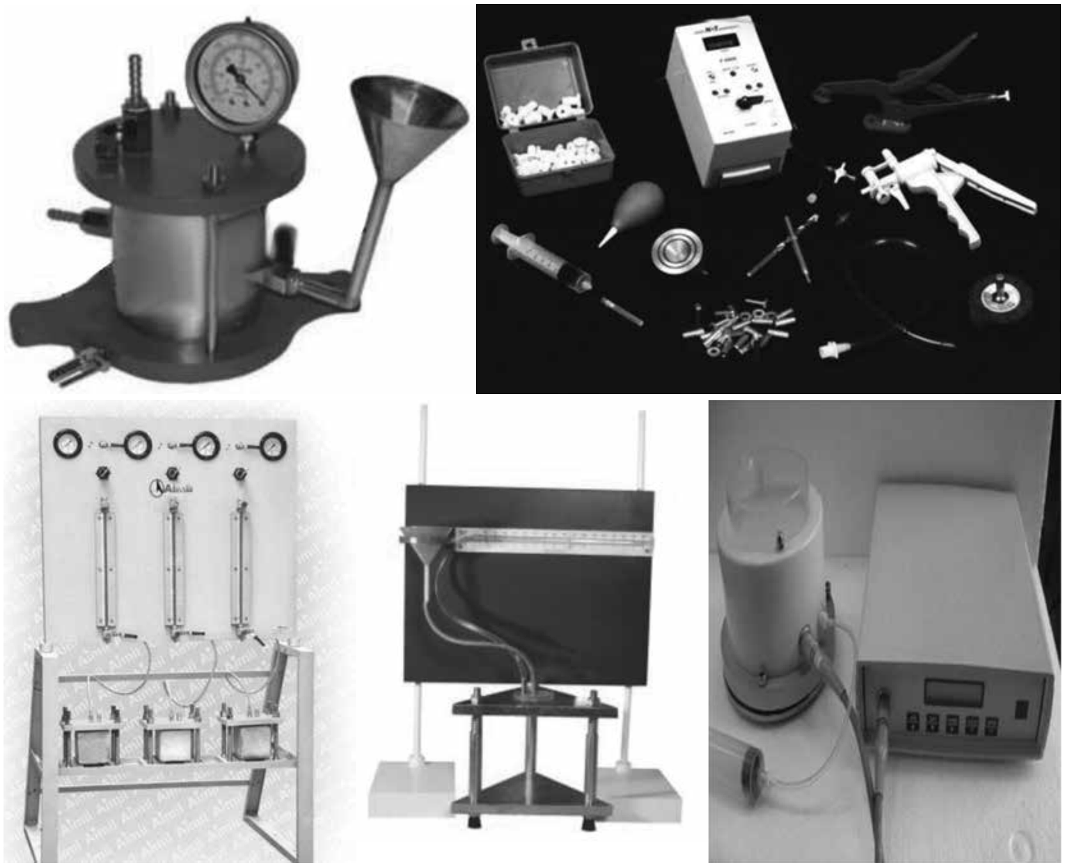


Fig. A22 Air/Water Permeability Test Apparatus

The test results may be affected due to: Moisture conditions; concrete mix; aggregate; surface finish and type; curing, age of concrete, cracking, water type and temperature may affect the permeability.

Many of the methods mentioned above are developed to be simple.

Although the in-situ tests require minimum surface preparation, some of the laboratory methods need conditioning of the specimen. While some tests are completed in 10-15 minutes after preparation and condition, other tests need longer times from 1-3 days, or more.

The methods do not measure permeability directly but provide res a 'Permeability Index' or permeability coefficient, or sometimes, which is related closely to the method of measurement. 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. For the ISAT, tests on oven dried specimens give reasonably consistent results but in other cases results are less reliable 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.

21. PETROGRAPHY TESTING

Petrography is used to determine and identify any potential Alkali-Aggregate Reaction (AAR) constituents.

Alkali-aggregate reaction occurs when minerals in certain aggregates react with the soluble alkaline components of the cement paste. There are two main forms of alkali-aggregate reaction namely, alkali-silica reaction and alkali-carbonate reaction.

The method involves visual examination of polished slabs under a binocular microscope, followed by detailed examination of thin sections under a polarizing microscope. The flowchart illustrating the progressive stages of a petrographic investigation with the level of evaluation increasing depending on the amount of detailed information required are given in **Fig. A23** below:

On-Site Investigation

Visible deterioration observed in the structure

↓

Laboratory Visual Identifications

Major components including aggregates, cement and related materials and any signs of deterioration

↓

Confirmation

Identification of specific mineral components of aggregates, cement and other related materials

↓

Quantitative Analysis

Statistical quantitative determination of air-void sizes, volumes and proportions of fly ash, slag, admixtures and similar components

↓

Detailed Laboratory Investigations

Detailed investigation of specific components and features of mineral interactions, deterioration and failures

Fig. A23 Petrographic Investigation Flow Chart

Samples for petrographic examination can be taken from lump samples or cores. Samples of mortar are impregnated with epoxy resin and sawn. The sections are prepared by careful grinding and polishing in a non-aqueous media. The polished surfaces are studied with at least 50x magnification and features of interest are measured by traditional point counting techniques or image analysis. More detailed studies use auxiliary methods like Scanning Electron Microscopy (SEM) coupled with Back Scattered Electron Imaging (BEI) to evaluate the hydraulic content. X-Ray Diffraction (X-RD) is also used for determination of some crystalline components.

Equipment consists of (i) Zoom stereo binocular microscope (**Fig. A24**), (ii) A high quality petrological photomicroscope fitted with a digital camera and (iii) A point counting stage for the petrological microscope to enable point counting of thin sections.

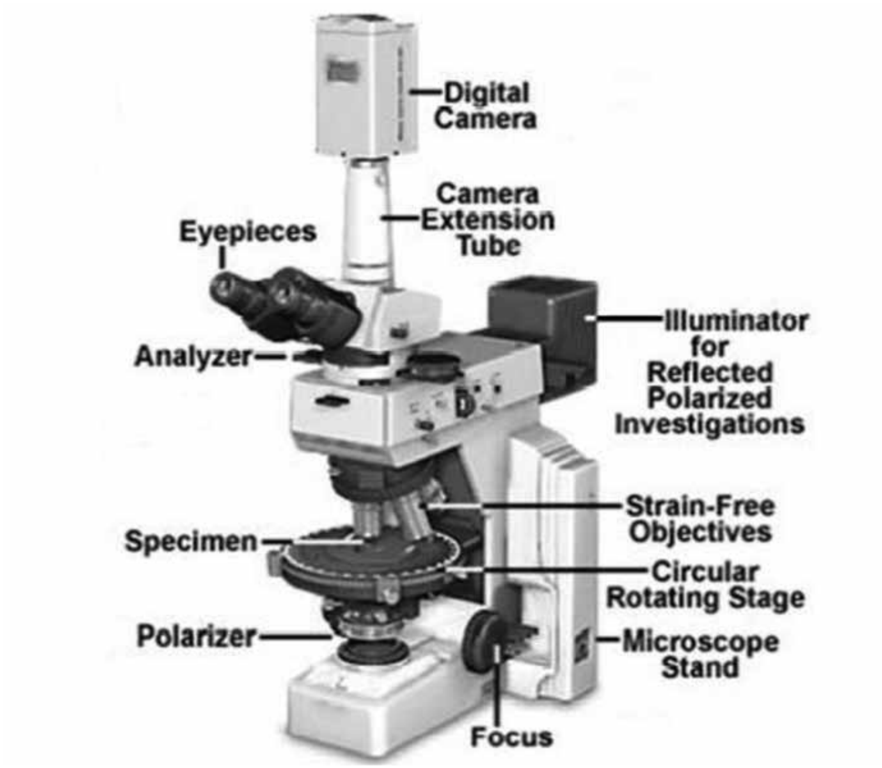


Fig. A24 Microscope

The results of a petrography analysis are used to establish the quality of the concrete by:

i. Determination in detail of the condition of concrete in a construction.
ii. Determination of the causes of interior quality, dis-tress, or deterioration of concrete in a construction.
iii. Determination of the probable future performance of the concrete.

Broken fragments of concrete are generally not use in petrographic examination, because the damage to the concrete cannot be clearly identified as a function of the sampling technique or representative of the real condition of the concrete.

Method can also used for identification of alkali-aggregate reaction in thin section and polished slabs. Petrographic Testing it is possible to diagnose the main cause of failure in a structure.

This technique is mostly to be used for assessment of the durability of reinforced concrete members where reinforcement corrosion is suspected. The method is also commonly used when repairing buildings of historical significance. Elemental mapping aids in the viewing of particle sizes and distribution.

22. Pull-Out Test

To determine the in-situ concrete compressive strength Pull-Out Test.

This test is based on the principle that the force required to pull out a cone of steel embedded in concrete is proportional to the strength of concrete **(Fig. A25)**. The insert is either cast into fresh concrete or installed in hardened concrete. The measured pull-out strength is indicative of the strength of concrete within the region represented by the conic frustum defined by the insert head and bearing ring. The pull-out strengths are indicative of the quality of the outer zone of concrete members and can be of benefit in evaluating the cover zone of reinforced concrete members.

In short the procedure is as under:

Inserts are placed at the location of tests. Such inserts are fixed to the location with bolts or proper fastening methods. The selected test surface shall be flat for drilling the core and undercutting the groove. Drill a core hole perpendicular to the surface to provide a reference point for subsequent operations. Use the expansion tool to position the expandable insert into the groove and expand the insert to its proper size. Place the bearing ring around the pullout insert shaft, connect the pullout shaft to the hydraulic ram, and tighten the pullout assembly snugly against the bearing surface, checking to see that the bearing ring is centered around the shaft and flush against the concrete. Apply load at a uniform rate so that the nominal normal stress on the assumed conical fracture surface increases. Record the maximum gage reading to the nearest half of the least division on the dial. It is to be borne in mind that levelling, curing period of grout in hole may affect the results.

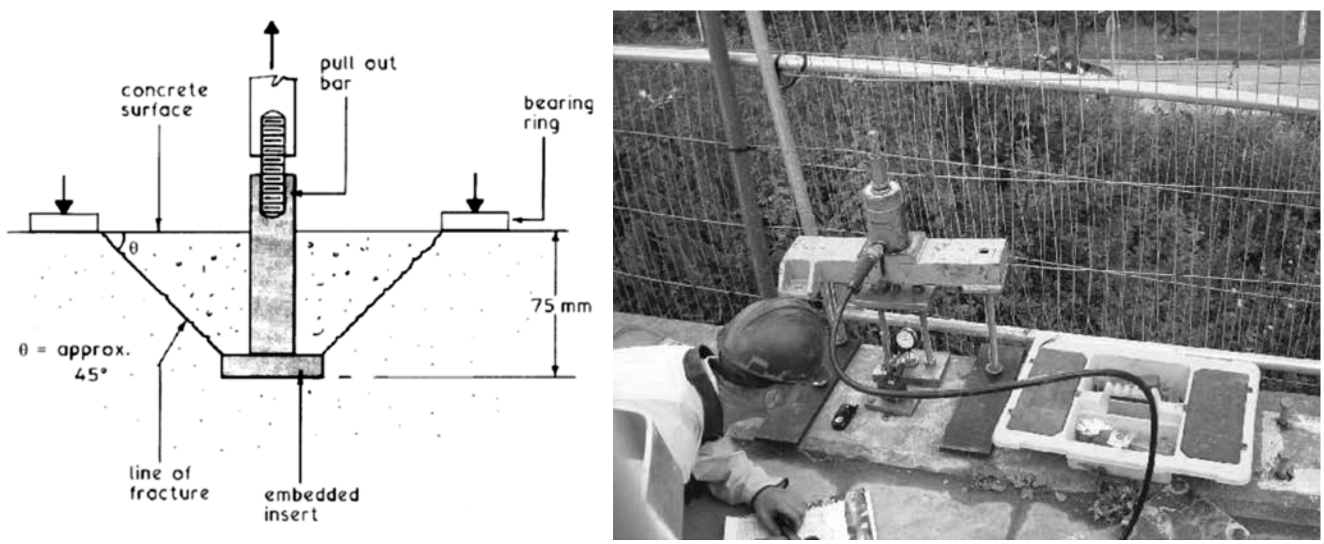


Fig. A25 Pull-Out Test

Levelling, curing period of grout in hole may affect the results.

For a given concrete and a given test apparatus, pullout strengths can be related to compressive strength test results. Such strength relationships are affected by the configuration of the embedded insert, bearing ring dimensions, depth of embedment and the type of aggregate (lightweight or normal weight). Before use, the relationships must be established for each test system and each new concrete mixture. Such relationships are more reliable if both pullout test specimens and compressive strength test specimens are of similar size, consolidated to similar density, and cured under similar conditions.

This test method does not provide statistical procedures to estimate other strength properties. The disadvantage of the pull out test is that it causes small damage to the concrete is required to be repaired.

If a strength relationship has been established experimentally and accepted by the specifier of tests, pullout tests are used to estimate the in-place strength of concrete to establish whether it has reached a specified level so that, like (i) post-tensioning may proceed, (ii) forms and shores may be removed, (iii) structure may be placed into service, or (iv) winter protection and curing may be terminated. In addition, post-installed pullout tests may be used to estimate the strength of concrete in existing construction. Pullout test can be used advantageously for inaccessible areas and section having dense rebar placement.

This test can be performed provided co-relation is established and accepted. Co-relation can are more reliable if both pullout test specimens and compressive strength test specimens are of similar size, consolidated to similar density, cured under similar conditions and has same age of concrete.

23. RESISTIVITY MEASUREMENT

Resistivity measurement is useful for identifying areas of reinforced concrete at risk from corrosion. For evaluating susceptibility of rebar corrosion in the given environment – concrete matrix.

This test works on a simple principle (**Fig. A26**) of current applied between the two outer probes and the potential difference measured between the two inner probes.

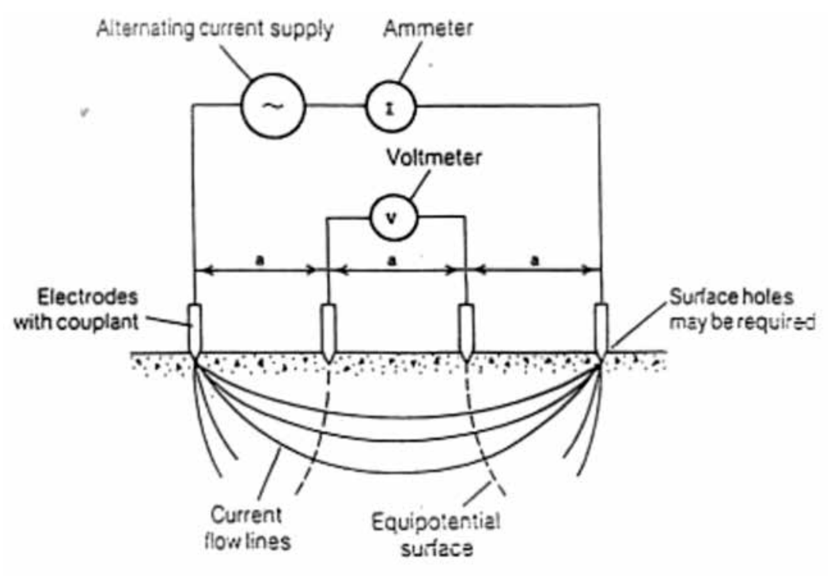


Fig. A26 Schematic of Wenner 4 probe Resistivity Meter

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 spacing.

A low frequency alternating electrical current is passed between the two outer electrodes whilst the voltage drop between the inner electrodes is measured.

For evaluating susceptibility of rebar corrosion in the given environment concrete matrix. Corrosion risk or activity of steel in concrete, the apparent resistivity (ρ) in "ohm-cm" may be expressed as:

$$\rho = 2\pi a(V/I)$$

Where, **V** is voltage drop, **I** is applied current, and **a** is electrode spacing and the resistivity values are given in the **Table A T3** below for the rate of corrosion.

Table No. A T3 Resistivity and Corrosion Rate

Sl. No.	Resistivity (ohm cm)	Likely Corrosion Rate
1	Less than 5,000	Very high
2	5,000 – 10,000	High
3	10,000 – 20,000	Low / Moderate
4	Greater than 20,000	Negligible

Few factors like moisture, salt content, temperature, water/cement ratio and mix proportions affects the results

Resistivity measurement is a fast, simple and cheap *in situ* non-destructive method to obtain information related to the corrosion hazard of embedded reinforcement. with little or no damage to the concrete structures.

To establish reliability of corrosion evaluationIt may be used in conjunction with other corrosion tests such as the half-cell potential measurement or linear polarization measurement method.

Conditioning of concrete surface required for meaningful results.

24. TRANSIENT DYNAMIC RESPONSE TEST

To locate delamination, cracking or loss of integrity in concrete the Transient Dynamic Response Test can be used.

The test procedure uses the propagation of wave in a media to determine the impedance and mobility of the section. The mobility of section, frequency of the wave, area of cross section where the test is applied, length of wave propagation can be used to calculate the compressive strength of the concrete.

In principle, for each step, the wave from an incident pulse travels through the section and is partly or wholly reflected at impedance changes within the member. If any of these returning reflections are in phase with the incident frequency, resonant makes the response of the section maximum. A graph of the maximum velocity against frequency of excitation would therefore show a series of resonating peaks with intervening troughs. This is the main principle on which TDR works is shown in **Fig. A27** below:

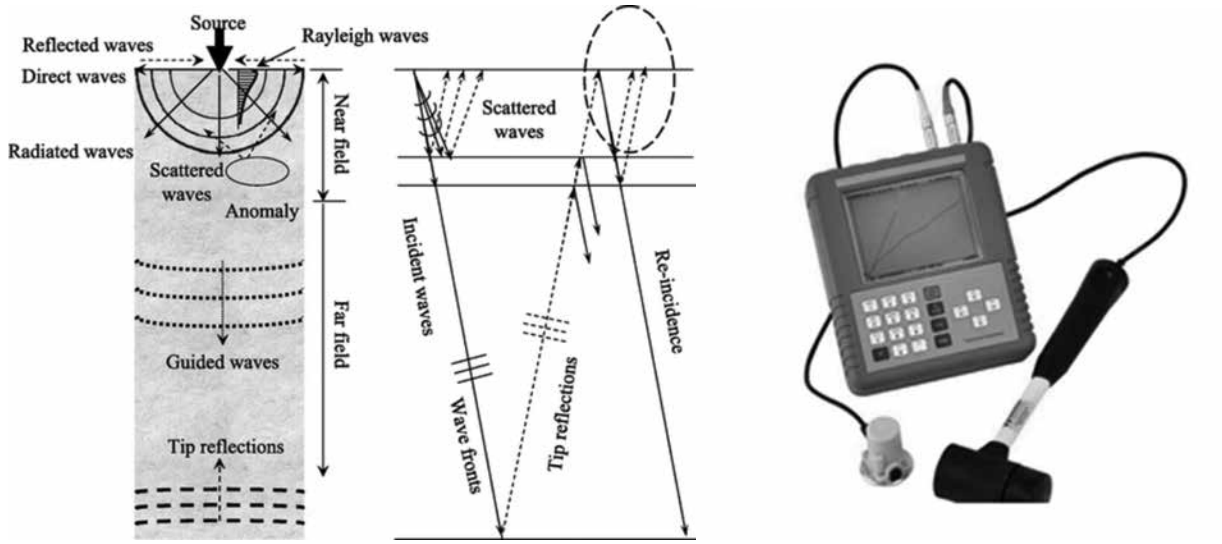


Fig. A27 The TDR Principle& Set up

Applying an impulse force to the section, usually by striking it with a light hand-held instrumented hammer, carries out the test. This impulse force is to be measured by means of a small electronic dynamic load cell. The response of the section to the hammer blow is to be measured by a velocity transducer (or geophone) held on the surface of the section. The force wave imparted onto the section and the resulting velocity response of the section are separately analyzed.

The tests is carried out first by striking the surface and recording the reading as an average of best correlated three reading from the four reading taken. Test measurements may be presented either an equi-potential contour map or the cumulative frequency diagram, indicating the magnitude of affected area of the concrete member.

Impedance at any given level in the section is therefore usually expressed by the relationship below:

$$\mathbf{z = \rho . c . A}$$

where ρ is the density of the material used, c is the propagation velocity of the stress wave and A is the cross sectional area of the section.

For slabs and pavements a diameter of 1000 mm. is be considered for the cross sectional area. It is imperative that the transmitter and the receiver are kept within this diameter.

The results of this test could be affected due to traffic moving on the bridge. As any movement causes vibrations that will also be recorded by the geophone.

One of the major advantage of this test is that it is rapid and if all sections are prepared a single team can test about a hundred sections. But this test cannot be performed on new concrete sections where the concrete is not fully set.

In the recent advances TDR evaluates mobility of the section, using it with available geometrical data and the frequency interval we can get the characteristic strength of a section.

25. TREPANNING TEST

Trepanning technique is developed to assess the in-situ stress under biaxial stress state.

The proposed technique employs a three element strain gage rosette to measure the strain release due to core-drilling (**Fig. A28**).

Fig. A28 Trepanning Equipment

The evaluation of in-situ stresses under bi-axial stress state by using concrete core trepanning technique was developed.

Experimental studies for assessment of stresses under biaxial stress condition can be carried out on ten specimens with different combination of loads.

The main advantage to the method is that the strains released on the core itself can be quite large, depending on the nature of the stress being investigated.

This method has several disadvantages when compared to the incremental core-drilling method like (i) The strain gauge must be bonded to the structure in question and must stay bonded through the coring process, (ii) The strain gauge must be aligned with the direction of maximum principal stress. This is not always known a priori and (iii) The wiring for the bonded gages must be kept undamaged and out of the way during the coring process, which might be difficult in a field situation.

26. CHEMICAL ANALYSIS

(A) Sulphate Content

The sulphate content of a sample is determined through precipitation of barium sulphate using barium chloride.

The chemical reagents used for the method are: (a).Hydrochloric acid (b) Dilute hydrochloric acid (c) Ammonium hydroxide solution (d) Barium chloride solution, and (e) Methyl red indicator solution.

The concrete sample is broken / crushed into a fine powder, passing 150 micron sieve (preferably 100 micron), ensuring in the process that the mortar part only is crushed and visible aggregate (coarse or fine) particles are excluded. Sufficient quantity (i.e. up to 25-30 grams) of powder is prepared so that at least three iterations can be carried out. The cement content of this sample is determined as per the procedure given BS 1881-Pt. 124, Section 5.9, or estimated from the mix proportions, if available.

Concrete powder sample of 5 ± 0.005 gm is weighed into a 400 ml beaker, in to which 50 ml of water and add 10 ml of concentrated hydrochloric acid are added. After adding 50 ml of hot water, the beaker is covered the and the contents are boiled gently for 5 min to 10 min. The cooled contents of the beaker are filtered through a medium ash less filter paper and the residue is washed thoroughly with hot dilute hydrochloric acid (1 part conc HCl + 49 parts water). Three drops of methyl red indicator are added to the filtrate (the liquid which has passed the filter paper) and the same is heated to boiling. The filtrate is just neutralized to yellow with dilute ammonium hydroxide solution. Immediately after that, 1 ml of concentrated hydrochloric acid is added to it and followed by drop--wise addition of 10 ml of barium chloride solution to the boiling solution. If excess ammonium hydroxide was added, 1 ml of concentrated hydrochloric acid may not be sufficient to obtain the required acid solution and the barium sulphate precipitate will then be contaminated. In this case the test shall be repeated. The solution is then boiled gently for 5 min, keeping the solution at just below boiling for 30 min. The solution is then allowed to remain at room temperature for 12 to 24 hr., and then filtered through a slow ashless filter paper, washing the residue free from chlorides with hot water. The filter paper and contents are transferred to a weighed silica or platinum crucible and burnt-off without flaming. The precipitate is ignited at 800 ºC to 900 ºC until a constant mass is achieved.

The sulphate content G, expressed as SO_3, as a percentage of the cement to the nearest 0.1 % (m/m) from the expression

$$G = \frac{L}{M_d} * 34.3 * \frac{100}{C^1}$$

Where,

M_d is the mass of the sample used (in gm);

C^1 is the cement content of the sample used (in %);

L is the mass of ignited barium sulphate (in gm).

IS: 2720-Part 27 (Indian standard methods of test for soils- Part 27- Determination of total soluble sulphates) is also based on similar principle. The same may also be adopted with suitable modifications.

Sulphate content of the concrete powder sample as percentage of cement content.

A maximum sulphate content of 4 % by weight of cement as recommended by IRC:112.

(B) Acid Soluble (total) Chloride Content

The chloride content in a concrete sample is determined through titration with silver nitrate solution by Volhard method.

The concrete sample is broken / crushed into a fine powder, passing 150 micron sieve (preferably 100 micron), ensuring in the process that the mortar part only is crushed and visible aggregate (coarse or fine) particles are excluded. Sufficient quantity (i.e. upto 25-30 grams) of powder is prepared so that at least three iterations can be carried out. The cement content of this sample is determined as per the procedure given BS 1881-Pt. 124, Section 5.9, or estimated from the mix proportions, if available.

The chemical reagents used for the testing are: (a) Nitric acid, relative density 1.4 (b) Silver nitrate standard solution, 0.1 mol/L (c) Thiocyanate standard solution, approx. 0.1 mol/L (d) Iron III indicator solution (Ammonium Ferric Sulphate solution added with 10 mL of nitric acid, (e) 3,5,5 - trimethylhexanol (nonyl alcohol).

A powdered sample of concrete weighing 5 ± 0.005 g is taken into a stoppered 500 ml conical flask. 50 ml of water and 10 ml of nitric acid are added to the sample in the conical flask. 50 ml of hot water is added to the mixture and the mixture is boiled for 4 to 5 min. The mixture is kept warm for 10 min to 15 min and then allowed to cool to room temperature. The mixture is then filtered through an ashless filter paper into a flask and the remainder is washed with hot water. A measured excess of the silver nitrate standard solution is added into the beaker. Then, 2 to 3 ml of nonyl alcohol is added, the flask is stoppered and shaken vigorously to coagulate the precipitate. Then 1 ml of iron III indicator solution is added and titrated with the thiocyanate solution to the first permanent red colour.

The acid soluble chloride content J of the concrete sample is calculated as a percentage of the cement to the nearest 0.01 % (m/m) from the expression:

$$J = \langle V_5 - \frac{mV_6}{0.1} \rangle \frac{0.3545}{M_c} * \frac{100}{C_1}$$

Where,

M_c is the mass of sample used (in g);

V_s is the volume of 0.1M silver nitrate solution added (in mL);

V_6 is the volume of thiocyanate solution used (in mL);

m is the molarity of the thiocyanate solution (in mol/L);

C_1 is the cement content of the sample used (in %).

Factors affecting the test results are:

i. Loss of material, particularly dust, during the crushing and grinding operations should be minimized for better results.

ii. Exposure to Atmospheric carbon dioxide.

IS 14959 (Indian standard-Determination of water soluble and acid soluble chlorides in mortar and concrete — Method of test) also is based on titration and can be adopted. However, the sampling method is slightly different.

Test method based on potentiometric principles using ion-selective electrodes are also available. They also may be adopted to determine the chloride content of concrete samples. Rapid test kits for in-situ chloride determination are also available and may be employed. However, in case of any dispute, the chemical method is taken as reference.

(C) Alkali Content

Method is used to determine the alkali oxide content will be determined.

The sampling procedures, Treatment of samples and analytical methods to be used on a sample of concrete to determine Alkali content is explained in BS 1881:Part 124.

The sodium oxide and potassium oxide contents as percentages to the nearest 0.01 % (m/m) of the sample used from the expression:

$$H=(0.2k/Mn)$$

Where, H is the alkali oxide content (in mg/L); k is the concentration of alkali (in mg/L); Mn is the mass of the sample (in g).

The sodium oxide equivalent content of the concrete Nb is calculated in kg/m³ from:

(a) where the analytical sample was used

$$N_b = \frac{(U + W + 0.5581)}{100} * r^9$$

Where, u is the sodium oxide content (in %); w is the potassium oxide content (in %); r^9 is the oven dried density of concrete (in kg/m³).

(b) where a sample of separate fines was used

$$N_b = \frac{N_e}{100} * \frac{c_1}{100} * r^9$$

Where, C^1 is the cement content (m/m) of the concrete,see 5.9 (in %).

Loss of material, particularly dust, during the crushing and grinding operations should be minimize for better results. Exposure to Atmospheric carbon dioxide affects the result.

The samples from other NDT tests like core cutting, pull out can be used.

27. CRACK MEASUREMENTS

The appearance of cracks is a phenomenon that cannot be predicted a priori, very often evaluating their effects is complicated and costly when using traditional techniques. The possibility to have monitoring systems that allow to accurately locate and to obtain crack width dimensions has become a challenge.

Width of Cracks can be found out.

The various equipments (**Fig. A29**) consist of: (i) Measuring Magnifier (ii) Crack Width Meter (iii) Crack Monitor (iv) Crack Meter (v) Field Microscope

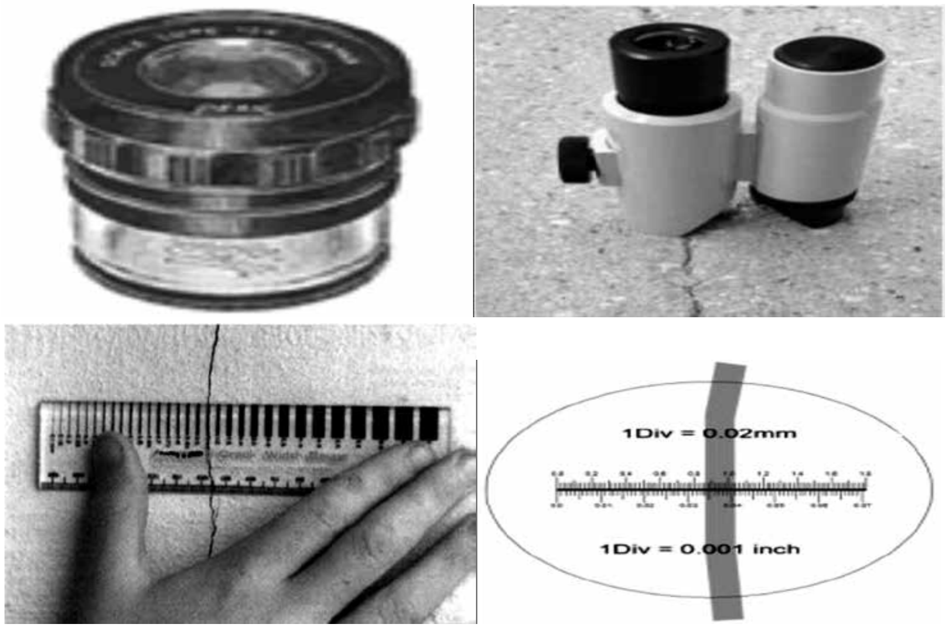


Fig. A29 Crack Measuring Equpments

Hot weather placement produced much wider cracks than cool weather placement.

Crack measurements may be affected due to: (i) Season of Year of Placement, (ii) Coarse Aggregate Type and Slab Temperature, (iii) Steel Reinforcement, (iv) Time of Crack Occurrence, (v) Crack Spacing, (vi) Temperature and (vii) Age of structure.

APPENDIX-II

RESEARCH AND DEVELOPMENT

A2.1 Introduction

Repairs and rehabilitation is as much an art as it is a science. The development of techniques and materials for repairs and rehabilitation is continuous process and it is likely that further improvement and advancement is going to help us understanding their behavior and performance in better way. However, performance of bridge structure and its response to loads and environment is subject to various interdependent combinations and processes. It may not be possible to attribute the deterioration to a single phenomenon.

These limitations should be well understood. This chapter has been added only for information to indicate some areas requiring research and development. Needless to say that the list given is not complete.

A2.2 Criteria of Practicality

In seeking to devise improved ways to rehabilitate and strengthen bridges, it is necessary to meet the criteria of practicality as below:

- Techniques must be robust and simple should not be unduly sensitive to workmanship defects;
- Techniques must be resistant to environmental condition;
- It is necessary that repair techniques be improved so that minimum delays are caused to traffic;
- Techniques must be relatively cost effective;
- Techniques should preferably involve proven technology and new ideas should be introduced with due care and attention.

A2.3 Goals of Research

The goals for future research and development will have to be (a) to establish durability oriented technology for design and construction of bridge to be accomplished by developing standards, codes, specifications and detailing and (b) to increase the service life of the existing by improving methods of investigation to quantify the level and rate of deterioration and by containing the future deterioration.

A2.4 Areas of Research

Different areas which call for intensive research and development effort to achieve better rehabilitation and strengthening of bridges are:

– Methods of detection and prediction of distress, damage or degradation of the existing structures and the extent of severity.
– Methods of protection of reinforcements (Ref. IRC:SP-80)
– Protective treatment of concrete surfaces.
– Techniques for identification of corrosion in pre-stressing steels & rebars.
– Efficacy of waterproofing systems.

- Loss of prestress in prestressed concrete members with passage of time in different environmental condition of exposure.
- Techniques for identification of degree & rate of corrosion.
- Development of techniques and materials for improvement of durability.
- Vacuum injections and other methods of grouting of cable ducts containing voids.
- Methods of increasing structural capacity of concrete members by application of external pre-stressing and bonding of steel plates.
- Design procedure for strengthening bridges by bonded plates and advanced composites long term performance of such methods of strengthening under adverse environment.
- Instrumentation techniques for monitoring long term behavior of structural members.
- Standards for grouting of cables in respect of viscosity, bleeding, shrinkage, strength, permeability and bond.
- Better characterization of the physical properties of deteriorating concrete and repair materials for use in established methods of analysis so as to measure more accurately the behaviour of damaged structures and to explore the ways of possible restoration of strength.
- Development of fatigue assessment and methods and techniques.
- Use of materials like fiber reinforced concrete, polymers and resins for repair and rehabilitation of concrete bridges.
- Techniques for repairs of fatigue distresses in steel structures especially the welded connections.
- Factors that affect performance of expansion joints and bearings like e.g. relative movements between decks and bearings. Also design that permit easier maintenance and replacement.
- Quality tests for evaluation of repairs on site.
- Development of expert systems for diagnosis and repair techniques for various combinations of distresses.
- In case of arch bridges, development of codal guidelines based on suitable R&D.
- Development of subjective as well as objective assessment criteria for individual component as well as structure as a whole.
- In case of arch bridges, development of codal guidelines based on suitable R&D.
- Development of deterioration models.
- Development of BMS Software.
- Artificial Intelligence Applications.

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the Code/Guidelines/Manual, etc. from the date specified therein)

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