STRUCTURAL HEALTH MONITORING USING DESTRUCTIVE, NON-DESTRUCTIVE EVALUATION

Project Report submitted in partial fulfillment of the requirement for the degree of

Bachelor of Technology

In

Civil Engineering

Under the Supervision of **Mr Abhilash Shukla**

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CERTIFICATE

This is to certify that project report entitled “Structural Health Monitoring Using Destructive and Non-Destructive Evaluation”, submitted by Mitesh Jhajharia and Akanksha Mehta in partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering to Jaypee University of Information Technology, Waknaghat, Solan has been carried out under my supervision.

This work has not been submitted partially or fully to any other University or Institute for the award of this or any other degree or diploma.

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ABSTRACT

Structures are assemblies of load carrying members capable of safely transferring the superimposed loads to the foundations. Their main and most looked after property is the strength of the material that they are made of. Concrete, as we all know, is an integral material used for construction purposes. Thus, strength of concrete used, is required to be 'known' before starting with any kind of analysis. In the recent past, various methods and techniques, called as Non-Destructive Evaluation (NDE) techniques, are being used for Structural Health Monitoring (SHM).

The concept of nondestructive testing (NDT) is to obtain material properties of in place specimens without the destruction of neither the specimen nor the structure from which it is taken. However, one problem that has been prevalent within the concrete industry for years is that the true properties of an in-place specimen have never been tested without leaving a certain degree of damage on the structure. For most cast-in-place concrete structures, construction specifications require that test cylinders be cast for 28-day strength determination. Usually, representative test specimens are cast from the same concrete mix as the larger structural elements. Unfortunately, test specimens are not an exact representation of *in-situ* concrete, and may be affected by variations in specimen type, size, and curing procedures.

The rebound hammer test is classified as a hardness test and is based on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges. The energy absorbed by the concrete is related to its strength. There is no unique relation between hardness and strength of concrete but experimental data relationships can be obtained from a given concrete. However, this relationship is dependent upon factors affecting the concrete surface such as degree of saturation, carbonation, temperature, surface preparation and location, and type of surface finish. A correlation between rebound number and strength of concrete structure is established, which can be used as well for strength estimation of concrete structures.

This work presents a study on the correlation and comparison between Destructive and a Non-Destructive Method (Rebound Hammer) of testing the compressive strength of normal cured concrete cubes, accelerated cured concrete cubes and concrete cubes made by replacing 25& cement with fly ash and employing the method of accelerated curing . Concrete cubes of 150mm x 150mm x 150mm were produced using concrete mix of grade 15N/mm^2, 20N/mm^2, 25N/mm^2 and 30N/mm^2 and first type of cubes cured for 7, 14, 21 and 28 days and second and third type cubes cured for 3 days at 90 degree Celsius . A total of 48 cubes were prepared for normal curing, 8 cubes were prepared for accelerated curing and 8 cubes were prepared with admixtures and accelerated cured and used for the study. To establish mathematical relationship between compressive strength obtained from rebound hammer and UTM the method of curve fitting was adopted. The Compressive strength by rebound hammer and by UTM was taken as the independent and dependent variable respectively.

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Chapter – 1
INTRODUCTION

To keep a high level of structural safety, durability and performance of the infrastructure in each country, an efficient system for early and regular structural assessment is urgently required. The quality assurance during and after the construction of new structures and after reconstruction processes and the characterization of material properties and damage as a function of time and environmental influences is more and more becoming a serious concern. Non-destructive testing (NDT) methods have a large potential to be part of such a system. NDT methods in general are widely used in several industry branches. Aircrafts, nuclear facilities, chemical plants, electronic devices and other safety critical installations are tested regularly with fast and reliable testing technologies. A variety of advanced NDT methods are available for metallic or composite materials.

In recent years, innovative NDT methods, which can be used for the assessment of existing structures, have become available for concrete structures, but are still not established for regular inspections. Therefore, the objective of this project is to study the applicability, performance, availability, complexity and restrictions of NDT.

The purpose of establishing standard procedures for nondestructive testing (NDT) of concrete structures is to qualify and quantify the material properties of in-situ concrete without intrusively examining the material properties. There are many techniques that are currently being research for the NDT of materials today. This report focuses on the one of the NDT methods relevant for the inspection and monitoring of concrete materials i.e. Rebound Hammer Test. The rebound hammer is an equipment which can effectively be used for qualitative evaluation and to calculate the compressive strength of a concrete structure without damaging it. Further destructive testing using UTM was performed and correlation between NDT and DT was drawn. Factors like curing and admixtures which affect the strength of concrete have also been taken into consideration while drawing the correlations.

Chapter – 2
LITERATURE REVIEW

2.1 Structural Health Monitoring

The process of implementing a damage detection and characterization strategy for engineering structures is referred to as **Structural Health Monitoring** (SHM). Here damage is defined as changes to the material and/or geometric properties of a structural system, including changes to the boundary conditions and system connectivity, which adversely affect the system's performance. The SHM process involves the observation of a system over time using periodically sampled dynamic response measurements from an array of sensors, the extraction of damage-sensitive features from these measurements, and the statistical analysis of these features to determine the current state of system health. For long term SHM, the output of this process is periodically updated information regarding the ability of the structure to perform its intended function in light of the inevitable aging and degradation resulting from operational environments. After extreme events, such as earthquakes or blast loading, SHM is used for rapid condition screening and aims to provide, in near real time, reliable information regarding the integrity of the structure.

Qualitative and non-continuous methods have long been used to evaluate structures for their capacity to serve their intended purpose. Since the beginning of the 19th century, railroad wheel-tappers have used the sound of a hammer striking the train wheel to evaluate if damage was present. In rotating machinery, vibration monitoring has been used for decades as a performance evaluation technique. Two techniques in the field of SHM are wave propagation based techniques and vibration based techniques. Broadly the literature for vibration based SHM can be divided into two aspects, the first wherein models are proposed for the damage to determine the dynamic characteristics, also known as the direct problem, and the second, wherein the dynamic characteristics are used to determine damage characteristics, also known as the inverse problem. In the last ten to fifteen years, SHM technologies have emerged creating an exciting new field within various branches of engineering.

2.1.1 Structural Health Monitoring Using Non-Destructive Testing

The quality of new concrete structures is dependent on many factors such as type of cement, type of aggregates, water cement ratio, curing, environmental conditions etc. Besides this, the control exercised during construction also contributes a lot to achieve the desired quality. The present system of checking slump and testing cubes, to assess the strength of concrete, in structure under construction, are not sufficient as the actual strength of the structure depend on many other factors such as proper compaction, effective curing also. Considering the above requirements, need of testing of hardened concrete in new structures as well as old structures, is there to asses the actual condition of structures. Non-Destructive Testing (NDT) techniques can be used effectively for investigation and evaluating the actual condition of the structures. These techniques are relatively quick, easy to use, and cheap and give a general indication of the required property of the concrete. This approach will enable us to find suspected zones, thereby reducing the time and cost of examining a large mass of concrete. The choice of a particular NDT method depends upon the property of concrete to be observed such as strength, corrosion, crack monitoring etc.

The subsequent testing of structure will largely depend upon the result of preliminary testing done with the appropriate NDT technique.

The NDT being fast, easy to use at site and relatively less expensive can be used for

(i) Testing any number of points and locations.
(ii) Assessing the structure for various distressed conditions.
(iii) Assessing damage due to fire, chemical attack, impact, age etc.
(iv) Detecting cracks, voids, fractures, honeycombs and weak locations.
(v) Assessing the actual condition of reinforcement.

Many of NDT methods used for concrete testing have their origin to the testing of more homogeneous, metallic system. These methods have a sound scientific basis, but heterogeneity of concrete makes interpretation of results somewhat difficult. There could be many parameters such as materials, mix, workmanship and environment, which influence the result of measurements.

Moreover the test measures some other property of concrete (e.g. hardness) yet the results are interpreted to assess the different property of the concrete e.g. (strength). Thus, interpretation of the result is very important and a difficult job where generalization is not possible. Even though operators can carry out the test but interpretation of results must be left to experts having experience and knowledge of application of such non-destructive tests.

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

A. For strength estimation of concrete:

(i) Rebound hammer test.

(ii) Ultrasonic Pulse Velocity Tester.

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

(iv) Pull off test.

(v) Pull out test

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

(i) Half cell potentiometer.

(ii) Resistively meter test.

(iii) Test for carbonation of concrete.

(iv) Test for chloride content of concrete.

(v) Profometer.

(vi) Micro cover meter

C. For detection of cracks/voids/ delaminating etc.:

(i) Infrared thermographic technique.

(ii) Acoustic Emission techniques.

(iii) Short Pulse Radar methods.

(iv) Stress wave propagation methods:

a) Pulse echo method.

b) Impact echo method.

c) Response method.

2.2 NON DESTRUCTIVE EVALUATION (NDE) METHODS

INTRODUCTION TO NDE METHODS

Concrete technologists practice NDE methods for

(a) Concrete strength determination. (b) Concrete damage detection.

2.2 (a) STRENGTH DETERMINATION BY NDE METHODS:

Strength determination of concrete is important because its elastic behavior & service behavior can be predicted from its strength characteristics. The conventional NDE methods typically measure certain properties of concrete from which an estimate of its strength and other characteristics can be made. Hence, they do not directly give the absolute values of strength.

2.2 (b) DAMAGE DETECTION BY NDE METHODS:

Global techniques: These techniques rely on global structural response for damage identification. Their main drawback is that since they rely on global response, they are not sensitive to localized damages. Thus, it is possible that some damages which may be present at various locations remain un-noticed.

Local techniques: These techniques employ localized structural analysis, for damage detection. Their main drawback is that accessories like probes and fixtures are required to be physically carried around the test structure for data recording. Thus, it no longer remains autonomous application of the technique. These techniques are often applied at few selected locations, by the instincts/experience of the engineer coupled with visual inspection. Hence, randomness creeps into the resulting data.

2.2.1 NDE METHODS IN PRACTICE

Visual inspection: The first stage in the evaluation of a concrete structure is to study the condition of concrete, to note any defects in the concrete, to note the presence of cracking and the cracking type (crack width, depth, spacing, density), the presence of rust marks on the surface, the presence of voids and the presence of apparently poorly compacted areas etc. Visual assessment determines whether or not to proceed with detailed investigation.

The Surface hardness method: This is based on the principle that the strength of concrete is proportional to its surface hardness. The calibration chart is valid for a particular type of cement, aggregates used, moisture content, and the age of the specimen.

The penetration technique: This is basically a hardness test, which provides a quick means of determining the relative strength of the concrete. The results of the test are influenced by surface smoothness of concrete and the type and hardness of the aggregate used.

Again, the calibration chart is valid for a particular type of cement, aggregates used, moisture content, and age of the specimen. The test may cause damage to the specimen which needs to be repaired.

The pull-out test: A pullout test involves casting the enlarged end of a steel rod after setting of concrete, to be tested and then measuring the force required to pull it out. The test measures the direct shear strength of concrete. This in turn is correlated with the compressive strength; thus a measurement of the in-place compressive strength is made. The test may cause damage to the specimen which needs to be repaired.

The rebound hammer test: The Schmidt rebound hammer is basically a surface hardness test with little apparent theoretical relationship between the strength of concrete and the rebound number of the hammer. Rebound hammers test the surface hardness of concrete, which cannot be converted directly to compressive strength. The method basically measures the modulus of elasticity of the near surface concrete. The principle is based on the absorption of part of the stored elastic energy of the spring through plastic deformation of the rock surface and the mechanical waves propagating through the stone while the remaining elastic energy causes the actual rebound of the hammer. The distance travelled by the mass, expressed as a percentage of the initial extension of the spring, is called the *Rebound number.* There is a considerable amount of scatter in rebound numbers because of the heterogeneous nature of near surface properties (principally due to near-surface aggregate particles).

There are several factors other than concrete strength that influence rebound hammer test results, including surface smoothness and finish, moisture content, coarse aggregate type, and the presence of carbonation. Although rebound hammers can be used to estimate concrete strength, the rebound numbers must be correlated with the compressive strength of molded specimens or cores taken from the structure.

Ultra-sonic pulse velocity test: This test involves measuring the velocity of sound through concrete for strength determination. Since, concrete is a multi-phase material, speed of sound in concrete depends on the relative concentration of its constituent materials, degree of compacting, moisture content, and the amount of discontinuities present. This technique is applied for measurements of composition (e.g. monitor the mixing materials during construction, to estimate the depth of damage caused by fire), strength estimation, homogeneity, elastic modulus and age, & to check presence of defects, crack depth and thickness measurement. Generally, high pulse velocity readings in concrete are indicative of concrete of good quality. The drawback is that this test requires large and expensive transducers. In addition, ultrasonic waves cannot be induced at right angles to the surface; hence, they cannot detect transverse cracks.

Acoustic emission technique: This technique utilizes the elastic waves generated by plastic deformations, moving dislocations, etc. for the analysis and detection of structural defects. However, there can be multiple travel paths available from the source to the sensors. Also, electrical interference or other mechanical noises hampers the quality of the emission signals.

Impact echo test: In this technique, a stress pulse is introduced at the surface of the structure, and as the pulse propagates through the structure, it is reflected by cracks and dislocations. Through the analysis of the reflected waves, the locations of the defects can be estimated. The main drawback of this technique is that it is insensitive to small sized cracks.

2.3 DESCRIPTION OF THE INSTRUMENT

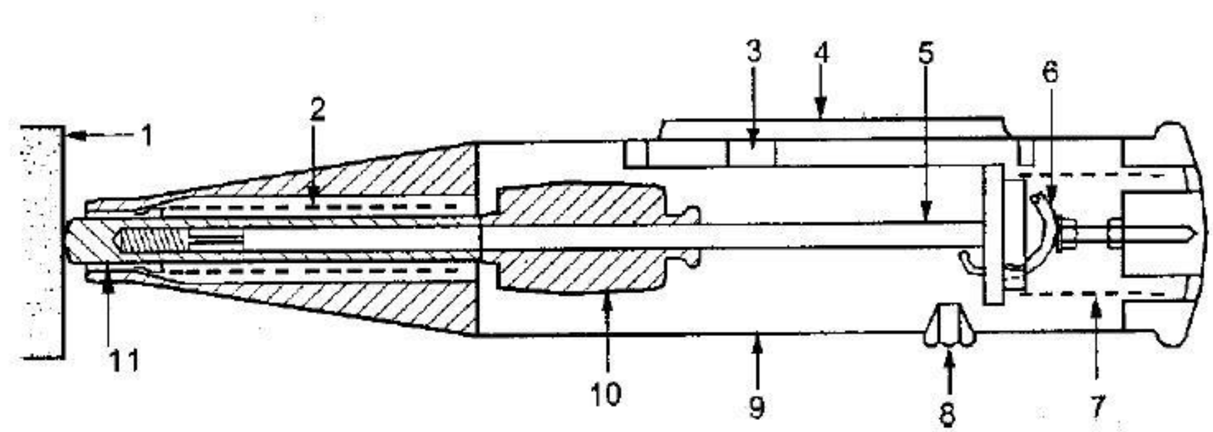
The following instruments were used in the project:

(i) Rebound Hammer (Schmidt Hammer) (Impact energy of the hammer is about 2.2 Nm).

(ii) Universal Testing Machine.

2.3 (a) Rebound Hammer (Schmidt Hammer)

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



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

Fig 2.1 Components of a Rebound Hammer

The rebound hammer method could be used for –

(i) Assessing the likely compressive strength of concrete with the help of suitable co-relations between rebound index and compressive strength.
(ii) Assessing the uniformity of concrete
(iii) Assessing the quality of concrete in relation to standard requirements.
(iv) Assessing the quality of one element of concrete in relation to another.

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

The test is classified as a hardness test and is based on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges. The energy absorbed by the concrete is related to its strength. Despite its apparent simplicity, the rebound hammer test involves complex problems of impact and the associated stress-wave propagation.

There is no unique relation between hardness and strength of concrete but experimental data relationships can be obtained from a given concrete. However, this relationship is dependent upon factors affecting the concrete surface such as degree of saturation, carbonation, temperature, surface preparation and location, and type of surface finish. The result is also affected by type of aggregate, mix proportions, hammer type, and hammer inclination. Areas exhibiting honeycombing, scaling, rough texture, or high porosity must be avoided. Concrete must be approximately of the same age, moisture conditions and same degree of carbonation (note that carbonated surfaces yield higher rebound values). It is clear then that the rebound number reflects only the surface of concrete. The results obtained are only representative of the outer concrete layer with a thickness of 30–50 mm.

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

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

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

Table 2.1 Impact Energy of Rebound Hammers

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

2.3 (b) Universal Testing Machine

"A universal testing machine is used to test the tensile stress and compressive strength of materials. It is named after the fact that it can perform many standard tensile and compression tests on materials, components, and structures."

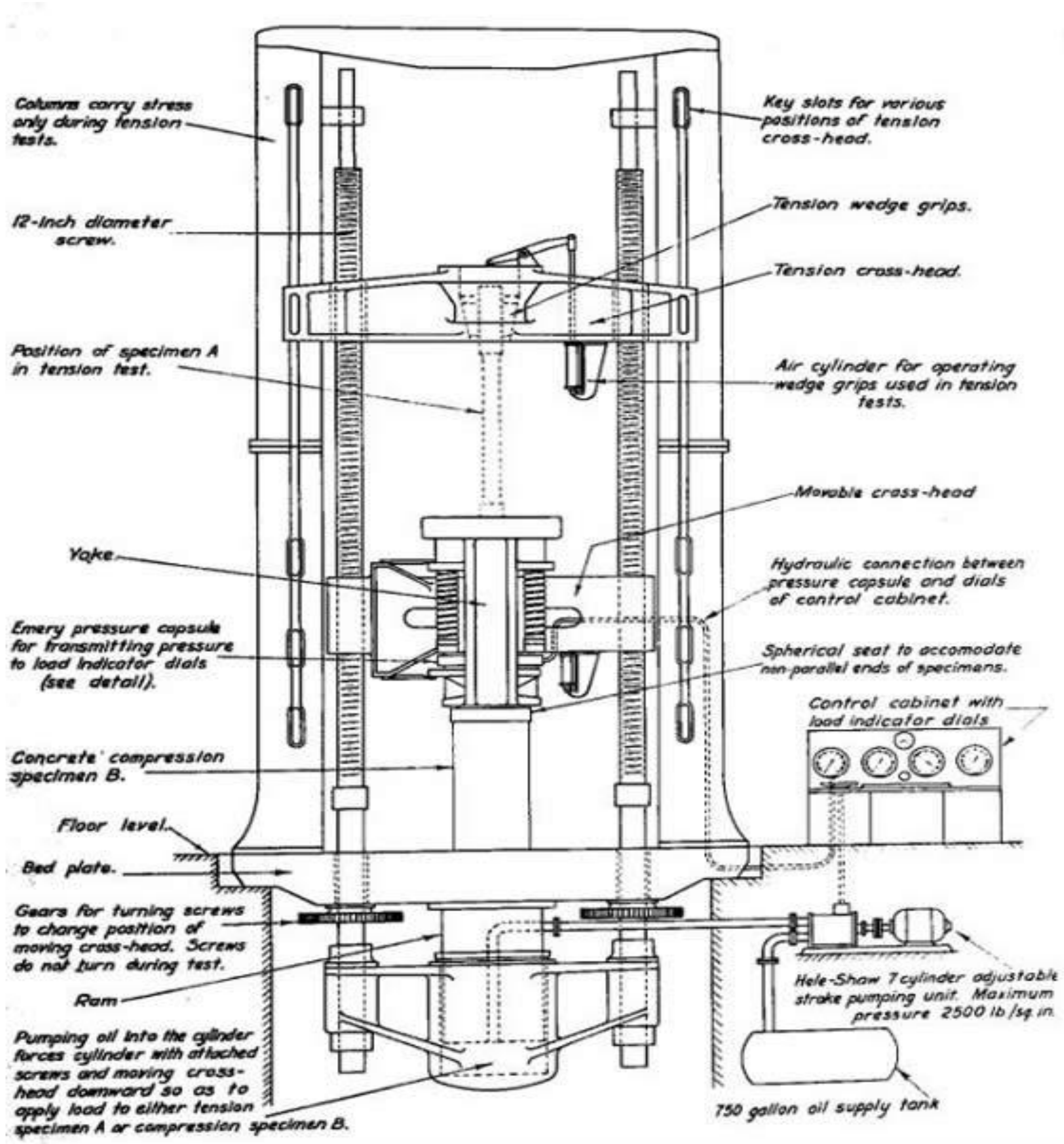


Fig 2.2 Components of Universal Testing Machiene

Components

- Load frame - Usually consisting of two strong supports for the machine. Some small machines have a single support.
- Load cell - A force transducer or other means of measuring the load is required. Periodic calibration is usually required by governing regulations or quality system.
- Cross head - A movable cross head (crosshead) is controlled to move up or down. Usually this is at a constant speed: sometimes called aconstant rate of extension (CRE) machine. Some machines can program the crosshead speed or conduct cyclical testing, testing at constant force, testing at constant deformation, etc. Electromechanical, servo-hydraulic, linear drive, and resonance drive are used.
- Means of measuring extension or deformation - Many tests require a measure of the response of the test specimen to the movement of the cross head. Extensometers are sometimes used.
- Output device - A means of providing the test result is needed. Some older machines have dial or digital displays and chart recorders. Many newer machines have a computer interface for analysis and printing.
- Conditioning - Many tests require controlled conditioning (temperature, humidity, pressure, etc.). The machine can be in a controlled room or a special environmental chamber can be placed around the test specimen for the test.
- Test fixtures, specimen holding jaws, and related sample making equipment are called for in many test methods.

Principle: Operation of the machine is by hydraulic transmission of load from the test specimen through pressure transducer to a separately house load indicator. The system is ideal since it replaces transmission of load through levers and knife edges, which are prone to wear and damage due to shock on rupture of test pieces.

Load is applied by a hydrostatically lubricated ram. Main cylinder pressure is transmitted to the pressure transducer housed in the control panel. The transducer gives the signal to the electronic display unit corresponding to the load exerted by the main ram. Simultaneously the digital electronic fitted on the straining unit gives the mechanical displacement to the electronic display unit. Both the signals are processed by the microprocessor and load and displacement is displayed on the digital readouts simultaneously.

2.4 ACCELERATED CURING

The rate of gain of strength of concrete depends on the reaction rate of cement and additions with water (Hydration). In common with all chemical reactions, the rate of reaction depends on reaction of temp. Higher reaction temperature gives higher rate of reaction e.g. the concrete gains strength more rapidly when its temp. is higher. Accelerated curing is the process by which the temperature of the concrete is raised artificially by applying external heat to speed up the rate of gain in strength.

High early concrete strengths are most efficiently produced by increasing the internal temperature of the concrete while maintaining high moisture content in the curing environment. Heating reduces the relative humidity of the air surrounding the concrete. Thus, moisture must be added to the heated air to maintain the same relative humidity of the air. If adequate moisture isn’t maintained in the curing environment, the concrete won’t develop maximum compressive strength, and cracking may occur. Durability of the concrete may also be reduced due to inadequate hydration of the cementations material.

Three heating methods are commonly used to accelerate curing:

1. Discharging steam or hot air directly in-to the curing environment puts the heating medium directly in contact with the concrete.

2. Enclosing steam or hot water in pipes heats the concrete by convection and radiation.

3. Attaching electrical resistance wires to the forms and covering them with insulation heats the product by heating the forms.

2.4.1 TYPES OF CURING

2.4.1 (a) Steam and heated-air curing

Circulating steam around the products is one of the most widely used accelerated curing methods, primarily due to the ease of producing and transporting steam to the prestressed member. It's an efficient method that increases the temperature and maintains a 100% relative humidity around the concrete products.

Steam can be produced in high- or low-pressure boilers, then piped to the casting bed, or generated by smaller steam packs located close to the products. An advantage of steam is that it contains relatively large quantities of heat per pound of steam at a relatively low temperature. This provides both an effective and economical method of transferring heat from the boilers to the concrete products. Internal temperature can also be in-creased by heating air and discharging it directly into the curing environment. There are two problems with this type of system. First, exhaust gases of unvented fossil-fu-el heaters contain carbon dioxide that com-bines with calcium hydroxide, a byproduct of cement hydration, forming weak calcium carbonates instead of strong calcium silicate hydrates. This produces a white powder on the concrete's surface. Second, reduced moisture in the air al-lows surface drying of the concrete. If heated air is used to accelerate curing, the products should be covered to prevent moisture loss or misted with water to in-crease the relative humidity of the surrounding air and prevent premature drying.

2.4.1 (b) Curing by jacketed heating

These are usually referred to as radiant heating systems. Hot water or steam in pipes transfers heat to the concrete by radiation or convection. In these systems the heating medium circulates back through the boiler, making this a more fuel-efficient curing system than steam curing.

Products must be covered because of the dry curing environment, but this environment also uses insulated forms that make the system more efficient. Conductive systems typically don't heat products as quickly as those using live steam, mainly due to inefficiencies of heat transfer.

Fig 2.3 Accelerated Curing Tank

2.4.1 (c) Curing by electrical heating

Electrical resistance heat has been used in several applications. Electric curing accurately controls temperature and is a reliable system requiring minimal labor. Well-insulated forms are used to conserve energy, but the system may not be cost effective in areas with high electric rates.

Tank will consist of a rectangular double walled metal cabinet, inside lined with stainless steel, outer powder coated. Easily replaceable high Wattage heaters are mounted in side the chamber. A slow speed stirrer is provided to circulate water inside the chamber to maintain the uniform temperature of water. A strong stainless steel perforated platform is provided for keeping the cubes and also have a lid with lifting handle to cover the chamber.

The temperature is indicated and controlled by a DIGITAL auto tuning PID Temp. Controller with soak timer to control the duration of heating and then to shut down the system without manual attendance. The front panel will have power supply indicating lamp, control action indication lamp and one main switch. Suitable for operation on 220 V, 50 Hz single phase, AC supply or 440 V 50 Hz, three phase AC supply, for bigger size of curing tank.

2.4.2 ACCELERATED CURING CYCLE

The accelerated curing cycle can be divided into three periods—preset, rising temperature, and maximum temperature (Figure 2.4).

Preset. Little or no cement hydration occurs during preset. Initial set ends the pre-set period. Heat shouldn't normally be applied until after initial set has occurred. Duration of the preset period is affected by admixture type and dosage, cement type, presence of Pozzolana or ground granulated blast furnace slag (GGBFS), initial concrete temperature, and air temperature in the curing environment.

Since the preset time of the concrete varies with different concrete mixtures and temperatures, testing is the best method for deter-mining the actual duration of the preset time. Use ASTM C 403, *Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance* to determine when initial set has occurred. Since many variables affect initial set, perform this test whenever there are changes in concrete ingredients, mix pro-portions, or concrete temperature. Perform it in the casting yard, not the laboratory, since ambient conditions affect setting time.

You can also detect initial set by monitoring the internal temperature of the concrete. An increase in temperature indicates that initial set has occurred and hydration has begun.It’s important not to add substantial quantities of heat to the concrete until initial set, especially with air-entrained concrete. Research shows that strength losses can occur if concrete is heated excessively prior to attaining initial set. This is mainly due to differential thermal expansions of the air, water, cement and aggregates, and to the low tensile strength of the concrete.

During the preset period, en-trained or entrapped air will expand at significantly different rates than the other materials and possibly cause internal cracking due to the concrete not being strong enough to resist these pressures. This will not only affect early strengths, but it will greatly affect ultimate strength and durability. Research shows that concrete should not be heated above 120° F until after the preset period. As the initial temperature of the concrete increases, the preset period shortens. If the concrete temperature is increased from 70° F to 90° F, it isn’t uncommon for the initial time of set to be reduced by 2 or more hours. If the temperature is reduced to 50° F, the initial setting time may double. After the concrete ingredients and proportions have been established, ambient temperature will have the greatest influence on preset time.

Rising temperature. After initial set, large amounts of heat are required to raise the concrete to maximum temperature. A significant portion of the required heat may be generated internally by cement hydration. Insulated blankets or heavy tarps should be used to control heat loss. Rate of rise may range from 20° F to 80° F per hour. The 28-day strengths aren’t significantly affected by variations in rates with-in this range, provided initial set has occurred before heat is applied.

Typical time temperature curve

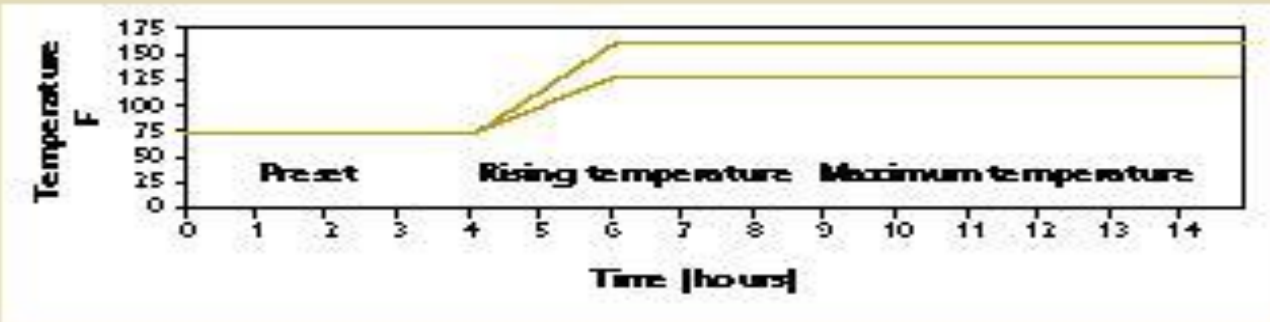


Fig 2.4 In the accelerated curing cycle, initial set marks the end of the preset period. After initial set, large amounts of heat are added to attain maximum temperature and maintain it for the time needed to reach specified strength.

Maximum temperature. Temperature is maintained at the maximum value for the time needed to attain specified strength. During this period, additional heat is needed to replace heat lost due to thermal conduction through the cover and cool air entering the curing environment. Maximum temperatures generally range from 130° F to 160° F. The higher the product temperature, the greater the heat loss; thus, more heat has to be added to the curing environment when high maximum temperatures are used.

Two strategies can be used after maximum temperature is reached. One, maintain the temperature by adding heat until the curing cycle is completed. Two, as an energy efficient strategy, maintain the maxi-mum temperature for a shorter time, shut off the heat, and allow the product to “soak” as the temperature slowly drops. If insulated covers are properly installed, the product doesn’t lose temperature quickly and will reach early strengths similar to those of products heated continuously throughout the maximum temperature period.

After the soaking period, the product is allowed to cool slowly enough to prevent differential thermal cracking. The proper cooling rate depends on product size and shape.

Several systems automate or semi-automate the curing cycle. These energy-saving systems can be set to add heat at the scheduled time, control the rate of temperature rise, and maintain the maximum temperature for a preset duration. They normally use thermal probes that monitor temperature within the product and add heat as needed to maintain the desired curing cycle.

2.5 ADMIXTURE: FLY ASH

Fly ash is used as a supplementary cementitious material (SCM) in the production of portland cement concrete. A supplementary cementitious material, when used in conjunction with portland cement, contributes to the properties of the hardened concrete through hydraulic or pozzolanic activity, or both. As such, SCM's include both pozzolans and hydraulic materials. A pozzolan is defined as a siliceous or siliceous and aluminous material that in itself possesses little or no cementitious value, but that will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds having cementitious properties. Pozzolans that are commonly used in concrete include fly ash, silica fume and a variety of natural pozzolans such as calcined clay and volcanic ash.

SCM's that are hydraulic in behavior include ground granulated blast furnace slag and fly ashes with high calcium contents (such fly ashes display both pozzolanic and hydraulic behavior). The potential for using fly ash as a supplementary cementitious material in concrete has been known almost since the start of the last century . The last 50 years has seen the use of fly ash in concrete grow dramatically with close to 15 million tons used in concrete. Historically, fly ash has been used in concrete at levels ranging from 15% to 25% by mass of the cementitious material component. The actual amount used varies widely depending on the application, the properties of the fly ash, specification limits, and the geographic location and climate. Higher levels (30% to 50%) have been used in massive structures (for example, foundations and dams) to control temperature rise. In recent decades, research has demonstrated that high dosage levels (40% to 60%) can be used in structural applications, producing concrete with good mechanical properties and durability.

Increasing the amount of fly ash in concrete is not without shortcomings. At high levels problems may be encountered with extended set times and slow strength development, leading to low early-age strengths and delays in the rate of construction. These drawbacks become particularly pronounced in cold-weather concreting. Also, the durability of the concrete may be compromised with regards to resistance to carbonation.

Figure 2.5 Fly ash, powder resembling cement, has been used in concrete since the 1930s

For any given situation there will be an optimum amount of fly ash that can be used in a concrete mixture which will maximize the technical, environmental, and economic benefits of fly ash use without significantly impacting the rate of construction or impairing the longterm performance of the finished product. The optimum amount of fly ash will be a function of wide range of parameters and must be determined on a case-by-case basis. For the purposes of this document the replacement levels shown in Table 2.2 will be used to represent low, moderate, high and very high levels of fly ash.

Level of Fly Ash % by mass of total cementitious material	Classification
<15	low
15-30	moderate
30-50	high
>50	Very high

Table 2.2 Dosage level of Fly Ash

Fly ash is a pozzolanic material. It is a finely-divided amorphous alumino-silicate with varying amounts of calcium, which when mixed with portland cement and water, will react with the calcium hydroxide released by the hydration of portland cement to produce various calcium-silicate hydrates (C-S-H) and calcium-aluminate hydrates. Some fly ashes with higher amounts of calcium will also display cementitious behavior by reacting with water to produce hydrates in the absence of a source of calcium hydroxide. These pozzolanic reactions are beneficial to the concrete in that they increase the quantity of the cementitious binder phase (C-S-H) and, to a lesser extent, calcium-aluminate hydrates, improving the longterm strength and reducing the permeability of the system. Both of these mechanisms enhance the durability of the concrete. The performance of fly ash in concrete is strongly influenced by its physical, mineralogical and chemical properties. The mineralogical and chemical composition are dependent to a large extent on the composition of the coal and since a wide range of domestic and imported coals (anthracite, bituminous, sub-bituminous and lignite) are burned in different generating stations in North America, the properties of the fly ash can be very different between sources and collection methods. The burning conditions within a power plant can also affect the properties of the fly ash.

Class	Description in ASTM C 618	Chemical Requirements
F	Fly ash normally produced from burning anthracite or bituminous coal that meets the applicable requirements for this class as given herein. This class of fly ash has pozzolanic properties.	$SiO_2 + Al_2O_3 + Fe_2O_3 \geq 70\%$
C	Fly ash normally produced from lignite or sub-bituminous coal that meets the applicable requirements for this class as given herein. This class of fly ash, in addition to having pozzolanic properties, also has some cementitious properties.	$SiO_2 + Al_2O_3 + Fe_2O_3 \geq 50\%$

Table 2.3 ASTM Specification for Fly Ash

2.5.1 Effect of Fly Ash on the Properties of Fresh Concrete

2.5.1 (a) Workability

The use of good quality fly ash with a high fineness and low carbon content reduces the water demand of concrete and, consequently, the use of fly ash should permit the concrete to be produced at lower water content when compared to a portland cement concrete of the same workability. Although the exact amount of water reduction varies widely with the nature of the fly ash and other parameters of the mix, a gross approximation is that each 10% of fly ash should allow a water reduction of at least 3%. A well-proportioned fly ash concrete mixture will have improved workability when compared with a portland cement concrete of the same slump. This means that, at a given slump, fly ash concrete flows and consolidates better than a conventional portland cement concrete when vibrated. The use of fly ash also improves the cohesiveness and reduces segregation of concrete.

2.5.1 (b) Bleeding

Generally fly ash will reduce the rate and amount of bleeding primarily due to the reduced water demand . Particular care is required to determine when the bleeding process has finished before any final finishing of exposed slabs. High levels of fly ash used in concrete with low water contents can virtually eliminate bleeding. Therefore, the freshly placed concrete should be finished as quickly as possible and immediately protected to prevent plastic shrinkage cracking when the ambient conditions are such that rapid evaporation of surface moisture is likely.

2.5.1 (c) Air Entrainment

Concrete containing low-calcium (Class F) fly ashes generally requires a higher dose of air-entraining admixture to achieve a satisfactory air-void system. This is mainly due to the presence of unburned carbon which absorbs the admixture. Consequently, higher doses of air-entraining admixture are required as either the fly ash content of the concrete increases or the carbon content of the fly ash increases. The carbon content of fly ash is usually measured indirectly by determining its loss-on-ignition (LOI). The increased demand for air entraining admixture should not present a significant problem to the concrete producer provided the carbon content of the fly ash does not vary significantly between deliveries. It has been shown that as the admixture dose required for a specific air content increases, the rate of air loss also increases.

2.5.1 (d) Setting Time

The impact of fly ash on the setting behavior of concrete is dependent not only on the composition and quantity of fly ash used, but also on the type and amount of cement, the water-to-cementitious materials ratio (w/cm), the type and amount of chemical admixtures, and the concrete temperature. During hot weather the amount of retardation due to fly ash tends to be small and is likely to be a benefit in many cases. During cold weather, the use of fly ash, especially at high levels of replacement, can lead to very significant delays in both the initial and final set. These delays may result in placement difficulties especially with regards to the timing of finishing operations for floor slabs and pavements or the provision of protection to prevent freezing of the plastic concrete. Practical considerations may require that the fly ash content is limited during cold-weather concreting. Higher-calcium fly ashes generally retard setting to a lesser degree than low-calcium fly ashes, probably because the hydraulic reactivity of fly ash increases with increasing calcium content.

2.5.1 (e) Heat of Hydration

The reduction in the rate of the heat produced and hence the internal temperature rise of the concrete has long been an incentive for using fly ash in mass concrete construction. In massive concrete pours where the rate of heat loss is small, the maximum temperature rise in fly ash concrete will primarily be a function of the amount and composition of the portland cement and fly ash used, together with the temperature of the concrete at the time of placing. Concrete with low portland cement contents and high fly ash contents are particularly suitable for minimizing autogenous temperature rises.

Most published work on the effects of fly ash on the rate of heat development and temperature rise in concrete have focused on low-calcium Class F fly ashes. Work by the Bureau of Reclamation indicated that the rate of heat development generally increases with the calcium content of the ash. Fly ashes high in calcium may produce little or no decrease in the heat of hydration (compared to plain portland cement) when used at normal replacement levels.

2.5.1 (f) Finishing and Curing

The use of fly ash can lead to significant retardation of the setting time, which means that finishing operations may have to be delayed. At normal temperatures, the rate of the pozzolanic reaction is slower than the rate of cement hydration, and fly ash concrete needs to be properly cured if the full benefits of its incorporation are to be realized. When high levels of fly ash are used it is generally recommended that the concrete is moist cured for a minimum period of 7 days. It has been recommended that the duration of curing be extended further (for example, to 14 days) where possible, or that a curing membrane be placed after 7 days of moist curing . If adequate curing cannot be provided in practice, the amount of fly ash used in the concrete should be limited.

CHAPTER – 3
TEST METHADOLOGY

3.1 REBOUND HAMMER

Before commencement of a test, the rebound hammer should be tested against the test anvil, to get reliable results. The testing anvil should be of steel having Brinell hardness number of about 5000 N/mm2. The supplier/manufacturer of the rebound hammer should indicate the range of readings on the anvil suitable for different types of rebound hammer.

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

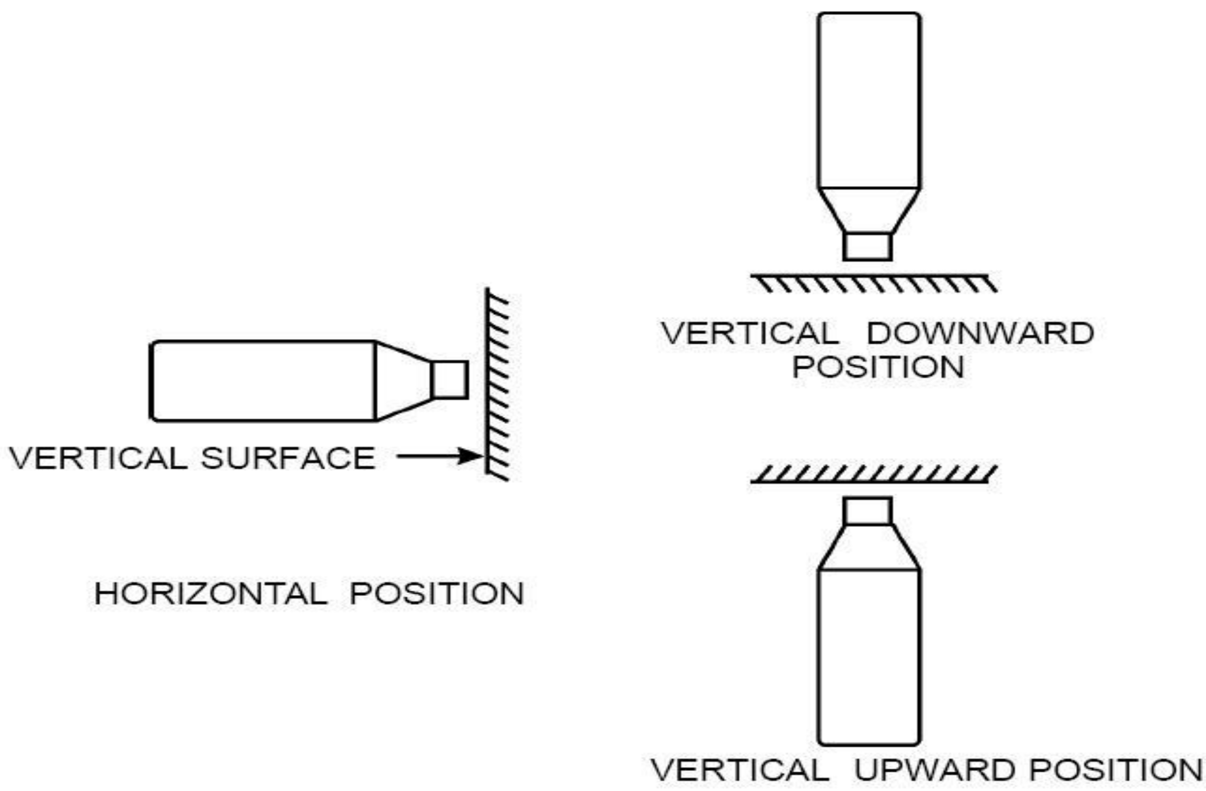


Fig 3.1 Various positions of Rebound Hammer

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

The following should be observed during testing –

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

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

(c) The test should not be conducted on rough surfaces resulting from incomplete compaction, loss of grout, spalled or tooled surfaces.

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

Procedure for obtaining correlation between Compressive Strength of Concrete and Rebound Number:

The most satisfactory way of establishing a correlation between compressive strength of concrete and its rebound number is to measure both the properties simultaneously on concrete cubes. The concrete cubes specimens are held in a compression testing machine under a fixed load, measurements of rebound number taken and then the compressive strength determined as per IS 516: 1959. The fixed load required is of the order of 7 N/mm2 when the impact energy of the hammer is about 2.2 Nm. The load should be increased for calibrating rebound hammers of greater impact energy and decreased for calibrating rebound hammers of lesser impact energy. The test specimens should be as large a mass as possible in order to minimize the size effect on the test result of a full scale structure. 150mm cube specimens are preferred for calibrating rebound hammers of lower impact energy (2.2Nm), whereas for rebound hammers of higher impact energy, for example 30 Nm, the test cubes should not be smaller than 300mm.

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

3.2 UNIVERSAL TESING MACHINE

METHOD OF TESTING

Initial Adjustment: - before testing adjust the pendulum with respect to capacity of the test i.e. 8 Tones; 10 Tones; 20 Tones; 40 Tones etc. For ex: - A specimen of 6 tones capacity gives more accurate result of 10 Tones capacity range instead of 20 Tones capacity range. These ranges of capacity are adjusted on the dial with the help of range selector knob.

Procedure for Performing Compression Test

Fix upper and lower pressure plates to the upper stationary head & lower table respectively. Place the specimen on the lower plate in order to grip. Then adjust zero by lifting the lower table select the proper job and complete upper and lower check adjustment. Apply some Greece to the tapered surface of specimen or groove. Then operate the upper cross head grip operation handle & grip the upper end of test specimen fully in to the groove. Keep the lower left valve in fully close position. Open the right valve & close it after lower table is slightly lifted. Adjust the lower points to zero with the help of adjusting knob. This is necessary to remove the dead weight of the lower table. Then lock the jobs in this position by operating job working handle. Then open the left control valve. The printer on dial gauge at which the specimen breaks slightly return back & corresponding load is known as breaking load & maximum load is known as the ultimate load.

CHAPTER – 4
OBJECTIVES OF THE PROJECT

The objectives of this project are:

1. Learn and identify different Structural Health Monitoring techniques:
   - Understanding SHM and its use
   - Studying different techniques used for SHM.
   - Performing different SHM techniques for better understanding.

2. Identifying one suitable technique based on feasibility which can be used to monitor health of concrete structures and comparing the results obtained with those obtained from destructive testing.
   - Identified NDE method to be adopted Rebound Hammer test.
   - Casting cubes to be tested of different grades and different curing period.
   - Performing tests destructive as well as non- destructive on specimens.
   - Using the data obtained to draw correlations between the two types of tests.

3. Draw correlation between destructive and non- destructive testing for normally cured, accelerated cured concrete samples and concrete samples formed by replacing 25% cement with fly ash and performing accelerated curing:
   - Casting cubes for different grades and keeping them in accelerated curing tank at 90 degree Celsius for 3 days.
   - Casting cubes by replacing 25% cement with flyash and performing accelerated curing.
   - Performing destructive and non-destructive tests on the samples.
   - Using the data obtained to draw correlations between different variables.

CHAPTER – 5

TEST RESULTS

5.1 Materials used:

Cement The test was conducted on Ambhuja Cement brand of OPC 43 grade. The cement test results obtained meets the standard specification; hence it is good for concrete works.

Coarse Aggregate: The aggregate crushing value (ACV) and aggregate impact value (AIV) of the aggregate used was 8.9% and 6.93% respectively. These values are below the maximum permissible values specified by IS 383 part III :1970 , the coarse aggregate used was good for concrete works.

Fine Aggregate: Fine aggregate used for the study has a specific gravity of 2.65 and a bulk density of specifies that fine aggregate should have an acceptable range between 2.4-2.7. This confirms that fine aggregate used is within the acceptable range and also good for concrete works.

5.2 Initial tests performed on materials:

5.2.1 Specific Gravity/Density Of Cement (IS:4031-Part 11-1988)

Specific gravity is defined as the ratio between the weight of a given volume of cement and weight of an equal volume of water.

Test Procedure:

i) Dry the Le-chatelier flask and fill with kerosene oil or Naptha to a point on the stem between and 1 ml.

ii) Dry the inside of the flask above the level of the liquid.

iii) Immerse the flask in a constant temp water bath maintained at room temp for sufficient time.

iv) Record the level of the kerosene oil in the flask as initial reading.

Introduce about 60 g of cement into the flask so that the level of kerosene rises to about say 22 ml mark. Splashing should be avoided and cement should not be allowed to adhere to the sides of the flask above the liquid.Insert the glass nipple into the flask and roll it gently in an inclined position to free the cement from air until no further air bubble rises to the surface of the liquid.Keep the flask again in constant temp water bath and note down the new liquid level as final reading.

Fig 5.1 Le chatelier flask

Calculation:

The difference between the first and final readings represents the volume of liquid displaced by the mass of cement used in test.The density is calculated as per the below mentioned formula to the second place of decimal.

$$Density = \frac{massofcement, g}{displacedvolume, cm^3}$$

Note: Two tests shall be carried out and the average is reported. If the difference between the two values differs by more than 0.03, the test shall be repeated.

Result: Specific Gravity of the cement 2.5

Precautions:

i) While pouring cement in the Lechatelier flask, care should be taken to avoid splashing and cement should not adhere to the inside of the flask above the liquid.

ii) The kerosene or Naptha should be completely free from water.

Technical Discussions

The result obtained by us is on the lower side of the limit specified by the code because of the following reasons:

i) Due to presence of two bulbs in the apparatus the cement got stuck in the first bulb and in order to push it down we had to use a broomstick to push the cement further as a result of which some material got stuck in the stick leading to false results.

ii) The cement was exposed to environment leading to lump formation inspite of sieving material before test it had hydrated to some extent because of the presence of moisture in the atmosphere.

Test Standard Reference: IS:4031(Part 11):1988-Methods of physical tests for hydraulic cement (Determination of density).

5.2.2 Soundness Of Cement By Le-Chateliers Method (IS:4031-Part 3-1988)

In the soundness test a specimen of hardened cement paste is boiled for fixed time so that any tendency to expand is speeded up and can be detected. Soundness means the ability to resist volume expansion.

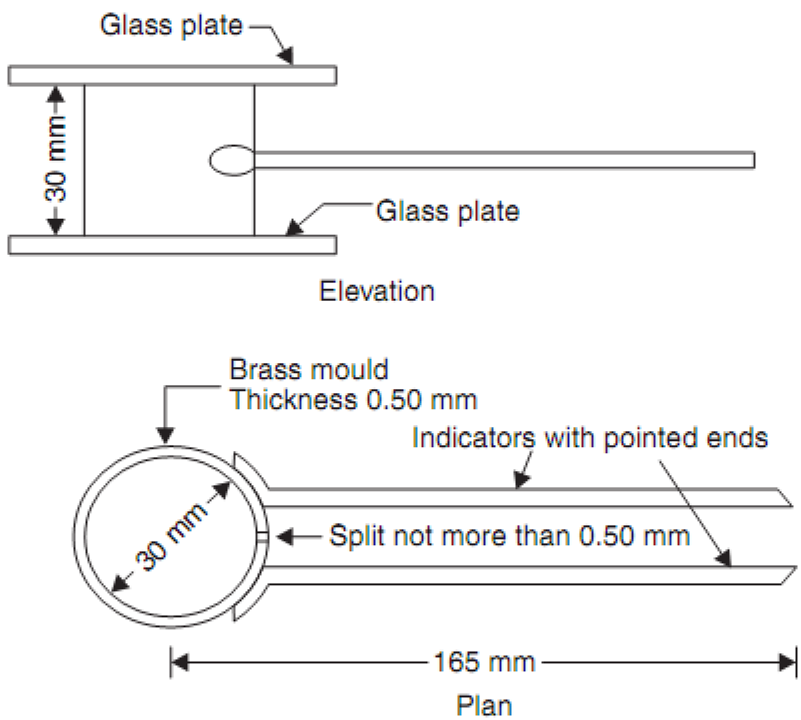


Fig 5.2 Le chatelier Apparatus

Environmental Conditions

Temperature	27 ± 2^0 C
Humidity	65 ± 5 %

Procedure

Before commencing setting time test, do the consistency test to obtain the water required to give the paste normal consistency (P).

Prepare a paste by adding 0.78 times the water required to give a paste of standard consistency (i.e. 0.78P).

i) Lightly oil the Le-chatelier mould and place it on a lightly oiled glass sheet.

ii) Fill the mould with the prepared cement paste. In the process of filling the mould keep the edge of the mould gently together.

iii) Cover the mould with another piece of lightly oiled glass sheet, place a small weight on this covering glass sheet.

iv) Submerge the whole assembly in water at a temperature of 27 ± 2^0 C and keep there for 24 hours.

v) Remove the whole assembly from water bath and measure the distance separating the indicator points to the nearest 0.5 mm (L_1).

vi) Again submerge the whole assembly in water bath and bring the temperature of water bath to boiling temperature in 25 to 30 minutes. Keep it at boiling temperature for a period of 3 hours.

vii) After completion of 3 hours, allow the temperature of the water bath to cool down to room temperature and remove the whole assembly from the water bath.

viii) Measure the distance between the two indicator points to the nearest 0.5 mm (L_2).

Calculations

Soundness/expansion of cement = L_1-L_2

L_1=Measurement taken after 24 hours of immersion in water at a temp. of 27 ± 2^0 C

L_2=Measurement taken after 3 hours of immersion in water at boiling temperature.

Calculate the mean of two values to the nearest 0.5 mm.

Note::In the event of cement failing to comply with the specified requirements, a further test should be made from another portion of the same sample in manner described above, but after aeration (done by spreading out to a depth of 75 mm at a relative humidity of 50 to 80% for a total period of 7 days).

Result: Soundness of the cement 6mm.

Precautions

i) All the measurements should be done accurately.

ii) Do not apply extra pressure while filling the moulds.

iii) During boiling water level should not fall below the height of the mould.

Type/Name of cement	**Reference Indian standard**	**Expansion (max.)**
OPC (33)	IS:269-1989	10 mm
OPC (43)	IS:8112-1989	10 mm
OPC (53)	IS:12269-1987	10 mm

Rapid hardening	IS:8041-1990	10 mm
Low heat cement	IS:12600-1989	10 mm
Super sulphated	IS:6909-1990	5 mm
Portland pozzolana	IS:1489-1991(part 1)	10 mm
PSC	IS:455-1976	10 mm
High alumina cement	IS:6452-1989	5 mm
SRC	IS:12330-1988	10 mm
Masonry cement	IS:3466-1988	10 mm
IRS-T-40	Railway standards	5 mm

Table 5.1 Standard Specifications for soundness of various cements

Technical Discussion

The test result obtained was on the lower side of the specified range because of the following reasons:

i) Presence of moisture in the atmosphere causing pre hydration of the cement sample.

ii) Low temperature.

Volume expansion in cement mortar or in cement concrete is caused by the presence of unburnt lime (CaO), dead burnt MgO and also $CaSO_4$.

By Le-chatelier method we can only find out presence of unburnt lime (CaO).

Presence of unburnt lime may develop cracks in the cement because of increase in volume.

Free lime (CaO) and magnesia (MgO) are known to react with water very slowly and increase in volume considerably, which result in cracking, distortion and disintegration.

Test Standard Reference: IS:4031(Part 3):1988-Methods of physical tests for hydraulic cement (Determination of soundness)

5.3.3 Initial & Final Setting Time (IS:4031-Part 5-1988)

Initial setting time is that time period between the time water is added to cement and time at which 1 mm square section needle fails to penetrate the cement paste, placed in the Vicat's mould 5 mm to 7 mm from the bottom of the mould.

Final setting time is that time period between the time water is added to cement and the time at which 1 mm needle makes an impression on the paste in the mould but 5 mm attachment does not make any impression.

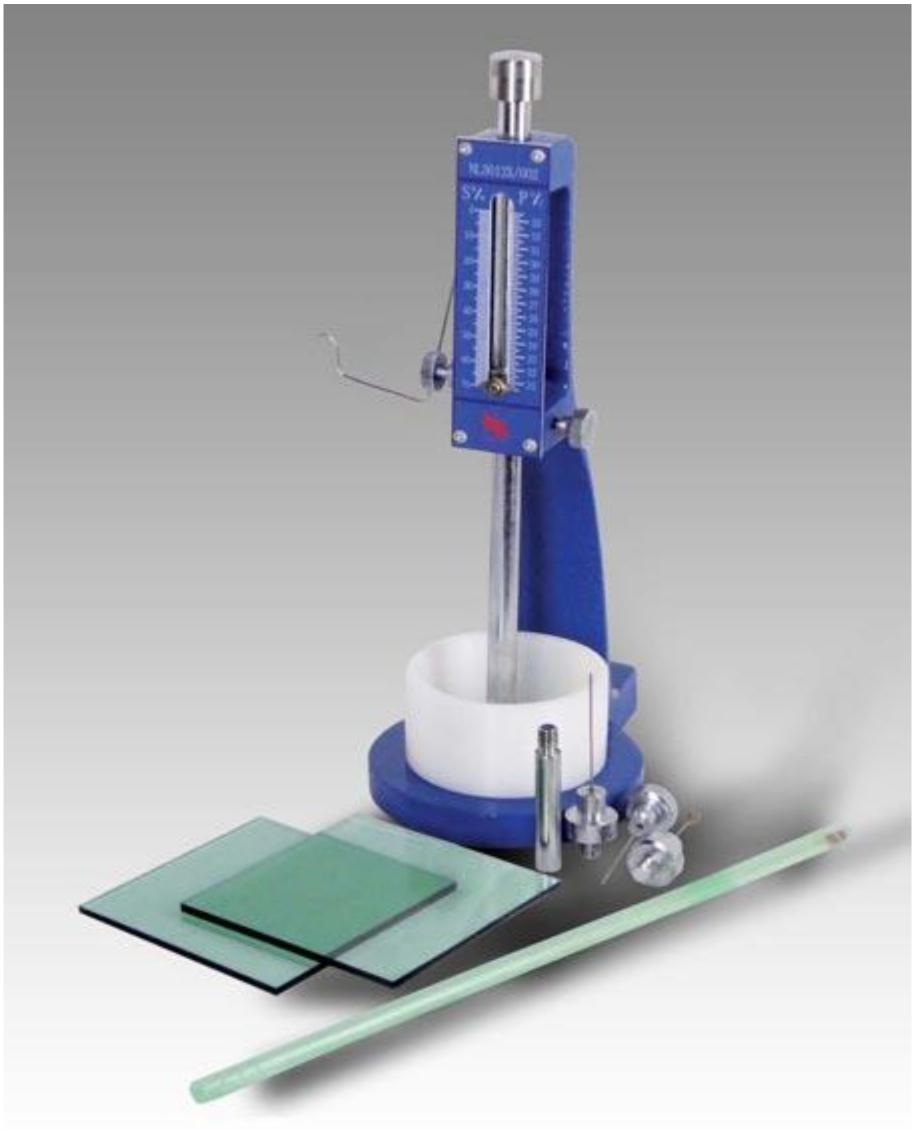


Fig 5.3 Vicat Apparatus

Procedure

(a)Test block preparation

i) Before commencing setting time test, do the consistency test to obtain the water required to give the paste normal consistency (P).

ii) Take 400 g of cement and prepare a neat cement paste with 0.85P of water by weight of cement.

iii) Gauge time is kept between 3 to 5 minutes. Start the stop watch at the instant when the water is added to the cement. Record this time (t_1).

iv) Fill the Vicat mould, resting on a glass plate, with the cement paste gauged as above. Fill the mould completely and smooth off the surface of the paste making it level with the top of the mould. The cement block thus prepared is called test block.

(b)Initial setting time

i) Place the test block confined in the mould and resting on the non-porous plate, under the rod bearing the needle.

ii) Lower the needle gently until it comes in contact with the surface of test block and quick release, allowing it to penetrate into the test block.

ii) In the beginning the needle completely pierces the test block. Repeat this procedure i.e. quickly releasing the needle after every 2 minutes till the needle fails to pierce the block for about 5 mm measured from the bottom of the mould. Note this time (t_2).

(c)Final setting time

i)For determining the final setting time, replace the needle of the Vicat's apparatus by the needle with an annular attachment.

ii) The cement is considered finally set when upon applying the final setting needle gently to the surface of the test block; the needle makes an impression thereon, while the attachment fails to do so. Record this time (t_3).

Calculation

Initial setting time=t_2-t_1

Final setting time=t_3-t_1,

Where,

t_1=Time at which water is first added to cement

t_2=Time when needle fails to penetrate 5 mm to 7 mm from bottom of the mould

t_3=Time when the needle makes an impression but the attachment fails to do so.

Result: IST 35 min FST 525 min.

Precautions

i) Release the initial and final setting time needles gently.

ii) The experiment should be performed away from vibration and other disturbances.

iii) Needle should be cleaned every time it is used.

iv) Position of the mould should be shifted slightly after each penetration to avoid penetration at the same place.

v) Test should be performed at the specified environmental conditions.

Type/Name Of Cement	Referenced Indian Stanadard	Initial Setting Time, mints (min.)	Final Setting Time, mints (max.)
OPC(33)	IS:269	30	600
OPC(43)	IS:8112	30	600

OPC(53)	IS:12269	30	600
SRC	IS:12330	30	600
PPC	IS:1489,P1	30	600
RHPC	IS:8041	30	600
PSC	IS:455	30	600
High alumina	IS:6452	30	600
Super sulphated	IS:6909	30	600
Low heat	IS:12600	60	600
Masonry cement	IS:3466	90	1440
IRS-T-40	Railway	60	600

Table no 5.2 Specifications of IST and FST for different type of cement.

Technical Discussion

i) It is essential that cement set neither too rapidly nor too slowly. In the first case there might be insufficient time to transport and place the concrete before it becomes too rigid. In the second case too long a setting period tends to slow up the work unduly, also it might postpone the actual use of the structure because of inadequate strength at the desired age.

ii) Setting should not be confused with hardening, which refers to the gain in mechanical strength after the certain degree of resistance to the penetration of a special attachment pressed into it.

iii) Setting time is the time required for stiffening of cement paste to a defined consistency.

iv) Indirectly related to the initial chemical reaction of cement with water to form aluminum-silicate compound.

v) Initial setting time is the time when the paste starts losing its plasticity.

vi) Initial setting time test is important for transportation, placing and compaction of cement concrete.

vii) Initial setting time duration is required to delay the process of hydration or hardening.

viii) Final setting time is the time when the paste completely loses its plasticity.

ix) It is the time taken for the cement paste or cement concrete to harden sufficiently and attain the shape of the mould in which it is cast.

x) Determination of final setting time period facilitates safe removal of scaffolding or form.

xi) During this period of time primary chemical reaction of cement with water is almost completed.

<u>Test Standard ReferenceIS:4031(Part 5):1988-Methods of physical tests for hydraulic cement (Determination of initial and final setting times)</u>

5.3.4 Specific Gravity & Water Absorption Of Aggregate (IS:2386-Part 3-1963)

Procedure

i) About 2kg of the aggregate sample is washed thoroughly to remove fines, drained and then placed in the wire basket and immersed in distilled water at a temperature between 22 to 32^0C with a cover of at least 50 mm of water above the top of the basket

ii) Immediately after the immersion the entrapped air is removed from the sample by lifting the basket containing it 25 mm above the base of the tank and allowing it to drop 25 times at the rate of about one drop per second. The basket and the aggregate should remain completely immersed in water for a period of 24±0.5 hours afterwards.

iii) The basket and the sample are then weighed while suspended in water at a temperature of 22 to 32^0C. The weight is noted while suspended in water (W_1) g.

iv) The basket and the aggregate are then removed from water and allowed to drain for a few minutes, after which the aggregates are transferred to one of the dry absorbent clothes.

v) The empty basket is then returned to the tank of water, jolted 25 times and weights in water (W_2) g.

vi) The aggregates placed in the dry absorbent clothes are surface dried till no further moisture could be removed by this clothe.

vii) Then the aggregate is transferred to the second dry cloth spread in a single layer, covered and allowed to dry for at least 10 minutes until the aggregates are completely surface dry. 10 to 60 minutes drying may be needed. The surface dried aggregate is then weighed W_3 g.

viii) The aggregate is placed in a shallow tray and kept in an oven maintained at a temperature of 110^0C for 24 hours. It is then removed from the oven, cooled in air tight container and weighed W_4 g.

Calculation

Weight of saturated aggregate suspended in water with basket = W_1 g

Weight of basket suspended in water = W_2 g

Weight of saturated aggregate in water = (W_1-W_2)g = W_s g

Weight of saturated surface dry aggregate in air = W_4 g

Weight of water equal to the volume of the aggregate = (W_3-W_s) g

$$\text{Specific gravity} = \frac{W_4}{W_3 - (W_1 - W_2)} \quad \text{Apparent sp.gravity} = \frac{W_4}{(W_4 - (W_1 - W_2))}$$

$$\text{Water absorption} = \frac{(W_3 - W_4)}{W_4} \times 100$$

Results: i) Specific Gravity of Fine Aggregate: 2.5

ii) Water Absorption ratio of fine aggregate: .5

iii) Specific Gravity of Coarse Aggregate: 2.6

iv)Water Absorption ratio of coarse aggregate: .6

Safety & Precautions

i) Use hand gloves while removing containers from oven after switching off the oven.

ii) Thoroughly clean & dry the container before testing.

iii) Special care should be taken that no outer air enters when using the balance.

iv) Use apron & safety shoes at the time of testing.

v) All parts of the equipment should always be kept clean.

Recommended Value:

The size of the aggregate and whether it has been artificially heated should be indicated. ISI specifies three methods of testing for the determination of the specific gravity of aggregates, according to the size of the aggregates. The three size ranges used are aggregates larger than 10 mm, 40 mm and smaller than 10 mm. The specific gravity of aggregates normally used in road construction ranges from about 2.5 to 3.0 with an average of about 2.68. Though high specific gravity is considered as an indication of high strength, it is not possible to judge the suitability of a sample road aggregate without finding the mechanical properties such as aggregate crushing, impact and abrasion values. Water absorption shall not be more than 0.6 per unit by weight.

Reference Standard IS : 2386 (Part 3) – 1963 – Method of test for aggregates for concrete (Part I) Particle size and shape.

5.3.5Workability Of Fresh Concrete Using Slump Cone Methods (IS:1199-1959)

For determination of consistency of concrete where the nominal maximum size of aggregate does not exceed 38 mm using slump test apparatus.

Fig 5.4Slump Cone Test Apparatus

Procedure

i) The internal surface of the mould is thoroughly cleaned by grease/oil and freed from superfluous moisture before commencing the test.

ii) The mould is placed on a smooth, horizontally leveled rigid and non-absorbent surface such as a plate.

iii)The mould is filled by concrete in four layers, each approximately one-quarter of height of the mould and each layer is tamp with 25 strokes by tamping rod of dia 16mm &length 600mm with pointed end in a uniform manner.

iv) After the top layer tamp, the concrete is struck off level with a trowel and any mortar leaked out between the mould and base plate is to be cleaned away.

v) The mould is removed from the concrete immediately by raising it slowly and carefully in a vertical direction.

Calculation

The slump is measured immediately by determining the difference between the height of the mould and that of the highest point of specimen.

Safety & Precautions

i) Use hand gloves & shoes while testing.

ii) Equipment should be cleaned thoroughly before testing & after testing.

iii) Use apron & goggles at the time of testing.

iv) The apparatus should remain free from vibrations during the test.

v) Petroleum jelly should be applied to the mould.

<u>Reference Standards IS : 1199 – 1959 – Method of sampling and analysis of concrete</u>

<u>5.3.6 Consistency Of Standard Cement Paste (IS:4031-Part4-1988)</u>

Standard consistency of a cement paste is defined as that consistency which will permit a vicat plunger having 10 mm dia and 50 mm length to penetrate to a depth of 33-35 mm from top of the mould.

Fig 5.5 Vicats Apparatus

Procedure

i) Take 400 g of cement and place it in the enameled tray.

ii) Mix about 25% water by weight of dry cement thoroughly to get a cement paste. Total time taken to obtain thoroughly mixed water cement paste i.e. “Gauging time” should not be more than 3 to 5 minutes.

iii) Fill the vicat mould, resting upon a glass plate, with this cement paste.

iv) After filling the mould completely, smoothen the surface of the paste, making it level with top of the mould.

.

v) Place the whole assembly (i.e. mould + cement paste + glass plate) under the rod bearing plunger

vi) Lower the plunger gently so as to touch the surface of the test block and quickly release the plunger allowing it to sink into the paste.

vii) Measure the depth of penetration and record it.

viii) Prepare trial pastes with varying percentages of water content and follow the steps (2 to 7) as described above, until the depth of penetration becomes 33 to 35 mm.

Calculation

Calculate percentage of water (P) by weight of dry cement required to prepare cement paste of standard consistency by following formula, and express it to the first place of decimal.

$$P = \frac{W}{C} \times 100$$

Where,

W=Quantity of water added

C=Quantity of cement used

Precautions

i) Gauging time should be strictly observed

ii) Room temperature should be well maintained as per test requirement.

iii) All apparatus used should be clean.

iv) The experiment should be performed away from vibrations and other disturbances.

Technical Discussion

i) This test helps to determine water content for other tests like initial and final setting time, soundness & compressive strength.

NAME OF TEST	AMOUNT OF WATER REQUIRED
Soundness(Le-chatelier method)	0.78 P (P=Consistency of standard cement paste)
Setting time	0.85 P (P=Consistency of standard cement paste)
Compressive strength	$(\frac{P}{4}+3)\%$ of combined mass of cement and sand.

Table 5.3 Tests dependent on consistency of cement

ii) Consistency refers to the relative mobility of a freshly mixed cement paste or mortar or its ability to flow. For a mortar the standard consistency is measured by flow table test.

iii) Generally the normal consistency for OPC ranges from 26 to 33%.

Test Standard Reference IS:4031(Part 4):1988-Methods of physical tests for hydraulic cement (Determination of consistency of standard cement paste)

Test (Unit)	Result	Standard Code Used	Limit Specified by Code
Specific Gravity of Cement)g/cc)	2.5	IS 4031(part-II): 1988	3.15
Soundness of Cement (mm)	6	IS 4031(Part-3): 1988	10
IST (mins)	35	IS 4031(part-5): 1988	30 (min)
FST(mins)	525	IS 4031(part-5): 1988	600
Specific Gravity of Fine Aggregate(g/cc)	2.5	IS 2386(part 3): 1963	2.74
Specific Gravity of Coarse Aggregate (g/cc)	2.6	IS 2386(part 3): 1963	2.74
Water Absorption of coarse aggregate	.5	IS 2386(part 3): 1963	.6
Water Absorption of fine aggregate	.6	IS 2386(part 3): 1963	.6
Slump Value	95mm	IS 1199:1959	75mm-120mm
Consistency	30%	IS:4031(Part 4):1988	27%-36%

Table 5.4 Results of the initial tests performed on the materials.

5.3 Rebound Hammer and UTM Results:

Preparation of Specimen: 12 cubes of each grade of concrete were cast, targeting at different mean strength. Further the cubes were cured for different no of days to ensure availability of wide range of compressive strength attained by these cubes. Size of each cube was 150x150x150mm.

Testing Of Specimen:

i) 10 readings (rebound number) were obtained for each cube, at different location on the surface of the specimen.

ii) The cubes were then given a load of 7 N/mm^2 (as specified by the IS CODE 13311) in the Compression Testing Machine and the Rebound Values were obtained.

iii) The cubes were then loaded up to their ultimate stress and the Breaking Load was obtained.

The following tables lists the Rebound numbers (rebound index), Mean Rebound Value, the Predicted Compressive Strength as predicted by the Rebound Hammer and the actual Compressive Strength as obtained by the Compression Testing Machine.

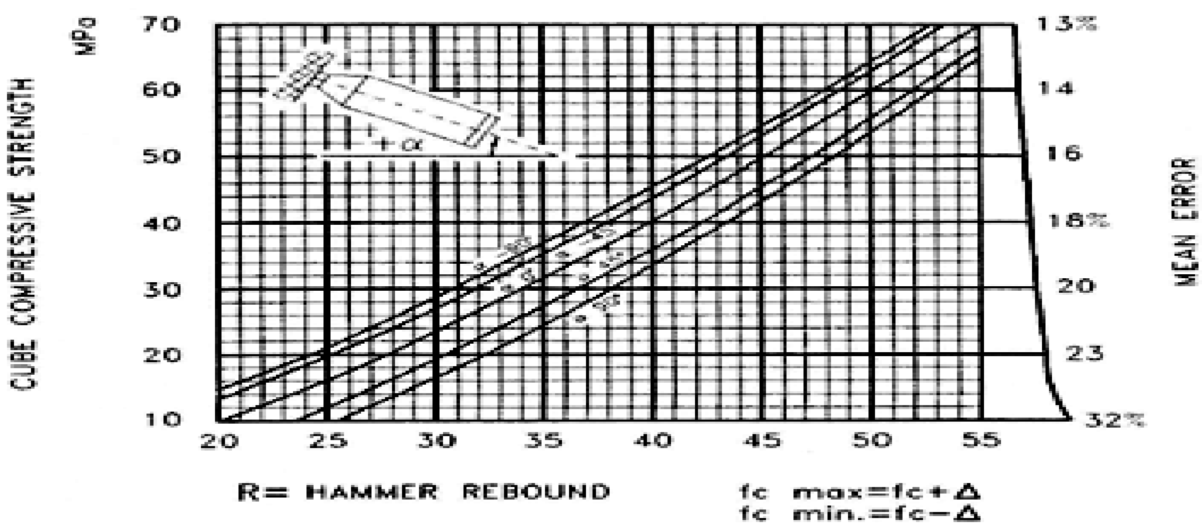


Fig 5.6 Rebound Number vs Compressive strength graph

TEST RESULTS FOR NORMALLY CURED CONCRETE SAMPLES

Age (in days)	7 DAYS			14 DAYS			21 DAYS			28DAYS		
Sample no.	S1	S2	S3	S1	S2	S3	S1	S2	S3	S1	S2	S3
Avg. RB	12	14	13	16	18	15	20	22	22	26	28	25
CS (NDT) (N/mm^2)	CBD	CBD	CBD	CBD	CBD	CBD	11.5	16	16	22	23	21
Cracking load from UTM(kN)	340	320	330	360	350	355	408	410	420	450	440	460
CS (DT) (N/mm^2)	15.1	14.2	14.7	16	15.5	15.7	18	18.2	18.7	20	19.6	20.44

Table 5.5 Compressive strength for M15 grade concrete

M20		M25		M30	
Compressive Strength	Rebound Number	Compressive Strength	Rebound Number	Compressive Strength	Rebound Number
Avg RN= 13	12 11 11 13 13 14 13 13 12 13	Avg RN= 15	15 14 13 15 16 12 16 17 15 12	Avg RN= 16	17 14 16 17 16 17 16 17 15 16

Table 5.6 Compressive Strength and Rebound number for concrete cured for 7 Days

	M20(N/mm^2)			M25(N/mm^2)			M30(N/mm^2)		
Sample number	S1	S2	S3	S1	S2	S3	S1	S2	S3
Compressive Strength	9	10	8.5	12.5	13	11	12	15	13

Table 5.7 Compressive strength from UTM for different concrete cured for 7days

M20		M25		M30	
Compressive Strength	Rebound Number	Compressive Strength	Rebound Number	Compressive Strength	Rebound Number
Avg RN= 16	18 15 14 16 15 14 15 16 17 14	Avg RN= 18	20 16 18 14 15 20 16 18 22 18	Avg RN= 20	18 22 19 18 26 21 20 21 22 18

Table 5.8 Compressive Strength and Rebound number for concrete cured for 14 Days

Grade	M20(N/mm^2)			M25(N/mm^2)			M30(N/mm^2)		
Sample number	S1	S2	S3	S1	S2	S3	S1	S2	S3
Compressive Strength	12	12.5	11	15	16	15.5	18	20	17

Table 5.9 Compressive strength from UTM for different concrete cured for 14days

M20		M25		M30	
Compressive Strength	Rebound Number	Compressive Strength	Rebound Number	Compressive Strength	Rebound Number
Avg RN=20	18	Avg RN= 23	22	Avg RN= 36	36
	20		24		34
	16		26		32
	22		24		40
	20		22		32
	24		24		34
	20		20		36
	24		26		40
	18		20		42
	20		22		35

Table 5.10 Compressive strength and Rebound Number for different concrete cured for 21days

Grade	M20 (N/mm^2)			M25 (N/mm^2)			M30 (N/mm^2)		
Sample	S1	S2	S3	S1	S2	S3	S1	S2	S3
Compressive strength	15	16	15.5	19	20	18.5	23	22.5	21

Table 5.11 Compressive strength and Rebound Number for different concrete cured for 21days

M20		M25		M30	
Compressive Strength	Rebound Number	Compressive Strength	Rebound Number	Compressive Strength	Rebound Number
Avg RN=26	27 22 22 28 26 24 34 29 25 23	Avg RN= 34	30 35 32 28 38 40 32 36 36 32	Avg RN= 45	44 46 41 42 46 47 50 39 45 49

Table 5.12 Compressive strength and Rebound Number for different concrete cured for 28 days

Grade	M20(N/mm^2)			M25(N/mm^2)			M30(N/mm^2)		
Samples	S1	S2	S3	S1	S2	S3	S1	S2	S3
Compressive Strenth	18	20	16	22.5	24	20	27	28	25

Table 5.13 Compressive strength and Rebound Number for different concrete cured for 28days

TEST RESULTS FOR ACCELERATED CURING CUBES

Preparation O f Specimen:- Cast two cubes for each grade of concrete for M15,M20,M25 and M30 and also cast two cubes each for same grades by replacing 8% of cement by Ultra Fine Slag. Place all the cubes in accelerated curing tank for steamed curing for three days. Size of each cube was 150x150x150mm.

Testing Of Specimen:

i) 10 readings (rebound number) were obtained for each cube, at different location on the surface of the specimen.

ii) The cubes were then given a load of 7 N/mm^2 (as specified by the IS CODE 13311) in the Compression Testing Machine and the Rebound Values were obtained.

iii) The cubes were then loaded up to their ultimate stress and the Breaking Load was obtained.

The following tables lists the Rebound numbers (rebound index), Mean Rebound Value, the Predicted Compressive Strength as predicted by the Rebound Hammer and the actual Compressive Strength as obtained by the Compression Testing Machine.

GRADE	M15		M20		M25		M30	
SAMPLE	S1	S2	S1	S2	S1	S2	S1	S2
COMPRESSIVE STRENGTH	24.8	26.2	22.67	26.67	33.55	31.78	26	24.4

Table 5.14 Compressive strength from destructive test.

GRADE	M15		M20		M25		M30	
SAMPLE	S1	S2	S1	S2	S1	S2	S1	S2
COMPRESSIVE STRENGTH	29.9	25.9	27.4	30.22	28.89	24.4	29.4	31.5

Table 5.15 Compressive strength from non destructive test.

M15 (sample 1)		M15 (sample 2)		M15-A (sample 1)		M15-A (sample 2)	
Compressive strength	Rebound number	Compressive strength	Rebound number	Compressive strength	Rebound number	Compressive strength	Rebound number
	31		32		29		28
	31		38		28		0
	30		32		31		29
	29		30		29		28
Avg	32	Avg	30	Avg	30	Avg	29
RN=29	30	RN=28.76	28	RN=26.87	30	RN=30	30
21Mpa	31	21Mpa	31	20Mpa	30	23Mpa	30
	30		32		30		28
	20		28		28		27
	28		29		29		29
	28		29		31		31

Table 5.16 compressive strength and rebound number from non-destructive test

M20 (sample 1)		M20 (sample 2)		M20-A (sample 1)		M20-A (sample 2)	
Compressive strength	Rebound number	Compressive strength	Rebound number	Compressive strength	Rebound number	Compressive strength	Rebound number
Avg RN=28.4 21.5Mpa	20 20 30 25 40 29 31 30 30 29	Avg RN=23.85 13.5Mpa	30 25 23 10 21 29 24 23 30 28	Avg RN=30.6 25.5Mpa	28 30 28 44 34 30 34 30 20 30	Avg RN=29.2 23.5Mpa	30 30 28 28 26 20 20 28 24 38

Table 5.17 compressive strength and rebound number from non-destructive test

M25 (sample 1)		M25 (sample 2)		M25-A (sample 1)		M25-A (sample 2)	
Compressive strength	Rebound number	Compressive strength	Rebound number	Compressive strength	Rebound number	Compressive strength	Rebound number
Avg RN=28 21.5Mpa	16 28 36 16 30 16 26 32 36 32 32 36	Avg RN=27.3 20Mpa	16 26 26 30 36 20 28 30 32 20 32 32	Avg RN=24.9 15.5Mpa	26 24 24 20 12 26 28 26 25 20 30 38	Avg RN=29.9 14Mpa	28 16 16 20 20 26 26 16 12 30 36 32

Table 5.18 compressive strength and rebound number from non-destructive test

M30 (sample 1)		M30 (sample 2)		M30-A (sample 1)		M30-A (sample 2)	
Compressive strength	Rebound number	Compressive strength	Rebound number	Compressive strength	Rebound number	Compressive strength	Rebound number
	25		14		27		14
	30		22		24		24
	16		26		15		23
	32		14		23		15
Avg	28	Avg	14	Avg RN=27	30	Avg	16
RN=25.9	14	RN=21.7	16	25.5Mpa	36	RN=25.6	16
18Mpa	30	11Mpa	16		34	24Mpa	20
	28		16		28		32
	30		28		16		26
	26		30		12		14
	30		26		10		32
	30		18		26		22

Table 5.19 compressive strength and rebound number from non-destructive test

CHAPTER - 6
DISSCUSSIONS

The Schmidt hammer provides an inexpensive, simple and quick method of obtaining an indication of concrete strength. The results are affected by factors such as smoothness of surface, size and shape of specimen, moisture condition of the concrete, type of cement and coarse aggregate, and extent of carbonation of surface.

The universal testing machine is gives the ultimate cracking load which is the concrete can bear before going to fail and the displacement of material. And then finally from the ultimate load we find out the compressive strength of that sample. This is very accurate method to find out the compressive strength of a sample.

The method presented is simple, quick, reliable, and covers wide ranges of concrete strengths. The method can be easily applied to concrete specimens as well as existing concrete structures. The final results were compared with previous ones from literature and also with actual results obtained from samples extracted from existing structures.

The results obtained by the rebound hammer test on 7 and 14 days age samples yielded the rebound no below 20 and the graph on the rebound hammer does not give compressive strength values for rebound number below 20 hence we were not able to draw graphs for 7 and 14 day age samples as we could only obtain the compressive strength values from UTM which alone could not help us establish any relationship.

The values for 21 and 28 days age concrete were successfully obtained and a relationship between the two compressive strengths was established.

For M15 grade the values obtained for M15 gave unexpected results because of the following reasons:

(i) The samples were prepared by taking w/c ratio as .44 (+2% to account for the various losses during the mixing process). The mixing was done in concrete mixer which already had some amount of dry mortar stuck to its surface, after dry mixing when water was poured inside the mixer some water was absorbed by the dry mortar in the mixer.

(ii) As a result of which the cement content in the mix was increased while the water content reduced which led to the false results. So Latin's formation and honey combing occurred hence at the time of placing there was uneven surface which further led to very low values of compressive strength.

The graph obtained gives a relationship between compressive strengths obtained from destructive and non-destructive tests. With the help of this graph we can find out the crushing strength which is obtained from UTM in the laboratories by comparing it with the rebound hammer test results obtained on site without actually performing destructive test on specimens at the construction site.

It is not possible to always perform the destructive tests on site and further it is a very cumbersome task and involves a lot of destruction which leads to large wastage of materials. Now by only performing non-destructive rebound hammer test we can get the value of the crushing strength from the graph hence it is very useful to reduce effort and time.

Also we established a relation between the age of concrete and its compressive strength which clearly was in accordance with the fact that the compressive strength of a concrete increases logarithmically and after a certain time reaches the maximum value on 100% hydration and on further increase in time the value of strength remains constant beyond a certain time period.

In the second phase of the project we wanted to further our study hence we decided to include factors like effect of curing type and adding of admixtures on the strength of concrete and further drawing correlations between the two strengths obtained by NDE and DE.

Internal concrete temperature is the most important factor affecting early compressive strength of concrete. Temperature is critical to meeting the dual concerns of higher early strength or reduced curing time.

High early concrete strengths are most efficiently produced by increasing the internal temperature of the concrete while maintaining high moisture content in the curing environment. Heating reduces the relative humidity of the air surrounding the concrete. Thus, moisture must be added to the heated air to maintain the same relative humidity of the air.

At heightened temperatures, the hydration process moves more rapidly and the formation of the Calcium Silicate Hydrate crystals is more rapid. The formation of the gel and colloid is more rapid and the rate of diffusion of the gel is also higher. However, the reaction being more rapid leaves lesser time for the hydration products to arrange suitably, hence the later age strength or the final compressive strength attained is lower in comparison to normally cured concrete. This has been termed as the crossover effect.

Accelerated curing techniques invariably involve high temperatures. This may induce thermal stresses in the concrete. Further, the water in the pores starts to exert pressure at higher temperatures. The combined effect of the pore pressure and thermal stresses causes a tensile stress within the body of the concrete. If the accelerated curing process is begun immediately after the concrete has been poured, then the concrete will not be able to withstand the tensile stresses as it requires time to gain some strength.

Moreover, these microcracks formed may then lead to the delayed formation of ettringite, which is formed by the transformation of metastable monosulfate.

Delayed ettringite formation (DEF) induces expansion in the concrete thereby weakening it. DEF is promoted by the formation of the cracks which enables the easy entry of water. Therefore, a delay period is allowed to elapse before the commencement of the curing process to allow the concrete to gain a certain minimum tensile strength.

The setting time of the concrete is an important criterion to determine the delay period. Generally, the delay period is equal to the initial setting time which has been found to give satisfactory results. Lesser delay periods result in compressive strength losses. In case of M15, M20, M25 a delay period of 3 and 2 days was provided but for M30 the curing was done after removing the cube from the casing after 24 hours thus explaining less compressive strength in case of M30.

Fly ash increases the later age strength of concrete as it reacts with calcium hydroxide and turns it into calcium-silicate-hydrates (C-S-H). However fly ash replaced cement have higher activation energy and therefore, their rate of hydration is lower as compared to ordinary Portland cement (OPC). This results in lower early age strength as compared to OPC. Accelerated curing techniques radically help to increase the rate of strength gain and hence the samples in which fly ash had been added have shown high compressive strength in case of accelerated curing.

Properly cured concrete made with fly ash creates a denser product because the size of the pores is reduced. This increases strength and reduces permeability. Which was observed from the results.

Since fly ash particles are spherical and in the same size range as portland cement, a reduction in the amount of water needed for mixing and placing concrete can be obtained. The use of fly ash can result in better workability, pumpability, cohesiveness, finish, ultimate strength, and durability.

The fine particles in fly ash help to reduce bleeding and segregation and improve pumpability and finishing, especially in lean mixes which can be seen in M15 and M20 samples. As grade increased i.e. in case of M25 the strength declined.

Strength in concrete depends on many factors, the most important of which is the ratio of water to cement. Good quality fly ash generally improves workability or at least produces the same workability with less water. The reduction in water leads to improved strength. And hence even on reducing the water cement ratio from .5 in case of normal concrete to .45 in case of additive concrete the strength achieved was more.

CHAPTER – 7

CONCLUSIONS

Structural health monitoring is an evolving field in civil engineering. It is very important to study the health of an existing structure in a country like India with rich cultural heritage and monuments. In an effort to study SHM we analysed different techniques used in SHM. Non-Destructive Evaluation and Wireless sensors are the most commonly used methods however due to non feasibility of resources we decided to analyse one NDE Rebound Hammer test and link it with DE i.e. Destructive analysis and draw suitable correlations between the two while inducing other factors such as age of concrete, type of curing , grade of concrete and admixtures added.
A total of 72 cubes were cast and correlations drawn by curve fitting method.

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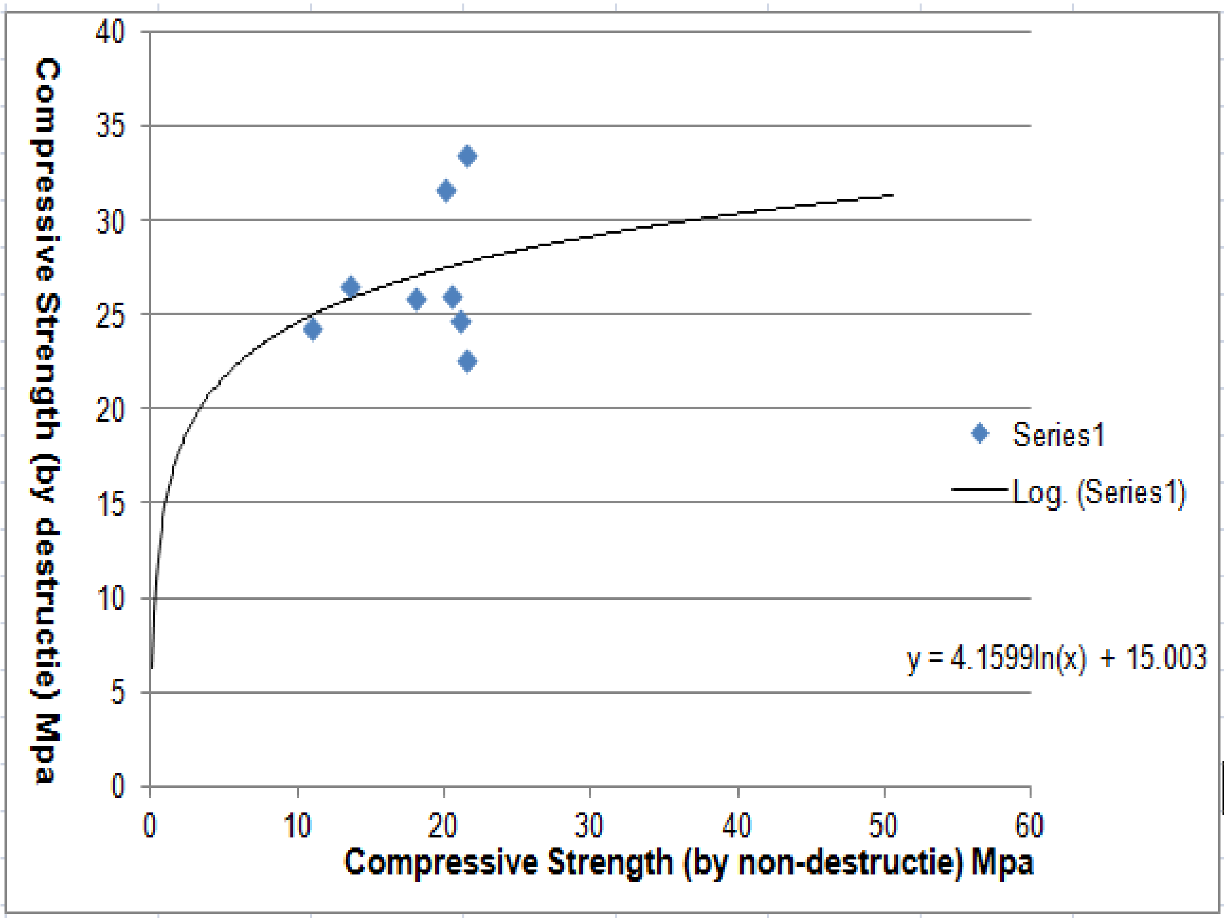
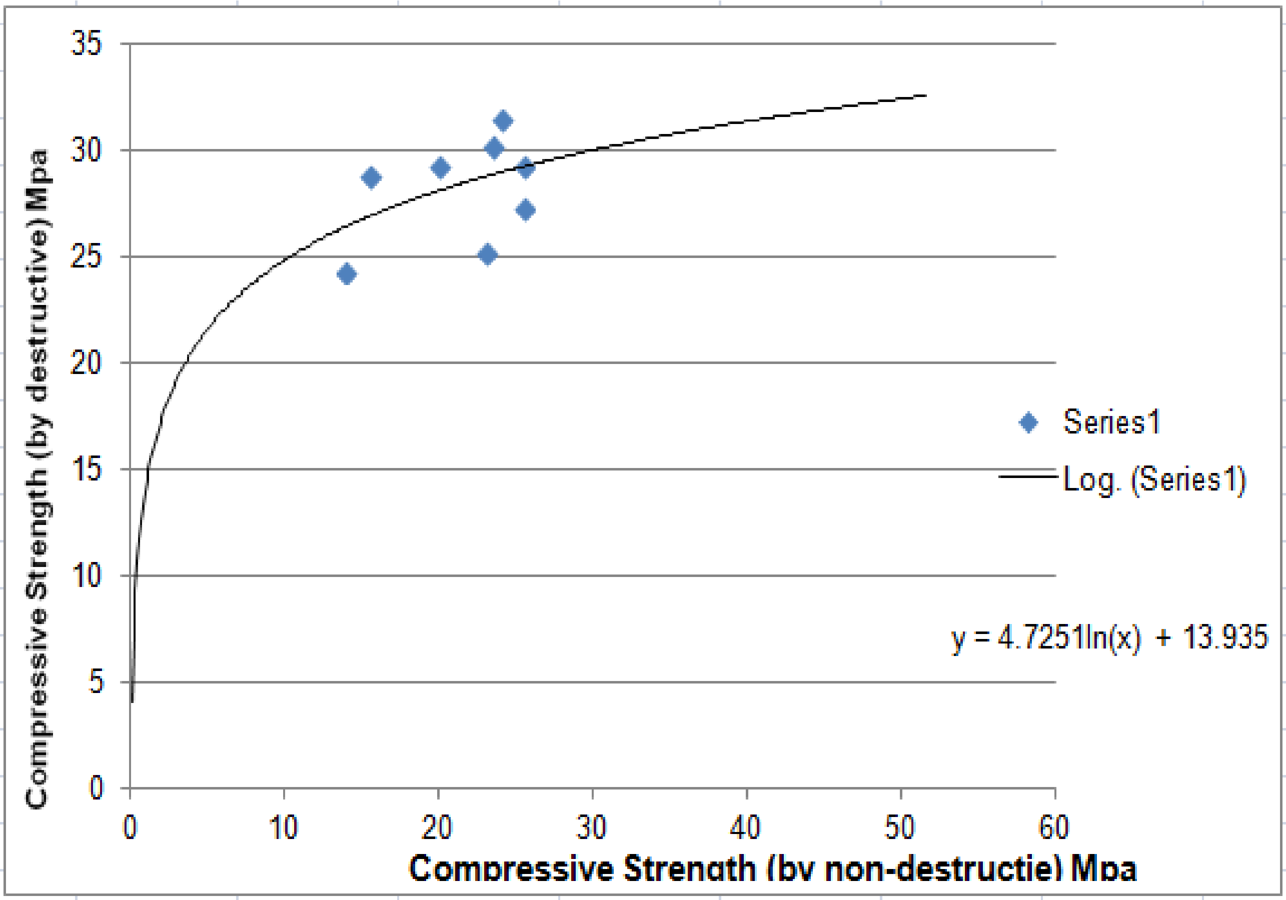
ANNEXURE

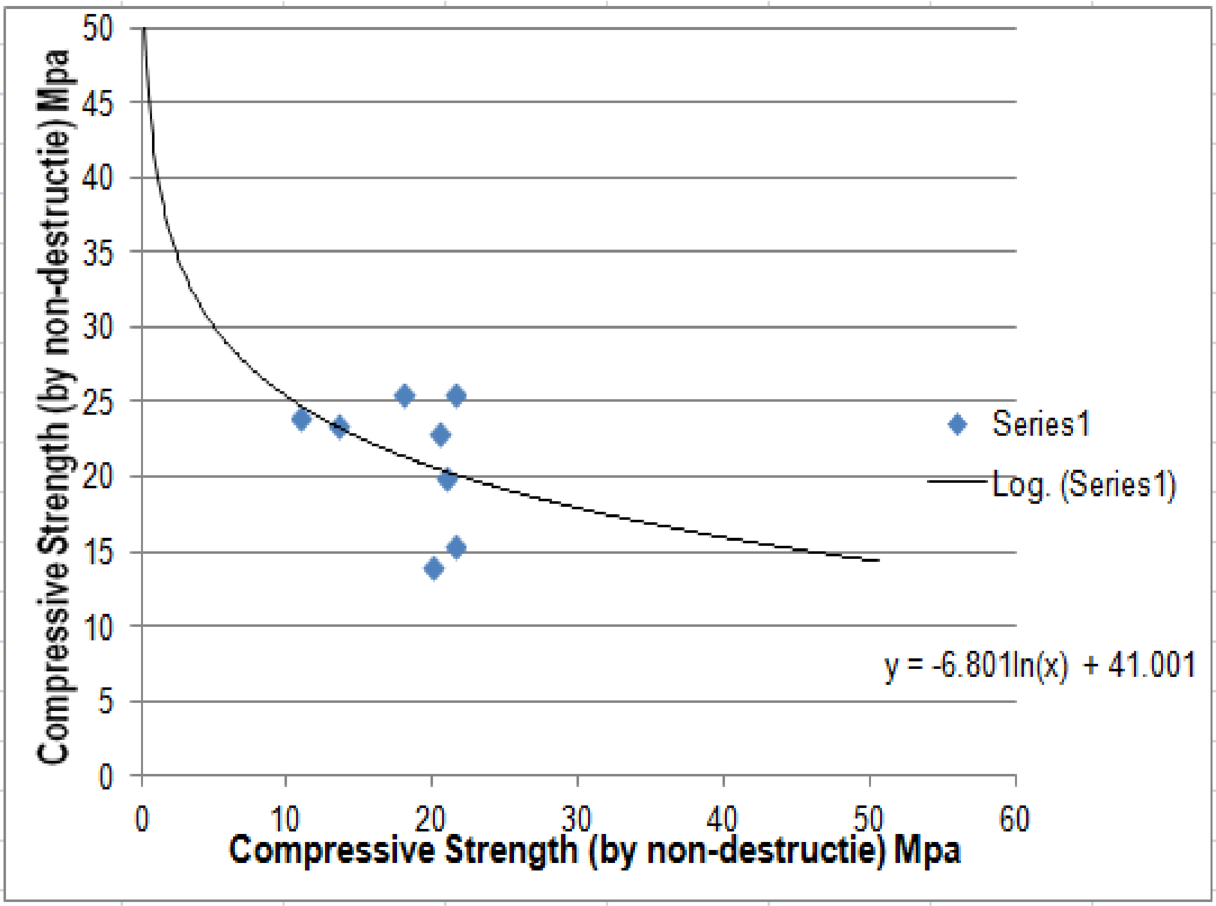
1. Correlation between destructive and non-destructive compressive strength for normal samples with accelerated curing



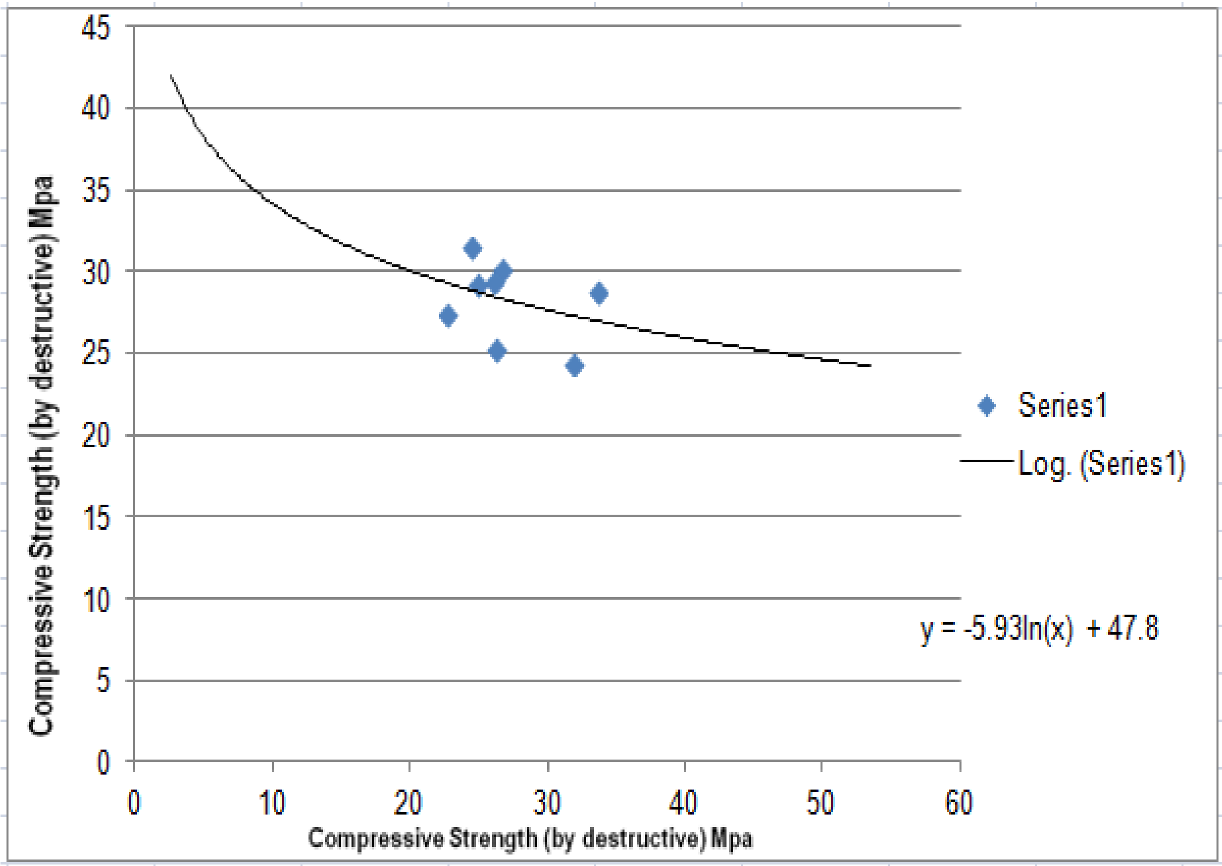
2. Correlation between destructive and non-destructive compressive strength for samples with additives accelerated curing



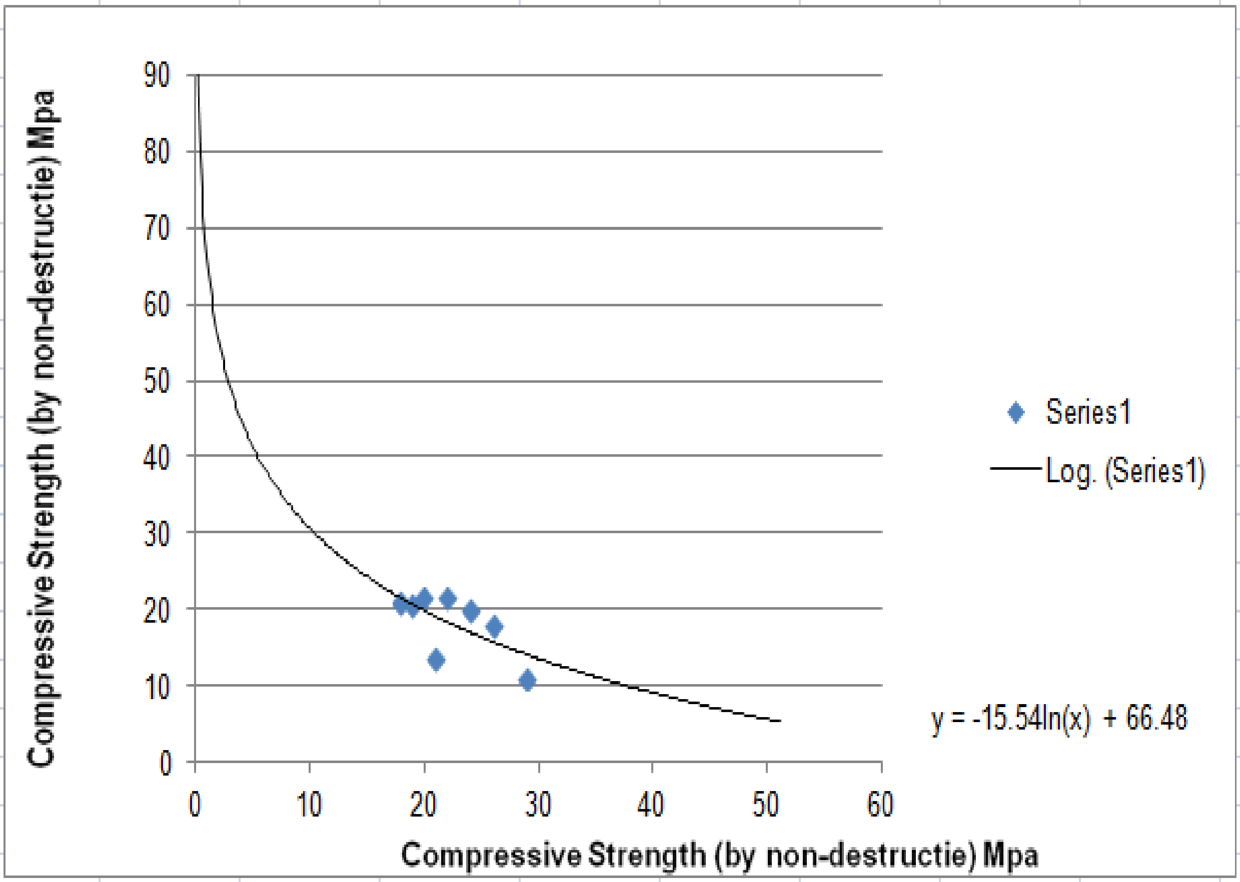
3. Correlation between Non-destructive compressive strength for normal accelerated and with additives accelerated cubes



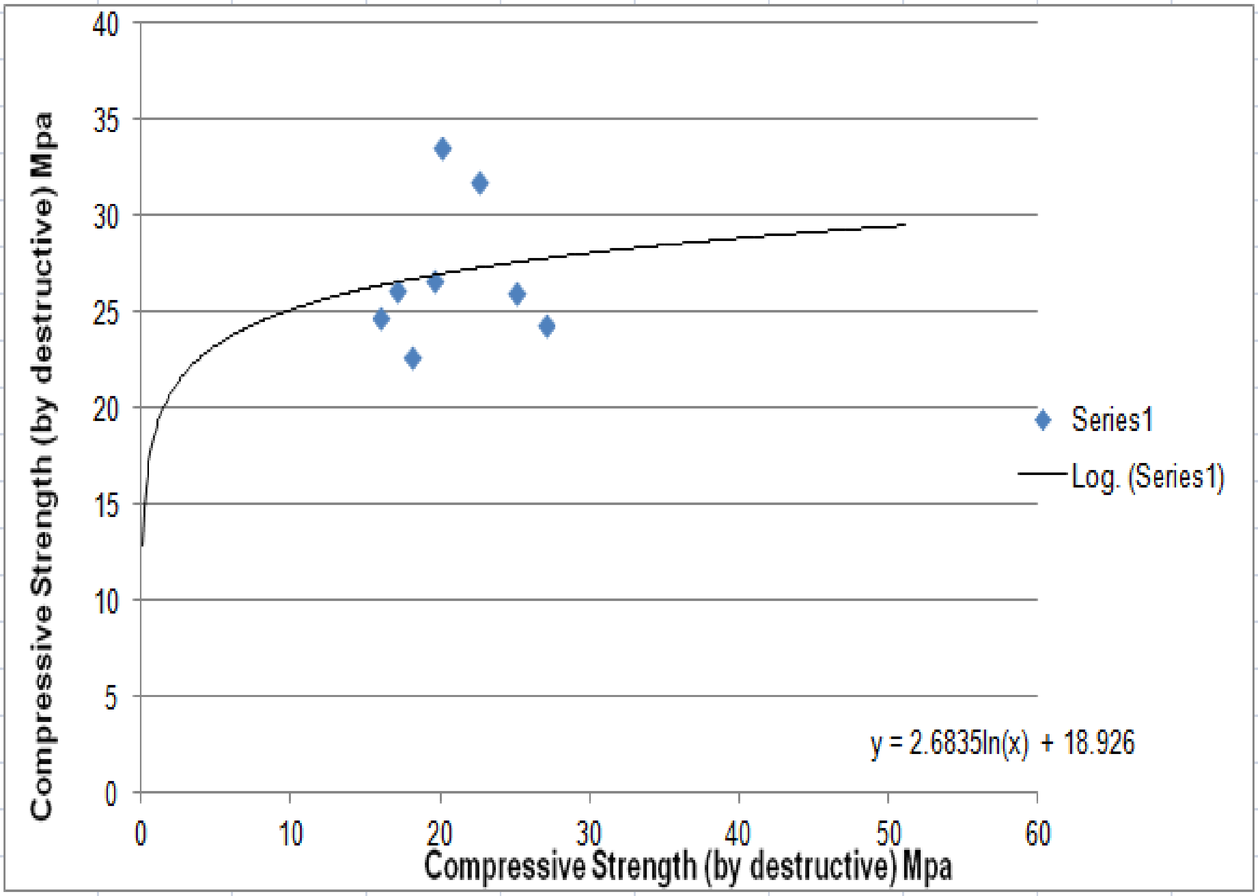
4. **Correlation between destructive compressive strength for normal accelerated and with additives accelerated cubes**



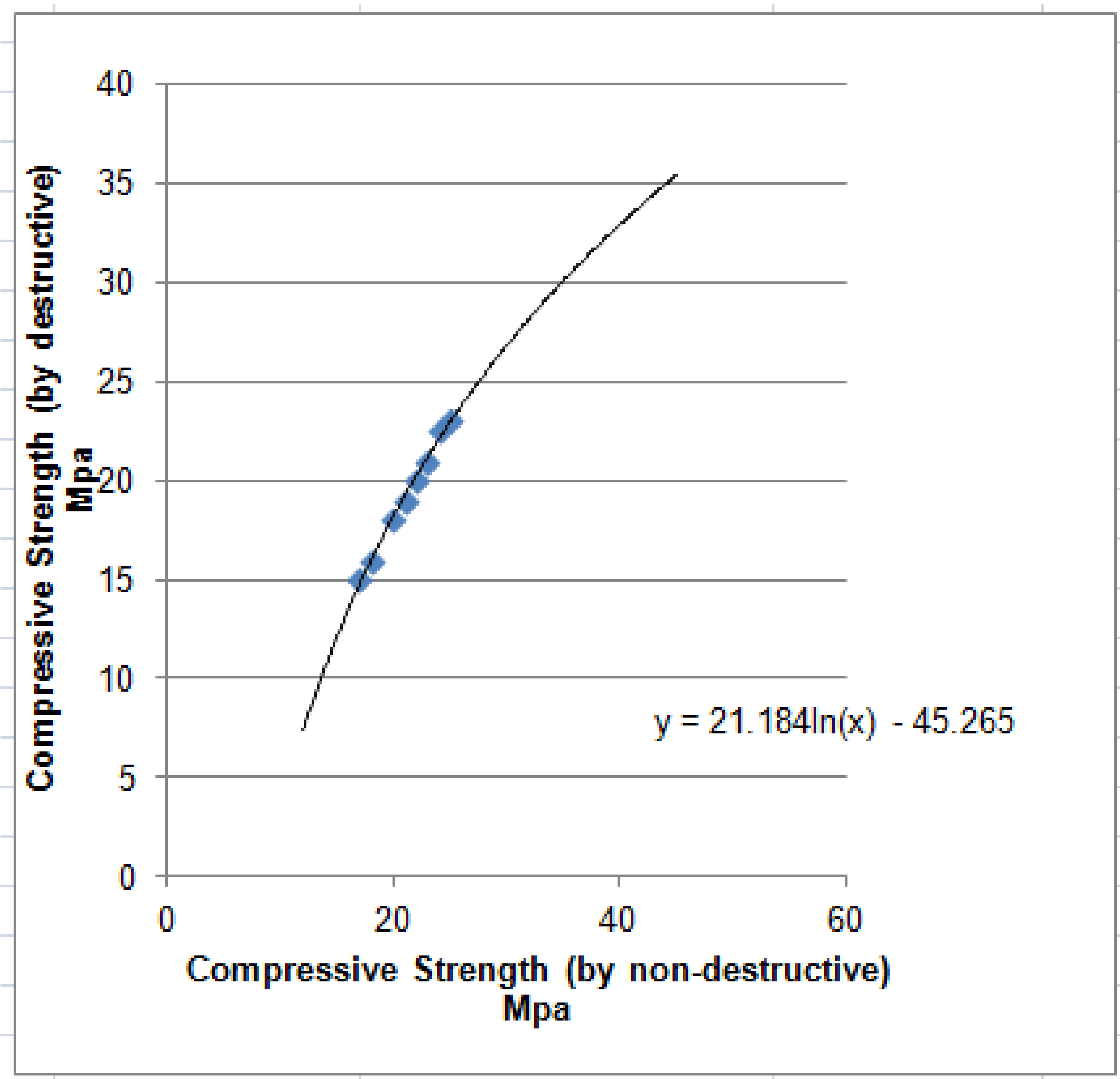
5. Correlation between Non-destructive test for normal 28 days cured and accelerated cubes



6. Correlation between Non-destructive test for normal 28 days cured and accelerated cubes



7. Correlation between Non-destructive testing and Destructive testing for 21 days cured concrete samples



7. Correlation between Non-destructive testing and Destructive testing for 21 days cured concrete samples