SEISMIC RETROFITTING OF RCC FRAME BUILDING

BY

NITIN J PATEL

10MCLC11

DEPARTMENT OF CIVIL ENGINEERING

AHMEDABAD-382481

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SEISMIC RETROFITTING OF RCC FRAME BUILDING

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering

(Computer Aided Structural Analysis and Design)

By

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MAY 2012

Declaration

This is to certify that

i) The thesis comprises my original work towards the degree of Master of Technology in Civil Engineering(Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

NITIN J PATEL

Certificate

This is to certify that the Major Project entitled "**Seismic Retrofitting of RCC Frame Building**" submitted by **Nitin Patel (10MCLC11)**, towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

Mr. Vijay D. Shah
Guide,
Structural Consultant,
Bhoomi Consultancy Pvt Ltd,
Ahmedabad.

Dr. P. H. Shah
Professor and Head,
Department of Civil Engineering,
Institute of Technology,
Nirma University, Ahmedabad.

Dr. K. Kotecha
Director,
Institute of Technology,
Nirma University,
Ahmedabad.

Examiner

Date of Examination

Abstract

The lack of adequate knowledge of structural behavior under seismic loads has claimed many lives and caused extensive property loss. A lack of good design and inadequate earthquake resistance cause damages to man made structures. The recent earthquake has revealed that the seismic strengthening or retrofitting of the building are one of the challenging task the structural engineer faces. The seismic retrofitting of building is the most effective methods of mitigating the seismic hazards. The purpose of retrofitting is to upgrade the strength and the ductility of existing damaged building so that it can withstand the future earthquake with minimum damage. The strengthening or retrofitting of existing concrete structures to resist higher design loads, correct strength loss due to deterioration, correct design or construction deficiencies, or increase ductility has traditionally been accomplished using conventional materials and construction techniques. Steel or concrete jackets, GFRP or CFRP wrapping and addition of new shear walls are just some of the many traditional techniques available.

Present work is focused on the Retrofit of an existing building, defining deficiencies, providing appropriate retrofitting and obtaining performance of a retrofitted RCC building. G+2 RCC school building is considered as case study for retrofitting. Various Non Destructive tests (NDT) are performed as a part for primary inspection of school building. It helps to ensure the deficient structural members.

ETABS software is used for modeling and analysis of structural member. After identifying of deficient structural member, suitable retrofitting methods are adopted. For retrofitting of deficient beam, GFRP wrapping has been done to increase flexural capacity. Jacketing has been done to take care of moment capacity for retrofitting of deficient columns. To take care of seismic forces the existing foundation is to be strengthen by combined footing which integrate member of columns.

Acknowledgement

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NITIN J PATEL

10MCLC11

Abbreviation, Notation and Nomenclature

d' .... Depth of compression reinforcement from the highly compressed face

d .... Effective depth of beam

b .... Breadth of beam, or shorter dimension of a rectangular column

E_s .... Modulus of elasticity of steel

e .... Eccentricity

f_{ck} .... characteristic cube compressive strength of concrete

f_y .... Characteristic strength of steel

M_u .... Bending moment

P_u .... Axial load on a compression member

V .... Shear force

τ_c .... Shear stress in concrete

τ_v .... Nominal shear stress

ϕ .... Diameter of bar

S_v .... Spacing of stirrups

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Chapter 1

Introduction

1.1 General

Earthquakes are one of the major natural hazards to life on the earth and have affected countless cities and villages on almost every continent. The damaged caused by earthquakes are mostly man mad structures. The casualties from the earthquakes suffered during the last decade have made it necessary to control and access buildings that have been constructed without any regard to appropriate seismic design characteristics. Earthquake poses an important challenge for the art and science of structural engineering to construct safe structures by proper design procedures. The lack of adequate knowledge of structural behavior under seismic loads has claimed many lives and caused extensive property loss for many decades.Existing reinforced-concrete (RC) structures may require strengthening to limit earthquake damage. Thus structure need to Seismic retrofit after Earthquake.

Earthquakes in Bhuj and Jammu Kashmir demonstrated the power of nature and the catastrophic impact of such power upon urban cities. Casualties and damage associated with older buildings, which were designed and constructed using codes that are now known to provide inadequate safety, are far worse than that for newer buildings which have been designed and built in accordance with more stringent code requirements. Earthquake occurrence is uncertain and can be possible partially for faults. The earthquake predictions cannot be eliminated by the earthquake events. Therefore the earthquake resistant structure is the only solution for the damaging effect of earthquake on structures. Also in case of damaged structures the retrofitting of structures is the effective method of mitigating the seismic hazard. Seismic up gradation of structures aims at improving the seismic performance of deficient structures, in terms of their strength, stiffness and ductility so that they can withstand seismic effect, current specification and maintaining desired performance level. Seismic strengthening is enhancing the capability of the structure for improves performance level.

1.2 Need of Retrofitting

Before going for the retrofitting of structures it is essential to have an idea about the reasons of failure of the structures and need of retrofitting. Seismic retrofitting is the modification of existing structures to make them more resistant to seismic activity, ground motion due to earthquakes.

Generally, registration and evaluation of state should be performed as a whole and be ready at a suitable time before design of the rehabilitation process is commenced. It is significant that the evaluation is based on the entire damage picture, the causes of damage.

Fig. 1.1 shows the general causes of damage to concrete and General causes of reinforcement corrosion are shown in fig. 1.2

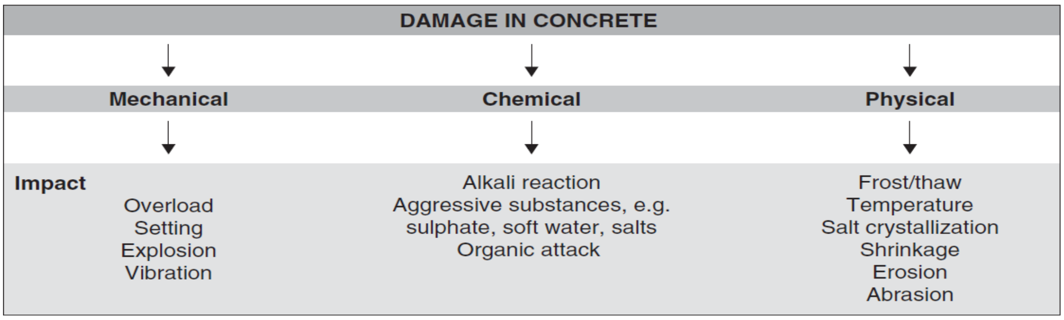

Figure 1.1: General Causes of Damage to Concrete

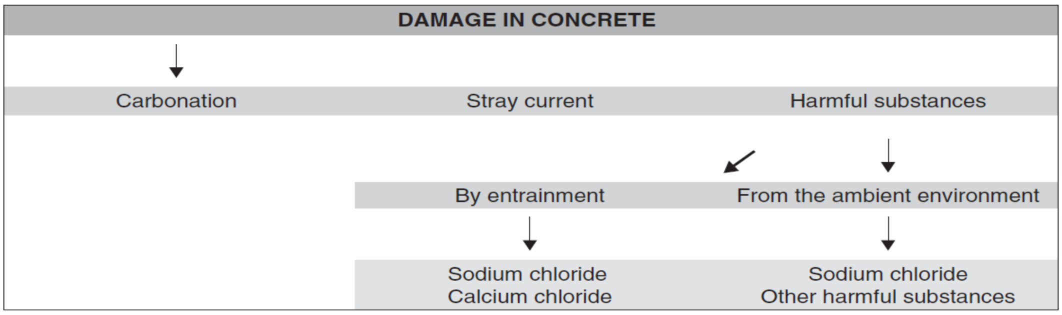

Figure 1.2: General Causes of Reinforcement Corrosion

Some other causes leading to retrofitting are as below:

- Changes in Code Provisions
- Change of use
- Corrosion of Embedded Reinforcements
- Design Errors
- Settlement and Movement
- Accidental Loadings

1.3 Background

Experience in past earthquake around the world indicates that concrete frames infilled with unreinforced masonry have been particularly prone to collapse. With the improvements in the earthquake engineering for new construction, more recent effort focused on the seismic behavior of older reinforced concrete frames. The effort of evaluating and improving seismic performance of building requires a detailed investigation of their deficiencies with regard to strength, stiffness, deformation capacity. However, over the past 25 years there has been a gradual shift from this position with the realization that increasing strength may not enhance safety, nor necessarily reduce damage. Before going for the retrofitting of structures it is essential to have an idea about the reasons of failure of the structures and need of retrofitting. In case of reinforced concrete buildings it is necessary to know about the failure pattern of structures. Generally R.C.C. buildings are designed from a detailed analysis for dead, live and seismic loads. The reinforced buildings often get damaged in earthquakes because of lack of good design and faulty reinforcement detailing practice.

The main causes of damage of reinforced concrete buildings are as follows:

- Improper design of lateral load resisting systems
- Omission of loading
- Inadequate detailing of reinforcement in beams, columns, and beam column joints from ductility point of view
- Poor quality of construction
- Inadequate diaphragm action of roof and floors
- Inadequate treatment of non-structural components like infill walls, staircase etc.
- Inadequate splicing of reinforcement in columns and beams can lead to failure

1.4 Objective of Study

For any building it is necessary to study the seismic behavior under external loading and its deficiencies, so the appropriate retrofit measure can be adopted very well. Following are the main objective of the study:

- To visual inspection of existing building.
- To evaluate existing capacity by Non destructive testing.
- To Analysis and Design of RCC frame building using ETABS software.
- To Study retrofitting techniques for RCC frame building.
  - Jacketing of Structural Member With Concrete
  - Glass Fiber Wrapping
  - Additional Shear Wall
  - Retrofitting of Foundation
- To evaluate the capacity and performance of retrofitted building.

1.5 Scope of work

The scope of work is as follows:

- Selection of an existing R.C.C building.
- Physical inspection & non destructive testing of building.
- Analysis of existing structural system.
- Summarizing Deficiencies of existing structural members.
- Suggest the appropriate retrofit measure for all deficient members.
- Comparison of the performance of the Retrofitted and non retrofitted building.

1.6 Compilation of Report

The report divided into various chapters as follows:

Introduction of Seismic Retrofitting of Building is discussed in **Chapter 1**.

Literature review is discussed in **Chapter 2**. It includes technical papers, books and journals. It also includes the focused on IITK Guidelines, ACI 440.2R-08, NDT method etc.

Fundamentals of retrofitting discussed in **Chapter 3**. It also discuss the different retrofitting techniques such as Jacketing, Wrapping and Additional Shear wall.

Chapter 4 covers Case study of a school building with physical inspection and NDT results. It gives idea about why retrofitting of school building and how many members have distress and damages.

In **Chapter 5**, 3-D modeling, analysis and design of G+2 RCC school building is carried out with the help of ETABS software.

Summary, Conclusions and Possibility for future scope of work are presented in **Chapter 6**.

Chapter 2

Literature Survey

2.1 General

Literature survey is important to review the work done in the area of Retrofitting of RCC structure. To take up the specific need to perform the analysis, the literature like technical papers, journals and books need to be referred. The prime important in the review was to understand the Fundamentals of the retrofitting and different retrofitting techniques of reinforced concrete building and assessment.

2.2 Literature Review

Various literatures related to Retrofitting are studied and brief review is presented below.

2.2.1 Guidelines of Retrofitting

IITK-GSDMA GUIDELINES [1] has discussed a systematic procedure for the seismic evaluation of buildings which can be applied consistently to a rather wide range of buildings. Though not applicable to all building types, the document also discusses some cost-effective strengthening schemes for existing older buildings identified as seismically deficient during the evaluation process. This document provided a method to assess the ability of an existing building to reach an adequate level of performance related to life safety of occupants.This document also provides some example of additional shear walls and jacketing of columns and beams. Therefore, the emphasis is on identification of unfavorable characteristics of the building that could result in damage to either part of a building or the entire structure.

ACI 440.2R-08 [2] gives general information on the history and use of FRP strengthening systems; a description of the unique material properties of FRP; and committee recommendations on the engineering, construction, and inspection of FRP systems used to strengthen concrete structures. FRP materials are lightweight, noncorrosive, and exhibit high tensile strength. FRP systems can also be used in areas with limited access where traditional techniques would be difficult to implement. This document provides guidance for the selection, design, and installation of FRP systems for externally strengthening concrete structures. This information can be used to select an FRP system for increasing the strength and stiffness of reinforced concrete beams or the ductility of columns and other applications. The document gives guidance on proper detailing and installation of FRP systems to prevent many types of debonding failure modes.FRP systems can be used to rehabilitate or restore the strength of a deteriorated structural member, retrofit or strengthen a sound structural member to resist increased loads due to changes in use of the structure, or address design or construction errors.

Eurocode 8 Part 3 [3] provided criteria for the evaluation of the seismic performance of existing individual structures and set forth criteria for the design of the repair/strengthening measures.The provisions of this Standard are applicable to all categories of buildings, the repair or strengthening of monuments and historical buildings often requires different types of provisions and approaches.

CPWD Handbook [4] discussed the causes of deterioration and consequent repair/rehabilitation strategy at optimum cost needs a scientific evaluation and solution.Also discussed nondestructive evaluation techniques, analysis and design of repairs, repair material, rehabilitation and retrofitting methods.

Boen T. [5] discussed the retrofitting method, need of retrofitting and step of retrofitting. Also includes basic causes of earthquake, materials used for retrofitting, retrofitting strategy etc.

Oehlers D. [6] explained that compression face plates are less likely to debond than tension face plates; results of thirteen tests on debonding due to vertical shear forces (critical diagonal crack debonding) and curvature (flexural end plate debonding) are described; and critical diagonal crack and flexural end plate debonding models are developed for compression face plates that can be used to ensure that beams with tension face plates that are extended into the compression faces do not debond prematurely.

Seismic Retrofit of Concrete Building Structures given by **Moehle J.** [7] , two general approaches usually are considered for a seismic rehabilitation. The first, illustrated in Figure A, involves global modification of the structural system. In this approach, the modifications to the structural system are designed so that the design demands, often denoted by target displacement, on the existing structural and nonstructural components are less than their capacities. Common approaches include addition of structural walls, steel braces, or base isolators. Passive energy dissipation schemes are not common for reinforced concrete frames because the displacements required for them to be effective often are beyond the displacement capacities of the existing components. Active control is rarely used. Another approach, illustrated in Figure B, involves local modification of isolated components of the structural and nonstructural system. In this approach, the objective is to increase the deformation capacity of deficient components so that they will not reach their specified limit state as the building responds at the design level. Common approaches include addition of concrete, steel, or fiber reinforced polymer composite (FRPC) jackets.

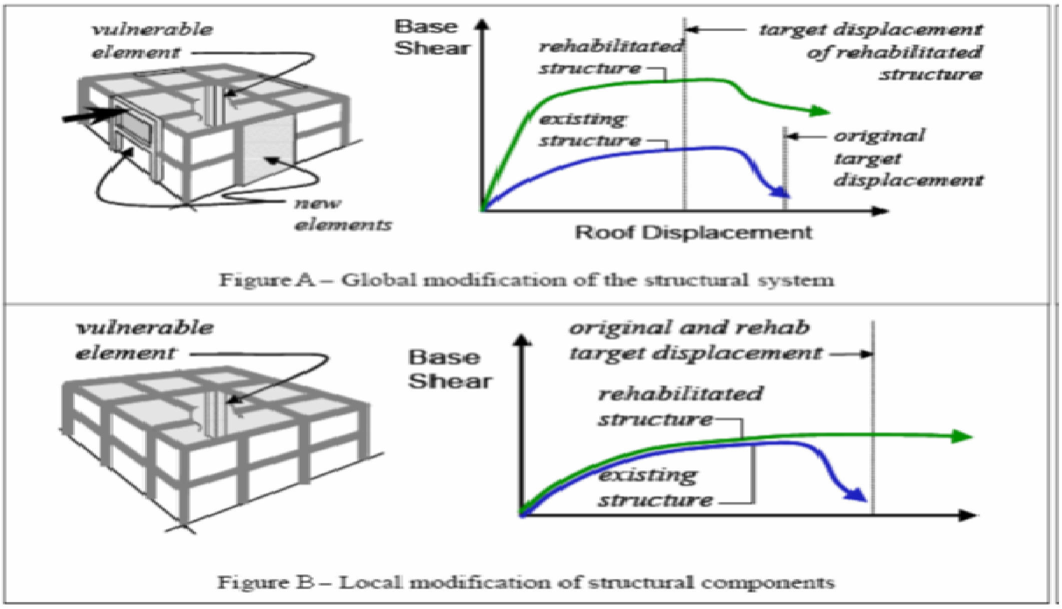

Figure 2.1: Global and Local Retrofit Approaches

Boroschek R. [8] described the need for structural retrofitting arises from several possible situations earthquake damage, change in code requirements and change in performance objectives of the structure. In order to established the need for retrofitting,a vulnerability study should be undertaken to identify existing conditions, vulnerability and expected behavior for different expected earthquake demands. The vulnerable study should be able to identify the main weakness and strengths of the structure.It's expected performance for the regional seismic hazard a description of damage severity and functional disruption for each health service and the convenience of a function, structural and non structural retrofit.

The followings are the basic steps in retrofitting process:

- Preliminary vulnerability study
- Identification of existing structural characteristics
- Identification of existing damage
- Determination of site seismic hazard
- Determination of site characteristics
- Establishing occupancy requirements
- Identification of economic restrictions
- Establishing social issues
- Consideration of historic status and local jurisdiction requirements
- Establishing possible target building performance levels
- Selection of the rehabilitation method (iterative procedure)

2.2.2 NONDESTRUCTIVE TEST

GUIDEBOOK ON NONDESTRUCTIVE TESTING OF CONCRETE [9] described advantage, disadvantage and procedure of different nondestructive test methods.

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

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

- Half-cell electrical potential method, used to detect the corrosion potential of reinforcing bars in concrete.
- Schmidt/rebound hammer test, used to evaluate the surface hardness of concrete.
- Carbonation depth measurement test, used to determine whether moisture has reached the depth of the reinforcing bars and hence corrosion may be occurring.
- Permeability test, used to measure the flow of water through the concrete.
- Penetration resistance or Windsor probe test, used to measure the surface hardness and hence the strength of the surface and near surface layers of the concrete.
- Covermeter testing, used to measure the distance of steel reinforcing bars beneath the Surface of the concrete and also possibly to measure the diameter of the reinforcing bars.
- Radiographic testing used to detect voids in the concrete and the position of stressing ducts.
- Ultrasonic pulse velocity testing, mainly used to measure the sound velocity of the concrete and hence the compressive strength of the concrete.
- Sonic methods using an instrumented hammer providing both sonic echo and transmission methods.
- Tomographic modelling, which uses the data from ultrasonic transmission tests in two or more directions to detect voids in concrete.
- Impact echo testing, used to detect voids, delamination and other anomalies in concrete.

- Ground penetrating radar or impulse radar testing, used to detect the position of reinforcing bars or stressing ducts.

- Infrared thermography, used to detect voids, delamination and other anomalies in concrete and also detect water entry points in buildings.

Malhotra and Carino [10] provided primarily information for practicing engineers engaged in quality control or investigations of hardened concrete.It gives two classes of nondestructive test methods for concrete. The first class consists of those methods that are used to estimate strength. The surface hardness, penetration resistance, pullout, break-off, pull-off, and maturity techniques belong to this category. Some of these methods are not truly nondestructive because they cause some surface damage, which is, however, minor compared with that produced by drilling a core. The second class includes stress wave propagation, ground probing radar, and infrared thermography techniques, which are used to locate delaminations, voids, and cracks in concrete. In addition, there are methods to provide information on steel reinforcement such as bar location, bar size, and whether the bars are corroding.

2.3 Summary

In this chapter, review of relevant literature is carried out. The review of literature gives the idea about the retrofitting, its need and fundamentals. Also discussed very useful different retrofitting techniques and designing.

Chapter 3

Fundamentals and Methods of Retrofitting

3.1 General

This chapter deals with retrofitting strategy used for the rehabilitation of the existing building. To obtain the performance of retrofitted building necessary strengthening has to be done for the weak elements of the structure. The different options available for the retrofitting of the deficient members of the building are discussed. The deficient members which are found out from existing capacity with the help of software and manual crosscheck. Accordingly the selection of retrofit strategy has been decided.

3.2 Concept of Retrofitting

Most of the loss of life and property in past earthquakes has occurred due to the collapse of non-engineered buildings. Non-engineered buildings are buildings which are spontaneously and informally constructed in various countries in the traditional manner using local available materials like stone, brick and wood, without any or little intervention by qualified architects and engineers in their design. in view of the continued use of such buildings, one of the efforts to reduce the earthquake risk in the future is to introduce earthquake resistance features in their existing structure/system . For existing buildings, retrofitting (strengthening) methods must be introduced. With little extra cost,the building can be made seismic resistance compare to demolition and constructing new building.

3.3 Systematic Approach for Retrofitting

Following are the various Retrofit Systems;

- Changing stiffness and/or strength
  - Addition of new RCC walls
  - Addition of braced frames
  - Thickening of existing shear walls
  - Carbon fiber reinforced plastic (CFRP) on Beam/Column
  - Glass fiber reinforced plastic (GFRP) on Beam/Column
  - Jacketing of Column/Beam
  - Combination of all above
- Increasing energy dissipation by providing new Viscous damping / Buckling restrained braced frame (BRBF)
- Modifying the character of the ground motion transmitted to the building using Base-isolation

3.4 Step of Retrofitting

Following are the steps of Retrofitting;

a. Determine as accurate as possible how the building behave when shaken by an earthquake

    (1) Check structural member physically

    (2) Check material quality of structural member

    (3) List all distress

b. Perform a static analysis for the building to get an idea of the causes of damage and determine the load paths when shaken by the earthquake.

c. Determine the causes of damage of components.

d. As soon as the type & reason of damage can be identified, repair and restoration of the components can be done separately in order that the original strength of the components can be restored.

e. If results of analysis indicate that the building with restored components can withstand the maximum expected earthquake for that area based on the latest code, then there is no need to retrofit

f. However, if the building with restored components was not designed or designed for a lower than the maximum expected earthquake forces specified by the latest code, then the building needs to be strengthened or retrofitted.

    (1) For strengthening, the restored building must be re-analyzed to identify which components must be strengthened.

    (2) For engineered buildings with severe damage and if the building needs to be strengthened, 3d non-linear analysis performance based design should be done.

g. After the strengthening works is completed, the building must be Re-analyzed to ensure that the strengthened building is earthquake resistant.

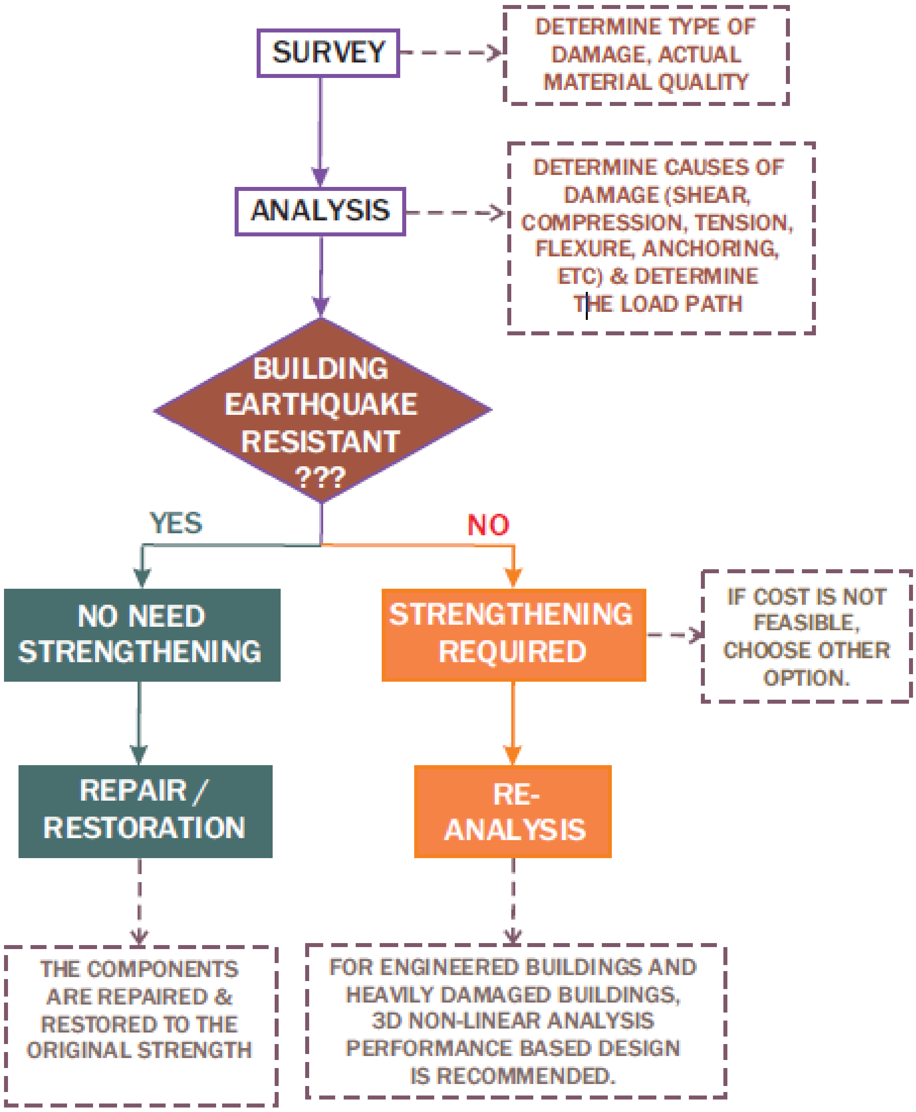

Figure 3.1: Steps of Retrofitting

Flow chart of Retrofitting is shown in above fig. 3.1

3.5 Retrofitting Techniques

While selecting the retrofitting strategy for the existing building the difficulties are arises during the modeling of that strategy. The retrofitting to the weak elements like beams and columns has been done by selecting the appropriate retrofit techniques so that it can be modeled properly in the software to get the desired results. The local retrofit approach is to upgrade the strength of the member, which are seismically deficient.
The most common method as given follows;

a. **Jacketing**

   (1) Concrete Jacketing

   (2) Steel Jacketing

   (3) Strap Jacketing

b. **Wrapping**

   (1) By using GFRP/ CFRP

   (2) By using steel fibres

c. **Additional Shear wall**

3.5.1 Jacketing of Structural Member

The most common method of enhancing the individual member strength is jacketing. Reinforced concrete jacketing increases the member size significantly.This has the advantage of increasing the member stiffness and is useful where deformations are to be controlled. If columns in a building are found to be slender, RC jacketing provides a better solution for avoiding buckling problems. It mainly includes the addition of concrete, steel or fibre reinforced polymer (FRP) jackets for use in confining reinforced concrete columns, beams, joints and foundation. The main purpose of jacketing is to increase concrete confinement by transverse fibre or reinforcement, to increase the shear strength by transverse reinforcement and flexural strength by longitudinal reinforcement.

The different options are available for the retrofitting of beams and columns as follows;

a. Jacketing of beams

b. Jacketing of columns

c. Jacketing of beam column joints

Jacketing of Columns

For deciding the jacketing thickness and the size of the jackets, the deficient column members are designed based on the GSDMA-IITK guidelines for seismic strengthening. Fig.3.2 shows the strengthening of the deficient column by means of concrete jacketing.

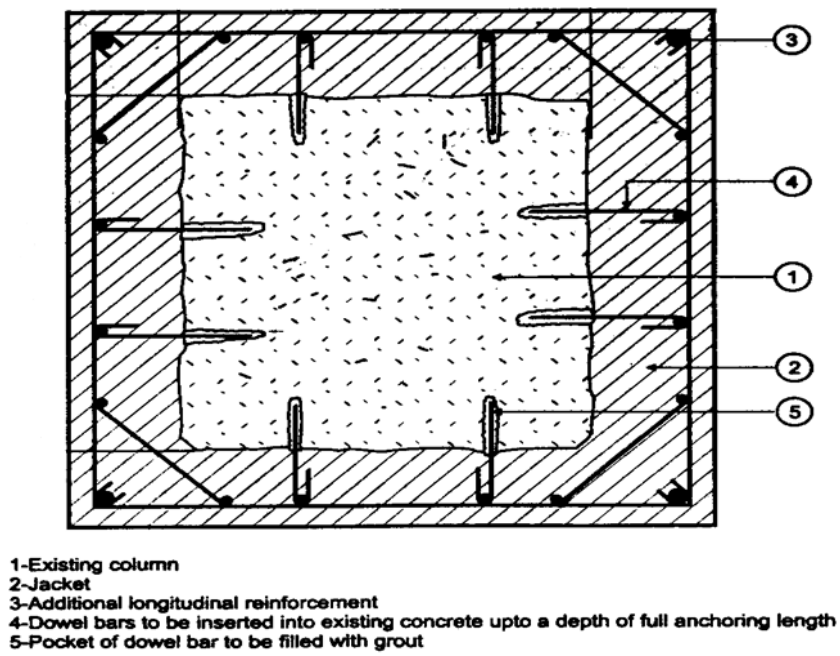

Figure 3.2: Strengthening Using Concrete Jacketing for Columns

3.5.2 GFRP Wrapping on Structural Member

The fibre wrap technique is a non-intrusive structural strengthening technique that increase the load carrying capacity (shear,flexural,compressive) and ductility of reinforced concrete members without causing any destruction or distress to the existing concrete. Composite materials made of fibers in a polymeric resin, also known as fiber-reinforced polymers (FRPs), have emerged as an alternative to traditional materials for repair and rehabilitation. For the purposes of this document, an FRP system is defined as the fibers and resins used to create the composite laminate, all applicable resins used to bond it to the concrete substrate, and all applied coatings used to protect the constituent materials. FRP materials are lightweight, noncorrosive, and exhibit high tensile strength. This document can be used to select an FRP system for increasing the strength and stiffness of reinforced concrete beams or the ductility of columns and other applications.Fig. 3.3 shows the GFRP on beams

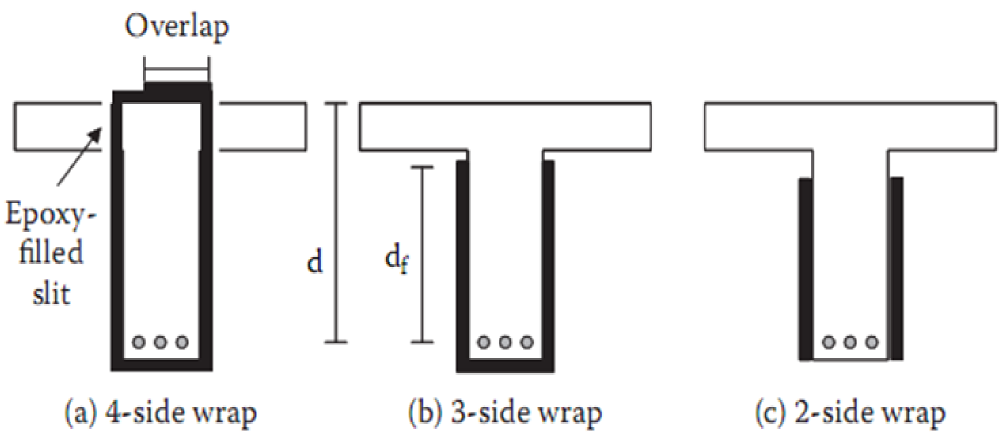

Figure 3.3: Fibre Wrap Technique for Improving Load Carrying Capacity of Beam

Enhancement in lateral drift, ductility and horizontal shear carrying capacities of a concrete member can be obtained by confinement of member by this method. The Flexural, shear and axial load carrying capacities of structural members can be enhanced by appropriate orientation of primary fibres of composites. The Resulting cured membrane not only strengthens RC member but also acts as an excellent barrier to corrosive agents, which are detrimental to concrete and reinforcement.The System is useful for its structural enhancement and protection capabilities under severe environmental conditions. It can be used for retrofitting of a wide variety of structures that include bridges, flyovers, chimneys, water tanks, buildings, large diameter pipes, industrial plants, jetties, sea-front and underwater structures.

3.5.3 Addition of New RC Shear Wall

Addition of new reinforced concrete shear walls (in Fig.3.4) provides the best option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures. The design of shear walls shall be done as per IS: 13920.

Incorporation of new structural components in an existing building will change the dynamic behavior of the whole structure considerably during the earthquake. The choice of the type, number and size of the added elements depends on the particularities of the existing structure and the functional layout of the building. Shear walls, because of their large stiffness and lateral strength, may provide the most significant part of the earthquake resistance of the building structure. Shear walls are used for strengthening RC frames, especially with open storeys. Shear walls can also be added with stiffness that is considerably higher than that of the old structure.

It could be replaced by earthquake resisting shear walls, or new walls could be added with no significant disturbance of the functional layout. In such cases, the new shear walls should have sufficient strength and stiffness to provide the entire lateral force resistance. Shear walls considerably add to the mass of the structure. The existing structure must be compatible with the strengthening elements and able to deform without failure in future earthquakes.The advantages of this is very easily **torsional irregularity** can be taken care.

Fig.3.4 provides Additional new shear walls for improved seismic performance.

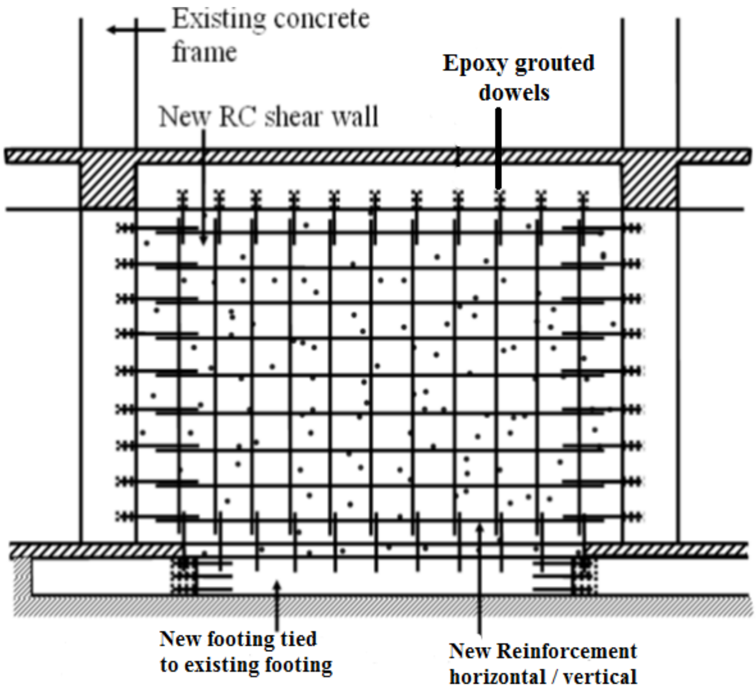

Figure 3.4: Additional New Shear Walls

3.6 Summary

In this chapter, Fundamental of retrofitting, need of retrofitting and Systematic Approach of retrofitting is discussed. To obtain the performance of retrofitted building necessary strengthening has to be done for the weak elements of the structure with the help of different retrofitting techniques such as Jacketing, Wrapping, Additional Shear wall.

Chapter 4

Physical Inspection and NDT Result of RCC Building

4.1 General

This chapter deals with general observation of an existing G+2 RCC school building at Ahmedabad with the help of different type of NDT equipments like Rebound hammer, Ultrasonic Pulse Velocity Tester, Half-cell Potential test and Profometer. Based on this observation, it can be decided regarding the present strength of the structural members/reinforcement of school building required retrofit or not.

4.2 Need of Non-Destructive Testing

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

4.3 Basic Methods for NDT of Concrete Structures

The following methods have been used for the NDT of concrete:

a. Rebound Hammer

b. Ultrasonic Pulse Velocity Tester

c. Half-cell Potential test

d. Profometer

4.3.1 Rebound Hammer (IS: 13311 Part 2-1992)

Rebound hammer is used for evaluating compressive strength and hardness of the structure and as such it is required to measure the available compressive strength and concrete quality of school building, this test is performed.

Main applications: It measures the surface hardness of concrete and provides an estimation of surface compressive strength, uniformity and quality of concrete.

Advantages: It gives accurate assessment of the strength of the surface layer of material. The entire structure can be tested in its 'as-built' condition.

Fig. 4.1 shows Schmidt rebound hammer.

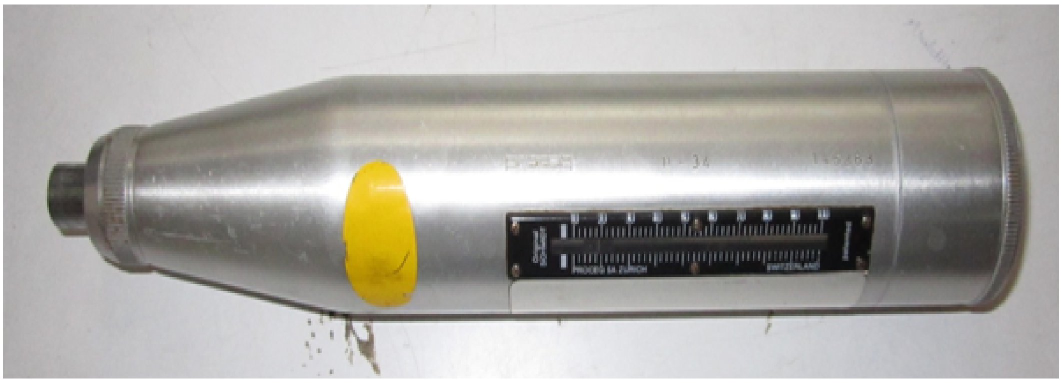

Figure 4.1: Schmidt Rebound Hammer

Limitations: Although the rebound hammer does provide a quick, inexpensive method of checking the uniformity of concrete, it has some serious limitations. The results are affected by: Smoothness of the test surface, Size, shape and rigidity of the specimen, Age of the specimen, Surface and internal moisture conditions of concrete, Type of coarse aggregate, Type of cement.

4.3.2 Ultrasonic Pulse Velocity Tester

Ultra pulse velocity is used to determine variability and quality of concrete by measuring pulse velocity. It is observed that almost all columns of school building are heavily damaged at ground floor and also cracks are seen. This test is performed for measure the defects like honeycombing, cracks, voids etc. The direct approach provides the greatest sensitivity and is therefore superior to the other arrangements.Thus, taken this method.

Main applications: Determination of the variability and quality of concrete by measuring pulse velocity. Using transmission method, the extent of such defects such as voids, honeycombing, cracks,fire damaged concrete and segregation may be determined.

Table 4.1: **Classification of the Quality of Concrete on the Basis of Pulse Velocity**

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

Criterion for concrete quality grading As per IS 13311 (Part 1):1992 is as in table4.1.

The pulse velocity (V) is given by

V= $\frac{L}{T}$

Where,

V = Pulse velocity

L = Path length,

T = The time taken by the pulse to traverse that length

Advantages: Excellent for determining the quality and uniformity of concrete. It can rapidly survey large areas and thick members. Path lengths of 10m to 15m can be inspected with suitable equipment.

Limitations: Proper surface preparation is required. Skill is required in the analysis of results as moisture variations and presence of metal reinforcement can affect results.

4.3.3 Half-Cell Potential Test (ASTMC 876-91)

Half-cell potential test (CANIN corrosion analyser) was performed to check the corrosion level in elements of school building like columns, chhajjas and lintel beams.

Table 4.2: **Criteria of Concrete Corrosion**

Phase of corrosion activity	Potential as measured by Copper Half cell
Initial Phase-Corrosion activity not taking place	< -200 mV
Transient Phase-Corrosion activity uncertain	-200 mV to -350 mV
Final Phase-Corrosion occurring positively	> -350 mV

The potential measured at the surface of concrete can be interpreted as per table 4.2

Main applications: The half-cell provides a relatively quick method of assessing reinforcement corrosion over a wide area without the need for wholesale removal of the concrete cover. Quantitative measurements are made so that a structure can be monitored over a period of time and any deterioration can be noted.

Advantages: It is portable equipment. Field measurements can be readily made and results can be plotted in the form of equipotential contour diagram, which can indicate likely areas of corrosion. It appears to give reliable information.

Limitations: The main limitation is that it does not provide information on rate of corrosion. It also requires access to reinforcing bars to make electrical contact.

4.3.4 Profometer (Rebar Locator)

Profometer test was performed to check the available cover in columns and available diameter of column bars of school building as original structural detailing drawings were not available.

Main applications: It is used for determining the presence, location and depth of rebars in concrete and masonry components. Advanced versions of covermeter can also indicate bar diameter when cover is known. It is moderately easy to operate.

Advantages: The presence of closely spaced reinforcing bar, laps, transverse steel, metal tie, wires or aggregates with magnetic properties can give misleading results. The meter has several scales for different bar sizes, therefore the bar diameter must be known if a true indication of cover is to be obtained.

Limitations: The maximum range of the instrument for practical purposes is about 100 mm. It does not give indication of the quality of concrete cover or the degree of protection afforded to the reinforcement.

4.4 Visual Inspection of School Building

G+2 RCC school Building is taken as case studies.

Data of Building:

Location: Nava vadaj, Ahmedabad

Storey: G+2 Building

All Beam size: 230 X 600 mm

All Column size: 230 X 450 mm

Slab thickness: 150 mm

Wall thickness: 230 mm

Zone: III

Area: 517 m^2

Medium soil

Cohesion c = 0.05 kg/cm^2

Density γ = 18 kN/m^3

Friction angle $\phi = 26.5°$

Plan, Elevation, 3-D view of school building in below figures 4.2, 4.3, 4.4 and 4.5

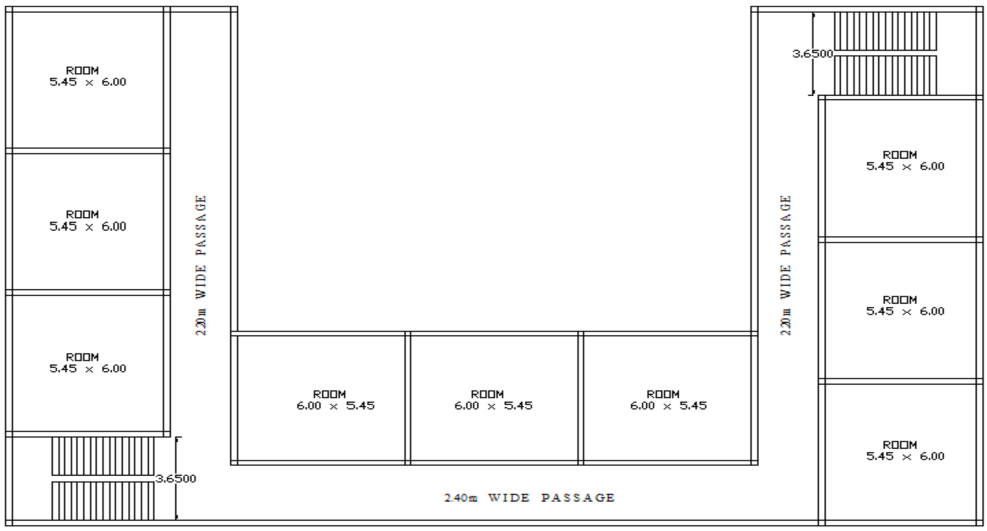

Figure 4.2: Floor Plan of G+2 RCC School Building

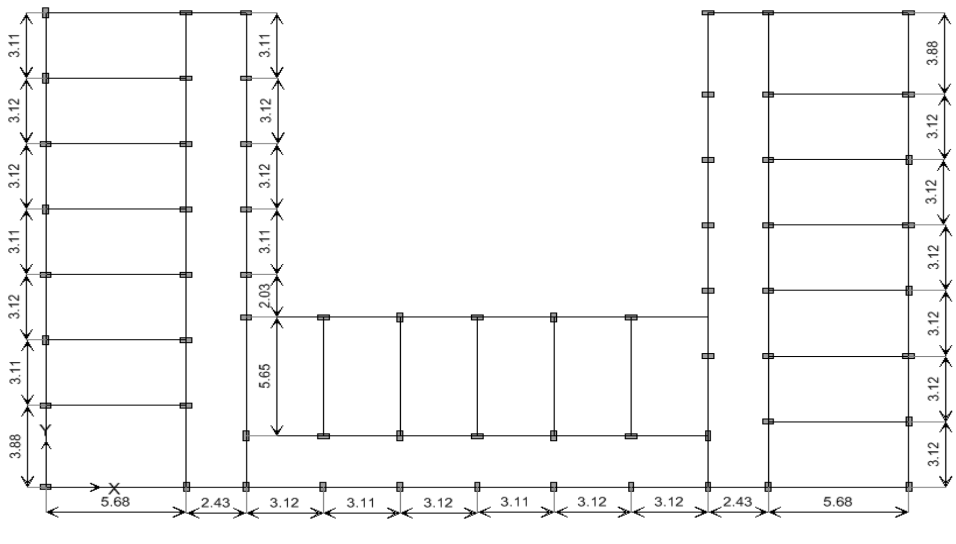

Figure 4.3: Dimension of the School Building

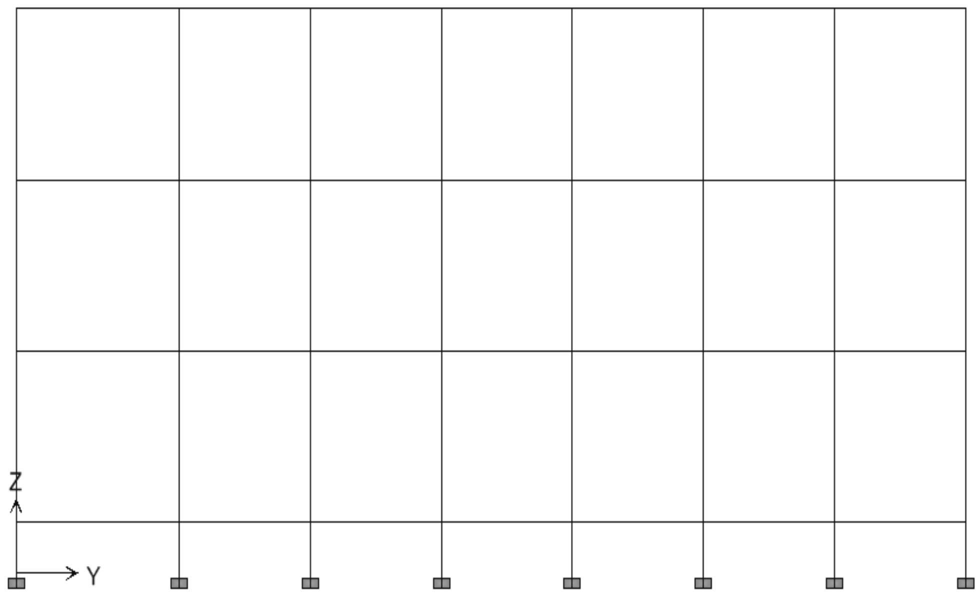

Figure 4.4: Elevation of Structural Model of G+2 RCC School Building

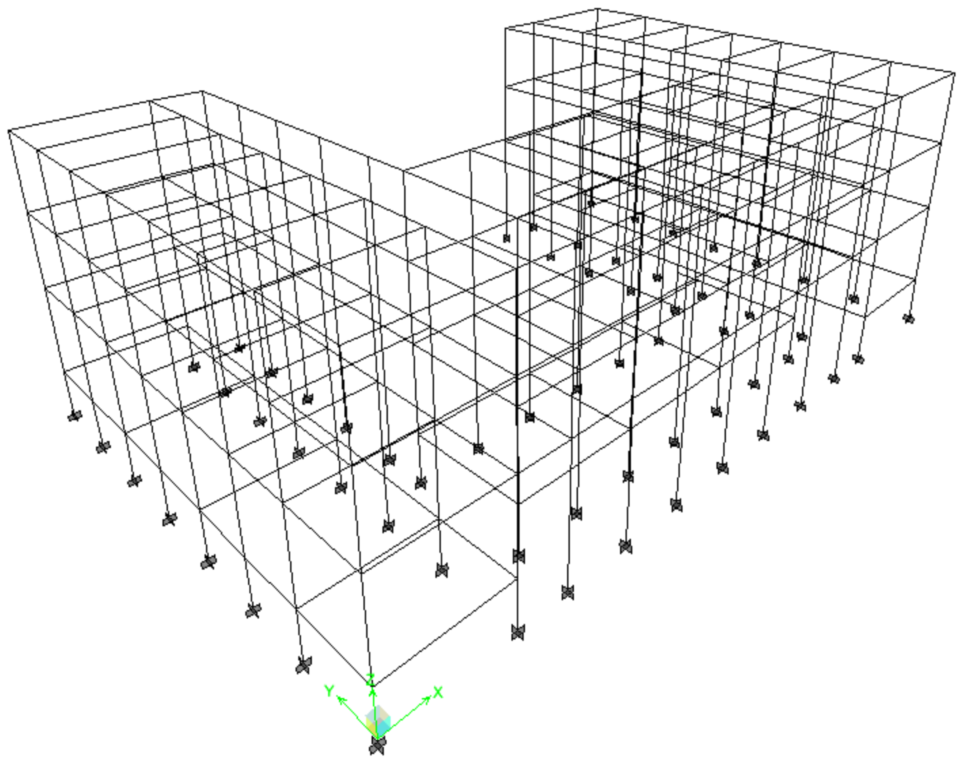

Figure 4.5: 3D View of Structural Model of G+2 RCC School Building

Some general plan of school building and details of column and chhajja number are shown in fig. 5.5, 4.7, 4.8 and 4.9.

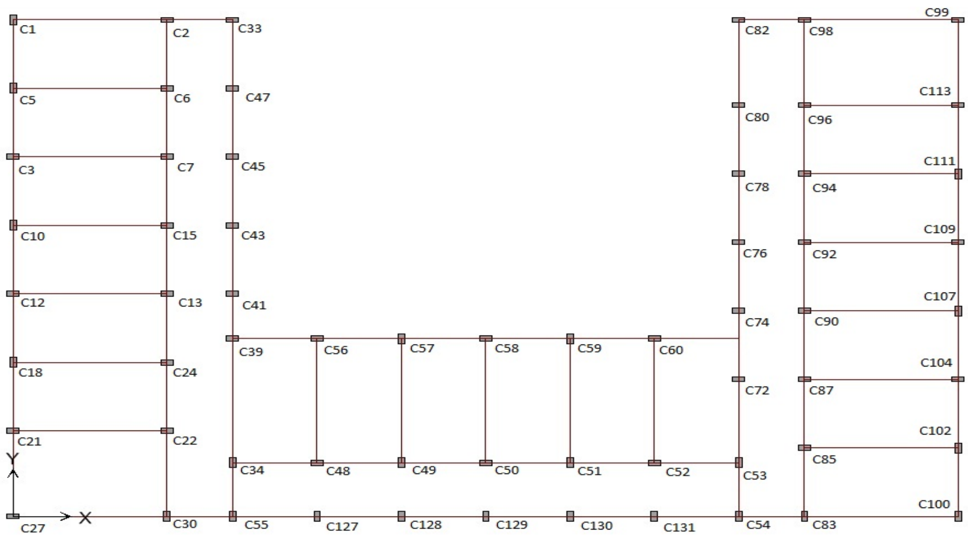

Figure 4.6: Column Number Location in Plan

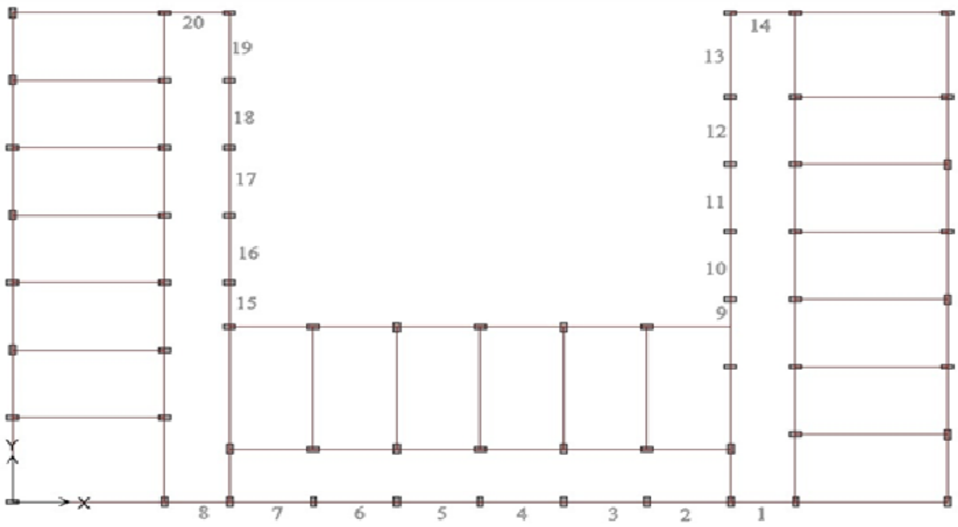

Figure 4.7: Chhajja Number Location at Ground Floor

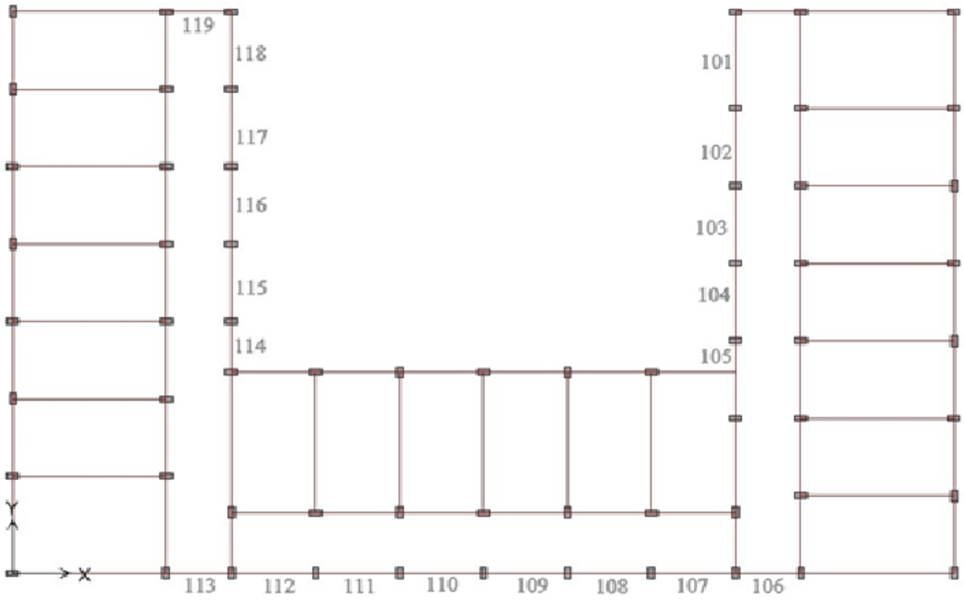

Figure 4.8: Chhajja Number Location at First Floor

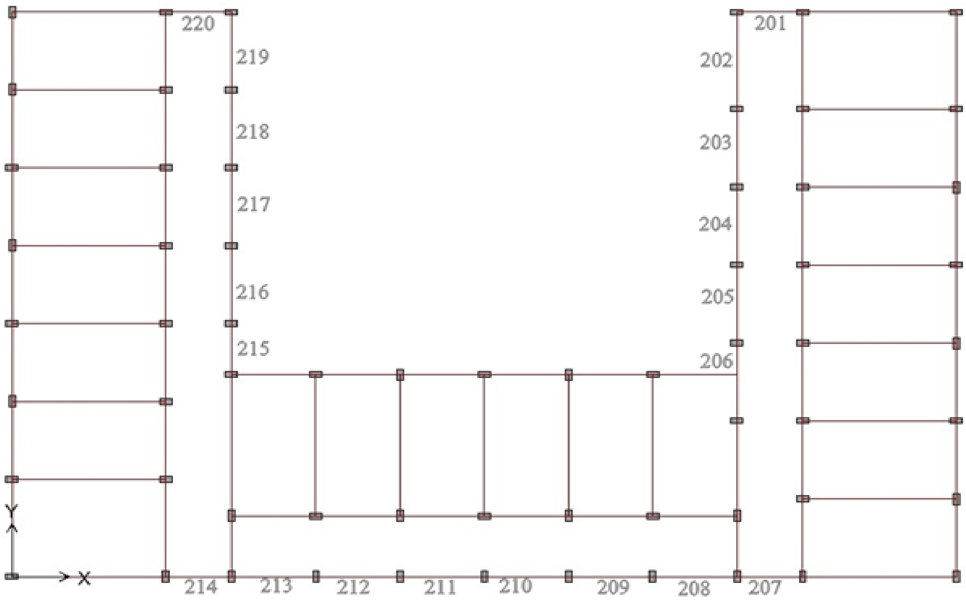

Figure 4.9: Chhajja Number Location at Second Floor

General observations carried out during the site inspection were shown in below subsequent figures:

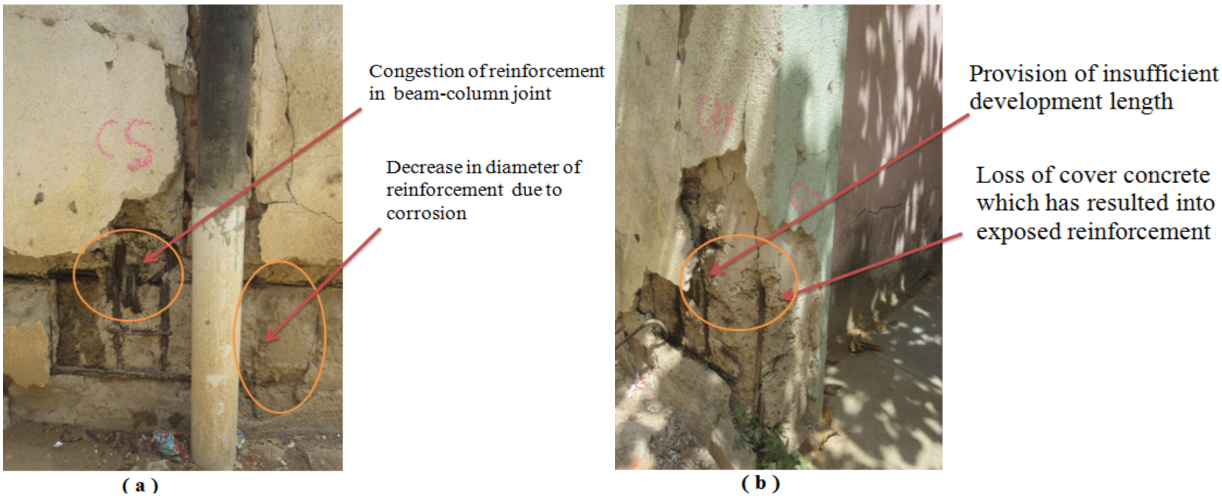

Figure 4.10: Exposed Reinforcement in Column due to Corrosion in [a] column no.5 , [b] column no.27

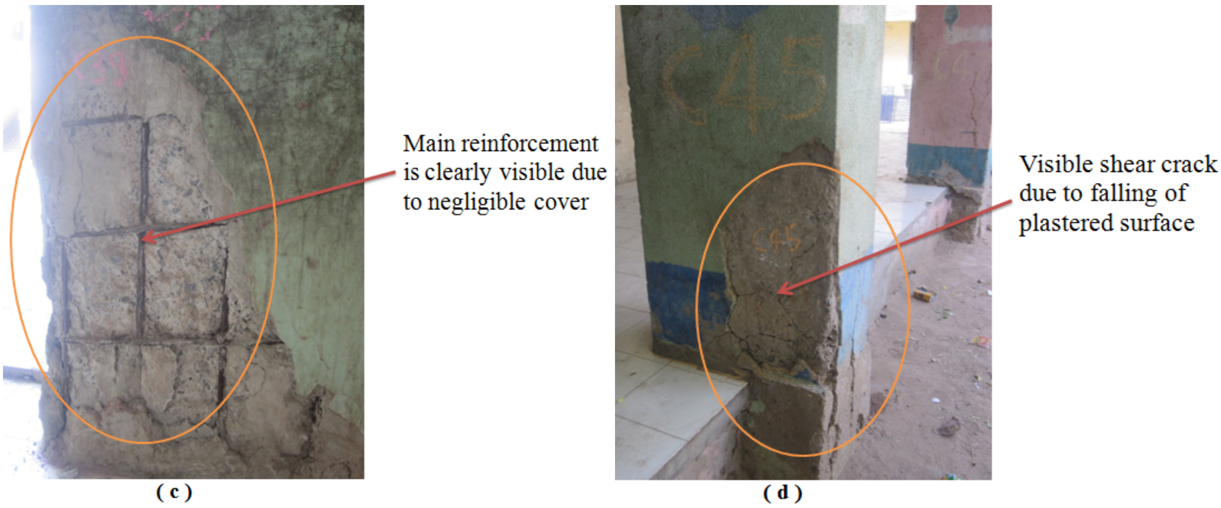

Figure 4.11: Exposed Reinforcement in Column due to Corrosion in [c] column no.39 , [d] column no.45

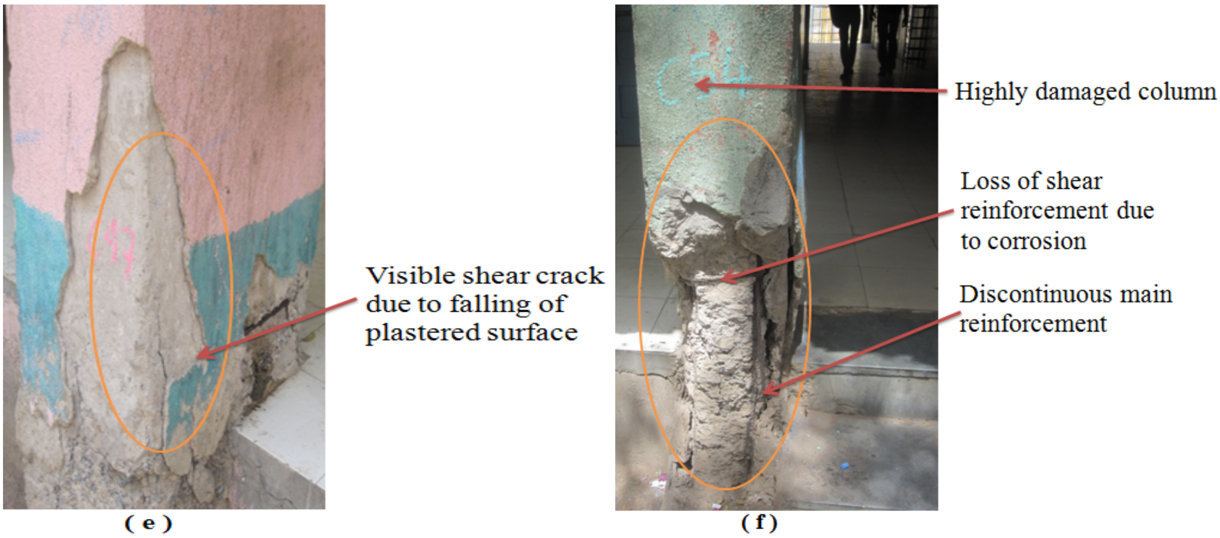

Figure 4.12: Exposed Reinforcement in Column due to Corrosion in [e] column no.47 , [f] column no.54

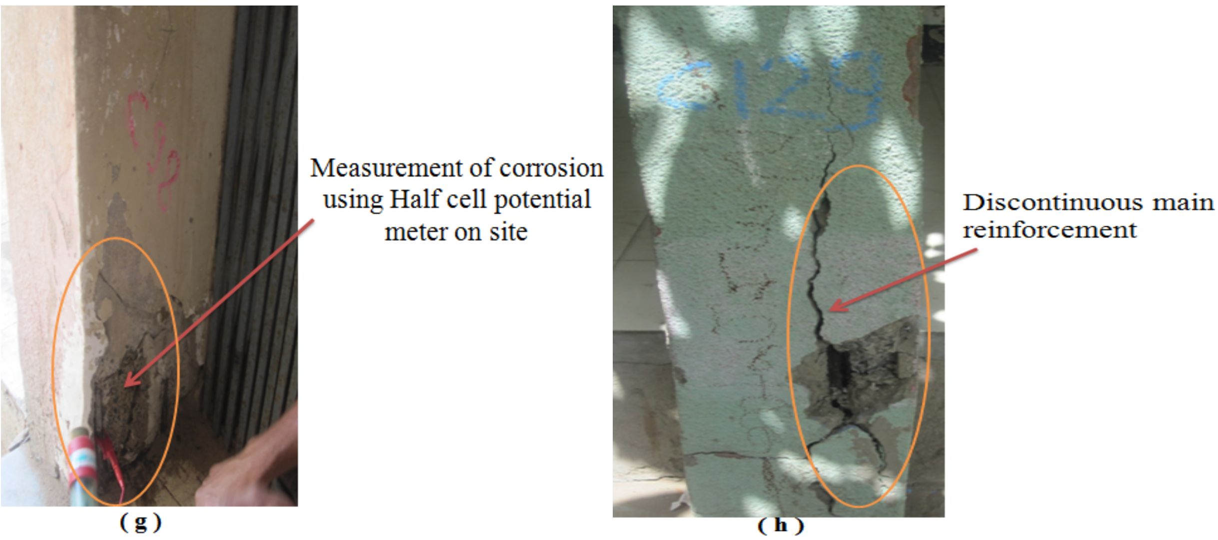

Figure 4.13: Exposed reinforcement in column due to Corrosion in [g] column no.98 , [h] column no.129

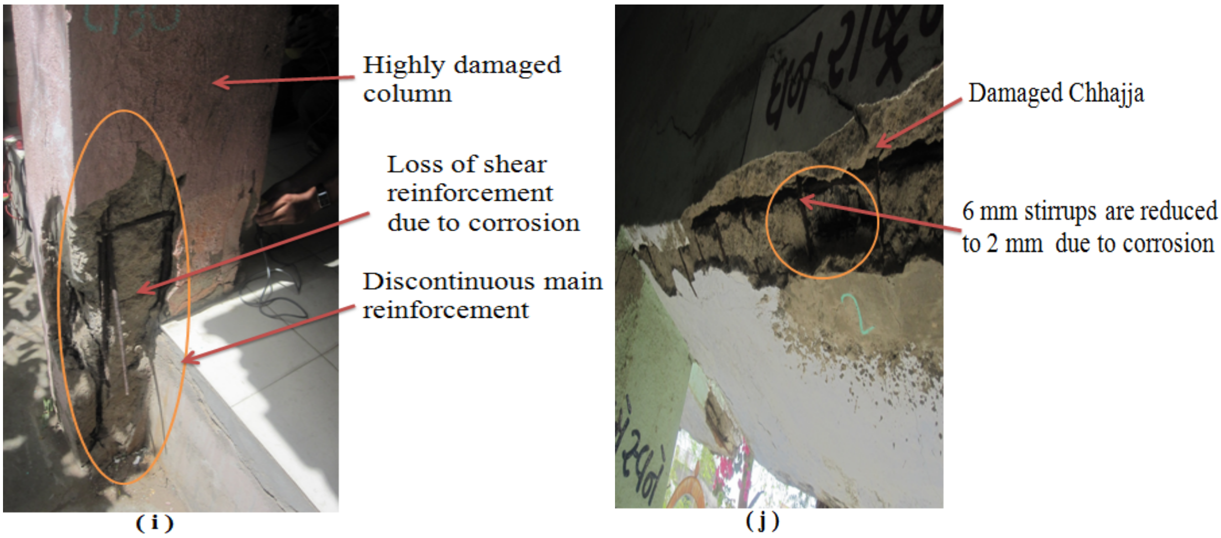

Figure 4.14: Exposed Reinforcement in Column and Chhajja in [i] column no.130 , [j] chhajja no.2

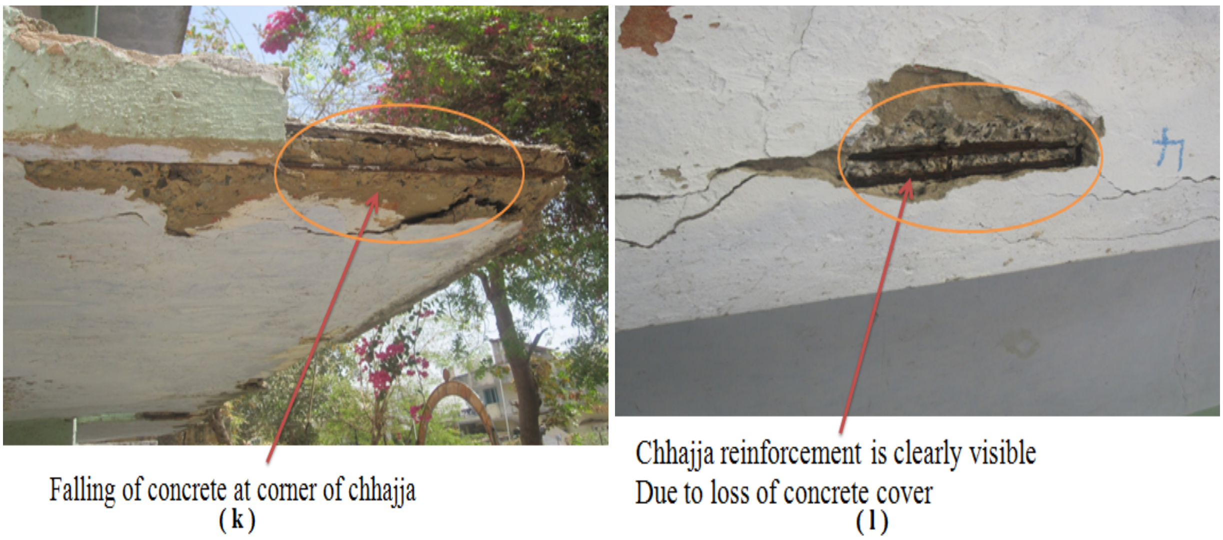

Figure 4.15: Exposed Reinforcement in Chhajja due to Corrosion [k] chhajja no.3 , [l] chhajja no.4

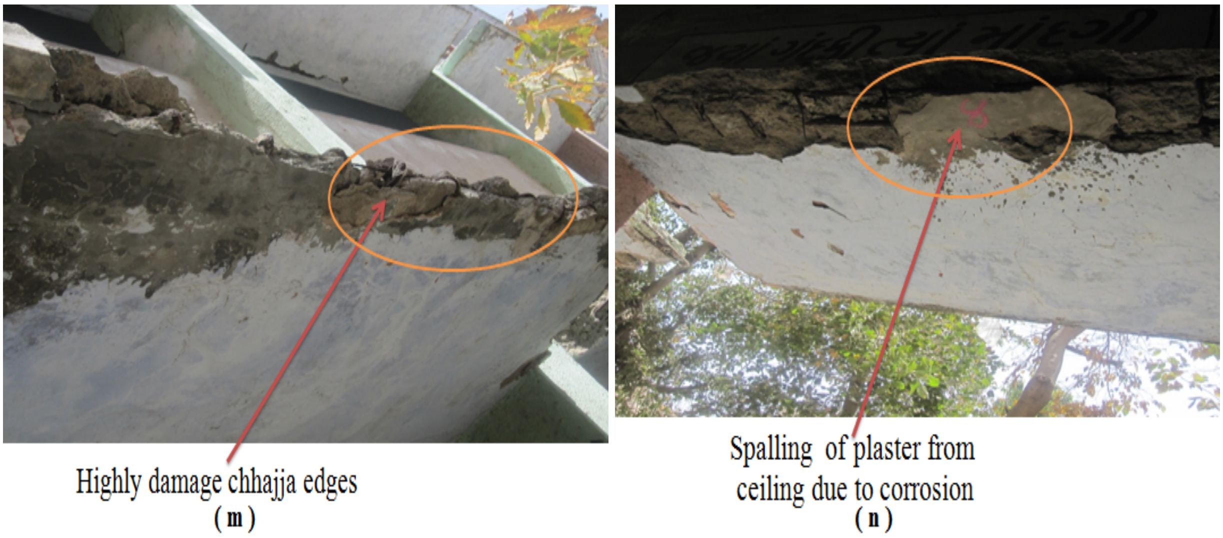

Figure 4.16: Exposed Reinforcement in Chhajja due to Corrosion [m] chhajja no.6 , [n] chhajja no.8

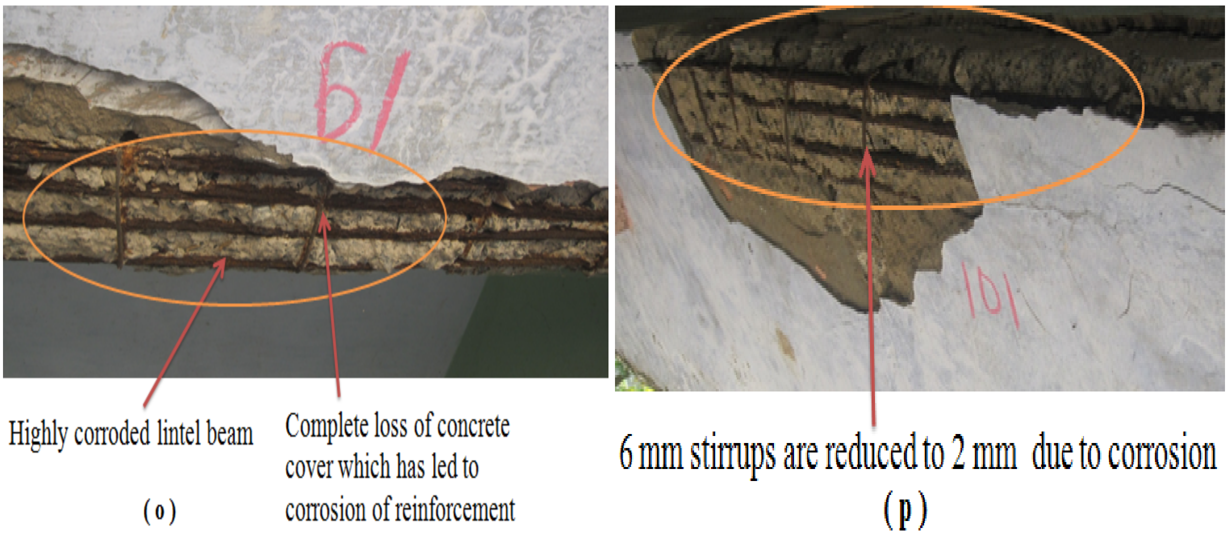

Figure 4.17: Exposed Reinforcement in Chhajja due to Corrosion [o] chhajja no.19 , [p] chhajja no.101

Figure 4.18: Exposed Reinforcement in Chhajja due to Corrosion [q] chhajja no.106 , [r] chhajja no.201

4.5 Summary of NDT Result of School Building

Non destructive tastings like rebound hammer, UPV, Profometer and Half-cell potentiometer were carried out for evaluating the performance of the structure.

All results of structural elements taken by NDT tests (Column and chhajja number shown in above figure) are shown in table 4.3

Table 4.3: **NDT Test Results of School Building**

		Ultrasonic Pulse Velocity Tester		Rebound hammer Ground Floor Result		Half-cell Potential test	
Sr no.	Identity of column no.	Pulse velocity (km/s)	Condition of concrete	Column surface	Rebound number	Potential as measured (millivolts)	Corrosion condition
1	54	2.85	Doubtful	Damaged surface	15	-237	Transient phase
				Finished surface	32		
2	54	2.85	Doubtful	Damaged surface	15	-246	Transient phase
				Finished surface	30		
3	27	1.14	Doubtful	Damaged surface	22	-270	Transient phase
				Finished surface	27		
4	47	0.78	Doubtful	Damaged surface	15	-156	Initial phase
				Finished surface	30		
5	45	1.78	Doubtful	Damaged surface	19	-300	Transient phase
				Finished surface	22		
6	76	1.89	Doubtful	Damaged surface	12	-220	Transient phase
				Finished surface	17		
7	130	2.81	Doubtful	Damaged surface	21	-330	Transient phase
				Finished surface	28		

		Ultrasonic Pulse Velocity Tester		Rebound hammer Ground Floor Result		Half-cell Potential test	
Sr no.	Identity of column no.	Pulse velocity (km/s)	Condition of concrete	Column surface	Rebound number	Potential as measured (millivolts)	Corrosion condition
8	5	2.74	Doubtful	Damaged surface	18	-351	Final phase
				Finished surface	23		
9	39	1.99	Doubtful	Damaged surface	16	-275	Transient phase
				Finished surface	17		
10	98	2.63	Doubtful	Damaged surface	16	N.A	N.A
				Finished surface	27		
11	1	N.A	N.A	Damaged surface	-	N.A	N.A
				Finished surface	15		
12	2	N.A	N.A	Damaged surface	23	N.A	N.A
				Finished surface	27		
13	33	2.11	Doubtful	Damaged surface	12	N.A	N.A
				Finished surface	22		

		Ultrasonic Pulse Velocity Tester		Rebound hammer Ground Floor Result		Half-cell Potential test	
Sr no.	Identity of column no.	Pulse velocity (km/s)	Condition of concrete	Column surface	Rebound number	Potential as measured (millivolts)	Corrosion condition
14	43	1.93	Doubtful	Damaged surface	11	N.A	N.A
				Finished surface	17		
15	41	2.14	Doubtful	Damaged surface	N.A	N.A	N.A
				Finished surface	20		
16	30	N.A	N.A	Damaged surface	-	N.A	N.A
				Finished surface	24		
17	55	2.22	Doubtful	Damaged surface	14	N.A	N.A
				Finished surface	22		
18	127	2.33	Doubtful	Damaged surface	-	N.A	N.A
				Finished surface	33		
19	128	2.09	Doubtful	Damaged surface	10	N.A	N.A
				Finished surface	33		

		Ultrasonic Pulse Velocity Tester		Rebound hammer Ground Floor Result		Half-cell Potential test	
Sr no.	Identity of column no.	Pulse velocity (km/s)	Condition of concrete	Column surface	Rebound number	Potential as measured (millivolts)	Corrosion condition
20	131	1.96	Doubtful	Damaged surface	-	N.A	N.A
				Finished surface	31		
21	83	N.A	N.A	Damaged surface	-	N.A	N.A
				Finished surface	21		
22	100	2.09	Doubtful	Damaged surface	-	N.A	N.A
				Finished surface	19		
23	74	2.06	Doubtful	Damaged surface	21	N.A	N.A
				Finished surface	15		
24	78	2.33	Doubtful	Damaged surface	16	N.A	N.A
				Finished surface	21		

		Ultrasonic Pulse Velocity Tester		Rebound hammer Ground Floor Result		Half-cell Potential test	
Sr no.	Identity of column no.	Pulse velocity (km/s)	Condition of concrete	Column surface	Rebound number	Potential as measured (millivolts)	Corrosion condition
25	80	2.30	Doubtful	Damaged surface	15	N.A	N.A
				Finished surface	22		
26	82	2.16	Doubtful	Damaged surface	18	N.A	N.A
				Finished surface	22		
27	99	N.A	N.A	Damaged surface	-	N.A	N.A
				Finished surface	17		

(Note: In above table 4.3, UPV test is not applicable for corner column because it is not possible to place of transducer on opposite side of corner column. Half-cell potential meter test is applicable when reinforcement is exposed. Thus, it is not possible for few column due to not reinforcement. N.A - Not Applicable)

Table 4.4: **Rebound Hammer Test Results of Columns at 1st and 2nd Floor**

Sr.no	Identity of Column no.	Rebound hammer test (For Finish surface)	
		First Floor Result	Second Floor Result
1	33	25	26
2	2	18	22
3	47	20	28
4	45	28	28
5	43	27	29
6	41	20	20
7	39	25	28
8	34	15	22
9	30	28	25
10	55	15	13
11	127	19	18
12	128	25	21
13	129	22	15
14	130	22	14
15	131	25	14
16	54	24	13
17	83	20	22
18	53	21	22
19	74	26	26
20	76	24	24
21	78	25	21
22	80	26	24
23	82	20	21
24	98	26	19
25	96	23	24

Table 4.5: **Half Cell Potential Test Results of Chhajja**

Sr.no	Identity of Chhajja no.	Half-cell Potential test	
		Potential as measured by copper half cell (millivolts)	Corrosion condition as per ASTM 876-91
1	101	-354	Final phase
2	2	-366	Final phase
3	109	-290	Transient phase
4	3	-310	Transient phase
5	4	-210	Transient phase
6	6	-207	Transient phase
7	8	-342	Transient phase
8	9	-283	Transient phase
9	19	-305	Transient phase
10	106	-309	Transient phase
11	201	-322	Transient phase
12	207	-306	Transient phase

Column size, main bar, stirrup (including plaster) has been carried out as below Fig. 4.19 with the help of profometer:

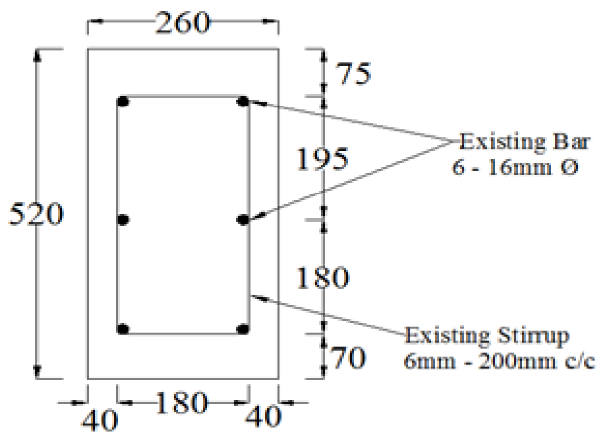

Figure 4.19: Existing Reinforcement Details of Column Carried out by Profometer

Figure 4.20: Reinforcement Details Shown by Profometer

Above Figure 4.20 shows meshing of reinforcement of column number 130 carried out by profometer.

4.6 Summary

The various tests are performed on structural elements of school building like columns, beams, chhajjas for evaluating the performance of the school building. It is concluded that concrete quality of columns is poor and almost all columns are in doubtful condition as indicated by UPV test. It is observed that, there are heavy corrosion in steel bars of columns and chhajjas as pointed by the results of half-cell potentiometer. Thus, Retrofitting is required from the above results.

Chapter 5

Analysis and Design of RCC Building

5.1 General

The chapter deals analysis and design of an existing school building. G+2 RCC school Building is taken as case studies which is Located at Nava vadaj, Ahmedabad.It requires retrofitting as discussed in chapter 4.

5.2 Structural Details of Existing Building

Structural drawings such as Typical slab details, terrace slab details; Beam, column and foundation details are not available of Shool building. Assume concrete grade for beams, columns and slabs were M15 and reinforcement was Fe415. The beams size 230mm $\times$ 600mm and Column size 230mm $\times$ 450mm. All the slabs were 150mm thick. All the walls were 230mm thick.The concrete grades of columns were M15.

5.3 Modeling of G+2 Building

The modeling of an existing building has done in ETABS. The G+2 RCC building has modeled extensively to simulate an existing RCC building. The G+2 building has modeled as frame having infill walls. The material properties and geometrical properties of structural elements is defined into the software. The orientation of columns is provided and the beam offsets is given as per the drawings. The grid lines were formed at each location of columns. Typical storey height is taken as 3.35m and base storey height was taken as 3.35m for foundation depth. The beams were created as per the location in drawing and corresponding properties of beams and columns were assigned. The reinforcement of all the members were obtained from the structural drawings available and the details are input in the software.

Data of Building:

Location: Nava vadaj, Ahmedabad

Storey: G+2 Building

All Beam size: 230 × 600 mm

All Column size: 230 × 450 mm

Slab thickness: 150 mm

Wall thickness: 230 mm

Zone: III

Area: 517 m^2 Medium soil

The Plan and dimension of building are shown in Fig. 5.1 & 5.2 .

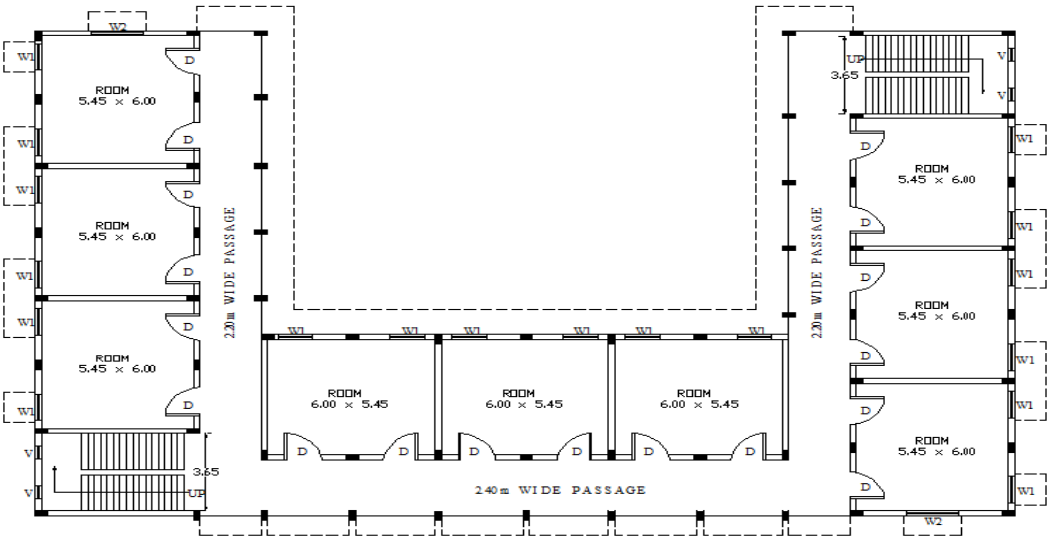

Figure 5.1: Floor Plan of G+2 RCC School Building

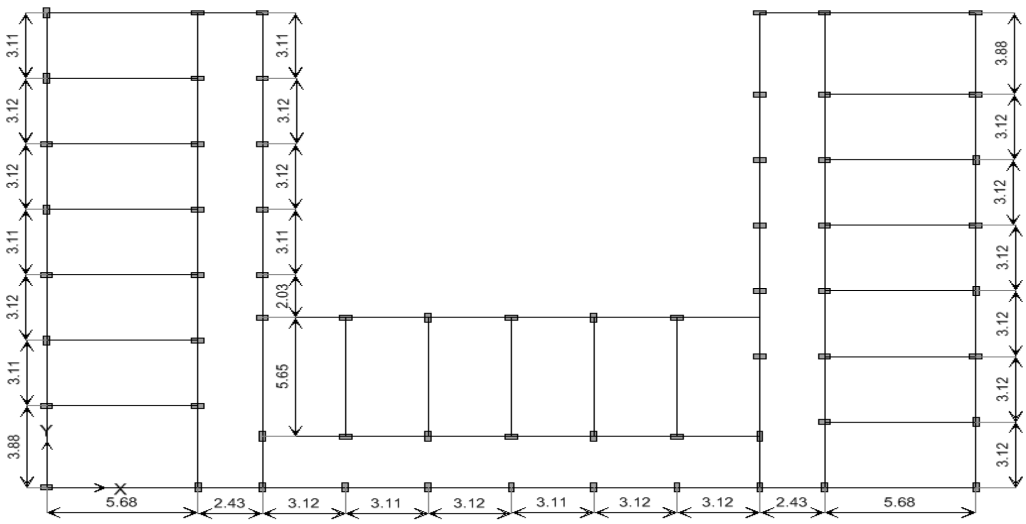

Figure 5.2: Dimension of the School Building

Fig. 5.3 and 5.4 shows the elevation and 3-D view of the school building.

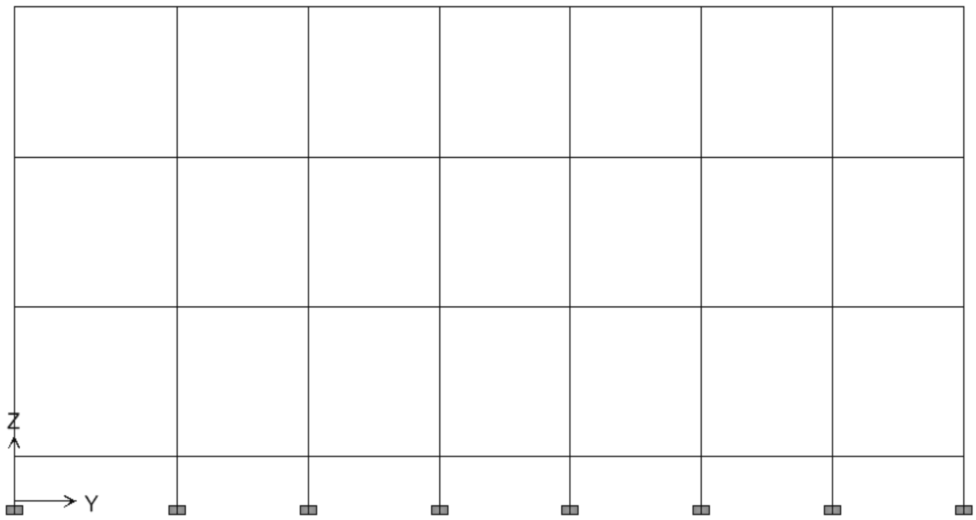

Figure 5.3: Elevation of Structural Model of G+2 RCC School Building

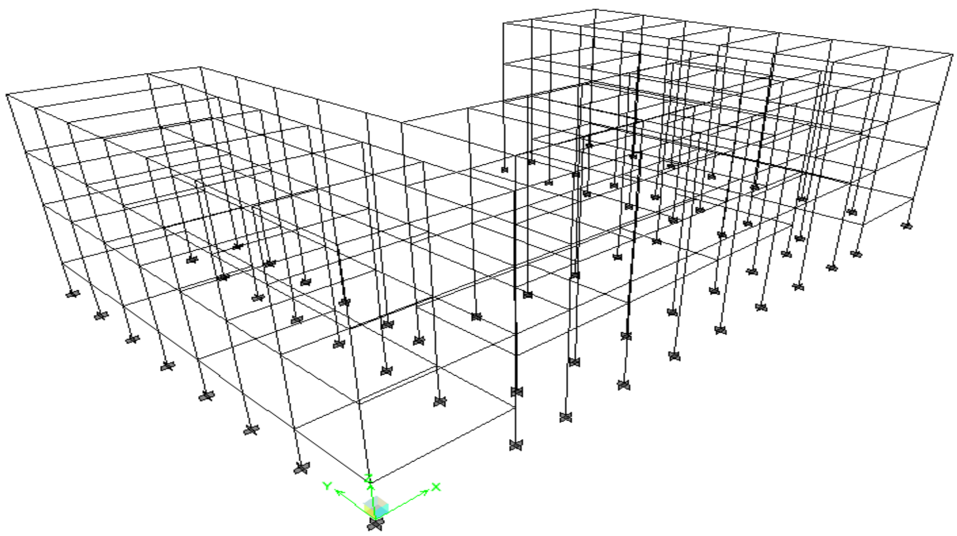

Figure 5.4: 3D View of Structural Model of G+2 RCC School Building

5.4 Load Cases

For analysis of model of new code,Four load cases were defined, Dead load, Live Load, Earthquake X, Earthquake Y, Wind X and Wind Y. Dead loads were considered by the software automatically.

Live Load = 3 kN/m^2

Live Load (for stair and passage) = 4 kN/m^2 (IS-875 Part-2)

Live Load (at roof) = 4 kN/m^2

Floor Finish Load = 1.25 kN/m^2

Floor Finish Load (at roof) = 2 kN/m^2

Wall Load (parapet) = 0.23 x 22 x 1= 5.06 kN/m

Wall Load on lower floors = 0.23 x 22 x (3.35-0.6) = 13.915 kN/m

Wall Load (Periphery of Passage)= 0.23 x 22 x 2.25 = 11.385 kN/m

Area of building : 517 m^2

Zone = III (Ahmedabad)

Zone Factor (Z) = 0.16

Response Reduction Factor (R) = 3 (OMRF) (Assumed as structural drawing is not available)

Importance Factor (I) = 1.5

Medium soil

Structural Response Factors (Sa/g) = 2.50

Time Period Ta = 0.159 sec (Manual calculation with infill panels)

Time Period Ta = 0.437 sec (Manual calculation without infill panels)

Time Period Ta = 0.567 (Old structure)

Time Period Ta = 0.38 (Retrofit structure)

5.5 Procedure for Retrofitting of Column

Procedure of Retrofitting of column is divided into 5 step as below:

a. To Find capacity of existing column

b. To Analyze the structure using ETABS

c. Grouping of columns

d. To Find deficiency

e. Identifying of column members which require retrofitting

a) To Find capacity of existing column

f_{ck} = 15 N/mm^2 , f_y = 415 N/mm^2

Size of column = 230 x 450 mm

steel Ast= 6-16mm ϕ (carried out from profometer)

percentage of steel (p_t) = 1.165 %

$A_{st} = 6 \times \frac{\pi}{4} \times 16^2 = 1206 \quad mm^2$

$A_g = 230 \times 450 = 103500 \quad mm^2$

clause 39.3, IS:456 - 2000 [11]

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{st} \tag{5.1}$$

Where

P_u = axial load on the member

A_c = Area of concrete

$P_u = 0.4\ f_{ck}[A_g - A_{st}] + 0.67\ f_y A_{st}$

$P_u = 0.4 \times 15 \times [103500 - 1206] + 0.67 \times 415 \times 1206$

P_u = 1154 kN

cover = 40 mm (from profometer)

Stirrup size = 6 mm

Find out moment in x direction:

d' = 40 + 6 + $\frac{12}{2}$ = 52 mm

$\frac{d'}{D} = \frac{52}{450} = 0.12$

Chart 33, SP 16

$\frac{P_u}{f_{ck}bD} = \frac{1154\times10^3}{15\times230\times450} = 0.7$

$\frac{p}{f_{ck}} = \frac{1.}{15} = 0.07$

$\frac{M_{ux}}{f_{ck}bD^2} = 0.055$

$M_{ux} = 0.055 \times 15 \times 230 \times 450^2$

$M_{ux} = 38$ kN.m

Find out moment in y direction:

$\frac{d'}{D} = \frac{52}{230} = 0.23$

Chart 34, SP 16-1980 [12]

$\frac{P_u}{f_{ck}bD} = \frac{1133.42\times10^3}{15\times230\times450} = 0.7$

$\frac{p}{f_{ck}} = \frac{1.09}{15} = 0.07$

$\frac{M_{uy}}{f_{ck}bD^2} = 0.08$

$M_{uy} = 0.08 \times 15 \times 230 \times 450^2$

$M_{uy} = 29$ kN.m

Table 5.1: **Existing Capacity of Column**

Existing Axial load carrying capacity	Existing moment capacity in X direction	Existing moment capacity in Y direction
1154 kN	38 kN.m	29 kN.m

b) To Analyze the structure using ETABS:

Etabs gives all type of result such as Axial loads, moment, Displacement etc. But we need axial load and moment for comparison between existing carrying capacity and new required capacity due to change of code and zone. Load and details of beam and column is considered as per above described in load cases.

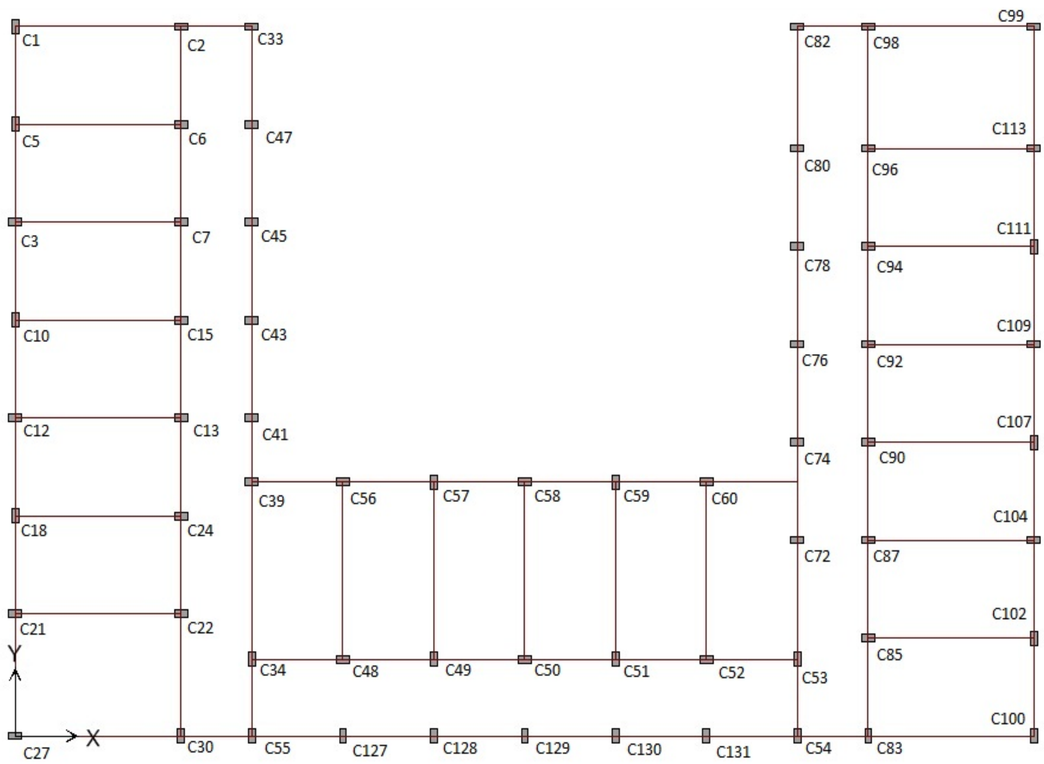

Figure 5.5: Location of Column in Plan

c) Grouping as per Percentage Requires /and Forces:

New axial load and moment are obtained from ETABS software.All the column are distributed among different groups and design is being done as per maximum load in a group.There are total 4 groups as mentioned below:

DCON19 = 1.5 D.L + 1.5 EQx, DCON20 = 1.5 D.L - 1.5 EQx

DCON21 = 1.5 D.L + 1.5 EQy, DCON22 = 1.5 D.L - 1.5 EQy

GROUP 1

Story	Column no.	Fx kN	Fy kN	Fz kN	Mx kN.m	My kN.m	Mz kN.m	Load
BASE	33	-20.42	-0.1	386.09	0.047	-65.86	-0.084	DCON19
BASE	41	0.05	-6.9	273.17	20.78	0.094	-0.003	DCON21
BASE	43	-0.02	-6.74	277.8	20.447	-0.04	-0.003	DCON21
BASE	45	-0.04	-6.77	292.87	20.517	-0.08	-0.003	DCON21
BASE	47	0.01	-6.75	314.39	-20.374	0.278	-0.026	DCON22
BASE	74	-1.78	-25.54	442.32	26.482	-1.679	-0.023	DCON21
BASE	76	-0.03	-6.85	292.48	20.694	-0.068	-0.003	DCON21
BASE	78	-0.03	-6.78	286.23	20.544	-0.052	-0.003	DCON21
BASE	80	0.15	6.83	343.84	-20.386	0.52	-0.026	DCON22
BASE	82	20.75	0.51	426.57	0.97	66.886	0.055	DCON20
BASE	127	-6.9	0.26	279.31	-0.785	-20.928	-0.084	DCON19
BASE	128	-6.92	0.08	291.82	-0.136	-20.974	-0.084	DCON19
BASE	129	-6.93	-0.06	292.03	0.448	-20.993	-0.084	DCON19
BASE	130	-6.93	-0.02	291.12	1.019	-20.995	-0.084	DCON19
BASE	131	-6.93	-0.32	287.48	1.534	-20.986	-0.084	DCON19

GROUP 2

Story	Column no.	Fx	Fy	Fz	Mx	My	Mz	Load
BASE	6	-20.07	0.29	664.35	-0.406	-61.849	0.089	DCON19
BASE	54	-0.01	20.52	428.37	-66.648	-0.108	-0.026	DCON22
BASE	55	-0.2	21.06	420.32	-68.278	-0.475	-0.026	DCON22
BASE	72	-0.17	-28.87	612.35	28.684	0.219	-0.138	DCON21

GROUP 3

Story	Column no.	Fx	Fy	Fz	Mx	My	Mz	Load
BASE	1	12.86	-45.17	688.13	61.068	8.792	0.191	DCON21
BASE	2	15.54	-2.54	693.08	2.002	50.192	0.209	DCON20
BASE	5	2.27	56.74	634.82	-68.045	2.224	-0.123	DCON22
BASE	10	1.37	51.33	558.85	-64.49	1.574	-0.008	DCON22
BASE	18	1.5	-51	555.29	64.363	1.675	0.029	DCON21
BASE	27	57.13	4.09	653.77	-2.186	70.816	0.039	DCON20
BASE	30	-12.9	37.85	787.41	-54.912	-8.658	-0.092	DCON22
BASE	48	-53.13	1.64	635.95	-1.901	-65.825	-0.044	DCON19
BASE	50	-0.54	6.48	658.23	-16.841	-0.66	-0.024	DCON22
BASE	52	52.34	1.43	628.18	-1.854	65.011	0.126	DCON20
BASE	53	-2.21	-30.77	546.8	56.865	-1.438	0.383	DCON21
BASE	56	-59.73	-2.1	665.81	1.913	-70.857	0.044	DCON19
BASE	58	-0.16	-6.75	620.88	17.389	-0.163	0.004	DCON21
BASE	60	-0.03	-7.9	617.53	17.867	-0.07	0.011	DCON21
BASE	98	-17.94	-5.35	699.24	3.915	-48.08	0.023	DCON19
BASE	99	-64.81	-6.28	673.41	4.701	-76.547	0.084	DCON19
BASE	100	-14.97	43.03	689.74	-58.356	-10.121	0.171	DCON22
BASE	102	-2.33	-57.18	638.46	68.524	-2.1	-0.076	DCON21
BASE	107	-1.37	-51.61	558.71	64.867	-1.559	0.003	DCON21
BASE	111	-1.47	-51.59	552.29	64.679	-1.659	0.04	DCON21

GROUP 4

Story	Column no.	Fx kN	Fy kN	Fz kN	Mx kN.m	My kN.m	Mz kN.m	Load
BASE	3	60.45	-1.39	780.51	1.378	75.595	0.085	DCON20
BASE	7	-60.08	0.54	830.55	-0.568	-75.254	-0.057	DCON19
BASE	12	61.13	-1.41	776.5	1.397	76.738	0.043	DCON20
BASE	13	-61.16	0.51	822.58	-0.545	-76.838	-0.053	DCON19
BASE	15	-19.8	0.57	680.66	-0.585	-62.314	-0.043	DCON19
BASE	21	60.97	-3.32	872.67	2.65	75.833	-0.655	DCON20
BASE	22	-62.41	-0.9	901.57	0.377	-76.973	0.669	DCON19
BASE	24	-20.7	0.62	677.51	-0.619	-64.134	-0.098	DCON19
BASE	34	1.68	-17.03	706.91	52.595	1.072	-0.184	DCON21
BASE	39	45.98	-13.2	785.17	9.212	61.87	-0.205	DCON20
BASE	49	-0.21	60.45	814.37	-76.018	-0.221	-0.038	DCON22
BASE	51	-0.27	59.99	813.99	-75.422	-0.262	0.028	DCON22
BASE	57	-0.03	-61.77	777.05	77.611	-0.04	-0.059	DCON21
BASE	59	-0.05	-61.74	778.06	77.615	-0.049	0.05	DCON21
BASE	83	13.17	35.01	774.91	-52.285	8.221	-0.265	DCON22
BASE	85	22.02	0.63	666.1	-0.541	64.527	-0.04	DCON20
BASE	87	61.6	0.32	829.6	-0.335	77.433	0.026	DCON20
BASE	90	20.02	0.32	681.11	-0.336	63.002	0.034	DCON20
BASE	92	60.82	0.36	822.39	-0.366	76.33	0.038	DCON20
BASE	94	20.14	0.22	677.46	-0.276	62.509	-0.024	DCON20
BASE	96	16.85	1.33	1060.04	-0.697	10.609	0.33	DCON2
BASE	104	-61.97	-1.78	781.88	1.788	-77.82	0.006	DCON19
BASE	109	-60.84	-1.78	775.53	1.773	-76.288	-0.044	DCON19
BASE	113	-59.6	0.31	877.48	0.4	-73.474	-0.62	DCON19

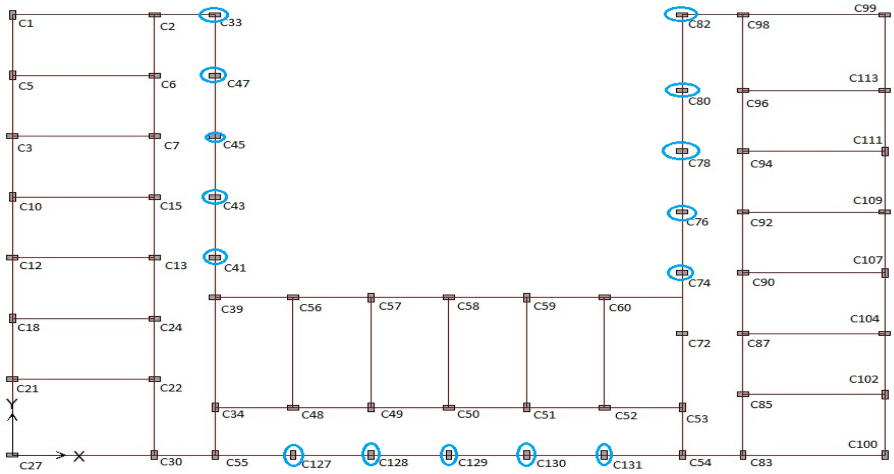

Figure 5.6: Column Grouping 1

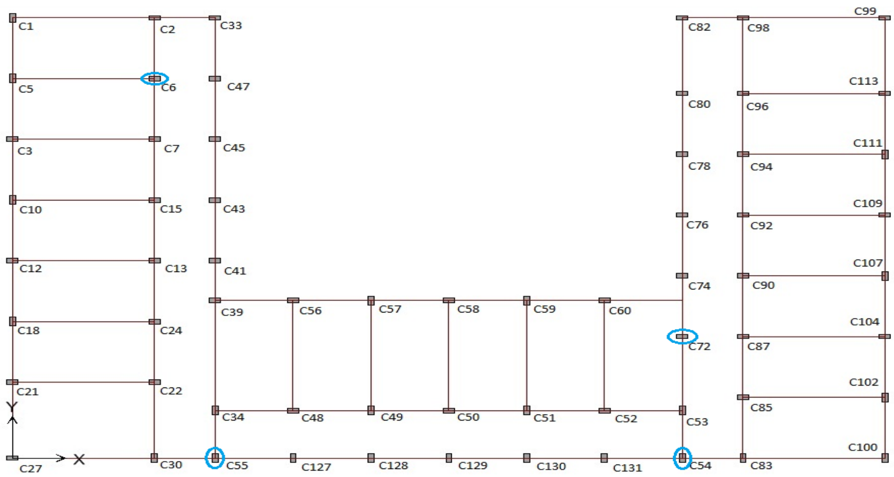

Figure 5.7: Column Grouping 2

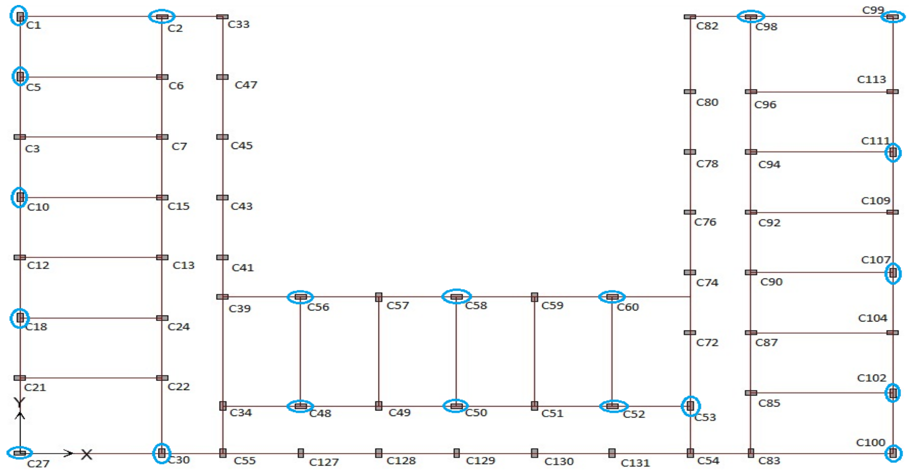

Figure 5.8: Column Grouping 3

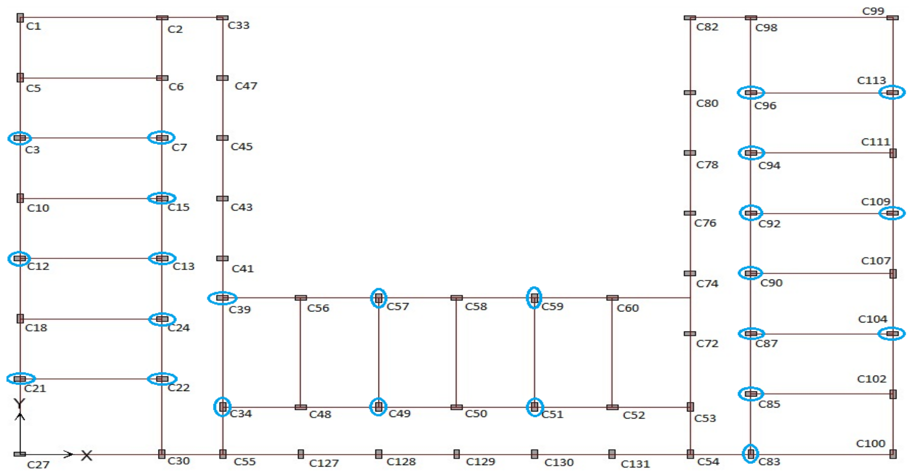

Figure 5.9: Column Grouping 4

d) To Find deficiency

Deficiency means comparison between new required capacity and existing capacity and find out which member will fail due to new load and moment. See in below tables, existing moment is less than new moments. Thus member will only fail due to moment because new earthquake load is not considered in old code. This school building made before 45 years.

Below tables of column group gives idea about which member required to be jacketed is shown by dark colours:

GROUP 1

		New analysis with EQ			Existing Capacity			Deficiency		
Story	Point	FZ	MX	MY	FZ	MX	MY	FZ	MX	MY
BASE	**33**	**386.09**	**0.05**	**65.86**	**1154**	**38**	**29**	**-767.91**	**-37.95**	**36.86**
BASE	41	273.17	20.78	0.09	1154	38	29	-880.83	-17.22	-28.90
BASE	43	277.8	20.45	0.04	1154	38	29	-876.2	-17.55	-28.96
BASE	45	292.87	20.52	0.08	1154	38	29	-861.13	-17.48	-28.92
BASE	47	314.39	20.37	0.27	1154	38	29	-839.61	-17.62	-28.72
BASE	74	442.32	26.48	1.67	1154	38	29	-711.68	-11.51	-27.32
BASE	76	292.48	20.69	0.06	1154	38	29	-861.52	-17.30	-28.93
BASE	78	286.23	20.54	0.05	1154	38	29	-867.77	-17.45	-28.94
BASE	80	343.84	20.38	0.52	1154	38	29	-810.16	-17.61	-28.48
BASE	**82**	**426.57**	**0.97**	**66.88**	**1154**	**38**	**29**	**-727.43**	**-37.03**	**37.88**
BASE	127	279.31	0.78	20.92	1154	38	29	-874.69	-37.21	-8.07
BASE	128	291.82	0.13	20.97	1154	38	29	-862.18	-37.86	-8.02
BASE	129	292.03	0.45	20.99	1154	38	29	-861.97	-37.55	-8.00
BASE	130	291.12	1.02	20.99	1154	38	29	-862.88	-36.98	-8.00
BASE	131	287.48	1.53	20.98	1154	38	29	-866.52	-36.46	-8.01

GROUP 2

		New analysis with EQ			Existing Capacity			Deficiency		
Story	Point	FZ	MX	MY	FZ	MX	MY	FZ	MX	MY
BASE	**6**	**664.35**	**0.41**	**61.84**	**1154**	**38**	**29**	**-489.65**	**-37.59**	**32.84**
BASE	**54**	**428.37**	**66.64**	**0.11**	**1154**	**38**	**29**	**-725.63**	**28.65**	**-28.89**
BASE	**55**	**420.32**	**68.27**	**0.47**	**1154**	**38**	**29**	**-733.68**	**30.27**	**-28.52**
BASE	72	612.35	28.68	0.22	1154	38	29	-541.65	-9.32	-28.78

GROUP 3

		New analysis with EQ			Existing Capacity			Deficiency		
Story	Point	FZ	MX	MY	FZ	MX	MY	FZ	MX	MY
BASE	**1**	**688.13**	**61.06**	**8.79**	**1154**	**38**	**29**	**-465.87**	**23.06**	**-20.21**
BASE	**2**	**693.08**	**2.01**	**50.19**	**1154**	**38**	**29**	**-460.92**	**-35.99**	**21.19**
BASE	**5**	**634.82**	**68.04**	**2.22**	**1154**	**38**	**29**	**-519.18**	**30.04**	**-26.77**
BASE	**10**	**558.85**	**64.49**	**1.57**	**1154**	**38**	**29**	**-595.15**	**26.49**	**-27.42**
BASE	**18**	**555.29**	**64.36**	**1.67**	**1154**	**38**	**29**	**-598.71**	**26.36**	**-27.32**
BASE	**27**	**653.77**	**2.18**	**70.81**	**1154**	**38**	**29**	**-500.23**	**-35.81**	**41.81**
BASE	**30**	**787.41**	**54.91**	**8.65**	**1154**	**38**	**29**	**-366.59**	**16.91**	**-20.34**
BASE	**48**	**635.95**	**1.91**	**65.82**	**1154**	**38**	**29**	**-518.05**	**-36.09**	**36.82**
BASE	50	658.23	16.84	0.66	1154	38	29	-495.77	-21.15	-28.34
BASE	**52**	**628.18**	**1.85**	**65.01**	**1154**	**38**	**29**	**-525.82**	**-36.14**	**36.01**
BASE	**53**	**546.8**	**56.86**	**1.43**	**1154**	**38**	**29**	**-607.2**	**18.86**	**-27.56**
BASE	**56**	**665.81**	**1.91**	**70.85**	**1154**	**38**	**29**	**-488.19**	**-36.08**	**41.85**
BASE	58	620.88	17.38	0.163	1154	38	29	-533.12	-20.61	-28.83
BASE	60	617.53	17.86	0.07	1154	38	29	-536.47	-20.13	-28.93
BASE	**98**	**699.24**	**3.91**	**48.08**	**1154**	**38**	**29**	**-454.76**	**-34.08**	**19.08**
BASE	**99**	**673.41**	**4.70**	**76.54**	**1154**	**38**	**29**	**-480.59**	**-33.29**	**47.54**
BASE	**100**	**689.74**	**58.35**	**10.12**	**1154**	**38**	**29**	**-464.26**	**20.35**	**-18.87**
BASE	**102**	**638.46**	**68.52**	**2.1**	**1154**	**38**	**29**	**-515.54**	**30.52**	**-26.9**
BASE	**107**	**558.71**	**64.86**	**1.55**	**1154**	**38**	**29**	**-595.29**	**26.86**	**-27.44**
BASE	**111**	**552.29**	**64.67**	**1.65**	**1154**	**38**	**29**	**-601.71**	**26.67**	**-27.34**

GROUP 4

		New analysis with EQ			Existing Capacity			Deficiency		
Story	Point	FZ	MX	MY	FZ	MX	MY	FZ	MX	MY
BASE	**3**	**780.51**	**1.37**	**75.59**	**1154**	**38**	**29**	**-373.49**	**-36.62**	**46.59**
BASE	**7**	**830.55**	**0.56**	**75.25**	**1154**	**38**	**29**	**-323.45**	**-37.43**	**46.25**
BASE	**12**	**776.5**	**1.39**	**76.73**	**1154**	**38**	**29**	**-377.5**	**-36.60**	**47.73**
BASE	**13**	**822.58**	**0.54**	**76.83**	**1154**	**38**	**29**	**-331.42**	**-37.45**	**47.83**
BASE	**15**	**680.66**	**0.58**	**62.31**	**1154**	**38**	**29**	**-473.34**	**-37.41**	**33.31**
BASE	**21**	**872.67**	**2.65**	**75.83**	**1154**	**38**	**29**	**-281.33**	**-35.35**	**46.83**
BASE	**22**	**901.57**	**0.37**	**76.97**	**1154**	**38**	**29**	**-252.43**	**-37.62**	**47.97**
BASE	**24**	**677.51**	**0.61**	**64.13**	**1154**	**38**	**29**	**-476.49**	**-37.38**	**35.13**
BASE	**34**	**706.91**	**52.59**	**1.07**	**1154**	**38**	**29**	**-447.09**	**14.59**	**-27.92**
BASE	**39**	**785.17**	**9.21**	**61.87**	**1154**	**38**	**29**	**-368.83**	**-28.78**	**32.87**
BASE	**49**	**814.37**	**76.01**	**0.22**	**1154**	**38**	**29**	**-339.63**	**38.01**	**-28.77**
BASE	**51**	**813.99**	**75.42**	**0.26**	**1154**	**38**	**29**	**-340.01**	**37.42**	**-28.73**
BASE	**57**	**777.05**	**77.61**	**0.04**	**1154**	**38**	**29**	**-376.95**	**39.61**	**-28.96**
BASE	**59**	**778.06**	**77.61**	**0.04**	**1154**	**38**	**29**	**-375.94**	**39.61**	**-28.95**
BASE	**83**	**774.91**	**52.28**	**8.22**	**1154**	**38**	**29**	**-379.09**	**14.28**	**-20.77**
BASE	**85**	**666.1**	**0.54**	**64.52**	**1154**	**38**	**29**	**-487.9**	**-37.45**	**35.52**
BASE	**87**	**829.6**	**0.33**	**77.43**	**1154**	**38**	**29**	**-324.4**	**-37.66**	**48.43**
BASE	**90**	**681.11**	**0.33**	**63.00**	**1154**	**38**	**29**	**-472.89**	**-37.66**	**34.00**
BASE	**92**	**822.39**	**0.36**	**76.33**	**1154**	**38**	**29**	**-331.61**	**-37.63**	**47.33**
BASE	**94**	**677.46**	**0.27**	**62.50**	**1154**	**38**	**29**	**-476.54**	**-37.72**	**33.50**
BASE	96	1060.04	0.69	10.60	1154	38	29	-93.96	-37.30	-18.39
BASE	**104**	**781.88**	**1.78**	**77.82**	**1154**	**38**	**29**	**-372.12**	**-36.21**	**48.82**
BASE	**109**	**775.53**	**1.77**	**76.28**	**1154**	**38**	**29**	**-378.47**	**-36.22**	**47.28**
BASE	**113**	**877.48**	**0.4**	**73.47**	**1154**	**38**	**29**	**-276.52**	**-37.6**	**44.47**

e) Identifying of column members which require retrofitting:

Above tables indicates that which member is deficient and which member will need jacketing.Column number 22 from group 4 is taken for design of jacketing is shown in Appendix A and same as for other groups members.Column size can be increased 100 mm from all four side, to provide ease in construction in terms of placing of main bar, stirrup,filling and vibrating during concreting and to maintain adequate cover.Here, below tables gives information about which member of column group need jacketing:

Table 5.2: **Jacketing Columns Location**

Group	No. of Column which are jacketing	Jacketing Size (ALL around 100 mm)
Group 1	33,82	430 × 650 mm
Group 2	6,54,55	430 × 650 mm
Group 3	1,2,5,10,18,27,30,48,52,53,56, 98,99,100,102,107,111	430 × 650 mm
Group 4	3,7,12,13,15,21,22,24,34,39,49,51,57, 59,83,85,87,90,92,94,104,109,113	430 × 650 mm

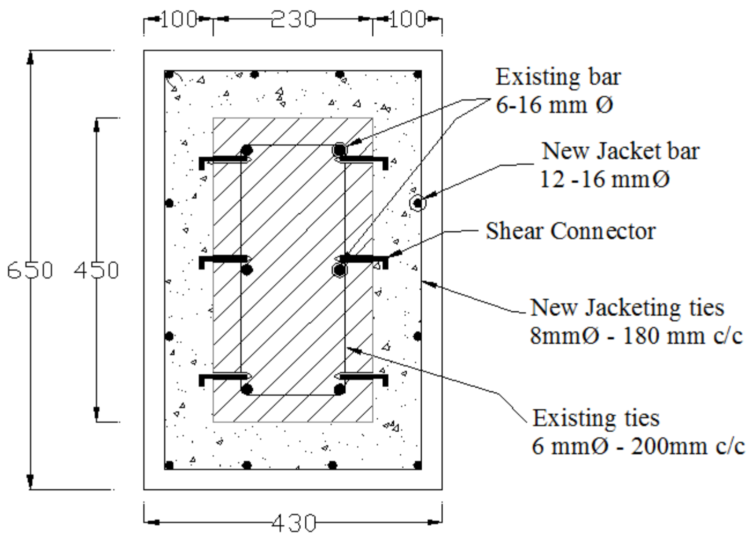

Figure 5.10: Reinforcement of Jacketing Column

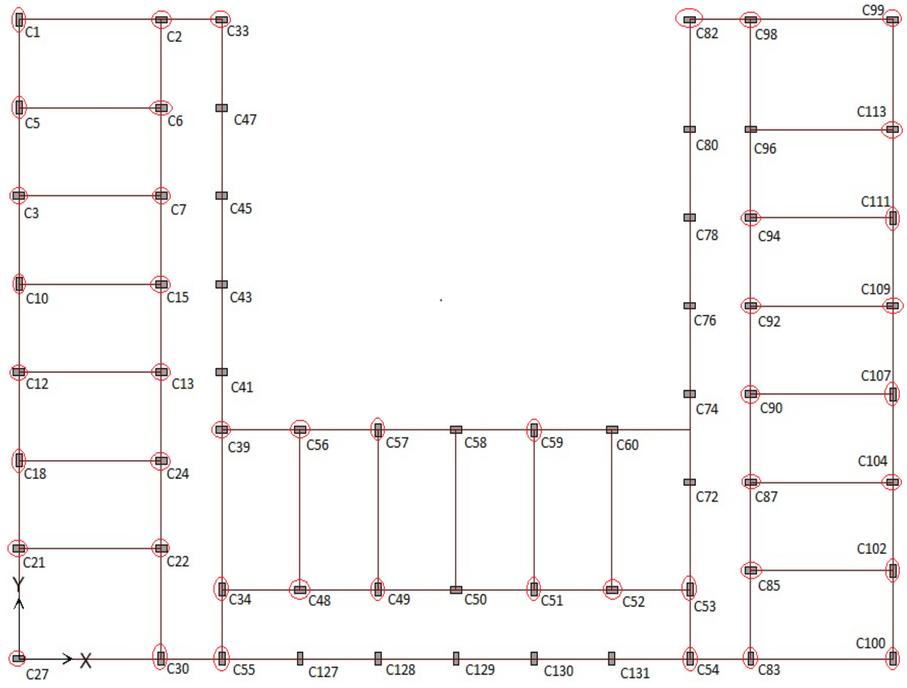

Figure 5.11: Jacketing Column location in Plan

Results comparison between Existing structure and Retrofitted structure

Existing Total Dead load = 22045 kN

After Retrofit, Total Dead load = 25176 kN

Time Period = 0.159 (Manually)

Time Period = 0.567 (Existing structure)

Time Period = 0.38 (Retrofit structure)

Displacement = 7.4 mm (Existing structure)

Displacement = 4.1 mm (Retrofit structure)

5.6 Procedure for Retrofitting of Beam Using GFRP

Procedure of Retrofitting of beam (As per ACI 440.2R) is divided into 5 step as below:

a. To Find capacity of existing beam

b. To Analyze the structure using ETABS

c. To Find deficiencies

d. To Identify the Beam which require retrofitting

a) To Find capacity of existing beam

f_{ck} = 15 N/mm^2 , f_y = 415 N/mm^2

Size of beam = 230 × 600 mm

Assume p_t = 0.857% (structure drawing not available and reinforcement not shown by profometer)

A_{st} = 2-25mm ϕ + 1-16mm ϕ =1183 mm^2

Cover = 30 mm (from profometer) , Stirrup size = 6 mm

Find out moment Capacity:

d' = 30 + 6 + 12 = 48 mm
d'/d = 48/600 = 0.08

Moment Capacity as per SP16 Table-49,pg-85

$\frac{M_u}{bd^2} = 2.53$

$M_u = 2.53 \times 230 \times 600^2 = 209$ kN.m

b) To Analyze the structure using ETABS:

3d analysis in ETABS with revised loads is done.

d) Find deficiency:

After comparing the moment given by Etabs and existing moment capacity of beam, Deficiencies are being carried out.

e) Identify of beam member which requires retrofitting:

6 members of beam are required to retrofitting as per deficiency shown in fig. 5.12. The retrofitting is proposed with GFRP as per ACI 440.2R. Design of only one member is shown here and same as for other 5 members.

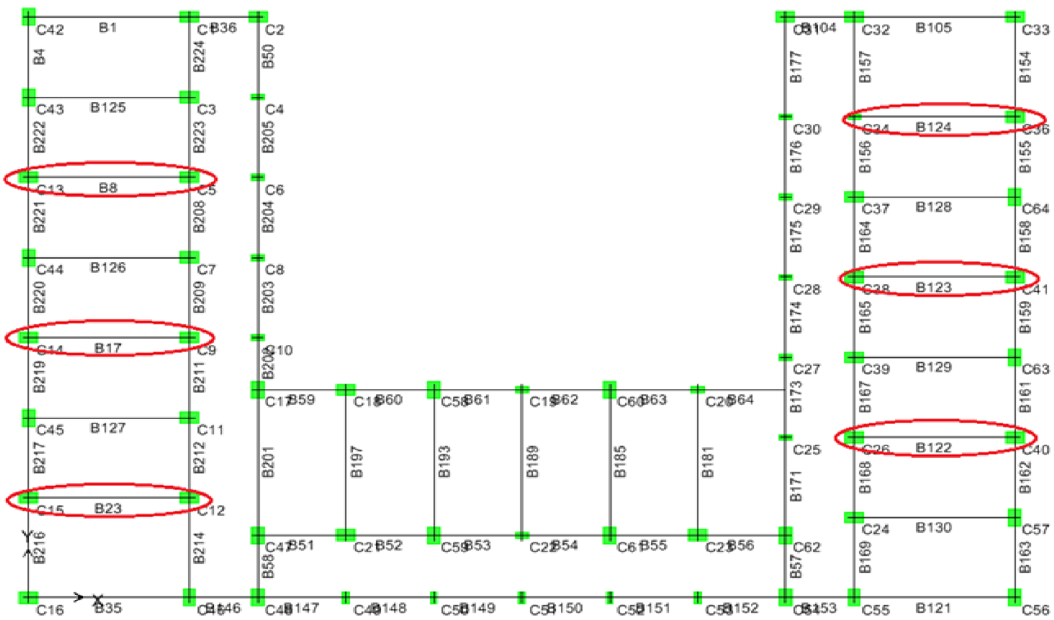

Figure 5.12: Location of Wrapping on Beam in Plan

5.6.1 Design of Flexural Strengthening of Beam as per ACI 440.2R

Data :

length of beam = 5.45m, Width of beam = 230 mm

Depth of beam D = 600 mm, Effective depth d = 566 mm

Clear cover = 20 mm

Stirrup = 6 mm

f_{ck} = 25 N/mm^2, f_y = 415 N/mm^2

Assume Steel in beam = 2-25 mm + 1-16 mm

NOS of bar = 3

Assume no. of plies = 1

Thickness per ply t_f = 1.02 mm

Ultimate tensile strength f_{fu}= 621.00 N/mm^2

Rupture strain ε^*_{fu}= 0.015 mm/mm

Modulus of elasticity of FRP laminates E_f = 37000 N/mm^2

Loadings	Existing	Anticipated
D.L =	3.269 N/mm	3.91 N/mm (20 % increase)
L.L =	0.69 N/mm	0.69 N/mm
D.L + L.L =	3.959 N/mm	4.60 N/mm
1.1D.L + 0.75LL =	4.114 N/mm	4.819 N/mm
Factored 1.2DL+1.6LL =	5.027 N/mm	5.796 N/mm
M_{DL}	12.138 kN.m	14.517 kN.m
M_{LL}	2.562 kN.m	2.562 kN.m
M_s	14.70 kN.m	17.08 kN.m
Unstregthened Moment M_u	22.05 kN.m	25.618 kN.m

STEP 1: FRP SYS DESIGN MATERIAL PROPERTIES

Environment reduction factor $C_E = 0.950$

Design ultimate tensile strength $f_{fu} = C_E f^*_{fu} = 589.95$ N/mm^2

design rupture strain $\varepsilon_{fu} = C_E \varepsilon^*_{fu} = 0.014$ mm/mm

STEP 2: PRELIMINARY CALCULATIONS

Factor relating depth of equivalent $\beta 1 = 1.05$ - $0.05\frac{f_c^{'}}{6.9} = 0.869$

$E_c = 23500.000\ N/mm_2$

$A_s = 942\ mm^2$

width $w_f = 230$ mm

Area of GFRP external reinforcement $A_f = n * t_f * w_f = 1 * 1.02 * 230 = 234.600 mm^2$

STEP 3: DETERMINE EXISTING STATE OF STRAIN ON SOFFIT

Ratio of depth of neutral axis to reinforcement depth measured from extreme compression fiber k = 0.334

$n_s = 8.511$

E_s= 200000 N/mm^2

distance from extreme compression fiber to the neutral axis, C = 187.54 mm

moment of inertia of cracked section transformed to concrete

$I_{cr} = 505702690 + 1751785962 = 2257488651\ mm^4$

strain level in concrete substrate at time of FRP installation

$\varepsilon_{bi} = \frac{M_{DL}(D_f - K_d)}{I_{cr} E_c}$ $\varepsilon b_i = 0.000129$

STEP 4: DETERMINE THE DESIGN STRAIN OF FRP SYSTEM

debonding strain of externally bonded FRP reinforcement

$\varepsilon_{fd} = 0.41 * \sqrt{\frac{25}{2*37000*1.02}}$

$\varepsilon_{fd} = 0.007 < 0.013$

STEP 5: ESTIMATE DEPTH OF NEUTRAL AXIS

c = 0.20 d = 0.20*564 = 112.30 mm

STEP 6: DETERMINE EFFECTIVE LEVEL OF STRAIN IN FRP R/F

$\varepsilon_{fe} = 0.003(\frac{d_f - c}{c}) - \varepsilon_{bi} \leq \varepsilon_{fd}$

$\varepsilon f_e = 0.013 \leq 0.007$

$\varepsilon f_e = \varepsilon f_d = 0.007$

strain level in concrete $\varepsilon_c = (\varepsilon f_e + \varepsilon b_i)(\frac{c}{d_f - c})$

$\varepsilon_c = 0.00175$

STEP 7: CALCULATE STRAIN IN EXISTING R/F STEEL

strain level in nonprestessed steel reinforcement

$\varepsilon_s = (\varepsilon f_e + \varepsilon b_i)(\frac{d-c}{d_f - c})$ εs = 0.0070

STEP 8 : CALCULATE STRESS LEVEL IN R/F STEEL AND FRP

stress in nonprestressed steel reinforcement

f_s = (200 kN/mm^2) * 0.0070 = 1.39 kN/mm_2

f_s = 1.39 kN/mm^2

effective stress in the FRP stress level attained at section failure

f_{fe} = 0.28 kN/mm^2

STEP 9 : CALCULATE THE INTERNAL FORCE RESULTANTS AND CHECK EQUILIBRIUM

maximum strain of unconfined concrete corresponding to $f_c^{'}$

$\varepsilon_c^{'} = 1.7\frac{f_c^{'}}{E_c}$

$\varepsilon^{'}{}_c = 0.002$

$\beta_1 = \frac{4\varepsilon_c^{'} - \varepsilon_c}{6\varepsilon_c^{'} - 2\varepsilon_c}$

$\beta_1 = 0.745$

multiplier on f_c' to determine intensity of an equivalent rectangular stress distribution

$\alpha_1 = \frac{3\varepsilon_c' \varepsilon_c - \varepsilon_c^2}{3\beta_1 \varepsilon_c'^2}$

$\alpha_1 = 0.877$

INITIAL ESTIMATE OF c

$c = \frac{A_s f_s + A_f f_{fe}}{\alpha_1 f_c' \beta_1 b}$ c = 180 mm > 112 mm (Safe)

STEP 10: CALCULATE FLEXURAL STRENGTH COMPONENTS

contribution of steel reinforcement to nominal flexural strength

$M_{ns} = A_s f_s (d - \frac{\beta_1 c}{2})$

$M_{ns} = 302.05$ kN.m

contribution of FRP reinforcement to nominal flexural strength

$M_{nf} = A_f f_{fe} (d_f - \frac{\beta_1 c}{2})$

$M_{nf} = 34.52$ kN.m

STEP 11 CALCULATE DESIGN FLEXURAL STRENGTH OF SECTION

strength reduction factor = = 0.9

$\phi M_n = \phi[M_{ns} + \psi M_{nf}] = 298.26$ kN.m > 25.618 k N - m (Safe)

strengthened section is capable of sustaining new required moment strength

STEP 12: CHECK SERVICE STRESSES IN R/F STEEL AND FRP

ρs = 0.0046

ρf = 0.0030

$E_s = 200$ kN/mm^2

$E_c = 23.50$ kN/mm^2

$E_f = 37$ kN/mm^2

$d_f = 600.0$ mm

d = 566.0 mm

k = 0.257

k_d = 145.56 mm

CALCULATION OF STRESS LEVEL IN R/F STEEL

M_s = 6.776 k N - m

εb_i = 0.00006

A_f = 234.60 mm^2

E_f = 37 kN/mm^2

d_f = 600.00 mm

d = 566.00 mm

k_d = 145.56 mm

E_s = 200 kN/mm^2

A_s = 602.88 mm^2

$f_{s,s}$ = 20.97 N/mm^2 <332 N/mm^2

Stress level in R/F Steel is within recommended limit

STEP 13: CHECK CREEP RUPTURE LIMIT AT SERVICE OF THE FRP

$f_{fu} = 590$

$f_{f,s} = f_{s,s}(\frac{E_f}{E_s})(\frac{d_f-kd}{d-kd}) - \varepsilon_{bi}E_f$

$f_{f,s} = 1.277 < 324.4725$ N/mm^2

Stress level in FRP is within recommended limit

Table 5.3 shows which members of beam are required gfrp wrapping and how many plies are required for wrapping.

Table 5.3: **Location of Wrapping on Beam Member**

Floor	No. of Beam which requires FRP wrapping	No. of plies (As per design)
Ground	8	1
Ground	17	1
Ground	23	1
Ground	122	1
Ground	123	1
Ground	124	1

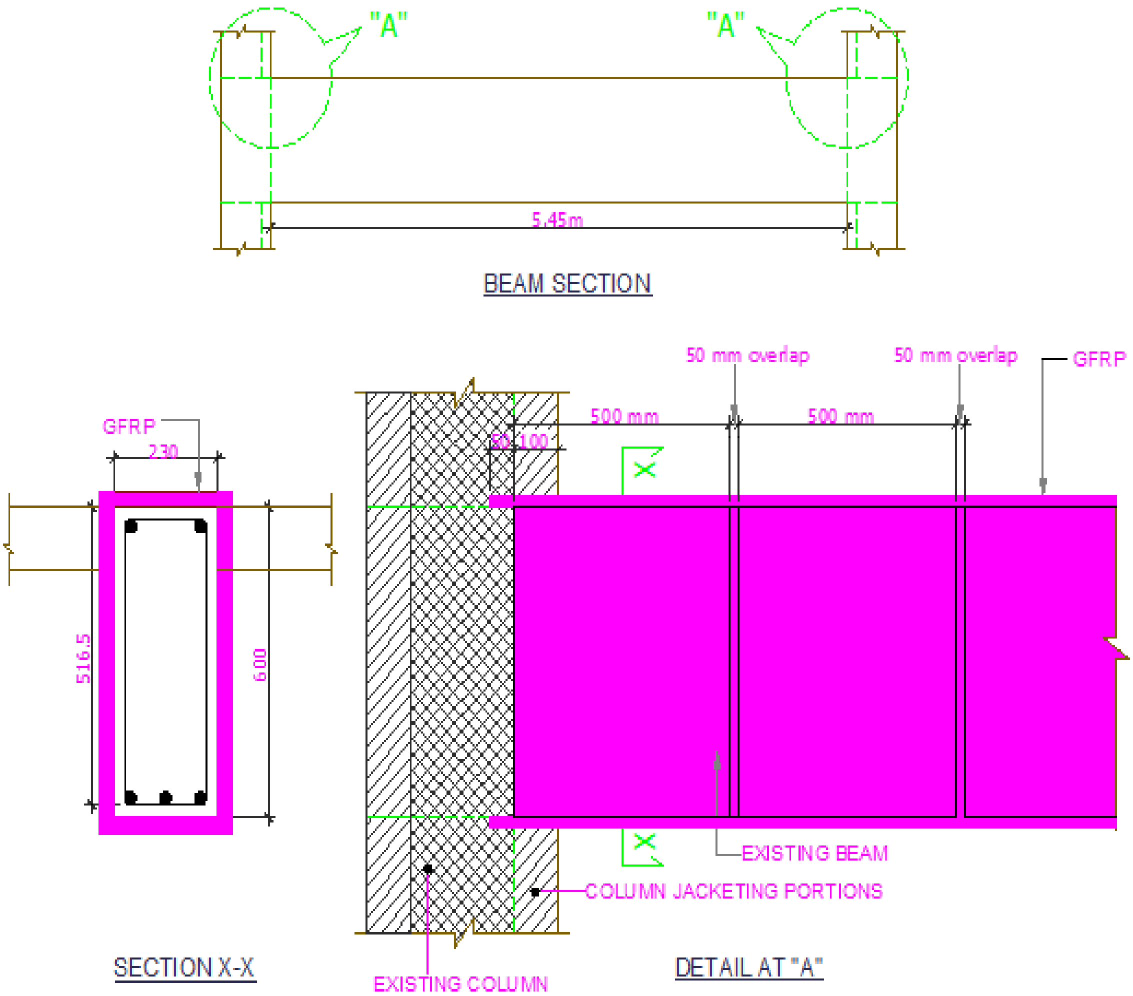

Figure 5.13: GFRP on Beam Member

5.7 Procedure for Retrofitting of Foundation Design

After Jacketing, Old foundation does not resist new moment and axial load, So Strengthening is necessary. Combined footing is the best option for retrofitting of existing building.

Combined footing design for strip number 8 (S-8) of school building is explained here and same as for other strip of building.

Fig. 5.14 shows all 9 strips of building and design of strip number 8 is explained below.

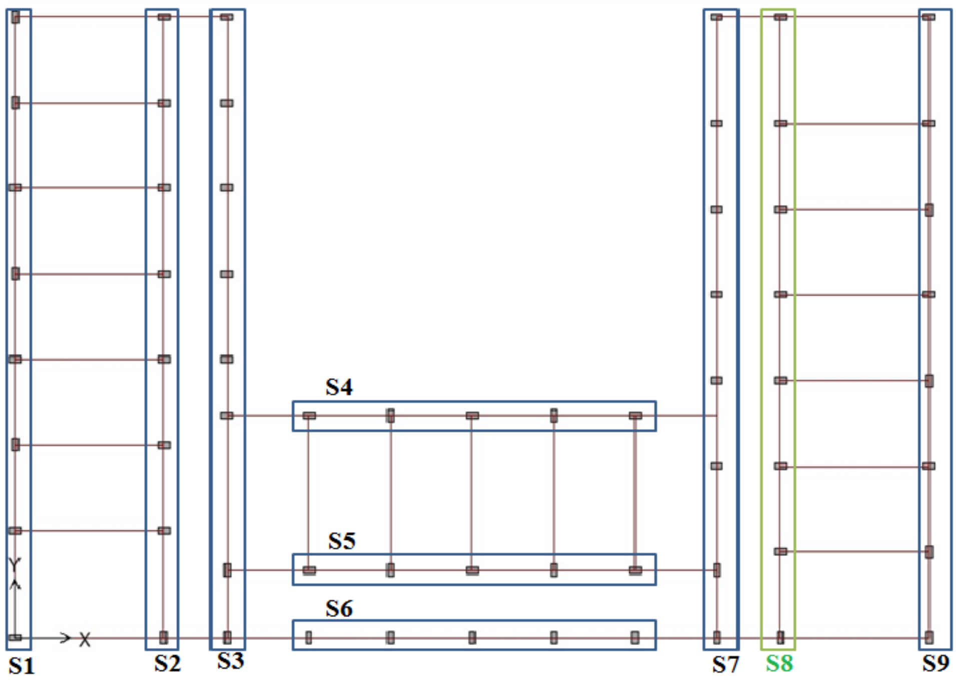

Figure 5.14: Strip of Foundation

Data :

Column size = 430 x 650mm

SBC = 160 kN/m^2

f_y = 415 N/mm^2 , f_{ck} = 20 N/mm^2

Soil density γ = 18 kN/m^3 , Density = 25 kN/m^3

Coefficient of friction μ = 0.4

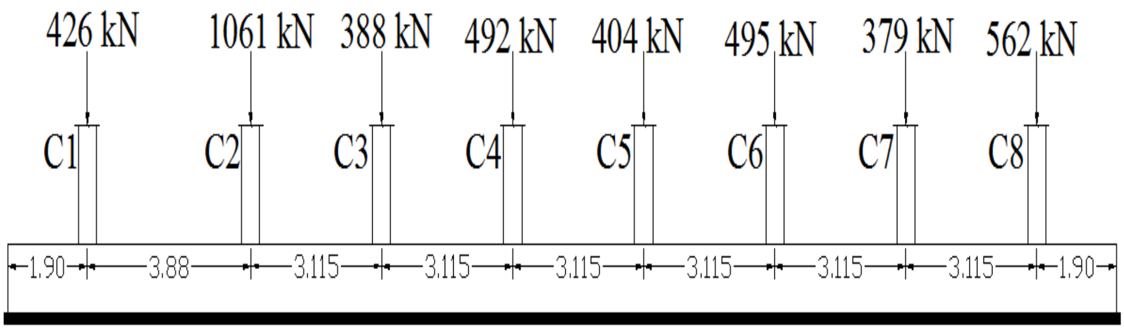

Figure 5.15: Load on Strip of Building

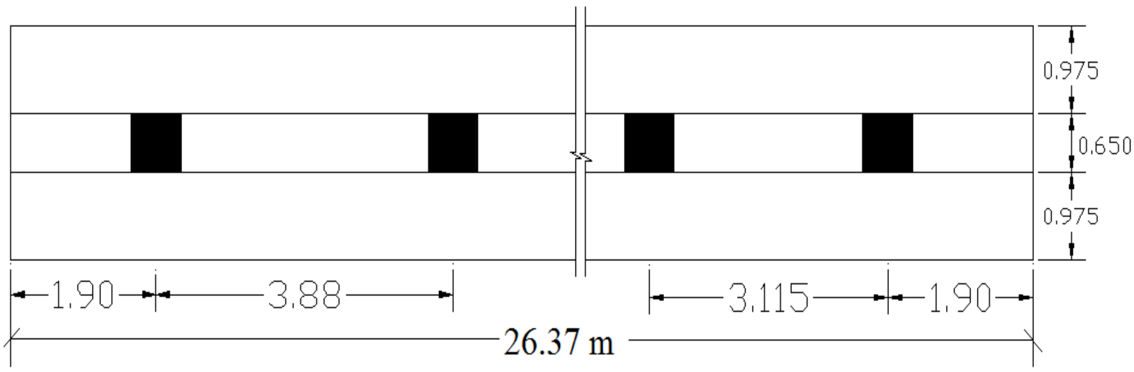

Figure 5.16: Plan of Strip

Above fig.5.15 shows new load on strip due to jacketing of column.

Total load on strip = 426+1061+388+492+404+495+379+562 = 4207 kN

Total load = 4207 kN

Footing self wt @12 % = 505 kN

Consider Load = 4712 kN

Area required = $\frac{4712}{160} = 29.45m^2$

Let width of footing = 2.6 m

C.G of loads from centre of C1

$\overline{X} = \frac{1061\times3.88+388\times6.995+492\times10.11+404\times13.225+495\times16.34+379\times19.455+562\times22.57}{4712}$

$\overline{X} = 9.607$ m

Extend the footing beyond center of C1 and C8 = 1.9 m

Provide 650 mm wide beam.

$p_{upmax} = \frac{P}{A} + \frac{M}{Z}$

$p_{upmax} = \frac{4712}{26.37\times2.6} + \frac{260}{\frac{1}{6}\times26.37^2\times2.6}$

$p_{upmax} = 69.58kN/m^2$

$p_{upmin} = 67.86kN/m^2$

$p_{u,up} = 1.5 \times 69.58 = 104.37kN/m^2$ on slab

$w_{u,up} = 2.6 \times 104.37 = 271.36kN/m$ on beam

Design Slab:

Consider 1 m length of slab

bending moment $M_u = \frac{0.975^2}{2} \times 104.37 = 49.61$ kN.m

$d_{req} = \sqrt{\frac{49.61\times10^6}{2.76\times1000}}$ (2.76 taken from SP16 for $f_y = 415$ and $f_{ck} = 20$)

$d_{req} = 134.06$ mm

D = 370mm (Larger depth for shear)

d = 370 - 50 - 10 = 310 mm

Shear at d = 310mm

$V_u = 0.49 \times 104.37 = 51.14$ kN

$\tau_v = \frac{51.14\times10^3}{1000\times310} = 0.158$ N/mm^2

$\frac{100A_s}{bd} = \frac{100\times491.72}{1000\times310} = 0.158$

$\tau_c = 0.28$

$\tau_v < \tau_c$ $\therefore$ ok.

$\frac{M_u}{bd^2} = \frac{49.61\times10^6}{1000\times310\times310} = 0.51$

Table - 2, SP-16, Page-48, $f_y = 415$ N/mm^2 , $f_{ck} = 20$ N/mm^2

$\therefore p_t = 0.146$ %

$A_{st} = \frac{0.146\times1000\times310}{100} = 453$ mm^2

Assume 10 mm ϕ

$\frac{c/c}{1000} = \frac{a_{st}}{A_{st}}$

$\therefore$ c/c = $\frac{1000\times\frac{\pi}{4}\times10^2}{453}$ = 173.37mm

Provide 10 mm ϕ @ 170 mm c/c = 462 mm^2

Development length

$L_d = 47 \times 12 = 564mm$

Anchorage available = 785-50 = 735mm

Distribution steel = $\frac{0.12}{100} \times 1000 \times 370 = 444\ mm^2$

Provide 8 mm ϕ @ 110 mm c/c = 456 mm^2

Design Beam:

Maximum moment = 625 kN.m

$d_{req} = \sqrt{\frac{625\times10^6}{2.76\times650}} = 590mm$

Try D = 1000 mm

d = 1000-50-20-10 = 920 mm

minimum reinforcement = $\frac{0.205}{100} \times 650 \times 920 = 1226\ mm^2$

$V_u = 565$ kN

$\tau_v = \frac{565\times10^3}{430\times920} = 1.43N/mm^2 < 2.8N/mm^2$

$\frac{100A_s}{bd} = \frac{100\times6\times\frac{\pi}{4}\times20^2}{430\times920} = 0.48$

$\tau_c = 0.475$

$\tau_c bd = 0.475 \times 430 \times 920 = 188$

V_{us} = 565-188 = 377 kN

$\frac{V_{us}}{d} = \frac{377\times1000}{920} = 410N/mm$

Shear resisted by minimum shear reinforcement using $8mm\phi$ two legged stirrups with $A_{sv} = 100mm^2$

$$S_v = \frac{0.87f_yA_{sv}}{0.4b} \tag{5.2}$$

$$S_v = \frac{0.87\times415\times100}{0.4\times430} = 210mm$$

$V_{usmin} = \frac{0.87\times415\times100\times920}{210} \times 10^{-3}$
$V_{usmin} = 158$ kN

Stirrups $8mm\phi - 210mmc/c$.

Calculation of design for flexure are tabulated in table b = 650mm, d = 920mm. Also,Provide 4 - 12 mm ϕ side face reinforcement as the depth of web of beam is more than 735mm

Bottom Reinforcement

Locaion	M_u kN.m	$\frac{M_u}{bd^2}$	$p_t(\%)$	A_{st} mm^2	Reinforcement	
					Bars	mm^2
C1	335	0.61	0.173	1035	4 - 25mm ϕ	1964 mm^2
C2	512	0.93	0.0.275	1645		
C3	420	0.76	0.22	1316		
C4	556	1.01	0.295	1764		
C5	523	0.95	0.28	1674		
C6	398	0.72	0.21	1256		
C7	287	0.52	0.15	897		
C8	335	0.61	0.173	1035		

Top Reinforcement

Locaion	M_u kN.m	$\frac{M_u}{bd^2}$	$p_t(\%)$	A_{st} mm^2	Reinforcement	
					Bars	mm^2
C1-C2	-301	0.55	0.158	945	4 - 20mm ϕ	1257 mm^2
C2-C3	-339	0.62	0.179	1070		
C3-C4	-398	0.72	0.206	1232		
C4-C5	-402	0.73	0.208	1244		
C5-C6	-377	0.69	0.20	1208		
C6-C7	-369	0.67	0.19	1136		
C7-C8	-318	0.58	0.162	969		

Check for Sliding :

wt of soil $= (2 - 1) \times (26.37 \times 2.6 - 0.65 \times 0.43) \times 18$

wt of soil = 1229.08 kN

wt of mat $= 8 \times 0.65 \times 0.43 \times 25 \times 2$

wt of mat = 111.8 kN

Total Axial load = Pu + wt of soil + wt of mat

Total Axial load = 4207 + 1229.08 + 111.8 = 5548 kN

$Load \times \mu = 5548 \times 0.4 = 2220$

Total Horizontal force of strip-1 columns shown in below table :

Column no.	Horizontal Load (kN)
98	17.94
96	16.85
94	20.14
92	60.82
90	20.02
87	61.6
85	22.02
83	13.17

Total Horizontal force (Sliding force) = 216 kN

Factor of safety $= \frac{2220}{216} = 10 > 3 \therefore$ Safe

Reinforcement details of New combined footing is shown in fig.5.17 , 5.18, 5.19 and 5.20.

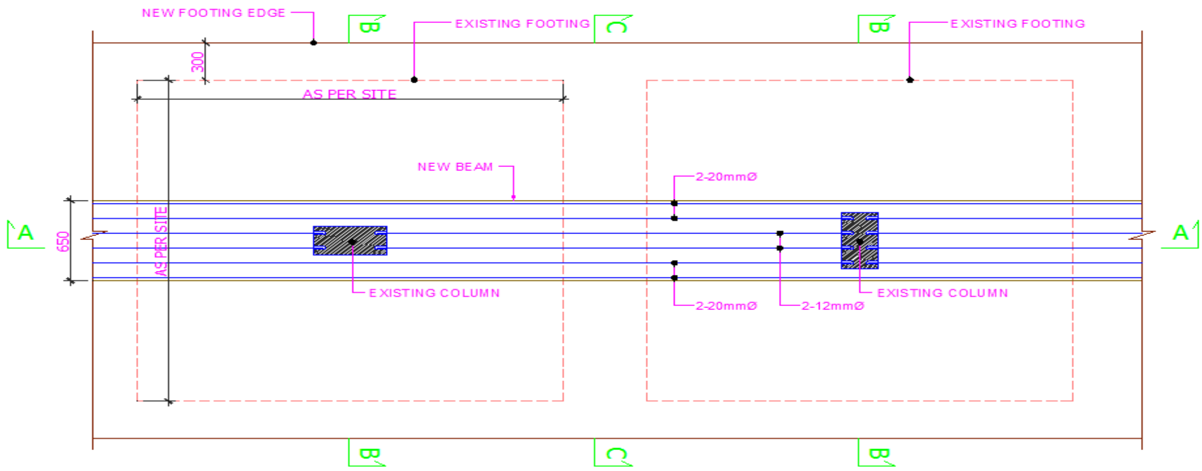

Figure 5.17: Reinforcement of Foundation in Plan

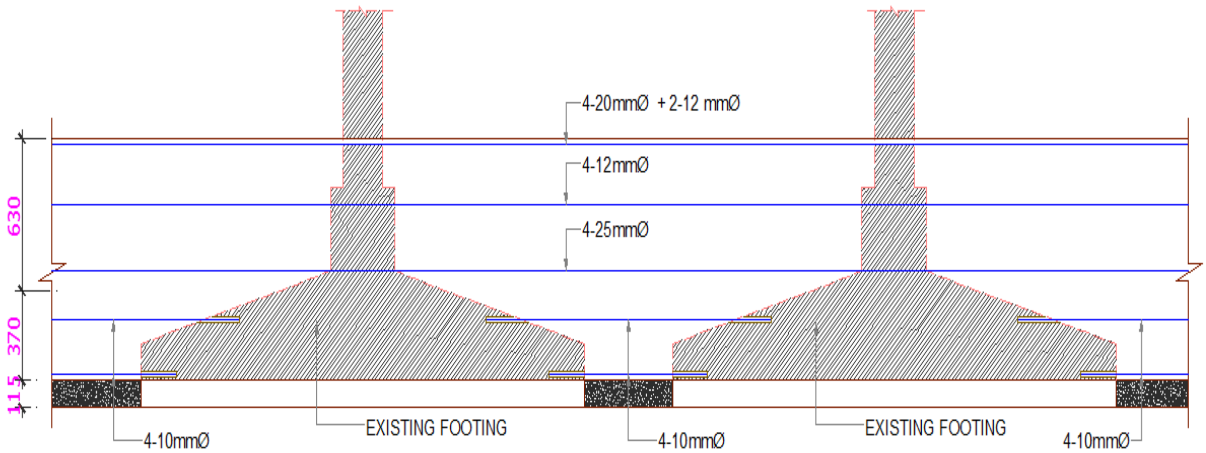

Figure 5.18: Reinforcement of Section A-A

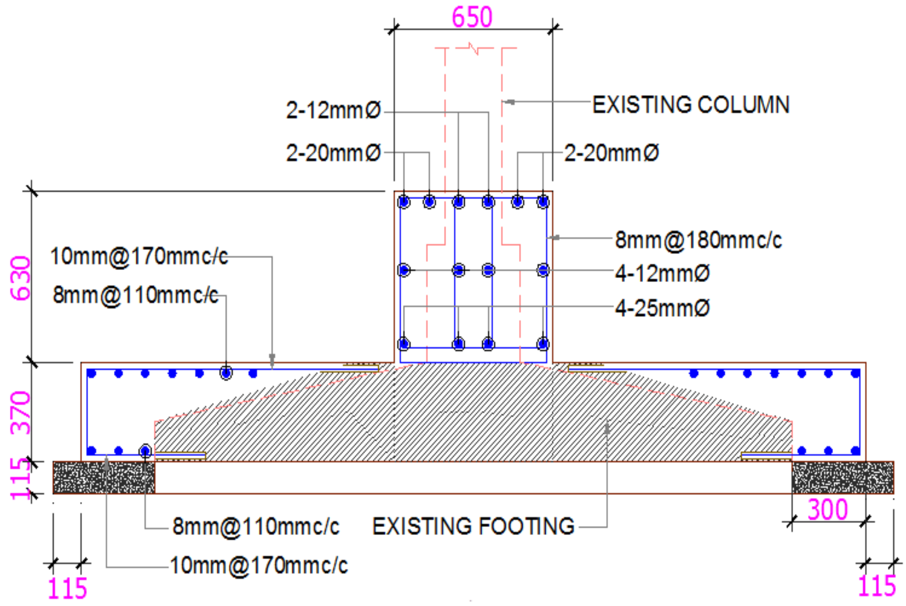

Figure 5.19: Reinforcement of Section B-B

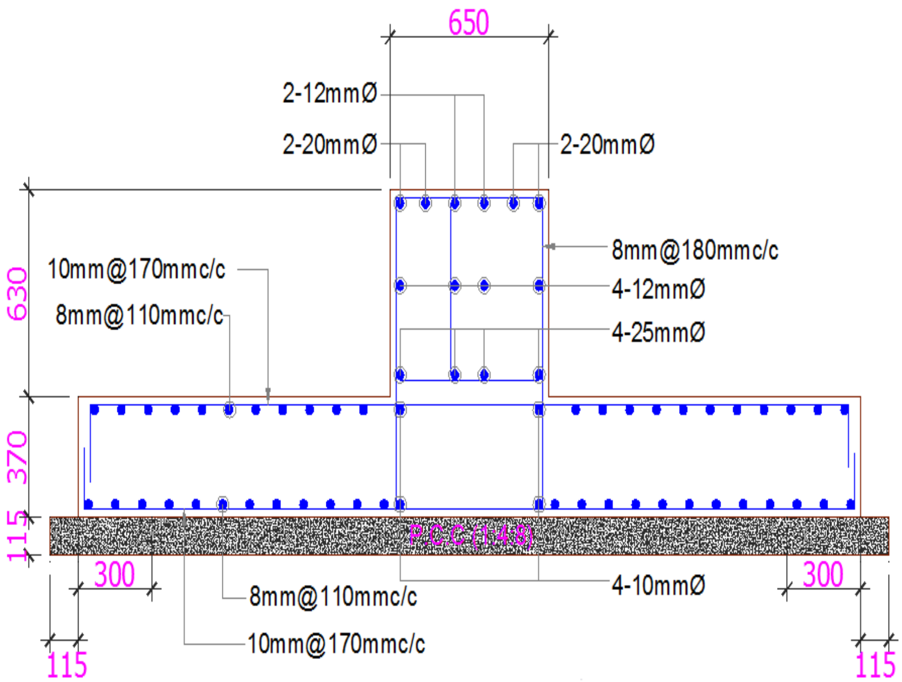

Figure 5.20: Reinforcement of Section C-C

5.8 Procedure of Retrofitting With Additional Shear Wall

- Shear wall helps to reduce Drift and torsion irregularity.
- Shear walls is introduced in such a manner that centre of mass matches with centre of stiffness(rigidity).

Before jacketing of columns, it has been observed that present structure has an eccentricity in both directions as shown in below fig. 5.21

X_{CM} = Centre of mass in X-direction

Y_{CM} = Centre of mass in Y-direction

X_{CR} = Centre of rigidity in X-direction

Y_{CR} = Centre of rigidity in Y-direction

$X_{CM} - X_{CR}$ = 17.45 - 17.57 = - 0.12 m

$Y_{CM} - Y_{CR}$ = 9.28 - 10.06 = - 0.78 m

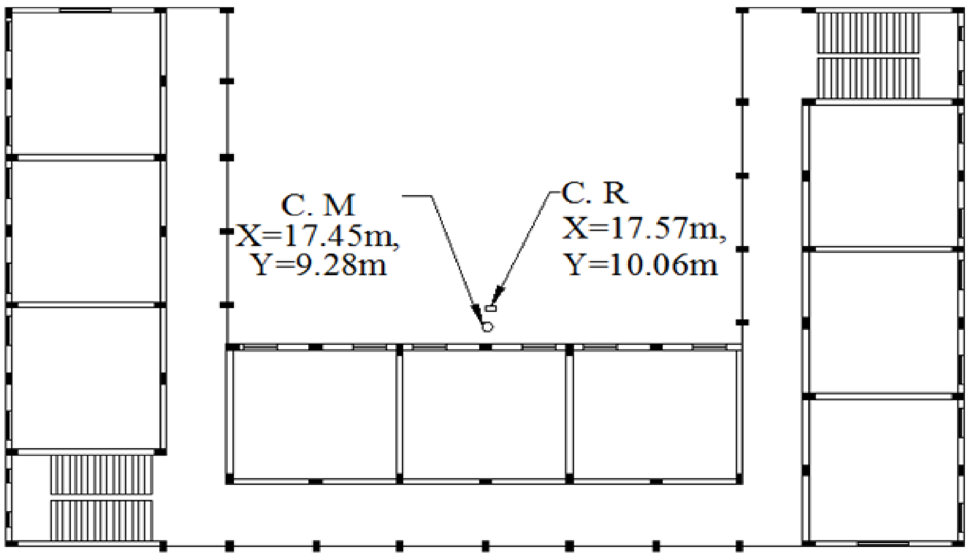

Figure 5.21: Position of C.M and C.R Before Jacketing

After jacketing of columns, it has been observed that eccentricity has been reduced only in Y direction as shown in below fig. 5.22

$X_{CM} - X_{CR}$ = 17.41 - 16.97 = 0.44 m

$Y_{CM} - Y_{CR}$ = 9.43 - 9.64 = - 0.21 m

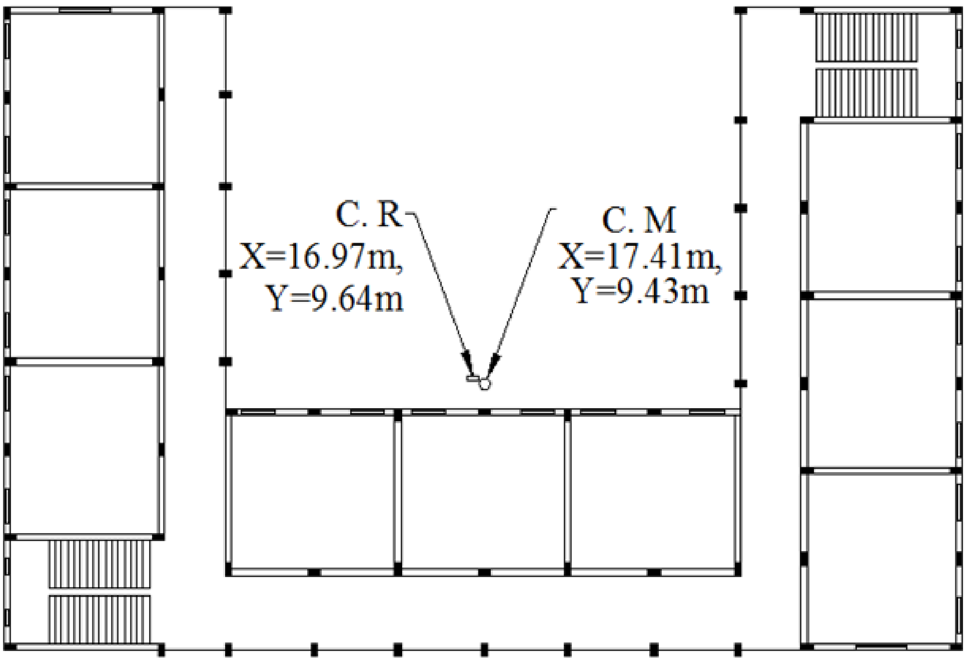

Figure 5.22: Position of C.M and C.R after Jacketing

For present case of school building, The shear wall has been introduced at different location in each case.
Two cases Out of them have been discussed below:

Case : 1 Shear wall is provided in two different location

As shown in fig. 5.23, Shear wall located along Y direction in left wing produce higher eccentricity in negative X-direction.

$X_{CM} - X_{CR}$ = 17.41 - 18.04 = - 0.63 m

$Y_{CM} - Y_{CR}$ = 9.42 - 9.7 = - 0.28 m

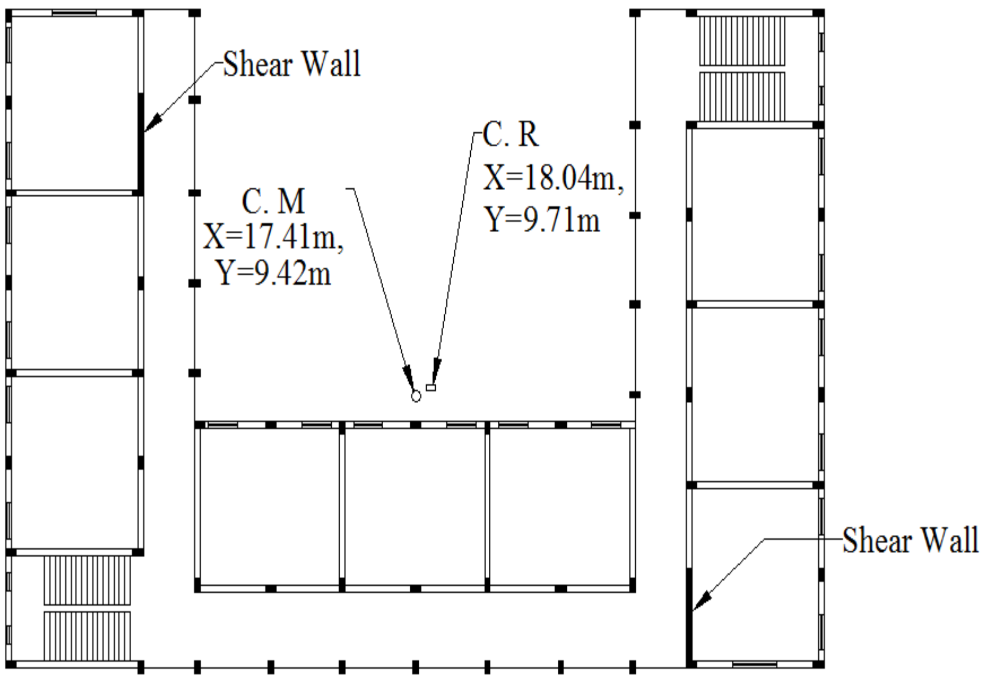

Figure 5.23: Position 1 of Shear Wall

Case : 2 Shear wall is provided in two different location

As shown in fig. 5.24, Shear wall located along Y direction in right wing produce higher eccentricity in positive X-direction.

$X_{CM} - X_{CR}$ = 17.41 - 16.69 = 0.72 m

$Y_{CM} - Y_{CR}$ = 9.42 - 9.65 = - 0.23 m

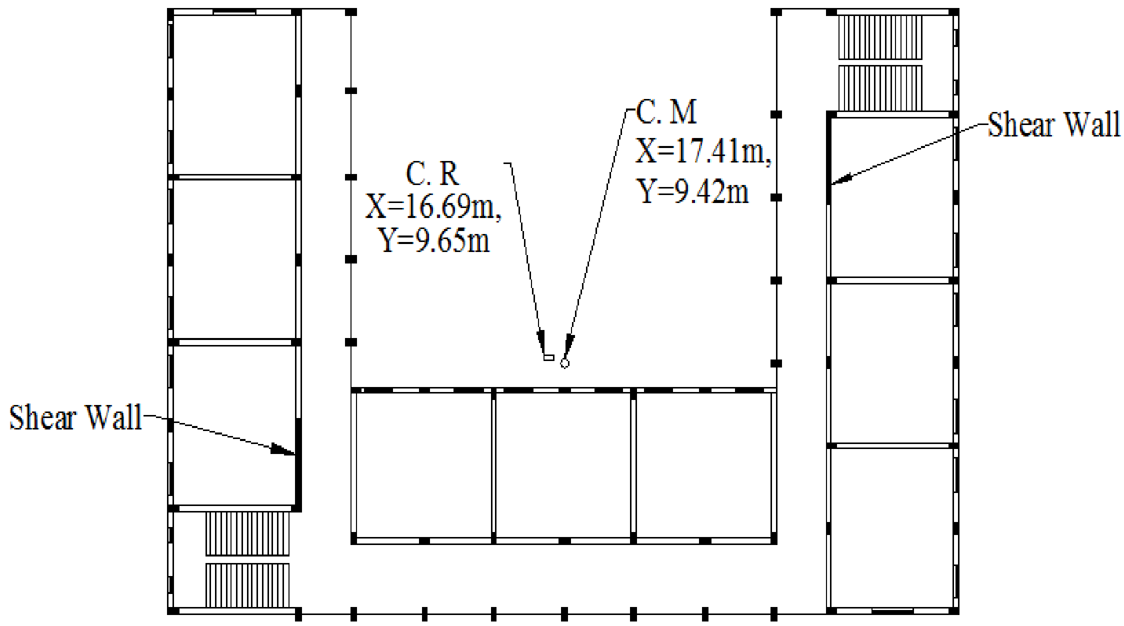

Figure 5.24: Position 2 of Shear Wall

Conclusion:

After placing shear wall in various location, it has been observed that shear wall does not help to minimize the eccentricity.

Providing more number of shear wall would have reduce the torsional irregularity but at the same time it would have not been cost effective solution.

Due to jacketing of column, eccentricity has been reduced in both the direction which could be cost effective solution compare to providing shear wall.

5.9 Summary

In this chapter, study of RCC school building is presented. ETABS software is used for analysis and design of structural members and found deficient member of building. Deficient member is retrofitted by retrofitting techniques such as Jacketing of Columns, GFRP wrapping on beams, combined footing and additional shear wall.

Chapter 6

Summary and Conclusions

6.1 Summary

Every building is having capacity to resist seismic forces to an anticipated seismic demand, if demand is more compared to capacity, building needs to be retrofitted to upgrade its seismic capacity. The objective of the work undertaken is to determine performance of an existing building, for that an existing G+2 RCC school building at Nava vadaj, Ahmedabad was cited.

Non destructive tests are carried out on deficient structural members to check the reinforcement strength strength.The result shows poor structural strength.

After comparing between existing capacity and required capacity, the elements of columns and beams are showing the deficiency

All Columns are decided to be jacketed with concrete grade M25 using shear connector and sizes of column are increased from 230 x 450 mm to 430 x 650 mm. Additional main reinforcement $12-16mm\phi$ and stirrups $8mm\phi$- 180 mm c/c have been provided in each jacketed column.

For Beams strengthening, Glass fiber reinforced polymer (GFRP) strips are provided. 6 number of beams required to be retrofitted with 1 ply of GFRP to increase the flexural capacity.

As the building is having isolated footing only, combined foundation has been provided to take care of additional moment at foundation level due to seismic forces. Combined foundation consists of strap beam connecting series of columns. Base of individual columns has been connected with slab.

The overall performance of the building after retrofitting shows increase in the base shear due to the increased stiffness of the building and reduced relative displacement.

6.2 Conclusion

a. Non destructive tests(NDT) have been performed and following conclusions are obtained:

   (1) After performing half cell potential meter test, corrosion has been observed in existing ground floor columns, chhajja and lintel beam. In addition to that at 2nd floor some part of chhajjas exhibits corrosion.

   (2) After performing Ultrasonic Pulse Velocity test, most of the column shows poor concrete quality.

   (3) After performing rebound hammer test, it has been observed that most of the column at ground storey shows poor concrete strength.

b. From Analysis of existing building with bare frame, it has been observed that 45 numbers of columns are required jacketing.Design of column jacketing is based on IITK-GSDMA GUIDELINES 2005.

c. After analysis of present school building, it has been observed that 6 beams are deficient in flexure and needs to be retrofitted with GFRP plies as per ACI 440.2R.

d. Retrofitting of columns increases lateral load on sub structure. Existing foundation was not design to take care of additional seismic forces. Hence retrofitting of foundation is necessary. Construction point of view combined footing would be ideal option.

e. Building is having irregularity shape, Torsion irregularity has been removed by providing jacketing.

6.3 Future Scope Of Work

Retrofitting of building can be done with other alternatives for examples:

- Steel jacketing of Columns
- CFRP on beams
- Strengthening individual footing
- Expansion joint
- Steel bracing

Appendix A

Design of The Deficient Members of Columns:

Design of only column member 22 from Group 4 is shown here and same as for other groups members.

Design of Columns for Group 4 as per IITK Guidelines

DATA

Size of column b = 430 mm $\times$ 650 mm

compressive strength of concrete f_{ck} = 25 N / mm^2

Characteristic strength of steel f_y = 415 N /mm^2

Axial loadP_u = 901 KN

Moment in X-direction M_{ux} = 0.377 KN.m

Moment in Y-direction M_{uy} = 76 KN.m

Length of column L = 3.5 m

Effective length of columnL_{effx} = 6.7 m

L_{effy} = 6.7 m

Cover = 40 mm

Dia of stirrups = 8 mm

Dia. of Bar = 16 mm

Effective cover = 56 mm

$d^{'}$/D = 0.086

$d^{'}$/b = 0.14

(A) Check for slenderness

L_{effx}/D = 10.31

L_{effy}/D = 15.58

(B) Initial Moments:

(1) Moment Due to Minimum eccentricity

e_{minx}= L/500 + D/30 = 28.37 mm

e_{minx}= L/500 + b/30 = 21.03 mm

$M_{ux,min} = P_u \times e_{minx}$ = 25.59 KN m

$M_{uy,min} = P_u \times e_{miny}$ = 18.97 KN m

(2) Additional moment

M_{ux} = 25.96 KN m

M_{uy} = 94.97 KN m

(C) Check for biaxial bending

$\frac{P_u}{f_{ck} \times b \times d}$ = 0.13

$\frac{M_{ux}}{f_{ck} \times b \times d^2}$ =0.01

$\frac{M_{uy}}{f_{ck} \times b \times d^2}$ =0.02

p/f_{ck} = 0.02 from sp 16, Chart 33

providing p_t = 0.8

$A_s = p_t$ b D /100

$A_s = 2236\ mm^2$

Using 16 mm dia bars for main reinforcement,

no. of bars reqd.= 11.211

no. of bars provd.= 12

no. of bars in each face = 4

Ast provd = 2413 $mm^2 > 2236\ mm^2$ (Safe)

(D)Check for Slenderness Limit

Unsupported length /60 = 56 mm < 650 mm

$P_{uz} = 0.45\ f_{ck}$(bD-A_{sc}) + 0.75 $f_y\ A_{sc}$

$P_{uz} = 3868 > 902 = P_u$

$P_u/P_{uz} = 0.23$

α n = 1

p prov = 0.86 %

p prov /f_{ck} = 0.035

From Chart no. 32 of SP16

M_{ux}/ (f_{ck} b D^2) = 0.05

M_u = 227 kN.m > M_{uy} ($\therefore$ Safe) $\frac{M_{ux}}{M_{ux1}}^{\alpha^n} + \frac{M_{uy}}{M_{uy1}}^{\alpha^n} = 0.53 < 1$

So, Safe

(E) Shear Check :

V = 72.5 kN

Moment Capacity of Beam

$M^{br}_{u,lim}$ = 221.6 kN.m

$M^{bl}_{u,lim}$ = 221.6 kN.m

h_{st} = 3.5 m

$V_u = 1.4\frac{M_u^{bl}+M_u^{br}}{h_{st}}$

V_u = 188.97 kN

(F) Check for Shear Strength(Clause 40.4, IS 456:2000)

Assuming reinforcement of one face is in tension

$A_s = 804.224 mm^2$

$\frac{100 A_s}{bD} = 0.29$ %

d = 594 mm

For M25, $\tau_c = 0.386 MPa$

$V_u = V_{us} + \tau_c bd$

$V_u = 90$ kN

$A_{sv} = 100.528 mm^2$ (2 legged)

$V_{usprovd} = \frac{0.87 f_y A_{sv} d}{S_v}$

$V_{usprovd} = 216$ kN

(G) Lateral Ties

Provide 8 mm dia bar

Spacing of bars

S = $[\frac{f_y}{\sqrt{f_{ck}}}]$ $[\frac{d_h}{t_j}]$

provide 8 mm- 180 mm c/c

Jacketing figure is shown in 5.10 and table of column location for jacketing is shown in 5.2

Appendix B

List of Paper Communicated

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