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Retrofitting of Old Buildings Through FRP

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in

STRUCTURAL ENGINEERING

By

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Certificate

This is to certify that **Ms. Apeksha Bhadouria** student of Delhi College of Engineering, Delhi worked under our guidance on major project entitled **"Retrofitting of old building through FRP"** being submitted in partial fulfilment of the requirement for the award of the degree of "**MASTER OF ENGINEERING IN STRUCTURE",** submitted in the department of **CIVIL ENGINEERING,** of Delhi University, Delhi is an authentic record of my work carried out under the supervision of **Prof.(Mrs.) P. R. BOSE, (H.O.D Civil Engg.)** & **Mr. Alok Verma (Lecturer)**, **Deppt. of Civil Engg. Delhi College of Engg., during the session 2005-2006**

The matter embodied in this dissertation has not been submitted by me for award of any other degree.

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Abstract

Strengthening of concrete structures in flexure with externally epoxy-bonded fiber-reinforced plastic (FRP) laminates is becoming an increasingly popular retrofit technique among researchers and engineers worldwide. This has been due to both the needs for maintaining and upgrading essential infrastructure in all the parts of the world, and well known advantages of FRP composite including both corrosion resistance and ease for site handling due to light weight. Flexural Strengthening of RC beams using bonded FRP soffit plates has become popular over the last few years

Plated reinforced concrete structures are a new and unique form of structure which has similar failure mechanisms or behaviours as in both reinforced concrete structures and composite steel and concrete structures.However plated structures also have many new failure mechanisms that are not covered in reinforced concrete and composite design manuals.

FRP plated beams can fail in various modes, which can be broadly classified into flexural failures,shear failures and debonding failures.If debonding failure occurs ,the ultimate tensile strength of the FRP cannot be fully utilized.In plated beams,debonding failures often govern the strength.Many different debonding failure modes are possible and are discussed . Advanced design rules are available for quantifying the various plate debonding mechanisms and consequently the shear and flexural capacities of the plated sections.

Upgradation of code from IS 1893 :1984 to IS 1893(Part I):2002 has made structures deficient.Base shear has increased by about 0.9 times that of old code. Flexural strength has decreased due to increase in the base shear.

A Case study was performed taking storey variation and it was observed that flexural strength should be increased.

Strengthening through soffit plating could be a remedy ,a case study was taken and storey drift was observed to be reduced ,thus retrofitting of beams through FRP could be a measure to retrofit beams

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Chapter 1. Introduction

1.1 Repair, strengthening, retrofit

The issue of upgrading the existing civil engineering infrastructure has been one of great importance for over a decade. Deterioration of bridge decks, beams, girders and columns, buildings, parking structures and others may be attributed to ageing, environmentally induced degradation, poor initial design and/or construction, lack of maintenance, and to accidental events such as earthquakes. The infrastructure’s increasing decay is frequently combined with the need for upgrading so that structures can meet more stringent design requirements (e.g. increased traffic volumes in bridges exceeding the initial design loads), and hence the aspect of civil engineering infrastructure renewal has received considerable attention over the past few years throughout the world. At the same time, seismic retrofit has become at least equally important, especially in areas of high seismic risk.

1.2 Purpose of seismic strengthening

Many structural engineers believe that the purpose of seismic strengthening is to upgrade the structure, to the maximum extent practical, into conformance with the lateral force requirements of the current building code. In reality this is not the purpose of seismic strengthening, but instead a method for achieving seismic upgrade, and often an inappropriate one.

As stated by the Structural Engineers Association of California (12-1) (SEAOC), the purpose of earthquake resistance provisions incorporated into the building codes is to maintain public safety in extreme earthquakes likely to occur at the building's site. Such provisions are intended to safeguard against major failures and loss of life, not to limit damage, maintain functions, or provide for easy repair. Specifically, it is expected that buildings designed to conform with the provisions of the building code would be able to:

- ✓ Resist a minor level of earthquake ground motion without damage;

- ✓ Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some non-structural damage;
- ✓ Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as non-structural damage.

These performance objectives were specifically formulated by SEAOC to apply to a broad range of structures and occupancies, based on trade-offs between public safety and economics. They were intended to apply to the general population of structures likely to be constructed and were specifically formulated under the influence of the seismicity of California, a region subject to frequent moderate magnitude earthquakes and occasional great earthquakes. These objectives can be reasonably attained in the design of new structures by carefully conforming to four basic sets of provisions specified by the code: strength, materials selection, structural detailing, and construction quality.

1.2.1 Seismic Strengthening Considerations

Since current building codes do not in general apply to existing structures, the implicit performance objectives of these codes need not be rigidly adhered to for seismic upgrades. It is therefore extremely important that the structural engineer work with the building owner to carefully define the intended purpose of seismic strengthening based on specific safety and economic performance objectives. These are likely to vary considerably from one structure to another based on several key factors. These factors include:

- ✓ Economic value of the structure and remaining years of service life.
- ✓ Occupancy of the structure including the number of persons at risk within the structure, as well as the potential for structural failure to result in release of hazardous substances and injuries outside the structure.
- ✓ Function of the structure and the economic or societal cost, which would result from loss of service due to earthquake, induced damage.

- ✓ Historic significance of the structure and the effects of seismic upgrades on the cultural resource.
- ✓ The site-specific seismic hazard.
- ✓ The relative cost of achieving upgrades to various criteria.

As an example, most people would agree that it is not appropriate to upgrade an unoccupied warehouse to the same level of reliability as a building with high occupancy. Similarly, a building expected to remain in service for 10 years need not have the same level of reliability as a building expected to provide service for 100 years. Reconciliation of these complex issues requires both qualitative and quantitative evaluation. Selection of appropriate design criteria cannot be made until these evaluations have been performed.

1.2.2 New Design Versus Retrofit Design Approaches

The basic design procedure for new structures consists of the selection of an appropriate level of lateral forces for design purposes, and then providing a complete, appropriately detailed, lateral force resisting system to carry these forces from the mass levels to the foundations. Deformations are checked as a secondary issue, and except for the design of flexible structures, they are not likely to control the design.

Deformation control can be relegated to a secondary consideration in the design of many new structures to code life-safety requirements because the modern materials and ductile detailing practices specified by present codes allow new structures to experience large deformations while experiencing limited damage. Older structures however do not have the advantage of this inherent ductility. Therefore, control of deformations becomes an extremely important issue in the design of seismic retrofits. Given a ground motion criteria, and the desired performance level for that ground motion, the real task of seismic retrofit becomes one of controlling structural deformations, in response to that ground motion, to within acceptable levels. If the objective is to avoid collapse, then deformations must be controlled to an extent where stability of the vertical load carrying system is not lost. If post-earthquake functionality is the objective, then deformations must be controlled to an extent where unrecoverable cracking and bending of structural (and non-structural) elements is small enough to avoid the cosmetic appearance of an unsafe structure. This limited deformation level is necessary to ensure continued operation.

Following a major earthquake, municipal building inspectors (with the assistance of local structural engineers) will perform a rapid screening assessment and make judgments as to which buildings are obviously unsafe, which are obviously safe, and which require further evaluation to ascertain whether the buildings are safe or not. Unless the building is tagged as obviously safe the local government may limit the use of the building until it can be proven safe. There are three primary types of deformations, which must be considered and controlled, in a seismic retrofit design. These are: global deformations, elemental deformations and inter-structural deformations.

Global deformations are the only type explicitly controlled by the building codes and are classically considered by reviewing interstory drift (see Chapter 7). The basic concern is that large inter-story drifts can result in P-delta instabilities. Control of inter-story drift can also e used as a means of limiting damage to nonstructural Elements of a structure (fascia, partitions, ceilings, utilities, etc.). It is less effective as a means of limiting damage to individual structural elements.

Elemental deformations are the amount of distortion experienced by an individual element of a structure such as a beam, column, shear wall, or diaphragm. Building codes have very few provisions to directly control these deformations. They rely on ductility to ensure that individual elements will not adversely fail at the global deformation levels predicted for the structure. In existing structures, with questionable ductility, it is critical to evaluate the deformation of each element and to ensure that expected damage to the element, at the given deformation level, is acceptable. This requirement extends to elements normally considered to participate in the lateral force resisting system as well as those that do not. For example, a common mode of collapse for older concrete structures is a punching shear failure of flat slabs at interior columns . This results from excessive rotation plus vertical accelerations (and induced punching shear concentrations) at the slab-column joint.

Often, the flat system is not considered to participate in the lateral force resisting system for a retrofitted structure. However, if the rotational deformation of these joints is not maintained below a damage threshold, the classic punching shear failure can still occur. Limiting calculated member stresses at realistic estimates of global structural deformation can sometimes control elemental deformations.

Inter-structural deformations are those that relate to the differential movement between elements of the structure. Failures, which result from a lack of such control, include classic failures of masonry walls, which have not been anchored to diaphragms, or failures resulting from bearing connections slipping off beam seats. Building codes control these deformations by requiring interconnection of all portions of structures and the provision of continuity ties. These same "code" techniques can be effective as retrofits for an existing structure. However, in some cases provision of continuity is not practical (for example at an expansion joint of a structure). In such cases, realistic estimate of expected deformations and ensuring that stability is maintained at these deformation levels is the most effective design procedure.

Strengthening of RC beam can be accomplished using several methods

(a)Traditional methods

- ✓ improve the appearance only
- ✓ structure does not regain strength
- ✓ do not arrest further deterioration

(b) Modern methods

- ✓ structure regains strength
- ✓ deterioration is arrested to a great extent
- ✓ arrest further corrosion

Modern method consist of strengthening of structures by

1.Adding new members

- ✓ Shearwall
- ✓ Bracings
- ✓ Buttress

2.Strengthing of Existing members

- ✓ Steel Plate Bonding
- ✓ FRP Plate Bonding

Externally bonded steel plates or FRP plates/ fabrics, since the use of steel plates is associated with some drawbacks such as difficulties of transporting handling and installing, corrosion of plates, limited delivery lengths and the need to prepare the steel surface for bonding, FRP have been adopted as it improves

- ✓ Flexural Strength
- ✓ Shear strength
- ✓ Load carrying capacity
- ✓ Stiffness
- ✓ Ductility

1.3 Historical Background

Fiber-reinforced polymer (FRP) composite materials were first introduced in the early 1940s. Although composite materials have been used in limited architectural applications since the 1950s, their use in construction, and particularly concrete applications, remained a novelty into the 1980s. Some use of FRP reinforcing products was introduced in Europe and Asia in the 1970s and 1980s.

The FRP plate bonding technology was first investigated at Swiss Federal Laboratory for Materials Testing and Research (EMPA) where tests on RC beams strengthened with CFRP plates started in 1984.

1.4 Technology

The pultruded FRP laminate reinforcing consists of bonding the FRP strip with the concrete structure using a high-strength epoxy resin as the adhesive. The FRP strips are manufactured using a pultrusion process. The pultrusion principle is comparable with a continuous press. Normally 24,000 parallel filaments are pulled through the impregnated bath, formed into strips under heat, and hardened. These strips are uni-directional; the fibers are oriented only in the longitudinal direction. Correspondingly, the strip strength in this direction is proportional to the fiber strength and, thus, very high. Strips are produced with strengths of approximately 3,000 MPa in the longitudinal direction, and with a thickness of up to 1.5 mm and widths of up to 150 mm In order to achieve an optimum composite action, the preparation of the bonding surfaces of the strip and concrete is critical. The strips must have the outermost layer of their bonding face, normally matrix-rich, removed to expose the fibers. Just before the bonding, the bonding surface is carefully cleaned with acetone. The concrete surface is treated by sand blasting, high pressure water jets, stoking, or grinding. Shortly before the bonding, it is cleaned with a vacuum cleaner. Concrete must be at least 6 weeks old, and have a minimum tensile strength of 1.5 MPa. Highly filled epoxy resin adhesive is used for the bonding.

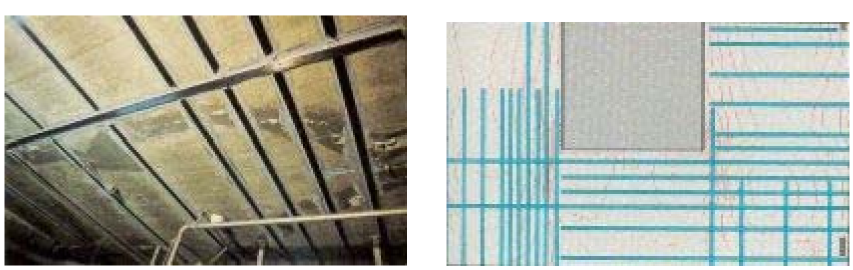

Fig.1.1FRP laminates

1.5 Externally bonded FRP reinforcement (EBR)

Recent developments related to materials, methods and techniques for structural strengthening have been enormous. One of today's state-of-the-art techniques is the use of fibre reinforced polymer (FRP) composites, which are currently viewed by structural engineers as "new" and highly promising materials in the construction industry. Composite materials for strengthening of civil engineering structures are available today mainly in the form of thin unidirectional *strips* (with thickness in the order of 1 mm) made by pultrusion flexible *sheets* or *fabrics*, made of fibres in one or at least two different directions, respectively (and sometimes pre-impregnated with resin). For comparison with steel, typical stress-strain diagrams for unidirectional composites under short-term monotonic loading are given in Fig. 1.2.

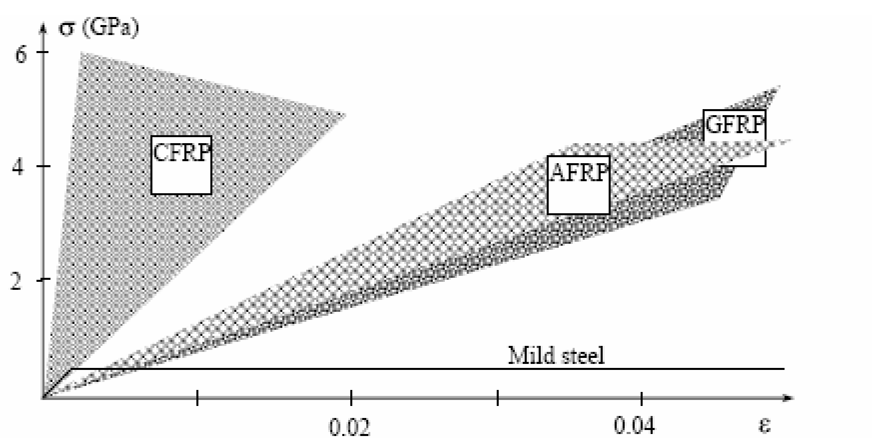

Fig 1.2: Uniaxial tension stress-strain diagrams for different FRPs and steel.(Ref –1)

1.5.1 Advantages of using FRP

The main advantages of FRP plates are their:

1. High strength to weight ratio. (about ¼ of steel),
2. Elimination of the need for scaffolding and reduction in labour costs,

3. Very high tensile strength (both static and long term, for certain types of FRP materials),

4. Stiffness that may be tailored to the design requirements,

5.Large deformation capacity; and practically unlimited availability in FRP sizes and FRP geometry and dimensions,

6. High corrosion resistance,

7. Reduced interruptions to existing services,

8. FRP plates are normally at least twice but can be over 10 times as strong as steel plates while their weight is only 20% of that of steel.

1.5.2 Disadvantages of FRP

Composites suffer from certain disadvantages too, which are not to be neglected by engineers: contrary to steel, which behaves in an elasto plastic manner, composites in general are linear elastic to failure (although the latter occurs at large strains) without any significant yielding or plastic deformation, leading to reduced ductility.

Additionally, the cost of materials on a weight basis is several times higher than that for steel (but when cost comparisons are made on a strength basis, they become less unfavorable).

Moreover, some FRP materials, e.g. carbon and aramid, have incompatible thermal expansion coefficients with concrete. Finally, their exposure to high temperatures (e.g. in case of fire) may cause premature degradation and collapse (some epoxy resins start softening at about 45-70^{o}C). Hence FRP materials should not be thought of as a blind replacement of steel (or other materials) in structural intervention applications. Instead, the advantages offered by them should be evaluated against potential drawbacks, and final decisions regarding their use should be based on consideration of several factors, including not only mechanical performance aspects, but also contractibility and long-term durability.

Although this technology has been used successfully in Japan and Europe, the usage of composite materials like FRP is still not widely recognized in the industry. The lack of knowledge of the technology and the simplicity of it will make some people hesitant to use it.

One barrier to the widespread use of these technologies using FRP materials for infrastructure repair is that they have not been broadly accepted by any building codes. Some concerns still remain about FRP's fire resistance, long-term creep characteristics, and aging due to ultraviolet rays or degradation of bond forces with time. Also, a full understanding of failure behavior and design models that would reflect the improvements in strength and stiffness of rehabilitated or retrofitted concrete structural members still need to be developed.

1.6 Applications of EBR

Composites have found their way as strengthening materials of reinforced concrete (RC) elements (such as beams, slabs, columns etc.) in thousands of applications worldwide, where conventional strengthening techniques may be problematic. For instance, one of the popular techniques for upgrading RC elements has traditionally involved the use of steel plates epoxy bonded to the external surfaces (e.g. tension zones) of beams and slabs. This technique is simple and effective as far as both cost and mechanical performance is concerned, but suffers from several disadvantages: corrosion of the steel plates resulting in bond deterioration; difficulty in manipulating heavy steel plates in tight construction sites; need for scaffolding; and limitation in available plate lengths (which are required in case of flexural strengthening of long girders), resulting in the need for joints.

Replacing the steel plates with FRP strips (Fig. 1-3 a, b) provides satisfactory solutions to the problems described above.

Another common technique for the strengthening of RC structures involves the construction of reinforced concrete (either cast in-place or shotcrete) jackets (shells) around existing elements. Jacketing is clearly quite effective as far as strength, stiffness and ductility is concerned, but it is labour intensive, it often causes disruption of occupancy and it provides RC elements, in many cases, with undesirable weight and stiffness increase. Jackets may also be made of steel; but in this case protection from corrosion is a major issue. The conventional jackets may be replaced with FRP fabrics or sheets wrapped around RC elements (Fig. 1-2 c, d), thus providing substantial increase in strength (axial, flexural, shear, torsional) and ductility without much affecting the stiffness. The range of applicability of EBR in RC structures is increasing constantly: a typical example is the recently developed technique of shear strengthening in beam-column joints (Fig. 1-3).

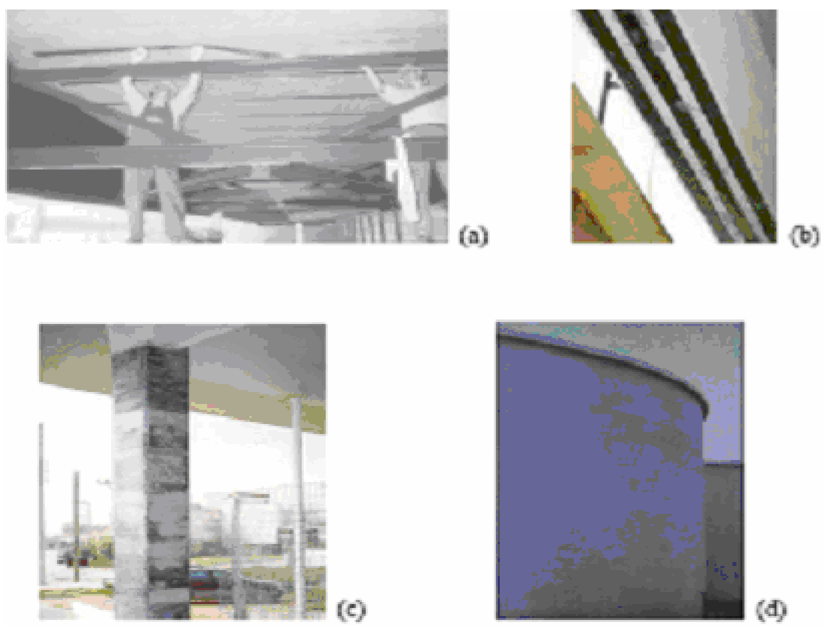

Fig. 1.3 Application of FRP

1.7 Objectives of Present Study

The purpose of this thesis is to compare different debonding models on different parameters and to obtain a conclusion for the different parameters to be provided by any user to avoid major debonding failures.The objective is divided into the following:

i. To study the present state of art and state of practice for retrofitting of structures by literature survey.
ii. To study different codes and guidelines for the requirement of retrofitting of structures.
iii. To numerically compare debonding strength models.
iv. To develop a designing model with the help of a programming language.
v. To provide FRP flexural strengthening of the beams to a structure that has been designed in accordance with the old code(IS 1893 –1984)
vi. To suggest the scope of further study

1.8 Thesis Organization

After discussing the importance of retrofitting of structures, different techniques for retrofitting structures is given in the *Introductory* chapter. Advantages of FRP over other materials is also discussed, we tried our best to organize this thesis in very systematic way which includes these chapters

Chapter-2 In this chapter of the thesis, we commence on to the materials for FRP strengthening and various types of FRP strengthening systems are discussed.

Chapter-3 Flexural strengthening of Beams through FRP is discussed along with the various failure modes. Literature survey for debonding is also quoted in this section.

Chapter –4 Starting from the Comparison of different debonding strength models based on various parameters, a case study is taken to calculate the thickness of FRP required based on different debonding strength models.

Chapter-5 A case study is taken on the basis of revision of Indian Standard Code IS 1893-1984 to IS 1893- 2002 and the requirement of retrofitting is explained. Most important and all the essence of the project work lies in this section, where demonstration of the problem of seismic upgrade of code is discussed using STAAD PRO.

Chapter-6 FPR is calculated for the case study to retrofit the old structure.

Chapter-7 Conclusion symbolizes the whole work and so this section is having some key words, which reflect my labour and dedication for this project work under the guidance of my guides.

Last but not the least, *References* give completeness to my thesis and my project work.

Chapter 2. FRP strengthening materials and techniques

2.1 Introduction

This chapter provides general information on FRP materials used in concrete strengthening, on concepts and techniques for their application, and on recently developed advanced methods of FRP applications as externally bonded reinforcement of concrete.

Fiber Reinforced Polymer (FRP) composites is defined as a polymer (plastic) matrix, either thermoset or thermoplastic, that is reinforced (combined) with a fiber or other reinforcing material with a sufficient aspect ratio (length to thickness) to provide a discernable reinforcing function in one or more directions. FRP composites are different from traditional construction materials such as steel or aluminum. FRP composites are anisotropic (properties only apparent in the direction of the applied load) whereas steel or aluminum is isotropic (uniform properties in all directions, independent of applied load). Therefore, FRP composite properties are directional, meaning that the best mechanical properties are in the direction of the fiber placement. Composites are similar to reinforced concrete where the rebar is embedded in an isotropic matrix called concrete.

2.2 Materials for FRP strengthening

The selection of materials for different strengthening systems is a critical process. Every system is unique in the sense that the fibres and the resin components are designed to work together. This implies that a resin system for one strengthening system will not automatically work properly for another. Furthermore, a resin system for the fibres will not necessarily provide a good bond to concrete. This implies that only systems that have been tested and applied in full scale on reinforced concrete structures shall be used in

FRP strengthening. Today there are several types of FRP strengthening systems, which are summarized below:

a) Wet lay-up systems
b) Systems based on prefabricated elements
c) Special systems, e.g. automated wrapping, prestressing etc.

These systems correspond to several manufacturers and suppliers, and are based on different configurations, types of fibres, adhesives, etc. Also, the suitability of each system depends on the type of structure that shall be strengthened. For example, prefabricated strips are generally best suited for plane and straight surfaces, whereas sheets or fabrics are more flexible and can be used to plane as well as to convex surfaces. Automated wrapping can be preferable in cases when many columns need to be strengthened at the same site.

Practical execution and application conditions, for example cleanness and temperature, are very important, in achieving a good bond. A dirty surface will never provide a good bond.

The adhesives undergo a chemical process during hardening that needs a temperature above 10^0C to start. If the temperature drops, the hardening process delays. In the following sections the three main components, namely adhesives, resin matrices and fibres of an FRP strengthening material system is discussed briefly.

2.3 Adhesives

The purpose of the adhesive is to provide a shear load path between the concrete surface and the composite material, so that full composite action may develop. Only the most common type of structural adhesives will be discussed here, namely epoxy adhesive, which is the result of mixing an epoxy resin (polymer) with a hardener. Depending on the application demands, the adhesive may contain fillers, softening inclusions, toughening additives and others. The successful application of an epoxy adhesive system requires the preparation of an adequate specification, which must include such provisions as adherent materials, mixing/application temperatures and techniques, curing temperatures, surface preparation techniques, thermal expansion, creep properties, abrasion and chemical resistance.

Epoxy adhesives have several advantages over other polymers as adhesive agents for civil engineering use, namely:

1. High surface activity and good wetting properties for a variety of substrates may be formulated to have a long open time.
2. High cured cohesive strength; joint failure may be dictated by adherent strength may be toughened by the inclusion of dispersed rubbery phase
3. Lack of by-products from curing reaction minimizes shrinkage and allows the bonding of large areas with only contact pressure
4. Low shrinkage compared with polyesters, acrylics and vinyl types
5. Low creep and superior strength retention under sustained load
6. Can be made thixotropic for application to vertical surfaces
7. Able to accommodate irregular or thick bond line.

2.4 Matrices

The matrix for a structural composite material can either be of thermosetting type or of thermoplastic type, with the first being the most common one. The function of the matrix is to protect the fibres against abrasion or environmental corrosion, to bind the fibres together and to distribute the load. The matrix has a strong influence on several mechanical properties of the composite, such as the transverse modulus and strength, the shear properties and the properties in compression. Physical and chemical characteristics of the matrix such as melting or curing temperature, viscosity and reactivity with fibres influence the choice of the fabrication process.

Hence, proper selection of the matrix material for a composite system requires that all these factors be taken into account. Epoxy resins, polyester and vinyl ester are the most common polymeric matrix materials used with high-performance reinforcing fibres. They are thermosetting polymers with good processibility and good chemical resistance. Epoxies have, in general, better mechanical properties than polyesters and vinyl esters, and outstanding durability, whereas polyesters and vinyl esters are cheaper.

2.5 Fibres

A great majority of materials are stronger and stiffer in the fibrous form than as a bulk material. A high fibre aspect ratio (length/diameter ratio) permits very effective transfer of load via matrix materials to the fibres, thus enabling full advantage of the properties of the fibres to be taken. Therefore, fibres are very effective and attractive reinforcement materials.

Fibres can be manufactured in continuous or discontinuous (chopped) form, but here only continuous fibres are considered. Such fibres have a diameter in the order of 5-20 μm, and can be manufactured as unidirectional or bi-directional reinforcement. The fibres used for strengthening all exhibit a linear elastic behaviour up to failure and do not have a pronounced yield plateau as for steel.

There are mainly three types of fibres that are used for strengthening of civil engineering structures, namely glass, aramid and carbon fibres. It should be recognised that the physical and mechanical properties can vary a great for a given type of fibre as well of course the different fibre types.

2.5.1 Glass Fibers

Glass fibers have typical properties of hardness, corrosion resistance, and inertness; they are also lightweight, flexible, and inexpensive. These properties make glass fibers attractive in low-cost infrastructure applications. All glass fibers have a similar stiffness, but different strengths and resistance to environmental degradation. Glass fibres for continuous fibre reinforcement are classified into three types:

1. E-glass fibres,
2. S-glass and
3. Alkali resistant AR-glass fibres.

E-glass fibres, which contain high amounts of boric acid and aluminate, are disadvantageous in having low alkali resistance. E-glass fibers are used when high tensile strength and good chemical resistance are required, which makes these fibers a preference in structural applications because of their good mechanical performance, corrosion resistance, and low cost.

S glass fibres are stronger and stiffer than E-glass, but still not resistant to alkali. S-2 glass fibers (S for strength) have the highest strength, but their limitation is that they are considerably more expensive.

To prevent glass fibre from being eroded by cement-alkali, a considerable amount of zircon is added to produce alkali resistance glass fibres; such fibres have mechanical properties similar to E glass.An important aspect of glass fibres is their low cost

2.5.2 Carbon Fibers

Carbon fibers, also known as graphite fibers, are lightweight and strong with outstanding chemical resistance. They are highly used in the aerospace industry. Contrary to glass fibers, which have one stiffness value, carbon fibers are available in a broad range of stiffness values. Two main raw materials used when manufacturing carbon fibers affect their properties. These are Polyacrylonitrile and pitch Polyacrylonitrile fibers. They govern the high performance markets in aerospace applications since they are made with a variety of stiffness and strength values. Pitch fibers are less expensive but have lower strength. Carbon fibres are normally either based on pitch or PAN, as raw material. Pitch fibres are fabricated by using refined petroleum or coal pitch that is passed through a thin nozzle and stabilized by heating. PAN fibres are made of poly acrylonitrile that is carbonised through burning. The diameter of pitch-type fibres measures approximately 9-18μ m and that of the PAN-type measures 5-8 μm. The structure of this carbon fibre varies according to the 8 orientation of the crystals; the higher the carbonation degree, the higher the orientation degree and rigidity as a result of growing crystals.

The pitch base carbon fibres offer general purpose and high strength/elasticity materials. The PAN-type carbon fibres yield high strength materials and high elasticity materials. Typical properties of various types of fibre materials are provided in Table 2.1.

Material	Elastic modulus (Gpa)	Tensile strength (MPa)	Ultimate tensile Strain (%)
Carbon			
High strength	215-235	3500-4800	1.4-2.0
Ultra high strength	215-235	3500-6000	1.5-2.3
High modulus	350-500	2500-3100	0.5-0.9
Ultra high modulus	500-700	2100-2400	0.2-0.4
Glass			
E	70	1900-3000	3.0-4.5
S	85-90	3500-4800	4.5-5.5
Aramid			
Low modulus	70-80	3500-4100	4.3-5.0
High modulus	115-130	3500-4000	2.5-3.5

Table :2.1 Typical properties of various types of fibre materials .

2.5.3 Aramid Fibers

The best known aramid fibers have trade names such as Kevlar, Technora, and Twaron. Aramid fibers have high energy absorption during failure, which makes them ideal for impact and ballistic protection. They have a low density, allowing them to have a high tensile strength-to-weight ratio and a high modulus-to-weight ratio, which makes them especially attractive for aircraft and body armor type applications.

Aramid fibers are made of polymer materials, which give the fibers the characteristics of the polymer. These fibers have a low compressive strength; in addition, they creep, absorb moisture, and are sensitive to Ultraviolet light. Their mechanical properties also vary considerably with temperature.

2.6 Fiber Forms

Most fibers can be obtained as prepreg tape in which an epoxy resin and a fiberglass backing hold fibers together. The production of the prepreg tape is labor intensive and thus induces additional cost. Consequently, most of the new applications in composites tend to use fibers in their simplest, unprocessed forms. For example, pultrusion and filament winding use roving or tow and resin to produce the final product without intermediate operations. Woven or stitched fabrics facilitate the fabrication of laminates in resin transfer molding and other processes.

2.6.1 Discontinuous and Continuous Fibers

Composites are reinforced with two different types of fibers: continuous or discontinuous. Continuous fibers are long fibers that usually attain maximum values in properties such as strength and stiffness due to the low number and size of surface defects. Continuous fibers are usually oriented along the direction in which the load is applied.

Discontinuous fibers are short fibers that are obtained by chopping the continuous fibers; they can also be directly produced as short-fibers to reduce fabrication costs. The aspect ratio (length over diameter) significantly affects the properties of short fiber composites. The orientation of the discontinuous fibers cannot be easily controlled and is assumed to be random, and these fibers usually have lower strength when compared to the continuous fibers.

2.6.2 Mat, Fabric, and Veil

Randomly oriented chopped filaments, short fibers, form a mat or swirled filaments loosely held together with a small amount of adhesive. A veil is a thin mat used as a surfacing layer to improve corrosion resistance of the composite. Veils and mats have fibers randomly oriented in all directions.

On the other hand, fabrics (Figure 2) are a two-dimensional reinforcement. Woven fabrics can be made by knitting but generally are made by weaving of yarns. No woven fabrics are made directly of strands, without the intermediate twisting of the strands into yarns. Stitched non woven fabrics can be made into very heavy fabrics thus reducing the time and cost of composite processing, provided that they can be adequately infiltrated with resins. Fabrics can be created with off-angle layers (e.g., + or – 45°) allowing several advantages in design and performance.

(a) (b)

Figure 2 Illustration showing the difference between

(b) Randomly oriented fibers and (b) Unidirectional woven fabric

Chapter 3. Flexural Strengthening of Beams

3.1 Introduction

Reinforced concrete elements, such as beams and columns, may be strengthened in flexure through the use of FRP composites epoxy-bonded to their tension zones, with the direction of fibres parallel to that of high tensile stresses (member axis). Prior to the application of FRP plate, the soffit of the RC beam must be prepared. The purpose of the adequate surface preparation is to improve the bond with the FRP,and provide an even surface. Application procedure may consist of following steps:

- ✓ Remove all unsound concrete and loose particles
- ✓ Arrest the corrosion
- ✓ Make it in it's original shape
- ✓ Grind the corner
- ✓ Make the surface smooth and undulation free
- ✓ Apply the adhesive
- ✓ Press the Wrap or strip by hand
- ✓ Use roller to remove air
- ✓ Put another coat of adhesive
- ✓ Apply protective coatings

The FRP plate may be a prefabriced plate, in which case some preparation of the bonding surface of the FRP plate may be necessary.[1]

The concept is illustrated in Fig.3-1, which also shows a practical application.

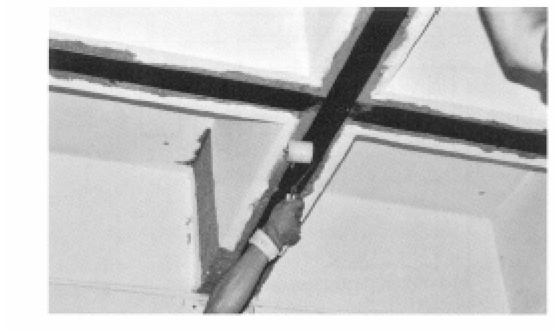

Fig. 3.1: Flexural strengthening of RC beams with CFRP strips

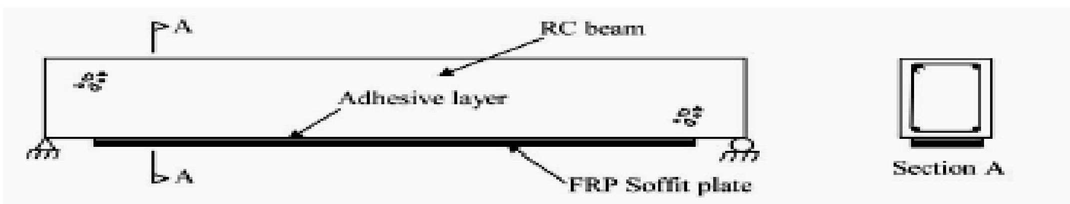

Fig. 3.2: RC beam bonded with FRP soffit plate.

Care should be taken during application

- ✓ Proper orientation of fiber
- ✓ Choice of epoxy as a adhesive
- ✓ Proper mixing of epoxy
- ✓ Entrapped air
- ✓ Apply fabric as temperature is falling

3.2 Debonding and bond failure modes

Bond is necessary to transfer forces from the concrete into the FRP, hence *bond failure modes* have to be taken into account properly. Bond failure in the case of EBR implies the complete loss of composite action between the concrete and the FRP reinforcement, and occurs at the interface between the EBR and the concrete substrate. On the other hand, localised *debonding*, means a local failure in the bond zone between concrete and EBR. In this case the reduction in bond strength between concrete and FRP reinforcement is limited to a small area, e.g. a loss in bond length of 2 mm next to a crack in a flexural member. Therefore localised debonding is not in itself a failure mode, which will definitely cause a loss of the load carrying capacity of a member with EBR. When localized debonding propagates, and composite action is lost in such a way that the FRP reinforcement is not able to take loads anymore, this failure is called *peeling-off*. If no stress redistribution from the externally bonded FRP reinforcement to the embedded reinforcement is possible, peeling-off will be a sudden and brittle failure.

Bond failure may occur at different interfaces between the concrete and the FRP reinforcement, as described below.

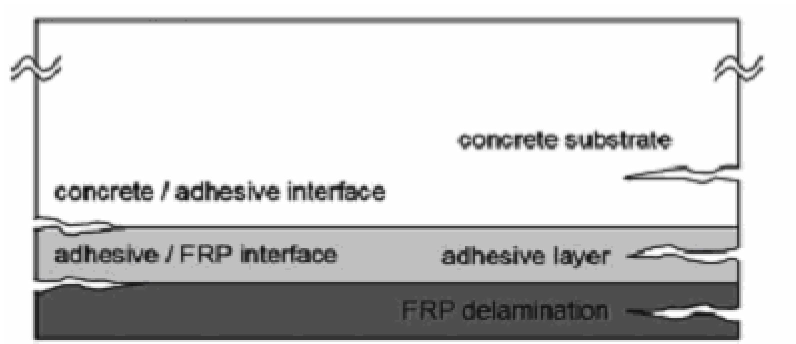

Fig. 3.3: Bond failures at different interfaces between the concrete and the FRP

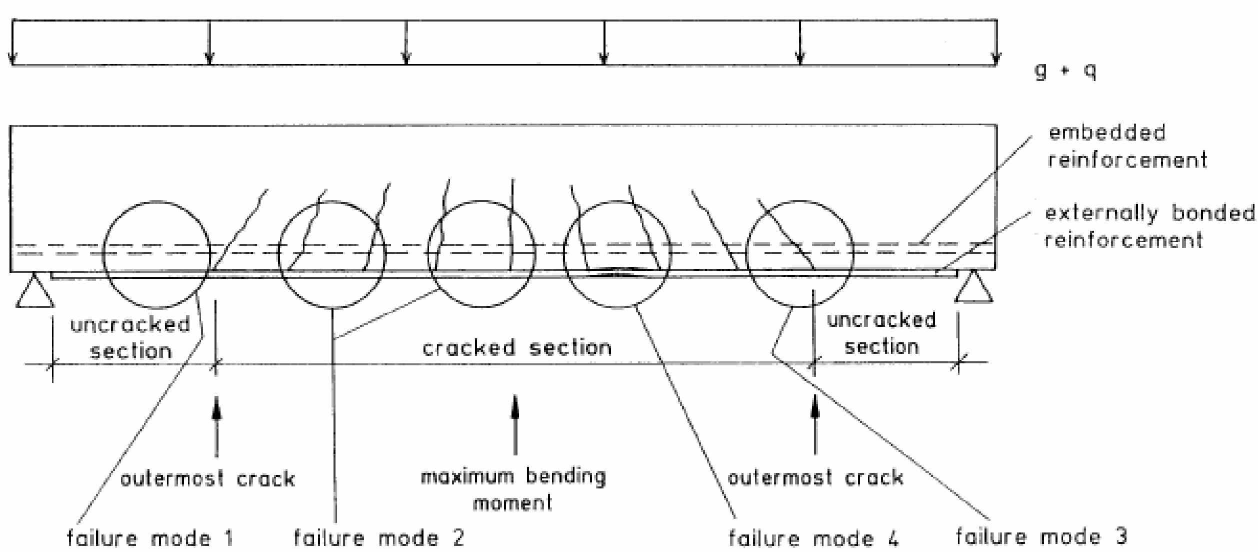

Fig.: 3.4 Bond failure modes of a concrete member with EBR

Mode 1: peeling-off in an uncracked anchorage zone

The FRP may peel-off in the anchorage zone as a result of bond shear fracture through the concrete.

Mode 2: peeling-off caused at flexural cracks

Flexural (vertical) cracks in the concrete may propagate horizontally and thus cause peeling-off of the FRP in regions far from the anchorage.

Mode 3: peeling-off caused at shear cracks

Shear cracking in the concrete generally results in both horizontal and vertical opening, which may lead to FRP peeling-off. However, in elements with sufficient internal (and external) shear reinforcement (as well as in slabs) the effect of vertical crack opening on peeling-off is negligible

Mode 4: peeling-off caused by the unevenness of the concrete surface

The unevenness or roughness of the concrete surface may result in localized debonding of the FRP, which may propagate and cause peeling-off.

3.3 Classification of Failure Modes

A number of failure modes for RC beams bonded with FRP soffit plates have been observed as:

a) Flexural Failure by FRP rupture
b) Flexural Failure by crushing of compressive concrete.
c) Shear Failure.
d) Concrete Cover separation.
e) Plate end interfacial debonding.
f) Intermediate Flexural crack.

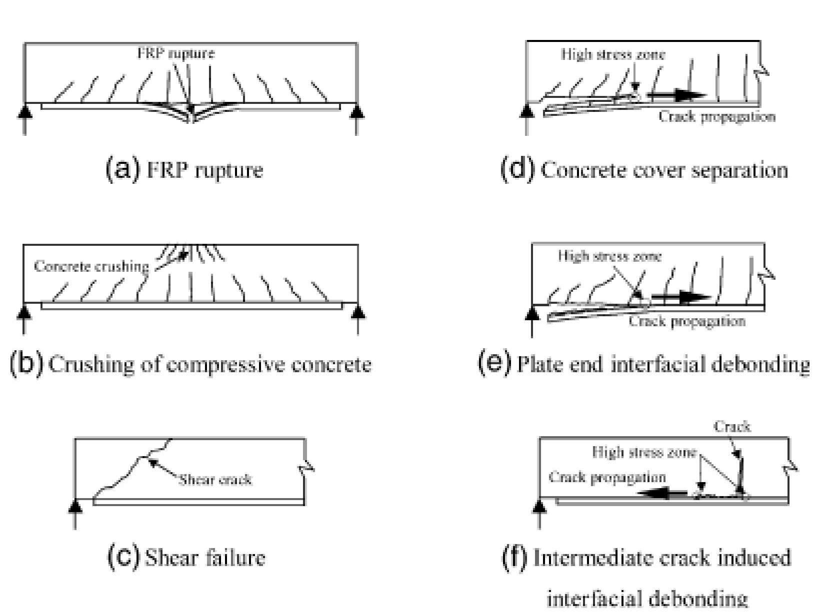

Fig.3.5: Failure Modes

3.3.1 Flexural Failure

If the ends of the plate are properly anchored, the ultimate flexural capacity of the beam is reached when either the FRP plate fails by tensile rupture or the compressive concrete is crushed. This is very similar to the classical flexural failure of RC beams, except for small differences due to the brittleness of the bonded FRP plate .FRP rupture generally occurs following the yielding of the longitudinal steel bars, although steel yielding may not have been reached if the steel bars are located quite far away from the tension face.

The strength gain and the ductility reduction are the two main consequences of flexural strengthening of RC beams using FRP plates. Beams which may fail by concrete crushing when alarge amount of FRP is used also show much reduced ductility. [1]

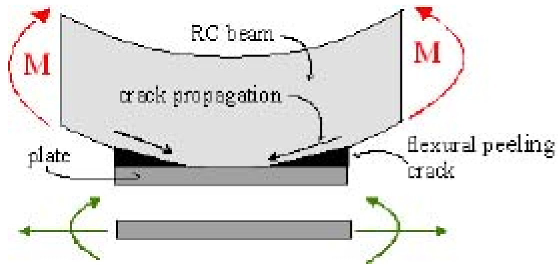

Fig. 3.6: Flexural peeling mechanism

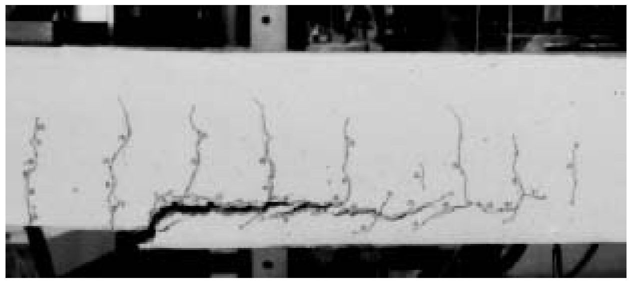

Fig 3.7: Plate-end in predominantly flexure region

3.3.2 Shear Failure

The strengthened beam can fail brittlely in shear,the shear failure mode can be made critical by flexural strengthening. Shear strengthening of the RC beams should be carried out simultaneously to ensure that the required flexural strength is not compromised shear Failure and that flexural failure still precedes shear failure[3].

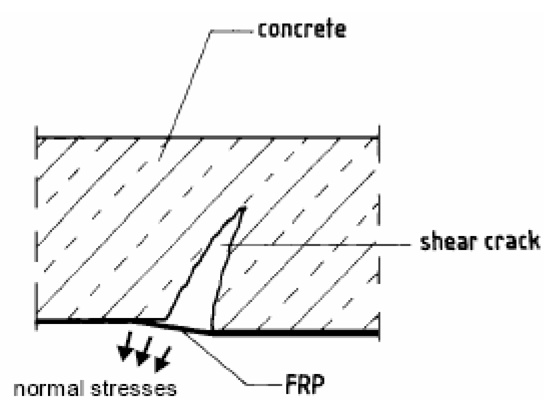

Fig. 3.8 Peeling-off caused at shear cracks.

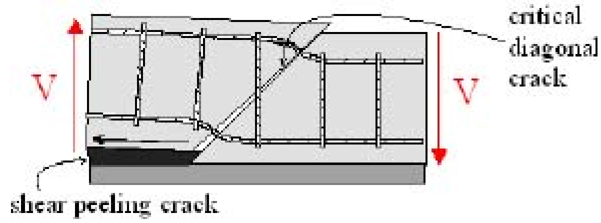

Fig.3.9: Shear peeling mechanism

Fig. 3.10: Plate-end in predominantly shear region

3.3.3 Plate end debonding failures

Premature failure may occur before the ultimate flexural capacity of the beam is reached owing to debonding.

3.3.3.1 Concrete Cover Separation

It is generally believed that failure of the concrete cover is initiated by the formation of a crack at or near the plate end, due to high interfacial shear and normal stresses caused by the abrupt termination of the plate here.

Once a crack forms in the concrete at or near the plate end, the crack propagates to the level of the tension reinforcement and then progresses horizontally, along the level of the reinforcement, resulting in the separation of the concrete cover (Fig. 3a). Fig. 3b shows a close up view of the detached plate end, where the tension reinforcement of the beam can be clearly seen. As the failure occurs away from the bond line, this is not a debonding failure mode in strict terms, although it does stem from stress concentration near the plate end.

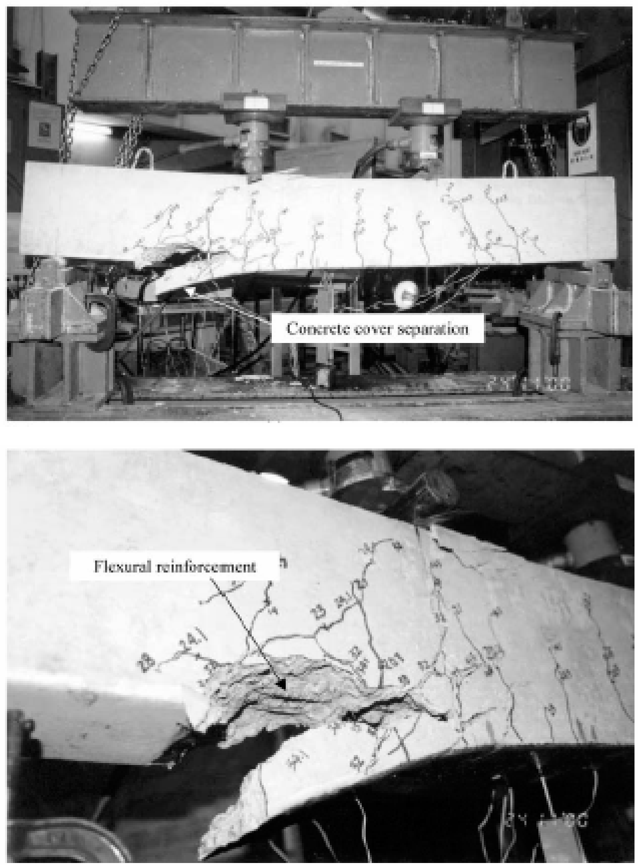

Fig.3.11: Failure Modes

3.3.3.2 Plate-end Interfacial Debonding

Debonding between the FRP plate and the RC beam (Fig. 2e), that propagates away from the plate end, has also been observed in experiments and is herein referred to as *plate end interfacial debonding*. The general consensus among researchers is that debonding failures of this form are initiated by high interfacial shear and normal stresses near the plate end that exceed the strength of the weakest element, generally the concrete. Upon debonding, a thin layer of concrete generally remains attached to the plate. This suggests that failure generally occurs in the concrete adjacent to the concrete-to-adhesive interface.

As both plate end debonding failure modes are due to the same cause (i.e. high interfacial shear and normal stresses near the plate end), plate end interfacial debonding has not been differentiated from concrete cover separation in many of the existing plate end debonding strength models.

3.3.4 Intermediate Crack-induced Interfacial Dedonding

Debonding may initiate at a flexural or a mixed flexural shear crack away from the plate ends and then propagate towards one of the plate ends as shown in fig……Debonding generally occurs in the concrete ,adjacent to the adhesive-to-concrete interface, and is referred to here in as intermediate flexural crack –induced debonding or intermediate shear crack –induced debonding.

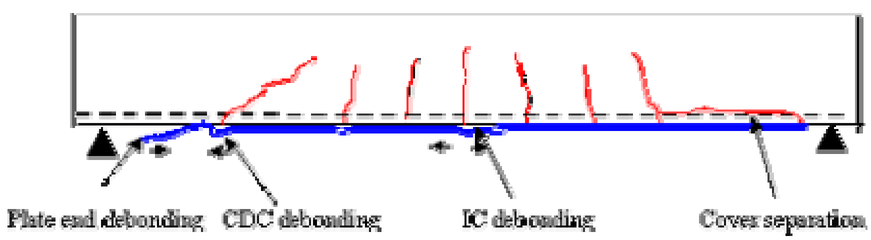

Fig. 3.12: Debonding failure modes in flexurally-strengthened RC beams

3.4 Literature Review

Several strength models for plate-end dedonding have been developed for FRP-plated RC beamsin the last decade (Varastehpour and Hamelin’s 1997, Saadatmanesh and Malek’s model 1998 , Wang and Ling’s model 1998 Ahmed and van Gemert’s model 1999, Tumialan et al.’s model 1999)

In addition,a number of strength models have also been developed for plate end debonding in steel-plated RC beams((Oehlers' model 1992, Ziraba et al.'s models1994, Jansze's model1997 Raoof and Zhang's model 1997).

Plate end debonding occurs either by concrete cover separation or in the concrete adjacent to the adhesive-to-concrete interface. Although the different mechanical properties of the bonded plate are expected to play a significant role, strength models can be expected to be applicable to different types of plates if the geometrical and material properties of plates are properly included. It is thus not unreasonable to expect that models developed for steel plated beams may well be applicable to FRP-plated beams, with modifications if needed.

In terms of approaches, existing debonding strength models can be divided into three categories, namely

(a) shear capacity-based models(Oehlers' model, Jansze's model, Ahmed and van Gemert's model)

(b) concrete tooth-based models(Raoof and Zhang's model, Wang and Ling's model ,Raoof and Hassanen's models)

(c) interfacial stress-based models(Ziraba et al.'s models, Varastehpour and Hamelin's model Saadatmanesh and Malek's model ,Tumialan et al.'s model)

It was seen that Oehlers' model(1992) is not purely based on shear capacity,as it also takes into account the interaction between shear and bending.The models developed specifically for FRP- strengthened RC beams follow the interfacial stress–based approach (Varastehpour and Hamelin's model Saadatmanesh and Malek's model ,Tumialan et al.'s model).

On the other hand, three of the five models specifiacally developed for steel plated beams follow either the shear capacity approach (Oehlers' model (1992) and Jansze's model (1997). or a combined approach (Ziraba et al.'s models1994).

Chapter 4. Comparison of debonding strength models

4.1 Shear capacity based models:

The common feature of these models is that the debonding failure strength is related to the shear strength of the concrete with no or only partial contribution of the steel shear reinforcement. The debonding strength is generally given as the shear force acting at the plate end, with or without taking into account the effect of any coexistent moment. The interfacial stresses between the plate and the beam need not be evaluated.

Table 2:Comparison of different Shear capacity models:

Oehlers' model	*Jansze's model*	***Ahmed and van Gemert's model***
Steel plate	Steel plate	FRP plate
Shear debonding occurs when the shear force at the plate end reaches the shear capacity of the concrete in the RC beam without the contribution of steel shear reinforcement	Shear debonding occurs on the initialization of shear cracking in an RC beam	Shear debonding occurs as concrete cover seperation
Shear reinforcement is not included for shear resistance	Shear reinforcement is not included for shear resistance	Shear reinforcement is included for shear resistance
Model can be used for terminating near the supports	Model can not be used for terminating near the supports	Model can not be used for terminating near the supports
$V_{db,end} = \dfrac{1.17}{\dfrac{a}{Mdb,f} + \dfrac{1}{Vdb,s}}$	$V_{db,end} = \tau_{PES} b_c d$	$V_{db,end} = \left(\tau_{PES} + \Delta\tau \, \mathrm{mod}\right) b_c d$

4.2 Concrete tooth models :

Concrete tooth models make use of the concept of a concrete “tooth” between two adjacent cracks deforming like a cantilever under the action of horizontal shear stresses at the base of the beam (Fig. 4.1). Debonding is deemed to occur when these shear stresses lead to tensile stresses at the root of the “tooth” that exceed the tensile strength of the concrete. The stress in the soffit plate at debonding can then be determined by defining an effective length for the plate for end anchorage over which a uniform shear stress is assumed.

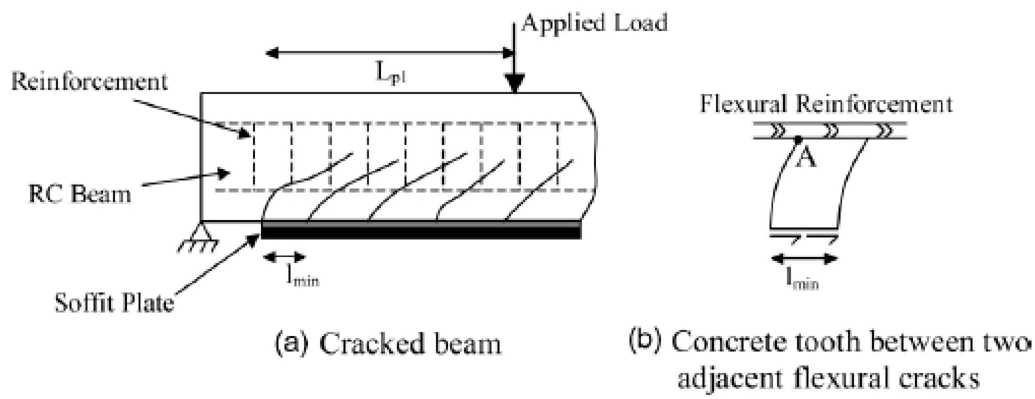

Fig.4.1 (a) Cracked beam (b) Concrete tooth between two adjacent flexural cracks

Table 3: Comparision of different Concrete tooth models:

Raoof and Zhang's model	Wang and Ling's model	Raoof and Hassanen's models
Steel plated beams	Steel plated beams	FRP plated beams
The effective length for end anchorage be the smaller of the length of the soffit plate in the shear span $Lp1$ and the lengths $Lp2$ for steel plated RC beams that failed by plate end debonding $L_{P,2} = l_{\min}(2l - 0.25 l_{\min})$ $l_{\min} \leq 72mm$ $L_{p,2} = 3l_{\min}$ $l_{\min} > 72mm$	T he effective length for end anchorage was taken to be the total plate length in the shear span.	The effective length of the plate for end anchorage was taken to be the smaller of the plate length in the shear span $Lp1$ and the the lengths $Lp2$ for FRP-plated RC beams those failed by plate end debonding 1) $L_{p,2} = l_{\min}(24 - 0.5 l_{\min})$ $l_{\min} \leq 40mm$ $L_{P,2} = 4l_{\min}$ $l_{\min} > 40mm$ 2) $L_{P,2} = l_{\min}(11.6 - 0.17 l_{\min})$ $l_{\min} \leq 56.5mm$ $L_{p,2} = 2l_{\min}$ $l_{\min} > 56.5mm$
One expressions was presented by them for the effective length of the steel plate for end anchorage	One expressions was presented by them for the effective length of the steel plate for end anchorage	They presented two expressions for the effective length of the FRP plate for end anchorage with two corresponding values for the bond strength between the FRP and the concrete.
$l_{\min} = \frac{A_e f_{ct}}{u(\Sigma O_{bar} + b_{frp})}$	$l_{\min} = \frac{A_e f_{ct}}{u_s \Sigma O_{bars} + u_{frp} b_{frp}}$	$l_{\min} = \frac{A_e f_{ct}}{u(\Sigma O_{bar} + b_{frp})}$

4.3 Interfacial stress based models

Interfacial stress based debonding strength models make use of interfacial stresses

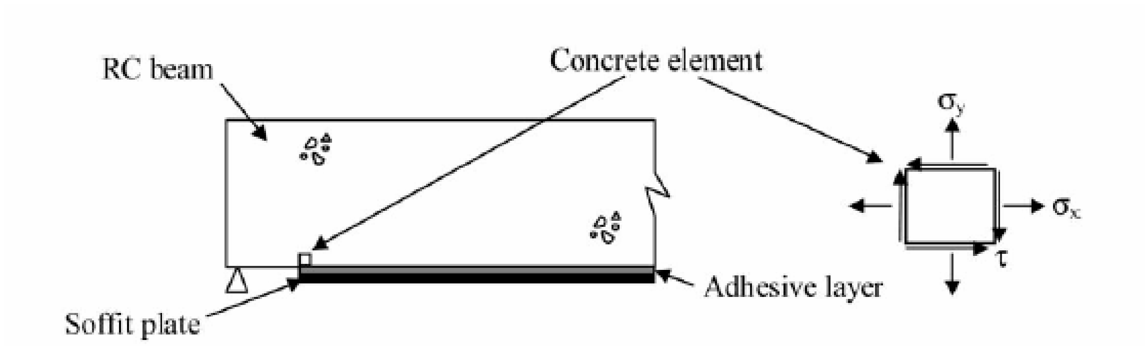

Fig 4.2: Stresses acting on a concrete element adjacent to the plate end

Table 4: Comparision of different Interfacial stress based models

Ziraba et al.'s models	***Varastehpour and Hamelin's model***	***Saadatmanesh and Malek's model***	Tumialan et al.'s model
Steel plated beams	FRP plated beams	FRP plated beams	FRP plated beams
Two models for predicting interfacial debonding	One model for predicting interfacial debonding	One model for predicting interfacial debonding	One model for predicting interfacial debonding
Can be used for plates terminated near supports	Can be used for plates terminated near supports	Can not be used for plates terminated near supports	Can not be used for plates terminated near supports

Chapter 5. Indian Code recommendations for Retrofitting (IS:13935)

5.1 Seismic Strengthening

The main purpose of the seismic strengthening is to upgrade the seismic resistance of a damaged building while repairing so that it becomes safer under future earthquake occurrences. This work may involve some of the following actions:

a) Increasing the lateral strength in one or both directions by increasing column and wall areas or the number of walls and columns.

b) Giving unity to the structure, by providing a proper connection between its resisting elements, in such a way that inertia forces generated by the vibration of the building can be transmitted to the members that have the ability to resist them. Typical important aspects are the connections between roofs or floors and walls, between intersecting walls and between walls and foundations.

c) Eliminating features that are sources of weakness or that produce concentration of stresses in some members. Asymmetrical plan distribution of resisting members, abrupt changes of stiffness from one floor to the other, concentration of large masses and large openings in walls without a proper peripheral reinforcement are examples of defects of this kind.

d) Avoiding the possibility of brittle modes of failure by proper reinforcement and connection of resisting members.

5.2 Seismic Retrofitting

Many existing buildings do not meet the seismic strength requirements of present earthquake codes due to original structural inadequacies and material degradation due to time or alterations carried out during use over the years. Their earthquake resistance can be upgraded to the level of the present day codes by appropriate seismic retrofitting techniques, such as mentioned earlier.

5.3 Strengthening or Retrofitting vs. Reconstruction

Replacement of damaged buildings or existing unsafe buildings by reconstruction is, generally, avoided due to a number of reasons, the main ones among them being:

a) Higher cost than that of strengthening or retrofitting,
b) Preservation of historical architecture, and
c) Maintaining functional social and cultural environment.

In most instances, however, the relative cost of retrofitting to reconstruction cost determines the decision. As a thumb rule, if the cost of repair and seismic strengthening is less than about 50 percent of the reconstruction cost, the retrofitting is adopted. This may also require less working time and much less dislocation in the living style of the population. On the other hand reconstruction may offer the possibility of modernization of the habitat and may be preferred by well-to-do communities.

Cost wise the building construction including the seismic code provisions in the first instance, works out the cheaper in terms of its own safety and that of the occupants.

Retrofitting an existing inadequate building may involve as much as 4 to 5 times the initial extra expenditure required on seismic resisting features. Repair and seismic strengthening of a damaged building may even be 5 to 10 times as expensive. It is therefore very much safe as well as cost-effective to construct earthquake resistant buildings at the initial stage itself according to the relevant seismic IS codes.

5.4 Selection of materials and techniques

The most common materials for repair works of various types buildings are cement and steel. In many situations suitable admixture may be added to cement mortar/cement concrete to improve their properties, such as, non-shrink-age, bond, etc. Steel may be required in many forms like bolts, rods, angles, beams, channels, expanded metal and welded wire fabric.

Wood and bamboo are the most common material for providing temporary supports and scaffolding, etc, and will be required in the form of rounds, sleepers, planks, etc Besides the above, special materials and techniques are available for best results in the repair and strengthening operations. These should be selected appropriately depending on the nature and cost of the building that is to be repaired, materials availability and feasibility and use of available skills, etc. Some special materials and techniques are described below.

1 Shotcrete

Shotcrete is cement mortar or cement concrete (with coarse aggregate size maximum 10 mm)conveyed through a hose and pneumatically placed under high velocity on to a prepared concrete or masonry surface. The force of the jet impingement on the surface compacts the shotcrete material and produces a dence homogeneous mass. Basically there are two methods of shotcreting; wet mix process and dry mix process. In the wet mix process, all the ingredients including water are mixed together before they enter the delivery hose. In the dry mix process, the mixture of damp sand and cement is passed through the delivery hose to the nozzle where the water is added. The dry mix process is generally used in the repair of concrete elements. The bond between the prepared concrete surface of the damaged member and the layer of shotcrete is ensured with the application of suitable epoxy adhesive formulation. The shear transfer between the existing and new layer of concrete is ensured with the provision of shear keys.

2 Epoxy Resins

Epoxy resins are excellent binding agents with high tensile strength. These are chemical preparations the compositions of which can be changed as per requirements. The epoxy components are mixed just prior to application. Some products are of low viscosity and can be injected in fine cracks too. The higher viscosity epoxy resin can be used for surface coating or filling larger cracks or holes. The epoxy resins may also be used for gluing steel plates to the distress members.

3 Epoxy Mortar

For larger void spaces, it is possible to combine the epoxy resins of either low viscosity or higher viscosity with sand aggregate to form epoxy mortar. Epoxy mortar mixture has higher compressive strength, higher tensile strength and a lower modulus of elasticity than cement concrete. The sand aggregate mixed to form the epoxy mortar increases its modulus of elasticity.

4 Quick-Setting Cement Mortar

This material is a non-hydrous magnesium phosphate cement with two components, that is, a liquid and a dry powder, which can be mixed in a manner similar to cement concrete.

5 Mechanical Anchors

Mechanical type of anchors employ wedging action to provide anchorage. Some of the anchors provide both shear and tension resistance. Such anchors are manufactured to give sufficient strength. Alternatively, chemical anchors bonded in drilled holes through polymer adhesives can be used.

Note: All though Indian Code does not recommend FRP retrofitting ,but it recommends epoxy resins for strengthening hence following the recommendations of the different international code FRP could be used in India.

5.5 Design codes used Internationally

Existing publications currently available for use development of codes and guidelines for FRP reinforced concrete are as follows:

USA

ACI 440.1R-03 (2003) “Guide for the Design and Construction of Concrete Reinforced with FRP Bars,” ACI Committee 440, American Concrete Institute, Farmington Hills, Mich.

ACI 440.3R-04 (2004) "Guide for Test Methods for Fiber Reinforced Polymers (FRP) for Reinforcing and Strengthening Concrete Structures," ACI Committee 440, American Concrete Institute, Farmington Hills, Mich.

ACI 440R (1996) “State-of-the-art Report on Fiber Reinforced Plastic Reinforcement for Concrete Structures,” American Concrete Institute Committee 440, 67 p.

ACI 440.2R-02 (2002) “Guide for Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures,” ACI Committee 440.

Canada

CAN/CSA-S6_00 “Canadian Highway Bridge Design Code” Canadian Standards Association, Toronto, Ontari

CAN/CSA-S806-02, “Design and Construction of Building Components with Fibre-Reinforced Polymers”, Canadian Standards Association, Toronto, Ontario, Canada, (May 2002), 187p.o, Canada, (December 2000), 734p.

ISIS Canada 2001a “Reinforcing Concrete Structures with Fiber Reinforced Polymers,” Design Manual No. 3, The Canadian Network of Centers of Excellence on Intelligent Sensing for Innovative Structures, ISIS Canada Corporation, Winnipeg, Manitoba, Canada, 158p.

Europe

FIP Task Group 9.3 “FRP Reinforcement for Concrete Structures” (1999)
Report # STF 22 A 98741 “Eurocrete Modifications to NS3473 When Using FRP Reinforcement”, Norway (1998)

Japan

Japan Society of Civil Engineers (JSCE) 1997 “Recommendation for Design and Construction of Concrete Structures Using Continuous Fiber Reinforced Materials,” Concrete Engineering Series 23, ed. by A. Machida, Research Committee on Continuous Fiber Reinforcing Materials, Tokyo, Japan, 325 p.

China

Proceedings of international symposium on bond Behaviour of FRP in structures (BBFS 2005), Dec, 7-9,Hong Kong, China, pp 45-54© 2005 international institute for FRP in construction design proposals for the debonding strengths of FRP strengthened RC beams in the Chinese Design Code.

Chapter 6. Case Study

6.1 Introduction

Now a days, for the construction of Civil Engineering Structures, we are using the recommendation of IS 456 2000 and IS 1893(Part 1)-2002,and those structures which were made before the revision of these code (IS 456 1978 & IS 1893 1984), need a retrofit these could be well understood from the clauses mentioned below:

6.2 Design Criteria for Multi-storied Buildings

Design Seismic Base Shear in accordance with IS 1893-1984

The base shear V_B is given by the following formula:

$V_B = kc\,\alpha_h$ W

Where,

k= performance factor depending on the structural framing system and brittleness or ductility of construction

c = a coefficient defining the flexibility of structure with the

increase in number of storeys depending upon fundamental

time period T

α_h = design seismic coefficient

W= total dead load + appropriate amount of live load as per code

T= fundamental time period of the building in seconds

NOTE - The fundamental time period may either be established by experimental observations on similar buildings or calculated by any rational method of analysis.

In the absence of such data T may be determined as follows for multistoried buildings:

a) For moment resisting frames without bracing or shear walls for resisting the lateral loads

T=0.1n

Where,

n= number of storeys including basement storeys.

b) For all others where

$$T = \frac{0.09H}{\sqrt{d}}$$

H = total height of the main structure of the building in meters,

d = maximum base dimension of building in meters in a direction parallel to the applied seismic force.

6.3 Design Seismic Base Shear in accordance with IS 1893(Part 1)-2002

The total design lateral force or design seismic base shear (V_B)along any principal direction shall be determined by the following expression:

$$V_B = A_h W$$

where,

Ah = Design horizontal acceleration spectrum, using the fundamental natural period T, direction of vibration, and the design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{ZIS_a}{2Rg}$$

Z=Zone factor is for the

I=Importance factor,

R=Response reduction factor.

S_a/g = Average response acceleration coefficient, and based on appropriate natural periods and, the approximate fundamental natural period of vibration (T_a), in seconds, of a moment-resisting frame building without brick infill panels may be estimated by the empirical expression:

$T_a = 0.075\, h^{0.75}$ for RC frame building

$= 0.085\, h^{0.75}$ for steel frame building

Where,

h = Height of building, in m. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns.

But it includes the basement storeys, when they are not so connected.

The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09}{\sqrt{d}} h$$

where,

h= Height of building

d= Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force damping of structure.

W= Seismic weight of the building.

6.4 Case study

Observation from IS 1893 1984 and IS 1893 2002 brings some conclusion such as the structures that are made following the old code needs a retrofitting this can be well understood from a case study:

Considering a symmetric building of 8 stoery, on analysis through the recommendations of IS 1893 :1984 and IS 1893(Part 1): 2002 on calculation of Base Shear and Distribution of design forces to different floor levels we obtain certain conclusions.

				v b	543.3209		
W1	1467.371	h 1	3.5	w1 h1^2	17975.29	Q 1	2.694836
W2	1467.377	h2	7	w2h2^2	71901.47	Q 2	10.77939
W3	1467.376	h3	10.5	w3h3^2	161778.2	Q 3	24.25361
W4	1467.375	h4	14	w4h4^2	287605.5	Q 4	43.1175

W5	1467.377	h5	17.5	w5h5^2	449384.2	Q 5	67.37119
W6	1467.374	h6	21	w6h6^2	647111.9	Q 6	97.01431
W7	1467.377	h7	24.5	w7h7^2	880793	Q 7	132.0475
W8	1412.69	h8	28	w8h8^2	1107549	Q 8	166.0425
sum	11684.32			sum	3624099		543.3209
W1	1467.371	h 1	3.5	w1 h1^2	17975.29	Q 1	5.180119
W2	1467.377	h2	7	w2h2^2	71901.47	Q 2	20.72056
W3	1467.376	h3	10.5	w3h3^2	161778.2	Q 3	46.62123
W4	1467.375	h4	14	w4h4^2	287605.5	Q 4	82.88213
W5	1467.377	h5	17.5	w5h5^2	449384.2	Q 5	129.5035
W6	1467.374	h6	21	w6h6^2	647111.9	Q 6	186.4847
W7	1467.377	h7	24.5	w7h7^2	880793	Q 7	253.8269
W8	1412.69	h8	28	w8h8^2	1107549	Q 8	319.1734
sum					3624099		1044.392
		T a	0.912914	sa/g	1.489735		
		A h	0.089384				

Calculation of base shear through IS 1893 (Part 1) 2002 and through IS 1893 1984 shows that base shear for a case of Zone (iv), Medium soil, Sensitive building (I=1.5) is about 0.92 times of that IS 1893 1984.

The analysis through Staad Pro 2004 of both the cases gives us moment variation in a frame of building as

Frames of a similar building with different floors were again taken and it was observed that:

(1) 3*4

(2) 4*4

(3) 6*4

(4) 8*4

(1) 3*4

	3 storey		
Beam no	2002	1984	Diff (KN m)
5040	-287.208	-124.962	-162.246
5040	320.638	181.637	139.001
5041	-207.085	-81.161	-125.924
5041	299.706	171.964	127.742
5042	-210.404	-83.091	-127.313
5042	294.872	169.818	125.054
5043	-231.796	-94.581	-137.215
5043	356.421	196.852	159.569
5044	-234.742	-94.947	-139.795
5044	279.893	158.667	121.226
5045	-186.022	-71.684	-114.338
5045	274.033	159.582	114.451
5046	-182.079	-68.474	-113.605
5046	267.903	155.279	112.624
5047	-197.592	-80.075	-117.517
5047	313.216	179.032	134.184
5048	-106.512	-36.587	-69.925
5048	152.554	96.577	55.977
5049	-64.689	-10.616	-54.073
5049	152.381	98.559	53.822
5050	-61.713	-8.489	-53.224
5050	149.246	96.352	52.894
5051	-65.886	-12.643	-53.243
5051	164.082	98.71	65.372

(2) 4*4

	4 storey		
	2002	1984	DIFF(KNm)
Beam no	Moment	Moment	
5040	-401.9	-159.442	-242.458
5040	419.87	211.558	208.312
5041	-303.232	-111.931	-191.301
5041	396.634	202.731	193.903

5042	-307.929	-114.323	-193.606
5042	391.311	200.614	190.697
5043	-335.555	-128.383	-207.172
5043	476.561	235.865	240.696
5044	-378.74	-144.79	-233.95
5044	403.982	200.765	203.217
5045	-311.307	-116.402	-194.905
5045	399.975	204.465	195.51
5046	-309.902	-114.98	-194.922
5046	395.268	201.558	193.71
5047	-330.87	-129.919	-200.951
5047	465.659	235.189	230.47
5048	-268.025	-92.065	-175.96
5048	306.027	154.289	151.738
5049	-234.038	-82.244	-151.794
5049	319.679	168.504	151.175
5050	-228.442	-78.15	-150.292
5050	314.676	164.677	149.999
5051	-239.203	-91.064	-148.139
5051	361.764	191.34	170.424
5052	-110.746	-28.759	-81.987
5052	152.279	87.859	64.42
5053	-85.845	-15.56	-70.285
5053	169.029	100.674	68.355
5054	-78.445	-10.689	-67.756
5054	167.567	98.476	69.091
5055	-77.536	-15.558	-61.978
5055	179.784	101.979	77.805

	6 storey		
	2002	1984	DIFF (KNm)
Beam no	Moment	Moment	
5040	-1142.33	-703.663	-438.67
5040	1056.453	679.297	377.156
5041	-903.923	-552.254	-351.669
5041	1004.042	647.987	356.055
5042	-916.897	-560.902	-355.995
5042	993.166	641.625	351.541
5043	-977.943	-600.992	-376.951
5043	1223.622	785.284	438.338
5044	-1157.25	-709.354	-447.893
5044	1082.16	692.235	389.925
5045	-990.64	-607.381	-383.259
5045	1079.182	695.244	383.938
5046	-992.045	-608.225	-383.82

5046	1076.558	693.552	383.006
5047	-1020.59	-631.073	-389.52
5047	1257.594	810.358	447.236
5048	-1050.19	-639.699	-410.489
5048	981.869	626.75	355.119
5049	-932.328	-570.862	-361.466
5049	1016.033	655.33	360.703
5050	-929.671	-569.144	-360.527
5050	1016.95	655.86	361.09
5051	-932.514	-578.012	-354.502
5051	1161.927	752.432	409.495
5052	-855.438	-515.472	-339.966
5052	807.046	514.992	292.054
5053	-793.421	-483.544	-309.877
5053	872.227	564.674	307.553
5054	-786.548	-479.229	-307.319
5054	877.178	567.801	309.377
5055	-766.444	-475.239	-291.205
5055	975.202	636.59	338.612
5056	-547.756	-320.049	-227.707
5056	538.971	344.963	194.008
5057	-565.598	-340.441	-225.157
5057	639.35	418.113	221.237
5058	-553.591	-332.631	-220.96
5058	647.658	423.088	224.57
5059	-505.036	-312.054	-192.982
5059	676.43	450.359	226.071
5060	-206.219	-111.846	-94.373
5060	211.65	142.251	69.399
5061	-242.355	-137.617	-104.738
5061	304.555	207.653	96.902
5062	-217.811	-121.083	-96.728
5062	321.758	217.392	104.366
5063	-161.227	-92.409	-68.818
5063	300.086	206.783	93.303

(4) 8*4

	1984	2002	
	Mz kNm	Mz kNm	DIFF
5040	-232.073	-485.138	-253.065
5040	273.513	490.911	217.398
5041	-176.552	-380.94	-204.388
5041	267.08	473.768	206.688
5042	-181.13	-387.777	-206.647
5042	266.609	470.901	204.292
5043	-197.826	-415.093	-217.267
5043	316.515	569.353	252.838

5044	-230.72	-492.883	-262.163
5044	274.941	502.836	227.895
5045	-202.431	-429.552	-227.121
5045	288.591	515.684	227.093
5046	-203.829	-430.841	-227.012
5046	289.933	516.865	226.932
5047	-219.027	-446.663	-227.636
5047	337.361	599.076	261.715
5048	-211.476	-462.796	-251.32
5048	255.491	472.556	217.065
5049	-199.86	-424.431	-224.571
5049	283.793	507.399	223.606
5050	-201.022	-424.507	-223.485
5050	286.874	511.164	224.29
5051	-214.842	-431.513	-216.671
5051	332.839	583.481	250.642
5052	-184.104	-415.768	-231.664
5052	229.957	429.061	199.104
5053	-187.496	-400.539	-213.043
5053	269.417	480.602	211.185
5054	-188.415	-399.437	-211.022
5054	274.137	486.806	212.669
5055	-201.269	-399.838	-198.569
5055	317.327	548.077	230.75
5056	-147.136	-349.025	-201.889
5056	196.414	368.886	172.472
5057	-164.182	-355.693	-191.511
5057	244.532	433.49	188.958
5058	-165.051	-353.806	-188.755
5058	250.652	441.697	191.045
5059	-176.891	-348.676	-171.785
5059	289.453	490.182	200.729
5060	-97.509	-256.71	-159.201
5060	151.853	286.458	134.605
5061	-127.888	-286.057	-158.169
5061	207.022	362.007	154.985
5062	-128.839	-283.58	-154.741
5062	214.54	372.154	157.614
5063	-139.141	-272.888	-133.747
5063	246.207	403.977	157.77
5064	-29.147	-129.334	-100.187
5064	93.168	176.419	83.251
5065	-78.962	-191.626	-112.664
5065	156.848	265.744	108.896
5066	-79.035	-187.668	-108.633
5066	164.728	276.792	112.064

5067	-86.365	-168.678	-82.313
5067	186.435	285.056	98.621
5068	0	-21.403	-21.403
5068	41.852	66.342	24.49
5069	-18.708	-72.711	-54.003
5069	91.722	139.227	47.505
5070	-11.134	-58.482	-47.348
5070	98.001	151.625	53.624
5071	-10.753	-34.774	-24.021

BENDING MOMENT %INCREASE DUE TO UPGRADATION OF CODE

BEAM NO/STOREYS	5040	5044	5048	5052	5056	5060	5064	5068
4	49.61345	50.30348	49.58321	53.32203				
6	55.52152	56.32841	56.66039	56.7104	56.24023	48.7863		
8	79.48361	82.88869	84.95994	86.58314	87.81044	88.64165	89.35579	58.51572

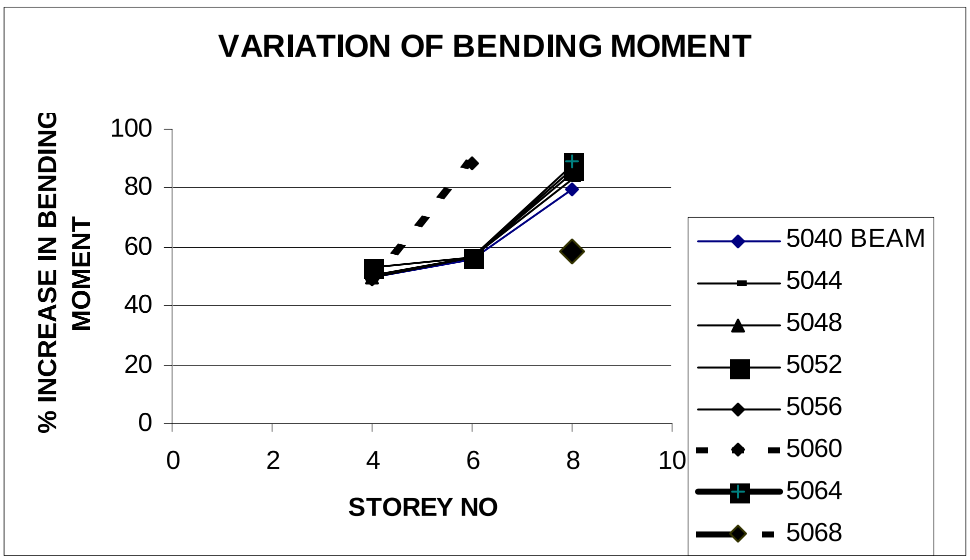

Fig 6.1 Variation of bending moment along different storey

On considering the moment variation along each floor

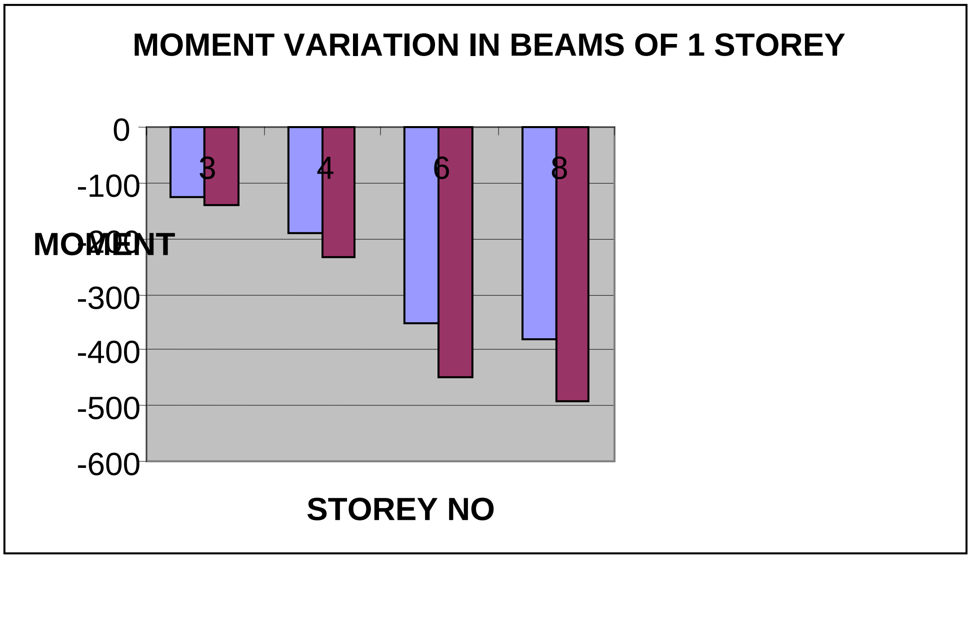

Fig.6.2: Negative Moment Variation in Beams of 1 storey

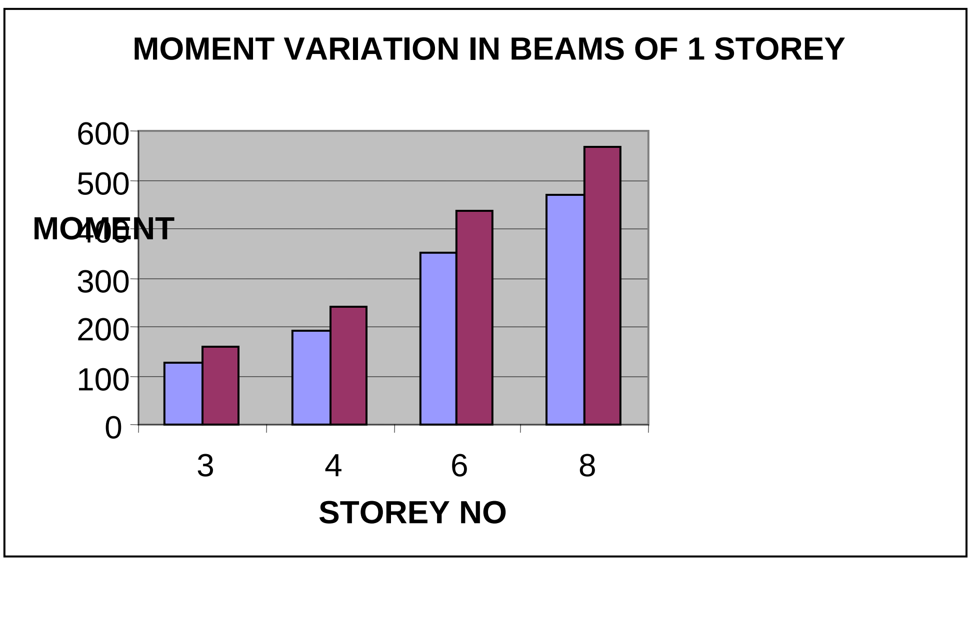

Fig.6.3: Positive Moment Variation in Beams of 1 storey

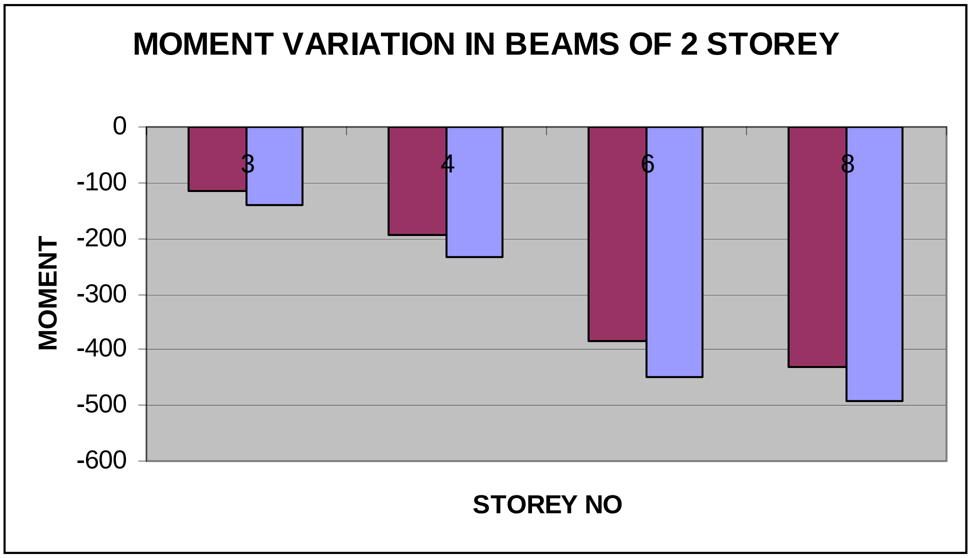

Fig.6.4: Negative Moment Variation in Beams of 2 storey

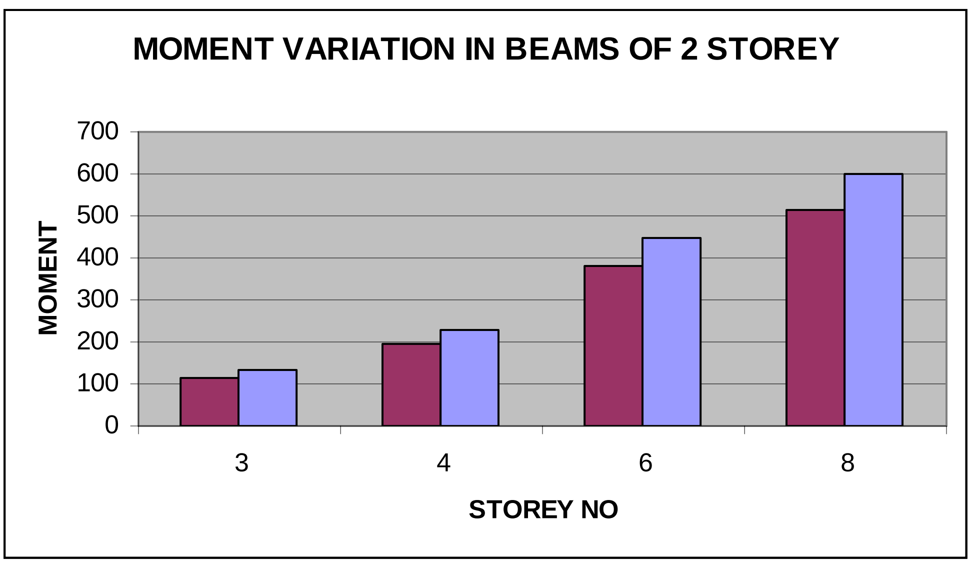

Fig.6.5: Positive Moment Variation in Beams of 2 storey

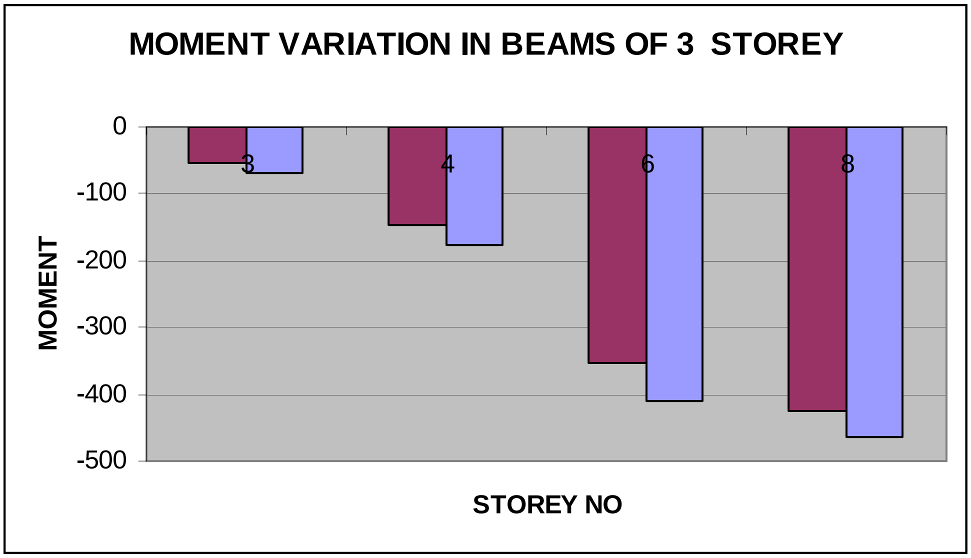

Fig.6.6: Negative Moment Variation in Beams of 3 storey

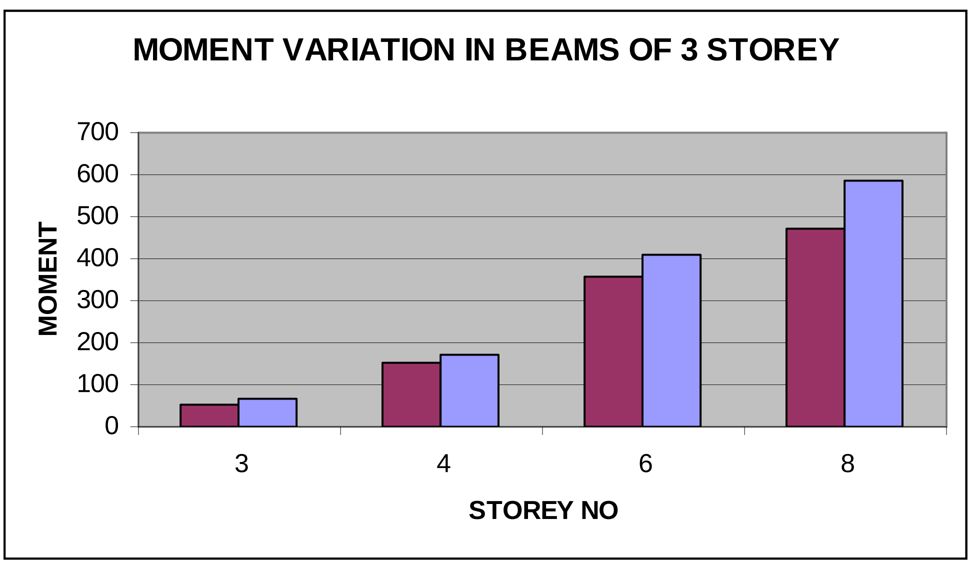

Fig.6.7: Positive Moment Variation in Beams of 3 storey

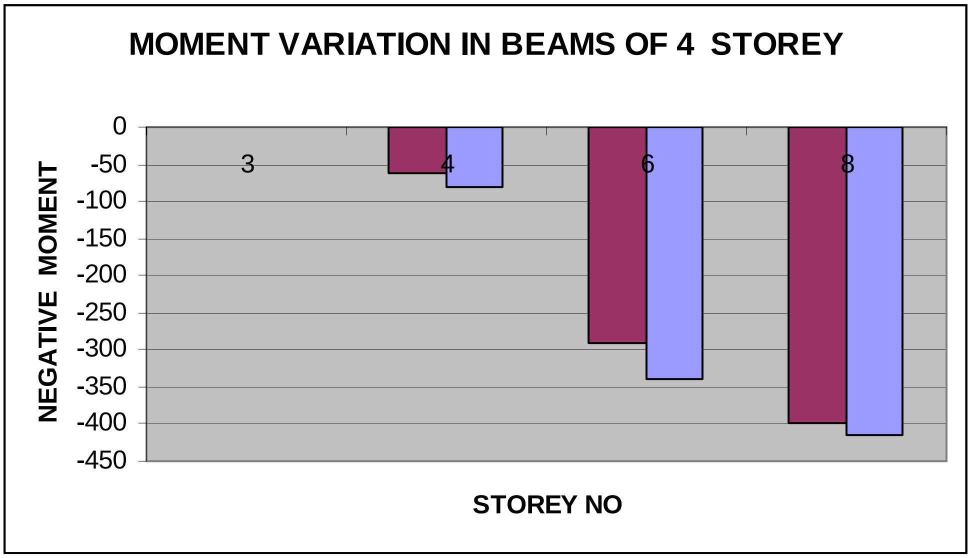

Fig.6.8: Negative Moment Variation in Beams of 4 storey

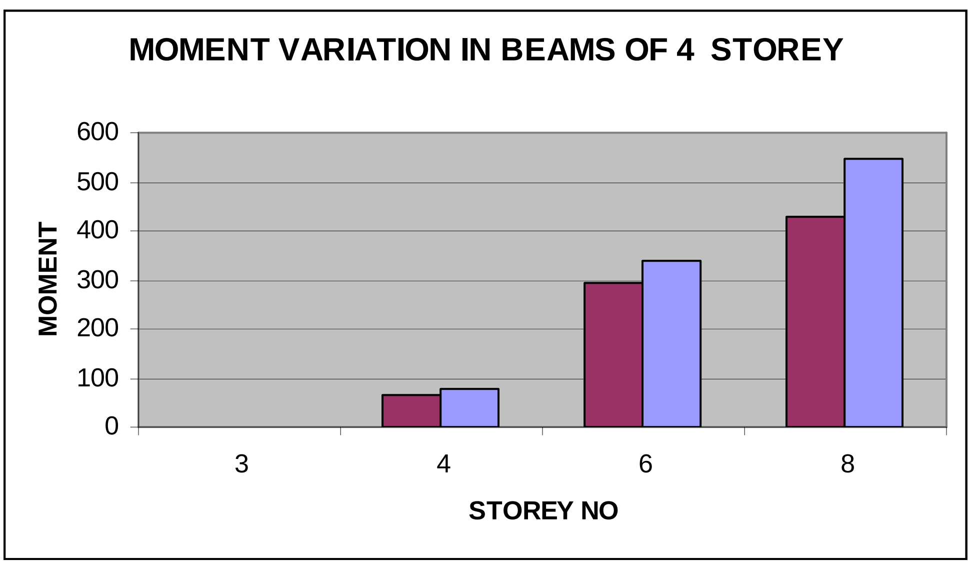

Fig.6.9: Positive Moment Variation in Beams of 4 storey

Conclusion:

1. ***Bay shear has increase by 0.9 times from the revision of code.***
2. ***As no of floors increase moment goes no increasing.***
3. ***Structure with higher no of floors require to strengthened.***
4. ***Storey drift also increased due to revision of code.***
5. ***For safety of life and in order to avoid causalities structure needs a retrofit.***

Chapter 7. Retrofitting through FRP

7.1 Design Equations

Assumptions:

- ✓ A plane section remains plane even after bending.
- ✓ A partial safety factor γ_{frp} is taken as 1.25

Based on above assumptions, the strains in the FRP, ε_{frp} and those in the steel bars, ε_{si} ,are related to the extreme compression fiber strain of concrete, ε_{cf} as follows:

$$\varepsilon_{frp} = \varepsilon_{cf} \frac{x - d_{frp}}{x} \quad \text{----(1)}$$

$$\varepsilon_{si} = \varepsilon_{cf} \frac{x - d_{si}}{x} \quad \text{-----(2)}$$

Where x, d_{si} and d_{frp} are the distances from the extreme concrete compression fiber to the neutral axis, the centroid of steel bars in layers i and the centroid of FRP respectively. The strain in FRP, ε_{frp} , is known, and hence

$$\varepsilon_{cf} = \varepsilon_{frp} \frac{x}{x - d_{frp}} \quad \text{-------(3)}$$

According to stress strain Fig 7, compressive stress in the concrete are given by fig 7

$$\sigma_c = 5500\left(\sqrt{\frac{f_{cu}}{\gamma_c}}\varepsilon_c - \frac{4100}{2}\varepsilon_c^2\right) \quad \text{if } 0 \le \varepsilon_c \le \varepsilon_{co} \quad \text{----------(4)}$$

and

$$\sigma_c = 0.67\frac{f_{cu}}{\gamma_c} \quad \text{if } \varepsilon_{co} \le \varepsilon_c \le 0.0035 \quad \text{----------(5)}$$

Where σ_c is the compressive concrete stress, ε_c is the compressive concrete strain which has a value of

$$\varepsilon_{co} = \frac{1}{4100}\sqrt{\frac{f_{cu}}{\gamma_c}} \quad \text{-----------(6)}$$

At peak stress of concrete, f_{cu} is the cube compressive strength of concrete, and γ_c is the partial safety factor for concrete.

For any given ε_{cf}, the total concrete compressive force C is

$$C = k_1 \frac{f_{cu}}{\gamma_c} b_c x \quad \text{-------------(7)}$$

Where b_c, is the beam width and k_1, is the mean stress factor

$$k_1 = \frac{\int_0^{\varepsilon_{cf}} \sigma_c d\varepsilon_c}{\left(\frac{f_{cu}}{\gamma_c}\right)\varepsilon_{cf}} \quad \text{--------(8)}$$

On substitution equations (6) & (7)

$$k_1 = 0.67\left(\frac{\varepsilon_{cf}}{\varepsilon_{co}} - \frac{\varepsilon_{cf}^2}{3\varepsilon_{cf}}\right) \quad \text{If } 0 \le \varepsilon_{cf} \le \varepsilon_{co} \quad \text{---------(9)}$$

and

$$k_1 = 0.67\left(1 - \frac{\varepsilon_{co}}{3\varepsilon_{cf}}\right) \quad \text{If } \varepsilon_{co} \le \varepsilon_{cf} \le 0.0035 \quad \text{----------(10)}$$

The depth of neutral axis x, can be determined

$$k_1 \frac{f_{cu}}{\gamma_c} b_c x + \sum_{i=0}^{n} \sigma_{si} A_{si} + \sigma_{frp} A_{frp} = 0 \quad \text{--------(11)}$$

Where σ_{si} and σ_{frp} are the stress in the steel bars and the FRP respectively, A_{si} is the total area of steel in layer i, n is the total number of steel layers, and A_{frp} is the area of the FRP, σ_{si} and σ_{frp} are given by

$$\sigma_{si} = E_{si}\varepsilon_{si} \quad \text{If } |\varepsilon_{si}| < \frac{f_y}{\gamma_s E_s} \quad \text{------(12)}$$

$$\sigma_{si} = \frac{\varepsilon_{si}}{|\varepsilon_{si}|}\frac{f_y}{\gamma_s} \quad \text{If} \quad |\varepsilon_{si}| \ge \frac{f_y}{\gamma_s E_s} \quad \text{----------(13)}$$

$$\sigma_{frp} = E_{frp}\varepsilon_{frp} \geq \frac{-f_{frp}}{\gamma_{frp}} \quad \text{----------(14)}$$

Where E_s and E_{frp} are the modulus of elasticity of steel bars and FRP respectively, f_y is the yield strength of steel, f_{frp} is the tensile strength of FRP, and γ_s and γ_{frp} are the partial safety factors for steel, and FRP respectively, material properties.

The position of concrete compression force C is defined by D, which is the distance from the extreme concrete compression fiber to the line of action of concrete compression force .D can be related to the height of the compression zone through

$$D = k_2 x \quad \text{-----------(15)}$$

Where the centroid factor k_2 of the compression force is given by

$$k_2 = 1 - \frac{\int_0^{\varepsilon_{cf}} \varepsilon_c \sigma_c d\varepsilon_c}{\varepsilon_{cf}\int_0^{\varepsilon_{cf}} \sigma_c d\varepsilon_c} \quad \text{----------(16)}$$

On simplification

$$k_2 = \frac{\frac{1}{3} - \frac{\varepsilon_{cf}}{(12\varepsilon_{c0})}}{1 - \frac{\varepsilon_{cf}}{(3\varepsilon_{co})}} \quad \text{If } 0 \leq \varepsilon_{cf} \leq \varepsilon_{co} \quad \text{---------(17)}$$

and

$$k_2 = \frac{\frac{\varepsilon_{cf}}{2} + \frac{\varepsilon_{co}^2}{12\varepsilon_{cf}} - \frac{\varepsilon_{co}}{3}}{\varepsilon_{cf} - \frac{\varepsilon_{co}}{3}} \quad \text{If } \varepsilon_{co} \leq \varepsilon_{cf} \leq 0.0035 \qquad \text{---------(18)}$$

The moment capacity of the beam M_U is finally determined by

$$M_U = k_1 \frac{f_{cu}}{\gamma_c} b_c x\left(\frac{h}{2} - k_2 x\right) + \sum_{i=1}^{n} \sigma_{si} A_{si}\left(\frac{h}{2} - d_{si}\right) + \sigma_{frp} A_{frp}\left(\frac{h}{2} - d_{frp}\right) \quad \text{---------(19)}$$

$$\rho_{frp,cr} = -\left(\frac{k_1\left(\frac{f_{cu}}{\gamma_c}\right)\left(\frac{x_{cr}}{h}\right) + \sum_{i=1}^{n} \sigma_{si}\rho_{si}}{\frac{f_{frp}}{\gamma_{frp}}}\right) \text{--------20}$$

The critical depth of neutral axis is given by, first of all is calculated

$$x_{cr} = \frac{0.0035}{0.0035 + \frac{f_{frp}}{(\gamma_{frp} E_{frp})}} d_{frp} \text{-----------(21)}$$

In case of concrete crushing $\varepsilon_{cf} = 0.0035$ and ε_{frp} is found from equation (1). The depth of neutral axis *x* can be calculated from (21) making use of equation (1), (2), (12), (13) and (14) area of FRP is known.

7.2 Stress limitation

Under service load conditions it is required to limit stresses in the concrete, steel and FRP to prevent damage or excessive creep of the concrete, steel yielding and excessive creep or creep rupture of the FRP. If external tensile reinforcement is added and as the compression force equals the total tensile force, a significant change in the state of concrete stress may be expected. To prevent excessive compression, producing longitudinal cracks and irreversible strains, the following limitations for the concrete compressive stress apply:

$$\sigma_c \leq 0.45 f_{ck} \text{ ----(22)}$$

$\sigma_c = E_c \varepsilon_c$ Hence ε_c is calculated.

$$\sigma_s = E_s \varepsilon_c \left(\frac{d - x_e}{x_e} \right) \leq 0.80 f_{yk} \text{ --------(23) }$$ x_e is calculated

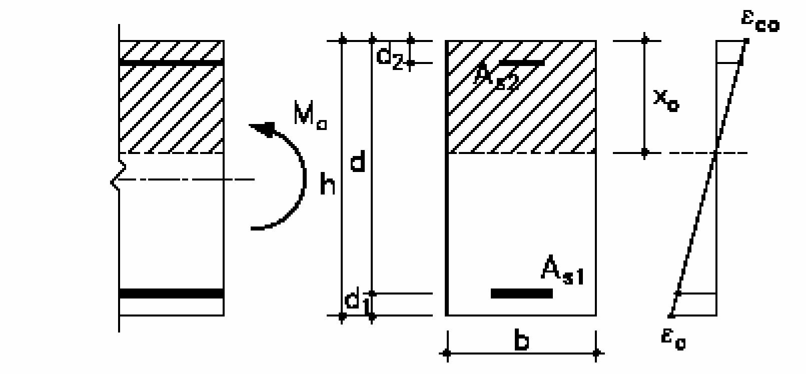

Fig.7.1: Stress Block

Based on the transformed cracked section, the neutral axis depth x_o can be solved from:

$$\frac{1}{2} b_c x_o^2 = \alpha_s A_{si} \left(d - x_o \right) \text{---------(24)}$$

Where α_s = Es/Ec. The concrete strain ε_{co} at the top fiber calculated in equation (6)

$$\sigma_f = E_f \left(\varepsilon_c \left(\frac{h - x_e}{x_e} \right) - \varepsilon_o \right) \leq \eta f_{fk} \text{ ---------(25)}$$

Where η < 1 is the FRP stress limitation coefficient. This coefficient depends on the type of FRP and should be obtained through experiments. Based on creep rupture tests, indicative values of η = 0.8, 0.5 and 0.3 may be suggested for CFRP, AFRP and GFRP, respectively. Note that as the design is often governed by the SLS, relative low FRP strains at service load may be expected, so that FRP creep rupture is typically not of concern.

In the case of FRP rupture, $\varepsilon_{frp} = -\varepsilon_{frp,rup.}$, the failure mode involving steel yielding / FRP fracture is theoretically possible. However, it is quite likely that premature FRP debonding will precede FRP fracture and hence this mechanism will not be activated. For the sake of completeness we may state here that the analysis for this mechanism may be done along the lines of the previous section.

7.3 Case study

A Structure of 8*8 is considered and it is seen that in the first floor it has maximum difference of moment due to revision of code. A program in excel is made to calculated the area of FRP required for strengthening / retrofitting of structure.

Storey no	1					
NO of floors	3	4	6	8		
Beam NO						
5041	-125.924	-191.301	-351.669	-380.94	MIN MOMENT	
5040	-162.246	-242.458	-438.67	-492.883	MAXI MOMENT	
5042	125.054	190.697	351.541	470.901	MIN MOMENT	
5043	159.569	240.696	438.338	569.353	MAXI MOMENT	
2						
5046	-113.605	-194.905	-383.259	-429.552	MIN MOMENT	
5044	-139.795	-233.95	-447.893	-492.883	MAXI MOMENT	
5047	112.624	193.71	383.006	515.684	MIN MOMENT	
5046	134.184	230.47	447.236	599.076	MAXI MOMENT	
3						
5050	-53.224	-148.139	-354.502	-424.431	MIN MOMENT	
5048	-69.925	-175.96	-410.489	-462.796	MAXI MOMENT	
5050	52.894	151.175	355.119	472.556	MIN MOMENT	
5051	65.372	170	409.495	583.481	MAXI MOMENT	
4						
		-61.978	-291.205	-399.838	MIN MOMENT	
		-81.987	-339.966	-415.768	MAXI MOMENT	
		64.42	292.054	429.061	MIN MOMENT	
		77.805	338.612	548.077	MAXI MOMENT	

5			-192.982	-348.676	MIN MOMENT	
			-227.707	-355.693	MAXI MOMENT	
			194.008	368.886	MIN MOMENT	
			226.071	490.182	MAXI MOMENT	
6			-68.818	-256.71	MIN MOMENT	
			-104.738	-286.057	MAXI MOMENT	
			69.399	286.458	MIN MOMENT	
			96.902	403.977	MAXI MOMENT	

B E A M N O. 5047 D E S I G N R E S U L T S

M25 Fe415 (Main) Fe415 (Sec.)

LENGTH: 4000.0 mm SIZE: 350.0 mm X 600.0 mm COVER: 25.0 mm

SUMMARY OF REINF. AREA (Sq.mm)

SECTION	0.0 mm	1000.0 mm	2000.0 mm	3000.0 mm	4000.0 mm
TOP REINF.	0.00 (Sq. mm)	0.00 (Sq. mm)	0.00 (Sq. mm)	1006.22 (Sq. mm)	2426.01 (Sq. mm)
BOTTOM REINF.	1657.26 (Sq. mm)	805.68 (Sq. mm)	407.90 (Sq. mm)	0.00 (Sq. mm)	75.87 (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION	0.0 mm	1000.0 mm	2000.0 mm	3000.0 mm	4000.0 mm
TOP REINF.	2-25í 1 layer(s)	2-25í 1 layer(s)	2-25í 1 layer(s)	3-25í 1 layer(s)	5-25í 1 layer(s)
BOTTOM REINF.	15-12í 2 layer(s)	8-12í 1 layer(s)	4-12í 1 layer(s)	2-12í 1 layer(s)	3-12í 1 layer(s)
SHEAR REINF.	2 legged 8í @ 250 mm c/c	2 legged 8í @ 250 mm c/c	2 legged 8í @ 200 mm c/c	2 legged 8í @ 250 mm c/c	2 legged 8í @ 250 mm c/c

This beam was designed for a moment of 337.361 KNM and it was observed that a moment of 599.076 KNM is to be resisted by the due to the upgrade of seismic design code from IS 1893 :1894 to IS 1893 (Part 1) :2002 now the beam is to be strengthened for a moment of 261.715 KNM.

EXCEL PROGRAMMING

f frp	2800		ε co	0.001091	f cu	30	b c	350
γ frp	1.25		K 1	0.600399	γ c	1.5	h	500
E FRP	165000		K 2	0.451069	A si	1250	E s	200000
d frp	500				fy	415	d si	450
ε cf	0.0035		ε frp	-0.01358	a sc	600	d	475
x cr	102.4845		ε si	-0.01187			d 2	25
		A FRP	943.5115					
				ε f	0.0107	0.010701		
				ε sc	0.013576			
				a frp,cr	911.1676			
		M u	599000000					
		A	451086.96	X1	-289.952			
		B	1665830					
		C	1629090.9					
		D	4202.7917					
		DK2	-1895.751					
		B"	-451.308					
		C"	-214930.2					
		X	741.2603					
E c	27386.13		ALPHA	7.302967				
b	9128.709							
a	175							
c	4564355							
x o 1	137.5098							
X o 2	-189.674							
ε o	0.002875							

NO OF STROREYS	BEFORE STRENGTHENING	AFTER STRENGTHENING	NO OF STROREYS	BEFORE STRENGTHENING	AFTER STRENGTHENING
1	0.4219	0.4129	1	0.6261	0.5575
2	0.9805	0.9529	2	1.4187	1.2875
3	1.4692	1.4248	3	2.1323	1.9543
			4	2.7189	2.506
4	1.7856	1.7304	5	3.1498	2.9134
			6	3.994	3.154

NO OF STROREYS	BEFORE STRENGTHENING	AFTER STRENGTHENING
1	0.5732	0.5153
2	1.3572	1.174
3	1.8309	1.4572
4	2.9157	2.4608
5	3.6228	3.0393
6	4.23343	3.5357
7	4.7059	3.9135
8	4.9944	4.1336

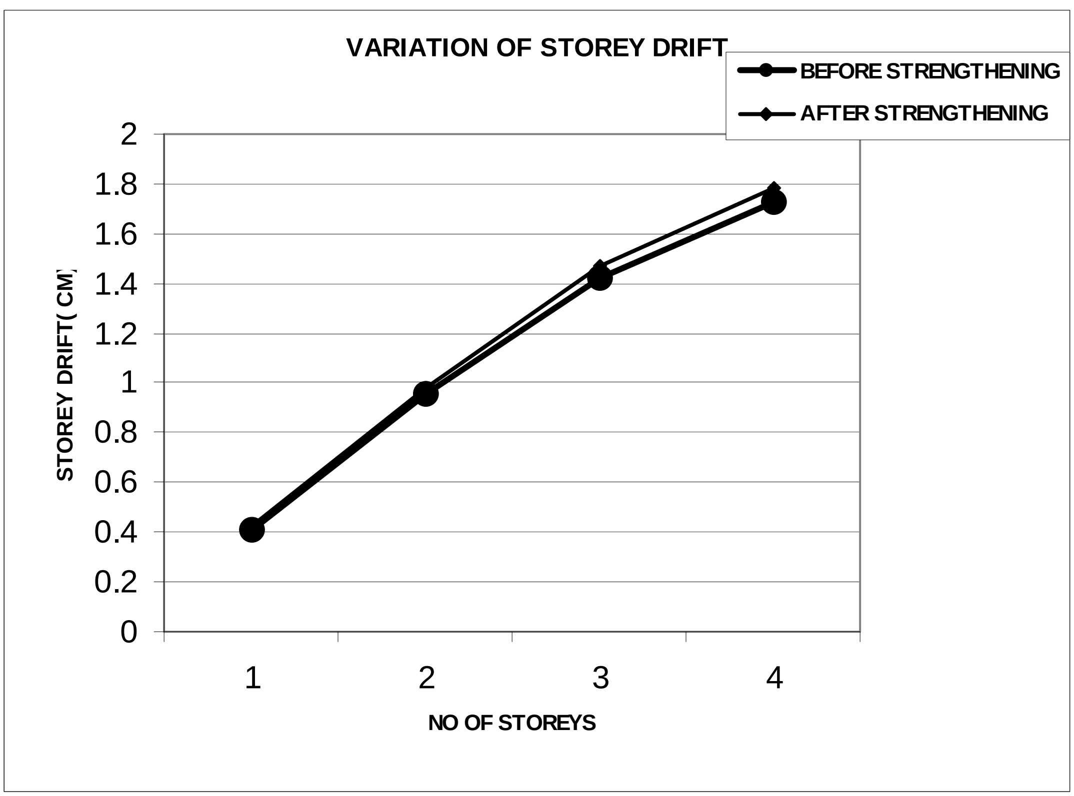

Fig 7.2: Variation of storey drift 4 storey

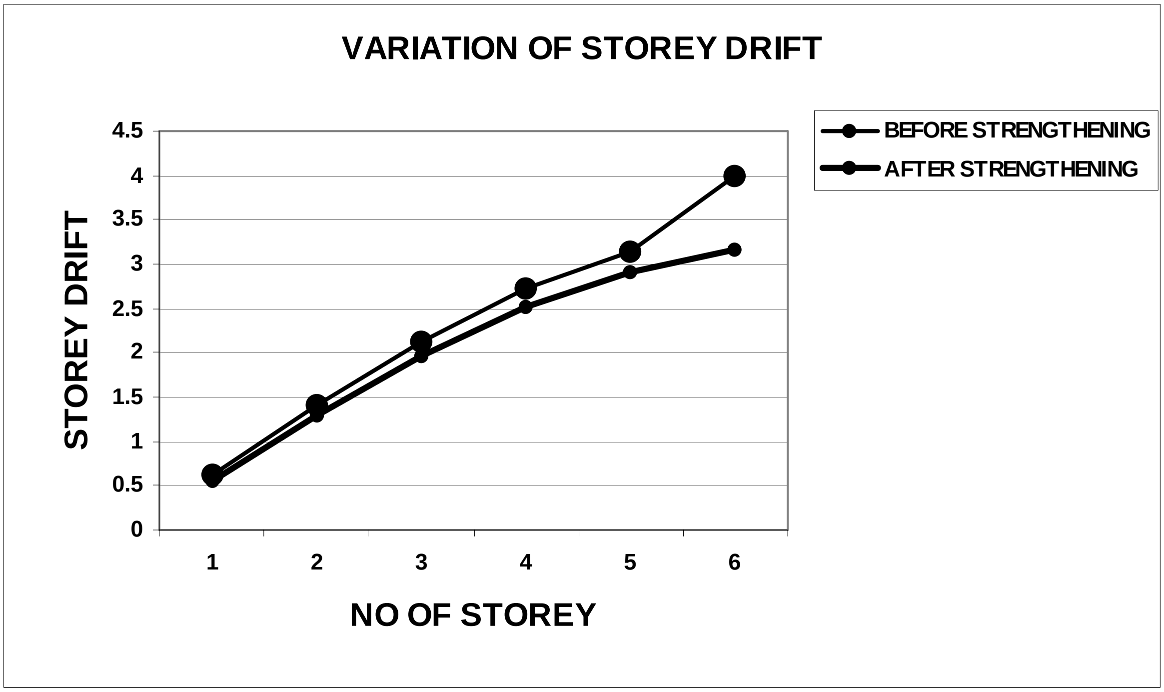

Fig 7.3: Variation of storey drift 6 storey

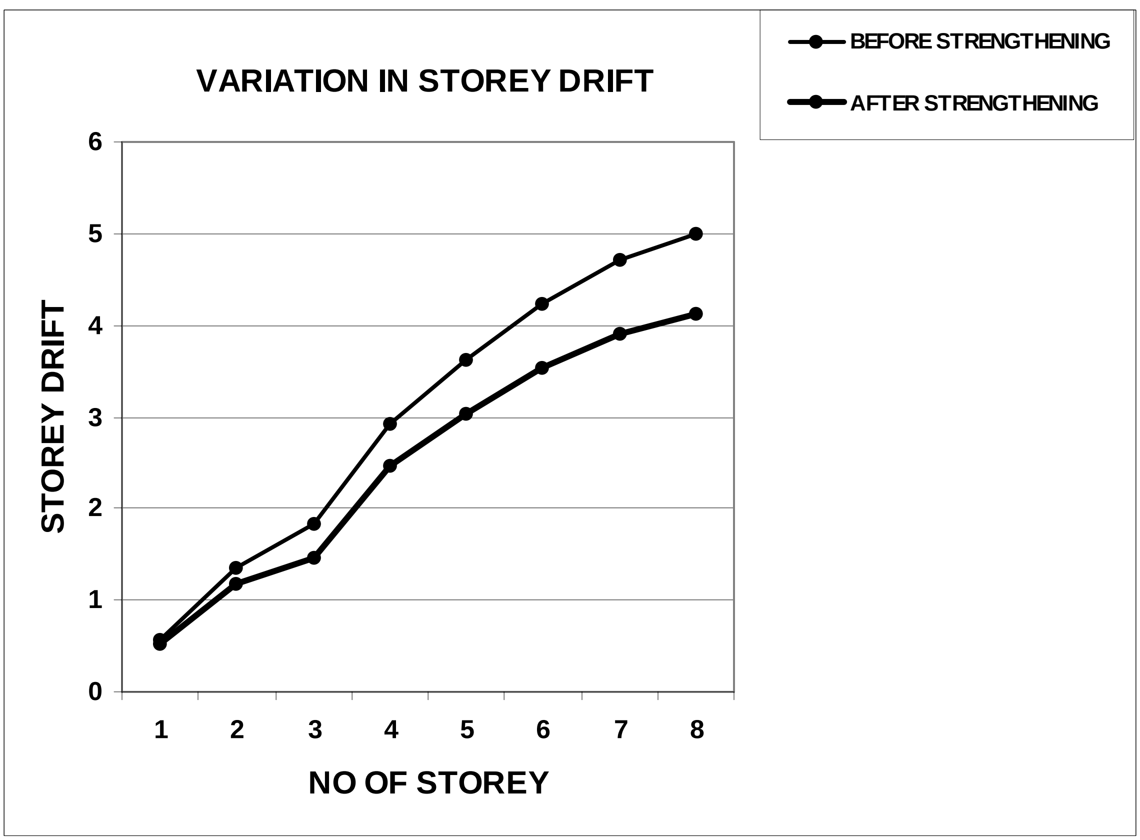

Fig 7.4: Variation of storey drift 8 storey

Conclusions:

1 On the application of FRP depth of neutral axis shifts downward

2 Compression concrete provides

3 FRP thickness required is less hence head room is not effected.

4 Storey drift after strengthening of beams has reduced from there original values.

5 As the no of storeys increase difference of drift before strengthening to after strengthening increases.

Discussion:

On Strengthening Storey drift has been reduced this gives the idea that the structure has been retrofitted.

Chapter 8:Conclusions & Scope of future work:

Conclusions:

- ✓ Upgradation of code has made changes in Base shear.
- ✓ Retrofitting has become an essential.
- ✓ FRP Strengthening technique could be used.
- ✓ Storey drift has been controlled through soffit plating of beams.

Scope of future work:

- ✓ Different debonding models could be used to calculate the thickness of FRP.
- ✓ FRP Strengthening could be used for different beam sizes and design charts could be provided.
- ✓ Shear strengthening of structural members could be compared.

References:

1 “Externally bonded FRPreinforcement for RC structures” fib Bulletin 14 A technical report“

2 Teng JG, Chen JF, Smith ST, Lam L.” RC structures strengthened with FRP composites.” Hong Kong, China: Research Centre for Advanced Technology in Structural Engineering, Department of Civil and Structural Engineering, The Hong Kong Polytechnic University, 2000.

3 Oehlers DJ. Discussion on “Theoretical study of the behaviour of reinforced concrete beams strengthened by externally bonded steel plates” by Roberts TM and Haji-Kazemi H. Proc Inst Civil Engnrs Part 2 1989;87:651–63.

4 Oehlers DJ. Discussion on “Prediction of peeling failure of reinforced concrete beams with externally bonded plates” by
Zhang S, Raoof M, Wood LA. Proc Inst Civil Engnrs Struct Build 1995;122:493–6.

5 Oehlers DJ, Ali M. Discussion on “Predictions of the maximum plate end stresses of FRP strengthened beams: Part II” by Quantrill RJ, Hollaway LC, Thorne AM. Mag Concrete Res 1998;50(1):91–2.

6 Smith ST, Teng JG. FRP-strengthened RC beams. I: Review of debonding strength models. Eng ng Struct 2001;24(4):385–395.

7 Smith ST, Teng JG. FRP-strengthened RC beams. II: Assesment of debonding strength models. Engng Struct 2001;24(4):385–395.

8 Raoof M, Hassanen MAH. Peeling failure of reinforced concrete beams with fibre-reinforced plastic or steel plates glued to their soffits. Proc Inst Civil Engnrs Struct Build 2000;140(August):291–305.

9 Raoof M, Zhang S. An insight into the structural behaviour of reinforced concrete beams with externally bonded plates. Proc Inst Civil Engnrs: Struct Build 1997;122(November):477–92.

10 Ziraba YN, Baluch MH, Basunbul IA, Sharif AM, Azad AK, Al- GJ. Guidelines towards the design of reinforced concrete beams with external plates. ACI Struct J 1994;91(6):639–46.

11 Sami Rizkalla , Tarek Hassan and Nahla Hassan Design recommendations for the use of the FRP as reinforcement and strengthening of concrete structures.

12 IS 456: 2000

13 IS 1893(Part I) :2002

14 IS 1893: 1984

Appendix 1

STAAD input

STAAD SPACE EIGHT STOREY
START JOB INFORMATION
ENGINEER DATE 05-Jul-06
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
51 0 3.5 4; 52 4 3.5 4; 53 8 3.5 4; 54 12 3.5 4; 55 16 3.5 4; 56 0 7 4;
57 4 7 4; 58 8 7 4; 59 12 7 4; 60 16 7 4; 61 0 10.5 4; 62 4 10.5 4;
63 8 10.5 4; 64 12 10.5 4; 65 16 10.5 4; 66 0 14 4; 67 4 14 4; 68 8 14 4;
69 12 14 4; 70 16 14 4; 71 0 17.5 4; 72 4 17.5 4; 73 8 17.5 4; 74 12 17.5 4;
75 16 17.5 4; 76 0 21 4; 77 4 21 4; 78 8 21 4; 79 12 21 4; 80 16 21 4;
81 0 24.5 4; 82 4 24.5 4; 83 8 24.5 4; 84 12 24.5 4; 85 16 24.5 4; 86 0 28 4;
87 4 28 4; 88 8 28 4; 89 12 28 4; 90 16 28 4; 91 0 0 8; 92 4 0 8; 93 8 0 8;
94 12 0 8; 95 16 0 8; 96 0 3.5 8; 97 4 3.5 8; 98 8 3.5 8; 99 12 3.5 8;
100 16 3.5 8; 101 0 7 8; 102 4 7 8; 103 8 7 8; 104 12 7 8; 105 16 7 8;
106 0 10.5 8; 107 4 10.5 8; 108 8 10.5 8; 109 12 10.5 8; 110 16 10.5 8;
111 0 14 8; 112 4 14 8; 113 8 14 8; 114 12 14 8; 115 16 14 8; 116 0 17.5 8;
117 4 17.5 8; 118 8 17.5 8; 119 12 17.5 8; 120 16 17.5 8; 121 0 21 8;
122 4 21 8; 123 8 21 8; 124 12 21 8; 125 16 21 8; 126 0 24.5 8; 127 4 24.5 8;
128 8 24.5 8; 129 12 24.5 8; 130 16 24.5 8; 131 0 28 8; 132 4 28 8; 133 8 28 8;
134 12 28 8; 135 16 28 8; 141 0 3.5 12; 142 4 3.5 12; 143 8 3.5 12;
144 12 3.5 12; 145 16 3.5 12; 146 0 7 12; 147 4 7 12; 148 8 7 12; 149 12 7 12;
150 16 7 12; 151 0 10.5 12; 152 4 10.5 12; 153 8 10.5 12; 154 12 10.5 12;
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175 16 24.5 12; 176 0 28 12; 177 4 28 12; 178 8 28 12; 179 12 28 12;
180 16 28 12;
MEMBER INCIDENCES
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5215 52 51;
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ISOTROPIC CONCRETE
E 2.5e+007
POISSON 0.17
DENSITY 25
ALPHA 1e-005
DAMP 0.05
END DEFINE MATERIAL
CONSTANTS
MATERIAL CONCRETE MEMB 5000 TO 5215
MEMBER PROPERTY INDIAN
5000 TO 5039 PRIS YD 0.5 ZD 0.5
5040 TO 5215 PRIS YD 0.6 ZD 0.35
SUPPORTS
91 TO 95 FIXED

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51 TO 90 141 TO 180 FIXED BUT FX FZ MX MY MZ
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*DEFINE 1893 LOAD
*ZONE 0.24 RF 3 I 1.5 SS 2 ST 2
*SELFWEIGHT
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*JOINT LOAD
*1 FX 10.938
*2 FX 10.938
*3 FX 10.938
*4 FX 10.938
*5 FX 10.938
*6 FX 72.575
*7 FX 121.8
*8 FX 115.976
*9 FX 121.8
*10 FX 72.575
*11 FX 73.75
*12 FX 121.846
*13 FX 117.257
*14 FX 121.846
*15 FX 73.75
*16 FX 73.614
*17 FX 121.852
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*194 FX 121.846
*195 FX 73.75
*196 FX 73.614
*197 FX 121.852
*198 FX 117.096
*199 FX 121.852
*200 FX 73.614
*201 FX 73.63
*202 FX 121.85
*203 FX 117.116
*204 FX 121.85
*205 FX 73.63
*206 FX 73.633
*207 FX 121.85
*208 FX 117.12
*209 FX 121.85
*210 FX 73.633
*211 FX 73.59
*212 FX 121.855
*213 FX 117.066
*214 FX 121.855
*215 FX 73.59
*216 FX 73.948
*217 FX 121.834
*218 FX 117.496
*219 FX 121.834
*220 FX 73.948
*221 FX 59.949
*222 FX 110.838
*223 FX 103.149
*224 FX 110.838
*225 FX 59.949
*1 FZ 10.938
*2 FZ 10.938
*3 FZ 10.938
*4 FZ 10.938
*5 FZ 10.938
*6 FZ 72.575
*7 FZ 121.8
*8 FZ 115.976

*9 FZ 121.8
*10 FZ 72.575
*11 FZ 73.75
*12 FZ 121.846
*13 FZ 117.257
*14 FZ 121.846
*15 FZ 73.75
*16 FZ 73.614
*17 FZ 121.852
*18 FZ 117.096
*19 FZ 121.852
*20 FZ 73.614
*21 FZ 73.63
*22 FZ 121.85
*23 FZ 117.116
*24 FZ 121.85
*25 FZ 73.63
*26 FZ 73.633
*27 FZ 121.85
*28 FZ 117.12
*29 FZ 121.85
*30 FZ 73.633
*31 FZ 73.59
*32 FZ 121.855
*33 FZ 117.066
*34 FZ 121.855
*35 FZ 73.59
*36 FZ 73.948
*37 FZ 121.834
*38 FZ 117.496
*39 FZ 121.834
*40 FZ 73.948
*41 FZ 59.949
*42 FZ 110.838
*43 FZ 103.149
*44 FZ 110.838
*45 FZ 59.949
*51 FZ 121.8
*52 FZ 206.764
*53 FZ 198.374
*54 FZ 206.764
*55 FZ 121.8
*56 FZ 121.846
*57 FZ 204.49
*58 FZ 197.796
*59 FZ 204.49

*60 FZ 121.846
*61 FZ 121.852
*62 FZ 204.766
*63 FZ 197.851
*64 FZ 204.766
*65 FZ 121.852
*66 FZ 121.85
*67 FZ 204.734
*68 FZ 197.846
*69 FZ 204.734
*70 FZ 121.85
*71 FZ 121.85
*72 FZ 204.727
*73 FZ 197.845
*74 FZ 204.727
*75 FZ 121.85
*76 FZ 121.855
*77 FZ 204.816
*78 FZ 197.859
*79 FZ 204.816
*80 FZ 121.855
*81 FZ 121.834
*82 FZ 204.087
*83 FZ 197.719
*84 FZ 204.087
*85 FZ 121.834
*86 FZ 110.838
*87 FZ 199.14
*88 FZ 188.225
*89 FZ 199.14
*90 FZ 110.838
*91 FZ 10.938
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*96 FZ 115.976
*97 FZ 198.374
*98 FZ 189.72
*99 FZ 198.374
*100 FZ 115.976
*101 FZ 117.257
*102 FZ 197.796
*103 FZ 190.933
*104 FZ 197.796
*105 FZ 117.257

*106 FZ 117.096
*107 FZ 197.851
*108 FZ 190.751
*109 FZ 197.851
*110 FZ 117.096
*111 FZ 117.116
*112 FZ 197.846
*113 FZ 190.775
*114 FZ 197.846
*115 FZ 117.116
*116 FZ 117.12
*117 FZ 197.845
*118 FZ 190.78
*119 FZ 197.845
*120 FZ 117.12
*121 FZ 117.066
*122 FZ 197.859
*123 FZ 190.714
*124 FZ 197.859
*125 FZ 117.066
*126 FZ 117.496
*127 FZ 197.719
*128 FZ 191.208
*129 FZ 197.719
*130 FZ 117.496
*131 FZ 103.149
*132 FZ 188.225
*133 FZ 176.891
*134 FZ 188.225
*135 FZ 103.149
*141 FZ 121.8
*142 FZ 206.764
*143 FZ 198.374
*144 FZ 206.764
*145 FZ 121.8
*146 FZ 121.846
*147 FZ 204.49
*148 FZ 197.796
*149 FZ 204.49
*150 FZ 121.846
*151 FZ 121.852
*152 FZ 204.766
*153 FZ 197.851
*154 FZ 204.766
*155 FZ 121.852
*156 FZ 121.85

*157 FZ 204.734
*158 FZ 197.846
*159 FZ 204.734
*160 FZ 121.85
*161 FZ 121.85
*162 FZ 204.727
*163 FZ 197.845
*164 FZ 204.727
*165 FZ 121.85
*166 FZ 121.855
*167 FZ 204.816
*168 FZ 197.859
*169 FZ 204.816
*170 FZ 121.855
*171 FZ 121.834
*172 FZ 204.087
*173 FZ 197.719
*174 FZ 204.087
*175 FZ 121.834
*176 FZ 110.838
*177 FZ 199.14
*178 FZ 188.225
*179 FZ 199.14
*180 FZ 110.838
*181 FZ 10.938
*182 FZ 10.938
*183 FZ 10.938
*184 FZ 10.938
*185 FZ 10.938
*186 FZ 72.575
*187 FZ 121.8
*188 FZ 115.976
*189 FZ 121.8
*190 FZ 72.575
*191 FZ 73.75
*192 FZ 121.846
*193 FZ 117.257
*194 FZ 121.846
*195 FZ 73.75
*196 FZ 73.614
*197 FZ 121.852
*198 FZ 117.096
*199 FZ 121.852
*200 FZ 73.614
*201 FZ 73.63
*202 FZ 121.85

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*203 FZ 117.116
*204 FZ 121.85
*205 FZ 73.63
*206 FZ 73.633
*207 FZ 121.85
*208 FZ 117.12
*209 FZ 121.85
*210 FZ 73.633
*211 FZ 73.59
*212 FZ 121.855
*213 FZ 117.066
*214 FZ 121.855
*215 FZ 73.59
*216 FZ 73.948
*217 FZ 121.834
*218 FZ 117.496
*219 FZ 121.834
*220 FZ 73.948
*221 FZ 59.949
*222 FZ 110.838
*223 FZ 103.149
*224 FZ 110.838
*225 FZ 59.949
*SPECTRUM CQC 1893 TOR X 0.06 ACC SCALE 1 DAMP 0.05 LIN MIS
*SOIL TYPE 2
*LOAD 2 EQZ
*SPECTRUM CQC 1893 TOR Z 0.06 ACC SCALE 1 DAMP 0.05 LIN MIS
*SOIL TYPE 2
LOAD 1 DL
SELFWEIGHT Y -1
FLOOR LOAD
YRANGE 0 28 FLOAD -5 XRANGE 0 16 ZRANGE 0 16
LOAD 2 LL
FLOOR LOAD
YRANGE 0 28 FLOAD -4 XRANGE 0 16 ZRANGE 4 12
LOAD 3 EQ
JOINT LOAD
96 FX 2.69484
101 FX 10.7794
106 FX 24.2536
111 FX 43.1175
116 FX 67.3712
121 FX 97.0143
126 FX 132.048
131 FX 166.043
LOAD COMB 4 1.5*(DL+LL)
```

```
1 1.5 2 1.5
LOAD COMB 5 1.2*(DL+LL+EQ)
1 1.2 2 1.2 3 1.2
LOAD COMB 6 1.2*(DL+LL-EQ)
1 1.2 2 1.2 3 -1.2
LOAD COMB 7 1.5*(DL+EQ)
1 1.5 3 1.5
LOAD COMB 8 1.5*(DL-EQ)
1 1.5 3 -1.5
LOAD COMB 9 0.9*DL+1.5*EQ
1 0.9 3 1.5
LOAD COMB 10 0.9*DL-1.5*EQ
1 0.9 3 -1.5
PERFORM ANALYSIS
LOAD LIST 7
START CONCRETE DESIGN
CODE INDIAN
FC 25000 MEMB 5040 TO 5071
FYMAIN 415000 MEMB 5040 TO 5071
FYSEC 415000 MEMB 5040 TO 5071
MAXMAIN 32 MEMB 5040 TO 5071
MAXSEC 25 MEMB 5040 TO 5071
MINMAIN 10 MEMB 5040 TO 5071
MINSEC 8 MEMB 5040 TO 5071
DESIGN BEAM 5040 TO 5071
FINISH
20002
```