Ain Shams University
Faculty of Enginering
Structural Eng. Dept.

Repair and Strengthening of Reinforced Concrete Structures

Submitted by

Eng. MOHAMED ABD ELMONEAM ZAKY

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Supervised by

ASSOCIATED PROF. DR. KHALID MOHAMED MORSY

STRUCTURAL ENGINEERING DEPARTMENT

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ABSTRACT

Repair and strengthening of damaged or vulnerable reinforced concrete structures is important in order to guarantee the safety of residents or users. Beams are important structural elements for withstanding loads, so finding the efficient repair and strengthening methods are necessary in terms of maintaining the safety of the structures.

This research study investigated various repair, retrofit, and strengthening techniques for reinforced concrete beams. The comparison and summary of each repair and strengthening method are provided in this thesis.

The thesis involves the literature review of current experimental test of repair and strengthening techniques for reinforced concrete beams. The experimental studies were summarized by describing the specimens and loading details, All the methods in the research were categorized into five chapters: section enlargement and concrete jacketing, external reinforcement, steel plates, unbonded-type strengthening, and concrete repairs. The installation procedures were summarized and the advantages, shortcomings, and considerations of each method were also discussed in the thesis.

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Chapter (1)

Introduction to Repair and Strengthening of Reinforced Concrete Structures

1 . 1 Introduction and Definitions :-

In design process we must take **two** important points in Considerations:-

1-**Safety** (the structure is safe under applied stresses) .

2-**Serviceability** (the structure in a good appearance such as

" no Deformations – no Cracks".

1.1.1Terminology of Repair & Strengthening

Repair :-

It is the process of increasing the performance of structure from minimum performance to initial performance .

This process are depending on available cost for the repair process.

High cost means high performance.

Strengthening :-

It is the process of increasing the performance of structures more than the initial performance that the structure was designed on it .

We need this process when the loads that on structures were increased .

It is the process of adding capacity to a member of structure.

Maintenance :-

It is the simple repair process to increase the performance of structure .

This process was applied at periodic time .

The next graph shows the main difference between

(repair – strengthening – maintenance).

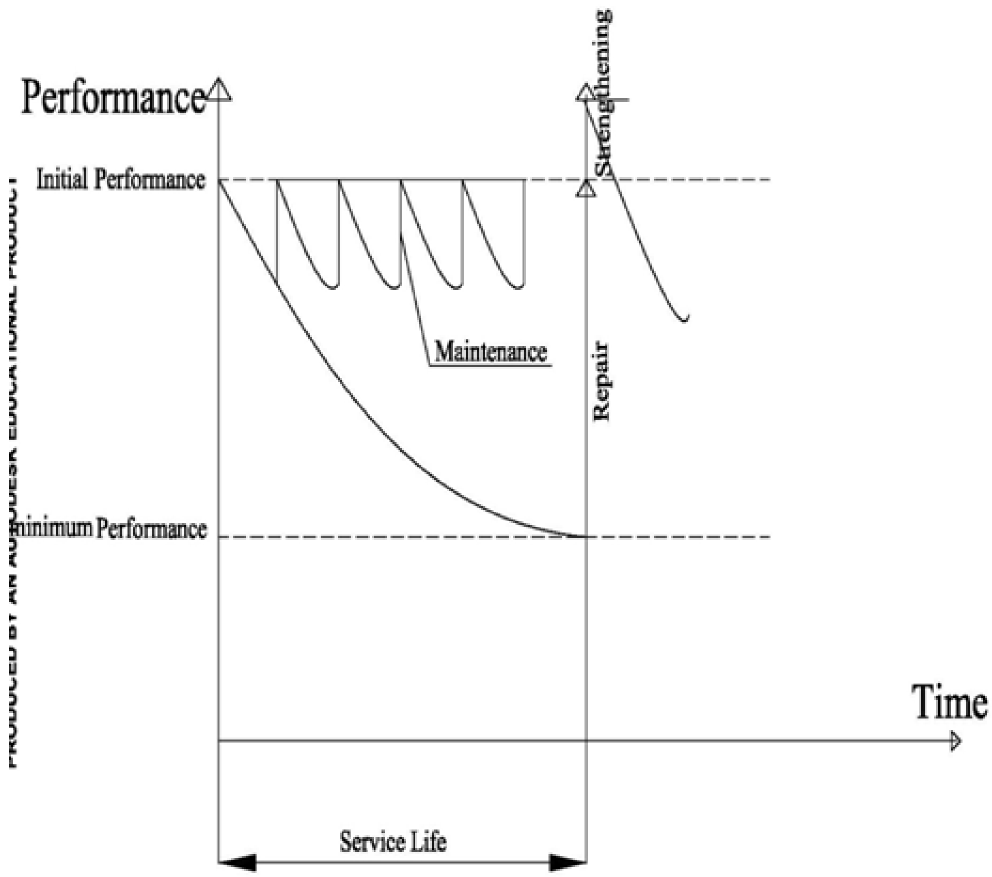


Fig 1.1 The main difference between

(Repair – Strengthening – Maintenance)

1.1.2 Basic Philosophy

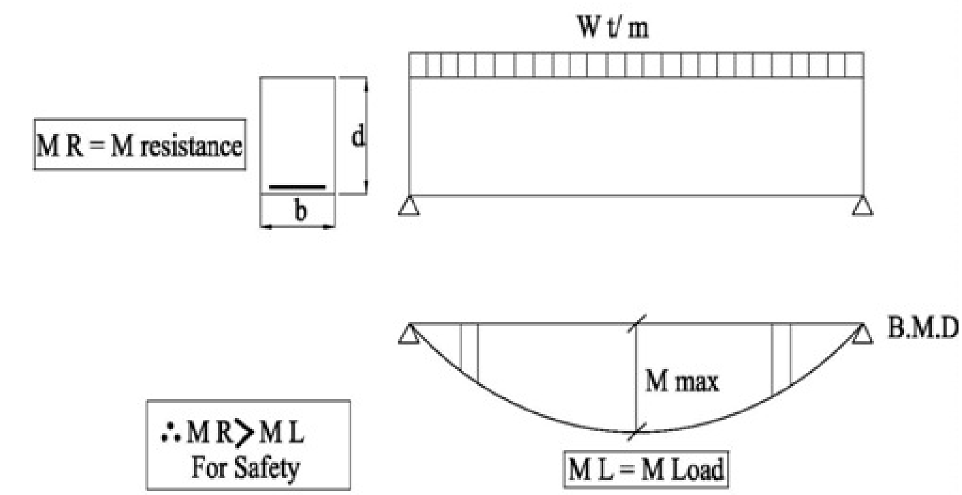


Fig 1.2- The main idea of design of reinforced concrete members

In strengthening

The resistance of structure element increased and subsequently the capacity of element increased for an additional load coming to the element.

1.1.3 Repair & Strengthening Techniques

A)-Increasing Resistance :-

- increasing cross section .

- add R.F.T .

- add Bonded material (steel plates - F.R.P) .

B)-Without Increase Resistance :-

- Reduce Dead Load . (change using of the structure).

- Change structure system.(add support for the beam).

1.1.4General Steps for Evaluate any Structure :-

A-Visual inspection :-

(discovere any problem in the structure such as cracking – spalling – disintegration , revision for statically system).

B-Non destructive tests :-

(Ultra sonic test – Core test – Loading test).

C-Selection of repair and strengthening materials :

(effective materials with optimum cost and suitable properties).

D-**Design process**

E-**Economic condition for owner**

1.2 Repair and Rehabilitation of Structures

This research deals with the latest techniques in repair and rehabilitation of structures. The various causes of structural failure and the principles of rehabilitation of structures are discussed.
Major repair that are to be carried out in Brick walls, Plaster walls and RCC members are explained in detail and an in-depth analysis into Reinforced Cement Concrete repair options like

- Shotcrete method (Guniting)
- Form and Pump Method

The research also deals with the comparatively new Form and Pump technique developed for the past 20 years are discussed. Finally a case study of rehabilitation of the Freedom Towers (Florida,USA) was conducted where the latest techniques such as hydrodemolition, Abrasive blast cleaning, Form and Pump placement and Checkerboard pattern of construction are extensively applied.

1.2.1 Introduction To Rehabilitation and Repairs :-

A large stock of existing structures and infrastructure are deteriorated with use and time and might have passed their design life and require retrofitting and rehabilitation. The cost of retrofitting various infrastructures is estimated in the lakhs of rupees.To Overcome the ill effects caused by these deteriorated buildings Repair and Rehabilitation works are carried out from time to time. Many of the existing structures were designed to codes that have since been modified and upgraded. Change in use or higher loads and performance demands require modifications and strengthening of structural elements.

1.2.3 Why do some structures fall down ?

→Site Selection and Site Development Errors :-

Failures often result from unwise land use or site selection decisions. Certain sites are more vulnerable to failure. The most obvious examples are sites located in regions of significant seismic activity, in coastal regions, or in flood plains. Other sites pose problems related to specific soil conditions such as expansive soils or permafrost in cold regions.

→DESIGN ERRORS:

These failures include errors in concept; lack of structural redundancy; failure to consider a load or combination of loads; deficient connection details; calculation errors;
misuse of computer software; detailing problems including selection of incompatible materials, failure to consider maintenance requirements and durability; inadequate or inconsistent specifications for materials or expected quality of work and unclear communication of design intent.

→CONSTRUCTION ERRORS:

Such errors may involve excavation and equipment accidents; improper
sequencing; inadequate temporary support; excessive construction loads; premature removal of shoring or formwork; and non-conformance to design intent.

→MATERIAL DEFICIENCIES:

While it is true that most problems with materials are the result of human errors.
involving a lack of understanding about materials, there are failures that can be attributed to unforeseeable inconsistencies in materials.

→OPERATIONAL ERRORS

Failures can occur after occupancy of a facility as the result of owner/operator errors. These may include alterations made to the structure, change in use, negligent
overloading and
inadequate maintenance.

1.2.4 PRINCIPLES OF REHABILITATION

→ELIMINATION

Remove the materials that cause damage to buildings. This is no easy matter, because everything from the floor to the roofing may contain various
undesirable materials in the form of additives and admixtures.

→SEPARATION

Some things just can't be eliminated, but can still be protected. Use
sealants or foil backed drywall to separate structures

from damage causing sources.

→VENTILATION

Controlled, filtered ventilation may be the only way to insure that the air we bring indoors is ideal. High humidity air or extremely low humidity air can cause significant damage to concrete, plaster and brick walls.

1.2.5 GENERAL AREAS OF REPAIR/REHABILITATION WORK

- Repair, removal, replacement and maintenance of mechanical supports, sanitary treatment plant and pipelines.
- Repair and modifications to diffuser ports, aeration systems, and discharge pipelines.
- Installation and maintenance of dewatering structures.
- Pile restoration and wood pile concrete encapsulation.
- Anode installation for cathodic protection.
- Repair and replacement of trash-rack and debris screen.

1.2.6 MAJOR TYPES OF REPAIR

- Brick Wall Repairs
- Plaster Wall Repairs
- RCC Repairs

A) BRICK WALLS

Basically, brick is durable and long-lived as long as the mortar joints are sound. Brick houses are susceptible to moisture - more so than wooden framed houses - but require very little maintenance.

- **PROBLEMS WITH BRICK** (STRUCTURAL PROBLEMS)

- Deteriorated Pointing affects many old houses. Mortar starts to disintegrate between the bricks, which can cause the entire wall to collapse, or single bricks to crumble.
- Dirty or stained brickwork can be caused by moisture, time, dirt along with rain or sprinklers.
- Efflorescence results from bricks getting wet, which leaves deposits of salts that are drawn out of the masonry as the moisture evaporates the brickwork and find the source of the moisture.
- Spalled brickwork is also common. Once bricks have been wet, the expansion of freezing water breaks off the top

surface of the brick, leaving the inner surface exposed. After a time, most of these bricks will crumble completely.

A couple of Don'ts for brick

- Don't assume that old mortar needs to be replaced. Old mortar is usually of a higher lime content than the newer replacement mortar we are likely to find to repoint, and the high portland cement content of new mortar can damage old walls beyond repair.

- Don't seal bricks with a water repellent (i.e., water seal) - it can mean that any moisture that is already in the brick stays in the brick, and interior moisture may not be able to escape.

- Don't use hydrochloric acid to clean brick, it can cause discoloration or mottling that is permanent.

- Never sandblast old brick! Sandblasting can damage the hard surface of fired brick and open the bricks up to water damage.

- Never use expansion joints in historic masonry - they can pulverize brick and ruin mortar joints.

REPAIR WORK

Cleaning Brickwork

- For normal dirt and grime, simply use plain water, rinsing with a hose and scrubbing with a stiff bristled brush.
- For stubborn stains add 1/2c ammonia to a bucket of water.
- Don't use a powerwasher except as a last resort - if we have a crumbling brick problem, this will make it worse (old windows don't stand up to high pressure water very well).

Removal of Organic Growth

A moist brick will often lead to growth a variety of molds and mosses.

- First, scrape the moss or mold off the surface with a non-metallic spatula (the same kind used on Teflon).

- Second, apply a wash of 1 part bleach to 4 parts water to kill the spores.
- After a couple of days, scrape again and rewash. It will probably take a few applications to kill everything off.

B) PLASTER WALLS

Should you repair or replace?

It is usually better to go in favour of repairing plaster walls, regardless of what they look like. But honestly, this is not always possible.

Basically, if:

- there is more than 1 large hole per 4 x 8 area, or
- there are more than 3-4 cracks in $100ft^2$, or
- the cracks are more than 1/4" wide.

Then replace the section of wall. It will take more time and failed attempts to repair this wall than it is worth. Old plaster should be cherished - it is stronger and more soundproof than current walls made of gypsum board or sheetrock. Even cracking or crumbling plaster walls should be repaired, not replaced.

- **PLASTER DAMAGE** (NON-STRUCTURAL PROBLEMS)

  Plaster is pretty tough stuff, but like any wall, it's going to get banged or gouged, and age will take it's toll.

- Impact Damage can be serious problem in an old house. Over the years, the walls are going to get banged and dented. Generally we have to replace the plaster 6-12" from the visible hole to reach plaster that is still keyed to the lath tightly.

- Nearly every wall has a few nail holes. These can usually be fixed with a tiny bit of spackle applied with the finger. Not perfect, but they will be unnoticeable when the wall is painted.

- Water is the enemy of plaster. Brownish stains on the walls or ceilings are evidence for bowing out of plaster. Water-damaged plaster can be very friable.

- Old walls and old houses often have cracks. Stress cracks are a sign of possible structural shifting, extreme temperature changes, incorrect plaster mix, improper curing or leaks. Diagonal cracks over doorways signal settlement, or a nearby source of vibration, such as a highway or railroad.

1.2.7 REPAIRS

- For repair of minor cracks, use fiberglass mesh tape then go over with a wide trowel and joint compound. There are also plaster patch compounds available that are excellent.
- For larger cracks and holes, we need to remove all the debris and enlarge the crack until we reach solid plaster and fill the crack with joint compound or plaster patch.

- If we choose to put wallboard over the plaster, use the following tips:

  - Apply wallboard horizontally
  - Use the largest boards available.
  - Use screws, not nails, 12" apart in ceilings, 16" on walls
  - Use a floating joint - the wall holds up the ceiling sheets
  - Use corner clips at all corners
  - Use fiberglass mesh tape, not paper, and special compound that is available for plaster walls.
  - Caulk interior corners with acrylic latex caulk its not historically correct, but the effect is smooth and unnoticeable.

1.2.8 RCC STRUCTURES

PROBLEMS IN RCC STRUCTURES (STRUCTURAL PROBLEMS)

- Flexure, Shear, Torsion, Shrinkage And Tension cracks
- Splitting, Diagonal, Horizontal cracks in Columns
- Rusting, Buckling, Bending, Twisting Distress in Steel structures

1.2.9 METHOD OF REPAIR FOR RCC STRUCTURES

A) WETMIX SHOTCRETE

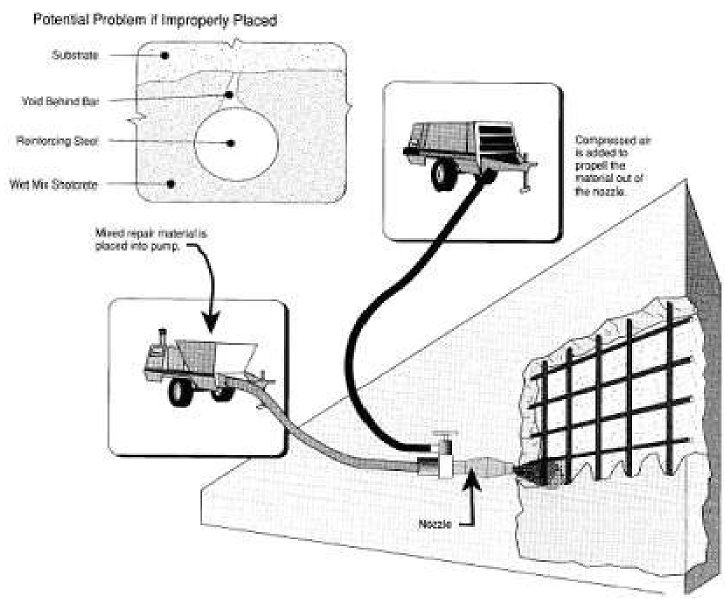


Fig 1.3 – (Describe wet mix shotcrete)

Wet mix shotcrete is a method that involves premixing of all ingredients including binder, water, aggregates and admixtures .The premixed repair materials are deposited into a pump which transports the materials to an exit nozzle where compressed air is introduced. The repair material is propelled onto the substrate with compressed air. Admixtures can be used to enhance durability. Air entrainment is required for freeze- thaw resistance. Fig(1.3).

B)→DRYMIX SHOTCRETE

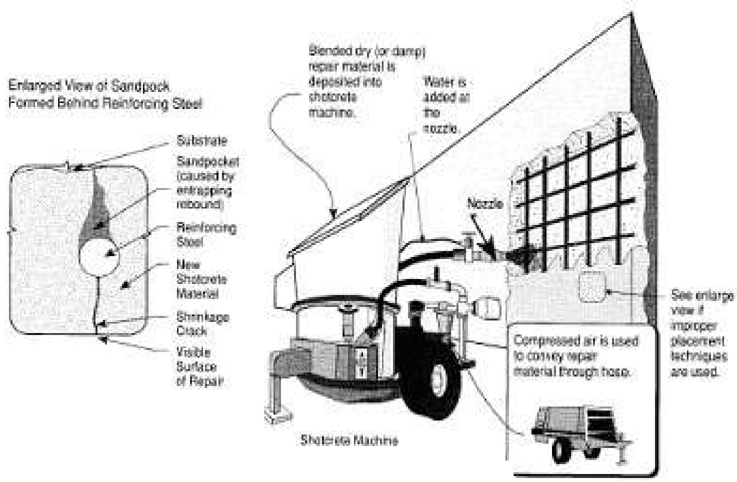


Fig 1.4 – describe dry mix shotcrete

- Problems associated with Drymix Shotcrete :
  - Presence of voids due to encapsulated rebound
  - Shrinkage cracking caused by high cement content , improper curing or excessive water control .

Dry mixing involves premixing of binders and aggregates which are fed into special mechanical feeder metering the premixed materials into a hose.The mix is jetted out along with compressed air from a nozzle connected to the hose having a water ring outfitted to it. This mix is injected to the repair spot.The resultant hardened properties include increased flexural, compressive strengths and more durability.

C) FORM AND PUMP TECHNIQUE

The form and pump repair method is a two step process of constructing formwork and pumping repair material into the cavity confined by formwork and existing concrete.

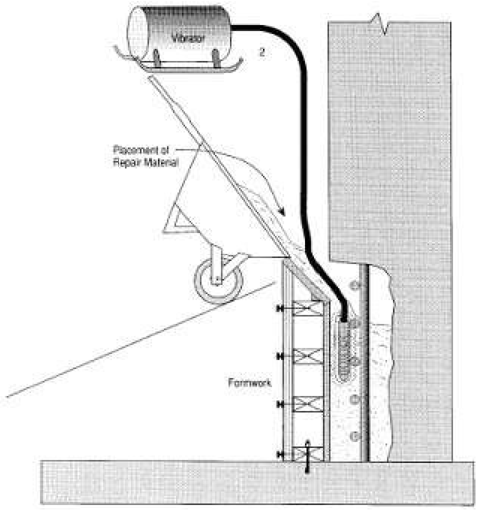
The form and pump technique allows use of different materials. Repair materials are mixed and pumped into the cavity. When the cavity is full, pump pressure is exerted into the form causing the repair material to consolidate and make contact with existing concrete surfaces.

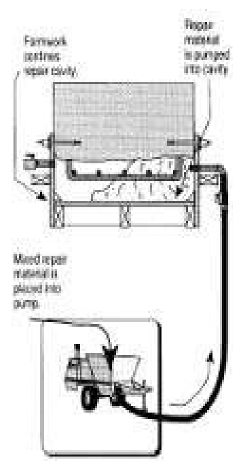
1.2.10 SURFACE REPAIR OF VERTICAL LOCATION(COLUMN)

One of the most common methods of surface repair of vertical and overhead location is placement of formwork and casting of repair material into the prepared cavity. The repair material must be of low shrinkage and necessary flow ability. Rodding or internal vibration is necessary to remove air and provide intimate contact for placing concrete substrate. In some applications complete filling of the cavity may be difficult. In those cases a final step of dry packing the remaining cavity works well.

1.2.11 SURFACE REPAIR OF OVERHEAD LOCATION (BEAM)

There are many techniques available to restore damaged or deteriorated concrete structures. Each surface repair techniques offer advantages and limitations depending upon the conditions of the repair project. Form a pump technique is relatively new method which has been developed as a viable alternative to Shotcrete(gunite), hand placement and grouted preplaced aggregate techniques.





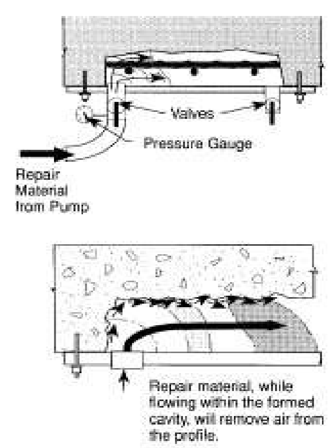


Fig 1.5 – describe Surface Repair of column & Beam

1.2.12.ADVANTAGES OF FORM AND PUMP TECHNIQUE:

- The use of almost any type of repair material- from fine grained mortar to course grained cement concrete.
- Placement is not limited by depth of repair, or by size or density of reinforcements.
- The pressurization process provides full encapsulation of exposed reinforcing steel.
- The formwork protects the repair material during curing process.

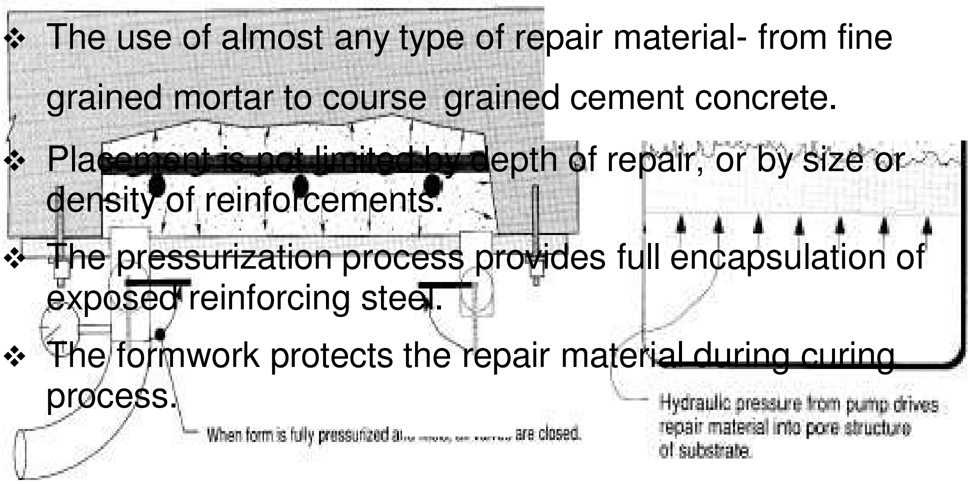


Fig 1.6 – Pump and Form Technique

1.2.13 PLACEMENT OF THE MATERIALS

The sequence of material placement into the formed cavity depends upon the geometrics involved. Vertical surfaces start at the lowest point, filling in a manner that prevents air entrapment. Arrangements for ports for pump line attachments are usually in grid form. When the flow is without the intrusion of air , the pump is shut off temporarily, the port closed off and pump line connected to the adjacent port which has seen flow. The sequence is carried out until the cavity is filled. Once the cavity is filled, the full line pressure is available to pressurize the formed cavity.

1.2.14 Selection of Mateials

Constructability requirements for materials used in form and pump method are limited only by their ability to be pumped and flow characteristics. The materials in-place properties like low drying shrinkage, compatibility, thermal and elastic properties. Drying shrinkage can cause cracking, delamination, inability to carry loads and low durability. Pumpability and flowability can be brought into the materials by additives and admixtures. Prepacked repair materials which are designed for pumping and incorporating shrinkage compensating additives are appropriate for many applications.

1.2.15 Compatibility of Repair and Substrate:-

The term " compatibility " has become very popular in the field of concrete repairs .It is always associated with the durability of repairs in general and with the load – carrying capacity of structural repairs . It had been suggested that failed repairs are the consequent of imperfect choices (the selection of repair materials incompatible with the substrate in a given environment .

Compatibility as shown in Figure 1.7 is the balance of physical , chemical , and electrochemical properties and dimensions between a repair material and the substrate that will ensure that the repair can withstand all the stresses induced by volume changes and chemical and electrochemical effects without distress and deterioration over a design period of time .

Recently , the selection of a repair material has been shifted from compressive strength , and low permeability to the combination of properties collectively called compatibility with existing substrate.

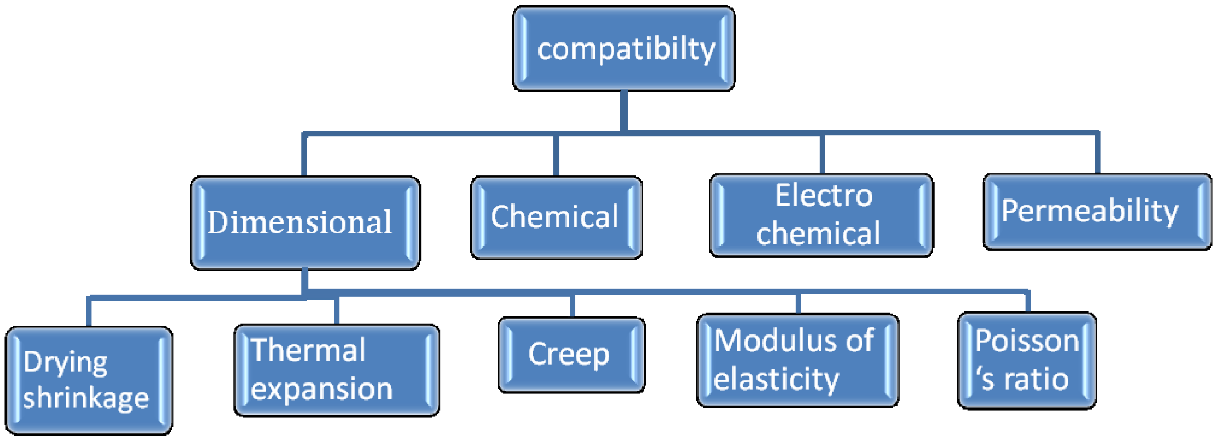


Fig. 1.7 Factors affecting compatibility of repair materials

1.3 Evaluation and Rehabilitation of Reinforced Concrete Structures:-

The extent of deterioration to concrete structures globally is occurring at an alarming rate, which challenges engineers on this continent and throughout the world on a daily basis. This includes damage to bridges, buildings, parking structures, environmental facilities, as well as other structures. Unfortunately, repair costs can be staggering. Delaying repairs usually results in much more costly repairs later. Furthermore, if concrete deterioration or damage is not addressed, some of these structures eventually may cease to be serviceable and worse yet, failures could occur.

There are a multitude of methods and materials available to repair concrete. Additionally, there is an abundance of references which deal with this problem. The International Committee of Concrete Repair (ICRI) and some committees within the American Concrete Institute (ACI) as well as other organizations throughout the world are devoted to developing methods for repair and to disseminate information to professionals regarding the repair of concrete.

Two excellent resources that cover this topic in great depth are:

1. ACI 546R-96, *Concrete Repair Guide*
2. *Concrete Repair Manual*, published jointly by ICRI and the ACI. This is a compilation of various repair documents currently available in North America, and will soon be expanded to include documents from Europe

Concrete repair, strengthening and renovation is an immense subject. The intent of this paper is to present an overview of the topic. This will include a discussion of how to approach a concrete repair program, as well as introducing some of the commonly used repair techniques and materials.

1.3.1 Typical Concrete Problems:

- Poor Quality Concrete
- Corrosion-related
- deterioration Carbonation
- Freeze-thaw damage
- Earthquake damage
- Design-related
- Substandard "Halo of Anodic" Ring effect

Poor quality concrete: The quality of concrete in a structure will impact the long-term performance. Good durable properly-consolidated concrete, placed with the minimum of honeycombing and internal shrinkage, will provide an environment that should protect the embedded reinforcing for years before repairs are required, if ever.

Corrosion-related deterioration: Corrosion of embedded reinforcing steel is the most common cause of concrete deterioration. When the iron in steel is exposed to water, oxygen, and chlorides, it oxidizes and produces corrosion (rust). The oxidized metal can expand up to 10 times its original volume, resulting in intense bursting forces in the surrounding concrete. This will eventually lead to cracking and delamination.

Carbonation: In normal concrete, the reinforcing is protected by the naturally high alkalinity of the concrete with a pH of about 12. A passivating layer of stable mineral scale is formed on the reinforcing which protects it from corrosion. Carbonation is the reduction of the protective alkalinity of the concrete. It is caused by the absorption of carbon dioxide and moisture which lowers the pH to 10 or less and renders the reinforcing vulnerable to corrosion. Reinforcing steel embedded in carbonated concrete will corrode in the presence of water and oxygen.

Freeze-thaw damage: Freeze-thaw damage is more likely to occur in poor quality concrete, especially if it is not air-entrained. This is a problem in the colder climates with a wide variation of temperature on a daily basis.

Earthquake damage: This is a problem here in Mexico City as well as at various other locations throughout the world. There are methods to modify and/or strengthen existing structures to meet current earthquake standards. Earthquake issues are not addressed by ACI 546 but are instead the responsibility of ACI Committees 341 and 369.

Design-related problems: Improper design or detailing can occasionally result in damage or deterioration to that structure. The lack of proper expansion joints in large concrete tanks, for example, will often result in significant cracking.

Substandard workmanship: Misplaced reinforcing, for example, is a common problem in concrete structures. This often results in severe cracking, and eventually leads to delamination because of corrosion.

Environmentally-related problems: Structures located along seacoasts, or in northern climates where deicing salts are used, for example, often have serious problems with corrosion of the underlying reinforcing steel because of its contact with chlorides.

“Halo of Anodic” Ring effect: It is common for the same reinforcing bars to extend from a repaired area to an adjacent un-repaired, contaminated concrete. Because the same bar extends into two distinctly different environments, conditions result in an electrochemical process, which fosters corrosion where the new repair and parent concrete meet (bond line). The build-up of rust at the surface of the reinforcing, usually in the original concrete, results in spalling, typically around the perimeter of repair patches.

1.3.2 Repair Methodology:

There must be a basic understanding of the underlying causes for the concrete deficiencies prior to selecting repair techniques or materials. Quite often it is not possible to eliminate the reasons that caused the deterioration. An example of this is inadequate concrete cover over reinforcing bars on the façade of a building. The best option is to use repair techniques which take into account the inadequate cover and to provide an appropriate protection system.

1.3.3 Concrete Repair Program:-

The basic steps for a repair program include:

Current condition evaluation
Selection of repair methods and repair materials
Preparation of Repair
Documents Bidding/Negotiation
process Execution of work

1.3.3.1 Current condition evaluation:

The objective of the condition evaluation is to find the cause for the deterioration or distress and to determine the extent of corrective work that is required. The tasks for a condition evaluation ideally/should include:

- Review of all available drawings, reports, maintenance records or any other pertinent documents.
-
- Perform structural analysis for the deteriorated condition if warranted, to determine if there are any safety issues.

- Visually inspect the entire structure. As much of it as possible should be sounded to determine the extent of deterioration. Record all results.

- Implement appropriate testing program. The purpose of the testing program is to evaluate the extent of corrosion activity, the properties of the concrete and reinforcing, the condition of concrete and to collect any other data that might be useful to determine the cause for the deterioration and to help establish an appropriate repair program.

Carefully evaluate all laboratory results and other findings from the testing program.

Analyze findings to determine the cause for the distress.
Prepare an evaluation report including findings, conclusions, and recommended repair program along with an opinion of cost to implement the repairs.

The concrete testing program should be designed to yield the necessary information to properly assess the structure. The testing program may include:

- Determination of the locations of delaminating by sounding with a hammer or chain drag or any other device. Note that good concrete, when tapped, has a pinging sound whereas delaminated has a dull sound.
- Determination of chloride levels of the concrete. Reinforcing steel in concrete with high chloride levels is more likely to corrode.
- Determination of corrosion activity within the concrete. This is accomplished by such methods as copper-copper sulfate half cell.
- Taking cores from existing concrete to perform compression tests to determine its strength. This can be supplemented by Windsor Probe or non-destructive tests, such as a Swiss Hammer.
- Determination of internal tensile strength of concrete or at bond line of repairs by performing bond or pull-out tests.
- Determination of physical properties of concrete, such as internal cracking, freeze-thaw damage, estimate of air void system and other pertinent data, with a Petrographic Analysis (microscopic evaluation).
    - Determination of levels of carbonation.
    - Testing to determine if there is any internal cracking or voids with impact- echo tests.
    - Testing with radar to locate reinforcing steel.
    - Testing with pachometer to locate reinforcing steel.

1.3.3.2 <u>Selection of repair methods and repair materials:</u>

A.Considerations prior to implementing repair program
Sometimes it makes more sense to repair the ongoing problems than to eliminate the cause. This might be the case for a parking garage scheduled to be replaced in a few years. Spot repairs would be appropriate, as opposed to a more aggressive repair program. The objective would be to keep the structure safe and operational.
When selecting repair methods and materials, outside constraints must be considered such as:

- Limited access to work areas
- Operating schedule (when owner will allow work to take place)
- Budget limitations
- Required useful life of structure (The repair program should be consistent with objective of owner. For example, the minimum repairs should be done if the structure is to be demolished in a few years.)
- Weather implications
- There may be constraints imposed by governmental agencies which have to be considered. This could include regulations regarding:
    - Airborne vapor and/or particles
    - Noise
    - Hazardous waste
    - Other governmental restrictions

Structural safety issues must be considered prior to implementing repairs, as well as throughout the duration of the repair program. If, for example, during an initial evaluation there is a concern regarding the structural integrity of a parking deck, it would be prudent to provide temporary shoring immediately.

Before selecting repair materials, consult with manufacturers to get a sense of what materials are available, the methods of installation, cost effectiveness and technical feasibility to use certain products. Be aware that manufacturers or vendors may not inform you of the limitation of their materials. It is important that the specifier carefully researches the repair materials that might be appropriate for the project before making a final selection.

The following factors should be considered when selecting a repair material:

- Coefficient of thermal expansion
- Shrinkage Permeability.
- Modulus of elasticity
- Chemical properties (pH close to12 is desirable)
- Electrical properties
- Color and texture properties

The following references are now available regarding repair materials:
ICRI Guideline No. 03733 *Guide for Selecting and Specifying*

- *Materials for Repair of Concrete Surfaces*.
- ACI 546R-96 *Concrete Repair Guide*, Chapter 3.

1.3.4 Concrete Protection

Concrete protection systems may be incorporated into a repair program to extend the life of repairs and minimize future deterioration. In some cases, they will improve the appearance of the repaired structures.

1)Surface applied Protection systems include:

- Penetrating sealers -- materials which, after application, are generally within the substrate of the concrete. Such products include boiled linseed oil, silanes, siloxanes and high molecular weight methacrylates.
- Surface sealers -- products of 10 mils (0.25 mm) or less in thickness that generally lay on the surface of the concrete. Such products include varieties of epoxies, polyurethanes, methyl methacrylates, moisture-cured urethanes and acrylic resins.
- High-build coatings -- materials with a dry thickness greater than 10 mils (0.25 mm) and less than 30 mils (0.75 mm) applied to the surface of the concrete. Such products include acrylics, styrene-butadienes, polyvinyl acetates, chlorinated rubbers, urethanes, polyesters, and epoxies.
- Membranes -- systems with a thickness of greater than 30 mils (0.7mm) and less than 250 mils (6 mm). Such products include urethanes, acrylics, epoxies, neoprenes, cement, polymer concrete, and asphaltic products.
- Overlays -- products over 250 mils (6 mm) in thickness that are, in general, bonded to the surface of the concrete. Such products include concrete, polymer concrete, and polymer- modified concrete.

The selection factors when comparing the various systems and products include:

- Track record Cost .
- Appearance.
- VOC (volatile organic compounds) compliance with governmental regulatory agencies.
- Compatibility with substrate.
- Durability and performance.

1)Cathodic Protection
The corrosion process is an electrochemical process where anodic and cathodic areas are formed on the steel. When the anodic and cathodic areas are electrically continuous and in the same electrolyte, corrosion at the anodic areas will occur. The corrosion is created as an electrical current flow occurs through the corrosion cell, anodes cathode and electrolyte. For reinforcing bars embedded in concrete, unless mitigated, the corrosion will continue, ultimately resulting in cracking, delamination and spalling of the concrete adjacent to the reinforcing. (From ACI 546R-96, Section 4.3).

An effective method to control the corrosion of steel in contaminated concrete is cathodic protection. The basic principal is to make the embedded reinforcing steel cathodic, thereby preventing further corrosion of steel. This can be accomplished by electrically connecting the reinforcing steel to another metal that becomes the anode, with or without the application of an external power supply.

Cathodic protection systems without an external power source are referred to as sacrificial systems. The metal used to protect the steel is "less noble" or more prone to corrosion than the steel.
Zinc is commonly used for this purpose.

Cathodic protection systems using an external power source are referred to as impressed current system. This method incorporates an external power supply to force a small amount of external current through the reinforcing steel. The purpose of this current is to counteract the flow of current caused by the corrosion process. A metal that corrodes at a very slow rate, such as platinum, is typically provided to serve as an anode.

1.3.5 Surface Preparation

Probably the most important task to achieve successful concrete repairs is the surface preparation. Good surface preparation requires that minimal damage be done to the remaining concrete at the bond line. Any loose concrete as a result of micro-cracking must be removed. This can be achieved by abrasive blasting or high pressure water jetting. The surface also needs to be clean, free of contaminants, and roughened to an appropriate amplitude for the selected material.
The edges of concrete patches should have shoulders to avoid feathering. This is usually accomplished by saw cutting. Repair patches should be made as regular as possible and re-entrant corners should be avoided. See attached Drawing RS1-1 reproduced from ICRI Surface Preparation Guideline 03730.

It is difficult to determine the extent of concrete removal until the work is implemented. The actual quantity of required concrete repairs may vary from the work as shown and specified in the repair documents. Initially the extent of the area to be repaired is sounded with a hammer, chain, or other device to determine the approximate area of removal. All delaminated, unsound, or otherwise unsuitable concrete must be removed The concrete removal must extend around the existing reinforcing with approximately ¾ Inch (19mm) clearance. At mats of reinforcing, if the lower layer is not corroded and tight, the removal need not extend below the lower layer. If the lower layer of reinforcing is also corroded, the removal must extend below the lower layer. Refer to Drawings RS1-1, RS1-2A and RS1-2B, which are based on ICRI Surface Preparation Guideline 03730.

Impact methods for concrete removal, such as jackhammers, can result in micro-cracking of the substrate. The lightest hammers possible should be used to minimize this impact. Care must be taken to remove all loose concrete prior to installing any overlays or repair materials.

For larger projects, hydrodemolition might be an option. This method removes concrete with high pressure water jets which is efficient and leaves a rough profile. After the surface is thoroughly cleaned, it usually is very satisfactory for bonding new concrete. Although, micro-cracking is minimized, containment and subsequent disposal of the water can be a problem.

Note: A typical "Concrete Repair Procedure" is attached to this paper as an example of the steps required to attain proper concrete repairs. It is intended to be used only as a guide and must be modified appropriately for each project. The author cannot assume any liability for its use on any project and does not warrant its accuracy.

References for methods of removal and preparation include:

- ACI 546R-96, *Concrete Repair Guide*, Chapter 2
- ICRI Guideline No. 03730, *Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion*

1.3.6 Repair Concept:-

The method of repair should be designed and detailed to mitigate the damage or deterioration as much as possible. Sometimes it is necessary to rebuild portions of the structure as part of the repair program. There are times when strengthening is required, in addition to repairs. Each project must be carefully evaluated, as previously discussed, prior to selecting a repair method appropriate for that job.
Deterioration adjacent to concrete repair patches commonly occur because of the “Halo of Anodic” Ring effect. When the same bar extends into two distinctly different environments, conditions result in an electrochemical process which may result in corrosion where the new and parent concrete meet (bond line). The build-up of rust at the surface of the reinforcing, usually in the original concrete, leads to spalling, typically around the perimeter of repair patches. This problem must be addressed in repair programs to delay the onset of future corrosion and the need for repairs. The methods to mitigate this problem include:

- Protective coatings
- Migrating corrosion
- inhibitors
- Cathodic Protection-impressed current
  Cathodic Protection-passive

The most common repair methods include:-

1)Cast-in-place concrete — Repair by conventional concrete is generally the replacement of defective concrete with new concrete.

2) Form and pour — Formed and poured concrete is a method of replacing damaged or deteriorated concrete by placing a repair mortar in a formed cavity.

3) Troweling — This method is used for shallow and/or limited areas of repair. These repairs are generally made with Portland cement mortars, proprietary cementitious materials or polymer-modified grouts or polymer grouts and mortars.

4) Dry packing — Dry packing is the hand placement of a very dry Portland cement mortar, which is tamped or rammed into place.

5) Preplaced aggregate — In this method the aggregate is placed in the forms first and the voids are filled by pumping in a cementitious or resinous grout. A benefit of this method is a significant reduction of dry shrinkage.

6)Shotcrete — This method involves pneumatically conveying concrete or mortar through a hose at a high velocity onto a surface.

7) Injection grouting — This is a common method for filling cracks or open joints. The materials used for this method can be either cementitious or chemical.

8) External Reinforcing — This method incorporates steel elements, Fiber Reinforced Polymers (FRP), Post-tensioning cables or other materials placed outside or on the surface of structural members to strengthen them.

1.3.7 Repair Documents

On many, if not most repair projects, it is difficult to establish the exact scope of work. The documents must be structured to allow for a fair and competitive bid process and to protect the owner for negotiated projects. This can be accomplished by providing the contractors bidding the projects with specified quantities.
This requires that the specifier perform a reasonable evaluation of the existing conditions to establish the scope of work. Based on the findings, the specifier must establish quantities for each type of repair or task to be performed by the contractor. The contractors will be asked to provide unit prices for each type of repair so that adjustments can be made for the actual quantities of work performed.
The repair details provided on the documents should be for the known conditions that need repair. It is usually necessary to modify details during construction for actual field conditions, which may vary from the assumed conditions. Re- engineering during construction is not uncommon.
When deterioration is particularly severe or when extensive concrete removal is anticipated, the project documents should caution the contractor that temporary structural support should be anticipated. Special attention should be given to the structural components under repair as well as to adjacent framing. Redistribution of loading during the work should be anticipated and considered during the preparation of the construction drawings. Contingency provisions should be included in the drawings and specifications for addressing potential increases in the scope of work.
The parameters for concrete removal should be defined and, if possible, boundaries of concrete removal and replacement shown on the documents. The specifications and drawings need to establish the criteria for preparation and provide adequate information to provide a standard for acceptance.

1.3.7.1-Bidding/Negotiation process

The selection of a qualified concrete repair contractor is an important aspect of the repair process. Not all repair contractors are proficient in all phases of repair work. If possible, select contractors who have demonstrated competency on projects with work similar to the project being bid.

A Pre-Bid conference should be held with all of the bidders, the engineer and the owner present. This will provide an opportunity for the bidders to ask questions and to increase their awareness of the project objectives and the scope and nature of work. It is best that the meeting be held at the repair site.

Prior to the start of work, a Pre-Construction Meeting should be held which would include the owner, engineer, contractor and their project manager, superintendent, and material suppliers. The contractor should present their schedule for the project at this meeting. The frequency of meeting, field reports, submittals and other items pertaining to the delivery of the project should be discussed at this meeting.

1.3.7.2-Execution of work

The repair work should be executed in accordance with the project documents. Typically the documents will be based on specified quantities. It is thus important that the actual quantity of each type of work be carefully determined and documented. The owner should be informed immediately of any overruns in the specified quantities.

The repair process, especially concrete removal and reinforcing repair, may alter the load distribution of the structure and of the members being repaired. Proper shoring and bracing needs to be provided throughout of the construction.

All of the required tasks, including concrete removal and surface preparation, need to be done in accordance with the specifications and drawings and to good industry practice. The installation of materials should be done in strict compliance with the manufacturer's instructions and the specifications.

Quality control throughout the repair process is essential to any successful project. Appropriate inspections by the engineer and/or testing/inspection agency and contractor needs to be performed on a regular basis. This should be supplemented with field testing as deemed appropriate by the engineer and testing agency.

1.4.Summary and Conclusion

Concrete repair projects are very challenging, as is true with most repair and renovation projects. It is imperative that the engineer understands the reasons which led to the damage and/or deterioration prior to developing a repair program. The underlying causes should be corrected, although this is not always possible. As a minimum, all unsafe conditions must be corrected, and if necessary, temporary shoring or bracing provided, as soon as they are identified.

The owner must be included when formulating a repair program, especially determining the project objectives. Because budget constraints often control the approach to a repair program, it is important that the owner has a clear understanding of what is being done. Furthermore, the owner should be apprised of the anticipated life of the repairs and the long-term costs to maintain that structure, after the repairs are implemented.

Periodic maintenance of structures is essential.
Each and every problem should be properly analysed and then the appropriate repair methods undertaken
Primary design of the building reflects its performance in long run.
Each repair technique is suitable only for the particular application for which it is meant for.
Form and Pump technique which has become the alternative for grouting, gunneting nowadays is also cost effective in large scale operations
Cost should not be significant planning factor in rehabilitation though it is a deciding factor.

Chapter (2)

Repair and Strengthening

Materials and Techniques

2.1- Repair and Strengthening Materials

2.1.1- Introduction

This Chapter contains descriptions of the Various categories of materials that are available for repair and strengthening of concrete structures . Typical properties , advantages , disadvantages or limitations and typical applications will be discussed for each material.

2.1.2- Cementitious Materials

In order to match the properties of the concrete being repaired and strengthened as closely as possible , Portland cement concrete and mortar or other cementitious compositions are frequently the best choices for repair materials.

2.1.2.1-Conventional Concrete

Conventional concrete is composed of Portland cement, aggregates, and water. Admixtures are frequently used to entrain air, accelerate or retard hydration, improve workability, reduce mixing water requirements, increase strength, or alter other properties of the concrete. Pozzolanic materials, such as fly ash or silica fume, may be used in conjunction with Portland cement for economy, or to provide specific properties such as reduced early heat of hydration, improved Later-age strength development, or increased resistance to alkali-aggregate reaction and sulfate attack.

Concrete proportion must be selected to provide workability, density, strength, and durability necessary for the particular application. To minimize shrinkage cracking, the repair and strengthening concrete should have a water-cement ratio as low as possible and a coarse aggregate content as high as possible.

Conventional concrete is readily available, well understood, economical, and relatively easy to produce, place, finish, and cure. Generally, concrete mixtures can be proportioned to match the properties of the underlying concrete; therefore conventional concrete is applicable to a wide range of repairs.

Conventional concrete without admixtures should not be used in repairs and strengthening where the aggressive environment that caused the original concrete to deteriorate has not been eliminated unless a reduced service life is acceptable. When used as a bonded overlay, the shrinkage properties of the repair and strengthening material are critical since the new material is being placed on a material that has exhibited essentially all of the shrinkage that it will experience. Full consideration of the shrinkage properties and the curing procedure should be addressed in thespecification for the repair strengthening procedure.

Conventional concrete is often used in repair and strengthening involving relatively thick sections and large volumes of repair and strengthening material. Typically, conventional concrete is appropriate for partial-and full-depth repairs and resurfacingoverlays where the minimum thickness is greater than about 100 mm on walls piers, and hydraulic structuresconventional concrete is particularly suitable for repair and strengthening in marine environments because the typically high humidity in such environments minimizes the potential for shrinkage.[1]

2.1.2.2–ConventionalMortar

Conventional mortar is a mixture of Portland cement, fine aggregate, and water. Water - reducing admixtures, expansive agents, and other modifiers are often used with conventional mortar to minimize shrinkage.

The advantages of conventional mortar are similar to those of conventional concrete. In addition, mortar can be placed in thinner sections. A wide variety of prepackaged mortars is available. They are particularly appropriate for small repair and strengthening.

Mortars generally exhibitincreased drying shrinkage compared to concrete because of their higher water volume, higher unit cement content, and higher paste- aggregate ratio.

Conventional mortar can be used in the same situations as conventional concrete wherever thin repair sections are required.

2.1.2.3-Dry Pack Mortar

Dry pack mortar may consist of one part cement, two and one- half to three parts sand, or prepackaged proprietary materials, and only enough water so the mortar will stick together when molded into a ball by slight pressure of the hands and will not exude water but will leave the hands damp. Curing is critical because of the low initial water content of dry pack mortar.

Because of its low water- cement ratio, dry pack exhibits very little shrinkage. Therefore, the patch remains tight and is of good quality with respect to durability, strength, and water tightness. If the patch must match the color of the surrounding concrete, a blend of gray and white Portland cement may be used. Normally, about one-third white cement is adequate, but the precise proportions can only be determined by trial.

Dry pack is not well suited for patching shallow depressions or for patching areas requiring filling behind exposed reinforcement, or for patching holes extending entirely through concrete sections. Without adequate curing, dry pack repairs are subject to failure.

Dry pack can be used for filling large or small cavities form tie holes, or any cavity that allows for adequate compaction. Such repairs can be accomplished on vertical and overhead surfaces without forms. Dry pack can also be used for filling narrow slots cut for the repair of dormant cracks, however, it is not recommended for filling or repairing active cracks.

2.1.2.4-Ferrocement

Ferrocement is a term used to describe a form of reinforced concrete that differs from conventional reinforced or prestressed concrete primarily by the manner in which the reinforcing elements are dispersed and arranged. Ferrocement is commonly constructed of hydraulic cement mortar reinforced with closely spaced layers of continuous and relatively small diameter wire mesh. The mesh may be made of steel or other suitable materials. [2]

Ferrocement has a very high tensile strength – to – weight ratio and superior cracking behavior in comparison to reinforced concrete.

Since no formwork is required, ferrocement is especially suitable for repair and strengthening of structures with curved surfaces such as shells, and free- form shapes.

The use of ferrocement in a repair situation will simply be limited by the nature of the repair and strengthening.

2.1.2.5- Fiber – reinforced Concrete

Fiber- reinforced concrete is conventional concrete with either metallic for polymeric fibers added to achieve greater resistance to plastic shrinkage and service- related cracking. In most applications, fiber reinforcing is not intended as primary reinforcement. Fiber reinforced concrete has been used for repair and strengthening using conventional and shotcrete placement methods. [3]

The fibers are added during concrete production and are in the concrete when it is placed. These fibers can be used to provide reinforcing in thin overlays that are not thick enough to include reinforcing bars.

Fiber reinforced concrete has been used for overlays of concrete pavements, slope stabilization, and reinforcement of structures such as arches or domes. Reinforced concrete structures have been repaired with fiber- reinforced shotcrete. Areas subject to shock or vibration loading, where plastic shrinkage cracking is a problem, or where blast resistance is required, could benefit from the addition of fiber reinforcement.

The addition of fibers reduces the slump and can cause workability problems for inexperienced workers. Rust stains may occur at the surface of steel fiber- reinforced concrete due to corrosion of fibers at the surface. In patching applications, the electrical conductivity of the patch material, when using metallic fibers, could influence corrosionactivity when patches are installed around previously damaged reinforcement. For other applications, a wicking effect suggests that permeability may be higher than for conventional concrete systems of equivalent thickness. Curing and protection of fiber- reinforced concrete should be similar to that for equivalent conventionally reinforced concrete.

2.1.2.6- Grouts

The grouts described herein are categorized as either hydraulic cement or chemical.

a)Cement Grouts

Cement grouts are mixtures of hydraulic cement, aggregate, and admixtures that when mixed with water produces a trowellabl, flowable, or pumpable consistency without segregation of the constituents. Admixtures are frequently included in the grout to accelerate, or retard time of setting, minimize shrinkage, improvepumpability or workability, or to improve the durability of the grout. Mineral fillers may be used for reasons of economy when substantial quantities of grout are required.

Cement grouts are economical, readily available, easy to install, and compatible with concrete. Admixtures can be used to modify cement grouts to meet specific job requirements at relatively low cost. Admixtures to minimize shrinkage are available on the market.

Cement grouts may be used for repairs by injection only where the width of the opening is sufficient to accept the solid particles suspended in the grout. Normally, the minimum crack width at the point of introduction should be about 3 mm.

Typical applications of hydraulic cement grout may vary from grout slurries for bonding old concrete to new concrete to filling of large dormant cracks or to filling of voids around or under a concrete structure. No shrink cement grouts may be used to repair spalledor honeycombed concrete or to install anchor bolts in hardened concrete.

b) Chemical Grouts

Chemical grouts consist of solutions of chemicals that react to form either a gel or a solid precipitate as opposed to cement grouts that consist of suspensions of solid particles in a fluid. The reaction in the solution may involve only the constituents of the solution, or it may include the interaction of the solution with other substances, such as water, encountered in the use of the grout. The reaction causes a decrease in fluidity and a tendency to solidify and fill voids in the material into which the grout has been injected.

The advantages of chemical grouts include their applicability in moist environments, their wide ranges of gel setting time, and their low viscosities. Cracks in concrete as narrow as 0.05 mm have been filled with chemical grout. Rigid chemical grouts, such as epoxies, exhibit excellent bond to clean, dry substrates, and some will bond to wet concrete. These grouts can restore the full strength of a cracked concrete member. Gel- type or foam chemical grouts, such as acryl amides and polyurethanes, are particularly suited for use in control of water flow through cracks and joints. Some gel grouts can be formulated at viscosities near that of water so they can be injected into almost any opening that water will flow through.

Chemical grout are more expensive than cement grout. Also, a high degree of skill is needed for satisfactory use of chemical grouts. Chemical bonding agents, such as epoxies, have relatively short pot life and working times at high ambient temperatures. Gel grouts should not be used torestore strength to a structural member. Most gel or foam grouts are water solutions and will exhibitshrinkage if allowed to dry in service.

Repair of fine cracks, either to prevent moisture migration along the crack or to restore the integrity of a structural member, is one of the most frequent applications of chemical grout. Some grouts, such as epoxies, are frequently used as bonding agents.

2.1.2.7- Low Slump Dense Concrete

Low slump dense concrete (LSDC) is a special form of conventional concrete. It generally has a moderate to high cement factor, a water- cement ratio less than 0.40, and exhibits working slumps of 50 mm or less. LSDC generally gains strength rapidly and is distinctive because of its high density and reduced permeability.

Overlays of LSCD with a minimum thickness of only 38 mm have provided up to 20 years of service when properly installed. The cost of LSDC is relatively low, and it can be placed using conventional equipment with slight modifications. Compared to structural grade concrete, LSDC provides reduced chloride permeability.

LSDC`s require maximum consolidation effort to achieve optimum density, or the use of a high- range water-reducing admixture (HRWRA) to improve workability of the concrete and reduce the compaction effort needed to provide bond to the reinforcing steel and to the underlyingconcrete. These low water- cement ratio concretes generally require at least7 days of continuous moist curing obtaining adequate hydration. LSCD permits galvanic corrosion even with a 0.32 water cement ratio and 25 mm cover. Drying shrinkage cracks, depending on crack width and depth, can increase chloride ion intrusion resulting in corrosion of the reinforcing steel in bridge deck overlays.

LSDC is frequently used as an overlay or final wearing course in a composite repair and strengthening to obtain a high (acceptable) quality, abrasion resistant, and durable concrete surface.

2.1.2.8- Magnesium Phosphate Concretes and Mortars

Magnesium phosphate concretes and mortars (MPC) are based on a hydraulic cement system that is different from Portland cement. Unlike Portland and some modified Portland cement concretes which require moist curing for optimum property development, these systems produce their best properties upon air curing- similar to epoxy concretes. Rapid strength developmentand heat are produced although retarded versions are available that produce less heat.

Setting times of 10 to 20minutes are typically encountered at room temperatures, and early strength development of 14 MP a within 2 hours is regularly obtained. Retarded versions with extended setting times of 45 to 60 minutes at room temperature are also available. Salt- scale resistance is similar to Portland cement based concrete materials. When extended with aggregates, abrasion resistance of MPC is similar to

PCC.Neat magnesium phosphate cement will naturally have lower abrasion resistance, similar to Portland cement mortars.

Patching applications are the most common use of MPC. It is frequently cost effective for rapid repairs where a short down time is important. The common uses are in highway, bridge deck, airport, tunnel, and industrial repairs.Repairs in a cold- weather environment are important applications. Due to the exothermic nature of the reaction, heating of the materials and the substrates is not usually necessaryunless the temperature is below freezing. MPC is useful for cold weather embedments and anchoring because of its high bond strength and low shrinkage rate.

2.1.2.9- Preplaced- aggregate Concrete

Preplaced- aggregate concrete is produced by placing coarse aggregate in a form and later injecting a Portland cement- stand grout (usually with admixtures), or a resinous material to fill the voids. Preplaced- aggregate concrete differs from conventional concrete in that it contains a higher percentage of coarse aggregate.

Because of the point – to – point contact of the coarse aggregate, drying shrinkage of preplaced- aggregate concrete is about one- half that of conventional concrete. Because the aggregate is preplaced and the grout pumped under pressure, segregation is not a problem and virtually all substrate voids will be filled with mortar.

Typically, preplaced- aggregate concrete is used on large repair projects, particularly where underwater concrete placement is required or when conventional placing of concrete would be difficult. Typical applications have included underwater repair of stilling basins, dams, bridge, abutments and footings.Preplaced- aggregate concrete has also been used to repair and strengthening beams and columns in industrial plants, water tanks and other similar facilities, as well as caissons for underpinning existing structures.

2.1.2.10- Rapid - Setting Cement

Rapid- setting cementitious materials are characterized by short setting times. Some may exhibit very rapid strength development with compressive strengths in excess of 6.9 MP a within 3 hours. Type III Portland cement with accelerators has been used for the patching of concrete for a long time and has been morewidely used than most other materials in full depth sections. [7]

Rapid- settings cements provide accelerated strength development that which allows the repair to be placed into service more quickly than conventional repair materials. This advantage is of importance in repair of highways and bridges because of the reduced protection times, lower traffic control costs, and improved safety.

Rabid- settings cements are especially useful in repair and strengthening situations where an early return to traffic is required, such as repair of pavements, bridge decks, and airport runways.

2.1.2.11-Shotcrete

Shotcrete is a mixture of Portland cement, sand, and water “ shot’’ into place by compressed air. In addition to these materials, shotcrete can also contain coarse aggregate, fibers, and admixtures. Properly applied shotcrete is a structurally adequate and durable repair material which is capable of excellent bond with existing concrete or other construction materials.

The successful application of shotcrete is dependent on the training, skill, and experience of the nozzleman. The nozzlemanshould be required to demonstrate his skill by placing a test panel that reflects the site conditions. His performance should be evaluated and approved before he is allowed on the job. Dust and rebound require special attention in indoor application.

2.1.2.12- Shrinkage-compensating Concrete

Shrinkage-compensating concrete is an expansive- cement concrete which is used to minimize cracking caused by drying shrinkage the basic materials and methods are similar to produce high – quality Portland cement concrete. consequently, the characteristics of shrinkage-compensating concrete are, in most respects, similar to those of Portland – cement concrete.

When properly restrainedby reinforcement, shrinkage – compensating concrete will expand an amount equale to or slightly greater than the anticipated drying shrinkage. Subsequent drying shrinkage will reduce these expansive strains but, a residual expansion will remain in the concrete, thereby eliminating shrinkage cracking.

Shrinkage- compensating concrete has been used to minimize cracking caused by drying shrinkage in replacement concrete slabs, pavements, bridge decks, and structures. Also, shrinkage- compensating concrete has been used to reduse warping tendencies where concrete is exposed to single face drying and carbonation shrinkage.

2.1.2.13- Silica- Fume Concrete

Silica fume, a by-product in the manufacture of silicon and ferrosilicon alloys, is an efficient pozzolanic material. Adding silica fume and a high range water- reducing admixture to a concrete mixture will significantly increase compressive strength, decrease permeability, and thus improve durability. Silica fume is added to concrete in either liquid or powder form in quantities of 5 to 15 percentby weight or cement. Compressive strengths of 33 to 103 MP a can be attained with silica-fume concrete. [9]

The first major applications of silica – fume concrete in the United States were for repair and strengthening of hydraulic structures subjected to abrasion- erosion damage (Holland and Gutschow, 1987). The high strength of silica- fume concrete and the resulting abrasion- erosion resistance appear to offer an economical solution to abrasion- erosion problems, particularly in those areas where locally available aggregate otherwise might not make acceptable concrete. Silica-fume concrete has been used extensively in overlays on parking structures and bridge decks to reduce the intrusion of chloride ions into the concrete.

2.1.2.14- Bonding Materials

Bonding materials can be used to bond new repair materials to an existing prepared concrete substrate. Bonding materials are of three types: epoxy based, latex based, and cement based.

a)**Epoxy:**Care should be taken when using these materials in hot weather. High temperatures may cause premature curing and the creation of a bond break. Most epoxy resin bonding materials create amoisture barrier between the existing substrate and the repair material.

b)**Latex:**Latex bonding agents are classified as Type I – Redispersible and Type II – Non – redispersible Type I bonding agents can be applied to the bonding surface several days prior to placing the repair materials; however, the bond strength is less than that provided by Type II bonding agents. Type I bonding agents should not be used in areas subject to water, high humidity, or structural applications. Type II systems act as bond breakers once they have skinned over or cured.

c) **Cement:** Cement based systems have been used for many years. Cement bonding systems use neat Portland cement or a blend of Portland cement and fine aggregate filler generally proportioned one to one by weight. Water is added to provide a uniformly creamy consistency.

2.1.3- Polymer Materials

The improvement of properties of hardened concrete by the addition of polymers is well documented. This guide presents information on various types of polymer materials and on their storage, handling, and use, as well as on concrete formulations, equipment to be used, construction procedures, and applications. [11]

Three basic types of concrete materials use polymers to form composites, each type of material will be discussed in the following sections.

2.1.3.1- Polymer-impregnated Concrete

PIC is a hydrated Portland-cement concrete that has been impregnated with a monomer that is subsequently polymerized. Impregnation is usually done using monomers which contain a polymerization initiator that can be activated by heat. The most widely used monomer is methyl methacrylate, although other monomers have been used. With polymer loadings of 1.5 to 2.5 percent by weight, and depths of impregnation of at least 6 mm and up to 38 mm, significant improvements in durability can be achieved. It is important to achieve a complete shell of impregnated concrete at the exposed surfaces.

Almost all existing types of concrete, whether they were cast with impregnation in mind or not, can be impregnated provided the proper procedures are followed. The impregnation of concrete surfaces with a suitable polymer has been shown to improve several important properties, including abrasion resistance; resistance to penetration by, and damage from water, acids, salts, and other deleterious media; and resistance to cycles of freezing and thawing.

Polymer impregnation reduces the permeability of concrete and thereby increases its durability in exposure to aggressive agents. However, impregnation does not render the concrete completely impermeable, and since concrete is still exposed, aggressive agents, such as sulfuric acid, will attack the concrete slowly. Cracks that are not sealed could serve as channels for ingress of aggressive agents into the concrete thereby defeating the purpose of the surface impregnation treatment. Cracks are also likely to occur during drying of the concrete prior to application of the monomer, and all of these cracks may not be filled during the impregnation process. All cracks must be filled with the polymer to achieve a reduction in permeability and corresponding increase in durability.

Polymer impregnation has been applied to existing concrete structures to improve durability, reduce maintenance requirements, and restore deteriorated concrete. The process has been used in a variety of applications including bridge decks, spillways, stilling basins, curbstones, concrete pipes and mortar-lined steel pipes, and deteriorated buildings.

2.1.3.2- Polymer-modified Concrete:

Polymer-modified concrete (PMC) has at times been called polymer-portland-cement concrete (PPCC) and latex-modified concrete (LMC). It is identified as Portland cement and aggregate combined at the time of mixing with organic polymers that are dispersed or redispersed in water. This dispersion is called a latex, and the organic polymer is a substance composed of thousands of simple molecules combined into large molecules. The simple molecules are known as monomers and the reaction that combines them is called polymerization. The polymer may be a homopolymer if it is made by the polymerization of one monomer or a copolymer if two or more monomers are polymerized.

Polymer dispersions are added to the concrete to improve the properties of the final product. These properties include improved bond strength to concrete substrates, increased flexibility and impact resistance, improved resistance to penetration by water and by dissolved salts, and improved resistance to frost action.

Of the wide variety of polymers investigated for use in PMC, polymers made by emulsion polymerization have been the most widely used and accepted. Styrene butadiene and acrylic latexes have been the most effective and predictable for concrete restoration. Other latexes commonly used include polymers and copolymers of ving1 acetate. [12]

When emulsified and mixed with concrete, epoxies provide excellent freeze-thaw resistance, significantly reduced permeability, and improved chemical resistance. Bond is excellent and flexural, compressive, and tensile strengths are high. However, epoxy emulsions have had limited use in concrete.

Latex-modified concrete (LMC) overlays have exhibited excellent long-term performance. Properly installed overlays are highly resistant to freeze-thaw damage, and they exhibit minimal bond failure after many years of service. LMC overlays installed on severely deteriorated bridge decks, after proper surface preparation, continue to perform many years after installation.

Mixing and handling of PMC is similar to conventional Portland cement concrete and mortar. Curing, however, is different. Whereas conventional concrete requires extended periods of moist curing, PMC generally requires one day to two days of moist curing followed by air curing. The PMC is placed in service when it has developed sufficient strength, which is dependent upon the hydration of the cement.

An advantage of latex-modified concrete is its good workability and ease of application when compared to similar systems. The bonding characteristics of latex-modified concrete are excellent and latex-modified concrete usually exhibits low permeability. Styrene-butadiene LMC has excellent durability for exterior exposures or environment where moisture is present. Surface discoloration will occur when the concrete is exposed to UV light. Where such discoloration is not acceptable, acrylic polymers should be used.

Like conventional concrete, latex modified concrete should be placed and cured at 7 to 30 °C with special precautions taken when either extreme is reached. It is recommended that mobile, continuous mixers, fitted with an additional storage tank for the latex, be used for large applications of LMC.

Like many mixtures with a low water-cementitious materials ratio, LMC has a tendency for plastic shrinkage cracking during field placement. Special precautions are necessary when the evaporation rate exceeds 0.5 $kg/m^2/hr$. latex modified concrete, similar to other Portland cement based materials, is susceptible to shrinkage cracking, which may allow ingress of chloride ions in some applications. The modulus of elasticity is generally lower compared to conventional concrete; therefore its use in vertical or axially loaded members should be carefully evaluated. [13-14] polyviny1 acetate should not be used in applications that may be exposed to moisture. Epoxy emulsions are more expensive than most latexes, and some are susceptible to color change and deterioration from exposure to sunlight.

PMC applications include overlays of bridge decks, parking structures and floors, and patching of any concrete surfaces. Styrene butadiene latex has been commonly used for repair and / or strengthening of bridges, parking decks and floors. Acrylic latexes have been used for floor repair and patching and are particularly suitable in exterior white cement applications where color retention is important.

Latex concrete is most commonly used for overlays. It is normally applied in sections ranging from 19 to 50 mm thick. These systems restore lost sections and provide a new, high-strength wearing surface that is very durable against weathering. Although used as overlay materials, polymer-modified concretes are effective patching materials. Since most patches and repairs in which PMC is used are relatively shallow, mixture proportions similar to those shown in ACI 548.3R should be considered.

2.1.3.3- Polymer Concrete:

PC is a composite material in which the aggregate is bound together in a dense matrix with a polymer binder. The composites do not contain a hydrated cement phase, although Portland cement can be used as an aggregate or filler. The term PC should never suggest a single product, but rather a family of products. Use of the term PC in this section also includes mortar.

PC has been made with a variety of resins and monomers including polyester, epoxy, furan, vinylester, methy1 methacrylate (MMA), and styrene. Polyester resins are attractive because of moderate cost, availability of a great variety of formulations, and moderately good PC properties. Furan resins are low cost, and highly resistant to chemical attack. Epoxy resins are generally higher in cost, but may offer advantages such as adhesion to wet surfaces. Detailed information on the use of epoxy compounds with concrete is available. [15]

The properties of PC are largely dependent upon the properties and the amount of the polymer used, modified somewhat by the effects of the aggregate and the filler materials. Typically, PC mixtures exhibit a) rapid curing, b) high tensile, flexural, and compressive strengths, c) good adhesion to most surfaces, d) good freeze-thaw durability, e) low permeability to water and aggressive solutions, and f) good chemical resistance.

PC can provide a fast-curing, high-strength patching material that is suitable for repair of Portland cement concretestructures. Pc is mixed, placed, and consolidated in a manner that is similar to conventional concrete. With some harsh mixtures, external vibration is required.

A wide variety of prepackaged polymer mortars is available which can be used as mortars or added to selected blends of aggregates. Depending upon the specific use, mortars may contain variable aggregate gradations intended to impart unique surface properties or aesthetic effects to the structure being repaired. Also, polymer mortars are available that are trowellable and specifically intended for overhead or vertical applications.

Epoxy mortars generally shrink less than polyester or acrylic mortars. Shrinkage of polyester and acrylic mortars can be reduced by using an optimum aggregate loading. The aggregate grading and the mixture proportions should be available from the polymer formulator.

Organic solvents may be needed to clean equipment when using polyesters and epoxies. Volatile systems such as MMA evaporate quickly and present no cleaning problems. However, such systems are potentially explosive and require nonsparking and explosion-proof equipment. It should be recognized that rapid curing generally means less time for placing and finishing operations. Working times for these materials are variable and, depending on ambient temperatures, may range from less than 15 minutes to more than one hour. Also, high or low ambient and concrete temperatures may significantly affect polymer cure time or performance.

The coefficients of thermal expansion of polymer materials are variable from one product to another, and are significantly higher than conventional concrete. Shrinkage characteristics of PC's must be closely evaluated so that unnecessary shrinkage cracking is avoided.

The modulus of elasticity of PC may be significantly lower than that of conventional concrete, especially at higher temperatures. Its use in load carrying members must be carefully considered. Only a limited number of polymer systems are appropriate for repair of wet concrete surfaces. In general, the aggregates used in PC should be dry in order to obtain the highest strengths.

High temperatures can adversely affect the physical properties of certain PC's, causing softening. Service temperatures should be evaluated prior to selecting PC systems for such use. Epoxy systems may burn out in fires where temperatures exceed 450 °F (230 °C) and can significantly soften at lower temperatures. Users of PC must consider its lack of fire resistance. Conventional concrete generally will not bond to cured PC, and compatibility of the systems should be considered.

Many PC patching materials are primarily designed for the repair and / or strengthening of highway structures where traffic conditions allow closing of a repair area for only a few hours. However, PC's are not limited to that usage and can be formulated for a wide variety of applications. PC is used in several types of applications; 1) fast-curing, high-strength patching of structures, and 2) thin (5 to 19 mm thick) overlays for floors and bridge decks.

Polymer mortars have been used in a variety of repair and / or strengthening where only thin sections (patches and overlays) are required. Polymers with high elongation and low modulus of elasticity are particularly suited for bridge overlays. PC overlays are especially well suited for use in areas where concrete is subject to chemical attack. [16]

2.1.4- Fiber Reinforcement Polymer Materials

FRP materials consists of a large number of small, continuous, directionalized, non-metallic fibers with advanced characteristics, bundled in a resin matrix. Depending on the type of fiber they are referred to as AFRP (aramid fiber based), CFRP (carbon fiber based) or GFRP (glass fiber based). Typically, the volume fraction of fibers in FRPs equals about 50-70% for strips and about 25-35% for sheets. Hence fibers are the principal stress bearing constituents, while the resin transfers stresses among fibers and protects them.

2.1.4.1- 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. 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.

When using epoxy adhesives there are two different time concepts that need to be taken into consideration. The first is the pot life and the second is the open time. Pot life represents the time one can work with the adhesive after mixing the resin and the hardener before it starts to harden before it starts to harden in the mixture vessel; for an epoxy adhesive, it may vary between a few seconds up to several years. Open time is the time that one can have at his / her disposal after the adhesive has been applied to the adherents and before they are joined together. Another important parameter to consider is the glass transition temperature, Tg. Most synthetic adhesives are based on polymeric materials, and as such they exhibit properties that are characteristic for polymers change from relatively hard, elastic, glass-like to relatively rubbery materials at a certain temperature. This temperature level is defined as glass transition temperature, and is different for different polymers.

Epoxy adhesives have several advantages over other polymers as adhesive agents for civil engineering use: [17]

a) High surface activity and good wetting properties for a variety of substrates.

b) May be formulated to have a long open time.

c) High cured cohesive strength; joint failure may be dictated by adherent strength.

d) May be toughened by the inclusion of dispersed rubbery phase.

e) Lack of by-products from curing reaction minimizes shrinkage and allows the bonding of large areas with only contact pressure.

f) Low shrinkage compared with polyesters, acrylics and viny1 types.

g) Low creep and superior strength retention under sustained load.

h) Can be made thixotropic for application to vertical surfaces.

i) Able to accommodate irregular or thick bond lines.

Typical properties for cold cured epoxy adhesives used in civil engineering applications are given in Table 3.1. for the sake of comparison, the same table provides information for concrete and mild steel too. [18]

Property (at 20 °C)	Cold-curing epoxy adhesive	Concrete	Mild steel
Density (kg/m3)	1100-1700	2350	7800
Young's modulus (GPa)	0.5-20	20-50	205
Shear modulus (GPa)	0.2-8	8-21	80
"Poisson's ratio	0.3-0.4	0.2	0.3
Tensile strength (MPa)	9-30	1-4	200-600
Shear strength (MPa)	10-30	2-5	200-600
Compressive strength (MPa)	55-110	25-150	200-600
Tensile strain at break (%)	0.5-5	0.015	25
Approximate fracture energy (Jm-2)	200-1000	100	105-106
Coefficient of thermal expansion (10-6/°C)	25-100	11-13	10-15
Water absorption: 7days – 25 °C (% w/w)	0.1-3	5	0
Glass transition temperature (°C)	45-80	--	--

Table 2.1: Comparison of typical properties for epoxy adhesives, concrete and steel.

2.1.4.2- 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 fibers 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. [19]

Epoxy resins, polyester and viny1ester 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 viny1esters, and outstanding durability, whereas polyesters and viny1esters are cheaper.

2.1.4.3- Fibres

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 recognized that the physical and mechanical properties can vary a great for a given type of fibre as well of course the different fibre types.

Glass fibres for continuous fibre reinforcement are classified into three types: E-glass fibres, S-glass and alkali resistant AR-glass fibres. E-glass fibres, Which contain high amount of boric acid and aluminate, are disadvantageous in having low alkali resistance. S-glass fibres are stronger and stiffer than E-glass, but still not resistant to alkali. 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.

Aramid fibres were first introduced in 1971, and today are produced by several manufacturers under various brand names. The structure of aramid fibre is anisotropic and gives higher strength and modulus in the fibre longitudinal direction. The diameter of aramid fibre is approximately 12 um. Aramid fibres respond elastically in tension but they exhibit non-linear and ductile behavior under compression; they also exhibit good toughness, damage tolerance and fatigue characteristics.

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 polyacry1onitrile that is carbonised through burning. The diameter of pitch-type fibres measures approximately 9-18 um and that of the PAN-type measures 5-8 um. The structure of this carbon fibre varies according to the 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 3.2. Note that values in this table are only indicative of static strength of unexposed fibers. [20]

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 2.2: Typical properties of fibres

2.2 Techniques of Repairing of Reinforced Concrete Structures :-

2.2.1. Introduction:-

- 3 Basic symptoms of distress in a concrete structure.
- Cracking, Spalling and Disintegration.
- Reasons for their development may be poor materials, poor design, poor construction practice, poor supervision or a combination.
- Repair of cracks usually does not involve strengthening.
- Repair of a structure showing spalling and disintegration, it is usual to find that there have been substantial losses of section and/or pronounced corrosion of the reinforcement.

2.2.2 Repairing Cracks:

- In order to determine whether the cracks are active or dormant, periodic observations are done utilizing various types of telltales.
    - By placing a mark at the end of the crack.
    - A pin or a toothpick is lightly wedged into the crack and it falls out if there is any extension of the defect.
    - A strip of notched tape works similarly :

      Movement is indicated by tearing of the tape.
    - The device using a typical vernier caliper is the most satisfactory of all.

      Both extension and compression are indicated.
    - If more accurate readings are desired, extensometers can be used.

- Where extreme accuracy is required resistance strain gauges can be glued across the crack.

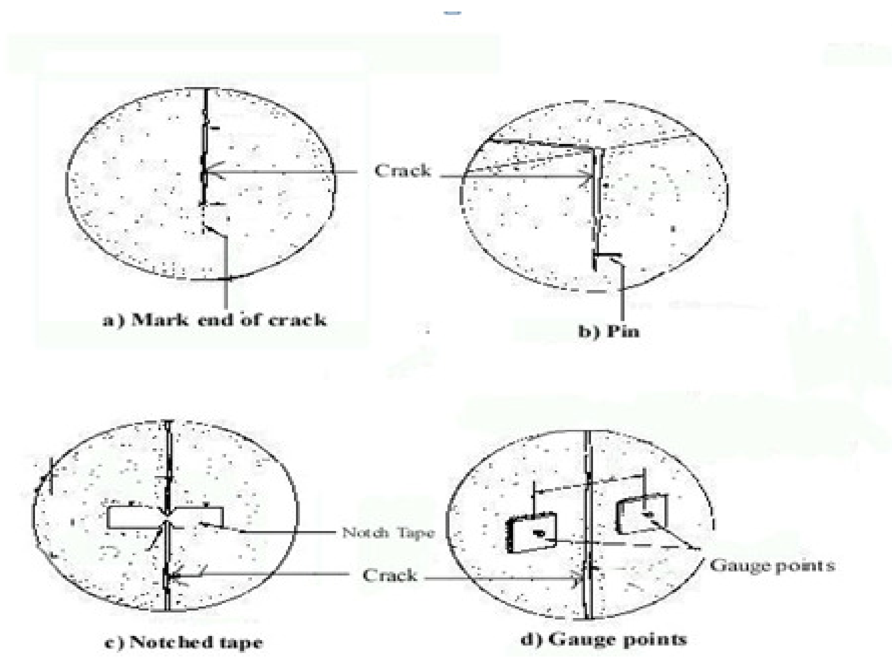


Fig. 2.1 Tell Tales

2.2.3 Types of Cracks:

- Active cracks and dormant cracks.
- The proper differentiation between active and dormant cracks is one of magnitude of movement, and the telltales are a measure of the difference.
- If the magnitude of the movement, measured over a reasonable period of time (say 6 months or 1 year), is sufficient to displace or show significantly on the telltales, we can treat the crack as an active one.
- If the movements are smaller, the crack may be considered as dormant.
- Cracks can also be divided into solitary or isolated cracks and pattern cracks.

- Generally, a solitary crack is due to a positive overstressing of the concrete either due to load or shrinkage.
- Overload cracks are fairly easily identified because they follow the lines demonstrated in laboratory load tests.

- In a long retaining wall or long channel, the regular formation of cracks indicates faults in the design rather than the construction, but an irregular distribution of solitary cracks may indicate poor construction as well as poor design.
- Regular patterns of cracks may occur in the surfacing of concrete and in thin slabs. These are called pattern cracks.

2.2.4.Techniques of Repairing Cracks

2.2.4.1 Bonding with Epoxies:

- Cracks in concrete may be bonded by the injection of epoxy bonding compounds under pressure.
- Usual practice is to:
  - Drill into the crack from the face of the concrete at several locations.
  - Inject water or a solvent to flush out the defect.
  - Allow the surface to dry.
  - Surface-seal the cracks between the injection points.
  - Inject the epoxy until it flows out of the adjacent sections of the crack or begins to bulge out the surface seals.

- Usually the epoxy is injected through holes of about ¾ inch in diameter and ¾ inch deep at 6 to 12 inches centers.
- Smaller spacing is used for finer cracks.
- The limitation of this method is that unless the crack is dormant or the cause of cracking is removed and thereby the crack is made dormant, it will probably recur, possibly somewhere else in the structure.
- Also, this technique is not applicable if the defects are actively leaking to the extent that they cannot be dried out, or where the cracks are numerous.

2.2.4.2 Routing and Sealing:

- This method involves enlarging the crack along its exposed face and filling and sealing it with a suitable material.
- The routing operation.
- Placing the sealant.
- This is a method where thorough water tightness of the joint is not required and where appearance is not important.

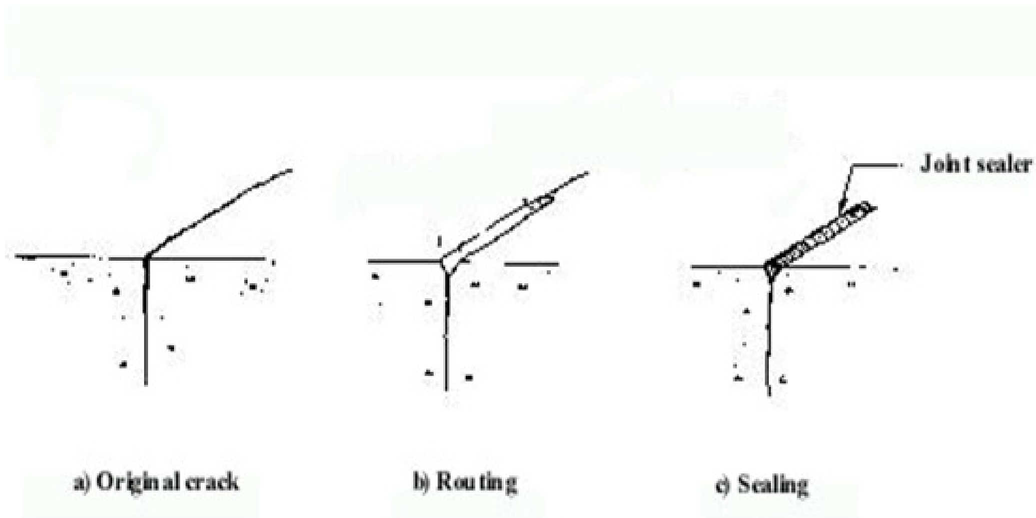


Fig. 2.2 Routing and Sealing

2.2.4.3. Stitching:

- Concrete can be stitched by iron or steel dogs.
- A series of stitches of different lengths should be used.
- Bend bars into the shape of a broad flat bottomed letter U between 1 foot and 3 feet long and with ends about 6 inches long.
- The stitching should be on the side, which is opening up first.

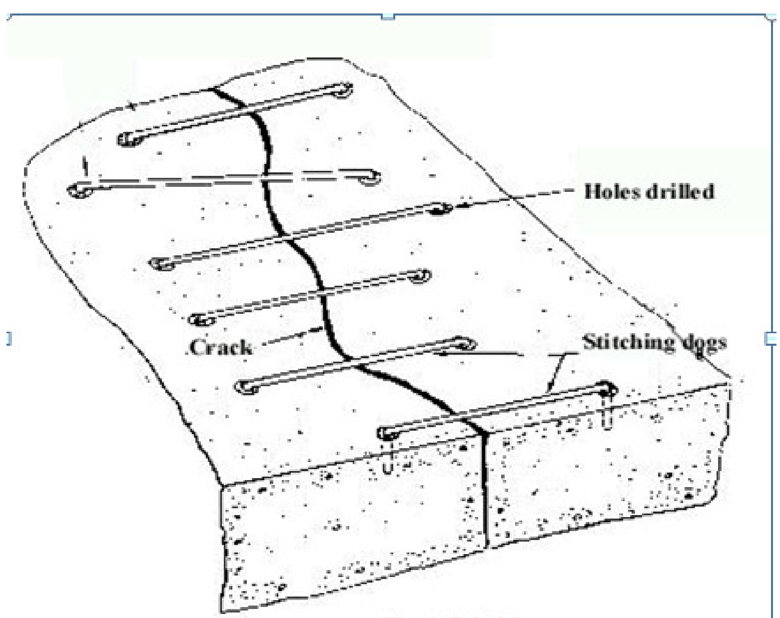


Fig. 2.3 Stitching

- If necessary, strengthen adjacent areas of the construction to take the additional stress.
- The stitching dogs should be of variable length and/or orientation and so located that the tension transmitted across the crack does not devolve on a single plane of the section, but is spread out over an area.
- In order to resist shear along the crack, it is necessary to use diagonal stitching.
- The lengths of dogs are random so that the anchor points do not form a plane of weakness.

2.2.4.4. External Stressing:

- Cracks can be closed by inducing a compressive force, sufficient to overcome the tension and to provide a residual compression.
- The principle is very similar to stitching, except that the stitches are tensioned; rather than plain bar dogs which apply no closing force to the crack.
- Some form of abutment is needed for providing an anchorage for the prestressing wires or rods.

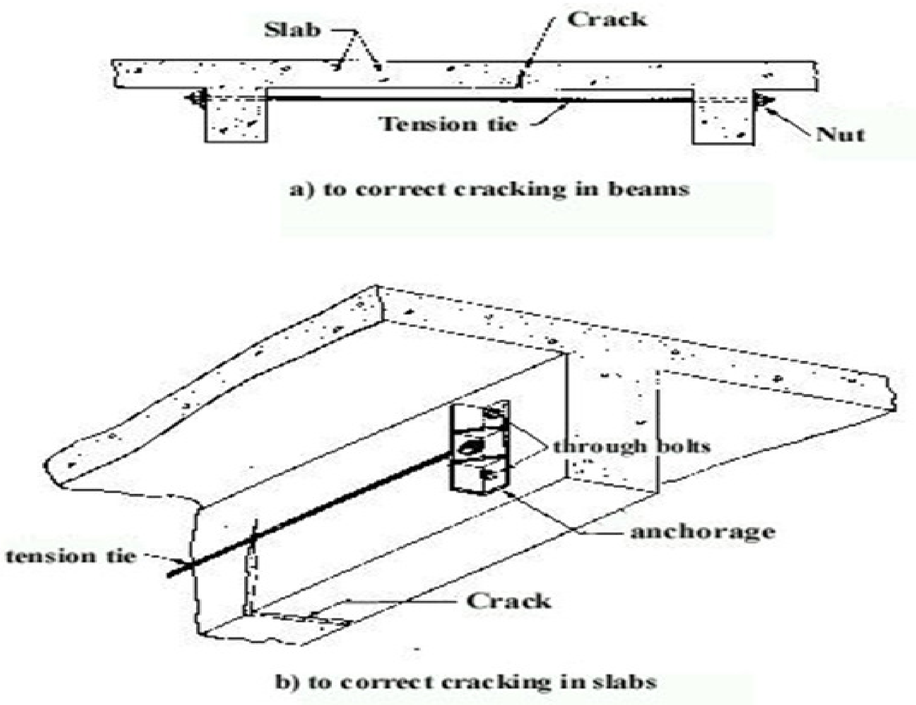


Fig. 2.4 External Stressing

2.2.4.5. Grouting:

- Same manner as the injection of an epoxy.
- Cleaning the concrete along the crack.
- Installing built-up seats at intervals along the crack.
- Sealing the crack between the seats with a cement paint or grout.
- Flushing the crack to clean it and test the seal; and then grouting the whole.

2.2.4.6. Blanketing:

- Similar to routing and sealing.
- Applicable for sealing active as well as dormant cracks.
- Preparing the chase is the first step.
- Usually the chase is cut square.
- The bottom should be chipped as smooth to facilitate breaking the bond between sealant and concrete.
- The sides of the chase should be prepared to provide a good bond with the sealant material.
- The first consideration in the selection of sealant materials is the amount of movement anticipated.
- And the extremes of temperature at which such movements will occur.
- Elastic sealants.
- Mastic sealants.
- Mortar-plugged joints.

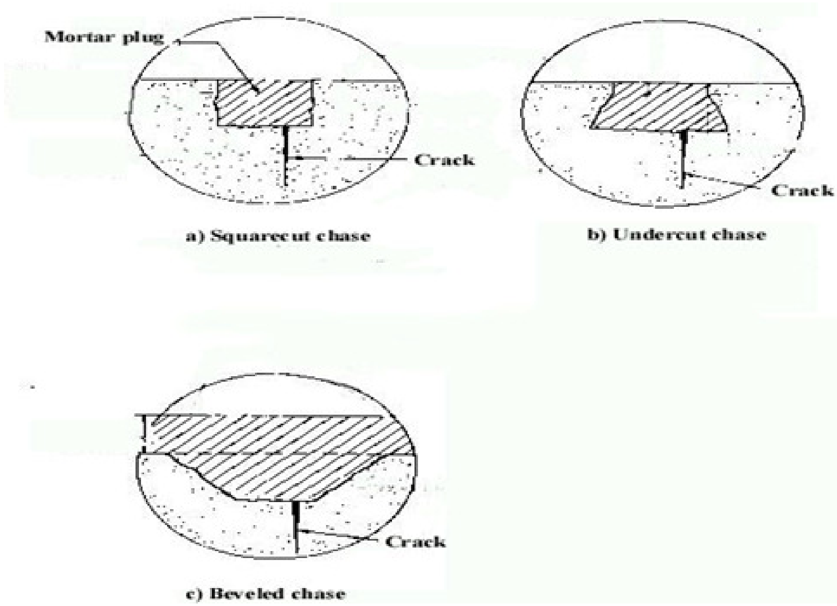


Fig. 2.5 Type of chase

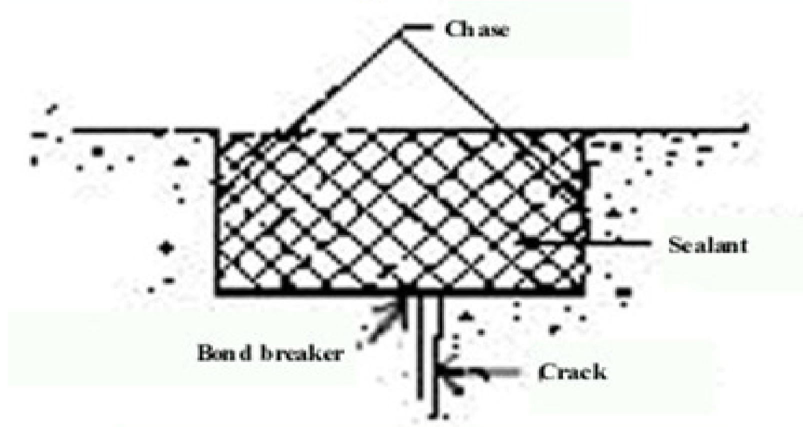


Fig. 2.6 Sealed chase

2.2.4.7 Use of Overlays:

- Sealing of an active crack by use of an overlay requires that the overlay be extensible and not flexible alone.
- Accordingly, an overlay which is flexible but not extensible, ie. Can be bent but cannot be stretched, will not seal a crack that is active.
- Gravel is typically used for roofs.
- Concrete or brick are used where fill is to be placed against the overlay.
- An asphalt block pavement also works well where the area is subjected to heavy traffic.

2.2.5.Repairing Spalling and Disintegration

- In the repair of a structure showing spalling and disintegration, it is usual to find that there have been substantial losses of section and/or pronounced corrosion of the reinforcement.
- Both are matters of concern from a structural viewpoint, and repair generally involves some urgency and some requirement for restoration of lost strength.

2.2.5.1. Jacketing:

- Primarily applicable to the repair of deteriorated columns, piers and piles.
- Jacketing consists of restoring or increasing the section of an existing member, principally a compression member, by encasement in new concrete.
- The form for the jacket should be provided with spacers to assure clearance between it and the existing concrete surface.
- The form may be temporary or permanent and may consist of timber, wrought iron, precast concrete or gauge metal, depending on the purpose and exposure.
- Timber,Wrought iron Gauge metal and other temporary forms can be used under certain conditions.
- Filling up the forms can be done by pumping the grout, by using prepacked concrete, by using a tremie, or, for subaqueous works, by dewatering the form and placing the concrete in the dry.
- The use of a grout having a cement-sand ratio by volume, between 1:2 and 1:3, is recommended.

- The richer grout is preferred for thinner sections and the leaner mixture for heavier sections.
- The forms should be filled to overflowing, the grout allowed to settle for about 20 minutes, and the forms refilled to overflowing.
- The outside of the forms should be vibrated during placing of the grout.

2.2.5.2. Gunning:

- Gunter is also known as shotcrete or pneumatically applied mortar.
- It can be used on vertical and overhead, as well as on horizontal surfaces and is particularly useful for restoring surfaces spalled due to corrosion of reinforcement.
- Gunite is a mixture of Portland cement, sand and water, shot into the place by compressed air.
- Sand and cement are mixed dry in a mixing chamber, and the dry mixture is then transferred by air pressure along a pipe or hose to a nozzle, where it is forcibly projected on to the surface to be coated.
- Water is added to the mixture by passing it through a spray injected at the nozzle.
- The flow of water at the nozzle can be controlled to give a mix of desired stiffness, which will adhere to the surface against which it is projected.

2.2.5.3. Prepacked Concrete:

- This method is particularly useful for carrying out the repair under water and elsewhere where accessibility is a problem.

- Prepacked concrete is made by filling forms with coarse aggregate and then filling the voids of the aggregate by pumping in a sand-cement grout.
- Prepacked concrete is used for refacing of structures, jacketing, filling of cavities in and under structures, and underpinning and enlarging piers, abutments, retaining walls and footings.
- Pumping of mortar should commence at the lowest point and proceed upward.
- Placing of grout should be a smooth, uninterrupted operation.

2.2.5.4. Dry pack:

- Drypacking is the hand placement of a very dry mortar and the subsequent tamping of the mortar into place, producing an intimate contact between the new and existing works.
- Because of the low water-cement ratio of the material, there is little shrinkage, and the patch remains tight. The usual mortar mix is 1:2.5 to 1:3.

2.2.5.5. Replacement of Concrete:

- This method consists of replacing the defective concrete with new concrete of conventional proportions, placed in a conventional manner.
- This method is a satisfactory and economical solution where the repair occurs in depth (at least beyond the reinforcement), and where the area to be repaired is accessible.
- This method is particularly indicated where a water-tight construction is required and where the deterioration extends completely through the original concrete section.
- Overlays.

- In addition to seal cracks, an overlay may also be used to restore a spalled or disintegrated surface.
- Overlays used include mortar, bituminous compounds, and epoxies.
- They should be bonded to the existing concrete surface.

2.3. Summary & Conclusions

- When repairing cracks, do not fill the crack with new concrete or mortar.
- A brittle overlay should not be used to seal an active crack.
- The restraints causing the cracks should be relieved, or otherwise the repair must be capable of accommodating future movements.
- Cracks should not be surface-sealed over corroded reinforcement, without encasing the bars.
- The methods adopted for repairing spalling and disintegration must be capable of restoring the lost strength.

CHAPTER 3

Strengthening of Reinforced Concrete

Structure Elements Using Concrete Jackets

3.1.Introduction

Repairing of reinforced concrete elements is required after a damage. Strengthening such elements is a method to increase the earthquake resistance. Thus, the strength of the structures can be moderately or significantly increased and the ductility can be improved.

Depending on the desired earthquake resistance, the level of the damage, the type of the elements and their connections, members can be repaired and / or strengthened by injection, removal and replacement of damaged parts or jacketing. For repair of concrete structures the advice of engineers is required.

Establishing bonds between old and new concrete is of importance. It can be done by chipping away the concrete cover of the original member and roughening its surface, by preparing the surface with glue (for instance, with epoxy prior to concreting), by additional welding of reinforcement or by formation of reinforced concrete or steel dowels.

Perfect confinement by close, adequate and appropriately shaped stirrups and ties contributes to the improvement of the ductility of the strengthened members. Redistribution of the internal force in structures due to member stiffness changes is very important.

Jacketing with steel profiles (angles and straps) is used for the strengthening of columns. The beam – to – column joint is difficult to be strengthen in this way. Jacketing by steel encasement is implemented by gluing of steel plates on the external surfaces of the original members. Steel plates as reinforcement are glued to the concrete by epoxy resin. This does not require any demolition, it is easy to implement and hardly increase the cross section of the members. Trained technicians are required for this work.

Jacketing by steel profiles or encasement requires special measures for fire and corrosion protection of the new steel profiles. Reinforced concrete jacketing does not need such a protection but the construction procedure is more difficult.

3.2. Strengthening of Reinforced Concrete Columns :-

Increasing flexural and shear strength, improving column ductility and rearrangement of column stiffness can be achieved by repairing techniques.

Column flexural strength increases with the enlargement of the concrete area and by adding reinforcement. Shear strength, and especially ductility, is improved by closely placed reinforcement ties (steal strips).

Damage to reinforced concrete columns, may include slight cracks (horizontal or diagonal) without damage to reinforcement, superficial damage in the concrete without damage to reinforcement, crushing of the concrete, buckling of reinforcement or rupture of ties. Based on the degree of damage, techniques such as injections, removal and replacement or jacketing can be applied.

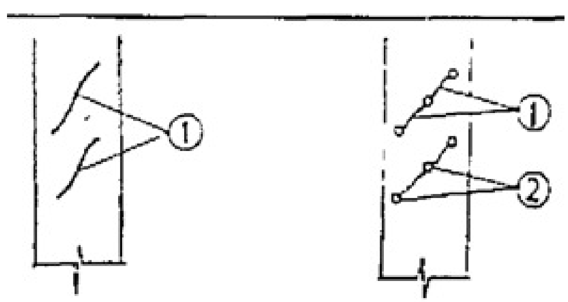
3.2.1.Local Repairs

Resin injections are applied only for damage columns with slight cracks without damaged concrete or reinforcement. Epoxy resin injection is suitable for cracks with width from 0.1 to 5 mm. cement grout injections can be applied for larger cracks (widths from 2 to 5 mm) (Figure 3.1).

Removal and replacement is necessary for heavily damaged columns with crushed concrete, buckled longitudinal bars or ruptured ties.

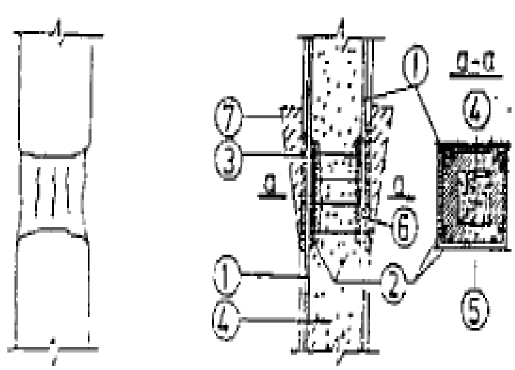
When the concrete is slightly damaged, the loose concrete is removed. The surfaces are roughened and dust is cleaned. Depending on the amount of concrete removed, some additional ties or reinforcement may be added and local jacketing is carried out (Figure 3.2).

When the vertical reinforcement is buckled, the ties are ruptured and the concrete is crushed, total removal and replacement of the damage parts must be carried out (Figure 3.3). If only repair is required, the original cross sections size will be maintained. If strengthening is necessary, the area of the column must be increased. Damaged and loose concrete should be removed, new vertical reinforcement inserted and welded to the existing reinforcement and new ties are placed Non-shrinkage concrete or concrete with low shrinkage properties should be used. Special attention must be paid to achieve a good bond between new and old concrete.



1-cracks 2- injection process

Fig.3.1- Cement Grout Injection



1-existing reinforcement.

2- added new reinforcement

3-added new ties.

4- old concrete.

5- new concrete .

6- welding.

7- mortar

Fig. 3.2 Local Jacketing

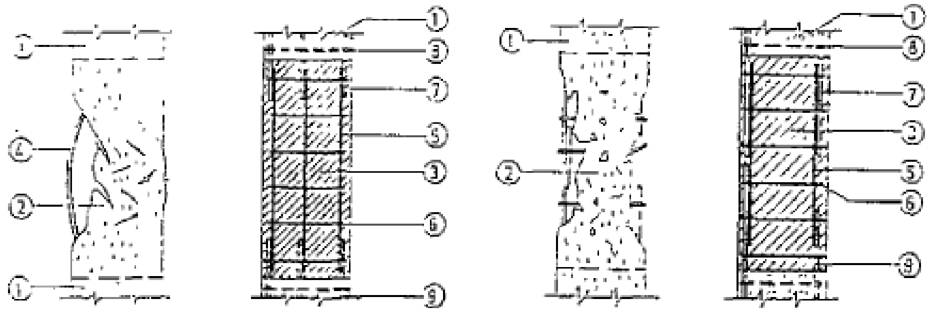


Fig. 3.3 Removal and placement of damaged parts

3.2.2.Reinforced Concrete Jacketing for Column

Jacketing should be applied in cases of heavy damage columns or in cases of insufficient column strength. This is really a strengthening procedure although, but it can also be used for repairing. Jacketing can be performed by adding reinforced concrete, steel profiles or steel encasement.

Reinforced concrete jacketing, depending on the space around the column, can be adding jacketing to one, two, three or four sides of the column (Figure 3.4). It is recommended that columns be jacketed on all four sides. In order to achieve the best bond between new and existing concrete, four-sided jacketing is best. In case one, two or three sided jacketing is all that is possible, the concrete cover in the jacketed parts of the existing column must be chipped away so that new ties can be welded to existing ties. In case of a four-sided jacketing, only roughening of the surface of the existing column may be required.

Jacketing only within one floor does not improve seismic response unless shear walls are also added. Adequate column strength can be achieved by passing the new vertical reinforcement through holes drilled in the slab and adding new concrete in the beam-column joint region (Figure 3.5) special attention should be paid to the good confinement of the vertical reinforcement near the floor beams.

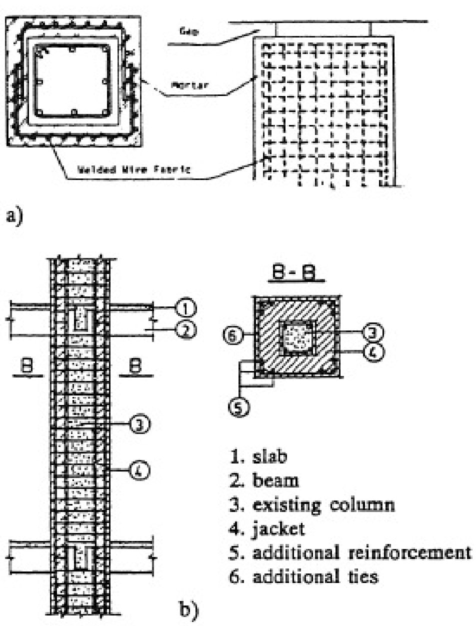
In the case of a one-sided jacket, adequate connection between existing and new concrete is achieved by closely spaced, well anchored, additional transversal reinforcement (Figure 3.6). The following solutions can be applied.

- Anchorage by ties to the existing vertical reinforcement (Figure 3.6(a)).
- Welding of additional ties to the column (Figure 3.6(b)).
- Connection by bent bars welded to the vertical reinforcement (Figure 3.6(c)).

Similar detailing is applied in case of two or three-sided jacketing.

In the usual four-sided jacketing, several solutions are possible (Figure 3.7):

- Jacketing with welded wire fabric and new concrete cover.
- Jacketing with connecting bent bars (Figure 3.7(a)).
- Jacketing with ties (Figure 3.7(b)).



1-Slab 2- beam. 3- old columns . 4- jackets.

5- added longitudinal reinforcement.

Fig. 3.4 column jackets

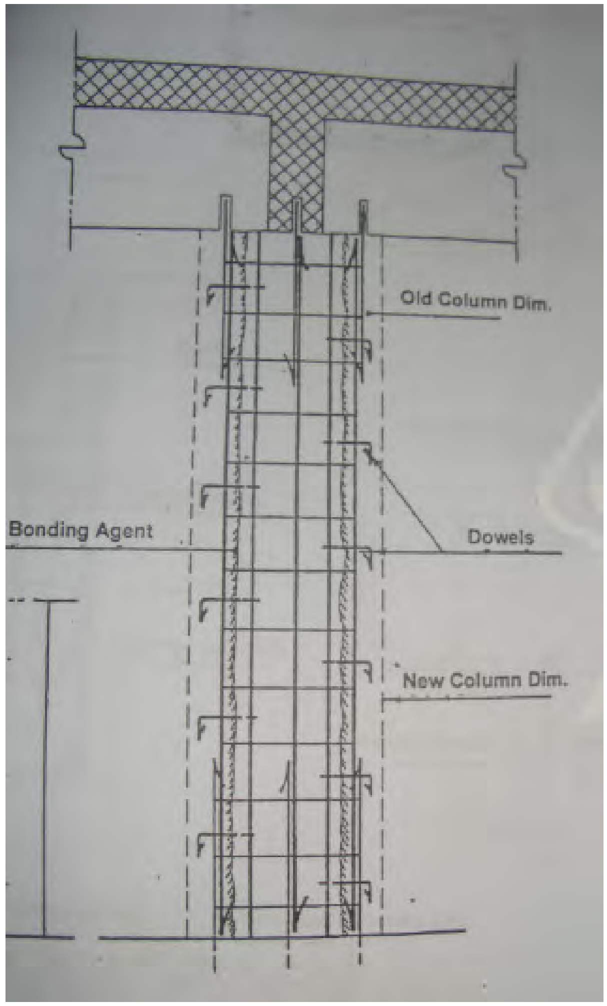
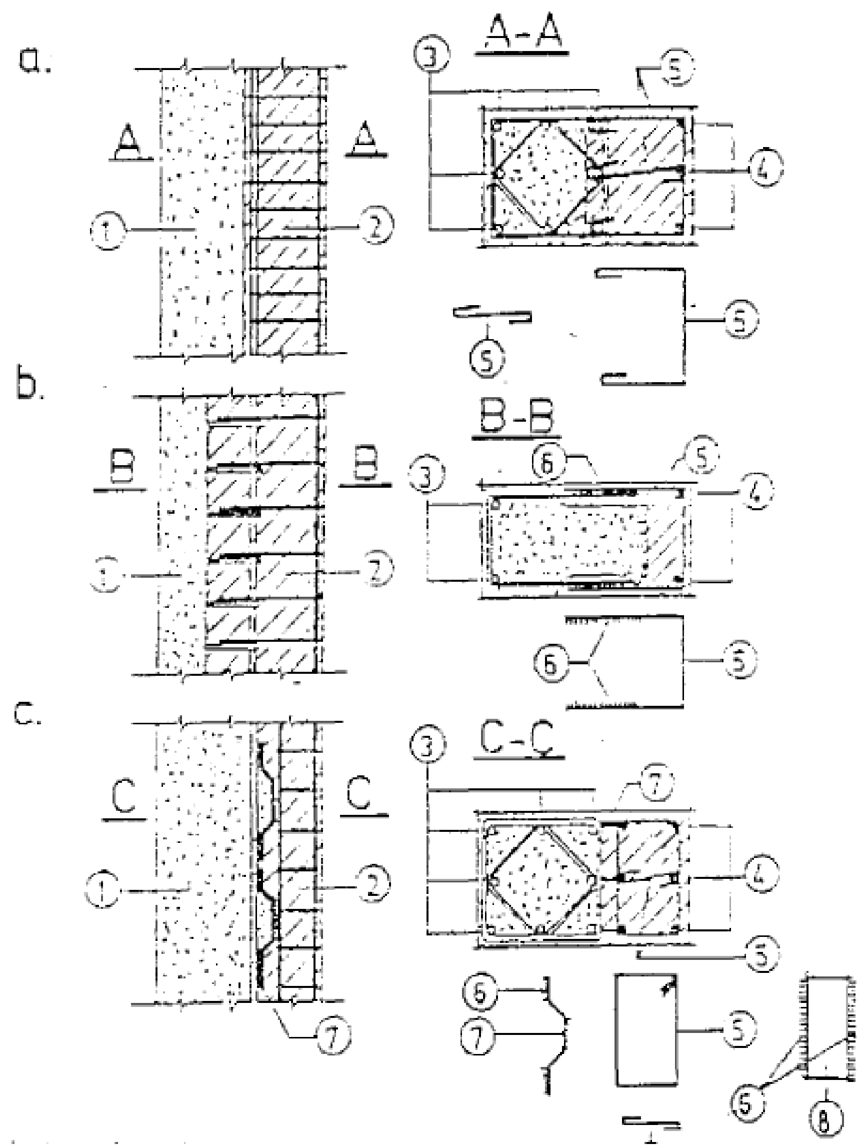


Fig. 3.5 Passing New Vertical Reinforcement for Adequate Columns Strength

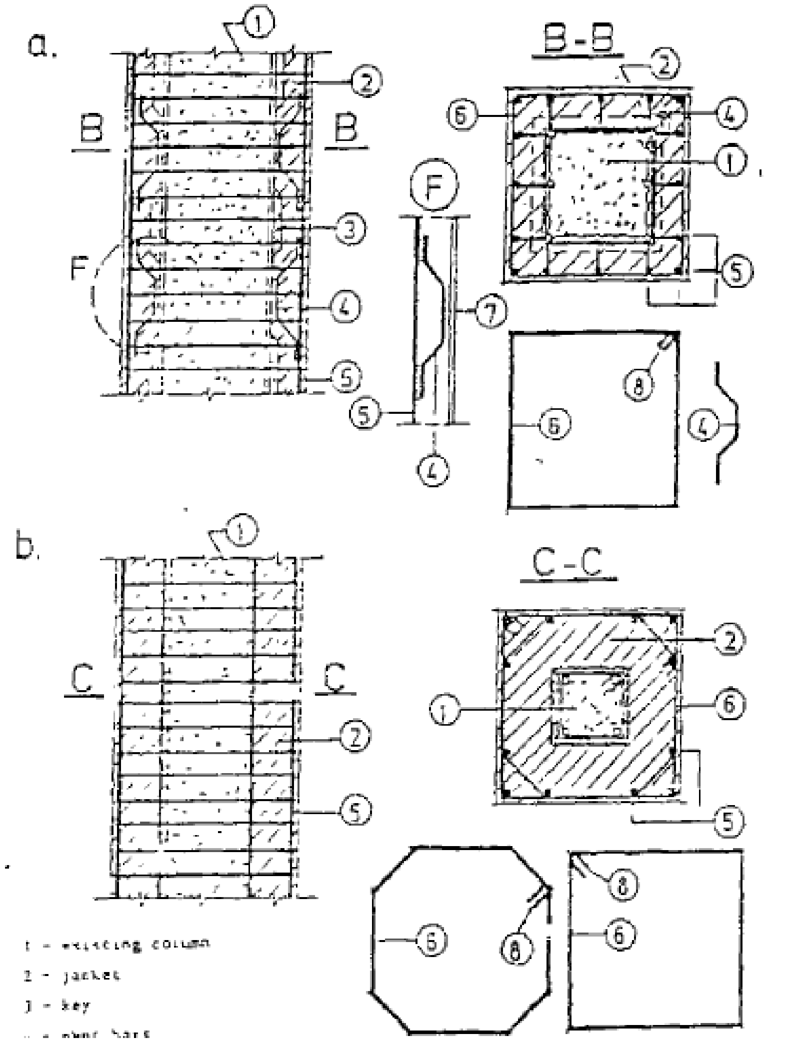


1-existing column 2- jacket 3- existing reinforcement

4- added longitudinal reinforcement . 5 – added ties

6- welding. 7- bent bars

Fig. 3.6 Additional Transverse Reinforcement in case of One- sided Jacket



1-existing column 2- jacket 3-Key

4- bent bars . 5 – added reinforcement

6- ties. 7- welding

8- alternative corners .

Fig. 3.7 Four – Sided Jacketing

3.2.3Reinforced Concrete Jacketing Conditions

- The strength of the new materials must be greater than those of the column.
- The thickness of the jacket should be at least 4 cm for shotcrete application or 10 cm for cast-in-situ concrete.
- The reinforcement should not be less than four bars for four-sided jacketing and bar diameter should be at least 14 mm.
- Every corner and alternate longitudinal bar should have lateral ties with an angle of bend of 135 degrees minimum. No intermediate bar should be father than 10 cm from corner of the ties. In some cases, it will be necessary to drill into the core of the column to place epoxy hooked ties into the hole or drill completely through the existing column core to install new ties.
- The ties should be minimum 8 mm, and at least 1/3 of the vertical bar diameter.
- Vertical spacing of ties is at most 20 cm. close to the joints the spacing should not exceed 10 cm, within a length of 1/4 the clear height. In addition, the spacing of ties should not exceed the thickness of the jacket.

Jackets can be made either with conventional or special cast-in-situ concrete or by shotcrete (gunite). For both methods, the existing concrete must be thoroughly roughened by chipping or heavy sandblasting and cleaned of all loose material, dust and grease. The surface should be thoroughly moistened before placing the concrete or shotcrete.

3.3: Strengthening of Reinforced Concrete Beams:-

The strengthening of beams is to provide strength and stiffness of beams to resist gravity and seismic loads the chosen procedure must provide strength and stiffness to the beams in relation to the columns. So that to avoid creating structures of the "strong girder – weak column type", which tend to force seismic hinging and distress into the column.

Depending on the type of damage (cracking crushing of concrete, rupture of reinforcement or ties). The techniques for repairing and strengthening beams are quite similar to those for columns.

3.3.1 Local Repairs

Injection is applied for repair of damaged beams with slight cracks only. Epoxy or cement grout injections are made.

Removal and replacement should be applied when heavy damage like crushing of concrete, deterioration of bond or rupture of reinforcement occurs. Before the removal of crushed concrete or rupture reinforcement, the damaged beam must be temporally supported. The replacement procedure for beams is similar to that of columns. More attention must be paid to compact the new concrete under existing beams or slabs, which is extremely difficult if placement access is not provided from the top surface of the beam.

3.3.2Reinforced Concrete Jacketing of Beams

Reinforced concrete jacketing is done by adding concrete on one. Three or four sides of a beam. To create bond between old and new concrete and for welding of the added reinforcement to the existing, the concrete cover must be chipped away. An irregular shaped concrete surface. Combined with anchoring welded stirrups. Provide good shear and tensile connection of the jacket to the beam.

Reliable anchorage of vertical bars in joint areas by sufficient length or by welding to anchors, is of great importance shear and ductility improvement can be provided by stirrups on all sides of the beam. The legs of the stirrup should penetrate and anchored into the slab at the top of the jacket.

One-sided jackets (Figure 3.8) adding strength only to the beam soffit, should be used only when it is necessary to increase the flexural strength of a beam. The connection between existing and new longitudinal reinforcement is achieved by connection bars (See Figure 3.8). The concrete cover should be away up to the reinforcement and higher at existing stirrups. Additional welded to the existing ones provide the connection between the existing and the newly-added concrete. The longitudinal reinforcement bars should anchored in the support region, by welding the reinforcement to a collar of angle profile, at the top of the column.

Sided jacketing (Figure 3.9) or beam encasement adds flexural and shear because of the increase of reinforcement area and concrete cross section. Additional longitudinal reinforcement should be connected with the existing reinforcement by diagonally welded bent bars or small steel plates. The stirrups through holes drdled in the slab and surround the whole beam. These holes also be used to place the concrete in the part of the jacket beneath the slab. Litional reinforcement for negative bending moments must be added over the surface into the beam region and outside of the existing column. Specialntion must be paid to the anchorage of the longitudinal bars in the joint on of the column jacket.

Heting three sides of the beam can also be installed beneath the soffit of the (figure 3.10) Shotcrete is the most feasible method of installing this type jacket. Its weakness is the anchorage of the new stirrups at the top of the. This detail is inferior to that shown in Figure 3.11 as the effectiveness he detail depends on the dynamic strength of the power driven nails and the finess of the strand to provide effective enchorage for the new stirrups. Reased strength be achieved by using a continuous steel plate with epoxy in bolts installed in the concrete, with the new stirrups welded to or hooked the steel plate.

3.3.3Reinforced Concrete Jacketing Should Conform to the Following:

The strength of the new materials should be greater than that of the existing beam.

The thickness of the jacket should not be less than 4 cm for shotcreteapplication or 8 cm for cast-in-situ concrete.

Top and bottomreinforcement should be anchored within column joint area with full development length, beginning from the face of the column, or continuous through the joint region.

At support regions, the stirrup spacing must not be more than 1/4 of the beam height. Outside this region, the stirrup spacing can be doubled.

The beams must not be made too stuff or strong with respect to the columns.

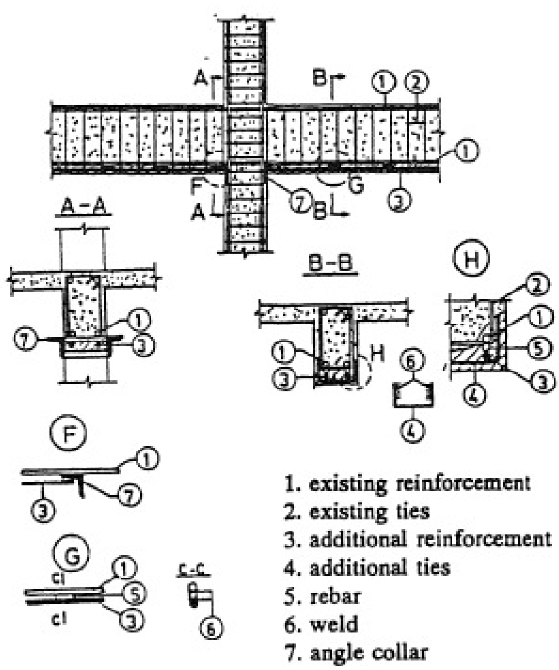


Fig. 3.8 one – Sided Jacketing for beam

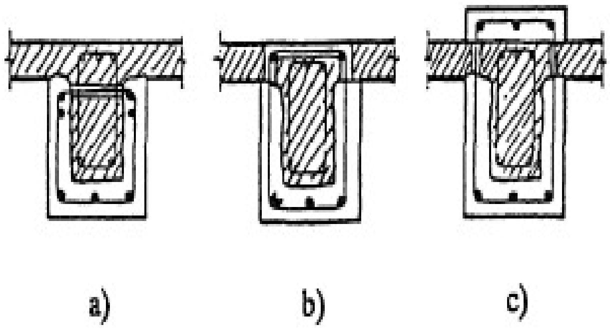


Fig. 3.9 Three - Four - Sided Jacketing (beam)

3.3.4 Repairing Gravity Load Capacity of Beams

Steel rods can be used to improve the shear resistance of damaged or undamaged beams. It can be performed by vertical or diagonal external clamps (Figures 3.10 a/b). The clamps consist of round rods with threads at the end tightened with nuts. Vertical clamps are fixed on angle profiles, but diagonal clamps are welded to longitudinal reinforcement to resist the longitudinal component of force, load reversal is anticipated, four-sided jacketing is the preferred method for strengthening.

Steel plate reinforcement is a new technique which can be used for beams subject primarily to static loading to improve their shear strength or midspanflexurestrength. The steel plates are attacked to concrete surface of the reinforce concrete members by gluing with epoxy resin. During the epoxy hardening, the steel plates must be clamped to the concrete member. It is recommended the steel plates also be anchored by either nails shot into the concrete or anchor bolts (wedged or epoxied). It is advisable for the beam to be smoothed withthin layer of expansive cement mortar for plates with thickness more than 3 mm. in this case; wedge anchor bolts must be applied. Special attention must be path to corrosion and fire protection, especially considering the total loss of epoxy res-strength at temperatures higher than 250 °C. This procedure is not recommended for beams subject to cyclic loading due to earthquake forces.

3.4. Strengthening of Reinforced Concrete Shear Walls

The walls, becauseof their great stiffness and lateral strength, provide the most earthquake resistance of the building. Therefore, a severely damaged or a poorlyigned shear wall must be repaired or strengthened in order that the structure's ngth for seismic force can be improved.

3.4.1Concrete Wall Repairs

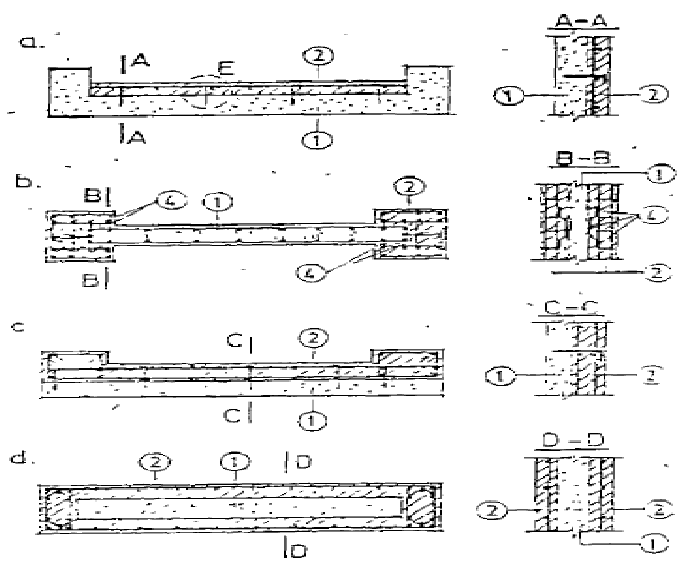
Ection with epoxy resin for crack repairs in shear has become a standard ctice over the last decade. If neither bond deterioration nor concrete crushing the occurred, epoxy injections are capable of restoring approximately the original wall strength. However, the repaired wall will never achieve the stiffness of the ginal wall. Not all of the small cracks can be epoxy injected. Another advantage is the rapid loss of strength of epoxy under fire. Epoxy repair of walls is simple, fast and economical, without changing the original wall size and hout evacuation of the inhabitants. Higher strength in walls than the original ength cannot be attained by epoxy injections. Therefore, if additional strength required, another technique must be used.

Removal and replacement should be applied for large cracks, partially crushed concrete and buckled reinforcement. After removing the loose concrete and toughening and cleaning the remaining surface, additional reinforcement or molded wire fabric should be placed. The choice of repair material (polymer mortar, cement mortar, concrete or shotcrete) depends on the degree of damage, the desired repair characteristics and the site conditions. Non-shrinkage or expansive cement for mortar and concrete is often desirable. Special care must repaid to adequately compact the new concrete, especially at contact regions with the existing concrete.

3.4.2.Increase of Wall Size

Thickening the wall with reinforced concrete should be applied when the original strength of a damaged or undamaged wall is insufficient. There are different ways and add strength to an existing concrete shear wall (3.10). Shotcrete is a sequently used technique in strengthening concrete shear walls.

When shear strength is to be increased, thickening the web with additional reinforced concrete is necessary (Figure 3.10). Web reinforcement (horizontal and vertical bars) is added. The reinforcement and the concrete must be anchored to the existing wall by roughening of the surface and by special anchor bolts. An appropriate solution is the application of epoxied bars with 90° hooks. Anchorage of the added web reinforcement can also be achieved by epoxying dowels in holes drilled into the wall.



1-existing wall 2- added wall 3-added column

4- welding . 5 – epoxies bar

Fig. 3.10 Adding Strength to existing Concrete Shear Wall

If an increase in flexural strength of a shear wall is required, reinforced flanges must be added to both ends of the wall (Figure 3.10). The new flange concrete must be confined by appropriately detailed, closely spaced hoops and cross ties. The anchorage of the flange concrete to the original wall is very important. It can be performed by welding of special bent bar connectors to the new and the old reinforcement or by epoxied anchor bars.

Shear forces between the shear wall and the floor slab must be transmitted. For this purpose, concrete dowel connections are made by opening holes through the slab. These holes also serve to concrete the new wall parts beneath the slab. Diagonal reinforcement bars passing through the slab and anchored in the upper story and lower story walls, provide an additional connection for shear transfer.

To provide bond between the old and the new concrete and for welding connection bars to the reinforcement, some concrete cover must be chipped away the original wall surface should be roughened, any paint or plaster removed additional shear is transmitted from the existing to the new wall by epoxied dowels in the existing surface.

Reinforced concrete thickening of shear walls should slao conform to the following provisions:

- The strength of the new materials should be greater than those of the existing walls.
- The web thickness of the new material should be at least 5 cm. the thickness of the new shear wall flanges should be at least 10 cm.
- Both the horizontal web reinforcement and the vertical web reinforcement must not be less than 0.0025 times the gross area of the wall thickening.
- The area of the vertical reinforcement concentrated at the wall ends must not be less than 0.0025 times the gross area of the newly added cross section.

- The end wall ties should not be less than 8 mm in diameter. The tie spacing should be no more than the thickness of the wall end thickening or miximum 15 cm.

- The new concrete should be anchored to the existing concrete with epoxied hooked dowels at a maximum of 60 cm in each direction and the existing wall surface should be thoroughly roughened.

3.5. Strengthening of Slabs

Slabs have to carry vertical gravity loads. However, they must also provide shragm action and be compatible with all lateral resistant elements of the cture. Therefore, slabs must possess strength and stiffness. Damages in slabs crally occur in locations with irregularities such as near large openings, at centration of earthquake forces in slabs close to widely spaced shear walls and staircase landings. Repair of slabs is necessary when damage occurs. Mgthening is applied when there is insufficient slab strength or for increased mgth in the region of newly-introduced shear walls.

3.5.1Repairs

Action should be applied for repair of cracks. Epoxy or cement grout can be add. Restoration of connection between the separated concrete parts can be ieved.

Removal and replacement procedures should be applied in cases of spalledconcrete and broken or buckled reinforcement. Floor slabs or staircase slabs can be repaired in this manner (Figure 3.11). After the removal of removal of the unsound reterials new reinforcement is incorporated and it is welded to the existing reinforcement. Concrete with better properties than the existing concrete should be used.

3.5.2.Increasing Slab Thickness:

Strengthening by thickening of slabs should be applied in cases of insufficient length or stiffness. The thickening can be done above or under the existing slab. The first case the flexural strength is increased because of the increased effective depth and ability to add negative reinforcement as reports. In the second case the flexural strength increases cause of the newly added tension reinforcement. In the case of normal concrete is used; in the case of the application of shotcrete is more suitable. The strengthening according to case provides greater floor slab stiffness for anhragm action and is strongly recommended. In case the performance will improved if the beams are also jacketed.

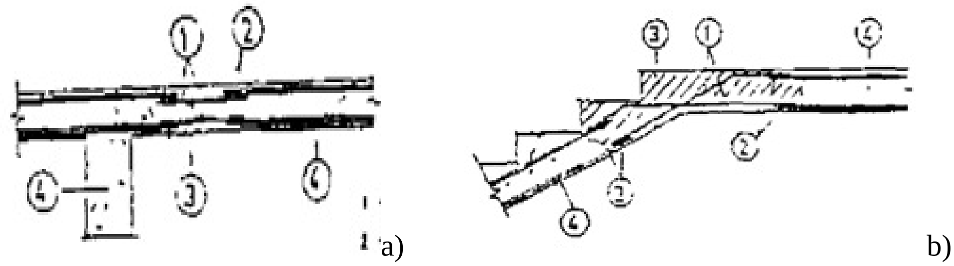
Or compatibility of the slab and the newly-added reinforce concrete, excellent wear bond is of great importance. It can be achieved by the following:

Reinforced concrete lugs

Rough surface, realized by gluing of sand grains with epoxy resin

- Epoxied steel dowel bars
- Additionally shaped concrete lugs in slab voids
- Steel dowels made either by steel angle, anchored by power concrete nails or by epoxied bolts or wedge anchor bolts

Roughening of the surface improve the bond between original and added concrete. It can be performed by sandblasting or by chipping with special mechanical equipment.



1- added reinforcement. 2- welding. 3- added concrete . 4- existing slab.

Fig. 3.11 Reinforcement of floor slabs or stair case slabs

3.6 Summary and Conclusion:

Each of these methods comes with a series of advantages and disadvantages. Some, like section enlargement, add considerable permanent load to the structure and may need more strengthening done to the other structural members. The external plate bonding technique and external post tensioning are susceptible to corrosion damage which may lead to failure of the strengthening system.

All of the repair techniques are very effective in increasing the element's carrying capacity or at least restoring the structural performance of the concrete members before deterioration. The selection of the most appropriate method to use will depend on several factors, such as the amount of strengthening required, the location where strengthening is required, architectural requirements, simplicity and speed of application, and total cost.

CHAPTER 4

Strengthening of Reinforced Concrete Structure Elements Using Steel Plates

4.1 Strengthening Design of Reinforced Concrete Beams by Addition of Steel Plates

4.1.1 Objective

This Chapter presents a synthesis of the strengthening design of reinforced concrete beams and columns by external reinforcement. The design criteria, the methods of analysis and the evaluation of the design resistant bending moment and shear are presented. Construction details for this type of strengthening are also presented.

4.1.2Keywords

Strengthening, reinforced concrete beams, design, bending, shear.

4.1.3 Introduction

The need to strength a structure is caused byproblems due to a wrong design, the degradation of the characteristics of thematerials along the time and the amplification of the load capacity caused by a new utilization of the building. Other causeis the publication of new design codes that increases the actions,such as the seismic action.

The design methods and criteria for thebending and shear strengthening of reinforced concrete beams byexternal reinforcement addition are presented in this paper.

The structure strengthening design must be precededby a strength assessment of the existent structure. This involvesa compilation of all the information related to the build erection and design, a structural inspection and a load capacity evaluation. Taking into account the objectives ofthe strengthening there are various types of interventions that can be adopted: demolition of all the structure, substitution of some structural elements, new structural elements introduction or the strengthening of the elements introduction or the strengthening of the existent elements.

The conception must be based on the structural assessment and the objectives to achieve. That conception must be established in the building, by economic reasons, and to reduce the consequences that the strengthening work causes in the non – structural elements and in the normal use of the building.

4.1.4 Strengthening design criteria

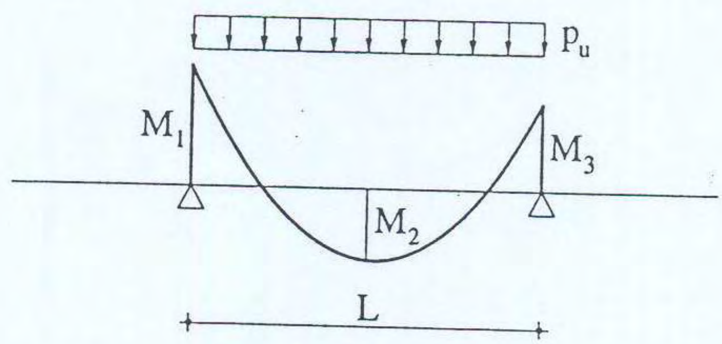
The strengthening design includes the Ultimate Limit States and the Limit States verifications. In the analytical evaluation of the structural resistant capacity there are used models that simulates the actions, the materials characteristics, the structure behavior, the design criteria and the safety level. In the strengthening design the serviceability Limit States verification, the existent damages and the lower stiffness of the strengthened elements must be considered. A reduction of the cross sections area and inertia is usually adopted.

In strengthening design the Ultimate Limit States verification is expressed by:

$$S_d = \gamma_F' \, S(F') \le R_d = \gamma_{n,R} \; R\left(\frac{f_k'}{\gamma_m'}\right) \tag{1}$$

Where S denotes the F loads effects, Y_F the actions partial safety factor, R the cross sections strength, f_K the materials characteristic strength and Y_m the partials materials safety factor. The subscript d means a design value and the superscript' refers a value that can be different from that one usually adopted in the design of a new structure. This is caused by the additional uncertainties related to the existent damage simulation and, s a consequence of the structure assessment, by the possibility to reduce the materials strength uncertainly.

The load effects evaluation in the strengthening design process is generally based on plastic models, figure 4.1, or elastic analysis followed by stresses resultant redistribution, as shown in figure 4.2, because the need to reduce the number of strengthened elements and the necessity to use the full strength capacity of the existing elements, the elastic model is not usually adequate.



$$p_u = \frac{(2\,M_2 - M_1 - M_3) + \sqrt{(M_1 + M_3 - 2M_2)^2 - (M_1 - M_3)^2}}{L^2/2}$$

Fig. 4.1- Plastic analysis – Ultimate load of a beam, p_u.

❖ Two different models can be used to evaluate the section strength:

- Modelling of the damage in the initial section and the additional materials behaviours, including the bond between the new and old materials;

- Simplified method based on the use of a global coefficient – monolithic – which takes into account all the factors that cause a section strength reduction. The design is made as a new structure, assuming that there are no damages and a perfect connection between the materials (monolithic). The values are than reduced by the strength monolithic coefficient, $Y_{n,R} \leq 1$, that depends on the strengthening type and technique. For the structures displacements evaluation are used monolithic coefficients of stiffness, $Y_{n,k} \leq 1$.

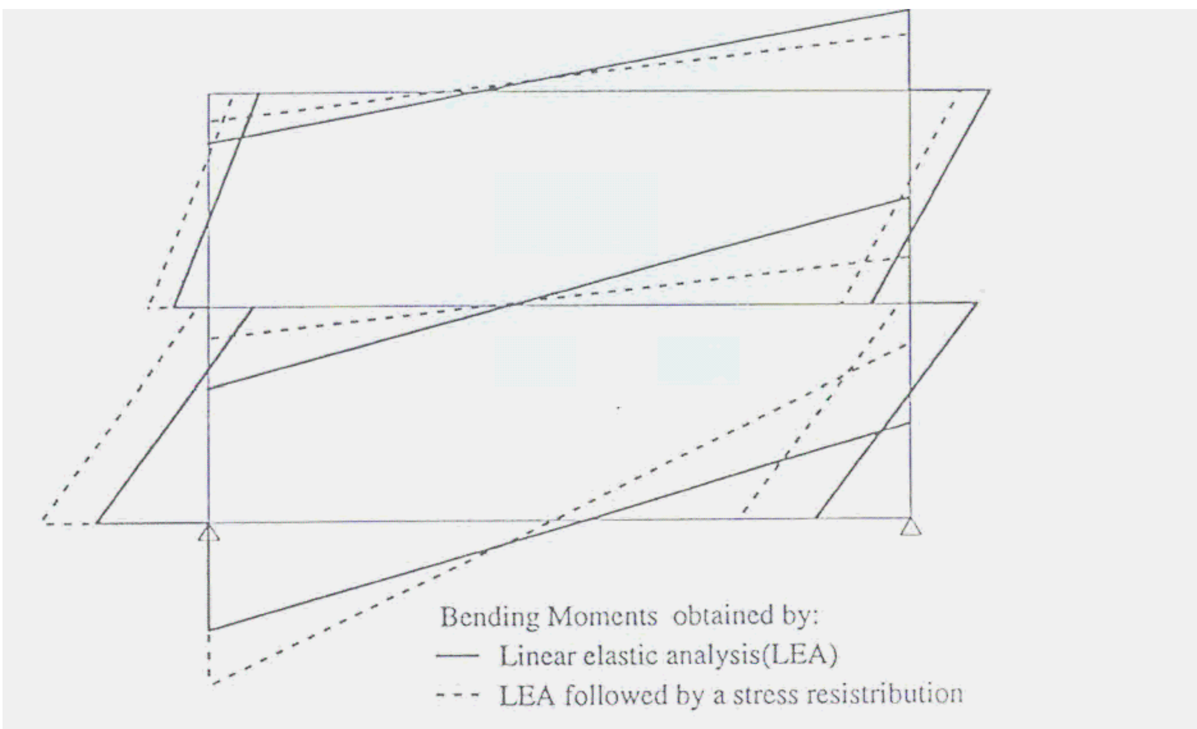


Fig. 4.2-: Elastic analysis following by a stress resultant redistribution.

4.1.5 Description of The Strengthening Technique

For the elements with insufficient reinforcements and a good concrete quality, the external reinforcements addition is the more adequate technique. There are usually used steel plates or hot rolled sections, mainly angle profiles as reinforcements. To mobilise the additional steel strength with low deformations of the strengthened element it is convenient the use of a low tensile strength steel, such as the Fe 360.

In terms of serviceability Limit states this type of strengthening increase the section inertia and stiffness and as a consequence of the resin injection that re-established the continuity of the element.

The additional steel reinforcements are connecting to the initial section by gluing with injected epoxy resin. The use of high strength steel bolts (steel anchor) particularly in the anchorage zones, near the ends of the plates is convenient. The strengthening efficiency is mainly depending on the connection behavior.

When the connection is only guaranteed by the resin a steel plate with a maximum 5 mm thick and 200 mm width is recommended. The resin thick must be comprehended between 1 and 3 mm. a higher resin thick leads to a lower bond capacity. Figure 3 and 4 shows the recommended steel plates dimension, j. Appleton and A. Gomes (1997)>

To allow the additional steel mobilization for the service loads must be removed from the structures during the strengthening execution.

A careful preparation of the concrete and steel surfaces is necessary to achieve a good bond quality. The concrete surface preparation is done by alight pneumntic needle hammer in order to increase the rough ness. A high roughness is inconvenient because it lead to an elevate resin thick. The steel plates are shotblastedand protected against the corrosion with a plastic film for transport and handling.the plastic film is removed before the steel plate application.

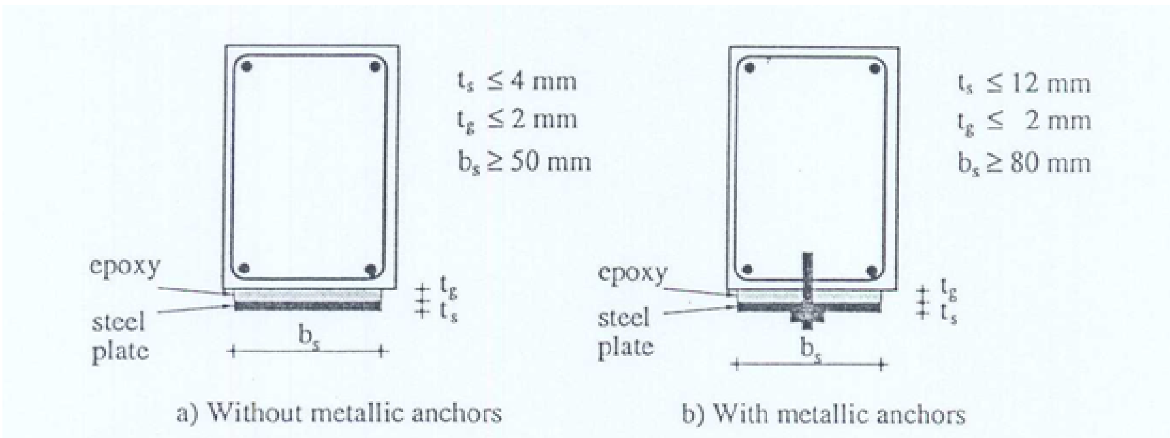


Fig. 4.3-: Flexural strengthening – recommended dimensions limits.

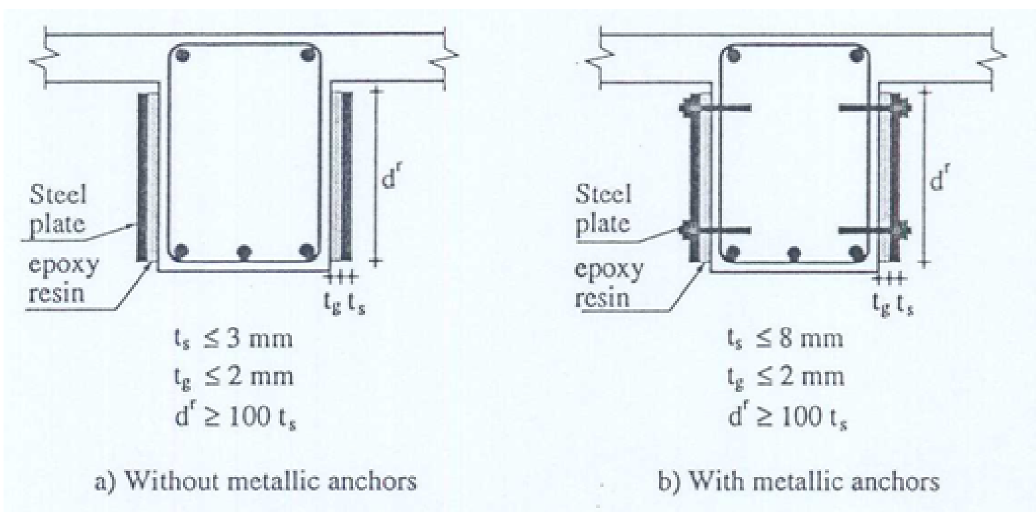


Fig. 4.4-: Shear strengthening – recommended dimensions limits.

After surface preparation, the reinforcing steel plates are installed free of adhesive, using the bolts placed into holes drilled in the concrete member. To allow for complete confinement of the airdrop, plate boundaries and both heads are sealed with an epoxy mortar. Small diameter tubes are left around the plate boundaries. The injection of the resin is made through those tubes. In the final of the injection, the tubes are blocked to allow some pressure in the resin. This causes the penetration of the resin in the small cavities and cracks.

The more relevant resin properties for this type of application are the viscosity, the pot life, the hardeningtime, the elasticity modulus and the strength. It is not convenient the application of the resin with a ambient temperature lower than 10 °C, J. Appleton and V. Silva (1992).

The strengthening of a concrete beams by addition of steel plates involves the followings steps:

- Concrete surface preparation;
- Positioning of the steel plates fixed with metallic anchors;
- Sealing of the steel plates boundaries with an epoxy mortar;
- Epoxy resin injection.

4.1.6 Beams strengthening Design:

4.1.6.1Bending

Figure 4.5a) shows the behavior model in the Ultimate Limit State. In structural elements design the global coefficient method – monolithic coefficients – can be use, as shown in figure 4.5b). According this method the section design bending strength, M_{rd}, is determined as a new element with two reinforcement layers. The resulting value of M_{rd} is then affected by the monolithic coefficient. For the steel plates is adapted the following monolithic coefficients, Eurocode 8 (1995): Bending $Y_{n, M}$ = 1.0, shear $Y_{n, V}$ = 0.9.

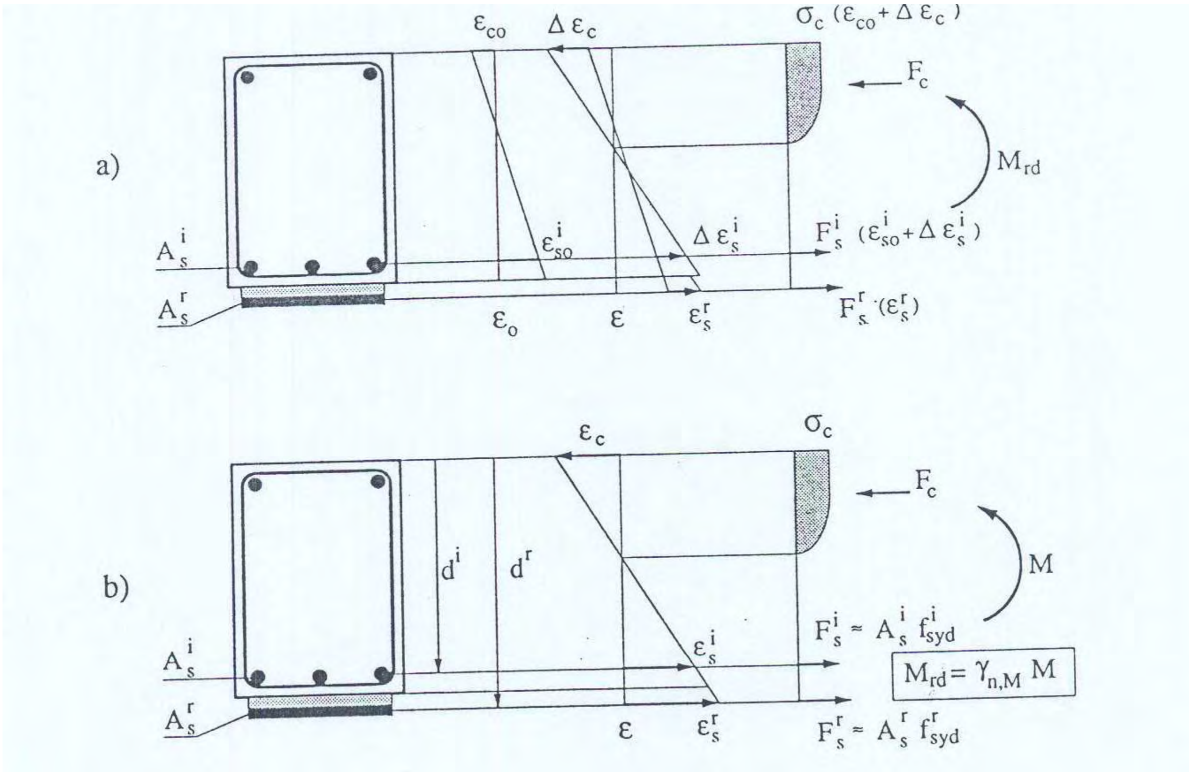


Fig. 4.5-: Bending strength models.

If the distance between the two reinforcement layers is small, the M_{rd}value can be determined by a equivalent reinforcement area, A_s^{eq}, with a tensile strength f^i_{syd}, positioned at the mechanical center. The design bending strength can be expressed by:

$$M_{rd} = A_s^{eq}\, z^{eq}\, f_{syd}^i = A_s^i\, z^i\, f_{syd}^i + A_s^r\, z^r\, f_{syd}^r \qquad (2)$$

Assuming z ≈ 0.9 d gives:

$$M_{rd} = A_s^{eq}\, 0.9\, d^{eq}\, f_{syd}^i = f_{syd}^i \left(A_s^i\, 0.9\, d^i\, f_{syd}^i + A_s^r\, 0.9\, d^r\, \frac{f_{syd}^r}{f_{syd}^i} \right) \qquad (3)$$

Usual design tables for reinforced concrete section can be used with the previous equation. The strengthening steel area is given by:

$$A_s^r = \frac{f_{syd}^i}{f_{syd}^r} \left(A_s^{eq} \frac{d^{eq}}{d^r} - A_s^i \frac{d^i}{d^r} \right) \qquad (4)$$

The connection between the steel plate and the concrete strength is determined assuming a plastic distribution of bond stresses as shown in figure 4.6.

The design connection strength verification can be expressed by following equations:

- Connection without bolts.

$$F_{sd} = A_s^r \, f_{syd}^r \le \tau_{rd} \, d \, \frac{L}{2} \qquad (5)$$

$$\tau_{rd} = \min(f_{ct,min}, 2\ \text{MPa}) \qquad (6)$$

- Connection with bolts.

$$F_{sd} = A_s^r \, f_{syd}^r \le nF_b + \tau_{rd} \, d \, \frac{L}{2} \qquad (7)$$

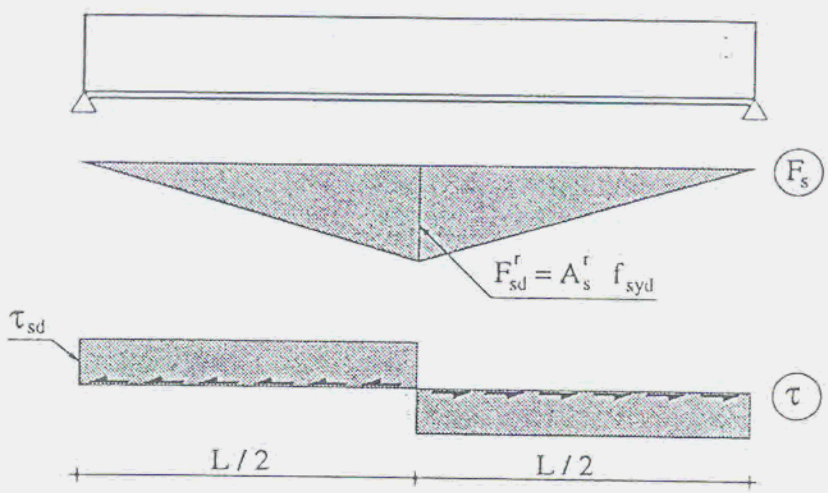


Fig. 4.6-: Distribution of bond stresses in the steel plate connection to the concrete.

F_b denotes the bolt design shear strength, n the number of bolts positioned in the length L/2, b the plate width and t_{rd} the design bond strength of the steel/resin/concrete connection. Is recommended a value of 0.5 MPA for t_{rd} when metallic anchors are used, J. Appleton and A. Gomes (1997).

In the connection serviceability verification, it is recommended an elastic model to the shear stresses evaluation. For the service load level, the connection behavior depends mainly on the resin. Experimental tests have shown when the metallic anchors were not applied early collapses occurs caused by the steel plates debond, J. Alfaiate (1986). The presence of the bolts in the plate extremities led to ahigher ultimate load capacity mobilizing the full strength of the steel plate. Other advantage in the use of the bolts is the fire strength.

To improve the connection strength, the steel plates must be extended until the edges of the beam and anchored to the column as shown in figure 4.7a). in a continuous beam the forces can be transferred along the columns by the connection of the steel plates to a collar usually made by angles as shown in figure 4.7b). this collar is also useful in column strengthening.

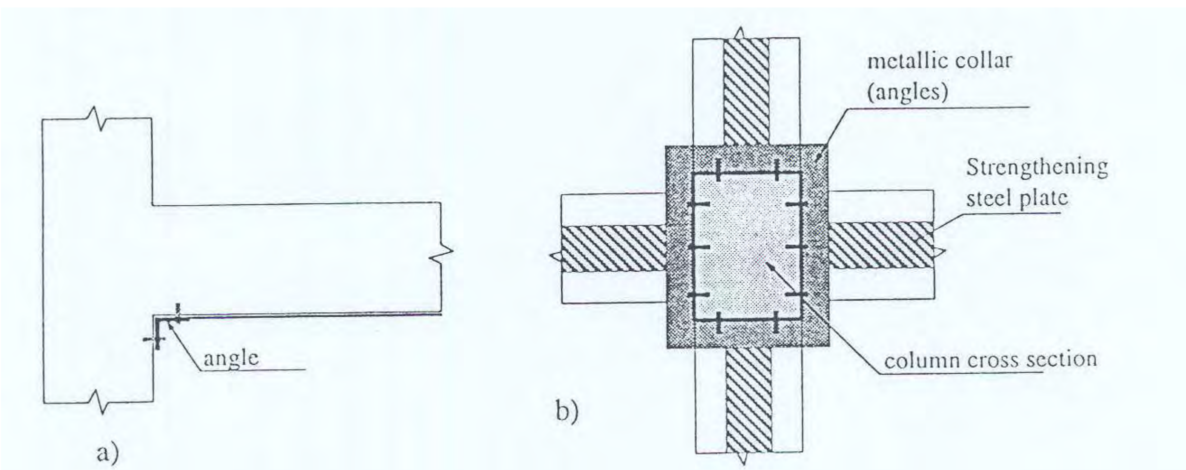


Fig. 4.7-: Connection details of the steel plates: a) Beam edges; b) Beam column connection.

4.1.6.2.Shear

If there are an insufficient transverse reinforcement, the shear strengthening by steel plates addition is adequate. The shear strength of a strengthened beam can be determined by the simplified method of the monolithic coefficients as expressed in the followings equations:

$$V_{sd} \leq V_{rd}^{max} \quad A_s^r = \tau_2 \; b_w \; d^i \tag{8}$$

$$V_{sd} \leq V_{rd} = V_{cd} + V_{wd} \tag{9}$$

$$V_{cd} = \tau_1 \; b_w \; d^i \tag{10}$$

$$V_{wd} = \gamma_{n,v} \left(0.9 d^i \frac{A_{sw}^i}{s} f_{syd}^i + 0.9 d^r \frac{A_{sw}^r}{s} f_{syd}^r \right) \tag{11}$$

Figure 4.8 shows details of the steel plates connection used to shear strengthening. Fig 3.8a) shows the solution obtained with a continuous steel plate which has the inconvenient of a large concrete surface to prepare and a higher difficulty to inject resin. The use of an angle profile in connection to the flange allows the application of mechanical steel anchors and led to a better solution. Fig 3.8b show the usual solution for the shear strengthening. The angle in the corner section makes the connection between the lateral and bottom plates.

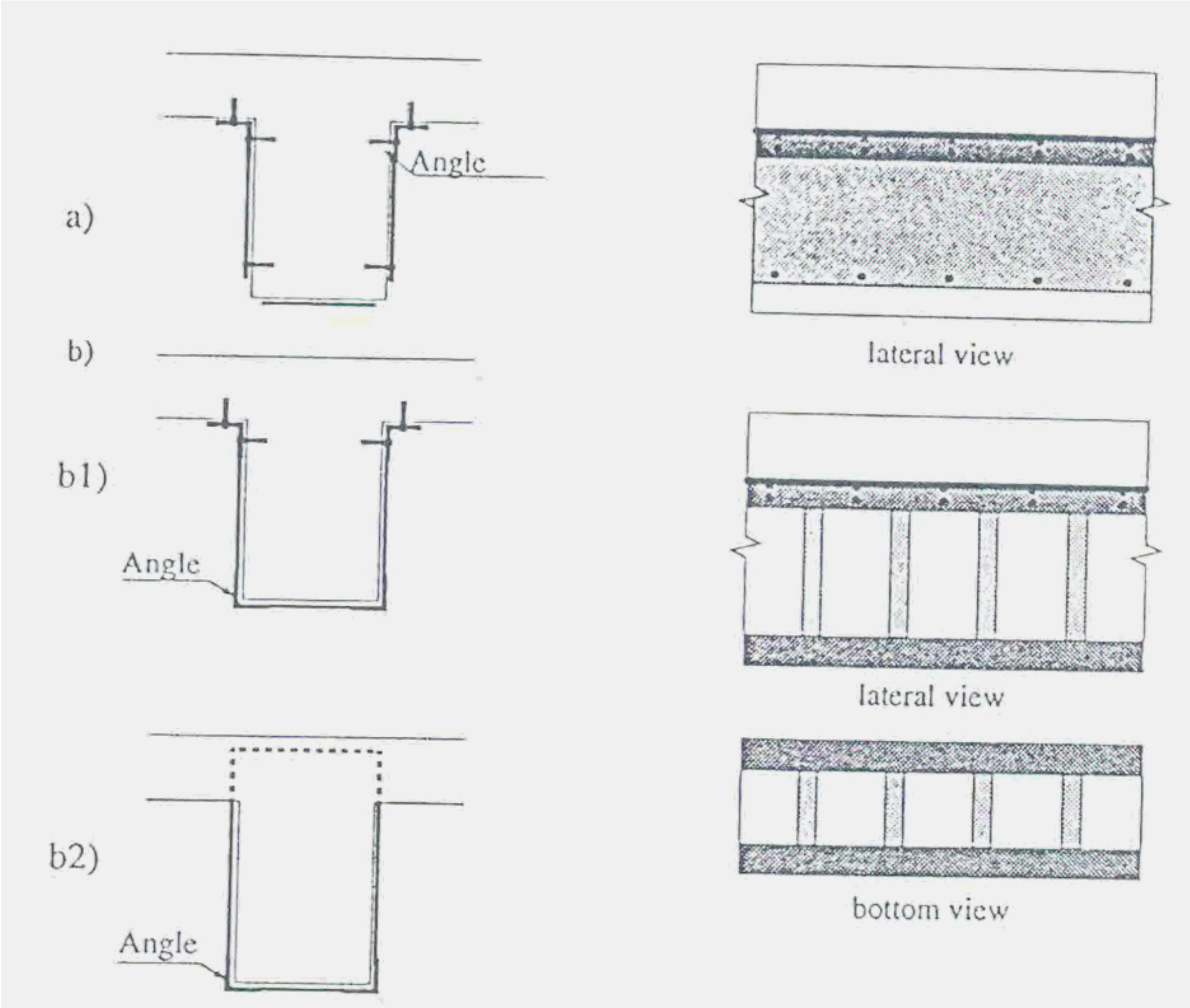


Fig. 4.8-: Shear strengthening details.

4.2. Strengthening Design of Reinforced Concrete Columns by Addition steel Plates

The columns strengthening scopes are the increase of the confinement, flexural strength and axial strength. The confinement level increase led to an improvement in the ductility and seismic behavior of the structure and to an axial capacity load increase.

The more adequate strengthening technique to the axial force capacity is the jacketing. The external addition of steel is more suitable in cases that have reinforcement insufficiency. Steel plates or profiles (angles) are usually applied.

The design of the columns strengthening is similar to the beams, by the application of the global coefficient method, as shown in figure 4.9. The monolithic coefficient for the bending and compression strengthening is: $Y_{n,\ MN}$ = 0.9, J. Appleton and A. Gomes (1997).

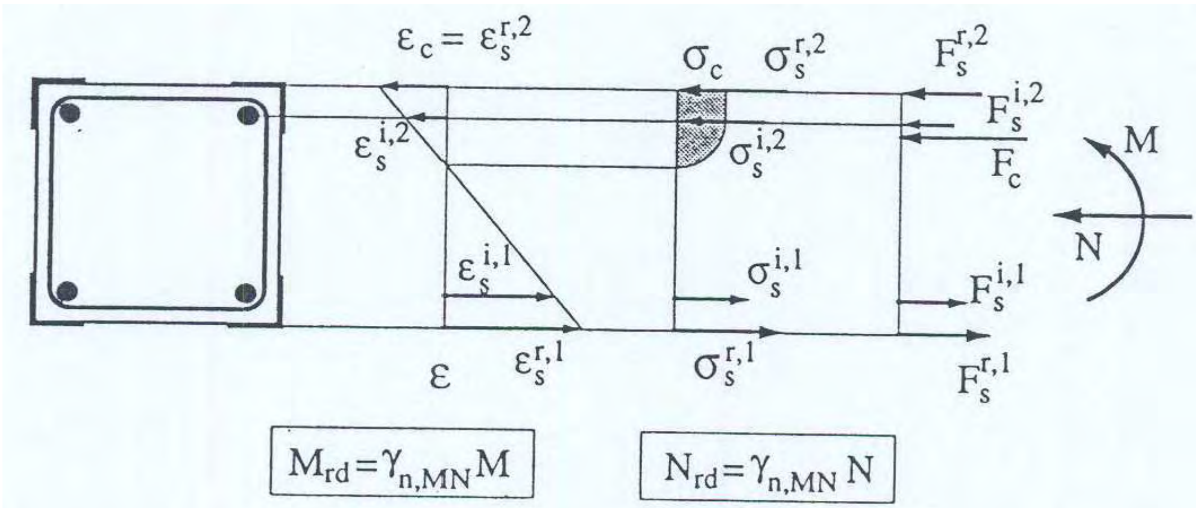


Fig. 4.9-: strength capacity evaluation of a column section

In a simplified way, the section strength can be determined as a new section. That is valid if the distance between the initial and additional reinforcements is small. In this case, it is obtained an equivalent steel area defined by the expression:

$$A_s^{eq} = A_s^i + A_s^r \frac{f_{syd}^r}{f_{syd}^i} \qquad (12)$$

The connection of the additional reinforcements is made by the epoxy resin complemented with steel connectors welded in the initial reinforcements, as shown in figure 4.10a). The application of steel bolt anchors is only possible if the strengthening plate has a sufficient dimension, figure 4.10b).

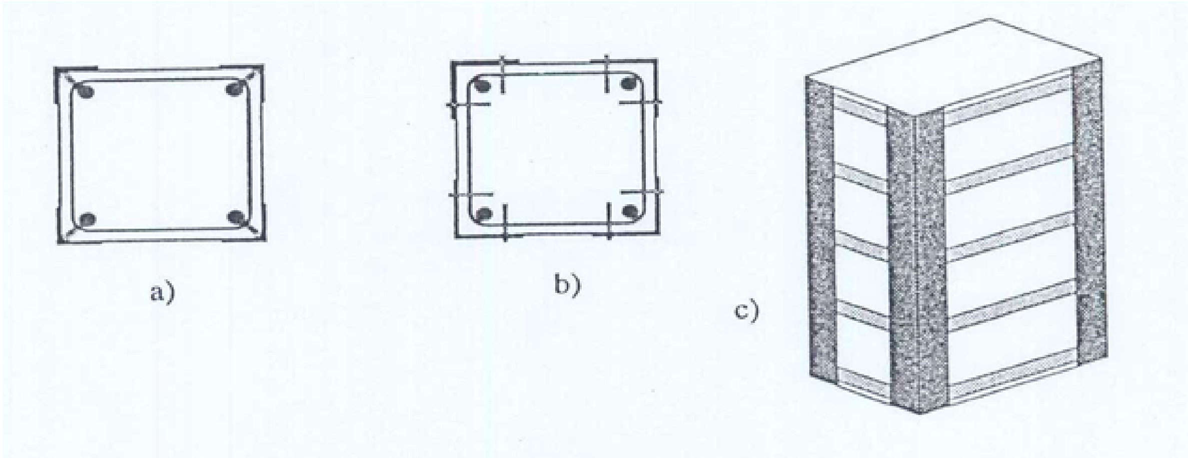


Fig. 4.10-: Column Strengthening details

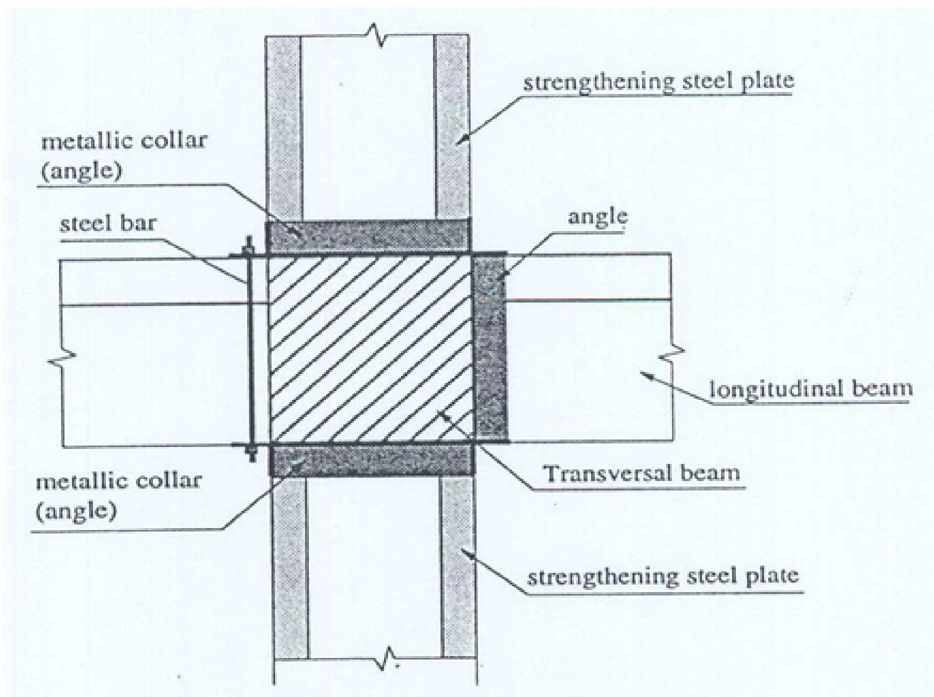


Fig. 4.11-: Strengthening details in a beam column joint.

Figure 4.11 shows the strengthening detail in a beam-column joint. The connection is composed by two collar linked through the slab by steel bars or a steel profiles.

Figure 4.12 shows the strengthening steel plates connection to the foundation. It is used a colar made by a steel angle profile which is connected by a steel anchor.

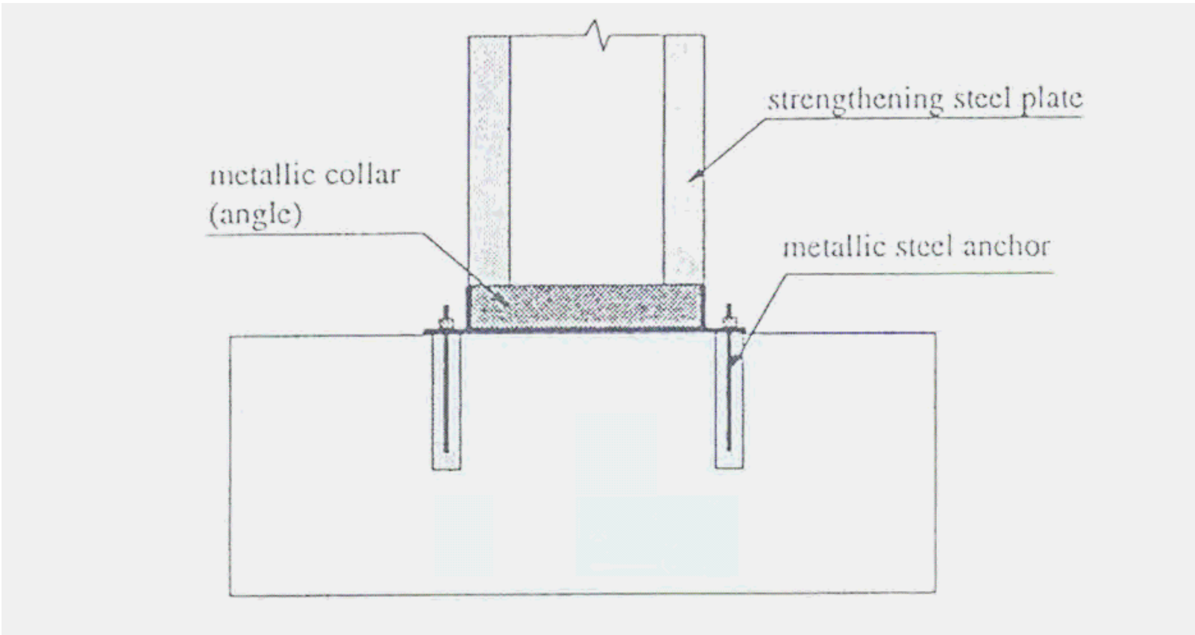


Fig. 4.12: Steel plate connection in the foundation.

Chapter 5

Strengthening of Reinforced Concrete Structure Elements Using F.R.P Materials

5.1.Introduction

For eight years New Zealand has been to the forefront in the development and use of FRP materials as a means of strengthening civil engineering structures. However, as of this day, there is no national guideline available which sets down recommendations for the design and detailing of FRP for strengthening of civil engineering structures. The authors have spent 3 years researching the technology and the available guidelines worldwide. This paper sets out proposals for interpretation of these guidlelines so that they may be used in the detailing of typical NZ structures.

5.1.1. Generic Information

The use of fibre reinforced polymers (FRP) as reinforcement for structures is rapidly gaining appeal. This is due to the many advantages these materials afford when compared to conventional steel reinforcement or concrete encasements. Their light weight, high strength-to-weight ratio, ease of handling and appplication, lack of requirement for heavy lifting and handling equipment and corrosion resistance are some factors that are advantageous in repair, retrofitting and rehabilitiation of civil engineering structures.

While no country yet has a national design code, several national guidelines [1-6] offer the state-of- the-art in selection of FRP systems and the design and detailing of structures incorporating FRP reinforcement.

However, there exists a divergence of opinion about certain aspects of the detailing between
guidelines. This is to be expected as the use of the relatively new material develops worldwide. Much research is being carried out at institutions around the world and it is expected that design criteria will continue to be enhanced as the results of this research become know in the coming years. This development process is akin to that which occurred in the 60s and 70s in the field of prestressed concrete.

The main areas of detailing where this divergence occurs are presented for discussion in this paper. Recommendations are given as to which guideline should be followed. However, as in all design and detailing carried out by responsible engineers, the final decision as to what criteria is chosen must rest with the designer.

5.1.2.Types and Properties Of FRP Used For Strengthening

The main fibre types used are carbon (CFRP), glass (GFRP) and aramid (AFRP). GFRP comes in two types – E-glass and AR glass. E-glass is the most common form used but it has the disadvantage that it is attacked by the alkali in fresh concrete or grout. AR-glass (AR = alkali resistant) is the answer to this.

E-modulus (hence ultimate strain and UTS) is the defining property of all FRPs and dictates the preferred uses for each generic type. Typical properties are given in Table 5.1

	E-modulus (GPa)	Ultimate Strain (%)	UTS (MPa)
CFRP (laminate)	165 - 215	1.3 – 1.4	2500 - 3000
CFRP (sheet)	240 - 640	0.4 – 1.6	2650 - 3800
GFRP (sheet)	65 - 75	4.3 – 4.5	2400
AFRP (sheet)	120	2.5	2900

Table 5. 1: Typical FRP Properties (dry fiber)

One of the governing properties used in design is the allowable strain in the FRP at ultimate limit state (ULS). It can be seen from Table 5.1 that there is a large range (0.4% – 4.5%) of ultimate strain of the various fibres, depending on the type. Hence selection of the correct material for each application is paramount to the design process. Table 5.2 provides a guideline to the selection of the type of material for each structural element and whether the requirement is for enhancement of confinement, flexure, axial load, ductility etc.

The bonding of the fibres to the substrate is affected by immersing the fibres in a matrix of epoxy resin.

The bonding material may be applied to either the surface or within slots cut in the cover concrete. Thus the FRP can be defined as a layer (or layers) of fibre embedded in a matrix of epoxy resin and bonded to (into) the concrete.

Element	Application	Glass Fibre Sheet (GFS)	Carbon Fibre Sheet (CFS)	Carbon Fibre Laminate (CFL)
	Fibre Direction	Uni-directional	Uni-directional	Uni-directional
Columns	*Fibre arrangement*	Woven	Straight	Straight
	Confinement	☼☼	☼☼	NA
	Flexure	☼	☼	☼☼☼
	Axial Load	☼☼	☼	☼☼☼
	Ductility	☼☼	NA	NA
	Durability	☼☼	☼	NA
Beams	Flexure	NA	☼	☼☼
	Shear	NA	☼☼	NA
Walls	Shear & flexure	☼☼	☼	☼
Slabs	Flexure	NA	☼	☼☼
Durability	Spalling	☼☼	☼	NA

☼ Possible use
☼☼ Preferred use
☼☼☼ Special application

Table 5. 2: Application Uses for F.R.P

Arrangements of the fibres vary from type to type. Table 5.3 sets out the comparison and description of the commonly used materials.

There is no fixed rule as to whether sheet or laminate should be used. Usually economy dictates the choice of one system or the other, but sometimes it is a design choice. Carbon (laminate or sheet) appears to be more economic for use in flexural or shear strengthening. Certainly, carbon has better fatigue properties than glass, so where the strengthening is used to carry often occurring fluctuating live loads, carbon should be chosen. Glass, because of its lower E-modulus, is more suitable for use in confinement of concrete, although it can, in certain circumstances, be used for flexural enhancement. Because of its low modulus, glass is seldom used for shear enhancement.

Laminates can only be applied to plane surfaces, therefore carbon, aramid or glass sheet are used on curved surfaces. Carbon sheet, on the other hand, is difficult to cut and handle in thin strips and therefore laminates are preferred, when narrow bands of FRP reinforcement are required.

Bi-directional glass fabrics are used for increasing the shear strength of masonry walls. Lighter fabrics are used where the substrate strengths are low, such as in old and historic masonry or brick buildings.

Composite Type	Fibre direction	Fibre arrangement	Typical application
Carbon Fibre Sheet (CFS)	Uni-directional	Straight	Increase in flexural and shear capacity; confinement
Aramid Fibre Sheet (AFS)	Uni-directional	Straight	Special applications
Glass Fibre Sheet (GFS)	Bi-directional	Woven	Increase in confinement and ductility
Carbon Fibre Laminate (CFL)	Uni-directional	Straight (partially pre-tensioned)	Increase in flexural capacity

Table 5. 3 – Fibre direction, arrangement and typical uses

The substrate to which the FRP is to be adhered, must have sufficient strength to transfer the loads from the FRP to the structure. Testing of the tensile strength of the substrate by pull-off tests is imperative. Table 5.4 sets out the minimum substrate strengths required for each of the FRP materials to be used efficiently:

Product	Minimum Tensile Strength (MPa)
Carbon Fibre Sheet (CFS)	> 1.0
Aramid Fibre Sheet (AFS)	> 1.0
Glass Fibre Sheet (GFS)	> 0.2
Carbon Fibre Laminate (CFL)	> 1.5

Table 5. 4 – Minimum substrate strengths for various F.RP materials

In some instances, the design minimum tensile strength may be increased when multiple in-situ pulloff tests indicate that the substrate strength is substantially higher than the figures given in the table. As an example, the design figure used at West Gate bridge was increased from 1.5 MPa to 1.9 MPa

5.2. Design Considerations

5.2.1 Design for flexural enhancement

The design of externally bonded FRP reinforcement (FRP EBR) for flexural members is based on limit state principles and relies upon the composite action between a reinforced or prestressed concrete element and the EBR. In general, strength, ductility and serviceability requirements must all be investigated. The design procedure is analogous to that for reinforced concrete beam and slab sections, with no axial load. The FRP strengthening materials are treated as additional reinforcement with different material properties. The only difference is the initial strains that are present in the concrete and reinforcement, due to the dead load at the time of applying the FRP.

Current design recommendations generally set acceptable levels of safety against the occurrence of both serviceability limit states (excessive deflections, cracking) and ultimate limit states (failure, stress rupture, fatigue). Possible failure modes and subsequent strains and stresses in each material (concrete, reinforcing steel and FRP) should be assessed at ULS and the avoidance of a brittle concrete failure ensured. In respect of the design of FRP systems for the seismic retrofit of a structure, attention is drawn to recommendations given in section 8.1 of [2].

The design procedure must consist of a verification of both limit states. In some cases, it may be expected that the SLS will govern the design.

For buildings (and other applicable structures), fire should also be included as a limit state as it will influence the properties of both the FRP and the adhesive used to attach it to the concrete.

Accidental loss of support from the FRP due to vandalism, impact etc, should be considered.

The safety concepts at ULS, adopted by most guidelines, are related to the different failure modes that may occur. Brittle failure modes, such as shear and torsion, should be avoided. In addition, and for the same reason, it should be guaranteed that the internal steel is sufficiently yielding at ULS so that the strengthened member will fail in a ductile manner, despite the brittle nature of concrete crushing, FRP rupture or bond failure. Hence the governing failure mode of a flexural member will be either steel yielding/concrete crushing (before FRP rupture or de-bonding) or steel yielding/FRP failure (either FRP rupture of bond failure) before concrete crushing. In all cases, verification that the shear (torsion) capacity of the strengthened member is larger than the acting shear (or torsion) forces is necessary. If needed, flexural strengthening must be combined with shear strengthening.

The design approach to strengthened sections is normally based upon a trial and error approach, which can be easily carried out by means of a simple spreadsheet. The initial type, size and length of the FRP reinforcements are selected at random. Then the flexural safety of the strengthened section is checked by analysing its limit states. If the safety check fails, or if the selected FRP strengthening elements are not economical, a new size or type of element is selected and the process is run again. Usually a few iterations are sufficient to arrive at a safe and economical solution.

Custom designed software exists for the design of FRP strengthening using CFRP laminates. One particularly good programme is available in the public domain on www.frp.at. It is written for either the ACI code (US), the British, French and German codes, as well as Eurocode 2. The properties of the FRP used in this programme are those relating to the products manufactured by the owner of the software.

The following assumptions are considered valid for the concept of design of FRP EBR:

- There is a perfect bond between the FRP and the bonded substrate. This is, in fact, achieved without difficulty in practice, and failure, if it occurs, is always in the substrate.
- Plane section remain plane (Bernoulli's principle).
- The stress-strain responses for concrete and steel reinforcement follow the idealised curves presented in current codes and standards.
- FRP has a linear elastic response.
- The tensile strength of the concrete is ignored.
- Loads which are in place at the time of application of the FRP cause the element being reinforced to act within its elastic limit.
- The existing conditions have been properly evaluated (this includes steel areas and properties, concrete strength, existing moments and shear forces, steel and concrete strains, etc).

For the ultimate and serviceability limit states, the design loading is obtained by multiplying the characteristic dead and imposed loads by appropriate load factors and strength reduction factors. Designers must incorporate factors from design codes acceptable to the location of the works. Figure 5 sets out load factors, partial factors of safety and material reduction factors for some codes. In addition, it is normal to use strength reduction factors when calculating ultimate strength. Some codes (EC2 and BS 8110, for example) use separate material strength reduction factors for reinforcing steel and concrete, while others (ACI 318, NZS 3101 and Austroads Bridge Design Code, for example) use global strength reduction factors for these two materials.

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CODE	LOAD FACTORS		MATERIAL STRENGTH REDUCTION FACTORS				STRENGTH REDUCTION FACTOR
	DEAD LOADS	LIVE LOADS	CONCRETE	STEEL REINFORCEMENT	FRP REINFORCEMENT		
	γ_G	γ_Q	γ_C	γ_S	STRENGTH	E-MODULUS	φ
BS 8110	1.4	1.6	1.5	1.15	VARIES	1.1	-
ACI 318	1.4	1.7	-	-	0.85	1.0	0.7 TO 0.9
NZS 3101 & NZS 4203	1.2	1.6	-	-	-	-	0.85
EURO CODE 2	1.35	1.5	1.5	1.15	VARIES	1.0	-
AUSTROADS	1.2	2.0	-	-	-	-	0.6 TO 0.8

Table 5. 5 – Load factors, Material Partial Safety Factors & Strength Reduction Factors of different Design Codes and FRP Guidelines.

The methods of incorporating strength reduction material factors for the FRP varies according to the FRP Design Guideline used.

The **UK Concrete Society TR 55** [1] recommends 3 separate factors, which relate to the method of manufacture, the type of FRP material and the degradation of the E-modulus over time.

The **German General Guidelines** [3] presently recommended reduction factors by limiting the allowable strain at ULS to between 0.4 and 0.7 of the ultimate strain and at SLS.

The draft **ACI 440** [2], as well as using a global strength reduction factor recommends a strength reduction factor for the FRP of 0.85 and an additional environmental strength reduction factor.

The ***FIB* Bulletin 14** [4] uses FRP material safety factors and also places limitations on the FRP strain at ULS and SLS.

Presently there are no Codes of Practice that include for the use of FRP as a reinforcement material. The designer therefore must take into account suitable limitations on the use of FRP, either by separate material reduction factors as per the UK and *fib* Guidelines, additional strength reduction factors used in conjunction with the global reduction factor, as for ACI 440, or a fixed upper limit of allowable strain, as per the German General Guideline and ACI 440.

All guidelines limit the stress/strain in the FRP to avoid de-bonding, which can occur in several mechanisms. In addition, due to the general decrease in ductility of a member strengthened with FRP, care must be taken to ensure ductility is preserved, by ensuring the internal steel will sufficiently yield at failure. This is done by limiting the depth of the compression zone at ULS.

5.2.2 Design values for material properties

As mentioned above, at the time of writing there are no Codes of Practice that set down the requirements for the design and execution of concrete strengthening using FRP. However, there are at least eight national guidelines that have been produced by recognised authorities and these can be accepted as state-of-the-art guidelines for the present. Nevertheless it must be recognised that the use of FRP as a strengthening medium, is a relatively new art and that research is being undertaken in many centres worldwide. The results of this research will undoubtedly cause the recommendations to be varied as experience is gained.

The various FRP Design Recommendations treat the strength reduction of the FRP material in different ways.

UK Concrete Society TR No. 55 [1]

TR 55 [1] postulates that the partial safety factors to be applied to the characteristic mechanical properties are a function of the type of fibre and the manufacturing/site application process. Thus

$$\gamma_{mF} = \gamma_{mf} \times \gamma_{mm}$$

Where □ *mf* depends on the type of fiber and □*mm* depends on the manufacturing and/or site application process. Typical values are given in (19) .

5.2.3 End conditions and development lengths

Members strengthened externally with FRP can fail prematurely as a result of local FRP separation. This can be caused by three different mechanisms: peeling, debonding and cover tension delamination.

Peeling failure may occur at the ends of the FRP where a discontinuity exists as a result of the abrupt termination of the plate. TR 55 [1] reports this phenomenum is normally associated with concentrated shear and normal stresses in the adhesive layer due to the FRP deformation that takes place under load. Peeling failure usually results in ripping of the cover concrete off the adjacent layer of steel reinforcement.
Debonding, unlike peeling, mostly occurs away from the plate end. It occurs if the bonding material is not up to specified strength or has not been properly applied. Debonding failure may also be indicative of inadequate preparation of the concrete substrate. More commonly, however, it is associated with the formation of wide flexural and shear cracks that occur as a result of the yielding of the embedded steel bars. The wide cracks generate high stresses in the FRP across the crack, which can only be dissipated by debonding. The cracks can then propagate towards the plate end, leading to FRP separation failure.

Cover tension delamination results from the normal stresses developed in a bonded FRP laminate. With this type of delamination, the existing reinforcing steel essentially acts as a bond breaker in a horizontal plane, and the reduced area of bulk concrete pulls away from the rest of the beam. The result is the entire cover layer of concrete delaminating from the member .

Peeling or end plate separation failure will be avoided by addressing two criteria:

(i) Limiting the longitudinal shear stress between the FRP and the substrate.

(ii) Anchoring the FRP by extending it beyond the point at which it is theoretically no longer required .

The word “theoretically” has produced intense international discussion. See section 6.4.2.1 and 6.4.2.2 of [19] for commentary and recommendation in this regard.

As a word of caution, the authors consider the limitations imposed by TR 55 [1] as deficient in certain aspects and designers should familiarize themselves with the limitations exposed by TR 55 and make the appropriate engineering decision for themselves. The authors recommend that the method developed by Onken & vom Berg [16] [reproduced in [19] [Figure 5.1] be used to determine end conditions in flexure.

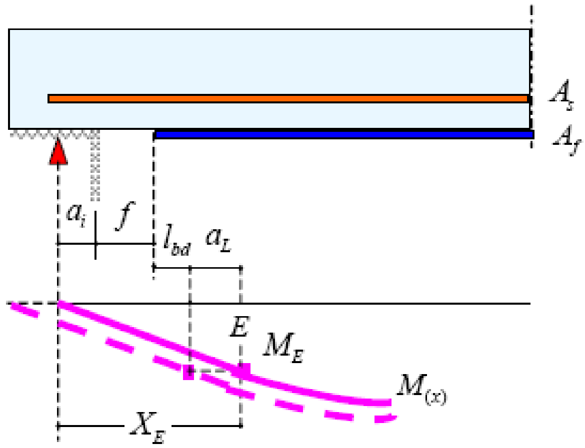

Fig. 5.1 - End condition considerations for FRP used in flexure – Onken and vom Bera [16]

5.2.4 Design for shear strengthening

Externally bonded FRP sheets can be used to increase the shear strength of reinforced concrete beams and columns. The shear strength of columns can be improved by wrapping with a continuous sheet of FRP to form a complete ring around the member. Shear strengthening of beams however, is likely to be more problematic when the beams are cast monolithically with slabs. Attention needs to be paid to anchoring the FRP at or through the beam/slab junction, ensuring that full anchorage occurs above the neutral axis (ie, in the compression zone). The FRP should be placed such that the principal fibre orientation, β , is either 45o or 90o to the longitudinal axis of the member.

- Increasing the shear strength can also promote ductile flexural failures.

ACI 440 [2] recommends that beams and columns on moment frames resisting seismic loads, at locations of expected plastic hinges, or at locations where stress reversal and post-yield flexural behaviour is expected, should only be strengthened for shear by completely wrapping the section with strips spaced less than h / 4 (clear spacing) where h is the depth (width) of the member .

- **Types of shear wraps :-**

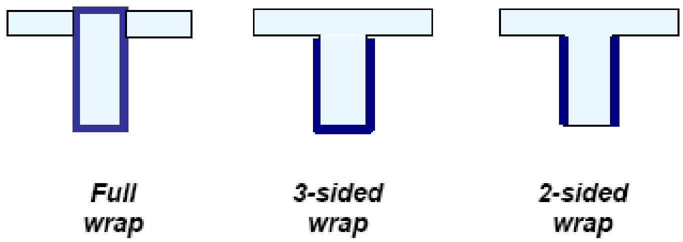

Fig. 5.2 - types of wrapping systems for shear reinforcement

Complete wrapping of the FRP around the section is the most efficient, followed by the 3-sided and the 2-sided wrap. In beam applications, especially T-beams where the neutral axis is mostly found in the slab portion of the beam, it is necessary to ensure the FRP is anchored in the compression zone (above the neutral axis). This is achieved by passing the FRP strip through slots cut in the slab and anchoring it on the top of the slab. Alternatively, anchors capable of transmitting the force from the FRP through into and beyond the mild steel reinforcement stirrups, can be used, if proper detailing of the load transfer from FRP to anchor is considered. **The 3-sided and 2-sided wraps should be used with absolute caution.**

The UK Concrete Society TR 55 [1], German General Guideline [3] and ACI 440 (draft) [2] all treat the shear situation differently. Depending on whether you are in an area where ACI 318 is used (global safety factors), or in Europe (partial material reduction factors), the requirements are quite different. Designers are recommended to study the appropriate code/guidelines, for detailed use .

Current research on shear strengthening with bonded FRP suggests that, as with conventional reinforced concrete, shear failure will occur due to two basic mechanisms, diagonal tension (resisted by shear stirrups) and diagonal compression (resisted by inclined concrete compression struts in tie and strut model).

For a detailed summary of the requirements of each of the three guidelines [19] should be consulted. This summary is not exhaustive and readers are advised to consult the appropriate document they are working with.

The authors recommend that designers use a method which makes sure connection of the FRP shear strengthening members follow the internal truss structure (Figure 5.3). In most cases, this will mean the anchorage of the FRP strips will be located within the compression zone of the concrete.

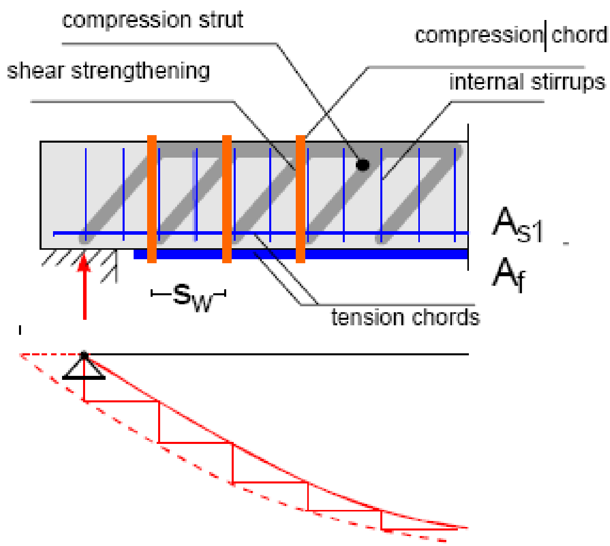

Fig. 5.3 - Connection of the FRP shear strengthening to the internal truss structure.

Spacing of FRP Laminate Strips As in the case with steel shear reinforcement, the spacing of FRP laminate strips should not be so wide as to allow the full formation of a diagonal crack without intercepting a strip. For this reason, if laminate strips are used, their spacing should not exceed the lesser of 0.8d and $w f + d / 4$ where:

d the effective depth of the beam and $w f$ the width of F.R.P laminate strips .

5.2.5 Design for Axial Load Enhancement

Retrofitting to enhance the axial compressive strength of concrete members using FRP material is commonly used. By wrapping a column with an FRP jacket, the shear, moment and axial load capacity, as well as the ductility, are improved. The column is wrapped with the FRP fibers in the hoop direction and this provides significant confinement to the concrete, thus leading to improvement in performance.

GFRP and CFRP are both very effective in enhancing axial performance. Creep of GFRP is not a concern with column wrapping because under normal service loads, the jacket remains virtually stress free.

Both circular and rectangular columns are able to be enhanced with FRP jackets. The most effective situation is the circular or oval jacket, but reasonable enhancement of rectangular columns is achievable, although less than that of square or circular columns.

The original theory and experimental work was carried out by Priestley [7] in 1988 and this has been followed by much research by others. The basis of the theory used widely today comes from the research work carried out by Wang Yung-Chih from 1996 – 2000, at the School of Engineering, University of Canterbury, New Zealand [8].

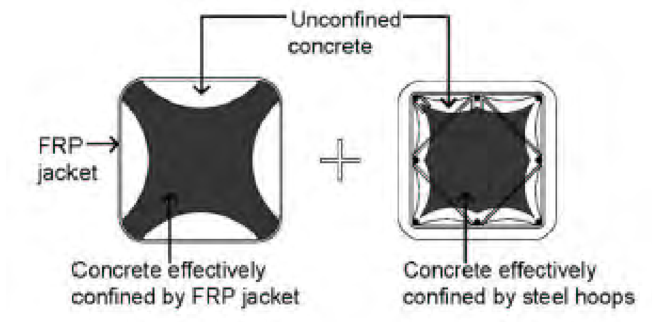

Fig. 5.4 - Dual Confinement Effect on a rectangular column with FRP jacket and internal steel hoops.

Figure 5.4 shows a cross section of a reinforced concrete rectangular column that is confined by an FRP jacket. The concentric compressive load carried by a short reinforced concrete column is the combination of the compressive loads carried by the concrete and the longitudinal reinforcing bars respectively.

Wang [8] assumes that the ultimate limit state in a concentrically loaded column is associated with 1% axial strain. With Poisson's ratio conservatively assumed to equal $\nu = 0.5$, at this strain level, the transverse strain at 1% axial strain is equal to 0.5%.

Wang [8] also postulates that the reinforcing steel behaves as an elasto-plastic material. The nominal compressive strength carried by the concrete, results from the stresses in three distinct regions shown in Figure 5.4. He further postulates that at 1% axial strain the unconfined concrete has reached its peak strength and has degraded to a residual strength to $0.3\,f'c$.

For design purposes it is necessary to reduce the nominal concentric strength to account for variations in the materials properties, scatter in the design equation, bending of the columns, nature and consequences of failure and reduction in load carrying capacity under long-term loads. This is done by strength reduction factors and material reduction factors.

The compressive load carried by the concrete results from the loads sustained by three distinct regions, viz, the unconfined concrete region, the effective area confined by the FRP jacket and the effective area of the concrete confined by both the FRP jacket and the steel stirrups. Hence the entire uni-axial stress-strain relationship for a concentrically loaded column wrapped with an FRP jacket can be obtained if the constitutive stress strain relation ships for each of the regions and for the reinforcing steel are known. The determination of the compressive strength of the confined concrete and the evaluation of the lateral confining pressure due to the elastic jacket and internal reinforcing stirrups is then able to be calculated [8].

5.2.6 Conclusions on design aspects

It is not possible, in the space allocated for a paper such as this, to provide a total picture of the state-of-the-art of this technology. However, suffice it to say that the use of FRP materials will greatly increase in the coming years. They have served their apprentiship and have proven to be economical and beneficial substitutes for the alternative methods of strengthening. The fact that we continue to upgrade our structures to increase ductility, load carrying capacity and seismic resistance will dictate that these materials will continue to be a strong participant in such activities.

New Zealand has lead the way for over 30 years in concrete technology – is it not time now for NZ in concrete technology – is it not time now for NZ to decide it must produce its own set of guidelines for use in this country? This is something the industry should carefully consider.

5.3 Strengthening of Reinforced Concrete Structures with CFRP Laminates

5.3.1 DEFINITION

REHABILITATION AND STRENGTHENING OF EXISTING CONCRETE STRUCTURES HAS (AND MORE IN FOCUS DURING THE LAST DECADE. ALL OVER THE WORLD TH STRUCTURES INTENDED FOR LIVING AND TRANSPORTATION. STRUCTURES ARE OF VARYING QUALITY AND FUNCTION, BUT THEY ARE ALL AGEING AND DETERIORATING OVER TIM STRUCTURES NEEDED IN 20 YEARS FROM NOW ABOUT 85-90 % OF THESE ALREADY BUILT. SOME OF THESE STRUCTURES WILL NEED TO BE REPLACED SINCE TH ARE IN SUCH BAD CONDITION. HOWEVER, IT IS NOT ONLY THE DETERIORATION PROCESSES THAT MAKE UPGRADING NECESSARY, ERRORS CAN HAVE BEEN MADE THE DESIGN OR CONSTRUCTION PHASE SO THAT THE STRUCTURE NEED STRENGTHENED BEFORE IT CAN BE USED. NEW AND INCREASED DEMANDS FROM THE TRANSPORTATION SECTOR CAN BE ANOTHER REASON FOR STRENGTHENING. IF A SITUATIONS SHOULD ARISE IT NEEDS TO BE DETERMINED WHETHER IT IS MORE EC TO STRENGTHEN THE EXISTING STRUCTURE OR TO REPLACE IT. THERE EXIST MANY DIFFE WAYS OF STRENGTHENING AN EXISTING CONCRETE STRUCTURE, SUCH AS SPRAYED CONCRETE, DIFFERENT TYPES OF CONCRETE OVERLAYS, PRE-TENSIONED CABLE THE OUTSIDE OF THE STRUCTURE, JUST TO MENTION A FEW. A STRENGTHEN M WAS USED QUITE EXTENSIVELY DURING THE MID 1970S IS STEEL PLATE BONDING, METHOD HAS GAINED RENAISSANCE THE LAST DECADE, BUT NOW AS FRP (F REINFORCED POLYMERS) PLATE BONDING. THIS TECHNIQUE MAY BE DEFINED AS IN WHICH A COMPOSITE PLATE OR SHEET OF RELATIVELY SMALL THICKNESS IS B WITH AN EPOXY ADHESIVE TO IN MOST CASES A CONCRETE STRUCTURE IN ORDER TO IMPROVE ITS STRUCTURAL BEHAVIOUR AND STRENGTH. THE SHEETS OR PLATE REQUIRE MUCH SPACE AND GIVE A COMPOSITE ACTION BETWEEN THE ADHER EXTENSIVE RESEARCH AND LABORATORY TESTING HAS BEEN CARRIED OUT ALL OVER WORLD AND AT MANY DIFFERENT LOCATIONS. THESE INVESTIGATIONS SHOW THAT THE METHOD IS VERY EFFECTIVE AND A CONSIDERABLE STRENGTHENING EFFECT ACHIEVED

AT LULEÅ UNIVERSITY OF TECHNOLOGY, RESEARCH IS TAKING PLACE IN THE F CFRP-STRENGTHENING, I.E., THE PROCESS OF STRENGTHENING CONCRETE MEMB BONDING CFRP (FIBRE REINFORCED POLYMER) PLATES OR SHEETS TO THEIR SURFACES. THE RESEARCH WORK STARTED IN 1988, THEN WITH STEEL PLATE TODAY CONTINUING WITH FRP MATERIALS. BOTH COMPREHENSIVE EXPERIMENTA THEORETICAL WORK HAS BEEN UNDERTAKEN. THE LABORATORY TESTS STRENGTHENING FOR BENDING AS WELL AS FOR SHEAR AND TORSION. IN SEVERAL FULL-SCALE TESTS HAVE BEEN PERFORMED. IN 1996 WORK WITH NSMR SURFACE MOUNTED REINFORCEMENT) STARTED. PILOT TESTS WERE PERFORMED A BENEFITS COMPARED TO TRADITIONAL PLATE BONDING WERE FOUND TO BE MANY.

IN THIS PART A BACKGROUND AND NEED TO THE USE OF NSMR IS DISCUSSED.

LABORATORY TESTS AND THEORY FOR CONCRETE BEAMS STRENGTHENED W
SURFACE MOUNTED CFRP REINFORCEMENT ARE BRIEFLY PRESENTED. IN ADDITION, THE
USE OF NSMR IN FIELD APPLICATIONS IS DISCUSSED AND A FIELD APPLICAT
PRESENTED.

5.3.2 Strengthening Concrete Structures in general

AS MOST OF US KNOW, CONCRETE IS A BUILDING MATERIAL WITH A HIGH COMPR
STRENGTH AND A POOR TENSILE STRENGTH. A BEAM WITHOUT ANY FO
REINFORCEMENT WILL CRACK AND FAIL WHEN SUBJECTED TO A RELATIVELY SMALL LOAD.
THE FAILURE OCCURS SUDDENLY IN MOST CASES AND IN A BRITTLE MANNE
MOSTCOMMON WAY TO REINFORCE A CONCRETE STRUCTURE IS TO USE
REINFORCING BARS THAT ARE PLACED IN THE STRUCTURE BEFORE THE CONCRETE IS CAS
SINCE A CONCRETE STRUCTURE USUALLY HAS A VERY LONG LIFE, IT IS QUITE COMMON
THAT THE DEMANDS ON THE STRUCTURE CHANGE WITH TIME. THE STRUCTU
HAVE TO CARRY LARGER LOADS AT A LATER DATE OR FULFILL NEW STANDARDS. IN EX
CASES, A STRUCTURE MAY NEED TO BE REPAIRED DUE TO AN ACCIDENT. A
REASON CAN BE THAT ERRORS HAVE BEEN MADE DURING THE DESIGN OR CONS
PHASE SO THAT THE STRUCTURE NEEDS TO BE STRENGTHENED BEFORE IT CAN B
ANY OF THESE SITUATIONS SHOULD ARISE IT NEEDS TO BE DETERMINED WHETHER IT IS
MORE ECONOMICAL TO STRENGTHEN THE STRUCTURE OR TO REPLACE IT. IT SHOULD ALSO
BE REMEMBERED THAT OVER THE PAST DECADE, THE ISSUE OF DETERIO
INFRASTRUCTURE HAS BECOME A TOPIC OF CRITICAL IMPORTANCE IN EUROPE
EQUAL EXTENT IN THE UNITED STATES AND JAPAN. THE DETERIORATION OF
SUPERSTRUCTURE ELEMENTS AND COLUMNS CAN BE TRACED TO REASONS RA
AGEING AND ENVIRONMENTALLY INDUCED DEGRADATION TO POOR INITIAL CONS
AND LACK OF MAINTENANCE. ADDED TO THE PROBLEMS OF DETERIORATION, ARE T
ISSUES RELATED TO THE NEEDS FOR HIGHER LOAD RATINGS AND INCREASED NUMBER
OF LANES TO ACCOMMODATE THE EVER-INCREASING TRAFFIC FLOW ON TH
ARTERIES. AS AN OVERALL RESULT, A SIGNIFICANT PORTION OF OUR INFRASTRU
CURRENTLY EITHER STRUCTURALLY OR FUNCTIONALLY DEFICIENT. BEYOND TH
VISIBLE CONSEQUENCES ASSOCIATED WITH CONTINUOUS RETROFIT AND REPA
STRUCTURAL COMPONENTS, ARE THE REAL CONSEQUENCES RELATED TO
PRODUCTION AND OVERALL ECONOMIES RELATED TO TIME AND RESOURCES CAUSED
DELAYS AND DETOURS. AS WE MOVE INTO THE TWENTY-FIRST CENTURY, THE RENEWAL OF
OUR LIFELINES BECOMES A CRITICAL ISSUE.

HOWEVER, TO KEEP A STRUCTURE AT THE SAME PERFORMANCE LEVEL IT N
BE MAINTAINED AT PREDESTINED TIME INTERVALS. IF LACK OF MAINTENANCE H
LOWERED THE PERFORMANCE LEVEL OF THE STRUCTURE, NEED FOR REPAI
ORIGINAL PERFORMANCE LEVEL CAN BE REQUIRED. IN CASES WHEN H
PERFORMANCE LEVELS ARE NEEDED, UPGRADING CAN BE NECESSARY. PERFORMAN
LEVEL MEANS LOAD CARRYING CAPACITY, DURABILITY, FUNCTION OR AE
APPEARANCE. UPGRADING REFERS TO STRENGTHENING, INCREASED DURABIL

CHANGE OF FUNCTION OR IMPROVED AESTHETIC APPEARANCE. IN THIS BOOK, STRENGTHENING IS DISCUSSED.

Restoration, reparation and reinforcement of old concrete structures are becoming increasingly common. If one considers the capital that has been invested in existing infrastructures, then it is not always economically viable to replace an existing structure with a new one. The challenge must be taken to develop relatively simple measures such as rebuilding, restoration, reparation and reinforcement that can be used to prolong the life of structures. An example of reinforcement would be strengthening an existing structure to carry greater loads. This places a great demand on both consultants and contractors. There are difficulties in assessing the most suitable method for an actual subject; as for example, two identical columns within the same structure can have totally different life spans depending on their individual microclimate. It is therefore important to analyse the problem thoroughly to be able to select the correct measure. The choice of an unsuitable reparation method can even deteriorate the structure's function. In the cases where reparation is appropriate, the intention should be to increase durability or load-bearing capacity. In comparison to building a new structure, strengthening an existing one is often more complicated since the structural conditions are already set. It can also be a problem to reach the areas that need to be strengthened. This is generally the case for traditional methods such as for example different kinds of reinforced overlays, shotcrete or post tensioned cables placed on the outside of the structure which normally need much space.

In recent years the development of the plate bonding repair technique has shown to be applicable to many existing strengthening problems in the building industry, not only for strengthening but also in cases of rebuilding and when mistakes have been made in the design or construction phase. This technique may be defined as one in which composite sheets or plates of relatively small thickness are bonded with an epoxy adhesive to, in most cases, a concrete structure to improve its structural behaviour and strength. The sheets or plates do not require much space and give a composite action between the adherents. The adhesive that is used to bond the fabric or the laminate to the concrete surface is a hardy two-component epoxy adhesive, which together with the fibre then becomes a plastic composite on the surface of the structure. The old structure and the new bonded material create a structural relationship that has a greater strength than the original structure.

THE QUESTION MUST BE ASKED WHY ADVANCED COMPOSITES ARE SUITABLE FO ENGINEERINGAPPLICATIONS. FIBRE REINFORCED POLYMER MATRIX COMPO MATERIALS HAVE A NUMBER OF ADVANTAGES WHEN COMPARED TO TRADI CONSTRUCTION MATERIALS SUCH AS STEEL, WOOD AND CONCRETE. FIBRE R POLYMERS (FRPS), OFFER EXCELLENT CORROSION RESISTANCE TO ENVIRONMEN

AS WELL AS THE ADVANTAGES OF HIGH STIFFNESS-TO-WEIGHT AND STRENGTH-TO-WEIG RATIOS WHEN COMPARED TO CONVENTIONAL CONSTRUCTION MATERIALS. OTHER ADVANTAGES OF FRPS INCLUDE LOW THERMAL EXPANSION, GOOD FAT PERFORMANCE AND DAMAGE TOLERANCE, NON-MAGNETIC PROPERTIES, THE EASE TRANSPORTATION AND HANDLING, LOW ENERGY CONSUMPTION DURING FABRI RAW MATERIAL AND STRUCTURE, AND THE POTENTIAL FOR REAL TIME MONITORI

PERHAPS THE BIGGEST ADVANTAGE OF FRPS IS TAILORABILITY. REINFORCEMENT ARRANGED ACCORDING TO THE LOADING CONDITIONS SO THAT A FRP STRUCT COMPONENT CAN BE OPTIMISED FOR PERFORMANCE. APPARENTLY THE HIGH COST OF FRPS COMPARED TO CONVENTIONAL MATERIALS HAS BEEN A MAJOR UNFAVO RESTRAINT. HOWEVER, A DIRECT COMPARISON OF THE UNIT PRICE BASIS MAY NO APPROPRIATE. WHEN INSTALLATION IS INCLUDED IN THE COST COMPARISON, FRPS CAN BE COMPETITIVE WITH CONVENTIONAL MATERIALS. THE LIGHT WEIGHT OF FRPS REDUCES TRANSPORTATION EXPENSES AND ALLOWS SOME PREFABRICATION TO TAKE PL FACTORY, WHICH REDUCES TIME AT THE JOB SITE. IF THE COMPARISONS INC CYCLE COSTS, FRPS CAN HAVE SIGNIFICANT ADVANTAGE.
THE UNIQUE PROPERTIES OF FRPS, LIKE HIGH CORROSION RESISTANCE, MAKE T CYCLE COST LOWER THAN THAT OF CONVENTIONAL MATERIALS. IN MANY CASES A COMPOSITE STRUCTURE CAN LAST MUCH LONGER THAN CONVENTIONAL MATERIALS, THUS ENSURING A LOWER LIFE-CYCLE COST IN MANY CASES. ALSO, INCREASING DEMA WILL DRIVE DOWN THE COST OF FRP. THE INTRODUCTION OF FIBRE REINFORCED IN CIVIL ENGINEERING STRUCTURES HAS PROGRESSED AT A VERY RAPID RATE YEARS.

THE BASIC IDEAS RELATED TO THE USE OF FRPS FOR STRUCTURAL STREN ALONG WITH EXAMPLES OF APPLICATION, HAVE BEEN PRESENTED BY TRIANTA (1998). THE PAST AND POTENTIAL FUTURE USE OF FRP STRENGTHENING REHABILITATION HAVE ALSO RECENTLY BEEN DOCUMENTED IN MANY CONF PROCEEDINGS (MEIER AND BETTI, 1997, TÄLJSTEN, 1997, BENMOKRANE AN RAHMAN, 1998), KEYNOTE LECTURES (MARUYAMA, 1997, NEALE AND LABOSSIÉRE, 1997) AND JOURNAL ARTICLES (THOMAS, 1998).

The most common way to strengthen structures has been for bending but shear strengthening is also often needed. The most common method has been to place sheets or laminates on the surface of the structure, however, further development of the plate bonding method has shown that it is favourable to place the laminates in the concrete cover of the structure. This method can be designated NSMR or Near Surface Mounted Reinforcement.

5.3.3 NSMR - A SHORT INTRODUCTION

THE USE OF NEAR SURFACE MOUNTED REINFORCEMENT FOR CONCRETE STRU

NOT A NEW INVENTION. A TYPE OF NSMR HAS BEEN USED SINCE THE 1940S, WHERE STEEL REINFORCEMENT WAS PLACED IN SLOTS IN THE CONCRETE COVER OR IN ADDITIONAL CONCRETE COVER THAT WAS CAST TO THE STRUCTURE. A TYP NSMR HAS BEEN USED SINCE THE INVENTION OF SHOTCRETE. HOWEVER, IN T APPLICATIONS IT WAS OFTEN DIFFICULT TO GET GOOD BOND TO THE ORIGINAL STRUCTURES SURFACE, AND IN SOME CASES, IT WAS NOT ALWAYS EASY TO CAST CONCRETE THE AROUND THE WHOLE STEEL REINFORCING BARS. FROM THE 1960S THE DEVELO STRONG ADHESIVES, SUCH AS EPOXIES, FOR THE CONSTRUCTION INDUSTRY MO METHOD FURTHER AHEAD BY BONDING THE STEEL BARS IN SAWED SLOTS CONCRETE COVER. HOWEVER, DUE TO THE CORROSION SENSITIVITY OF STEE ADDITIONAL CONCRETE COVER WAS NEEDED. FOR THESE APPLICATIONS EPOXY CO STEEL BARS HAVE ALSO BEEN USED, HOWEVER, IT HAS BEEN SHOWN OVER TIME THAT EPOXY COATED STEEL BARS ARE NOT ALWAYS CORROSION RESISTANT FO REASONS THAT NOT WILL BE DISCUSSED HERE. THE USE OF STEEL NSMR HA BEEN A GREAT SUCCESS. NEVERTHELESS, BY USING CFRP NSMR SOME OF THE DRAWBACKS PRESENTED CAN BE OVERCOME. FIRST OF ALL, CFRP NSMR DOES NOT CORRODE, SO THICK CONCRETE COVERS ARE NOT NEEDED. SECONDLY, THE LAMINATE CAN BE TAILOR-MADE FOR NEAR SURFACE APPLICATIONS, THIRD LIGHTWEIGHT OF THE CFRP LAMINATES MAKES THEM EASY TO MOUNT. FURTHER DEPENDING ON THE FORM OF THE LAMINATE AIR VOIDS BEHIND THE LAMINATES CAN BE AVOIDED. BOTH EPOXIES AND SYSTEMS USING HIGH QUALITY CEMENT MORTAR BE USED. HOWEVER, BEFORE PROCEEDING, A SHORT DESCRIPTION OF HOW UNDERTAKE A STRENGTHENING WORK WITH NSMR WILL BE GIVEN. IN PRACTICAL EXECUTION THE FOLLOWING

STEPS MUST IN GENERAL BE PERFORMED DURING STRENGTHENING:

- SAWING UP SLOTS IN THE CONCRETE COVER, DEPTH DEPENDING ON PRODU USED AND DEPTH OF CONCRETE COVER.
- CAREFUL CLEANING OF THE SLOTS AFTER SAWING, HIGH-PRESSURISED WAT APPROXIMATELY 100 - 150 BARS, IS RECOMMENDED. NO SAW MUD ALLOWED THE SLOT
- IF AN EPOXY SYSTEM IS USED, THE SLOT MUST BE DRY BEFORE BONDING. IF CEMENT SYSTEM IS USED, IT IS MOSTLY RECOMMENDED THAT THE EXISTING SURFACES ARE WET AT THE TIME OF CONCRETE MORTAR CASTING.
- ADHESIVE IS APPLIED IN THE SLOT OR WITH A CEMENT SYSTEM; CEMENT MO APPLIED IN THE SLOT.

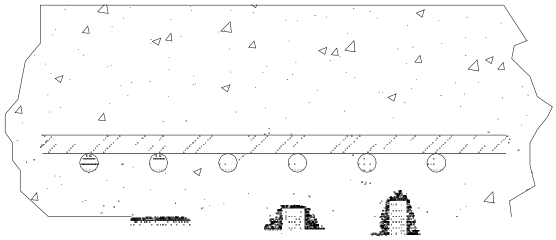

Plate Bonding NSMR

FIG. 5.5. COMPARISON BETWEEN LAMINATE PLATE BONDING AND NSMR

THE NSMR LAMINATES ARE MOUNTED IN THE SLOT AND THE EXCESS ADHESIV CEMENT MORTAR IS REMOVED WITH A SPATULA OR SIMILAR. IT IS INTERESTING TO COMPARE TRADITIONAL LAMINATE AND SHEET PLATE BONDING WITH NSMR, AND THIS IS DONE IN FIGURE 5.5 AND TABLE 5.6. IN FIGURE 5.5, THE DIFFERENCE BETWE LAMINATES AND NSMR CAN BE SEEN. THE FRACTURE ENERGY TO REMOVE NSMR MANY CASES MUCH LARGER THAN FOR BONDED LAMINATES.

FURTHERMORE, NSMR RESISTS END PEELING MUCH BETTER THAN BONDED LAM AND THEY ARE CONSIDERABLY MORE PROTECTED AGAINST FIRE, VANDALISM AND IM FROM E.G. VEHICLES. HOWEVER, IN SOME APPLICATIONS IT DEMANDS A GREATER EFFORT TO CARRY OUT THE WORK ON SITE. AN OVERVIEW OF THE CHARACTERISTICS AND SOME TYPICAL ASPECTS OF THESE THREE TY STRENGTHENING METHODS WITH FRP ARE IN TABLE 5.6, SEE FIB BULLETIN 14, 2001, TÄLJSTEN 2002.

TABLE 5.6 CHARACTERISTICS AND ASPECTS OF EXTERNALLY BONDED FRP REINFORC

	Laminates	**Sheets**	**NSMR**
Shape	Rectangular strips	Thin unidirectional or bi-directional fabrics	Rectangular strips or laminates
Dimension: thickness width	 Ca: 1.0 - 2.0 mm Ca: 50 - 150 mm	 Ca: 0.1 - 0.5 mm Ca: 200 - 600 mm	 Ca: 1.0 - 10.0 mm Ca: 10 - 30 mm
Use	Simple bonding of factory-made profiles with adhesives	Bonding and impregnation of the dry fibre with resin and curing at site	Simple bonding of factory made profiles with adhesive or cement mortar in pre-sawed slots in the concrete cover

Application aspects	For flat surfaces Thixotopic adhesive for bonding Not more than one layer recommended Stiffness of laminate and use of thixotropic adhesive allow for certain surface unevenness Simple in use Quality guaranteed from factory Suitable for strengthening in bending Needs to be protected against fire	Easy to apply on curved surfaces Low viscosity resin form bonding and impregnation Multiple layers can be used, more than 10 possible. Unevenness needs to be levelled out Need well documented quality systems Can easily be combined with finishing systems, such as plaster and paint Suitable for shear strengthening Needs to be protected against fire	For flat surfaces Depends on the distance to steel reinforcement A slot needs to be sawn up in the concrete cover The slot needs careful cleaning before bonding Bonded with a thixotropic adhesive Possible to use cement mortar for bonding Protected against impact and vandalism Suitable for strengthening in bending Minor protection against fire

5.3.4 Field applications

The field-application presented here was carried out during the fall of 1999. The reason for strengthening was a mistake at the construction site. The amount of steel reinforcement needed in a joint between a pre-cast concrete element and on site cast concrete was not sufficient and strengthening was demanded. The reason for choosing NSMR for this application was the CFRP laminates resistance to corrosion and stiffness and strength comparable to steel. The cross section of the laminates used was 5 x 35 mm, with a Young´s modulus of 160 GPa and an ultimate strain at failure corresponding to 1.6 %. Nevertheless, the main factor to the NSMR advantage was that the laminate could be placed in the concrete cover and that no great work effort was needed. However, laminate Plate Bonding was also discussed, but since the bridge has a long life and the sealing on the bridge deck is replaced every 20 years the risk of ripping the laminate off the surface was estimated to be very likely. Another reason was the wearing surface in form of warm asphalt that was going to be applied.

First the slots (40 x 8 mm) were sawn up in the concrete cover. The slots were cleaned carefully and allowed to dry before the adhesive was applied. The strengthening system chosen was BPE® NSMR. In figure 5.6a, the laminates and the slots in the concrete cover can be seen prior to bonding. In figure 5.6b the laminates are bonded in the slot and in figure 5.6 c the result after strengthening but before the surface is asphalted is shown.

a) Slots in front of the worker holding a NSMR laminate.
b) Bonding the NSMR laminates in the cleaned and dry slots
c) Results after bonding but before sealant and asphalt

FIG. 5.6. STRENGTHENING OF A CONCRETE JOINT WITH NSMR

THE CLIENT, THE SWEDISH ROAD AUTHORITY, CONSIDERED THE STRENGTHENIN SUCCESSFUL AND IT IS TODAY AN ACCEPTED METHOD OF STRENGTHENING C BRIDGES.

5.3.5 FUTURE WORK

SEVERAL RESEARCH PROGRAMS ARE ONGOING AT LULEÅ UNIVERSITY OF TECHN DIVISION OF STRUCTURAL ENGINEERING. A PROJECT THAT IS ALMOST FINISHED I "BEHAVIOR OF PLATE BONDED CONCRETE BEAMS IN COLD CLIMATE", WHERE CFRP STRENGTHENED CONCRETE BEAMS, BOTH LAMINATES AND NSMR, HAVE BEEN T AT - 30 ℃ AND COMPARED WITH BEAMS TESTED IN ROOM TEMPERATURE. THE PRE RESULTS FROM THESE TESTS ARE THAT NO DECREASE IN LOAD COULD BE NOTICED FOR BEAMS TESTED IN COLD CLIMATE. RESULTS FROM THESE TESTS WILL BE REPORTED DURING SPRING 2002, FOCUS WILL BE PLACED ON NSMR APPLICATIONS. FIR ANCHORAGE WILL BE STUDIED, SECONDLY THE POSSIBILITY TO USE PRE-STRESSED NS WILL BE INVESTIGATED FURTHER AND THIRDLY A FULL-SCALE APPLICATION TO STRENGTHEN A ROAD BRIDGE FOR THE SWEDISH ROAD AUTHORITIES WILL BE CARRIED OUT. IN ADDITION

ONGOING RESEARCH REGARDING SHEAR AND TORSION STRENGTHENING WILL BE CO

5.4 Strengthening Techniques of Reinforced Concrete Columns Using Fiber Reinforced Polymeric Materials

5.4.1 Why need to strengthening columns?

Fiber reinforced composite materials are becoming more frequently used in civil engineering structures. One of the most practical applications of these new materials concerns the strengthening of reinforced concrete columns by means of confinement with fibre composite sheets. In the literature, various theoretical models have been proposed to describe the behaviour of confined concrete columns. The principal advantages of this technique are the high strength-to-weight ratio, good fatigue properties, non-corroding characteristics of the fibre reinforced polymers (FRP), and the facility of its application. The maximum efficiency of confining systems using FRP materials is reached in case of columns with circular cross-section and is explained by the fact that the entire section of the column is involved into the confinement effect. Rectangular confining reinforcement is less efficient as the confinement action is mostly located at the corners This paper reveals the most utilized techniques of performing composite confining systems for reinforced concrete columns, with their advantages and also disadvantages.

5.4.2. Generic Information :-

Over the years, engineers have used different methods and techniques to retrofit existing structures by providing external confining stresses. For the past few years, the concept of jacketing has been investigated to provide such forces. Externally applied jackets have been used as a reinforcement to contain concrete for different reasons. Engineers have used traditional materials such as wood, steel, and concrete to confine and improve the structural behavior of concrete members [7].

Section enlargement is one of the methods used in retrofitting concrete members. Enlargement is the placement of reinforced concrete jacket around the existing structural member to achieve the desired section properties and performance. The main disadvantages of such system are the increase in the column size obtained after the jacket is constructed and the need to construct a new formwork.

Steel jacketing has been proven to be an effective technique to enhance the seismic performance of old bridge columns. The steel jacket is manufactured in two shell pieces and welded in the field around the column.

However, this method requires difficult welding work and, in a long term, the potential problem of corrosion remains unsolved [7].
Interesting in the use of flexible fiber reinforced plastic (FRP) sheets for the external wrapping of concrete compressed members is today a very popular theme, especially as regards estimating the effectiveness of this reinforcing technique in increasing the strength and ductility of members in seismic areas [1].
Several advantages are observed in using FRP wraps compared to the most common other techniques based on the use of steel reinforcements such as: the high-mechanical properties of the material (tensile strength and elasticity modulus) compared with its lightness; its insensitivity to corrosion; the ease of applying the reinforcing material, etc. [1].

5.4.3. FRP Confining Systems in Case of Reinforced Concrete Columns

Confinement is generally applied to members in compression, with the aim of enhancing their load carrying capacity or, in cases of seismic upgrading, to increase their ductility. Advanced FRP composite materials have only recently been recognized as reliable confinement devices for reinforced concrete elements. FRP, as opposed to steel that applies a constant confining pressure after yield, has an elastic behavior up to failure and therefore exerts its (passive) confining action on concrete specimens under axial load in a different way with respect to steel. In Fig. 5.7 it can be seen that, at a certain value of the normalized axial concrete strain, the steel reaches yielding and then, from that point on, it exerts a constant lateral (confining) pressure, while FRP exerts a continuously increasing confining action [3], [8].

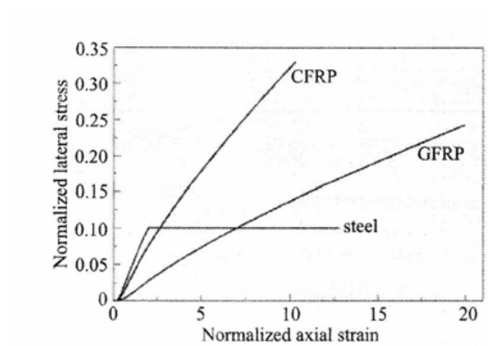

Fig. 5.7 – Comparison of confinement actions of steel and FRP materials.

Utilizing composite systems for confinement of concrete, a transfer of tensile stresses from concrete to the composite system is realized. The confining system changes the characteristic loading – strain behavior of concrete applying it a confining pressure. The maximum efficiency of confining systems using FRP materials is reached in case of columns with circular cross-section and is explained by the fact that the entire section of the column is involved into the confinement effect. The confining pressure is uniformly distributed on the entire cross-section of the element. In case of columns with noncircular cross-section only a part of it is subjected to the confining effect, and that part is

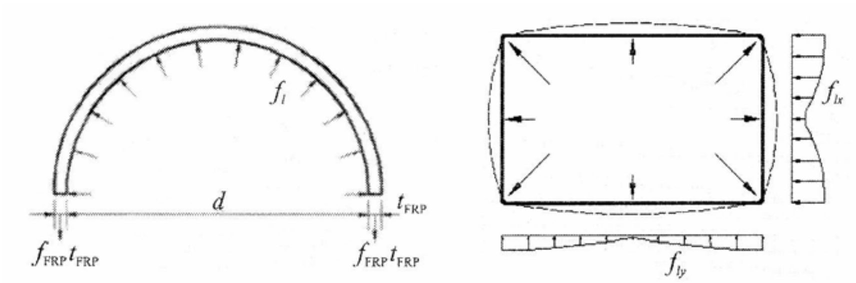

Fig. 5.8 – Distribution of confining pressure in case of columns with circular and rectangular

cross-section; fl – the confining pressure; fFRP – FRP longitudinal tensile strength; d – the diameter of the RC column cross-section; tFRP – thickness of the composite material; flx,y – the confining pressures given by x- and y-directions.

The main objectives of confinement are: (i) to prevent the concrete cover from spalling, (ii) to provide lateral support to the longitudinal reinforcement and (iii) to enhance concrete strength and deformation capacities. In the case of circular columns these goals can be achieved by applying external FRP jackets, either continuously all over the surface or discontinuously as strips. In the case of rectangular columns, the confinement can be provided with rectangular-shaped reinforcement, with corners rounded before application (the

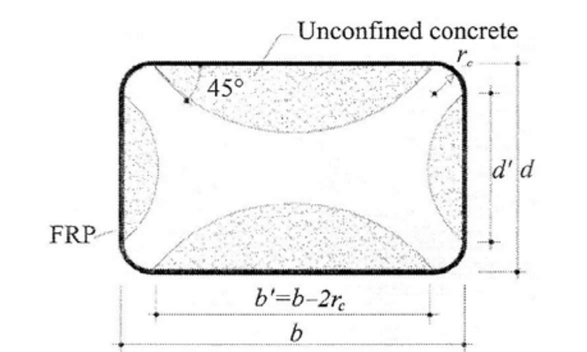

Fig. 5.9 – Effectively confined core for non-circular sections.

radius is about 15…25 mm, depending on the specifications given by the FRP jacket supplier). Rectangular confining reinforcement is less efficient as the confinement action is mostly located at the corners and a significant jacket thickness needs to be used between corners to restrain lateral dilation and column bar buckling. An alternative approach is to enclose the rectangular column within an externally cast circular or oval shape that provides the appropriate shape for the jacket (Fig.5.9) [8].

5.4.4. Confining Techniques

Today there are several types of FRP strengthening systems namely

a) wet lay-up systems;

b) systems based on prefabricated elements;

c) special automated wrapping systems.

a) Wet lay-up process represents the most commonly used technique, in which unidirectional fibre sheets or woven fabric sheets are impregnated with resins and wrapped around columns, with the main fibres oriented in the hoop direction [2]. Installation on the concrete surface requires saturating resin, usually after a primer has been applied. Two different processes can be used to apply the fabric [9]: (i) the fabric can be applied directly into the resin which has been applied uniformly onto the concrete surface, (ii) the fabric can be impregnated with the resin in a saturator machine and then applied wet to the sealed substrate.

The wrapping can be realized continuously around the entire element or partially, using sheets of FRP disposed in spiral or in distinct sections. There can be applied variable number of layers (from same material or distinct ones), obtaining different thicknesses of the confining layer, depending on the required element strength (Fig. 5.10) [4].

Fig. 5.10 – Wet lay-up confining system.

b) When prefabricated FRP jackets are used, the jackets are fabricated in half circles or half rectangles and circles with a slit or in continuous rolls, so that they can be opened up and placed around columns (Fig. 5.11). This can be considered as technical most elaborated system, but the major problems emerge in the closure area of the composite layer because of insufficient overlapping [2].

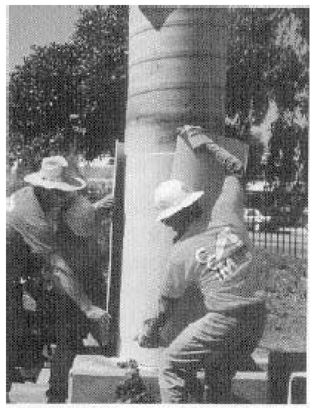

Fig. 5.11 – Confining system based on prefabricated FRP elements.

c) The FRP automated wrapping technique through winding of tow or tape was first developed in Japan in the early 90s and a little later in the USA. The technique, shown in Fig. 5.12, involves continuous winding of wet fibres under a slight angle around columns by means of a robot. Key advantage of the technique, apart from good quality control, is the rapid installation [9].

Fig. 5.12 – Automated RC column wrapping.

Choosing the confining technique depends on the following factors: a) the importance and the volume of the strengthening work;

b) the type of composite materials used;

c) the working gang experience;

d) working conditions and the period of time available to perform the job;

e) the financial availability.

Table 5.7 presents the main advantages and disadvantages of the three execution methods for confining systems with composite materials [6].

Disadvantages	Advantages	
a) reduced quality control; b) high volume of work on site.	a) workability to a large type and shapes of columns; b) easy to handle.	Wet lay-up method
a) workability to a reduced shapes of columns; b) special equipments required.	a) high quality control; b) reduced volume of work on site.	Automated method
a) workability to a reduced shapes of columns; b) high costs.	a) the best quality control; b) reduced volume of work on site.	Confining with prefabricated elements

Table 5.7 Advantages and Disadvantages of the Methods Used for Confining withComposite Systems

These systems correspond to several manufacturers and suppliers and are based on different configurations, types of fibres, adhesives, etc. Practical execution and application conditions, for example cleaness 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°C to start. If the temperature drops, the hardening process delays.

5.5 Summary and Conclusion

There is no doubt that strengthening concrete structures with NSMR is an effective method. Compared to laminate plate bonding several benefits can be noticed, not only higher fracture energy at failure but also better protection against fire, vandalism and impact.
The pilot tests presented in this paper show promising strengthening results and a considerable strengthening effect could be noticed. Theory presented covers traditional design for bending, however, more work is needed to also cover anchorage and other types of strengthening applications.
The field application presented shows that it is easy to strengthen structures and the method is not only time saving but also beneficial from a financial point of view.

The following conclusions can be drawn up for the flexible bonding system reported in this paper:

1. FRP/concrete interfaces using the flexible bonding system need much longer anchorage length to develop fully its interfacial bond force-transferring capacity. With a sufficient long anchorage length, flexible bonding adhesive can achieve higher pullout capacity. FRP sheet/concrete interfaces with normal and flexible bonding system show big differences in their local bond stress-slip behaviours. The flexible bonding system has lower interfacial bonding stiffness but can ease the local stress concentration and can bear large interfacial slip while avoiding local debonding.
2. Compared to the normal adhesive bonding, flexible bonding system can improve the ultimate capacity of FRP strengthened RC beams. So the high strength, an advantage of FRP, can be utilized more efficiently. Different from those in pullout tests, the failure modes of strengthened RC members using the flexible bonding system may shift from the debonding of FRP/concrete interface to the ripping of concrete covercrete. The latter mode is triggered by the yielding of steel reinforcement rather than the stress concentration at FRP/concrete interfaces. Therefore, the flexible bonding system contributes a ductile failure. But the improving effects of the flexible bonding system may be eliminated in cases of larger FRP amount and shorter shear span.

3. Contribution of the flexible bonding system to the ultimate capacity

of FRP strengthened RC beams is mainly after the yielding of steel reinforcement. Hence the flexible bonding system is appreciated for the ultimate state strengthening rather than the serviceability strengthening purpose.

4. The strengthened RC members using flexible bonding system have an equal or better fatigue performance than those using normal adhesive bonding. Compared to the normal bonding system, the flexible bonding system contributes a smooth degradation of member stiffness although it leads to a comparatively low initial stiffness.
5. The flexible bonding system seems particularly favourable for the RC members with larger span. The strengthened FRP/RC composite system can keep its integrity up to a ductile failure and gain high strength efficiency in FRP.

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Case Studies

Case (A): Strengthening of Reinforced Concrete Columns Using Reinforcement Concrete Jackets:-

A-1 Project

- construction of Special clinics for children hospital (ain Shams university)

A-2Description of Building:-

The building consist of Four floors. (Ground – First – Second – third – Fourth)

The statically system is (solid slab).

The building are supported on reinforced concrete columns. (Skeleton System)

The columns are in good condition with respect to (visual inspection – Shmedit Hammer test – core test)

Fig.A-1view for the building

Fig. A-2 Photos of columns before strengthening

A-3 Problems

1-The Owner wanted to increase new floor which lead to increase load on the columns and footing .

2-By Redesign and recalculate the building finding (some columns are unsafe due to new loads .

3- finding corrosion in some columns

A-4 Solution

1- we need to reduce the new load which come to columns by made the new floor is (steel material) .

2-we need to increase capacity for some columns which are unsafe

By concrete jacketing .

Fig. A-3 Photos of columns Jacketing

Fig.A-3 Photos of columns Jacketing

Fig. A-4 Photos of column crack repair.

A-5 Method Of Application

1-Adding stirrups every (30- 50) c.m

2- remove concrete cover

3-remove corrosion from steel by wire brush .

4-paint the surface of steel bars by epoxy to protect bars from corrosion.

5- Adding sand or grout to steel surface .

6-Adding Shear Dowels to improve bonding between old and new concrete .

7- adding new reinforcement bars in two direction .

8- placement new concrete around columns .

Case (B): Strengthening of Reinforced Concrete Columns Using Steel Plate Jackets:-

B-1 Project

- construction of Office building of Kuaitian office (Elmaarag city – Elmaadi – Cairo)

B-2 Description of Building:-

The building consist of Four floors. (Basement – Ground – Four typical floor)

The statically system is (solid slab).

The building are supported on reinforced concrete columns. (Skeleton System)

Fig. B-1 view for the building

(Elmaarag city)

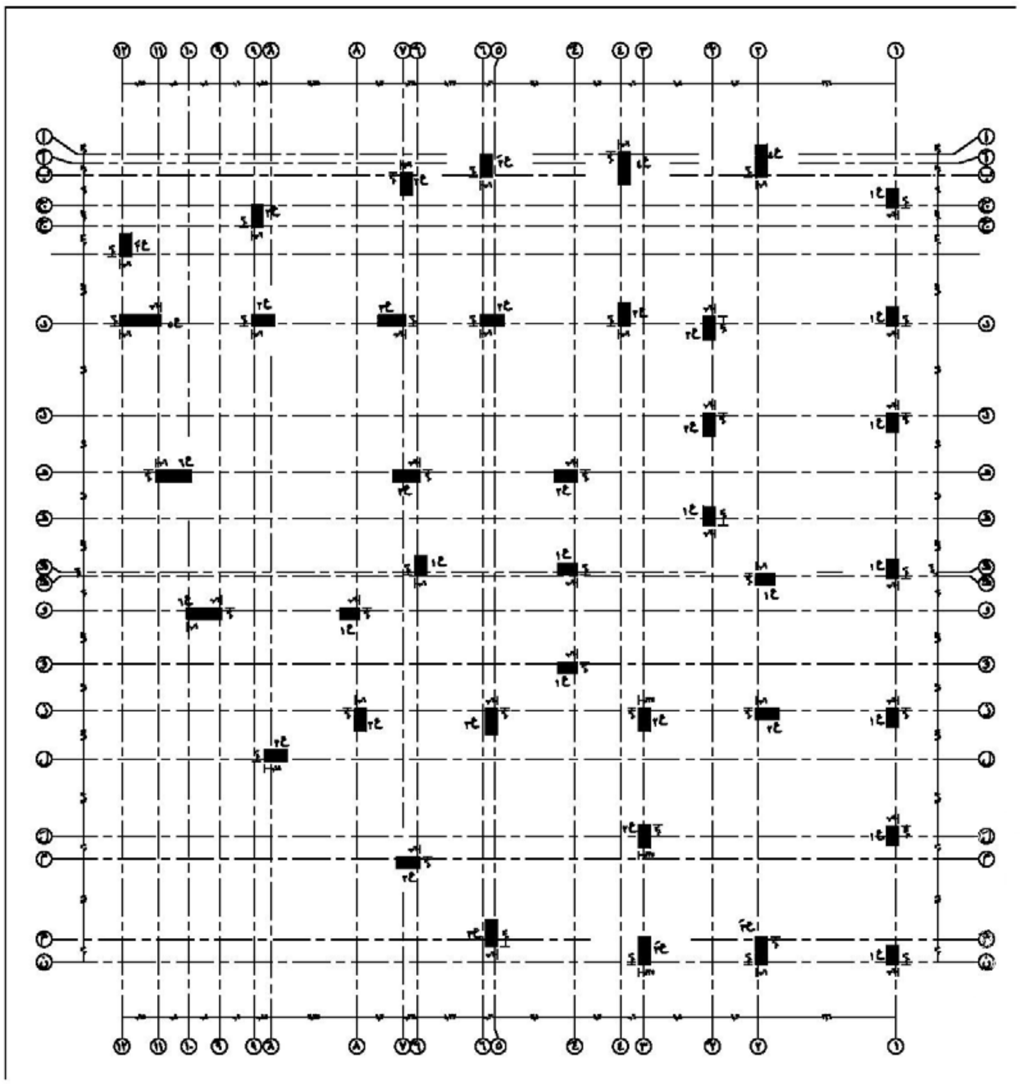

Fig. B-2 Columns & Axis plan

Fig. B-3 Column Photos before strngthening

B-3 Problems

1-The Owner wanted to increase new floor and change the using of building from residential building to office building which lead to increase load on the columns and footing .

2-By Redesign and recalculate the building finding (some columns are unsafe due to new loads .

3- The columns are not in good condition with respect to (visual inspection – Shmedit Hammer test – core test)

4-Average strength for concrete columns = 145 k.g / cm2

Fig. B-4 Core specimens

B-4 Technical Report for building:-

1-Visual inspection for all building.

2-take column dimensions for all columns in all floors.

3-indicate the upper level for reinforced concrete of footings and smells to measure the height of basement columns.

4-Shmedit hammer test for columns.

5-aplly core test to check for actual strength of the columns.

B-5 Solution:-

2-we need to increase capacity for some columns which are unsafe

By steel jacketing .

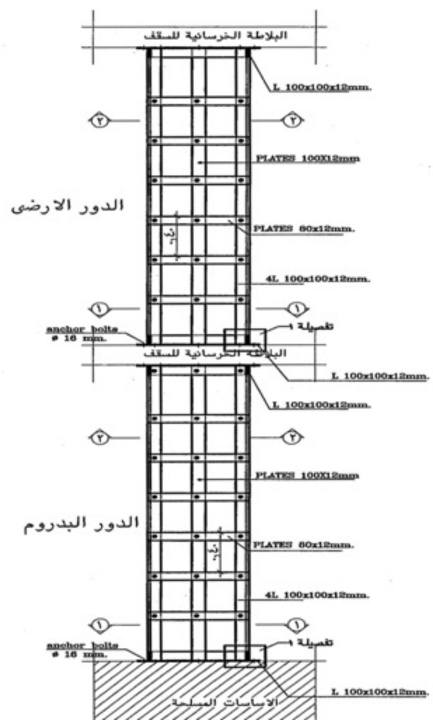

Fig. B.5 column jacket with steel plates

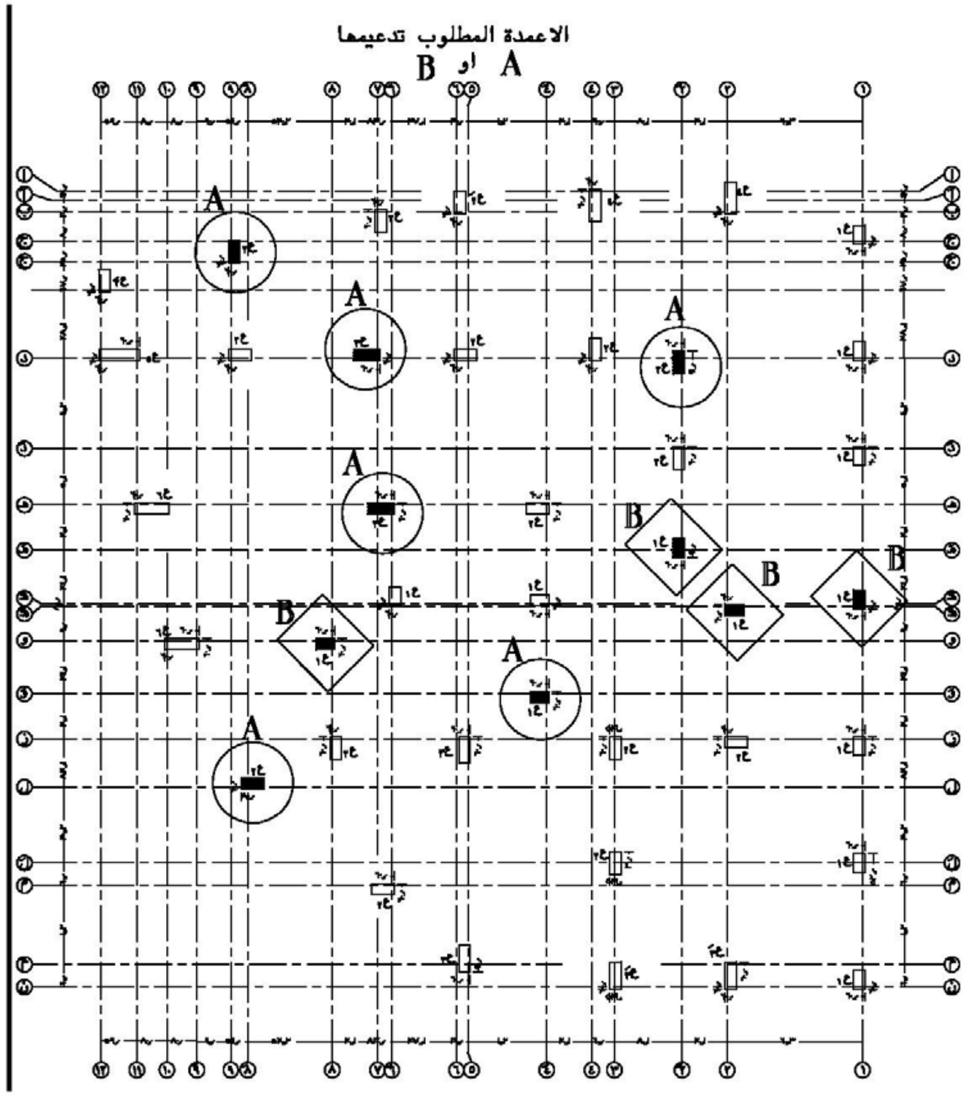

Fig. B- 6 Plan Column need to strengthening

B-6 Method of Application:-

1-Put angles in corner of columns

2-put steel plates at equal spacing for the column

3- cover the column by steel mesh .

4- placement concrete cover .

Fig. B-6 column steel jackets

Fig. B-6 column steel jackets.

Fig. B-7 Permanent Fixation by steel angle to carry new load

Fig. B-8 placement of concrete cover

Fig. B- 9 Photos of columns After finish placement of concrete

Case (C): Strengthening of Reinforced Concrete Slabs Using F.R.P Laminates:-

C-1 Project

- construction of National bank of Egypt (Elmaadi – Cairo)

C-2 Description of Building:-

The building consist of Four floors.

The statically system is (solid slab).

The building are supported on reinforced concrete columns. (Skeleton System)

Fig. C-1 view for the building

(National bank of Egypt)

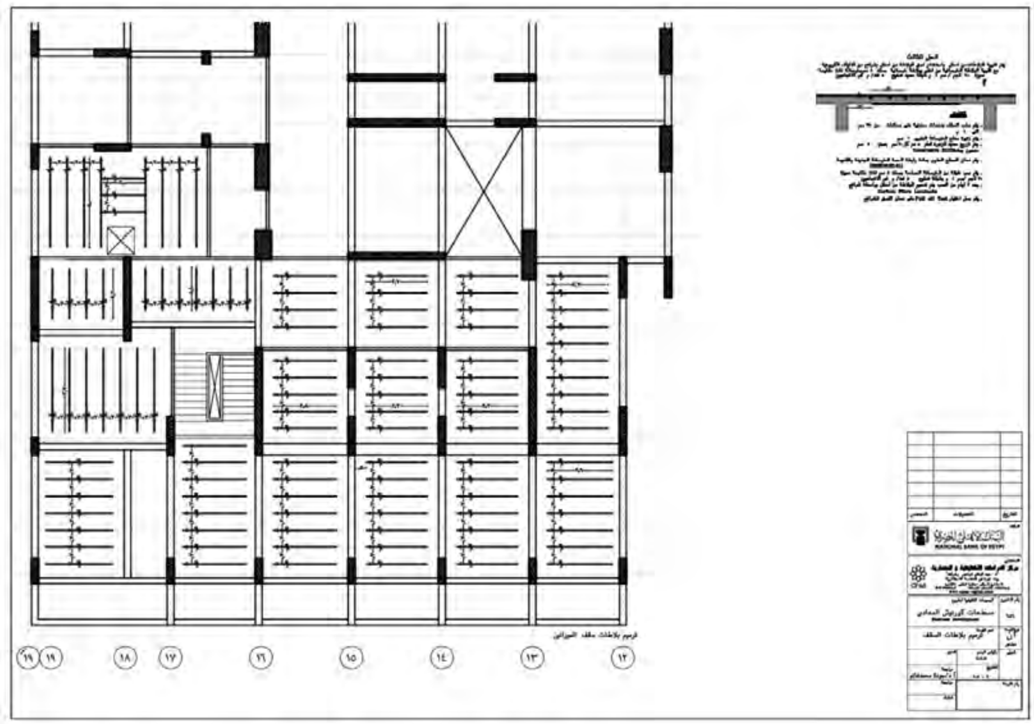

Fig. C-2 plan for slab need to strengthening

Fig. C-3 Slab before Strengthening

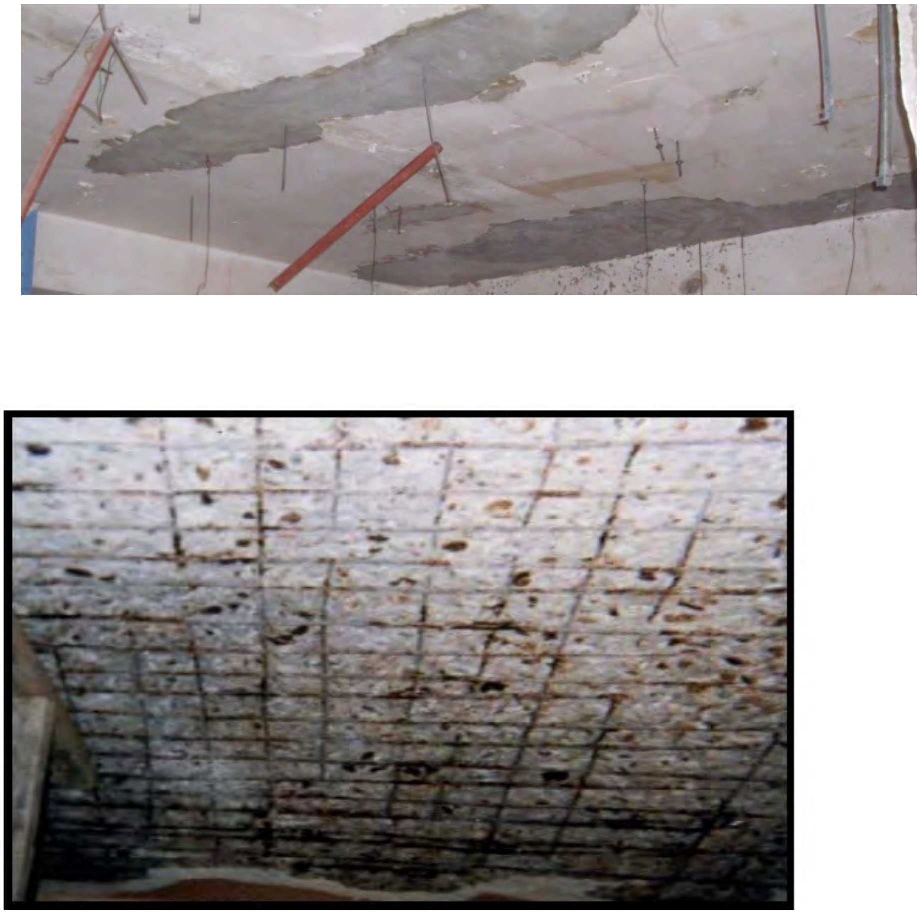

Fig. C-4 Concrete cover is fall from slab.

C-3 Problems

1-Finding high deformations in some spans of slabs .

2- finding Cracks in some slabs .

3- Concrete Cover is fall down from some span .

C-4 Technical Report for building:-

1-Visual inspection for all building.

2-take column dimensions for all columns in all floors.

3- Apply core test from slabs .

4-Shmedit hammer test for columns.

5-aplly core test to check for actual strength of the Slabs.

C-5 Solution:-

1-we need to crack injection.

2- we need to strengthening slab by CFRP material .

C-6 Method of Application

1-Transport the load of slab by using steel column .

2-Remove concrete cover and remove corrosion from steel bars by wire brush .

3- Paint the surface of steel by epoxy material to protect from corrosion .

4- Placement of concrete cover .

5- paint the new surface of concrete by epoxy materials to improve bonding between substrate concrete and CFRP material .

Fig. C-5 Crack injection for slab.

Fig. C- 6 Transform load from slab by Steel form

Fig. C-7 Transform load from slab by Steel form

Fig. C-8 Put CFRP to Strength Slab

Summary and Conclusion:-

1- FROM THE REVIEW OF THE PREVIOUS STUDIES, IT CAN BE CONCLUDED THAT EPOXY INJECTION IS AN EASY AND EFFECTIVE METHOD TO REPAIR THE CRACKED BEAMS. IT NOT ONLY REDUCES THE DEFLECTION AND CONTROL THE CRACKS BUT ALSO SLIGHTLY INCREASES THE WEIGHT OF BEAMS.

2- FURTHERMORE THIS METHOD IS INEXPENSIVE AND UNSOPHISTICATED. USING OPC MORTAR AND A FREE FLOWING MICRO-CONCRETE ARE TWO POSSIBLE METHODS TO REPAIR THE BEAMS UNDER CORROSION CONDITIONS. HOWEVER, FOR LONG TERM, A FREE FLOWING MICRO-CONCRETE CAN RESTORE HIGHER STRUCTURAL CAPACITY AND RESIST CHLORIDE BETTER UNDER ACCELERATED WEATHER CONDITIONS COMPARED TO USING OPC MORTAR.

3-THIS RESEARCH ANALYZED THE VARIOUS REINFORCED CONCRETE STRENGTHENING AND REPAIRING METHODS FOR REINFORCED CONCRETE STRUCTURES.

4-THE EXPERIMENTAL RESULTS THAT ARE SUMMARIZED FROM LITERATURE REVIEW WERE USED TO CONCLUDE THE PROS AND CONS OF CURRENT STRENGTHENING AND REPAIR METHODS.

5- THE SUGGESTIONS ABOUT RETROFIT AND REHABILITATION METHODS ARE PROVIDED IN THE THESIS FOR FUTURE WORK AND STUDY. THE PROBLEMS

AND EXPERIENCES REPORTED IN THIS RESEARCH WILL BE USED TO IMPROVE THE FUTURE REPAIR AND STRENGTHENING RESEARCH.

APPENDIX

عمل تقرير عن حالة المبنى :- (مشروع المقر الرئيسى للمكتب الكويتى بطريق الاتوستراد – مدينةالمعراج)

1- الفحص البصري للمبني بشكل عام.

2- رفع ابعاد الاعمدة لجميع الادوار.

3- تحديد منسوب ظهر الخرسانات المسلحة للقواعد والميدات لتحديد ارتفاع اعمدة البدروم.

4- اجراء اختبارات القلب الخرساني علي الاسقف بالادوار المختلفة .

5- اجراء اختبار مطرقة شميدت علي الاعمدة بالادوار المختلفة وذلك لتقدير مقاومة الخرسانة وفي حالة عدم تحقيقها المقاومة المطلوبة يمكن اجراء اختبار القلب الخرساني لها.

TECHNICAL REPORT ON BUILDING

مقدمة:

أثناء أعمال تطوير مبنى بيت الزكاة الكويتى تلاحظ سوء حالة خرسانة الأسقف لذلك تم إعـداد هذا التقرير بناءاً على طلب السيد مدير المكتب الكويتى للمشروعات الخيرية بالقاهرة وذلـك لمعاينة المبنى وإجراء الإختبارات اللازمة لتحديد السلامة الإنشائية للمبنى.

وصف المبنى:

- المبنى هيكلى خرسانى والحوائط من الطوب.
- المبنى مكون من دور بدروم ودور أرضى وعدد أربعة أدوار متكررة.

معاينة المبنى:

بمعاينة المبنى على الطبيعة بعد إزالة البياض من الأعمدة والكمرات والبلاطات تلاحظ الآتى:

١- سوء حالة خرسانة البلاطات.
٢- خرسانة الأعمدة والكمرات بحالة جيدة.
٣- ترحيل فى بعض الأعمدة بالأدوار المختلفة.

لذلك أثناء المعاينة تم عمل محضر بالموقع والتنبيه بالآتى:

١- رفع أبعاد الأعمدة لجميع الأدوار.
٢- رصد موقع كل عمود.
٣- تحديد منسوب ظهر الخرسانات المسلحة للقواعد والميدات لتحديـد إرتفـاع أعمـدة البدروم.
٤- إجراء إختبارات القلب الخرسانى على الأسقف بالادوار المختلفة.

P.O.BOX : 6 SARAY - EL KOBBA P.C. 11712
14, EL-SOBKY ST., HELIOPOLIS - CAIRO - EGYPT
Tel : (202) 4190843 / 744 / 271 - Fax : (202) 2919341
E-mail :info@cpas-egypt.com www.cpas-egypt.com

ص.ب: ٦ سراى القبة – رمز بريدى ١١٧١٢
١٤ شارع السبكى – خلف نادى هليوبوليس
مصر الجديدة – القاهرة – جمهورية مصر العربية
ت : ٨٤٣ / ٧٤٤ / ٤١٩٠٢٧١ (٢٠٢) ف : ٢٩١٩٣٤١

٥- إجراء إختبار مطرقة شميدت على الأعمدة بالأدوار المختلفة وذلك لتقدير مقاومة الخرسانة وفى حالة عدم تحقيقها المقاومة المطلوبة يمكن إجراء إختبار القلب الخرسانى لها.

الإختبارات:

- تم إجراء إختبار القلب الخرسانى عدد (١٠) للمبنى بواقع عدد (٢) قلب خرسانى لكل سقف.
- تم إجراء إختبار مطرقة شميدت عدد (٢٩) عمود بواقع عدد (٥) أعمده بجميع الأدوار وعدد (٤) عمود بالدور الأخير.
- تم إجراء إختبار التحليل الكيميائى لعينات القلب الخرسانى لسقف الأرضى والأول وقد أعطت نتائج القلب الخرسانى مقاومة أقل من المقاومة المطلوبة طبقاً لمواصفات المشروع وكما هو موضح بالتقرير المرفق يتراوح بين ٥٠ إلى ١٠٠ كجم/سم٢.
- وقد أعطت نتائج مطرقة شميدت مقاومة تحقق المقاومة المطلوبة طبقاً لمواصفات المشروع وكما هو موضح بالتقرير المرفق.

ومن التحليل الكيميائى:

- نلاحظ أن محتوى الكلوريدات أقل من المسموح بها طبقاً للكود المصرى لتصميم وتنفيذ المنشأت الخرسانية.
- محتوى الكبريتات عبارة ٠,٥١٤% من وزن الخرسانة
  يعادل ٣,٢٠% من وزن الأسمنت لسقف الدور الأرضى
  ٠,٦٤٢% من وزن الخرسانة
  يعادل ٤% من وزن الأسمنت

وهذه النسبة فى حدود المسموح به طبقاً للكود المصرى لتصميم وتنفيذ المنشأت الخرسانية.

RESULTS OF SHMEDIT HAMMER TEST

HELWAN UNIVERSITY
FACULTY OF ENG.
MATARIA – CAIRO
TEL : 2433813
CIVIL ENG. DEPARTMENT
PROPERTIES & STRENGTH OF MATERIALS LABORATORY

جامعة حلوان
كلية الهندسة
المطرية – القاهرة
ت : 2433813
قسم الهندسة المدنية
معمل خواص ومقاومة المواد

اختبار مطرقة شميدت

اسم المشروع : مقر بيت الزكاة الكويتي بالقاهره - مدينة المعراج - خلف كارفور
اسم العميل : أ.د/ جودة غانم (مركز الدراسات التخطيطية والمعمارية)

رقم النقطة	متوسط رقم الارتداد	مقاومة الضغط ($كجم/سم^2$)	مكان النقطـــــــــة
1	42.1	438	أعمدة دور البدروم
2	43.4	461	
3	35.3	316	
4	41.2	421	
5	43.9	471	
6	38.7	376	أعمدة الدور الارضي
7	39.4	388	
8	41.3	423	
9	39.1	383	
10	39.8	395	

ملاحظات:

- تم تحديد أماكن النقط بمعرفة العميـــــل
تم اجراء الاختبار بتاريخ 29/8/2006

التجارب والاختبارات
مهندس :
التوقيع :
التاريخ : ٢٢/٩/٢٠٠٦

الاشراف والمراجعة
دكتور مهندس :
التوقيع :
التاريخ : ٢٧/٥/٢٠٠٦

مدير المعمل
أ.د. شريف فخري محمد
التوقيع:
التاريخ: ١٧/٩/٢٠٠٦

عميد الكلية
أ.د. محمد عبد الحميد النشـــــار

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FACULTY OF ENG. | كلية الهندسة

MATARIA – CAIRO | المطرية – القاهرة

TEL : 2433813 | ت : 2433813

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PROPERTIES & STRENGTH OF MATERIALS LABORATORY | معمل خواص ومقاومة المواد

اختبار مطرقة شميدت

اسم المشروع : مقر بيت الزكاة الكويتي بالقاهره - مدينة المعراج - خلف كارفور

اسم العميل : أ.د/ جودة غانم (مركز الدراسات التخطيطية والمعمارية)

رقم النقطة	متوسط رقم الارتداد	مقاومة الضغط (كجم/سم^2)	مكان النقطـــــــــة
1	38.1	365	أعمدة الدور الاول
2	38.4	370	
3	37.5	353	
4	38.8	377	
5	39.8	395	
6	40.1	401	أعمدة الدور الثاني
7	40.7	412	
8	39.9	398	
9	37.9	361	
10	40.3	405	

ملاحظات:

- تم تحديد أماكن النقط بمعرفة العميــــل
تم اجراء الأختبار بتاريخ 29/8/2006

التجارب والاختبارات
مهندس : محمد توفيق أحمد
التوقيع : محمد توفيق
التاريخ : ٢٠٠٦/٩/١٢

الاشراف والمراجعة
دكتور مهندس : طارق ...
التوقيع :
التاريخ : ٢٠٠٦/٩/١٢

مدير المعمل
أ.د. شريف فخري محمد
التوقيع:
التاريخ: ٢٠٠٦/٩/١٧

عميد الكلية
٩/١٧
أ.د. محمد عبد المنعم النشـــــار

HELWAN UNIVERSITY

FACULTY OF ENG.
MATARIA – CAIRO
TEL : 2433813

كلية الهندسة
المطرية – القاهرة
ت : 2433813

CIVIL ENG. DEPARTMENT
PROPERTIES & STRENGTH OF MATERIALS LABORATORY

قسم الهندسة المدنية
معمل خواص ومقاومة المواد

اختبار مطرقة شميدت

اسم المشروع : مقر بيت الزكاة الكويتي بالقاهره - مدينة المعراج - خلف كارفور
اسم العميل : أ.د/ جودة غانم (مركز الدراسات التخطيطية والمعمارية)

رقم النقطة	متوسط رقم الارتداد	مقاومة الضغط (كجم/سم ^{2})	مكان النقطـــــة
1	44.5	482	أعمدة الدور الثالث
2	47.6	539	
3	43.1	456	
4	44.4	480	
5	41.9	434	
6	38.8	377	أعمدة الدور الرابع
7	38.0	363	
8	39.7	393	
9	39.5	391	

ملاحظات:

- تم تحديد أماكن النقط بمعرفة العميـــل
تم اجراء الاختبار بتاريخ 29/8/2006

التجارب والاختبارات
مهندس : محمد توفيق أحمد
التوقيع : محمد توفيق
التاريخ : ٢٠٠٦/٩/١٢

الاشراف والمراجعة
دكتور مهندس : طه الدمرداش
التوقيع :
التاريخ : ٢٠٠٦/٩/١٣

مدير المعمل
أ.د. شريف فخري محمد
التوقيع:
التاريخ: ٢٠٠٦/٩/١٧

عميد الكلية
أ.د. محمد عبد الحميد النشـــار

RESULTS OF CHEMICAL ANALYSIS TEST :-

CONSULTING ENGINEERING BUREAU
(C.E.B.)

المكتب الإستشاري الهندسي
(سيب)

<u>نتائج اختبار التحليل الكيميائي على عينات الخرسانه</u>

العميل : مركز الدراسات التخطيطية والمعمارية .

المشروع : مقر المكتب الكويتي للمشروعات الخيرية – مدينة المعراج – القاهرة .

المحتويات	عينة رقم ١ سقف الأرضى	عينة رقم ٢ سقف الأول
الكلوريدات على هيئة (كل) % من وزن الخرسانة	% ٠,٠٤٨	% ٠,٠٣٤٣
الكبريتات على هيئة (كب أ ٣) % من وزن الخرسانة	% ٠,٥١٤	% ٠,٦٤٢

ملاحظات :

١ – النتائج الموضحه بعاليه تسرى على العينات الموردة من الجهه طالبه الإختبار .

٢ – العينات المختبرة عبارة عن عينتين ، الأولى لسقف الأرضى والثانية لسقف الأول .

٣ – تعذر تحديد محتوى الأسمنت للعينتين لإحتوائهما على نسبة عالية جدا من المواد الكربونية (الجيرية) متمثلة بالركام الكبير الجيرى .

والله ولى التوفيق ،،،

المكتب الإستشاري الهندسي

م/ محمد عمرو عزت سلامه

المكتب الاستشاري الهندسي (سيب)

تحريرا فى : ٢٠٠٦/٩/١٧

31, Fallah St. - Mohandesseen - Tel : 3442118 - Fax: 3444313
Lab. : 3, Nablus St., Mohandesseen -Tel : 3047447

٣١ شارع الفلاح – مدينة المهندسين – تليفون : ٣٤٤٢١١٨ – فاكس : ٣٤٤٤٣١٣
المعمل : ٣ شارع نابلس – مدينة المهندسين تليفون : ٣٠٤٧٤٤٧

Test 19

<u>RESULTS OF CORE TEST FOR COLUMNS:-</u>

CAIRO UNIVERSITY
FACULTY OF ENGINEERING
MATERIALS TESTING LABORATORY

Your Ref. : 29 / 11 /2006 **Applicant:** المكتب الكويتى للمشروعات الخيرية
مبنى مقر المكتب بمدينة المعراج

Our Ref. No. : MTL/ 931 /2006 **Specimens:** عدد ٥ عينات قلب خرسانى موردة بمعرفة العميل

Date : 29 / 11 /2006

تقرير نتائج اختبار القلب الخرسانى

رقم	القطر (سم)	الارتفاع بعد الوسادة (سم)	حمل الكسر (كجم)	مقاومة الأسطوانة (كجم/سم٢)	معامل التصحيح	مقاومة المكعب المكافئ (كجم/سم٢)	ملاحظات
١	٩,٨	١٦,٢	١٠٤٧٠	١٣٨,٧	١,١٩	١٦٥,١	
٢	٩,٨	١٦,٥	٨٩٤٠	١١٨,٥	(*)١,٢٦	١٤٩,٣	
٣	٩,٨	١٤,٢	٨٤٧٠	١١٢,٢	(*)١,٢٢	١٣٦,٩	
٤	٩,٨	١٧,٢	٦٨٣٠	٩٠,٥	(*)١,٢٦	١١٤,٠	
٥	٩,٨	١٥,٧	٩٦٥٠	١٢٧,٩	(*)١,٢٦	١٦١,٢	

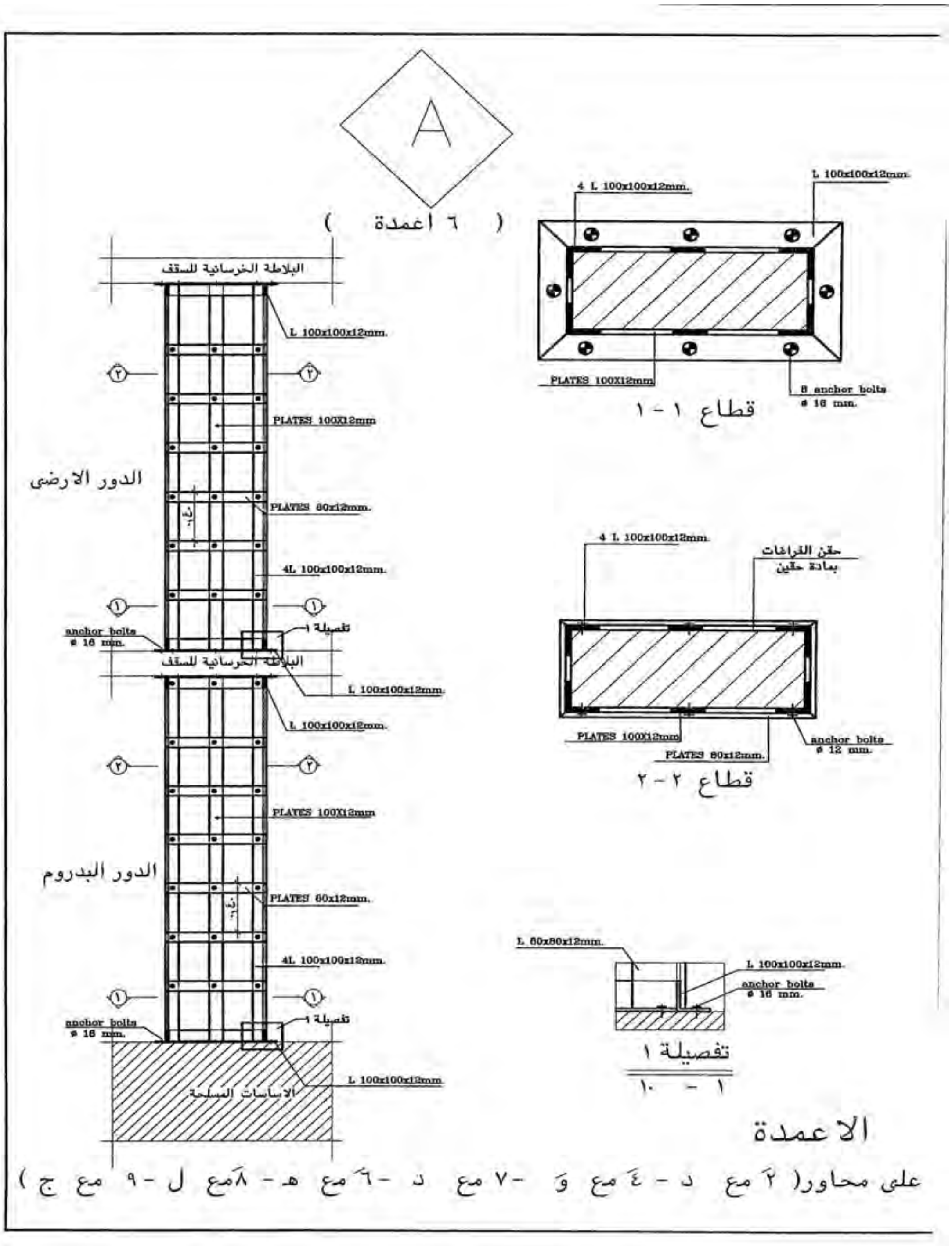
(*) صحح لوجود صلب التسليح

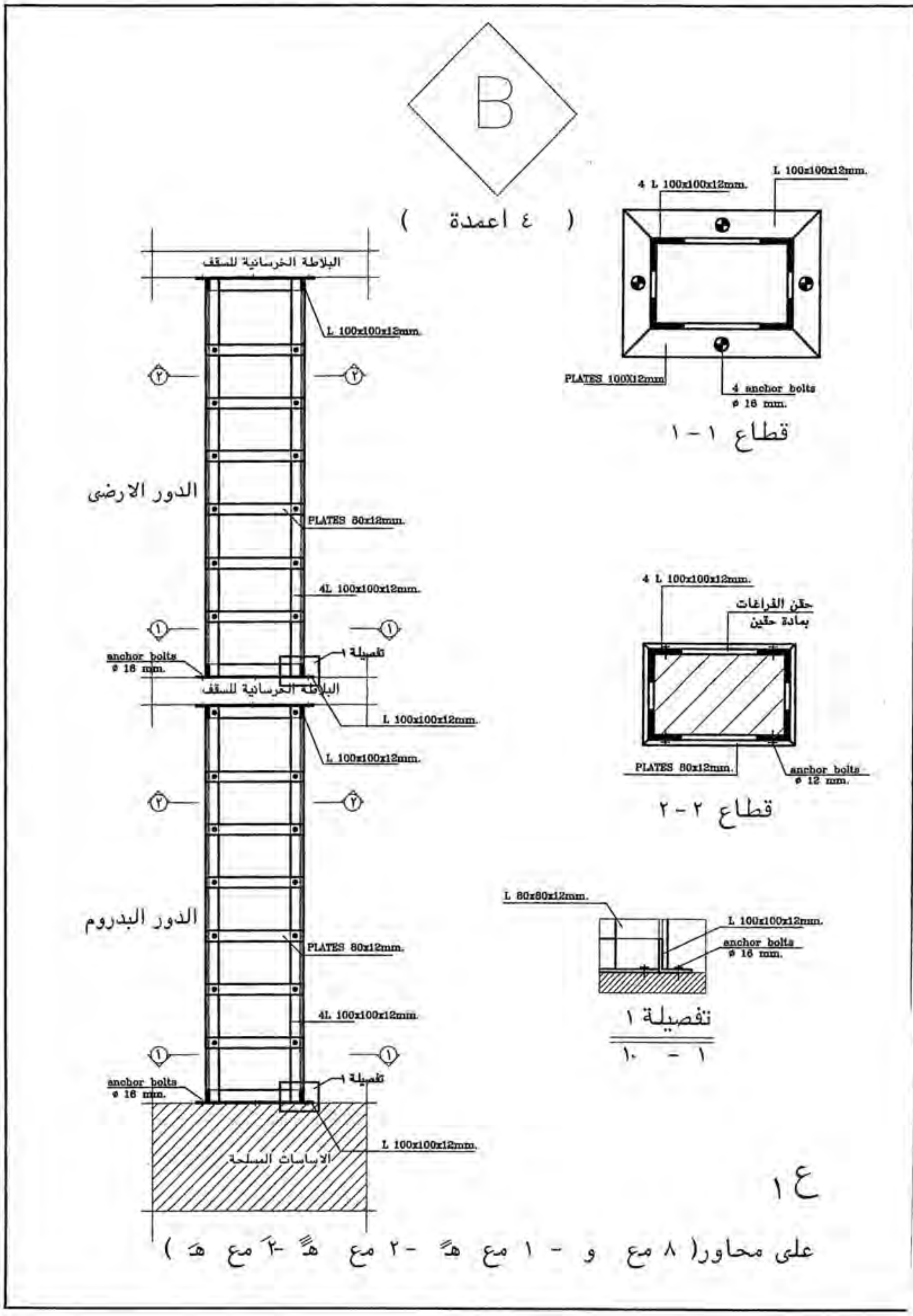
المتوسط ١٤٥ كجم/سم٢

مدير معمل اختبار المواد

أ.د. محمد سامح هلال

TECHNIQUE OF COLUMN STRENGTHENING:-

A
(٦ اعمدة)
البلاطة الخرسانية للسقف
L 100x100x12mm.
PLATES 100X12mm
الدور الارضى
PLATES 80x12mm.
4L 100x100x12mm.
anchor bolts
ø 16 mm.
تفصيلة ١
L 100x100x12mm.
الدور البدروم
الاساسات المسلحة
4 L 100x100x12mm.
PLATES 100X12mm
8 anchor bolts
ø 16 mm.
قطاع ١ - ١
حقن الفراغات
بمادة حقين
PLATES 80x12mm.
anchor bolts
ø 12 mm.
قطاع ٢ - ٢
L 80x80x12mm.
تفصيلة ١
١ - ١٠
الاعمدة
على محاور (٢ مع د - ٤ مع و - ٧ مع د - ٦ مع هـ - ٨ مع ل - ٩ مع ج)


B
(٤ اعمدة)
البلاطة الخرسانية للسقف
L 100x100x12mm.
الدور الارضى
PLATES 80x12mm.
4L 100x100x12mm.
anchor bolts
ø 16 mm.
تفصيلة ١
البلاطة الخرسانية للسقف
L 100x100x12mm.
الدور البدروم
PLATES 80x12mm.
4L 100x100x12mm.
الاساسات المسلحة
4 L 100x100x12mm.
L 100x100x12mm.
PLATES 100X12mm
4 anchor bolts
ø 16 mm.
قطاع ١ - ١
4 L 100x100x12mm.
حقن الفراغات
بمادة حقين
PLATES 80x12mm.
anchor bolts
ø 12 mm.
قطاع ٢ - ٢
L 80x80x12mm.
L 100x100x12mm.
anchor bolts
ø 16 mm.
تفصيلة ١
١ - ١٠
ع ١
على محاور(٨ مع و - ١ مع هً -٢ مع هً -٢ مع هَ)

TECHNICAL REPORT ON STRENGTHENING OF BUILDING:-

Center of Planning and Architectural Studies
"Prof. Dr. Abdelbaki Ibrahim & Partners"
Expertise House of Engineering Consultant

CPAS

مركز الدراسات التخطيطية والمعمارية
"أ.د / عبد الباقى إبراهيم وشركاه"
بيت خبرة في الهندسة الاستشارية

تقـــريـــر

عن أعمال التدعيم لمبنى المقر الرئيسي – بيت الزكاة الكويتي

مقدمة:

تم إعداد هذا التقرير بناءاً على طلب مدير مكتب القاهرة – المكتب الكويتي للمشروعات الخيرية بموجب الخطاب رقم ٨٢٩ المؤرخ ٢٠٠٦/١٢/١٢ بشأن تقديم دراسة كاملة عن كافة أعمال التدعيم بالمبنى الكائن على الطريق الدائري بجوار كارفور المعادي .

أولاً : وصف المبنى:

- المبنى هيكلى خرسانى والحوائط من الطوب.
- المبنى مكون من دور بدروم ودور أرضى وعدد أربعة أدوار متكررة.

ثانياً : الأختبارات التى أجريت:

١ – البلاطات :

١–١ تم عمل أختبار الضغط للقلب الخرسانى على أماكن في سقف البدروم وسقف الأرضي والأول و الثاني والثالث . بالإضافة إلى عمل أختبار تحليل كيميائي لعينة بسقف الأرضي والأول وبناءاً على ذلك تم التوصية بتكسير جميع البلاطات ذات البحور الأكبر من ٢,٢٥م مع الأحتفاظ بالحديد وإعادة صبها بخرسانة ذات مقاومة مميزة ٢٥٠كجم/سم٢ ومرفق نتيجة هذه الأختبارات(مرفق ١).

١–٢ تم عمل أختبارات تحميل لبلاطة كابولية بسقف الأول وعدد ٢بلاطة بسقف الثاني و بناءاً على نتائج الأختبارات (مرفق ٢نتائج الأختبارات) تم التوصية بعمل تدعيم للبلاطات الكابولية فقط عن طريق تكسير الخرسانة والأبقاء على الحديد وإضافة حديد جديد وصب خرسانة جديدة ذات مقاومة مميزة ٢٥٠كجم/سم٢ ومرفق أسلوب التدعيم (مرفق ٣) .

P.O.BOX : 6 SARAY - EL KOBBA P.C. 11712
14, EL-SOBKY ST., HELIOPOLIS - CAIRO - EGYPT
Tel : (202) 4190843 / 744 / 271 - Fax : (202) 2919341
E-mail :info@cpas-egypt.com www.cpas-egypt.com

ص.ب: ٦ سراى القبة – رمز بريدى ١١٧١٢
١٤ شارع السبكى – خلف نادى هليوبوليس
مصر الجديدة – القاهرة – جمهورية مصر العربية
ت : ٨٤٣ / ٧٤٤ / ٤١٩٠٢٧١ (٢٠٢) ف : ٢٩١٩٣٤١

٢- الأعمدة :

١-٣ تم عمل أختبارات المطرقة على عدد من الأعمدة في البدروم والأرضي والأول والثاني والثالث والرابع ومن نتائج الأختبارات وجد أن جميع الأعمدة آمنة ولا تحتاج لتدعيم (مرفق ٤نتائج الأختبار).

١-٤ تم عمل أختبار القلب الخرساني على بعض الأعمدة بالبدروم ومن نتائج الأختبارات (مرفق ٥) وجد أن المقاومة المميزة للخرسانة ضعيفة وبناءاً عليه تم التوصية بعمل تدعيم لبعض الأعمدة في البدروم والأرضي بأستخدام زوايا حديد في الأركان ومرفق أسلوب التدعيم (مرفق ٦) .

٣ - بالمعاينة و الكشف على الكمرات وجد أن حديد التسليح السفلي غير موجود في موضعه التصميمي وكذلك وجود أزاحة لأماكن الكانات الأمر الذي يدعو إلى الأحتياج لتدعيم بعض الكمرات (الكمرات ذات البحور أكبر من ٣,٠٠متر) نتيجة لذلك بأستخدام Carbon fiber أسفل الكمرات ومرفق صورة لهذه الكمرات التي تم الكشف عليها (مرفق ٧).

رئيس القسم الإنشائى

أ.د. / جودة محمد غانم
إستاذ الانشاءات هندسة المطرية – جامعة حلوان

رئيس المركز

د. محمد عبد الباقي إبراهيم
أستاذ م. بكلية الهندسة –جامعة عين شمس
استشارى التخطيط والتصميم العمرانى

P.O.BOX : 6 SARAY - EL KOBBA P.C. 11712
14, EL-SOBKY ST., HELIOPOLIS - CAIRO - EGYPT
Tel : (202) 4190843 / 744 / 271 - Fax : (202) 2919341
E-mail :info@cpas-egypt.com www.cpas-egypt.com
ص.ب: ٦ سراى القبة – رمز بريدى ١١٧١٢
١٤ شارع السبكى – خلف نادى هليوبوليس
مصر الجديدة – القاهرة – جمهورية مصر العربية
ت : ٨٤٣ / ٧٤٤ / ٤١٩٠٢٧١ (٢٠٢) ف : ٢٩١٩٣٤١