APPLICATION OF CFRP COMPOSITE FOR SUSTAINABLE SOLUTION OF CORRODED SLAB SYSTEM DUE TO LOW NOMINAL COVER

CASE STUDY ON NERD SLAB SYSTEM

Chathuri Kaushalya Ganewattha

198047V

Degree of Master of Science

Department of Civil Engineering

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A thesis submitted in partial fulfilment of the requirements for the degree Master of Science in Civil Engineering

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DECLARATION

I declare that this is my work. This thesis does not incorporate without acknowledgment any material previously submitted for a degree or diploma in any other university or institute of higher learning. To the best of my knowledge and belief, it does not contain any material previously published or written by another person except where the acknowledgment is made in the text.

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The above candidate has carried out research for the master's under my supervision.

Name of the supervisor: Prof. (Mrs.) J C P H Gamage

Signature of the supervisor: UOM Verified Signature Date: 26.03.2023

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C K Ganewattha

Department of Civil Engineering

University of Moratuwa

26.02.2023

ABSTRACT

Carbon Fiber Reinforced Polymer (CFRP) strengthening technique had been shown excellent performance in externally strengthening reinforced concrete (RC) elements due to their superior properties compared to the alternative strengthening techniques. A substantial number of studies have been done to study the behavior of externally bonded CFRP strengthened RC elements. However, as per the knowledge, while most studies have focused on the external strengthening of RC beams using CFRP, and very few studies have focused on strengthening the pre-stressed beams and slabs.

Pre-stressed concrete is most popular building technique in construction buildings. Steel corrosion is recognized as the most serious and dominant mechanisms of deterioration for concrete structures. Subsequently, the capacity of the pre-tension elements decreases after exposure to corrosion. NERD center slab system faces such unacceptable losses in load carrying capacity, stiffness, and ductility due to severe corrosion in pre-stressed beams. This study focuses on how CFRP can go for a load increment after reaching its ultimate load carrying capacity.

The test procedure was arranged in two stages. The first stage testing was used to show the performance/ behavior of composite slab specimen and pre-stressed beam, with the application of load. The second stage of testing was carried out to make comparison between the structural performance of retrofitted and strengthen specimens. Specimens were selected for retrofitting after application of loading in stage 1. The total of 12 specimens were exposed to the loading and behavior of each of the specimen were observed. Specimens were selected as slabs and eight number of them were composite slabs with or without shear links which have overall dimensions equal to 1800 * 600 mm and other four were pre-stressed beams with overall dimensions of 1800 mm in length. In this study, CFRP is proposed as the economical solution which does not touch the structural integrity of the structure.

All the specimens were tested using universal loading machine. In stage 1, specimens were loaded up to its ultimate failure. In stage 2, all the tested specimens were retrofitted using CFRP. In total number of six specimens were used for retrofitting. Another six specimens were also strengthened using CFRP before loading. In each stage of loading several observations were done. Such as mode of failure, cracking width distribution, ultimate load, and each composite panel's corresponding deflection were also recorded.

In stage 1, flexural and shear cracks propagated in the pre-stressed beam and the beam failed at the applied load of more than 50 kN. Stage 2 focused on the performance/ behavior of the retrofitted and strengthened specimen after application of CFRP. The results from stage 2 showed a considerable reduction (nearly 20%) in loading of retrofitted/ strengthened composite slabs compared to control specimens. Difference in failure pattern is caused for this discrepancy in load demand of second stage. The experimental results showed some satisfactory performance in regaining the lost strength of the composite specimens due to corrosion.

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CHAPTER 1: INTRODUCTION

1.1. Background

Concrete is a compound material which having relatively low tensile strength and ductility. If concrete comprised with steel reinforcement which having higher tensile strength or ductility, then it is called as reinforced concrete. Concrete is the most common material used for building different types of structures and components of the structures including slabs, walls, beams, columns, foundations, frames and more. Precast or cast-in-place concrete are the classifications of fabricating reinforced concrete. Normal reinforced concrete does not easily endure the stresses like wind action, earthquakes, vibrations, and other bending forces. Therefore, it is unsuitable for many structural applications. Pre-stressed concrete is another innovation in construction industry, and it is achieved by either pre-tensioning or post tensioning processes. At the present, this type of applications is commonly used for the construction of floor beams, piles, and railway sleepers. In addition to that it can be used for the structures such as bridges, water tanks, roofs, and runways. Precast concrete elements are cast and cured off site and can be joined to form a complete structure. Most efficient floor system is key to creating optimal building structures. It can be designed and implemented using such innovation techniques make significant impact on material costs, ultimate strength, operating costs, occupancy levels and end use of a building. (Darvin, Dolan and Nilson, 2016)

In 1987 another new method of reinforced concrete slab system was introduced by late Dr. A.N.S. Kulasinghe. The National Engineering Research Development (NERD) slab system comprises of trapezoidal-shaped pre-stressed beams at 600 mm intervals and 50mm thick in-situ concrete topping. This system had several advantages such as significant savings in materials, labor, and time, not requiring soffit plaster and works space below the slab and immediate availability after construction (Sanjaya et al. 2015).

Nevertheless, due to several reasons the reinforced slab systems might get subjected to deterioration over time. Embedded reinforcement corrosion is the major concern of concrete deterioration rapidly, especially when steel reinforcement exposes to the aggressive agents in the surrounding environment. Triantafyllou et al. (2019) mentioned that the influence of corrosion makes the structural failure mode from ductile to brittle failure and may lead to unpredictable failures of the structures. Corrosion of the steel cause for a reduction of the cross-sectional area and it is resulted in the decrease in the strength of the steel, and the loss of the bond strength between the steel bar and the concrete. Additionally, change of the ductility of rebar and cracking of surrounding concrete of the corroded reinforcements can be caused in reduction in ductility, the serviceability and the load carrying capacity of the structures. Further, deterioration of bond strength between steel-concrete interface causes for decrease in flexural strength. Hence, researchers have experimentally investigated the rehabilitation techniques for reinforced concrete structures with corroded steel reinforcement. (Alwash et al.,2019)

In recent times, most of researchers are directing their research towards several inadequacies like deterioration of materials or aging, poor maintenance of structural elements. They are making the efforts for creating an efficient and cost-effective technique of restoring affected structures. Numerous techniques are being applied in strengthening of concrete structures in several ways around the world. Such as external post-tensioning and pre-stressing, over-slabbing, or increasing the ratio of bonded reinforcement. External pre-stressing is one of the most appropriate techniques for strengthening or rehabilitating existing structures, and it make considerable cost savings, and reduction in construction time (Suntharavadivel, 2008). If improving the structural load carrying capacity is the main purposes of the existing structural member, tendons which is made by steel should be placed at the outside of the element, tensioned, and anchored at their ends. Fiber reinforced polymer (FRP) is the material which can provide the most efficient solutions rather than steel tendons and it provides most effective solutions such as increase the structural capacity of the prevailing structures when they need some renewal or retrofitting (Osimani, 2004).

FRP is a thin polymer layer that is comprised of different fibers such as glass FRP (GFRP), carbon FRP (CFRP), aramid FRP (AFRP), and basalt FRP (BFRP). Sheets bonded to concrete with adhesives, Laminates, dry fibers, or rods, and fasteners is different form of application of FRP composites (Siddika, 2019). Standard specifications are specifying the requirement for upgrading of concrete members. CFRP has high resistance to all forms of alkali and has high stiffness than GFRP while demanding low amounts of resin. Therefore, the selection of FRP for strengthening of defective concrete members are crucial and requires more attention to gain structural capacity of the members. Shear and flexural strengthening can be done by applying FRP as externally bonded (EB) laminates, near surface mounted (NSM) bars/ strips, mechanical anchorage systems, or grooving methods with or without adhesives. EB FRP can be done with the desired number of layers in different configurations and due to the lack of adequate cover, NSM cannot be used for NERD center slab systems. Hence flexural and shear capabilities can be formed by using side bonding, U-wrapping, or full wrapping of the specimens.

Many studies have been focused on behavior of RC structures when exposed to the aggressive environments and rehabilitation techniques for corroded specimens. They checked only for ultimate strength of the structures before and after retrofitting. Very few studies have been given the results for ultimate strength of corrosion defected pre-stressed structures and their strengthening/ rehabilitation with FRP. Therefore, the research study presented in this thesis was concentrated on experimental investigations of load carrying capacity of composite slab panels before and after strengthening/retrofitting with CFRP.

1.2. Research Objectives

- To investigate existing strengthening method to restore reinforced concrete slabs.
- To evaluate the strength and the service performance of the composite slab system introduced by NERD.
- To investigate the possibilities of strengthening the composite slab system using Fibre Reinforced Polymer (FRP) technology.
- To provide design recommendations to strengthen the NERD slab system with low nominal cover using CFRP.

1.3. Methodology

Literature survey was conducted to identify the behaviour of pre-stressed composite slabs when they were exposed to hazardous environment and the effectiveness of existing retrofitting method. The following flow chart indicates the research direction:

Identify the problems in the composite slab system (NERD System)

↓

Conduct a literature review to evaluate the existing strengthening methods to restore similar composite slabs.

↓

Determine the behavior of composite slab and the elements experimentally.

↓

Data Analysis, Conclusions and Recommendations

Figure 1.2: Summary of methodology

1.4. Significance of the Research

Propagation of corrosion cracks and losing the strength due to the exposure to aggressive environmental condition in NERD centre slab system remain as unsolved problems. Application of insulation layers on defected areas after removing corroded debris and preparing surface are cost effective than removing defected pre-stressed beams. Moreover, addition of high thick insulated materials increases the dead load of the structures and the appearance of the structures. Hence, developing suitable retrofitting method as sustainable solution is much necessary for the durability of the structures. The degradation prior exceeding the service period due to corrosion of reinforcement resulted from low nominal cover is a common problem in the composite slab system introduced by NERD. CFRP strengthening technique has been successfully used in strengthening reinforced concrete slabs. However, there is no evidence on the behaviour of such strengthened composite slab members. This investigation is focused on experimental quantification of behaviour of prestressed beam members, reinforced concrete slab and composite slab system without strengthening and strengthening using CFRP for strength enhancement or recovery due to cracking and corrosion.

1.5. Thesis Arrangement

This thesis consists of several chapters and their descriptions are explained as follows.

Chapter 1. NERD slab system is one of the prominent composite slab systems among other slab systems in Sri Lanka. Hence this chapter presents outline of the background, research significance and objectives followed by this study.

Chapter 2. Present a detailed literature survey which was carried out to identify the issues in pre-stressed beams, composite slabs compared to the NERD slabs and the available rectification methods. Behavior of CFRP compared to the other FRP composites when applying loads, surface preparation prior to the application and ways of applying CFRP for cracks and major drawbacks of existing systems are described with several examples. Developing suitable FRP application system for NERD center slab system as the research gap also clearly explained.

Chapter 3. Recently some defects were found in NERD slab system which were exposed to the aggressive environment. This chapter provide some information about defects seen on NERD slab panels due to low nominal cover before applying CFRP as the retrofitting plan.

Chapter 4. This research was carried in two different stages. This chapter represents the detailed description of the two stages including the notation of the specimens which were used for testing and their material properties, sample preparation for the testing, and the test setup.

Chapter 5. Experiment was conducted in two stages. Content of this chapter provides a discussion of the experimental results obtained from each stage.

Chapter 6. This chapter provides general and specific conclusions of the study. Conclusions were made according to the experimental results obtained from two stages.

Chapter 7. Retrofitting and strengthening are two different strategies. Basically, this research was attentive on the rectification of the slab system which exposed to the aggressive environmental conditions. Hence, this chapter provides the recommendations for the design and rectification of NERD slab.

CHAPTER 2: LITERATURE REVIEW

2.1 Concrete structures

Concrete is the most durable compound material which is composed with fine and coarse aggregate, and they are mixed together with a watery cement that hardens over time. Due to its superior fire resistance compared to the wooden construction, it is the second most used substances after water in the world. Concrete is very strong in compression and weak in tension. Inexpensive and easy to produce at site make it very popular in the construction industry. Reinforced cement concrete or RCC is a concrete that contains steel which is a viable solution for making tension to the concrete. Reinforced concrete is made up of concrete with steel laid in the formwork. Strength of concrete may be varied from light weight to high strength concrete. For larger civil engineering projects high strength concrete is used. (Mccormac & Brown, 2016)

2.2 Concrete slabs

Reinforced concrete slabs are known as the massive-scale flat plates. It is supported by reinforced concrete beams, walls, or columns; by masonry walls; by structural steel beams or columns; or by the ground. According to the supporting conditions, slabs are divided into two groups. They are one-way slabs (supports are provided in two sides and main reinforcements are provided in one direction only) and two-way slabs (supports are provided in four sides and main reinforcements are provided in two directions) (Shawkat, 2017). The thickness and the reinforcement requirement depend on the bending, the deflection, and the shear requirements (Jack & Russell, 2016). Obviously, slabs can be arranged as pre-stressed, or concrete can be poured into the mold of prior rebar positioning according to the requirement and load bearing capacity of the slabs. These manufacturing methods are used as per the feasibility of the project. Slabs made up of pre-stressed beams or hollow blocks, hollow core slab made with precast concrete, cast in situ slabs are several common slab designs for multi-storied buildings.

2.2.1. Pre-stressed slabs

The textbook of “Design of reinforced concrete” mentioned the theory of prestressed which is quite simple, and that theory has been used in numerous kinds of structures for many years. This slab system is used for larger available span widths with minimal support requirements, up to 3m, and thickness of the concrete board is 80 – 120 mm. It is mentioned as a best option for overcoming the concrete’s natural weakness in tension. In this slab system, bracing wire strings are used as longitudinal reinforcement and steel bars are laid over the pre-stressing reinforcement, as the transverse reinforcement. Although rods are trying to regain their original position after laying, it is prevented by the surrounding concrete to which the steel is bonded. Pre-stressed floor slab system does not require any plastering works due to its high surface quality. Pre-stressed concrete designs are a concept of inducing stresses throughout the entire structure. The result of this method is to obtain a better product to handle the vibrations and shocks than conventional concrete. Further, it can be used to form longer and thinner structures that can handle heavier loads too. Pre-tensioning and post tensioning are two ways of giving stability for the pre-stressed concrete structures and the method of tensioning is selected depending on the requirement and the facilitation provided. (Mccormac & Brown, 2016, Mohamed et. al., 2015)

2.2.2 Composite Slab (use in NERD, Sri Lanka)

2.2.2.1 Background

Dr. ANS Kulasinghe introduced an economically feasible slab system, especially for domestic buildings in Sri Lanka in 1987 (Sanjaya et al. 2015). As shown in Figure 2.1, the first NERD center (National Engineering Research Development) slab system was built at the Department of Machine Development and Fabrication, NERD Center, Sri Lanka.

Figure 2.1: First slab construction at the NERD center

This slab system was designed with trapezoidal-shaped pre-stressed beams at 600 mm intervals and with the thickness of 50 mm in-situ concrete topping. The Pre-stressed beams contain three nos. of 5 mm diameter, high tensile steel wires, and each wire was tensioned up to 20 kN. Pre-stressed beams and in-situ concrete toppings used were Grade 40 and Grade 30 concrete. A 50 mm x 50 mm welded GI mesh (Galvanized Iron mesh) was positioned at the center of the 50 mm thick In-situ topping as shown in Figure 2.2.

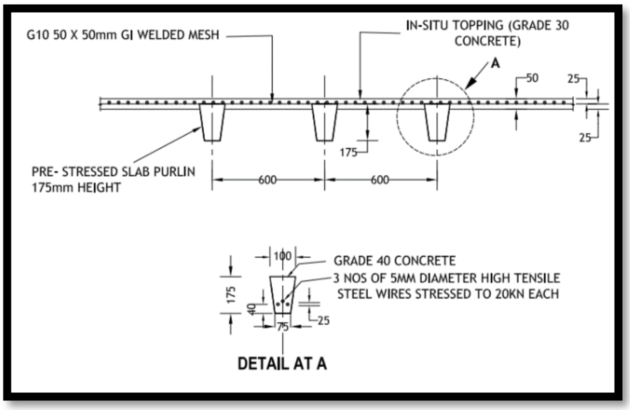


Figure 2.2: Typical slab system

Although the thickness of the situ topping, the pre-stressed force, and the concrete grades remain unchanged, the beams' depth has varied depending on the span, as given in Table 2.1. The detailed design of the composite slab system is attached in **Annex 01.**

Table 2.1: Variation of pre-stressed beam depth with the span

Span (mm)	**Beam depth (mm)**
Less than 3000	100
3000 to 3600	125
3600 to 4200	150
4200 to 4800	180

Note: These heights are designed for an imposed load of 1.5 kN/m^2

After a certain period of implementation, the original slab was modified by few engineers. They eliminated the practical difficulties faced during the construction, such as plywood formwork, hanging the formwork from the beam, and introduced an additional Ferro cement panel as a modification as shown in Figure 2.3.

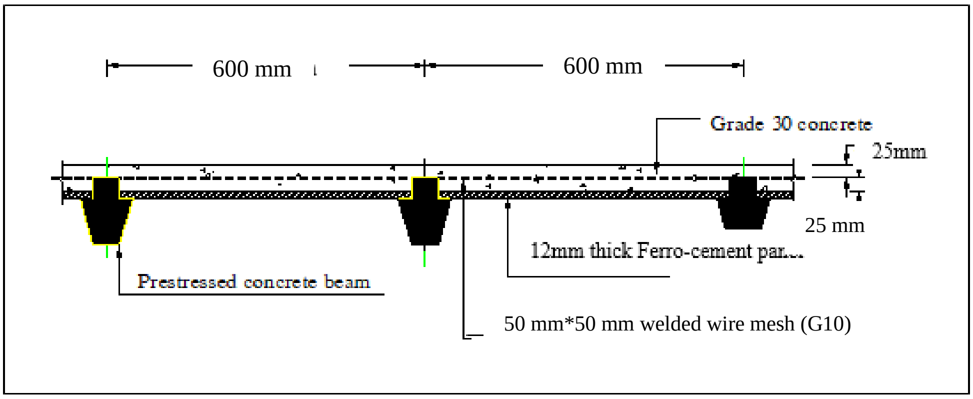


Figure 2.3: Modified slab system

For continuous construction purposes, the NERD center has trained over 800 contractors to construct the slab system. Over 50 technically qualified persons were instructed to manufacture the pre-stressed beams at their yards since the development of the slab system. Approximately 500 slab units are being constructed each year by these contractors at their pre-stressing yards. This slab system has successfully been used for more than 25 years in Sri Lanka due to the following constructional aspects.

1. This slab system gives significant savings in materials, labour, and time.
2. Dead load from the whole slab compared to the other slabs is considerably reduced by using a 50 mm thick slab.

3. Bottom working spaces are provided for the construction activities immediately after concreting the slab due to their no propping requirements at the construction stage.
4. The soffit plastering is not required.

Composite solid slabs give a positive bending moment at the in-situ topping surfaces used to develop live loads. Longitudinal shear links are used at the top of the beam to resist the horizontal shearing of the concrete. Hence, modification is done by applying shear links and in-situ topping after applying gypsum boards as shown in Figure 2.4.

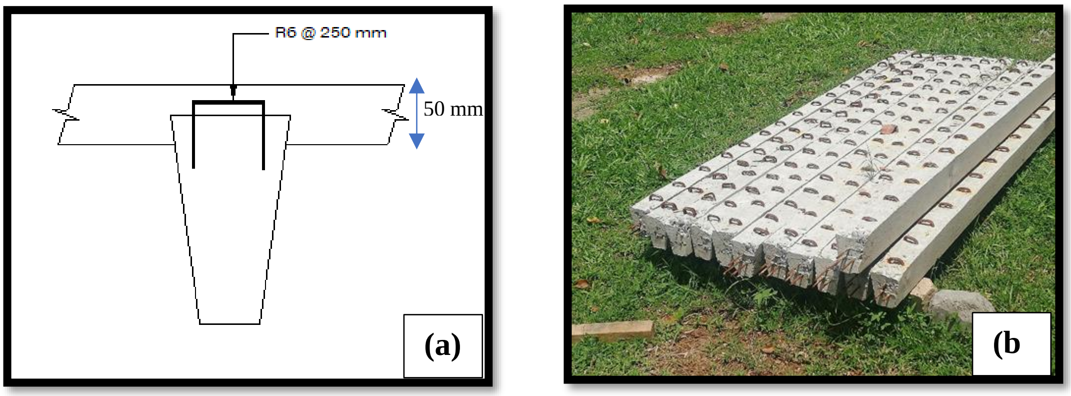


Figure 2.4: (a) Systematic diagram of modified connection; (b) Cast beams with modification.

NERD center slab system gained high popularity due to its economic feasibility. The projects carried out by applying the NERD slab system are attached in **Annex 02**. The comparison of cost between the NERD center slab system and the conventional slab system is given in Table 2.2. The summary of the cost calculation for 1 m^2 of NERD and traditional slab panels is attached in **Annex 03**. These cost figures are based on 2019 material prices, labor, and equipment supply charges. The summary of cost calculation for 1 m^2 of NERD and conventional slab panels are attached in **Annex 03**. These cost figures are based on 2019 material prices, labor and equipment supply charges. Table shows that more than 15% savings in rate compared to the conventional slab systems.

Table 2.2: comparison of rates of slabs

Item	**Description**	**Rate per Sqft with NERDC licensees' components**	**Soffit finishing work**	**Finishing with soffit NERDC licensees' components**	**Percentage saving compared to conventional slab**
					Percentage saving % NERDC yard components
1	50mm thick floor slab with G10 50mm x 50mm wire mesh with decorated panel Concrete 1:1 1/2 :3 (3/4")	4846	110	4956	15.06%
2	50mm thick floor slab with G10 50mm x 50mm wire mesh with suspended form work Concrete 1:1 1/2 :3 (3/4")	4465	330	4795	17.82%
3	5"Thick Conventional slab concrete 1:11/2":3"(3/4")	5835		5835	0.00%

Significant savings in materials, labor, and time are the main reasons for the cost savings. Pre-stressed beams are generally cast on NERD center, Ja Ela. In-situ toppings are applied in site.

2.2.2.2 Present method of construction

2.2.2.2.1 Stressing of pre-stressing wires

30 m long pre-stressed wires are typically used to manufacture pre-stressed concrete beams. To stress the wires, wires are fixed at one end, and jacking is done as per the special jacking arrangement introduced by Dr. ANS Kulasinghe at the other end, as shown in Figure 2.5. The wires, which have a tensile strength of 1670 kN/mm^2, are kept straight as much as possible by using intermediate supports before jacking. For pre-stressing, normal relaxation indented type wires of 5 mm diameter are used, and for the holding purposes of the wires, an anchor cone and an anchor pin are used at the end.

Figure 2.5: Stressing wires using special jacking arrangement.

The basic stress-strain relationship is given by

$$E = \frac{F/A}{e/l} \qquad (1)$$

E = Young's modulus (205 kN/mm^2)

P_j =Jacking Force (20kN)

A_t =Cross sectional area of the tendon (π x 2.5^2)

l = Initial length {30.49 m (=100 ft.)}

e = Elongation.

The elongation is used to control the jacking force. In the case of 20 kN jacking force, the expected elongation is nearly 150 mm for a 5 mm pre-stressing wire of 30 m in length.

2.2.2.2.2 The casting of pre-stressed concrete beams

To cast the pre-stressed beams, the long line method is used as shown in Figure 2.6. For the casting of the beams (with depths of 100 mm, 125 mm, 150 mm, 175 mm), 175 mm deep trapezoidal-shaped steel shutter is used. The nominal mix of 1:1:2 concrete with early strength gain admixtures and 15 mm metal size are used to cast the beams of pre-stressed purlins. Figure 2.7 shows the wire position of design of the pre-stressed beams.

Figure 2.6: Longline method of casting

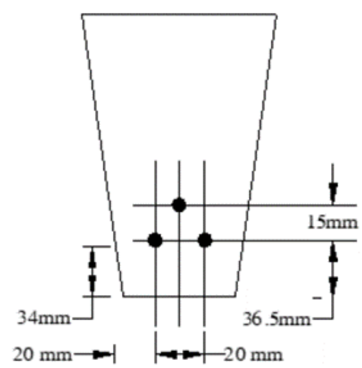


Figure 2.7: Wire position

To keep the wires in the required position, two types of endplates are used as shown in Figure 2.8.

Figure 2.8: Endplate types

2.2.2.2.3 Transfer of pre-stressing force

Formwork are expelled after one day of concreting, and pre-stress is switched to the concrete three days after concreting by cutting pre-stressing wires. Special admixtures are used to achieve the required strength at transfer within three days.

2.2.2.2.4 Transport and handling of beams

Pre-stressed beams are maintained in the supporting condition as simply supported when they are handled, lifted, and transported. Extra non-pre-stressed tor steel wire is used at the top of the beam in the case of slender beams so that it can withstand the additional tensile stresses.

2.2.2.2.5 Placing of Beams to construct floor slab.

According to the span (Table 2.1), selected pre-stressed beams are kept with the minimum length of 600 mm center to center distance.

2.2.2.2.6 Making the In-situ Topping.

Binding wires are used to hang the plywood shutter on pre-stressed beams, and a portion of the pre-stressed beam is inserted into the concrete, as shown in Figure 2.9. 50mm in-situ topping is cast utilizing the nominal mix of grade 30 (1:1 ½: 3, ¾") concrete whereas keeping the gauge 10, 50 mm x 50 mm welded mesh at the middle of the in-situ topping.

Figure 2.9: Casting the composite slab panel (a) Hang the plywood shutter (b) keeping the welded mesh at the center (c) Making the in-situ topping.

2.2.3 Structural integrity of concrete structures

Structural integrity is a parameter mainly related to structural reliability. The concrete structure's durability, serviceability, and strength will determine its reliability. Structural integrity is the system's ability to resist deforming and breaking excessively of its structural members. For the design of the structure, ultimate and serviceability limits serve basic guidelines, and it is advised to stay within the design limits and their design capacities. According to P. Pavii,2007, the structure's general integrity is weakened owing to excessive accidents and unfavorable threats such as congestion, fire, explosion, flood, and earthquake, and certain structural limit states may be encountered. Significant implications that may induce damage initiation and development in the concrete structure are the aggressive environment, construction quality, loading history, and maintenance quality. [Pavisic P, 2006] Reinforced concrete is the major component in construction that used worldwide, and corrosion of embedded reinforcing bars is the primary course of the deterioration of structural member. However, with pre-stressed concrete, the loss of effective cross-sectional area owing to cover concrete cracking, failures in the mechanical characteristics of rebar due to lower cross-sectional areas, and bond strength and stiffness between corroded rebar and concrete are not much more severely affected. But in the case of ultimate and serviceability limit, states may affect significantly. Many researchers have widely studied corrosion of steel reinforcement structures (Fernandez et al. 2018 c; Tahershamsi, 2016, Craig (); Triantafyllou 2019; Cairns et al. 2008c), and they viewed that reducing the carrying capacity makes a direct impact on structural safety. Moreover, corrosion creates more rust than the volume of Steel due to its' tensile force and exerts more tensile force on the concrete which leads to the spalling of the specimen. (Parajuli, 2016)

The reliability of the structure is associated with its durability, serviceability, and strength. Those are ensuring the comfort and longevity of the system. For design purposes, ultimate and serviceability limit states serve as basic guidelines for the system's reliability. Corrosion erodes the cross-sectional reinforcement area. Although corroded rebar does not contribute much to the strength of the pre-stressed structures, it directly affects states' ultimate limit state and serviceability limit states. (Parajuli,2016). Failure may happen due to reduced flexibility, and reduced cross-sectional areas lead to steel-concrete interface problems. Besides, corrosion produces a high amount of rust greater than the rebar area, which results in extra tensile force. When the tensile strength of the concrete is exceeded due to extra tensile force and the concrete tend to spall. However, due to the corrosion issue, a passive coating within the reinforcement prevents additional corrosion to a certain extent. When corrosion extends beyond the depth of the concrete cover, the reinforcement may deteriorate. This could be due to a lack of concrete cover due to faulty design or a waterproof membrane. Therefore, before deciding on the scope of corrective activities required, it is critical to study the cause of degradation through meticulous analysis and implications.

2.3. Deterioration of Concrete Structures

2.3.1. General literature on concrete and defects

Defects are the major types of things that can affect the concrete structurally or architecturally, and major defects are as follows.

Crack

- Crazing – developed in the surface of the concrete-like fine random cracks or fissures due to shrinkage of the surface layer. Most rarely, it will be more than 3mm deep.
- Disintegration – when concrete is breaking down or deteriorating into small fragments or particles.
- Plastic cracks – when concrete is just placed, finished and left to be exposed to the warm drying wind cracks can develop over the surface until few hours of the slab's life.
- Hardened cracks – When hardened concrete is exposed to moisture, it expands, and when exposed to low-humidity air, it shrinks.
- Scaling – when hardened concrete is exposed to freezing and thawing, local flaking or peeling on the concrete surface may occur.

✓ Delamination – Surface separation of the thin layer (typically thickness from 3mm to 6mm) and this resulting in an unbounded concrete layer between them. These are developed in dowelled concrete related to the timing of the final trowel (trapping air and water at the finishing operations). Overloading which occurs when the design load is lower than the applied load threatens the structure and its structural integrity. Overload causes cracks in the concrete, and these crack patterns indicate the types of loading stresses.

- ✓ Flexural crack – Crack propagation of concrete beams pre-stressed with single-strand tendons, Tehrani et al. (2014) provides a comprehensive understanding of this topic. They focused to find a solution for the flexural cracks that occur at the centre of the ties. There are two types of flexural cracks.

  Positive flexural cracks – cracks occur vertically at the bottom and the centre of the simply supported beam.

  Negative flexural cracks – when damage occurs above the supports at the top of the beams.

Also, the splitting cracks give some passage for the moisture to enter inside the concrete. If this moisture gets in contact with reinforcing bars, there may be a high chance for the steel corrosion to occur, and further corrosion may weaken the concrete specimen which can undergo further flexural cracking.

Spalling – The concrete spalling process removes surface concrete layer surrounding diameter of 150 mm or more with the depth of 25 mm or more. It exposes the aggregate beneath the surface, leaving it uneven and pitted. Mostly spalling in concrete may result in pressure which occurs due to the rust of corrosion making more volume than the embedded reinforcing bars. According to Deeny et al. (2013), spalling is the breaking off layers or chunks of concrete from the surface during the exposure based on their experience with the behavior of concrete structures that display performance on fire. Further, they discussed that spalling could be due to three different ways. Aggregate spalling, corner spalling (or sloughing off), and explosive spalling are examples of these types of spalling. The structural performance does not affect aggregate spalling in this case. But in the case of fire, corner spalling and fierce spalling pose a more significant threat to the structural stability when the structure is subjected to fire. Thus, the thickness of the slab and concrete compressive strength depend on concrete spalling, according to Taillefer et al. (2013). Low-quality concrete covering reinforcing Steel, improperly planned connections, and bond failure in two-course construction due to shrinkage differences between the topping and foundation courses can all cause concrete spalling. Furthermore, according to numerous researchers, the best technique to avoid spalling is to apply a repair material mixture after cleaning and coating the corroded steel bars.

Figure 2.10: Remove damaged concrete, cut spalling concrete, brush steel bars, and coat steel bars (the constructor)

Freeze-thaw deterioration – Many researchers have investigated how concrete performs when exposed to high temperatures and how high-strength and standard concrete function mechanically. (Xie et al., 2019). However, water in pores or cavities expands so that making considerable pressure and it will be more than the tensile strength of the concrete. This dilation may help with freeze-thaw deterioration. (Parajuli, 2016)

2.3.2. Failure due to steel corrosion

Corrosion prevention of structural Steel is an essential part of the overall integrity and the aesthetic of the structure. Before setting up a corrosion prevention method for structural Steel, it should be understood how much corrosion has occurred on the Steel and the ways of corrosion. Corrosion control is just one part of the steel selection process. The durability of a structure depends upon the materials and quality of Steel; surface treatment and exposure conditions are the dependencies of the durability. (Rao, 2010.) High alloy and low alloy steel have two different qualities of the Steel itself. Naturally, the high alloy steel is more corrosive resistant than the low alloy steel, and the low alloy steel needs a more comprehensive coating system to apply effectively to fend off the corrosion. According to the society of protective coatings, coatings are more effective at controlling uniform erosion than controlling localized attacks. Also, the following facts should be taken into attention in the early design process of the structure.

- Reduce exposure to the atmosphere.
- Stay away from dissimilar metals.
- Prevent water from building up.
- Avoid surface irregularities.

Hence, an exemplary system will balance the performance level and service life cycle at the least cost.

2.3.1.1. Initiation of steel corrosion in RC concrete

Steel is a product of iron, and the transformation of iron into Steel necessitates the use of energy. Corrosion is the formation of iron oxide or rust when it releases energy and returns to its original state. Corrosion is an electrochemical phenomenon (Figure 2.11), and the corrosion process is shown in Figure 2.12. (Parajuli, 2016) According to Quraishi et al. (2017), Dhawan et al. (2014), Soudki et al., () the reactions take place at the anode and the cathode as shown below.

Anode: $Fe = Fe^{2+} + 2e$ (Metallic iron) (2)

$FeO.(H_2O)_x$ (Rust)

Cathode: $1/2O_2 + H_2O + 2e = 2OH^-$ (3)

If the oxygen level is limited, the rate of corrosion is reduced. The following reaction which happens at the anodic side shows the formation of rust after iron dissolution of the reinforcement bars.

$Fe^{2+}+2OH^- \rightarrow Fe(OH)_2$ (Ferrous Hydroxide) (4)

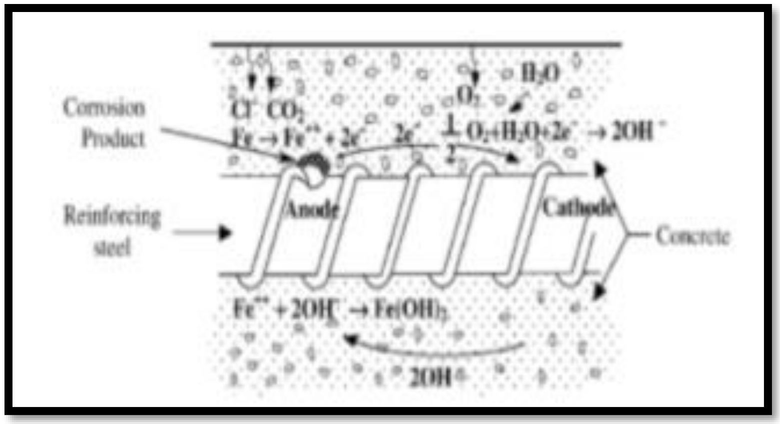


Figure 2.11: A simplified model of the electrochemical process of corrosion of Steel (Quraishi et al.,2017)

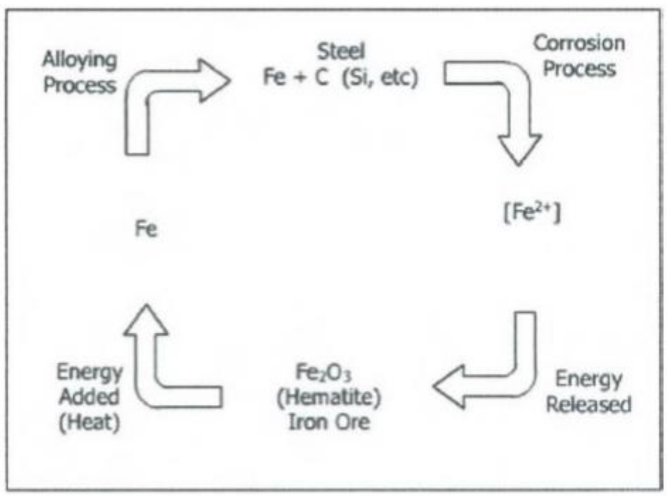


Figure 2.12: Extractive metallurgy in reverse (Parajuli B., 2016)

The loosing cross-section of the bars, volumetric expansion due to corrosion products, cracking concrete cover longitudinally, and delaminating the interface of bar-concrete due to formation of corrosion product are all direct effects of embedded reinforcement corrosion in structural concrete. In their paper "structural performance of corrosion damaged concrete beams," Cairns J. et al. (2008) pointed out that a direct effect of corrosion of embedded reinforcement in structural concrete could be seen as a loss of bar cross-section and volumetric expansion of corrosion. Several research studies have been carried out to establish the link between the degree of rebar corrosion and the flexural strength of the reinforced concrete after it has been affected by the rebar corrosion. (Lee et al.,1999; Fernandez et al., 2018; Craig Tahershamsi, 2016). The interrelationship of corrosion effect in deterioration is summarized in Figure 2.13.

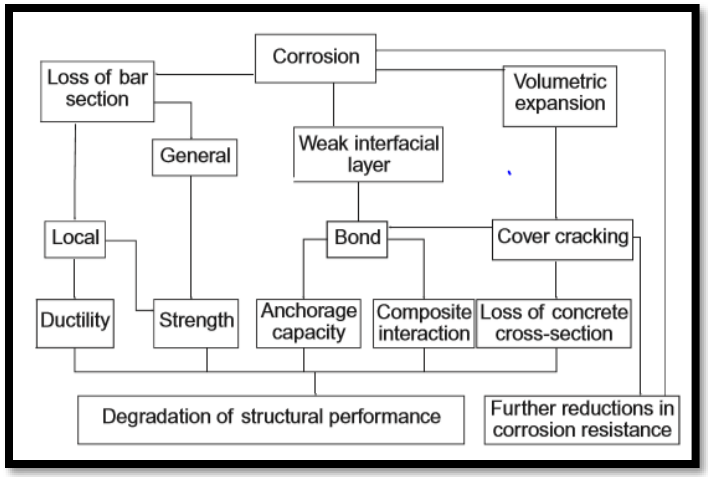


Figure 2.13: Corrosion effect of reinforcement on residual structural capacity (Cairns J. et al., 2008)

Several environmental factors may influence the durability of concrete. The most prevalent causes of degradation are corrosive substances such as deicing salts or saltwater salts. However, removing steel passivity can be a loss of alkalinity due to chloride attack or carbonation of concrete. Medeiros et al., (2015), Min Lee et al. (2018) have found that aggressive irons and climatic changes in the atmosphere may affect the deterioration of the embedded Steel in the structure. Carbonation leads to lower the alkalinity destroying the passive layer surrounding the reinforcing bars, which may cause corrosion. The carbonation process can be expressed as:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \qquad (5)$$

Owning to the atmospheric reaction of CO_2 with hydroxyl ion (OH^-), pore solution makes it acidic and results in the enhancement of steel corrosion, cracking, or spalling of concrete structures. Carbonation reaction in concrete to make $CaCO_3$ formation are given below.

$$CO_2+H_2O \rightarrow H_2CO_3 \qquad (6)$$

$$H_2CO_3+H_2O \rightarrow HCO^{-\,3} +H_3O^+ \qquad (7)$$

$$HCO^{-\,3} +H_2O \rightarrow CO_2^{-\,3} +H_3O^+ \qquad (8)$$

$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2H_2O$ (9)

Few researchers have studied the determination of carbonation depths. Lima and Medeiros conducted their research to find the carbonation depth in 57 years old concrete structures. Carbonation tests were performed on two specimens, and one is viaduct beams, and the other is columns after 57 years of exposure in an urban environment. The Fick's 2nd law to predict the carbonation penetration in concrete is given below.

$$x_{CO_2}(t) = k_{CO_2} \cdot \sqrt{t} \tag{10}$$

Xco_2 Carbonation depth (mm) Kco_2 Constant of carbonation

t time in years

They concluded that carbonation could behave differently in different locations. M. Tahershamsis' (2016) carried out his research of structural integrity when corrosion effects to the reinforcement in concrete structures and it provides a complete understanding of how it influences mechanical properties of reinforcement. The author has given two categories of corrosion as general and local. General corrosion occurs uniformly along the reinforcement, and generally, it may be caused by the carbonation. Nevertheless, localized corrosion is invariably associated with chloride contamination. Carbonation-induced corrosion, on the other hand, is slower than chloride-induced corrosion. (Ziga Smit, 2008)

Figure 2.14: Images of several types of rust damage. (When spalling occurred, localized corrosion (center) and overall corrosion owing to carbonation (left, right)) (Ziga Smit, 2008)

There are two forms of chlorides present in concrete, namely bound chloride, and free chloride. The damaged level at the rebar concrete interface using chloride ion concentration is shown in Table 2.3.

Table 2.3: Corrosion risk classification at different chloride levels (Quraishi, et al.,2017)

Serial No	**Risk of corrosion**	**Chloride Content (% wt of cement)**
1	Negligible	0.4
2	Possible	0.4 – 1.0
3	Probable	1.0 – 2.0
4	Certain	>2.0

Further, variation of volume of rust products are also given in Table 2.4.

Table 2.4: Volume variations of rust products (Quraishi et al.,2017)

Corrosion product	**Color**	**Volume in cm^3**
Fe	Earthy	1.3
FeO	Black	1.9
Fe_3O_4	Black	2.1
$Fe(OH)_2$	White	3.8
$Fe(OH)_3$	Brown	4.2
$Fe(OH)_3$, $3H_2O$	Yellow	6.4
Fe_2O_3	Red	2.5

Chloride penetration is caused by various accelerators and deicing salts present in the seawater which penetrate the protective oxide layer of the Steel, exceeding the permissible limits. Studies show that only water-soluble chlorides promote corrosion, and once it starts, chlorides are not actively involved in the corrosion rate, as shown in Figure 2.15. In addition to that, Figure 2.16 shows the process of corrosion by chloride irons and carbonation.

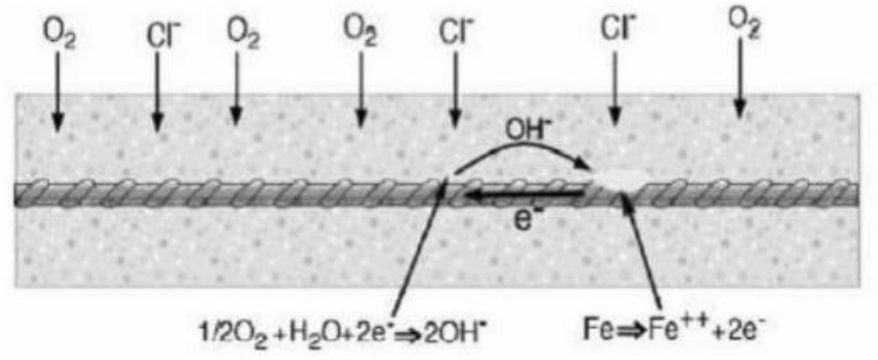


Figure 2.15: Chloride induced corrosion (Parajuli B., 2016)

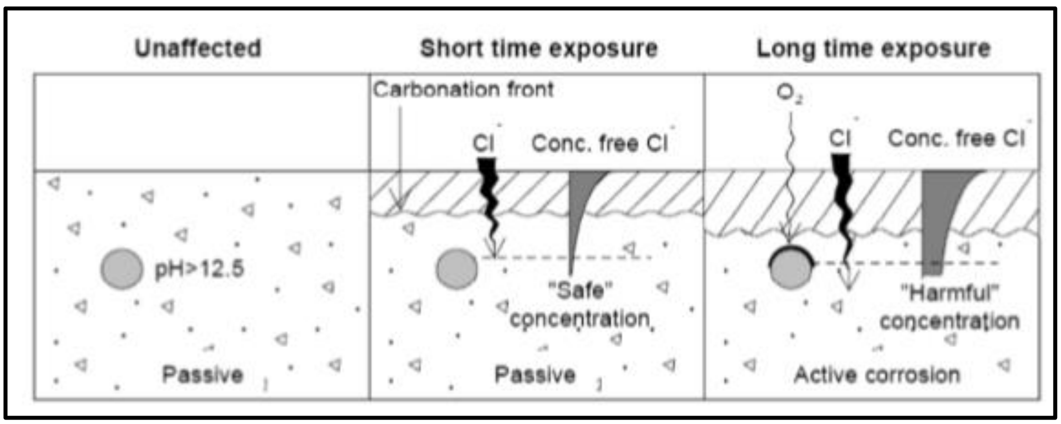


Figure 2.16: Corrosion by chloride irons and carbonation (Quraishi et al.,2017)

Alwash et al.,2019 has found that the carbonation and the chloride-induced corrosion of the embedded reinforced bars are vulnerable to the failure of concrete structures. According to their report, corrosion caused by several significant faults and corrosion of embedded Steel affect the mechanical behavior of the systems. Corrosion has harmed the stiffness, the serviceability, the ductility, and the load-bearing capacity. Moreover, corrosion of reinforced bars can significantly impact the structural safety by reducing the structures' carrying capacity and flexibility. Reduction in the ductility of the structure develops unexpected failures. If the bars get corroded, the average strain of the entire bar becomes lower than its' original bars. Thus, corrosion causes brittle behavior on reinforcement which results in failure at load. (Fernandez I. et al.,2018; Lee HS. Et al,1999; Alwash NA., et al, 2019; Tahershamsi M., 2016). Development of tensile stress due to volume expansion causes detrimental effects on the durability of the structure. For this reason, cracking and spalling of concrete may happen, as given in Figure 2.17. Further, the loss of concrete cover significantly reduces the load-bearing capacity of the structure. (Dhawan et al., 2014)

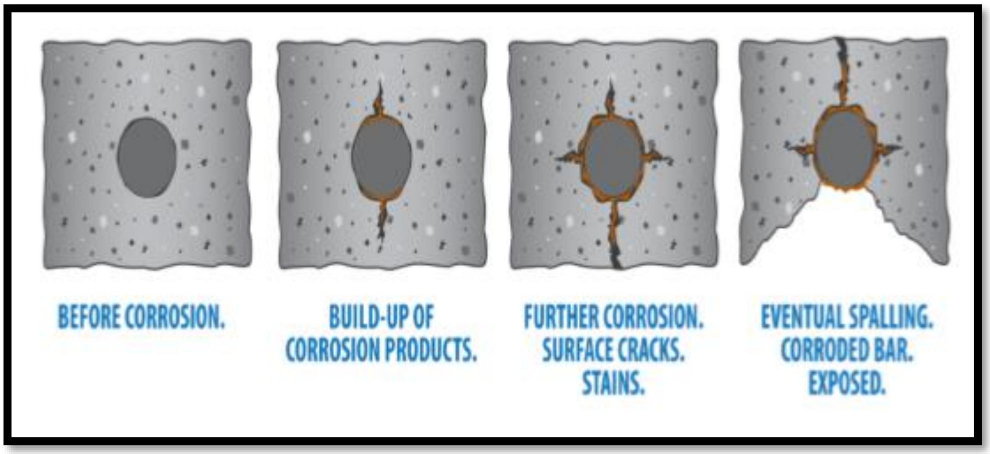


Figure 2.17: Progress of corrosion in concrete and eventual spalling (Dhawan et al., 2014)

In their work of "Concrete cracking prediction considering the filling proportion of strand corrosion products," Wang et al. (2016) has found that the crack widths have varied depending on the degree of corrosion. The triangle, rectangle, and trapezoidal stages of cracks are depicted schematically in Figure 2.18. The crack (crack A) inside concrete seems to be a triangle before cover cracking. Crack propagation (crack B) enters the surface as corrosion progresses. Cracks in the concrete surface occur, with a rectangle-shaped crack (crack C). When the fracture widens in the radial direction, it takes the shape of a trapezoid, as depicted in crack D.

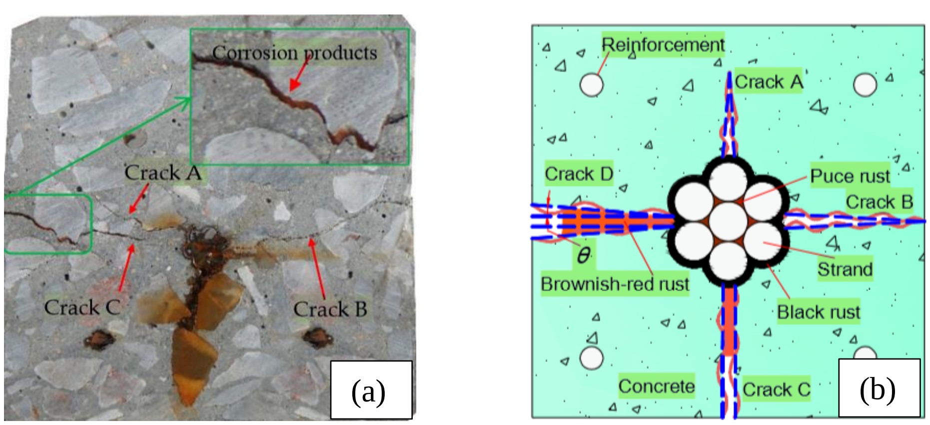


Figure 2.18: Schematic diagrams (a) Crack distribution (b) Simplified crack propagation (Wang et al., (2016))

2.3.2 Stress Corrosion Cracking in concrete

Several researchers studied stress corrosion cracking (SCC). (Vavpetic, 2008; Khalifeh, 2019; Ramamurthy, Atrens, 2013; Rao, 2019; Aly & Neto, 2014;) Stress corrosion cracking is usually produced by an aggressive agent when contacted with reinforcing Steel having mechanical stress. This sort of corrosion is most common in tendons that have been pre-stressed or post-tensioned. Deterioration can reduce the cross-section of the strands, create concrete cracking, reduce bond strength, cause pre-stress loss, and finally reduce the structural capacity of the PC. (Vavpetic, 2008, Dai et al. 2020). According to Dai et al., concrete cracking due to inducing corrosion can lower the bond strength and its losses the effective stress of prestress members by 6.6% from the initial stress. As a result, bond strength and effective pre-stress reduction linearly as corrosion loss approaching 34%. According to the research of Alireza Khalifeh (2019) & Aly & Mattar (2014), SCC is associated with three combinations such as tensile stress (applied and residual), environment, and some metallurgical conditions, as illustrated in Figure 2.19.

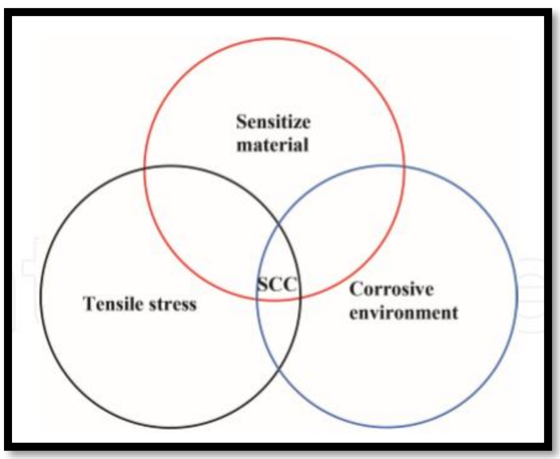


Figure 2.19: The essential requirements for SCC (Khalifeh A.,2019)

The stress corrosion cracking looks like brittle cracking which does not lose materials and visible corrosion products. It is represented as "river branched" (the crack of the primary stage is like a river on the surface of the material and secondary location, cracks look like a river with few branches). (Ramamurthy S. & Atrens, 2013; Aly & Neto,2014;) The report made by Vavpetic, (2008) mentioned three facts for the occurrence of stress corrosion cracking. It discussed the types of the steel erode due to stress corrosion, the point below at which the stress corrosion becomes extremely slow and the aggressive media which creates stress corrosion.

Figure 2.20 shows how a crack develops slowly but then spreads swiftly, resulting in an unanticipated failure. It is a very complex degradation process of initiation and propagation of stress corrosion cracking on the surface. The relationship among different factors in the corrosion process as shown in Figure 2.21.

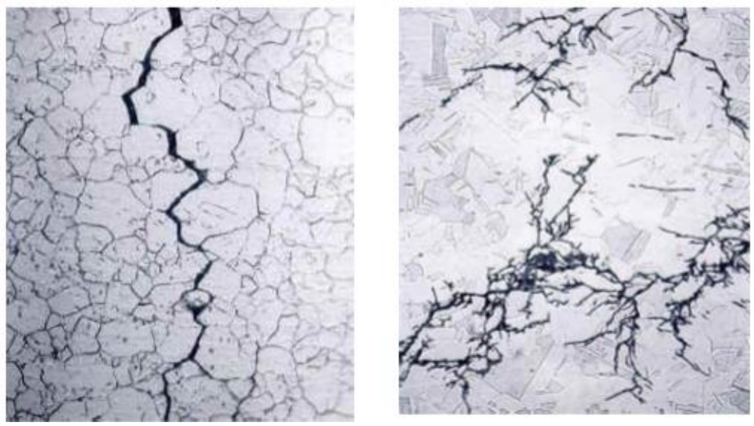
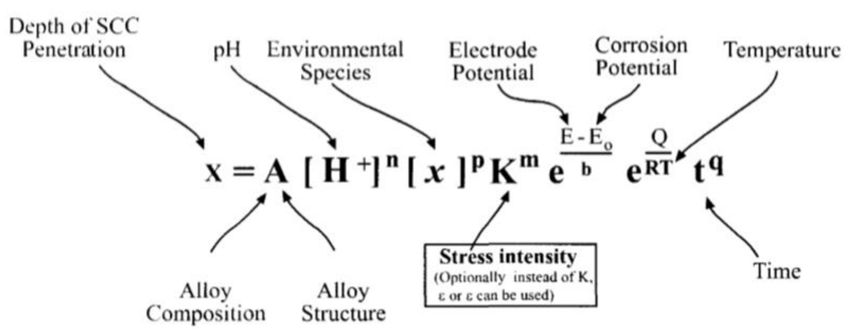


Figure 2.20: Start of stress corrosion cracking (Vavpetic, (2008))



(11

Figure 2.21: General relationship for SCC process (Aly & Neto,2014)

Cracks are the major phenomena when designing a reinforced concrete member. Calculation based limiting cracks cause for the unexpected brittle failures in reinforced concrete members. The abrupt reaction will have a dynamic effect, which could result in rebar rupture. Hence, the equation (12), given the minimum requirements that need to fulfil the brittle fracture of lightly reinforced concrete members and the conditions for the un-cracked state, are stated in the equation (2) for crack propagation in pure bending with an axially compressive force. According to Euro code 2, limited-width cracks in reinforced concrete structures do not indicate a lack of serviceability or durability (Apostolopoulos et al.,2019; Ahlgren & Edwin, 2017). If the concrete structure is pre-stressed, a compressive force may be added to the structure, and the propagation of cracks may get delayed. The compressive forces from the pre-stressing are more significant than the tensile forces from the service load in the case of full pre-stressing (Edwijn,2017).

$$A_{s,min} \geq 0.26 \frac{f_{ctm}}{f_{yk}} b_t d \geq 0.0013 b_t d \tag{12}$$

$A_{s,min}$ $minimum\ area\ of\ bending\ reinforcement$

f_{ctm} $mean\ axial\ tensile\ strength\ of\ concrete$

f_{yk} $characteristic\ yield\ strength\ of\ longitudinal\ reinforcement$

b_t $mean\ width\ of\ the\ part\ of\ the\ cross - section\ in\ tension$

d $effective\ depth\ of\ the\ cross\ section$

$$\sigma_{cn} + \sigma_{cm} \leq f_{ct,ft} \tag{13}$$

And where $f_{ct.ft} = k.f_{ctm}$

$$k = 1.6 - \frac{h}{1000} \geq 1$$

$k = height\ reduction\ factor$

$h = height\ of\ the\ beam$

The studies mentioned above focus on evaluating several reasons for joint corrosion impact. Typically bond failure in confinement reinforcing bars occur in two ways. Bond pullout failure can be expected due to the presence of sufficient cover and or transverse reinforcement. Concrete can withstand radial tensile forces generated at bond pullout failure by the bond transfer force. However, a loss happens by localized shearing of the concrete. The bond failure occurs when there is insufficient confinement and exceeding tensile strength of the concrete due to the development of longitudinal crack (Figure 2.22) parallel to the reinforcing bars. Loss of confinement and mechanical interlock result in the bond splitting failure, and cracking loosens that confinement in-between the concrete and the rebar. (Craig & Soudki, ;) Under typical circumstances, load cracks can form in PC beams. The JTG D62-2012 code (Ministry of Communication of the People's Republic of China 2012) and the Comite Euro-International du Beton—Federation International de la Précontrainte (CEB—FIP) Model Code 2010 (CEB—FIP 2011) define 0.10 and 0.20mm as critical cracks appeared at the serviceability limit state for PC members. (2019, Dai.) Further, several factors that make pre-stressing bars vulnerable to corrosion can be summarized as follows (Parajuli. 2016)

a. Voids in the vicinity of the tendons
b. The weakening of the passive alkaline layer of the concrete
c. Introduction of chloride irons in the concrete which starts the corrosion when get in contact with the water and the oxygen.
d. Unproperly sealed joints.
e. Insufficient concrete cover
f. High water-cement ratio caused Highly permeability in concrete.

In the NERD center slab system, the reasons for corrosion are majorly the insufficient concrete cover and the weakening of the alkaline protective layer of the concrete. But the point of high permeable concrete cannot be considered for the corrosion due to Grade 40 high strength concrete. In most cases, the slabs that deteriorate from the marine environment; chloride-induced corrosion may happen. Reinforced corrosion is thought to be limited to a mixture of acceptable quality of concrete, additive use, suitable cover, and crack width limits. Hence by the improvement of the above, the design life of the slab system can be enhanced.

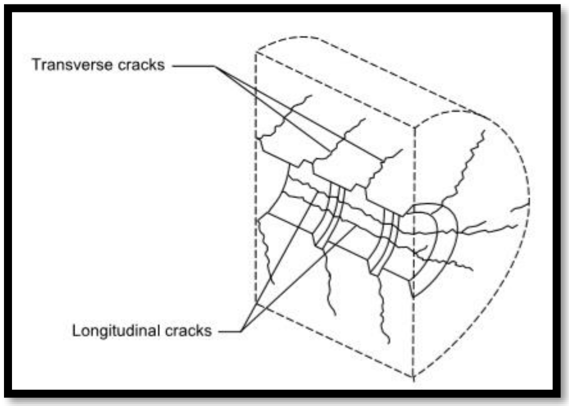


Figure 2.22: Longitudinal and transverse cracks caused by bond stresses (Tahershamsi, 2016)

According to previous research, less permeable concrete which is consolidated adequately with a lower value of water-cement ratio can greatly support to prevent the corrosion rate. Thus, if the Steel is treated with high electrical resistance of the epoxy coating reducing the cathodic area, the surrounding corrosion is minimized. Adequate concrete covering also minimizes the exposure to the environment. Further, proper attention is needed to avoid any defects from corrosion, and the end of pre-stressed steel should suitably be covered so that the tendons are protected from the deicing chemicals.

As seen in the preceding sections, pre-stressed member cracks appear on flexural members' top or bottom surfaces. If the cracks are large enough, pre-stressing tendons will erode with the cracks based under sustained or cyclic loads, which are applied on the member. If the thickness of the concrete member is less, it tends to crack and to spall, as mentioned earlier. Also, wide cracks lead to corrosion of pre-stressed tendons, and loss of pre-stressing makes structural integrity. The same situation is faced by the NERD center slab system which shows some shear and flexural cracks on the surface, and which should be minimized by using some strengthening techniques.

2.3.4. Degradation of pre-stressed RC structures

Pre-stressed structures are one of the significant findings in the world. It can avoid possible permanent cracks on concrete. However, pre-stress may tend to enhance the vulnerability of the member to fluster by reducing the natural frequency of vibration and wind exited oscillation (Mohammed, 2019). But, in the case of Steel, corrosion is one of the primary causes for its vulnerability as corrosion leads to structural deterioration.

Pre-stressed strand corrosion, on the other hand, would be much worse in pre-stressed concrete members than in reinforced concrete members. Unexpected brittle breakdowns occur from the collapse of deteriorated PC beams, causing in considerable economic losses. On the other hand, flexural cracks are regarded to be the most important element in forecasting flexural capacity (Dai et al., 2019). Consequently, the flexural flexibility of the corroded pre-stressed beams is critical for the serviceability and the safety.

The rust expansion ratio, which is demarcated as the volume of corrosion products divided by the volume of Steel used, can be used to assess the pre-stressed concrete structural deterioration. Many researchers have given little attention to corrosion-affected cracking in pre-stressed concrete compared to the reinforced concrete. Wang, et al., (2019), Costa & Appleton, (2002), Moawad, et al., (2016), Dai, et al., (2020), Apostolopoulos, (2018), Liu & Fan, (2019) are the few researchers who have carried out the research of flexural behavior of pre-stressed concrete under corrosion environment. The influence of modest corrosion rates (less than 2.87 percent) on flexural capacity is not substantial, according to Li and Yuan, but will result in a significant decrease of ultimate deflection for beams with wire rupture failure. According to Costa A., the life-threatening deterioration condition of the pre-stressing tendons is engendered by inferior materials and workmanship. As per the measurements, in very aggressive corrosion environments, corrosion rates are measured at 500 micrometers per year. Hence, supplementary protection measures must be employed where pre-stressing Steel is used. (Costa, 2002)

Concrete creep and shrinkage, pre-stressing strand stress relaxation, and corrosion are all potential causes of pre-tension losses. The effects of stress relaxation, concrete creep and shrinkage, and long-term pre-stress losses in pre-stressed concrete have all been studied extensively (Bruce et al., 2018, Apostolopoulos et al (2003)). Several regulations also provide methods for analyzing long-term pre-stress losses. The following three components that lead to pre-stressing steel corrosion are discussed in detail in Chapter 2.2-2.4 of ACI 222.2R-01 (American Concrete Institute 2005). (Source: Bruce et al., 2008)

- Metal characteristics (these have a minor influence on corrosion resistance)
- The concrete that surrounds the wire or strand's quality (this has a significant impact on corrosion resistance)
- Conditions of service (these have the most critical influence on corrosion resistance)

According to some researchers, the pre-stressing force in pre- or post-tensioned concrete beams is proportional to the corroded strand's cross-sectional area being reduced. Apart from the reduction of strand's cross-section, cracking of concrete and degradation of bonding are additional factors affecting the pre-stress loss. Furthermore, pre-tensioned concrete members have a greater impact on pre-stress loss than post-tensioned concrete members. In the electrochemically accelerated corrosion situation, the strand rust expansion ratio proposes to be 2.78 (Wang et al.,2019; Dai et al. 2020; Moawad et al.,2016).

The above research focuses on the fact that high-stress levels in the weld bead will accelerate stress corrosion cracking. Figure 2.23 depicts how corrosion expansion causes tensile stress to expand in the circumferential direction, causing the concrete to crack.

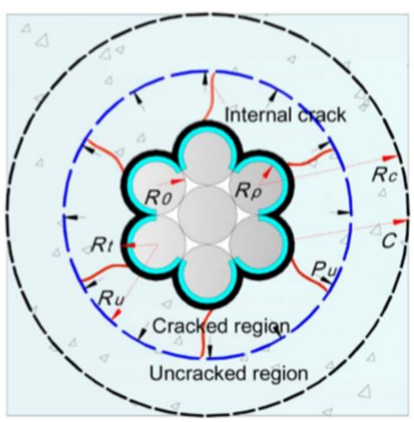


Figure 2.23: Strand corrosion causes concrete to crack.

Some of the corrosion-produced rust fills holes and crevices, while the rest contributes to the expansive pressure. According to some experts, x-ray and digital image processing techniques could be utilized to measure the amount of the corroded rebar loss of weight and fracture widths. Table 2.5 and 2.6 shows that the chemical composition and mechanical characteristics of the steel which is used in pre-stressing tendons. However, the evaluation of pre-stress loss caused by corrosion cracking in pre-tensioned concrete members needs further study.

Lei Wang and his team developed a model to predict concrete cracks under the combined effects of pre-stressing and steel reinforcement. They investigated corrosion-induced cracking in a variety of stress states using four pre-stress levels: 0, 0.25fp, 0.5fp, and 0.75fp (where fp was 1860 MPa). As shown in Table 2.7 below, the specimens are split into three groups based on corrosion time and building technology.

Table 2.5: Chemical composition of steel in terms of weight percentages

Type	**Fe**	**C**	**Mn**	**Si**	**P**	**S**	**Cr**	**Cu**	**Ni**	**Ti**	**Al**
Strand	97.862	0.82	0.74	0.21	0.012	0.006	0.17	0.09	0.03	0.03	0.03
Reinforcement	97.849	0.2	1.34	0.55	0.033	0.028	-	-	-	-	-

Table 2.6: Mechanical Characteristics of steel

Type	**Diameter (mm)**	**Yield strength (MPa)**	**Ultimate strength (MPa)**	**Elastic modulus (GPa)**
Strand	15.2	1830	1910	195
Deformed bars	6(8)	400	540	200

Table 2.7: Parameters of specimen

Type	**Pre-tensioned concrete**				**Pre-tensioned concrete**				**Post-tensioned concrete**			
	Group A				**Group B**				**Group C**			
Beam No	PA0	PA1	PA2	PA3	PB0	PB1	PB2	PB3	PC0	PC1	PC2	PC3

Prestress (MPa)	0	$0.25f_p$	$0.5f_p$	$0.75f_p$	0	$0.25f_p$	$0.5f_p$	$0.75f_p$	0	$0.25f_p$	$0.5f_p$	$0.75f_p$

The findings of the experiments show that steel strand corrosion and fracture width vary in numerous ways. According to the thick-walled cylinder theory, corrosion-induced cracking can occur when the tensile strength of the concrete is exceeded. It was also discovered that the increase in the strand tensile strength from 0 to 75% reduces the life-threatening time of cover cracking by 22% while increasing the crack propagation rate by 9%.

The model performed the four-point bending test is used to find out load-deflection behavior of the corrosion specimen, as illustrated in Figure 2.24. The test specimens which were used for the tests also are shown in Figure 2.25. Dai and his research group (2020) carried out the same experiment as mentioned above. The load cell from four-point bending test was used to measure apply loads consistently. Vertical deflections at the support, mid-span and loading points were measured using digital dial gauges. According to the experimental results, that model is proved the effect of concrete cracking and bond deterioration. They also concluded that high strand tension would hasten the corrosion-induced loss of pre-stress.

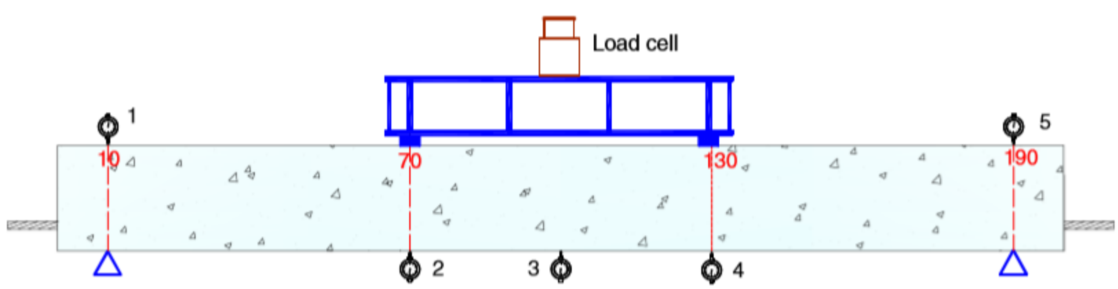


Figure 2.24: Diagram of load testing

Figure 2.25: Accelerated corrosion device

Moawad performed another test procedure on partially pre-stressed and corroded concrete beams. And his group stated that their experimental program has comprised of six beams with the sizes of (150mm x 400mm x 4500mm). The strands were stressed after 28 days of concrete casting. The test arrangement was divided into two parts. Six partially pre-stressed concrete beams were cast in the first step, and the four beams were corroded in the second step. As illustrated in Figure 2.26, the loads were applied as two concentrated loads using two hydraulic jacks with a capacity of 800 kN each.

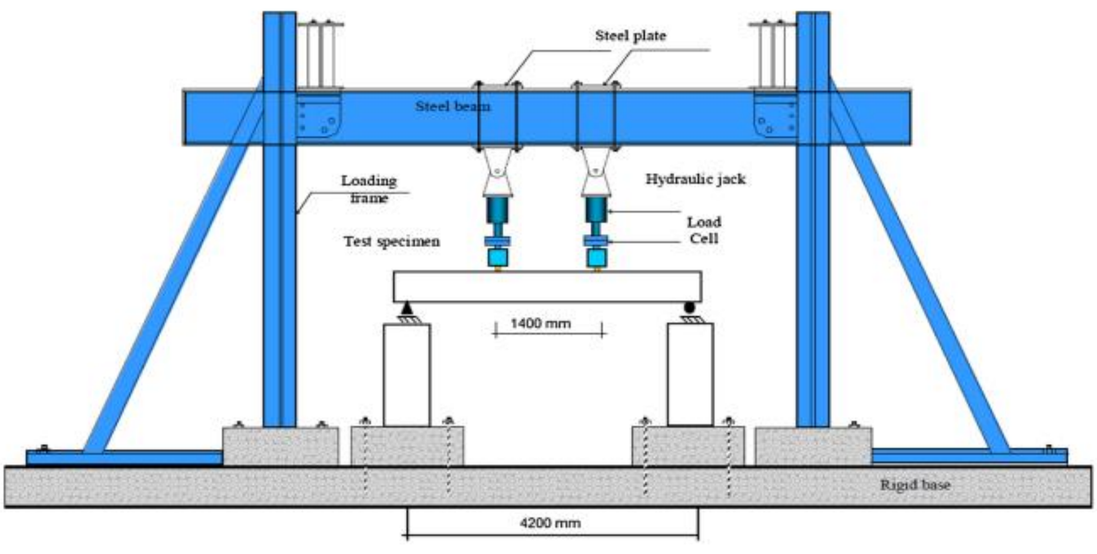


Figure 2.26: Test setup for tested beams (Moawad et al.,2016)

The failure of all parts of the pre-stressed specimens begins with the yield of the lower main reinforcement and then the fracture of the top concrete. The cracking pattern of the partially pre-stressed beams has appeared as irregular cracks at the concrete cover with varying thicknesses as illustrated in Figure 2.27. The crack pattern of the partially pre-stressed beams has scattered along their entire length.

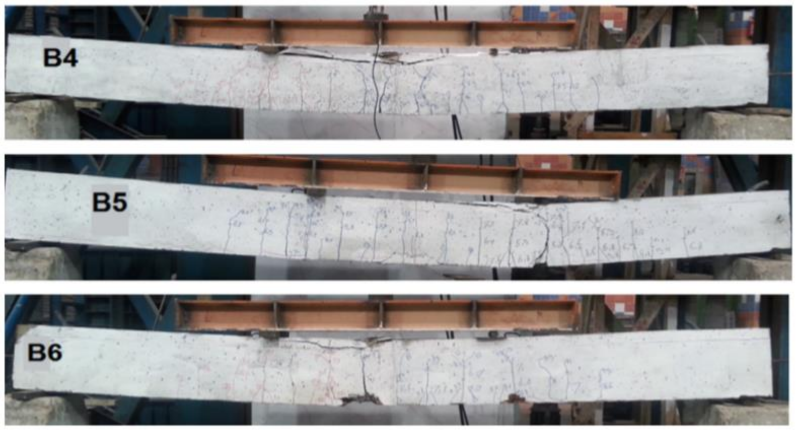


Figure 2.27: Crack pattern at the controlled specimens

Furthermore, they tested several samples which were partially exposed to corrosion. Consequently, they observed that a smaller amount of variance in behavior and also ultimate loading capacity and propagation of cracks along the beam. Figure 2.28 gives a clear image of cracks due to excessive stresses of corrosion.

Figure 2.28: the crack pattern of corroded concrete specimen

They concluded that non-pre-stressed steel reinforcement has a greater likelihood of deterioration unless partially pre-stressed tendons are used. Moisture and chloride cannot penetrate the pre-stressed tendons because of the cement grout. Ductworks, especially plastic ducts with no splices or connections, serve as a moisture and chloride barrier. Epoxy coating or complete grout filling in the plastic duct is an adequate protection solution for non-pre-stressed Steel.

2.4 Fibre Reinforced Polymer (FRP) materials

2.4.1 Background of FRP materials

Corrosion is one of the main durability issues which is faced by concrete infrastructure in the world. Many structures in harsh environments have experienced severe serviceability losses, or the need for replacement, refurbishment, or reinforcement occurred considerably faster than the projected time due to corrosion. As Steel corrodes, its cross-sectional area reduces, and two primary criteria are caused for this discrepancy of concrete. Second, corrosion produces a greater volume of rust than the original volume of steel, putting large tensile forces on the concrete around it, causing it to crack and spall. Expansive forces generated by corrosion can cause concrete to fracture, spall, and discolor, causing the reinforcement-concrete structural relationship to fail. If the stress corrosion cracking is prevented or suppressed to a certain extent, the structural strength of the corroded RC beam can be maintained (Soudki,2016).

Scientists pay their attention to repairing damaged and deteriorated buildings with realistic option such as fiber-reinforced polymer (FRP) systems. They chose that option due to its high tensile strength, lightweight, resistance to the corrosion, high strength, and easiness of installation. Traditional strengthening processes such as steel plate bonding, section expansion, and external post-tensioning have replaced FRP as a cost-effective solution for reinforcing and rehabilitating concrete structures (Aktas & Gunaslan, 2017, Grace et al.,1999). FRP is the important consistency of fibers that are substantial/stiff enough to transfer the loads together with a solid or weak interface. Because the matrix (binder) is the composite's stress moving element, it will allow for seamless load transfer between broken or damaged fibers and neighboring unbroken fibers (Ahlgren & Edwijn, (2017)). Hence, according to the Soudki et al., (2016), structural performance will be improved with combination of following mechanisms.1) Limiting concrete cross-sections reduces stress corrosion cracking and joint cracks, 2) Avoiding additional chloride penetration into concrete. As a result, the corrosion rate is reduced. To compensate for the loss in steel cross-section, higher flexural and shear resilience was added.

2.4.1.1 Material properties

The previous study has shown that the availability of explicit design guidelines, installation techniques, and building specifications are necessary for FRP strengthening. Although detailed design standards for traditional construction materials are available, FRP design specifications for flexural and shear strengthening are still working. FRP composites usually made of two entities, such as epoxy and fibers which are types of resins. The fibers are mixed with different materials to give mechanical properties to the FRP higher than the pre-stressing steels which has received more attraction from structural engineers. FRP composites combined with three composites, namely Glass, Carbon, and Aramid and the typical densities of FRP are as indicated in Table 2.6. Table 2.6 clearly demonstrates that the thickness of the Steel is more than that of the FRP composites. As a result, a significant reduction in density occurs which is a desired attribute as it lowers the structure's transportation and handling costs as well as its dead load (Setunge et al.,2002; Dasgupta 2018; Erki., et al.,1993). According to the Erki and Rizkalla (1993), further explanations can be given for the general advantages of FRP compared to Steel as below.

- High ratio of resistance to specific gravity. It is given that the density of FRP is more significant than ten to fifteen times the Steel.
- Fatigue characteristics of carbon and aramid fibre reinforcement are three times higher than Steel. However, glass FRP reinforcement has significantly below fatigue characteristics to Steel.
- Excellent resistance to the corrosion and neutrality to the electromagnetic power.
- Low axial thermal coefficient expansion, especially for CFRP composite materials.

Table 2.6: Difference in density of FRP composite & steel

Steel	**GFRP**	**CFRP**	**AFRP**
7900	1200 - 2100	1500 - 1600	1200 - 1500

Table 2.7 shows a better comparison of CFRP, BFRP, and GFRP reinforcements. Siddika et al. (2019) has found that BFRP has more strain control capability at failure than CFRP. Due to the high volume of silica, GFRP is extremely vulnerable to alkaline environments, and CFRP is extremely resistant to alkali forms. CFRP has chosen as a material of strength improvement and expansion resistance of concrete over GFRP or AFRP due to its high stiffness. As stated by the ACI committee report, "FRP can be used to rehabilitate or restore the strength of a deteriorated structural member, to retrofit or strengthen a sound structural member to resist increased loads due to address design or construction errors". However, code says that, before application, there should be some vital field investigations.

In addition to that, Figure 2.20 shows material characteristics of CFRP, GFRP, AFRP, and steel pre-stressing tendons. The figure shows that CFRP, AFRP and GFRP give linear behavior of tensile stress vs tensile strain graph. Pre-stressing wires behave like steel when subjected to the tensile forces as usual. After certain point, pre-stressing wires cannot receive higher loads.

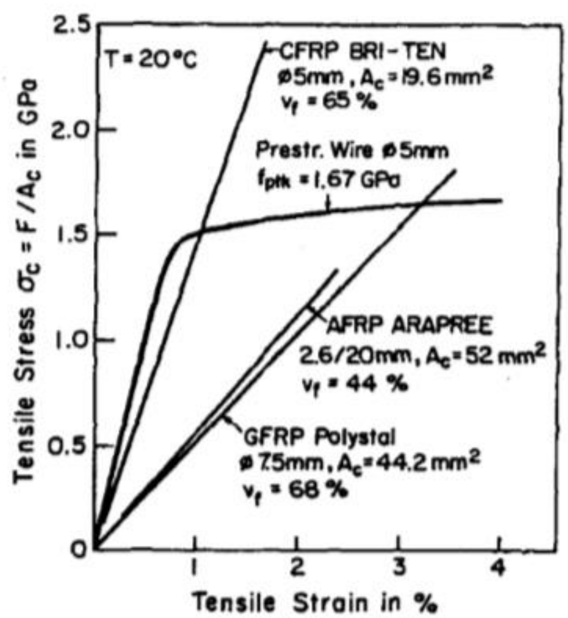


Figure 2.20: Material characteristics of CFRP, GFRP, AFRP, and Steel pre-stressing tendons (Erki et al.,1993)

Table 2.7: Properties of FRP composites (Siddika et al., (2019)

FRP Type	Unit weight (g/m^2)	Thickness of CFRP (mm)	Tensile strength (MPa)	Elastic modulus (GPa)	Rupture strain (%)	Ultimate strength (MPa)
GFRP bar	–	–	483–1600	35–51	1.2–3.1	–
CFRP bar	–	–	600–3690	120–580	0.5–1.7	–
AFRP bar	–	–	1720–2540	41–125	1.9–4.4	–
CFRP sheet	–	0.4	–	84	1.25	1050
CFRP sheet	200	0.115	3790	230	–	–
GFRP sheet	915	0.36	3240	72.4	–	–
CFRP	340	0.45	1548	89	1.74	–
CFRP sheet	300	0.165	4800	230	–	–
CFRP sheet	–	0.86	609	63.3	0.96	–
CFRP sheet	–	0.165	3550	235	1.50	–
GFRP sheet	–	0.353	1700	71	2.0	–
CFRP	–	1.4	2850	168		–
CFRP sheet	–	0.131	–	238	0.015	4300
CFRP Strip	–	1.2	–	165	0.017	3100
GFRP sheet	–	0.131	–	72.5	0.04	2276
CFRP sheet	–	0.111	–	242	1.7	4103
GFRP sheet	–	0.273	–	73	2.7	3400
CFRP sheet	–		–	212	1.58	3350
CFRP sheet	–	0.167	3522	258.9	–	–
BFRP sheet	300	0.12	1684	–	2.1	–
CFRP sheet	340	0.45	1500	–	1.65	–
CFRP sheet	–	0.348	2089.4	119.25	1.7	–
GFRP sheet	–	0.352	786.5	34.13	3.5	–

In general, strengthen concrete beams with FRP composites contribute significantly to improving the fatigue life, serviceability of the beam, and flexural strength as compared to un-strengthened beams. The usage of pre-stressed FRP improves ultimate and serviceability limit capacities significantly. According to research, pre-stressed FRP permits materials to better use their tensile capacity, which boosts the structural strength of degraded constructions. A variety of strengthening techniques such as external post-tensioning (EPT) using anchorage and non-anchorage systems, near-surface mounted (NSM), externally bonded reinforcing (EBR), have been used to investigate (Aslam et al., (2015), Karthick, et al., (2017)). Aslam et al. (2015) proposes of the strengthening process which is shown in Figure 2.21.

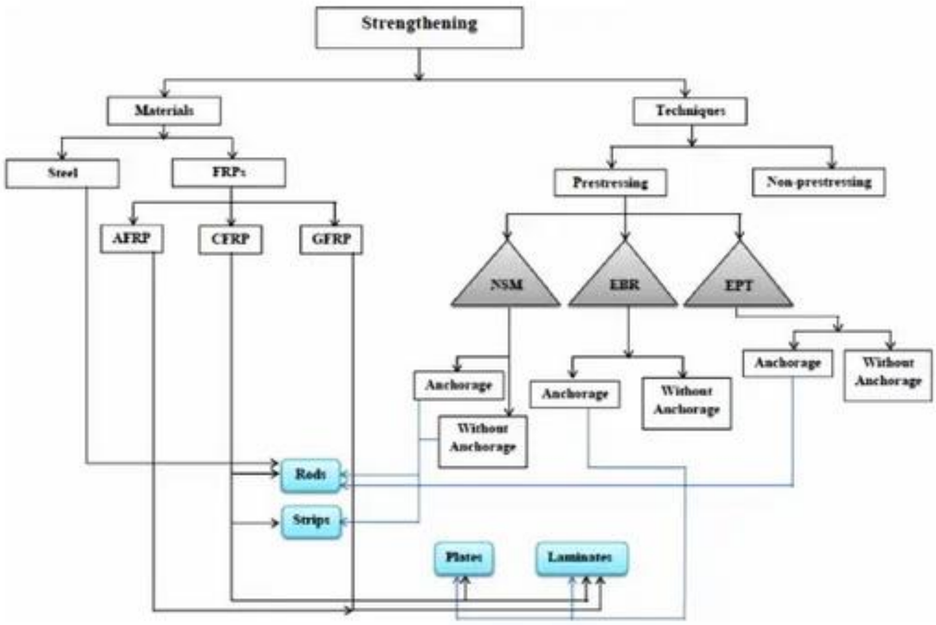


Figure 2.21: FRP Strengthening process.

Many researchers have paid their attention towards strengthening exhausted structures by the application of FRP for the last two decades. Rodrigo, 2015 researched old structures that had fallen below their carrying capacity due to the loss of shear strengthening. Wrapping CFRP sheets over prestressed concrete beams, spraying GFRP coatings, and embedding FRP bars deeply to the concrete beams are all listed as ways to improve overall structural integrity and strength. The design guidelines as listed in the European FIB report for externally bonded FRP composite reinforcement and the American ACI guidelines for strengthening of concrete can be reviewed under the functional class 03 of the Austroads code classification. The ACI material compliance standards are listed in the Table 2.8 below, and the Figure 2.22 depicts the stress-strain relationship for several materials at the ultimate loading condition.

Table 2.8: ACI compliance of the design materials (Rodrigo, 2015)

Material used	Design strength (MPa)	Elastic modulus (MPa)	Allowable strain
Concrete steel bars	21 (β_1= 0.91)	26,100[a]	0.003
Steel bars	400	200,000	0.002
CFRP strips (flexural)	0.85 * 2,800 = 2.380	165,000	0.85 * 0.017 = 0.01445
CFRP wrapping (shear)	0.85 * 3500 = 2,975	230,000	0.85 * 0.015 = 0.01275

[a] The loan term modulus of elasticity of 13,050 was used to account for creep of concrete.

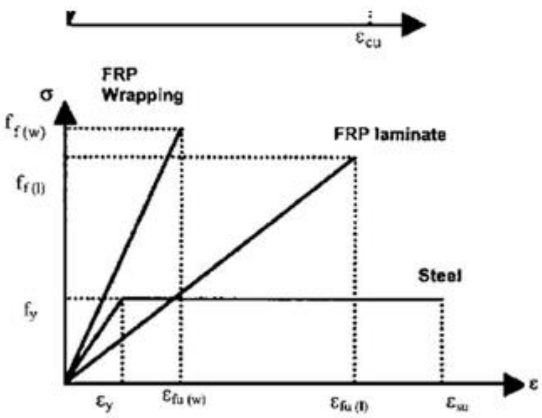


Figure 2.22: Stress-strain relationship under ULS for various constitutive materials (Rodrigo, 2015)

Also, researchers consider the possibility of FRP de-bonding and delamination prevention of FRP laminates which are calculated according to the guidelines of ACI as below. The equations below show several parameters, such as,

$$k_m = \frac{1}{60\varepsilon_{fu}}\left(\frac{90{,}000}{nE_f t_f}\right) < 0.9 \tag{14}$$

$$\varepsilon_f = \varepsilon_{cu} \frac{h-x}{x} - \varepsilon_0 \leq K_m \varepsilon_{fu} \tag{15}$$

(Rodrigo, 2015)

$k_m = bond\ depended\ coefficient$ $\varepsilon_{cu} = ultimate\ strain$

$\varepsilon_f = elastic\ modulus$ $\varepsilon_0 = initial\ strain\ before\ straining$

$T_f = thickness\ of\ the\ FRP\ strips$ $\varepsilon_{cu} = ultimate\ strain$

The engineers also used the following equation to compute the RC's design bond shear strength.

$$f_{cbd} = 1.8 \frac{f_{ctk}}{\gamma_c}$$

$$\varepsilon_{sl} < \varepsilon_{yd}: \frac{V_d}{0.95 d b_f \left(1 + \frac{A_{sl} E_s}{A_f E_f}\right)} < f_{cbd}$$

$$\varepsilon_{sl} \geq \varepsilon_{yd}: \frac{V_d}{\frac{Z_s + Z_f}{2} b_f} < f_{cbd} \tag{16}$$

(Rodrigo, 2015)

Strengthening with non-pre-stressed FRP sheet/plate on RC beams improves flexural capacity and stiffness, as shown in Figure 2.23.

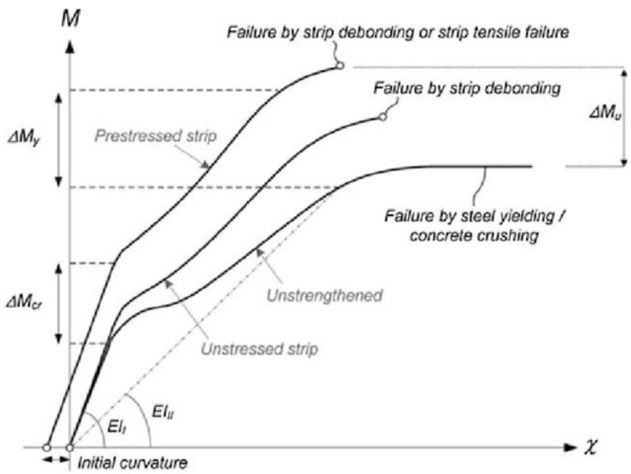


Figure 2.23: use of pre-stressed strip for flexural stiffness

It is evident from Figure 2.23 that FRP reinforcement shows higher value of modulus of elasticity to the lower value of average tensile strain. Lower bending moment capacity shows higher strain distribution along the section when the failure occurs in normal concrete with steel yielding. Hence, FRP application is more viable solution.

The maximum strength and general performance of reinforced concrete beams are significantly enhanced. It can reduce cracks evenly distributed over the RC beam, and summation of the minor damages are also reduced. If the FRP laminate is not sufficiently anchored to the RC beam, it will de-bond in a higher shear stress zone. Hence, prevention can be done by using special stirrup and anchorage systems (Ahlgren, 2017). Further, it is observed that flexural strengthening, shear strengthening, and axial confinement can be done using CFRP fibers and CFRP laminates, as shown in Figures 2.24.

Figure 2.24: Application of CFRP for (a) flexural stiffness (b) shear strengthening (c) axial confinement.

2.4.1.2 Surface preparation

In recent time, structurally deteriorated or damaged structures such as bridges, buildings, have been motivated by the researchers to find most efficient and cost-effective technique of refurbishing affected structures. FRP has been identified as an excellent strengthening material for retrofitting reinforced structures among other numerous ways. Due to its superior to resist of corrosion, high strength to weight ratio, high mechanical strength, low weight, and fast and inexpensive ways of restoration, many extensive researchers have carried out their studies through the application of FRP for retrofitting of structures. According to them research, surface preparation is most important part in achieving strong interaction between FRP and concrete successfully. According to the ACI 440.2R, externally bonded FRP should not be applied on corroded and contaminated environment. Cracks that are 0.3mm should be sealed with epoxy injection. Smaller cracks less than 0.3mm can be sealed with resin to prevent corrosion. All rough corners should be smoothed with putty to prevent stress concentrations before applying FRP on clear environment. If FRP application is done as the NSM system, the grooves should be cleaned, cleared, and ensured that it has adequate bond with concrete. Cutting groove should not be damage existing steel reinforcement. Application and mixing procedure should be in accordance with the specifications provided by the manufacturer.

2.4.1.3 Failure modes of FRP

As shown in Figure 2.25, there are five different failure scenarios when reinforcing concrete is enhanced with CFRP laminates.

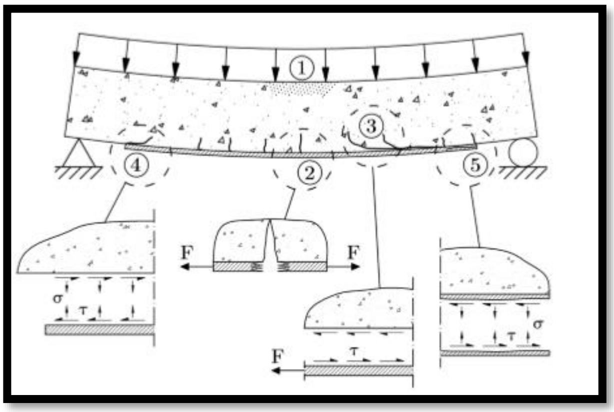


Figure 2.25: Failure scenarios in concrete with externally bonded CFRP. 1) Concrete crushing. 2) CFRP laminate rupture. 3) De-bonding of adhesion. 4) De-bonding of the endplate 5) Delamination of FRP. (Ahlgren P. & Edwijn J., 2017)

Several factors that can be caused for de-bonding CFRP laminate from the concrete. Crushing can occur when a beam is excessively reinforced or when the ultimate load of concrete reaches before reinforcement begins to yield according to the studies by Ahlgren P. & Edwijn J. (2017). A rupture could occur because of the CFRP brittleness failing. However, if the adhesive is correctly applied, still failure can occur by reason of the cover separation from the concrete, as shown in Figure 2.28.

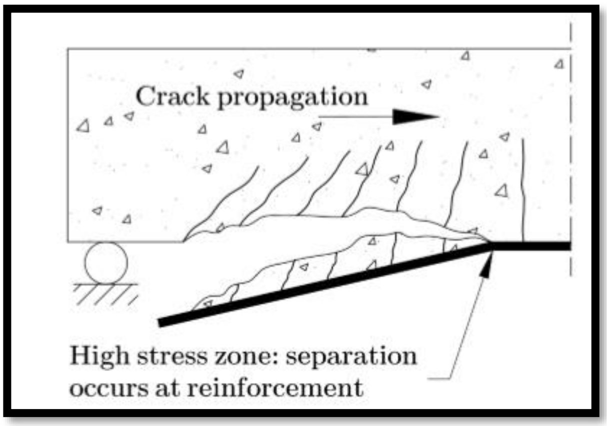


Figure 2.28: Concrete cover separation (Ahlgren & Edwijn, 2017)

2.5. Retrofitting RC structures with FRP

2.5.1. Corrosion- damaged reinforced concrete beams repaired with FRP.

Several research have studied the behavior of pre-stressed concrete beams under cyclic loading. Pre-tensioned pre-stressed concrete was tested before and after loading by Budiono. Four original and four repaired pre-stressed specimens were used in this study, and they were all exposed to quasi-static reversed cyclic loading. The original beams were loaded until they failed, then repaired using various techniques and re-tested under the same stress conditions. Control and restored specimens' behavior was examined and studied.

The specimen which was repaired had no fracture occurred at the pre-stressing strand. Holes were cleaned and injected with liquid polymer grout into the holes. The additives were employed to reduce the time required for the concrete to reach its strength from 28 to 7 days. The crack Patten they observed is given in Figures 2.29 and 2.30. This experiment concluded that a similar flexure and shear cracks pattern developed in both repaired and control beams. More considerable deflections were observed in which polymer grouting and re-cast concrete specimens. Further, they detected that the strength degradation of repaired beams due to cyclic loading was more severe than the original beams.

Figure 2.29: Crack pattern of original specimens (Budiono et al., 2000)

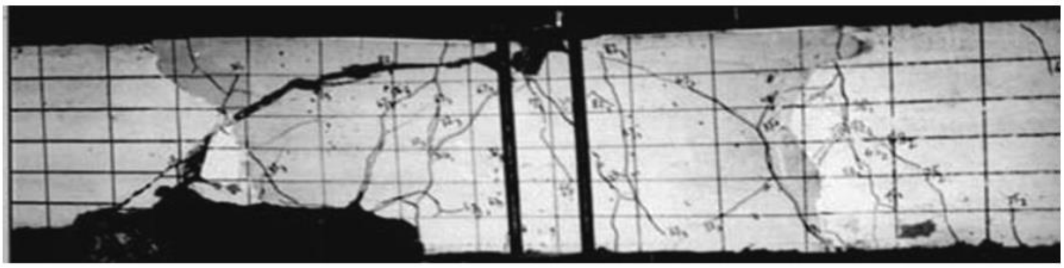


Figure 2.30: Crack pattern of required beams (Budiono et al.,2000)

One of the pilot projects was carried out by the Timisoara University of Technology. Float SC and his group were concerned about the retrofitting of two-way RC slabs using composite materials. Flexural behavior of two-way slabs without openings with uniformly loading was tested in this experiment. In most situations, RC slabs need spaces where they are not considered during the design of the building. These situations lead to unexpected failures and retrofitting can be done using FRP. Before retrofitting, all the specimens were tested up to a particular stage. After that, a mixed retrofitting solution of NSMR—FRP and EB-FRPP was applied as shown in Figure 2.3. They concluded that a 37.3% load capacity increment can be achieved.

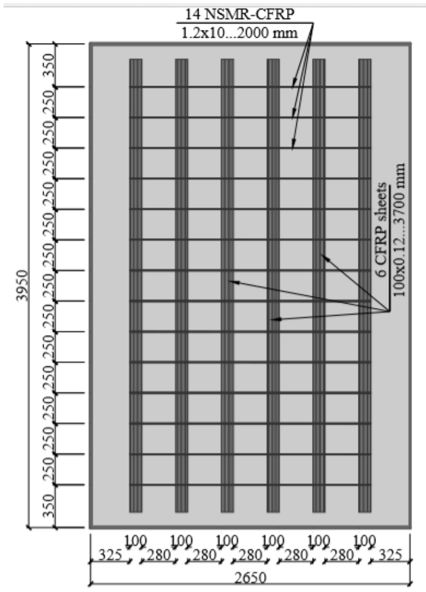


Figure 2.31: Retrofitting solution for full slab (Florut et al.,2007)

In 2017, Gribnyak et al. explored the structural stability of reinforced concrete beams strengthened with external carbon plates. The researchers successfully investigated the load resistance and failure modes of composite beams after application of externally bonded CFRP sheets. As shown in Figure 2.32, two different layouts of 2 x 6 pins (symbolized by A12) or 2 x 10 pins (symbolized by A20) were employed. The load-bearing capacity rose by 0% over the reference beams IB1Ref and IIB1Ref, while the pins resisted the shear stresses, de-bonding near the life-threatening cracks occur within the pure bending zone.

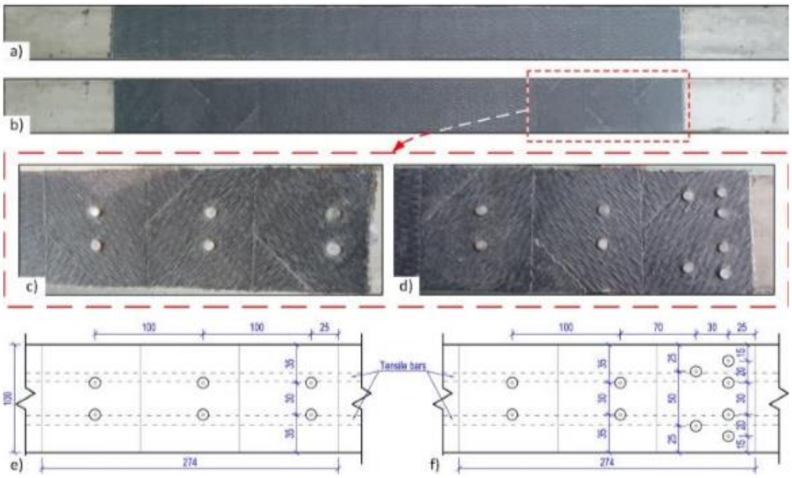


Figure 2.32: CFRP sheets without fastening are shown in a), with additional layout for hybrid jointing are shown in b), with layouts of 2 x 6 pins and 2 x 10 pins are shown in c) and d); and the respective dimensions of the joints are shown in e) and f)

Ahemed et al., 2011, evaluated the flexural performance of CFRP enhanced RC beams with various degrees of strengthening techniques. Figure 2.33 shows how CFRP laminates are stacked. When the second and third layers of CFRP plates are used, the ultimate load and the stiffness of the beam can be significantly increased. They concluded that beams strengthened with various CFRP layers couldn't reach their total flexural capacity because of the influence of detaching and premature de-bonding before achieving the beam's ultimate flexural capacity. However, when edge strips were used, the mode of failure altered from flexural (steel reinforcement yielding) to abrupt rupture of the CFRP laminates.

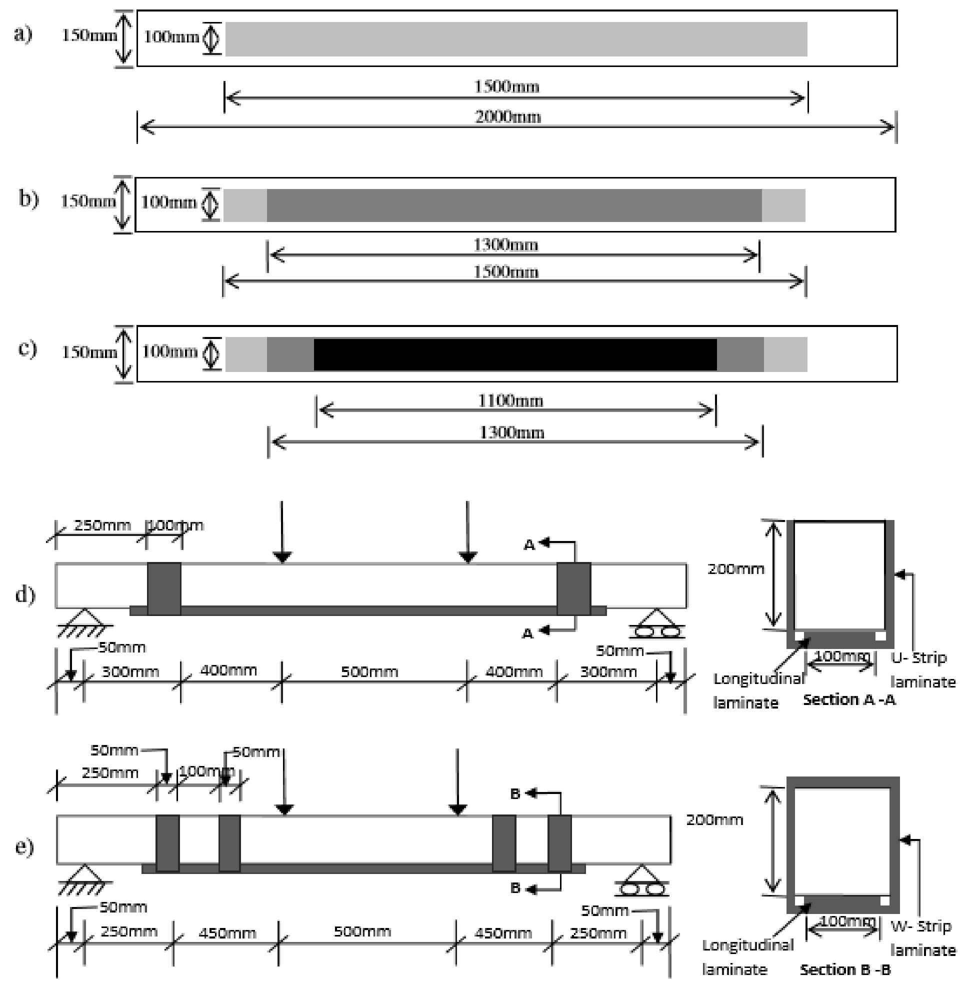


Figure 2.33: RC beam strengthening schemes: CFRP laminates comprising a) single, b) double, c) triple-reinforced beams with one U-shaped stripe and two W-shaped edge strips.

2.6. Summary of Literature Review

To conclude, shear and flexural strengthening can be done by using the CFRP application. The problem with de-bonding in the above investigations shows that U socketing can minimize the de-bonding failure of the beam. But in the case of shear strengthening, it can be eliminated by complete wrapping with CFRP, angle of application (refer to Figure 2.35), U wrap (refer to Figure 2.34), or U jacketing. (Kim et al.,2012). FRP reinforcement can be applied to the tension face of a concrete flexural member to increase flexural strength, and the fibers should be aligned along the length of the member. (ACI 440.2R-08, pp 21). Also, testing one-way slabs in bending subjected to concentration loads will introduce much higher curvature in the slab than the two-way slabs. According to Azizi and Talaeitaba, 2019, the EBROG sharing methodology is more effective than the EBR surface mounting technique.

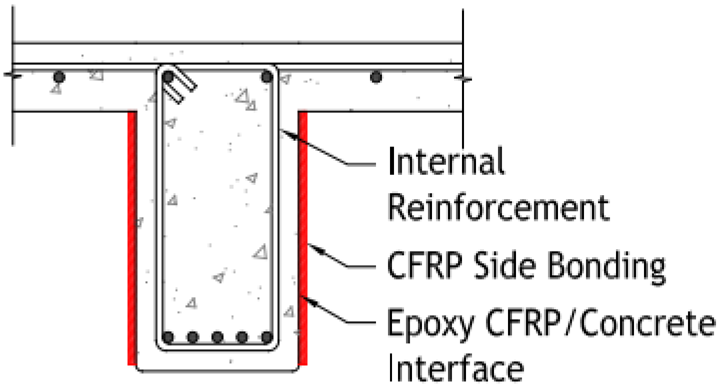


Figure 2.34: CFRP side bonding provides shear strength. (Kim Y.et al,2012)

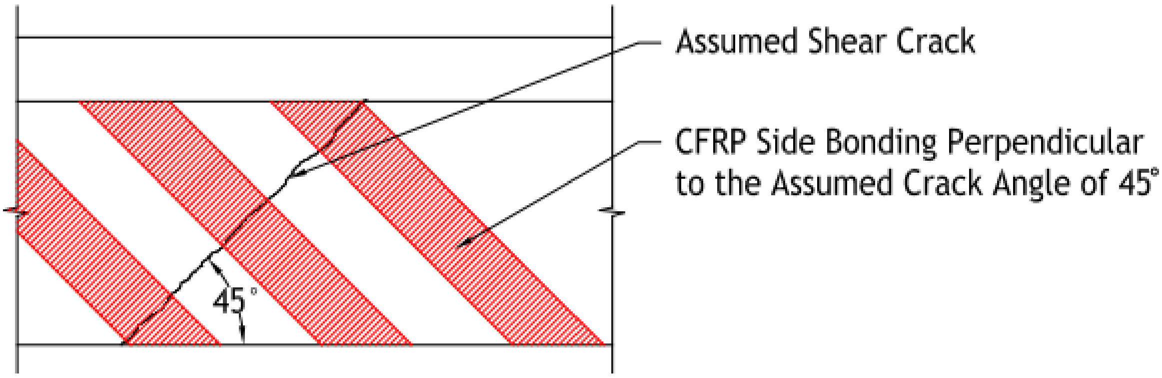


Figure 2.35: CFRP side bonding strips put perpendicular to a45' fracture angle (Kim Y. et al.,2012)

2.7. Future Research Needs

Future research needs to focus on several aspects. (a) design codes should be elaborated to structural designers so that dispose the calculation tool for future strengthening works. (b) assessments should be carried out to identify the durability of each system when they exposure to the different conditions. (c) application of prestressed tendons for strengthening is limited to the academia and it should be developed for practical purposes and suitable anchoring system.

Most of the researchers carried out their research for prestressed CFRP tendons. Very few studies have shown that prestressed sheets also potentially capable to play the role of tension reinforcement. Hence, methods should be developed to avoid the debonding at the corners.

CHAPTER 3: INTRODUCTION TO CASE STUDY

3.0. Background

Through the visual inspection, several corrosion related rust spots have been visualized at the bottom of the NERD center slab system. It can be seen that protective layers have been peeled off and initiation of long cracks at the lateral direction of the steel bars. Figure 3.10-3.12 shown cracks due to corrosion and expansion of steel bars. Some components of the concrete structures had been severely carbonized, and some concrete parts were also lost due to corrosion of steel bars, by affecting peoples' emotions. Hence, engineers should do relevant quality inspection to decide the retrofitting method before repairing the structures.

On-site examination and test results observed:

(1) The strength of prestressed beam and insitu concrete deck can fundamentally meet the requirements of design, and the strength of surface of concrete is smaller than that of the interior.

(2) The cracks of concrete along the area of retrofitting are longitudinally connected or not.

(3) The area which is affected for peeling off concrete layer.

(4) The amount of corrosion of steel bars, and the area of damaged at tension zone.

3.1. Structural Retrofitting Plan

Existing retrofitting and renovation technologies are divided into six categories. Such as (1) Method of enlarged the section; (2) method of enclose steel for retrofitting; (3) method of bonding of steel; (4) method of prestressed retrofitting; (5) the method of retrofitting by altering the way of structural strength; and (6) method of application of carbon fiber. Form the above mentioned renovation technologies, researchers have been proved that application of carbon fiber has several advantages such as high strength, high modulus, corrosion resistance, fatigue resistance, easy processing, light weight, and easy manual operation. Hence, due to its outstanding characteristics and high efficiency, CFRP can be used as the most capable method for retrofitting of the structures.

Corrosion of the steel bar causes the reduction of the area of cross-section, the strength of the steel bar, and the loss of the bond strength between the steel bar and the concrete interface. Further, it causes the strength to decrease. When strengthening, the amount of longitudinal carbon fiber fabric should be calculated according to the degree of structural corrosion to improve the bearing capacity. Hence, it is necessary to assess the ultimate strength of the pre-stressed beam with a 50 mm in-situ topping and the strength of the composite slab at failure so that proper adjustments can be made to prevent such a failure in the future.

Figure 3.10: Deterioration of NERD slab system

Figure 3.11: Deterioration of Pre-stressed NERD slab purlins in Centre for drug rehabilitation & human values development research studies, Pelmadulla, Sri Lanka, Constructed in 1993

Figure 3.12: Deterioration of Pre-stressed NERD slab purlins in house located in Ekala

CHAPTER 4: METHODOLOGY

4.1 Introduction to the proposed plan

The main objective of this research is to develop a suitable retrofitting method for corroded area of the NERD slab system which is cost effective and sustainable than the materials available in the market. FRP used as a retrofitted material and cracked area was cleaned and prepared for the application of FRP. This study assumes that corroded environments reduce the strength of the pre-stressed beams and regaining the reduced strength being the main target in this research. Hence, this research has been divided into two different stages which are described below.

Stage 1: Selected models were used as control specimens to check the load bearing capacity and the behavior when the load is applied. Crack patterns were observed for the supplication of FRP.

Stage 2: By observing the crack patterns, FRP application method were decided. Cracked samples were used as the retrofitted samples and other two samples from each model were used as strengthened samples.

4.2 Behavior of Composite Specimen- Stage 01

This section discusses the crack pattern of the pre-stressed composite specimens after four-point bending test. Three different models were simulated to predict the behavior when they were subjected to loading. Two samples were tested from each category and designations of each sample are given in the Table 4.1.

Table 4.1: Different models for testing (before FRP application)

Series	Designation	Type	Nos of specimens
		Control specimens	
Series 01	CLCP	Control panel with shear links	02
	CNCP	Control panel without shear links	02
	CB	Control beam	02

All the test specimens were 1800 mm * 600 mm

4.2.1. Load bearing capacity of the control specimen (Stage-I)

4.2.1.1 Material characteristics

4.2.1.1.1 Concrete

The average compressive strength of high strength concrete at 28 days is specified as 41.21 MPa for all beam specimens. Specific concrete with a compressive strength of 30.51 MPa was used for in situ toppings. In a nominal mix of 1:1:2 (Grade 40 concrete) concrete with early strength enhancement admixtures, the weight of ordinary Portland cement, locally available natural sand, and aggregate water-to-cement ratio remained constant at 0.5. Without admixtures, Grade 30 concrete was made with the same mix ratio. The topping was made from the same concrete batch as the beam, which was G40 and G30. After 28 days of curing, concrete cubes were tested for compressive strength in the lab. According to the ASTM C109 (ASTM C109/C109 M-02,2002) three cubes were cast from each batch and kept for curing for 28 days prior to testing. **Annex 04** contains the test results that were discovered.

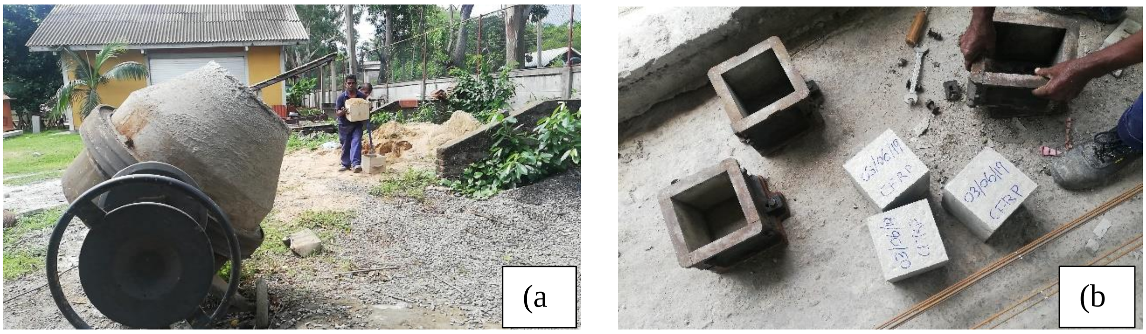


Figure 4.1: Materials (a) Cement & Aggregate; (b) Cubes for crushing

4.2.1.1.2 Steel

All beams were pre-stressed with high yield bars, 5mm diameter (T5). Characteristic breaking load is 32.7 kN & applied jacking force is 20 kN. It is around 60% of the expected strength of the tendon.

Table 4.2: Characteristic of the steel wires

Property	**Value**
Nominal diameter	5 mm
Nominal tensile strength	1670 N/mm^2

Characteristics breaking load	32.7 kN
Jacking force per 5mm wire	20 kN

4.2.2 Sample preparation & testing (Stage 1)

Before preparing composite panels, pre-stressed beams were cast according to the available specifications. Depth of the beam was decided as per the span of the slab. In this study beam was cast as 1800 mm (less than 3000 mm); hence the depth was taken as 100 mm. Beams were kept 600 mm intervals to make the composite behavior between pre-stressed beams and in-situ slab. 50 mm * 50 mm welded GI mess was used at the center of the 50 mm thick in-situ topping. Form work of the concrete slab was hung by using binding wires to the beam and poker vibrator was used for the better compaction of in-situ topping.

Figure 4.2: Sample preparation (a) pre-stressed beams with shear links (b) Hanging formwork and placing concrete (b) applying poker vibrator.

Prepared samples were tested using four-point bending test. The test arrangement was set to match the loads subjected to uniformly distributed loads over the central patch. Using this system, the inferior side becomes the tensioned one, and welded mess becomes the tensioned reinforcement. From the in-situ concrete slab, all the loads were transferred to the pre-stressed beams located at the edges of the experimental elements. A hydraulic jack with a force of 250 kN was utilized to induce the load. The I section was supported on two steel rollers that covered the entire width of the composite panel and transferred the loads in one single circle at a steady pace to the test beam through the hydraulic jack. Six elements were tested to the point of failure before being refitted with CFRP. Figure 4.3 illustrates a general overview (test setup and instrumentation) of the entire test setup. Two no of Dial gauges were used to measure the deflections up to 150 mm and the least count of 0.01 mm placed at the middle of the pre-stressed beams. The dial gauges were placed in the center of the composite panel such that when the load was transmitted, the deflection was critical in the middle of the span. The propagation of cracks was noted, as well as the mode of failure.

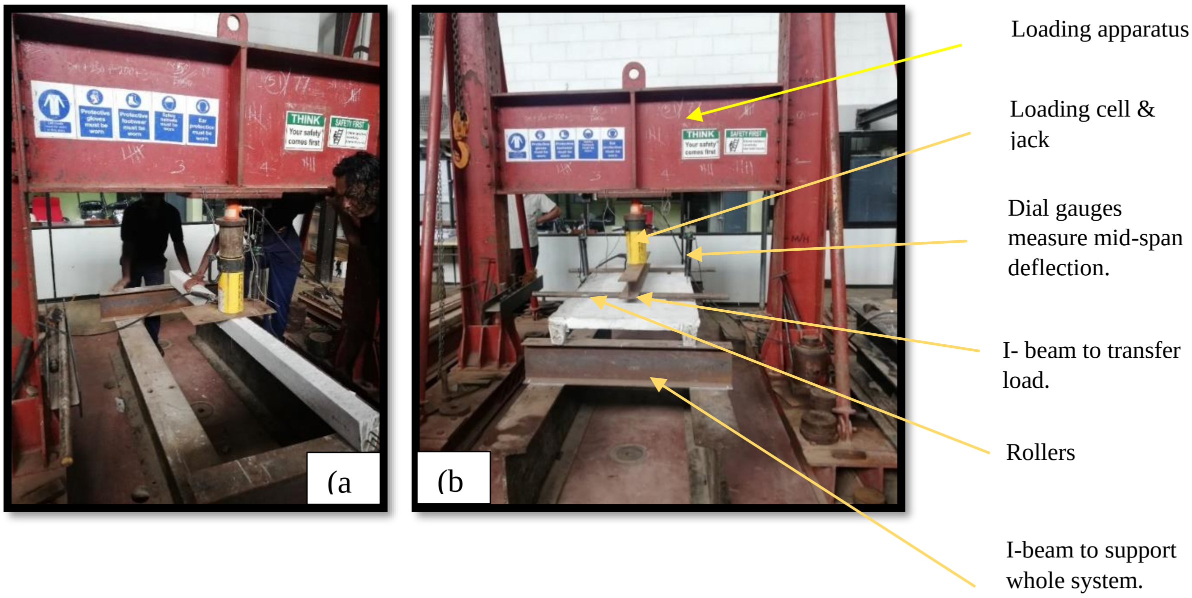


Figure 4.3: Test setup and instrumentation (a) loading setup of pre-stressed beam (b) loading setup of composite slab.

4.3. Behaviour of the Retrofitted and Strengthened Specimen- Stage II

4.3.1 Overview

Based on the cracks propagation after reaching its ultimate loading capacity as explained in the Chapter 4, CFRP was selected as an insulated material for concrete composites. Here it was assumed that the strength of concrete is lost substantially by corrosion; hence FRP was used as the retrofitting and strengthen material of concrete. As the next step, CFRP was applied as a retrofitting material of pre-cracked composite slabs and pre-stressed beams to regain the ultimate strength of the specimen. Another model was developed to study the behavior and the performance of the composite slabs. Conclusions were made with the percentage of increment of the strength of each specimen after loading.

This section mainly focuses on the materials prepared in stage 2. Crack pattern, behavior of loading, percentage of strengthening using CFRP are discussed. Further, retrofitting method before using CFRP in composite slabs are also explained. Table 5.1 shows the models prepared for the testing below.

Table 4.3: Different models for testing (after FRP application)

Series	Designation	Type	Nos of specimens
		Retrofitted specimens	
Series 02	RLCP	Retrofitted panel with shear links	02
	RNCP	Retrofitted panel without shear links	02
	RB	Retrofitted beam	02
		Strengthen specimens	
Series 02	SLCP	Strengthened panel with shear links	02
	SNCP	Strengthened panel without shear links	02
	SB	Strengthened beam	02

All the test specimens were 1800 mm * 600 mm.

4.3.2. Load bearing capacity of the control specimen (Stage-II)

4.3.2.1 Material characteristics

In situ concrete with 28-day compressive strength of Grade 30 and Grade 40 concrete was used to prepare the specimens with the dimensions of 1800 mm * 600 mm. 0.116 mm thick CFRP sheets (Technical data sheet, CFRP) were used to retrofit and strengthen the composite slabs and pre-stressed beams using two-part epoxy adhesive (Technical data sheet, adhesive) after preparing the surface of each specimen. Cracks repaired using crack repairing mortar available in the market. Through the removing of weakened concrete layer, strength can be restored using proper insulation and the characteristics of the materials used in each stage are listed below. All the properties of the materials were obtained from the technical data sheets which are attached in the Annexes.

4.3.2.1.1 FRP & Epoxy Adhesive

Table 4.4 and Table 4.5 presented the properties of uni directional CFRP (refer to Figure 4.4) and adhesive (refer to Figure 4.5).

Table 4.4: Properties of Carbon Fiber Fabrics

Product Name	**High strength carbon fiber fabric for structural strengthening**
Product code	X-Wrap C300
Sheet weight	300g/m2
Carbon content	95%
Net effective thickness	0.166mm
Tensile strength	4000 MPa
Elongation at break	2%
Modulus of elasticity	240 GPa

Table 4.5: Properties of Epoxy Adhesive

Product Name	**Lamination resin for X-wrap fabrics**	
Property	standard	Tropical
Viscosity 20’C	5000 cps	7000 cps
Pot life	40 mins at 25 C	40 mins at 35 C
Tensile strength 7d/20’C		45 MPa
Flexural strength 7d/20’C		60 MPa
Compressive modulus 7d/20’C		1.67 GPa
Glass transition temperature		65C

Figure 4.4: CFRP fabric

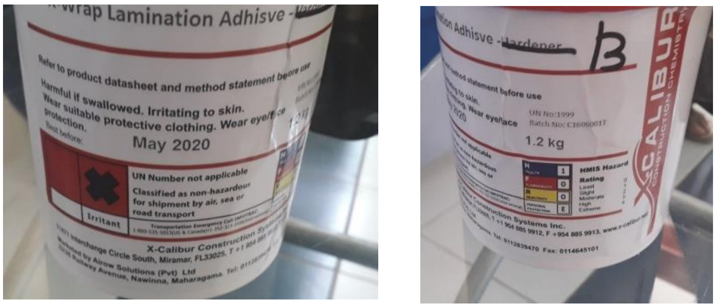


Figure 4.5: Epoxy Adhesive

4.3.3 Crack Repairing for Specimen Preparation and Material Properties

4.3.3.1 Overview

All specimens were prepared according to the details available in Chapter 2, literature review. In situ topping was placed on the pre-stressed beams and both shear & without shear links methods were used for preparation of specimens as mentioned earlier. Heavily damaged specimens were prepared after loading of composite slabs. Depth of the pre-stressed beams were based on the spans given in Table 2.1. Load increments after the application of CFRP for retrofitting and strengthening the specimen were compared with the control specimens which were measured using universal testing machine as shown in Figure 4.3. Two I sections were used to make simply supported at the end and a transient load was applied through the I section until the failure occurred. During the loading the propagation of cracks were inspected, and the flexural and shear cracks occurred were identified on pre-stressed beams.

It is observed that slab beams connections were heavily damaged, and crack sealing method should be used in this study according to the ACI guidelines. Therefore, the cracked area was completely removed from the beams and damaged area was repaired using an in-situ concrete repair grout. All the cracks were sealed using adhesive. Then the specimen was kept for fourteen days of curing under ambient conditions. To strengthen the specimens using CFRP was laid after following wet lay-up method, at the end of the exposure period, as describe below. As well as four non cracked specimens (used as control specimens) were also strengthened using CFRP with the same method. Detailed procedure including the crack repairing and the application of CFRP are given in the next subsection.

4.3.3.2 Crack Repairing and Material Specification

The concrete layer in the bonding area becomes weak due to the application of loads and the occurrence of some flexural and shear cracks. As per the guidelines of ACI Committee 440, 2017, weak part of the bonding area should be ground off using an angle grinder and a proper crack repairing method should be used. The repairing method and the material specifications are given below.

Step 01: Repair the spalling parts of PSC beams.

Using high-pressure water blasting, all traces of loose concrete or mortar, dust, grease, oil, and other contaminants were thoroughly eliminated. To achieve a keyed aggregate exposed surface, damaged or contaminated concrete was removed. The mortar was directly placed on the damp substrate and left to dry for 24 hours.

Table 4.6: Properties of the material (MasterEmaco S 5400, n.d.)

Product name	**Master-Emaco S 5400**
Appearance	Grey color
Layer thickness	Min. 5 mm Max. 50 mm
Density	Approx. 2.2 g/cm3
Compressive strength - After one day - After seven days - After twenty-eight days	 ≥16 MPa ≥45 MPa ≥70 MPa
Adhesion (28 days)	≥2 MPa
Application temperature	Between +5 and +35'C
Working time	45 – 60 min

Figure 4.6: Repairing of cracks using structural repair motor

Step 02: Seal the crack, place the injector bases at necessary places.

Starting at the bottom of the hole, the hall was bored and carefully cleaned using compressed air. The drill hole is cleaned with a steel brush before injecting pure epoxy resin into the hole. While carefully drawing back the static mixer, the adhesive was injected (Figure 4.6 (b)) into the hole, starting from the bottom. Anchors were inserted slowly with a rotational motion and left undisturbed for 24 hours to solidify.

Table 4.7: Product data (FastFix it, n.d.)

Product Name	Injection Cartridge (pure epoxy resin)
Product code	FX-E400 WE400
Product description	Two-part high-performance anchoring adhesive based on epoxy resin and devoid of solvents.
Product data	Colors: Part A: White Part B: Black Mixed: Gray Mix ratio: 3:1
Density	Part A: 1.70 kg/l Part B: 1.30 kg/l 1.68 kg/l (part A + B mixed) (+/_- 0.5kg)

Table 4.8: Get and loading times.

Application temperature (‘C)	Gel time (min)	Loading time (hr)
40	4	3
30	7	5
20	15	7
10	60	12

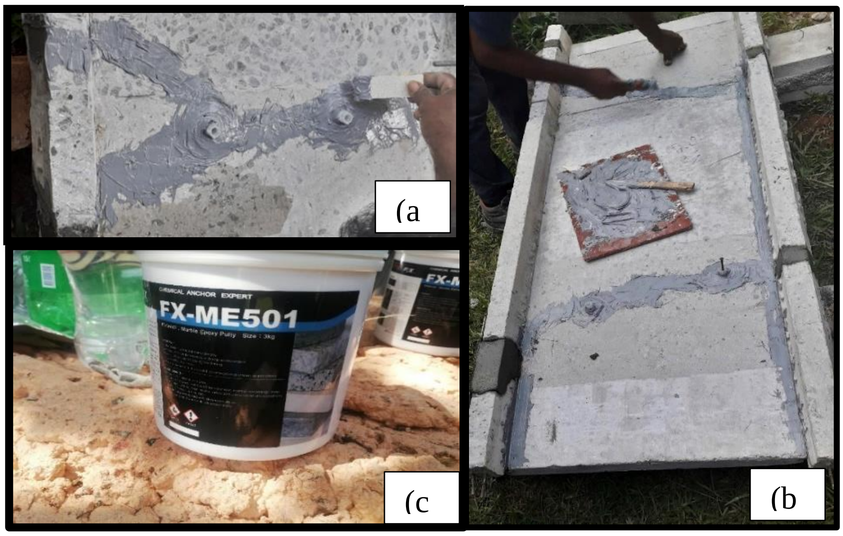


Figure 4.6: Strengthening procedure (a) Placing of injector bases (b) Sealing the cracks (c) Pure epoxy resin.

Step 03: Inject crack sealing adhesive using mighty injectors.

TamRex 220 was Injected using mighty injectors and allowed to cure the tested PSC beams and the composite slab panels for seven days.

<u>TamRex 220</u>

It's an epoxy resin with a high modulus and low viscosity that may be injected into structural cracks in concrete and set in a wet or damp environment. Some of full specification, including the performance data, are given in Table 4.9 below. The application of epoxy resin (Figure 4.7 (a)) for both composite slabs and pre-stressed beams are shown in Figure 4.7(b).

Table 4.9: Technical data for TamRex 220 (TamRex 220, n.d,)

All at 25'C	**Part (A)**	**Part (B)**	**Mixture**
Color	Clear	amber	Amber
Density	1.09 kg/l	0.98 kg/l	1.05 kg/l
Mix ratio			2:1 by volume
Minimum curing temp.			15'C
Compressive strength, 7 days at yield			80 N/mm2
Tensile strength, 7 days			25 N/mm2
Final cure	7-14 days	7 days	7 days

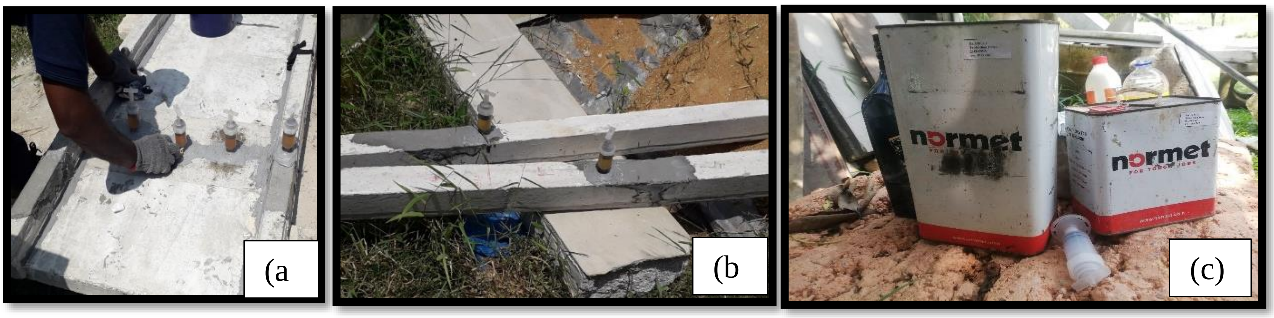


Figure 4.7: Inject epoxy resin (a) for composite panels (b) for pre-stressed beams (c) low viscosity long pot life epoxy injection resin, TamRez 220 TG

<u>Mighty injectors</u>

The powerful nozzle consists of a small storage container with a spring-loaded piston filled with an oiler. The information of the mighty injectors is given in Table 4.10. The high-performance nozzles (refer to Figure 4.8) are placed directly above the cracks and fixed with TamRez 310, which is also used to seal the surface between the nozzles. The advantage of this system is that the operator only needs to fill in the injector's form reservoir, which then applies a constant pressure from the internal spring mechanism.

Table 4.10: Product Specification (Normet Mighty Injector, n.d.)

Maximum length	110mm
Minimum length	95mm
Capsule inner diameter	27mm
Weight	42 gm
Maximum pressure	4.0 bar
Minimum pressure	2.2 bar
Maximum capacity	10 cc

Figure 4.8: Mighty injectors

Step 04: Make the surface is rough before applying CFRP.

<u>Surface preparation</u>

For the surface preparation, the substrate to be treated was cleaned by grinding to remove all contamination such as dust, oil, grease, surface coatings, etc. Sharp edges were smoothed up to give a radius of at least 20mm.

Figure 4.9: Surface preparation (a) Grinding off weak layer of concrete (b) Appearance of ground concrete surface.

Step 05: Apply CFRP for slab panels and allow it to cure.

After observing the crack pattern, the decision was taken to apply CFRP.

4.3.4 Design of the strengthening system

After observing the crack pattern, a simplified analytical approach was applied to determine the total amount of carbon fiber reinforced polymer (CFRP) that is to be placed on each element. For all the elements subjected to the flexural failure, the strengthening procedure consists of applying CFRP components on the tensioned side in the required direction and the quantity. CFRP was installed on the inferior side of the slab (fiber direction to longitudinal, bending direction) and along the soffit of the pre-stressed beam with edges in this project (fiber direction perpendicular to the crack). The properties of the carbon fiber fabric and adhesive are given in Table 4.11 and Table 4.12. The observed crack pattern on both the composite specimen and the pre-stressed beam and the way the FRP application was done are shown in Figure 4.10 & 4.11.

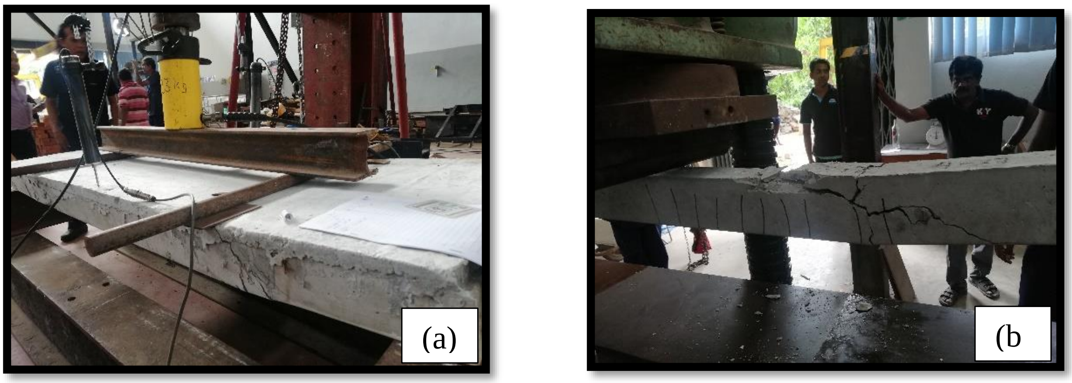


Figure 4.10: Crack pattern (a) composite slab (b) pre-stressed beams

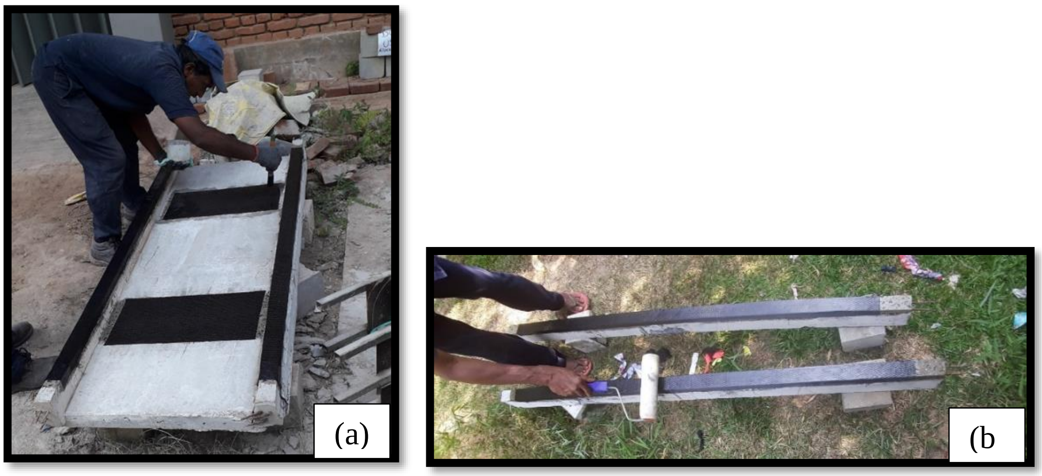


Figure 4.11: Strengthening procedure (a) FRP application for composite slab (b) FRP application for pre-stressed beam.

Table 4.11: Properties of Carbon Fiber Fabrics (X-Wrap C300, n.d.)

Product Name	**High strength carbon fiber fabric for structural strengthening**
Product code	X-Wrap C300
Sheet weight	300g/m2
Carbon content	95%
Net effective thickness	0.166mm
Tensile strength	4000 MPa
Elongation at break	2%
Modulus of elasticity	240 GPa

Table 4.12: Properties of Epoxy Adhesive (X-Wrap Lamination Adhesive, n.d.)

Product Name	**Lamination resin for X-wrap fabrics**	
Property	standard	Tropical
Viscosity 20'C	5000 cps	7000 cps
Pot life	40 mins at 25 C	40 mins at 35 C
Tensile strength 7d/20'C		45 MPa
Flexural strength 7d/20'C		60 MPa
Compressive modulus 7d/20'C		1.67 GPa
Glass transition temperature		65C

CHAPTER 5: RESULTS & DISCUSSION

5.1. Stage 1- Test Results

Deflections at the mid span of the slab, and deflections at the mid-point of the pre-stressed specimens, the crack width, and the load level were also measured. Average deflection value was obtained by two dial gauges to obtain precise results. Sets of test results for control specimen with or without shear links and pre-stressed beams are attached in **Annex 10**.

5.1.1 Control slabs with or without shear links

As expected, flexural cracks were appeared at the two supports and small cracks appeared at the ends with a further widening of flexural cracks (Figure 5.1.). When the applied load is more than 50 kN, the beam failed in the flexural mode at two supports as shown in Figure 5.2.

The major cracks observed were the flexural cracks and shear cracks were also observed with increased load. The load at the failure of slabs without shear links were less than the load at the failure of slabs with shear links due to its horizontal shear failure between pre-stressed beam and in situ slab.

Figure 5.1: Crack propagation on composite slabs

Figure 5.2: Slab failure at two supports

In addition, at the top of the mid span of the beam, some crushing may happen where the load was applied as in the Figure 5.3. Failure of pre-stressed beam was due to the flexural and the shear cracking as well as the crushing on the pre-stressed beams.

Figure 5.3: Cracking on pre-stressed beams

5.1.2. Comparison of load vs deflections of specimens

Figures 5.4 & 5.5 illustrate the variation of the loads against the deflection of each specimen. Figure 5.4 displays the variation of load vs deflection of composite slabs and Figure 5.5 displays the variation of load vs deflection of pre-stressed beams.

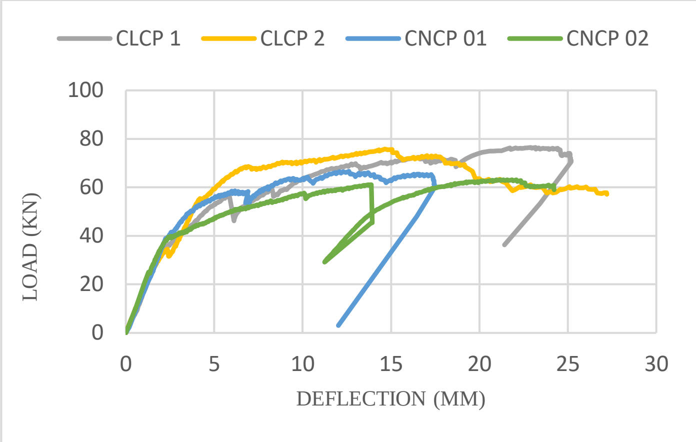


Figure 5.4: Variation of load vs deflection of composite slabs

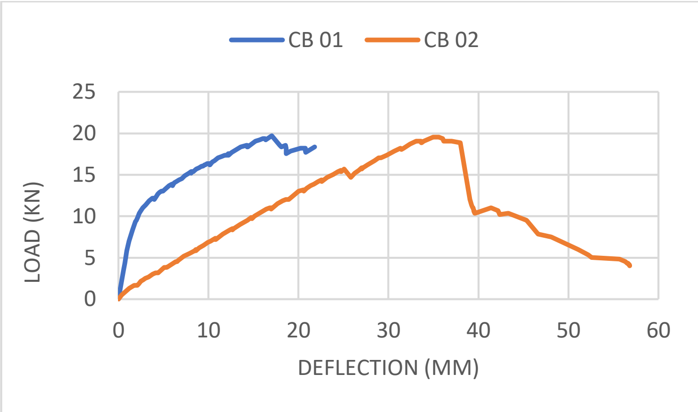


Figure 5.5: Variation of load vs deflection of pre-stressed beams

Composite slabs with shear links gave a maximum load with compared to the slabs without shear links at the failure. Pre-stressed beams without composite action gave lesser value compared to the composite slabs. Hence, it can be concluded that, composite behavior gave higher load bearing capacity. Further, it was noticed that the load bearing capacity was reduced after the composite slabs reached their maximum loading capacity.

5.1.3. Discussion

This chapter describe the series of test programs which were carried out to find the load bearing capacity of composite slabs and pre-stressed beams. Grade 30 and Grade 40 concrete were used for casting in situ topping and pre-stressed beams. To maintain the composite behavior, the in-situ toppings were laid on pre-stressed beams. Depth of the beams were chosen depending on the span and shear links were applied to reduce the horizontal shear failure of the composite slab.

Load bearing capacity of composite slabs was checked using four-point bending test and the load bearing of pre-stressed beams was checked by applying loads at the center. When the applied load was increased, flexural cracks initiated at the supports of the pre-stressed beams and the beams failed due to flexural and shear cracks. Load bearing capacity of composite slabs were much higher compared to the pre-stressed beams due to its composite action. In addition to that, load at failure of composite slabs with shear links also showed some additional value than that of without using shear links. The allowable load and load at failure are given as a summary in Table 5.1.

Table 5.1: Load variation of allowable load and load at failure

Model ID	**Description**	**Allowable load (kN)**	**Load at the failure (kN)**	
			Sample 01	Sample 02
CLCP	Deflection	9.72	54.4	67.5
	0.3 mm crack	14.23	46	55
CNCP	Deflection	9.72	52.3	56.4
	0.3 mm crack	14.23	32	46
CB	Deflection	2.42	14.5	5.1
	0.3 mm crack	3.56	18	14.3

5.2 Test program – Stage 2

All the retrofitted and strengthened specimens developed were tested at Structural Testing Laboratory, University of Moratuwa. Test arrangement is explained under the subsection 4.2.3. All prepared models were tested for four-point bending test.

5.2.1. Retrofitted slabs with or without shear links.

The obtained load and deflections values for retrofitted models from the four-point bending test are mentioned in Table 5.10. Deflections at the mid span of the slab, and mid span of the pre-stressed specimens, crack width, and load level were also measured. Average deflection value was obtained by two dial gauges to obtain the precise results. Sets of test results for retrofitted models with or without shear links and retrofitted pre-stressed beams are attached in **Annex 10**

5.2.1.1 Crack propagation

When the applied load increased, flexural cracks at the two supports of the composite slab appeared around 10 kN, as illustrated in Figure 5.6. Around 15 kN load, shear cracks appeared at the edges of the CFRP sheets and spalling of FRP started at two edges of the pre-stressed beams as illustrated in Figure 5.7. Retrofitted models ultimately failed in shear due to de-bonding of CFRP fabrics at 46 kN load as shown in Figure 5.8. In addition to that, models without shear links were failed due to loosen of composite behavior between pre-stressed beams and in situ concrete due to its horizontal shear failure as shown in Figure 5.9.

Figure 5.6: Initial flexural cracks

Figure 5.7: Shear cracks near CFRP wrappings

Figure 5.8: Models failed by de-bonding of CFRP

Figure 5.9: Loosening of strength between in-situ concrete and pre-stressed beam.

In addition to that, retrofitted pre-stressed beams failed due to the propagation of flexural cracks at the middle of the beams. The beam was crushed at the center as usual where the load was applied. All the pre-stressed beams failed by their ultimate load at 7 kN as displayed in Figure 5.10.

Figure 5.10: Pre-stressed beams failed due to crushing and propagation of flexural cracks.

5.2.1.2 Comparison of load vs deflections of specimens

The variation of the loads against the deflection of each specimen are illustrated in Figure 5.11 & 5.12 below.

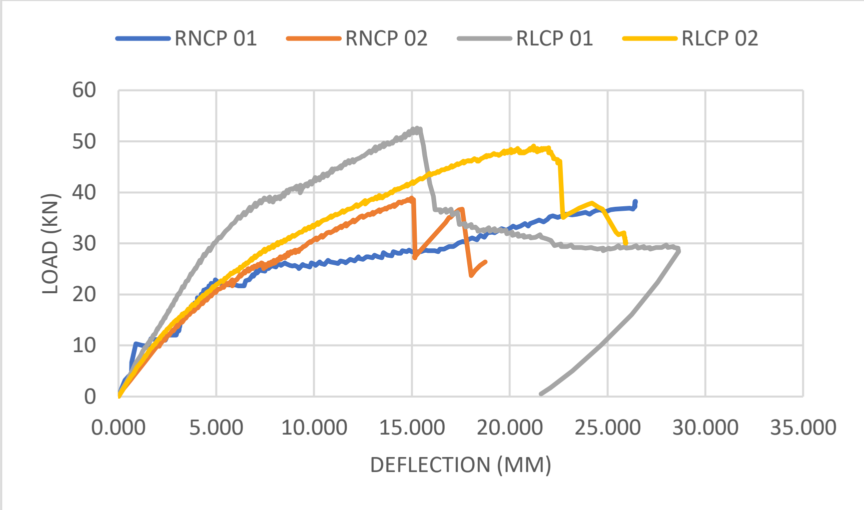


Figure 5.11: Variation of load vs deflection of composite slabs

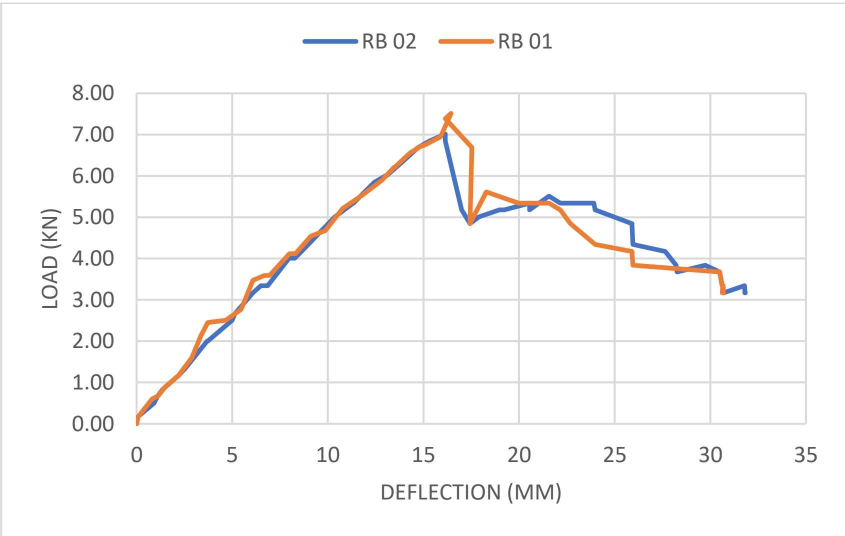


Figure 5.12: Variation of load vs deflection of pre-stressed beams

Figure 5.11 show the variation of load vs deflection of composite slabs and Figure 5.12 shown the variation of load vs deflection of pre-stressed beams. Composite slabs with shear links gave the maximum load compared to slabs without shear links at the failure as usual. Pre-stressed beams without composite action gave a lesser value compared to the composite slabs. Hence, researcher given concluding remarks that the composite behavior provides the higher load bearing capacity. Further, it was noticed that the load bearing capacity was reduced after composite slabs reached their maximum loading capacity.

5.2.2. Strengthened slabs with or without shear links.

The obtained load and deflections values for strengthened models from the four-point bending test are included in Table 5.11. Sets of test results for strengthened models with or without shear links and strengthened pre-stressed beams are attached in **Annex 11.**

5.2.2.1 Crack propagation

When the applied load was increased, flexural cracks at the two supports of the composite slabs appeared around 12 kN. Around 18 kN load, the shear cracks appeared at the edges of the CFRP sheets and spalling of FRP started at the two edges of the pre-stressed beams. Strengthen models ultimately failed in shear due to the de-bonding of CFRP fabrics at 50 kN load as shown in Figure 5.13. In addition to those models without shear links failed due to the loosening of the composite behavior between the pre-stressed beams and the in-situ concrete due to its horizontal shear failure.

Figure 5.13: Shear and flexural cracking on strengthened models with FRP.

Furthermore, as per the retrofitted pre-stressed beams, strengthened beam with FRP also failed because of the propagation of flexural cracks at the middle of the beam. When the load was applied, all the beams were crushed at the middle as usual. All the pre-stressed beams were failed by their ultimate load at 13 kN as illustrated in Figure 5.14.

Figure 5.14: Strengthened Pre-stressed beams failed due to crushing & propagation of flexural cracks.

5.2.2.2. Comparison of load vs deflections of specimens

The variation of the loads against the deflection of each specimen are illustrated in Figure 5.15 & 5.16 below.

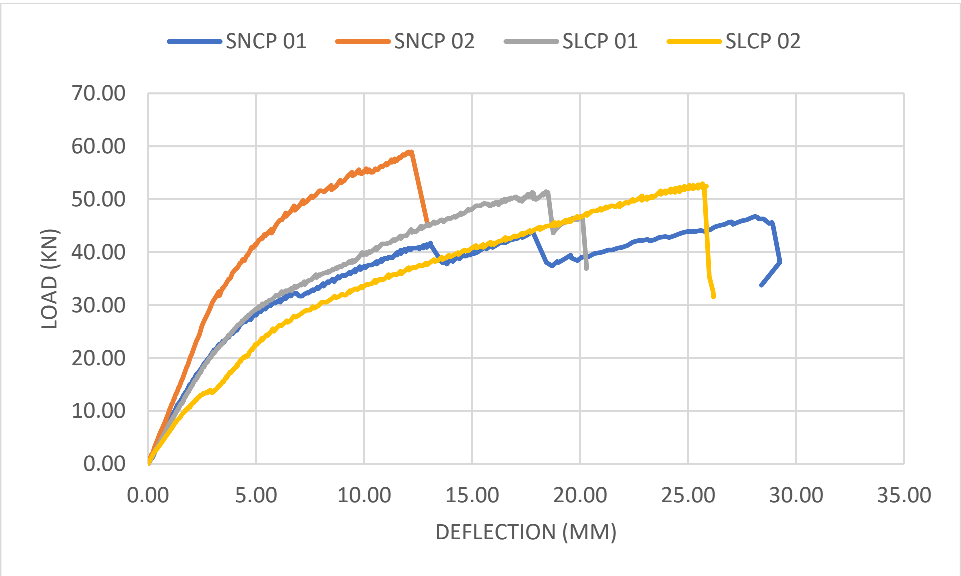


Figure 5.15: Variation of load vs deflection of composite slabs

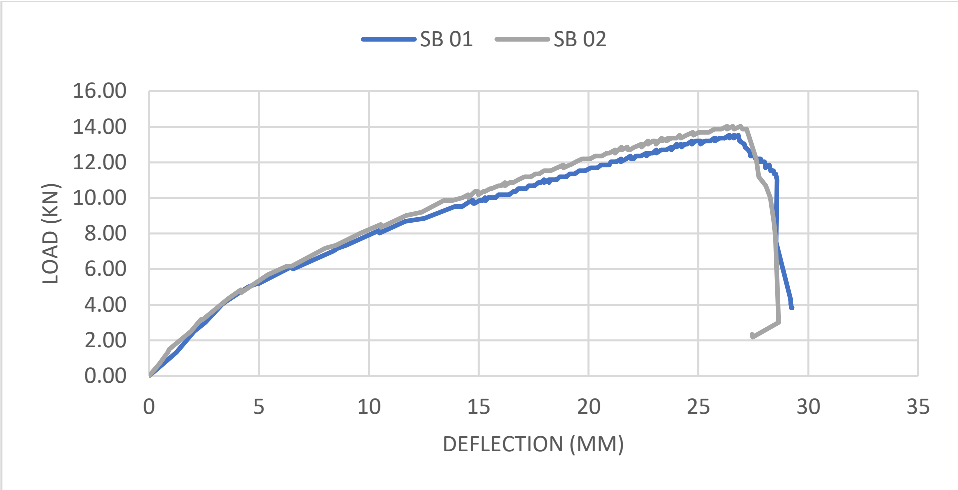


Figure 5.16: Variation of load vs deflection of pre-stressed beams

Load vs deflection variation of composite slabs and prestressed beams were shown in Figure 5.18 and Figure 5.19 respectively. Composite slabs with shear links gave the maximum load compared to slabs without shear links at the failure as usual. Pre-stressed beams without composite action gave lesser value compared to the composite slabs. Hence, it can be concluded that, the composite behavior provides higher load bearing capacity. Further, it was noticed that the load bearing capacity reduced after composite slabs reached their maximum loading capacity.

5.2.3. Discussion

In this chapter, a series of test programs were carried out to find a method to increase the load bearing capacity of composite slabs and pre-stressed beams which were exposed to adverse environment. Due to low nominal cover and/or problems encountered during the manufacturing process, they can get exposed to the bad weather. Hence, cracks can propagate which reduce the strength between pre-stressed tendons and concrete layer. In this study assumption was based on the fact that the insulation material can increase the strength of concrete. CFRP is an ideal material among other insulation materials and hence this study focused on the models retrofitted and strengthened using CFRP.

The main purpose of these two setups to make better comparison of shear and flexural load increment after the CFRP wraps on cracks that develop along the transfer length of prestressing beams and insitu topping. Load bearing capacity of strengthened/retrofitted composite slabs was checked using four-point bending test, and load bearing capacity of pre-stressed beams was checked by applying loads at the center. When the applied load increased, flexural cracks initiated at the supports of the pre-stressed beams and the beams failed due to flexural and shear cracks. In addition, some de-bonding failure also happened when the models reached their ultimate loading capacity. Application of FRP changed the loading pattern and some cracks propagated along the middle line of the in-situ slabs for both strengthened and retrofitted specimens. Load bearing capacity of strengthened/retrofitted composite slabs were lower compared to the control specimen due to the change in load distribution pattern. Allowable load and load at failure are given as a summary in Table 5.2.

Table 5.2: Load variation of allowable load and load at failure

Model ID	**Description**	**Allowable load (kN)**	**Load at the failure (kN)**	
			Sample 01	Sample 02
RLCP	Deflection	9.72	38.4	28.2
	0.3 mm crack	14.23	45	40
RNCP	Deflection	9.72	17.1	25.8
	0.3 mm crack	14.23	25	37
RB	Deflection	2.42	3.4	3.9
	0.3 mm crack	3.56	5	5.1
SLCP	Deflection	9.72	34.4	28.56
	0.3 mm crack	14.23	40	46
SNCP	Deflection	9.72	32.06	49.1
	0.3 mm crack	14.23	38.4	51
SB	Deflection	2.42	6.5	6.6
	0.3 mm crack	3.56	12	13.3

It can be observed that the average failure load of the strengthened specimens is 19.5% less than that of the control specimen (Failure pattern changed). In addition, the average failure load of retrofitted models is 38.84% less than the control specimen, 24.04% less than the strengthened specimen and 48% greater than the required load-carrying capacity.

CHAPTER 6: CONCLUSIONS

Based on the real field investigations, in this research, the effects of damage due to corrosion in composite slab system were studied. Compared to the reinforcing bars in concrete, pre-stressed strands are more vulnerable to corrosion and hence, it is easy for the corrosion agents to corrode pre-stressing strands. Aggressive environmental factors, loading conditions and low nominal cover become critical factors for initiation of corrosion and propagation of cracks.

Concrete are weak in tension and steel governs the tensile strength of concrete which maximizes the ultimate flexural capacity of the concrete. It is observed that poor workmanship, and lack of knowledge of the deterioration mechanisms results in insufficient planning and wrong estimation of environmental effects. Propagation of cracks may lead to corrosion and corrosion in one strand may eventually affect the other strands. Once at least two strands are break, the pre-stressed beams are completely lost its ductility and it may suffer for brittle failure. Hence, propagation of cracks can diminish the strength and make both the occupants and the structure at high risk.

This research focused on maximizing the structural capacity if it gets reduced due to the loss of strength in the pre-stressed beam. Under two stages, the testing was conducted for the initial analysis. In stage 01, six specimens were cracked by applying loading up to their failure points. Specimens with shear links were given high loading capacity at the failure, compared to the shear links without failure. Shear screws at the steel concrete interface enhance the load increment by 43.75% at failure since making the failure mode from brittle to ductile and minimizing the horizontal shear strength. Pre-stressed beams also gave higher value at failure compared to the allowable load.

In stage 02, all the pre-stressed beams were retrofitted using CFRP and another six un-cracked specimens were also strengthened using CFRP. CFRP was used according to the past literature and in a more ideal way with concrete structure having low nominal cover. The load demand at the failure was significantly lower compared to the expected capacity of the specimens. Change of loading pattern due to the application of CFRP resulted in the discrepancy in loading. Flexural cracks propagated at the middle and shear cracks propagated near the supports. All retrofitted and strengthened specimens failed due to de-bonding of CFRP near the supports. However, it was observed that the specimens without shear links were failed by the loosened composite behavior due to its horizontal shear failure. But in the case of pre-stressed beams, both retrofitted and strengthened specimens showed low loading capacity compared to the control specimens. It was observed that, when loading was applied, the behavior of the cracking changed. Hence, it was revealed in this study that this technique is more efficient for composite slabs than pre-stressed beams since this technique changes the loading pattern and enhances the loading capacity of the composite specimens.

6.1 Future Studies

Methodology of corrosion detection is still in its early stage and has a long way to go. According to the past literature, there are several techniques identified and, in this research, there was not any inspection equipment used except the visual inspection. It is not the best way to identify the stage of corrosion and the loss of pre-stressed force. If any other method is developed to identify the level of corrosion, then the inspection will not be tiresome and time consuming.

Discrepancy in the load demand and expected capacity of both strengthened and retrofitted specimens remained a mystery throughout this research. Some assumptions were required for the reasons behind these significant variations to conclude. The author would like to suggest to further investigate for the possible faults in the specimens and to use real corroded scenarios for the testing.

As per the literature few studies carried out to demonstrate the application of prestressed FRP sheets as the tension reinforcement. The authors believe that it provides additional strength along the length of the members and over the support. The future studies need to be evaluated long term behavior of the system, introducing suitable anchorage system to increase the initial prestressing levels, analysis the behavior when expose to the fire, how system affect to the creep properties.

CHAPTER 7: RECOMMENDATIONS

Based on the test results received from the experiment, the following recommendations can be made.

- It is important to obtain detailed visual inspection about the pre-stressed beams or in-situ concrete slab to be repaired before doing any strengthening or repairing. This inspection includes areas of corrosion, nature of corrosion, nature of crack propagation and depth of cracks. Strengthening should be done after inspection to avoiding the structure may perform in an undesirable fashion.
- Application of FRP along the bottom side of the beam is applicable for the flexural strengthening of the prestressed beam. FRP U stirrups are recommended when shear strengthening is accomplished at one third of length of both side of the beam. U stirrups provide additional support to the shear reinforcement to the beam, and also support for additional anchorage to the flexural reinforcement without making disturbances to the aesthetic of concrete surface.
- The methods which were used to prepare the surfaces of these specimens appeared to be adequate. Debonding were not observed in major areas during the tests. Hence, writer recommended to use these strengthening and retrofitting methods for future strengthening projects.

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Appendix 01

Detailed Design of the Composite Slab System

1.0 Design of elements

1.1 Design data

Imposed load (kN/m^2)	=	1.5
Beam depth (mm)	=	175
Jacking force per wire (kN)	=	20
Wire diameter (mm)	=	5.0
No of wires used	=	3
Beam spacing (mm)	=	600
Thickness of the in-situ topping	=	50
Span of the beam in feet	=	16
Cover to bottom steel (mm)	=	34
Strength at transfer; f_{ci} (N/mm^2)	=	30
Concrete grade, f_{cu} (N/mm^2)	=	40

1.2 Calculation of Longitudinal Stresses

Calculation of Sectional Properties

The following beam section was selected for the analysis.

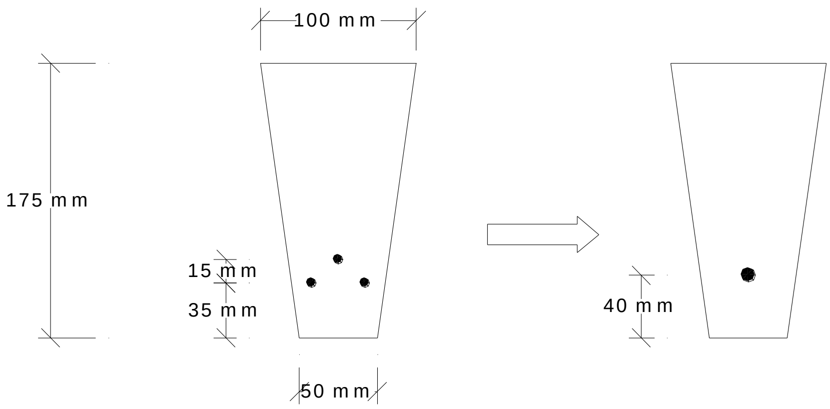

Figure 01: Section of PSC slab purlin

Cross sectional area of the pre- stressed beam A_c is given by.

$$A_c = \frac{(100+50)\times 175}{2}$$

$= 1.3125 \times 10^4\ mm^2$

Calculation of I value for pre-stressed beam; $I_{beam,na}$

The distance to centroid from bottom; ā is given by.

$$\left[100\times 175\times \frac{175}{2}\right] = \left[\frac{(100+50)\times 175}{2}\times \bar{a}\right] + \left[2\times \frac{(25\times 175)}{2}\times \frac{175}{3}\right]$$

$$1.531\times 10^6 = (1.3125\times 10^4\times \bar{a}) + 0.225\times 10^6$$

$$\bar{a} = 97.2\ mm$$

Therefore, second moment of area of the beam section about neutral axis is given by.

$$I_{beam,na} = \frac{175^3\times (50^2 + 4\times 50\times 100 + 100^2)}{36\times (50+100)}$$

$$= 3.226\times 10^7 mm^4$$

Assumptions:

1. 30 % of the jacking force is considered as pre-stress losses.

2. Based on the present practice, the maximum span that 175mm deep beam can support is 4.88 m (16 ft.)

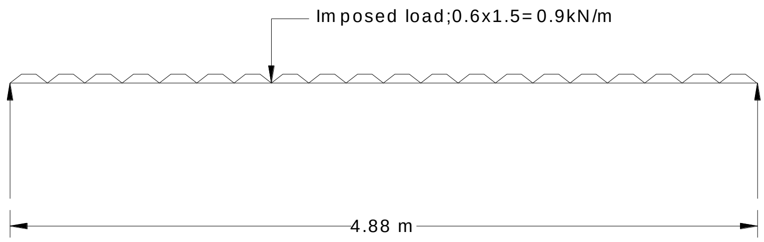

Figure 02: Arrangement of loading

Applied pre-stressing force;P_j =20kNx3=60 kN (i.e.Jacking force per wire)x(no. of wires)

Effective pre-stressing force;P_e = 60x0.7 = 42 kN (assuming 30% losses)

Stresses due to effective pre-stressed force (assuming compressive stresses are positive)

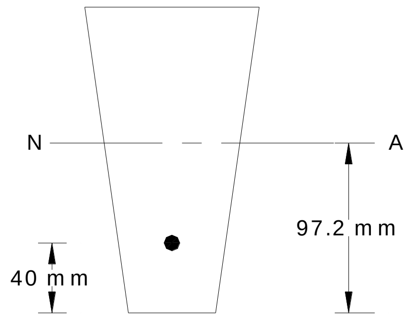

Figure 03: Eccentricity of PSC purlin

Eccentricity; e=97.2-40 =57.2 mm

Uniform stresses due to pre-stress = P_e/A_c

$$= \frac{42 \times 10^3}{(100 + 50) \times \frac{175}{2}}$$

$= 3.2 \text{ N/mm}^2$

Stress due to eccentricity at bottom fibers $= \dfrac{P_e \cdot e}{Z_b}$

$$= \frac{42 \times 10^3 \times 57.2}{(3.226 \times 10^7)/97.2}$$

$= 7.24 \text{ N/mm}^2$

Stress due to eccentricity at top fibers $= \dfrac{P_e . e}{Z_b}$

$$= \frac{42 \times 10^3 \times 57.2}{(3.226 \times 10^7)/77.8}$$

$= 5.79 \text{ N/mm}^2$

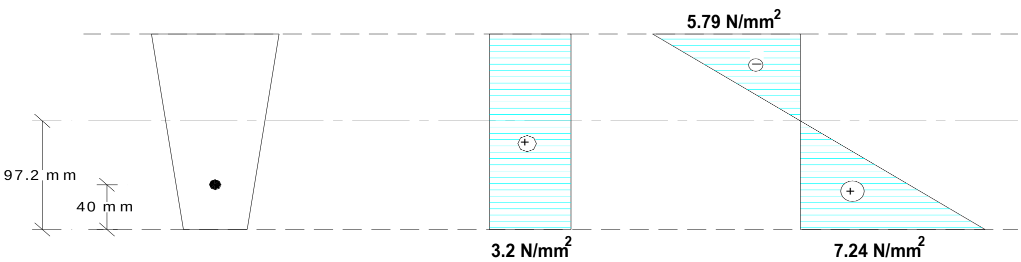

Figure 04: Stress due to effective PSC force

Stresses due to self-weight.

Dead weight; $\text{wd} = \dfrac{(100 + 50)}{2} \times 175 \times 10^{-6} \times 1 \times 24$

$= 0.315 \text{ kN/m}$

Stress due to self-weight at bottom fibers $= \dfrac{M_d}{Z_b}$

$$= \frac{\left(0.315 \times 4.88^2 \times 10^6\right)/8}{\left(3.226 \times 10^7\right)/97.2}$$

= 2.83 N/mm^2

Stress due to self-weight at top fibers $= \dfrac{M_d}{Z_b}$

$$= \frac{\left(0.315 \times 4.88^2 \times 10^6\right)/8}{\left(3.226 \times 10^7\right)/77.8}$$

= 2.26 N/mm^2

Stresses due to 50 mm thick In-situ topping.

Stress due to in- situ topping at bottom fibers $= \dfrac{M_{ds}}{Z_b}$

$$= \frac{\left(0.72 \times 4.88^2 \times 10^6\right)/8}{\left(3.226 \times 10^7\right)/97.2}$$

= 6.46 N/mm^2

Stress due to in-situ topping at top fibers $= \dfrac{M_{ds}}{Z_t}$

$$= \frac{\left(0.72 \times 4.88^2 \times 10^6\right)/8}{\left(3.226 \times 10^7\right)/77.8} \qquad = 5.15\text{N/mm}^2$$

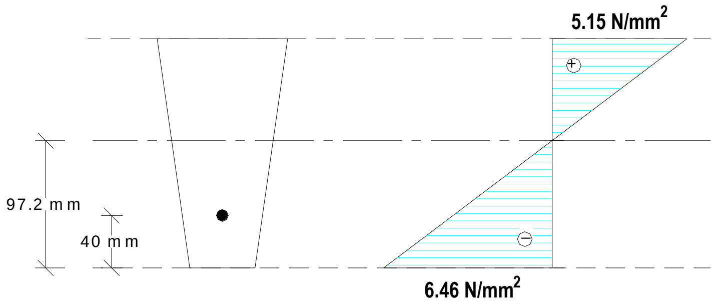

Figure 05: Stress due to 50 mm thick in-situ topping

Stresses due to Imposed load.

Imposed load; $w_i = 1.5 \times 0.6$

= 0.9 kN/m

Stress due to imposed load at bottom fibers $= \dfrac{M_{i\,max}}{Z_{b,comp}}$

$$= \frac{(0.9 \times 4.88^2 \times 10^6)/8}{(9.777 \times 10^7)/151.2}$$

= 4.14 N/mm^2

Stress due to imposed load at top fibers $= \dfrac{M_{i\,max}}{Z_{t,comp}}$

$$= \frac{(0.9 \times 4.88^2 \times 10^6)/8}{(9.777 \times 10^7)/48.8}$$

= 1.34 N/mm^2

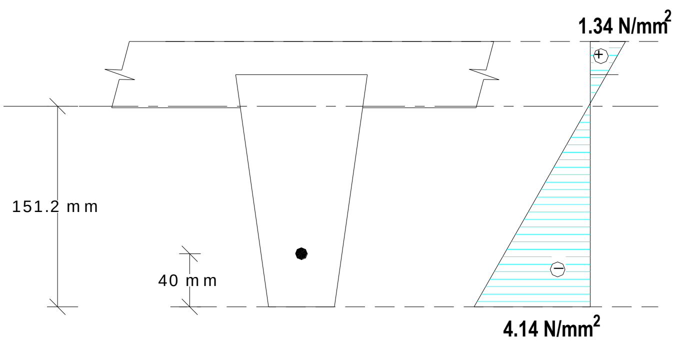

Figure 06: Stress due to imposed load

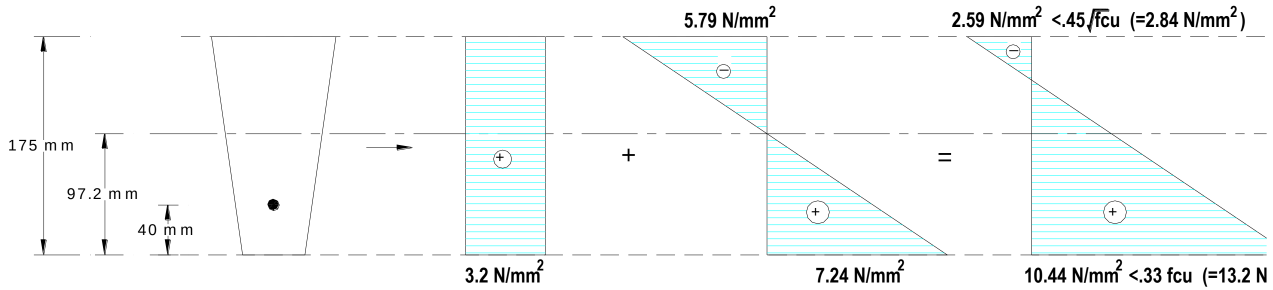

Figure 07: Stress at transfer

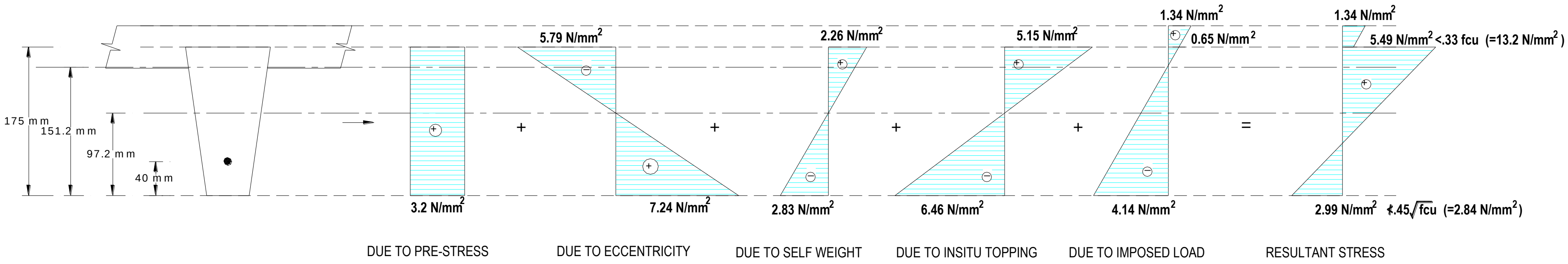

Figure 08 : Stress at servicebility condition

1.3 Horizontal Shear Transfer in Composite Beams

The composite behavior of the precast beam and the in-situ slab is only effective if the horizontal shear at the interface between two elements can be transferred. If no shear resistance exists and a load is applied to the composite beam, the slab would slide relative to the beam and the two elements behave separately. However, if sufficient shear resistance is provided, the slip will not take place and full composite action will be ensured.

Horizontal shear stress can be calculated by considering total compressive force in the slab transferred across at ultimate load. This approach has been recommended by BS8110 and if this condition is satisfied, it may be assumed that satisfactory horizontal shear resistance is provided at the SLS. BS 8110 recommends that the horizontal shear force (V_h) due to design ultimate load can be taken as follows.

- When the interface is in the tension zone: V_h = Total compression (or tension) due to ultimate bending moment
- When the interface is in the compression zone: V_h = The compression from that part of the compression zone above the interface due to ultimate bending moment

The average shear stress is calculated by dividing the horizontal shear force by the area obtained by multiplying the contact width by the beam length between the point of maximum moment and the point of zero moment. The average horizontal shear stress is then distributed along the length of the interface in proportion to the vertical shear force diagram. This is discussed in cl.5.4.7.2 of BS8110: Part1 :1985

$$V_h = (0.45\ f_{cu})bh$$

Average horizontal design shear stress; $(v_h)_{avg} = \dfrac{V_h}{b_i l} = \dfrac{V_h}{b_i (L_b / 2)}$

The design shear stress; $v_h = 2 \times (v_h)_{avg}$

According to the Cl.5.4.7.3 of BS8110: Part1 :1985 the area of nominal links should be at least 0.15% of the contact area. Area should not be excessive. Links should be adequately anchored on both side of the interface. Horizontal shear stress at interface between precast beam and in-situ topping can be calculated as given below.

Note: for all types of beams, interface is in the compression zone.

Average horizontal design shear stress; $(v_h)_{avg} = \dfrac{V_h}{b_i(L_b/2)}$

$$= \frac{0.45 \times 30 \times 600 \times 25}{150 \times 2440}$$

= 0.553 N/mm^2

Design shear stress $= 0.553 \times 2$

=1.106 N/mm^2

But from Table 5.5 of section 5 of BS 8110 (for as cast concrete of grade 30 with nominal links projecting into in-situ concrete);

Design ultimate horizontal shear stress at the interface=1.80 N/mm^2

Hence nominal amount of reinforcement is satisfactory.

Area of nominal links = 0.15% x contact area

= 0.15% x150x 1000

= 225 mm^2/m

No. of R6 links required = 225/56.55(area of R6 link)

= 4 links

Hence R6 links at 250mm spacing should be provided to transfer horizontal shear between pre-stressed beam and in-situ topping as shown below.

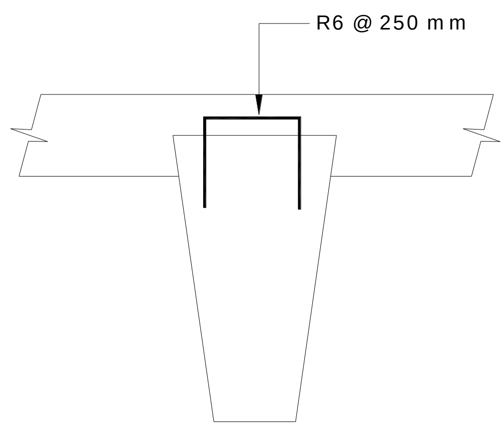

Figure 09: Shear link arrangement in PSC slab purlin

4.4 Design of 50 mm Thick Slab in Transverse Direction

Since the NERDC Composite slab is spanning in one direction, a span/ depth ratio of 34 is considered.

Hence effective depth $= \frac{0.6 \times 10^3}{34}$

=18 mm

We can use a cover of 20 mm (mild exposure condition).

Assume bar diameter of 3mm.

H=50 mm and d = 50-25-3/2 =23.5 mm

Loading (for one meter strip)

Self-weight =0.05x24x1	=1.2 kN/m
Finishes=1.0x1	=1.0 kN/m
Total dead (g_k)	= 2.2 kN/m
Imposed load =1.5x1.0	=1.5 kN/m
Total imposed (q_k)	= 1.5 kN/m
Design load	= 1.4x2.2+1.6x1.5
	=5.48 kN/m

Since $g_k > q_k$ and $q_k < 5$ kN/m, and if we assume that bay size>30m^2 for an interior panel bending moment and shear force can be obtained from Table 3.13/BS 8110.

Span moment	= (0.063) F.l
	= 0.063x5.48x0.6^2
	= 0.124 kN/m
Support moment	= (-0.063) F.l

$= -0.063 x 5.48 x 0.6^2$

$= -0.124$ kNm/m

Shear at support = 0.5F

= 0.5x5.48x0.6

= 1.64 kNm/m

Design for bending at span

$$k = \frac{M}{bd^2 f_{cu}}$$

$$= \frac{0.124 \times 10^6}{1000 \times 23.5^2 \times 30}$$

= 0.007<k' (=0.156)

Hence compression reinforcement is not necessary.

$$Z = d\left[0.5 + \sqrt{\left(0.25 - \frac{k}{0.9}\right)}\right]$$

$$= d\left[0.5 + \sqrt{\left(0.25 - \frac{0.005}{0.9}\right)}\right]$$

= 0.99d

Take Z= 0.95d

$$As = \frac{M}{0.87f_y Z}$$

$$= \frac{0.124 \times 10^6}{0.87 \times 250 \times 0.95 \times 28.5}$$

$=21.06\ mm^2/m$

Hence 3mm mild steel at 50mm spacing is satisfactory.

$\frac{100As}{Ac} = \frac{100 \times 7.07 \times 21}{1000 \times 50} = 0.29 > 0.24$; (from table 3.27 of BS8110 for f_y=250N/mm^2

and tension reinforcement for rectangular section)

Spacing<3d; (23.5mmx3=70.5mm)

Hence minimum steel and crack control O.K.

Check for deflection:

$$\frac{M}{bd^2} = \frac{0.124 \times 10^6}{1000 \times 28.5^2} = 0.153$$

$$f_s = \frac{5}{8} \times 250 \times \frac{21.06}{148.47} = 22.16$$

$$F_1 = 0.55 + \frac{477 - 22.16}{120(0.9 + 0.153)}$$

$= 4.14$

Hence take F_1=2 (Table3.11/BS8110)

Allowable span/depth=26x2=52

Actual span/depth =600/28.5 =21 <52   Hence satisfactory

<u>Check for shear:</u>

$$v = \frac{1.64 \times 10^3}{1000 \times 50} = 0.032$$

$$100\frac{A_s}{b_v d} = \frac{100 \times 148}{1000 \times 50} = 0.296$$

from Table 3.9 for d= 28.5mm and f_{cu}=30 N/mm^2

$$vc = \frac{0.79 \times (0.296)^{1/3} \times (400/28.5)^{1/4} \times (30/25)^{1/3}}{1.25}$$

$$= 0.865 > 0.032$$

Hence from Table 3.17/ BS 8110, shear reinforcement is not required.

Appendix 02

Past Projects

Item No.	**Project**	**Construction year**	**Duration**	**Total project cost**	**Floor area m^2**	**Cost per floor area of each**	**Used technologies**
1.0	Officer's quarters for Forest	1993	12Months	2.23 M	60	242,280.00	4" x 4" Pre-stressed columns
	Department				per each	0.24M	Pocket foundation
	Kithulgala						4" x 3" Plinth beam
	Imbulpitiya						4" x 3" tie beams
	Rambukwella						Micro tiles for roof
	Rikartan						
	Neluwa						
2.0	Construction of Two storied Model	1995	4Months	0.6M	140	565,320.00	5" x 5" Pre-stressed columns
						0.56M	Pocket foundation
							5" x 3" Plinth beam
							5" x 3" tie beams
							Micro tiles for roof
3.0	Girl's hostel at Maddewatta for	1995	12Months	11.1M			* Foundation with plinth beams
	University of Ruhuna						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Slipform walls /Cement sand

							block walls
4.0	Hostel building at RDTI	1997	18Months	7.68 M			* Foundation with plinth beams
	Pilimathalawa						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement soil block /Cement sand
							block walls
5.0	Electrical and Electronics Building	1998	12 Months	7.15M	1115	4,502,370.00	* Foundation with plinth beams
	at NERDC					4.5M	* 6" x 6" Pre-stressed Columns
						-	* NERDC composite floor slab
						-	* Concrete Door window frames
						-	* Ferro cement Canopies
						-	* PSC Trusses & PSC purlins for
						-	roof
						-	*Slipform walls /Cement sand
						-	block walls

6.0	Management Development &	1998	5 Months	.9M	150	605,700.00	* Foundation with plinth beams
	Training unit Anuradhapura					-	* 6" x 6" / 4"x 4" Pre-stressed
						-	Columns
						-	Slipform wall & Cement block walls
						-	Cement quarry dust mix floor
						-	Con. Door & window frame
						-	* PSC Trusses & PSC purlins for
						-	roof
						-	Slipform walls/ Cement block walls
7.0	Canteen & welfare Building	1999	11 Months	2.4 M		-	* Foundation with plinth beams
	at NERDC					-	* 6" x 6" Pre-stressed Columns
						-	* NERDC composite floor slab
						-	* Concrete Door window frames
						-	* Ferro cement Canopies
						-	* PSC Trusses & PSC purlins for
						-	roof
						-	*Cement soil block /Cement sand

						-	block walls
8.0	Minuwangoda fair	1999	24 Months	3.1 M	972 m^2	#VALUE!	* Foundation with plinth beams
						-	* 6" x 6" / 4"x 4" Pre-stressed
	4 Part Block			4.40 M	623 m^2	#VALUE!	Columns
						-	Slipform wall & Cement block walls
	2 Part Block			0.610 M	78 m^2	#VALUE!	Cement quarry dust mix floor
						-	Con. Door & window frame
	3 Part Block			0.120 M	156 m^2	#VALUE!	* PSC Trusses & PSC purlins for
						-	roof
9.0	MIRJE Office Building	2000	24 Months	4.41 M	435 m^2		* Foundation with plinth beams
	Movement for Inter Racial Justice						* 6" x 6" Pre-stressed Columns
	and Equality at Nugegoda						* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement soil block /Cement sand
							block walls

10.0	NAITA Building Project at	2000	12Months	1.40 M			* Foundation with plinth beams
	Matara						* 6" x 6" / 4"x 4" Pre-stressed
							Columns
							Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/ Cement block walls
11.0	Building complex for Hiriyala	2000	12 Months	40.0M	3500m^2		* Foundation with plinth beams
	Economic Centre						* 6" x 6" / 4"x 4" Pre-stressed
							Columns
							Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/ Cement block walls
12.0	Civil Engineering Department	2000	12 Months	3.5 M			* Foundation with plinth beams
	Building at NERDC						* 6" x 6" / 4"x 4" Pre-stressed
							Columns

							Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/ Cement block walls
13.0	Student Hostel for National	2000	12Months	18.9 M			* Foundation with plinth beams
	Institute of Technical Education						* 6" x 6" / 4"x 4" Pre-stressed
	Rathmalana						Columns
							Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/ Cement block walls
							* NERDC composite floor slab
14.0	IT Centre for University of	2002	12 Months	12.8 M			* Foundation with plinth beams
	Peradeniya						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof

							*Slipform walls /Cement sand
							block walls
15.0	Construction of quarters for STF	2002	12 Months				
	Police Constable barrack			.75M	$93m^2$		* Foundation with plinth beams
	Police Sergeants barrack			.57M	$56m^2$		* 6" x 6" / 4"x 4" Pre-stressed
	Police Officers quarters			.9M	$70m^2$		Columns
	Administration Building			1.6M	$150m^2$		Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/ Cement block walls
16.0	Construction of 3 houses and	2002	6 Months	3.5 M	IT Centre		* Foundation with plinth beams
	IT centre for Ranaviru				285 m^2		* 6" x 6" / 4"x 4" Pre-stressed
	Sewa Authority - Mailapitiya				Houses		Columns
					550 m^2 x 3		Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
17.0	Gymnasium at Saliyapura	2002	10Months				* NERDC composite floor slab

	Camp - Anuradhapura						
18.0	Residential Building for Sri	2002	4Months	2.2 M	345 m^2		* Foundation with plinth beams
	Lanka Army - Saliyapura Camp						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement soil block /Cement sand
							block walls
19.0	Agricultural Department	2002	24 Months	10.4 M	650m^2		* Foundation with plinth beams
	Building at NERDC						* 6" x 6" / 4"x 4" Pre-stressed
							Columns
							Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/ Cement block walls
							* NERDC composite floor slab
20.0	Twin Houses for Institute of	2002	3Months	2.5M	99 m^2		* Foundation with plinth beams
	Post Harvest Technology						* 6" x 6" / 4"x 4" Pre-stressed

							Columns
							Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/ Cement block walls
21.0	Construction of Officers	2003	10Months	3.30 M	408 m^2		* Foundation with plinth beams
	Quarters for Army Camp						* 6" x 6" / 4"x 4" Pre-stressed
	Panagoda						Columns
							Slipform wall & Cement block walls
							Cement quarry dust mix floor
							Con. Door & window frame
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/ Cement block walls
22.0	Hostel for Technology Park	2003	6Months	6.0M	525m^2		* Foundation with plinth beams
	at NERDC						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies

							* PSC Trusses & PSC purlins for
							roof
							*Slipform walls /Cement sand
							block walls
23.0	Office and Work Shop Building	2005	12 Months	9.12 M	613 m^2		* Foundation with plinth beams
	for the Renewable Energy						* 6" x 6" Pre-stressed Columns
	Department at NERDC						* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement sand block walls
24.0	Don Bosco Tsunami Housing	2005	24 Months	200M	750m^2 each		* Foundation with plinth beams
	Complex						* 6" x 6" Pre-stressed Columns
	3 storied Community hall +						* NERDC composite floor slab
	4 storied 13 Nos of buildings						* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement soil block /Cement sand
							block walls
25.0	North East Housing	2005	12Months	each .18M	each		* Foundation with plinth beams
	Reconstruction Programme				44.12 m^2		* 6" x 6" / 4"x 4" Pre-stressed

	(NEHRP)						Columns
	20 Houses						* Concrete Door window frames
							* PSC Trusses & PSC purlins for
							roof
							*Cement block walls
26.0	Workshop building for Renewable	2005	12 Months	9.6M	890m^2		* Foundation with plinth beams
	Energy Department at NERDC						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Slipform walls /Cement sand
							block walls
27.0	Engineering Museum at NERDC	2005	60 Months	23.0 M	3344 m^2		* Foundation with plinth beams
							* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* 6"slip form walls
							Pre-streesed wall panels
							* Ferro cement Canopies
28.0	Vidatha Resource Centres	2006	6Months	1.46 M	140m^2		* Foundation with plinth beams
	Horana						* 6" x 6" / 4" x 4" Pre-stressed
							Columns
							* Concrete Door window frames

							* PSC Trusses & PSC purlins for
							roof
							*Cement block walls
29.0	Vidatha Resource Centres	2007	12 Months	2.07 M	143.98 m^2		* Foundation with plinth beams
	Kesbewa						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement block walls
30.0	TMD Building at NERDC	2007	08 Months	10.15 M	44.24 m2		* Foundation with plinth beams
							* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement soil block walls
31.0	Vidatha Resource Centres	2007	07 Months	4.6 M	137.66 m^2		* Foundation with plinth beams
	Yatiyanthota						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies

							* PSC Trusses & PSC purlins for
							roof
32.0	Vidatha Resource Centres	2008	6 Months	1.51 M	$75m^2$		* Foundation with plinth beams
	Dodangoda						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement block walls
33.0	Hostel Building for	2008	12 Months	7.5M	$360m^2$		* Foundation with plinth beams
	Sabaragamuwa University						* 6" x 6" / 4"x 4" Pre-stressed
							Columns
							* Concrete Door window frames
							* PSC Trusses & PSC purlins for
							roof
							*Cement block walls
34.0	Training Centre for Vocational	2008					* Foundation with plinth beams
	Training Authority						* 6" x 6" / 4"x 4" Pre-stressed
							Columns
							* Concrete Door window frames
							* PSC Trusses & PSC purlins for
							roof
							*Cement block walls

35.0	Vidatha Resource Centres	2008	6 Months	2.18 M	140m^2		* Foundation with plinth beams
	Madurawala						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
36.0	Construction of Building for Sri	2010					* Foundation with plinth beams
	Lanka Prison at Wariyapola						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement block walls
37.0	Canteen Building for	2010	12 Months	16.00M			* Foundation with plinth beams
	Dharmapala Vidyalaya						* 6" x 6" Pre-stressed Columns
	Pannipitiya						* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement sand block walls

38.0	School Library Building	2011	24Months	.8M	$60m^2$		* Foundation with plinth beams
	Development Project in			per each	per each		*6" stabilized cement soil blocks
	Monaragala (25 buildings)						*Concrete Door & window frames
39.0	Vidatha Resource Centres	2011	3.5 Months	7.00 M	$185m^2$		* Foundation with plinth beams
	Karaweddi - Jaffna						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
40.0	Construction of Vidatha Resource	2011	6 Months	7.9M	$201.85\ m^2$		* Foundation with plinth beams
	Centre Imbulpe						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
41.0	Construction of Building for Vijaya	2011	8 Months	8.0M	$230m^2$		* Foundation with plinth beams
	Kumarathunga Memorial Hospiltal						* 6" x 6" Pre-stressed Columns
	(3 Storied)						* NERDC composite floor slab
							* Concrete Door window frames

							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							Slipform walls/Cement block walls
42.0	Nurses Quarters for	2011	24 Months	26 M	1073 m^2		* Foundation with plinth beams
	Polonnaruwa Base Hospital						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement soil block walls
43.0	Green Cabana for Air Force	2012	5 Months	1.4M	50m^2		* Foundation with plinth beams
	Camp - Ekala						*6"x6" pre-stressed columns
							*6" stabilized cement soil blocks
							NERD Slab
							*Concrete Door & window frames
							*Pre-cast Stair case including
							Pre -cast stringer beams with pre
							-stressed concrete steps
44.0	Construction of Vidatha Resource	2012	6 Months	7.9M	201.85 m^2		* Foundation with plinth beams
	Centre Nivithigala						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames

							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
45.0	Construction of Vidatha Resource	2012	6 Months	11.8M	201.85 m^2		* Foundation with plinth beams
	Centre Kiriella						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							* Cement soil blocks
46.0	Construction of Vidatha Resource	2012	6 Months	8.07M	192m^2		* Foundation with plinth beams
	Centre Opanayake						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
47.0	Construction of Vidatha Resource	2012	6 Months	8.67M	192m^2		* Foundation with plinth beams
	Centre Weligepola						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for

							roof
48.0	School Library Building	2012	24Months	1.0M	75m^2		* Foundation with plinth beams
	Development Project in			per each	per each		6" x 6" Pre-stressed columns
	Anuradhapura (25 buildings)						*6" stabilized cement soil blocks
							*Concrete Door & window frames
							*slipform walls
49.0	Model House at Technology	2013	5 Months	3.10 M	108 m^2		* Foundation with plinth beams
	Park						*6"x6" pre-stressed columns
							*6" stabilized cement soil blocks
							* NERDC composite floor slab
							*Concrete Door & window frames
							*Pre-cast Stair case including
							Pre cast stringer beams with pre
							stressed concrete steps
50.0	Two storied Accomodation	2013	12 Months	7.52 M			* Foundation with plinth beams
	Building at Panagoda Camp for						*6" x 6" pre-stressed columns
	Sri Lanka Army						*6" slip form walls
							* NERDC composite floor slab
							*Concrete Door & window frames
51.0	Construction of Vidatha Resource	2013	6 Months	7.90M	205m^2		* Foundation with plinth beams

	Centre Balangoda						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
52.0	Construction of Vidatha Resource	2013	6 Months	5.73M	138m^2		* Foundation with plinth beams
	Centre Pannala						* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
53.0	Laboratory Building for Vijaya	2013	12 Months	12.5 M	407 m^2		* Foundation with plinth beams
	Kumarathunga Hospital at Seeduwa						*6"x6" pre-stressed columns
	(3 storied Building)						* NERDC composite floor slab
							*Concrete Door & window frames
							*Cement sand block walls
54.0	Library Building at NERDC	2013	12 Months	6.5 M	225 m^2		* Foundation with plinth beams
							* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for

							roof
							*Cement soil block walls
55.0	Construction of school buildings	2013	9Months	12.0M	260m^2		* Foundation with plinth beams
	Watareka						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							* Slipform walls /Cement sand
							block walls
56.0	Construction of school buildings	2013	9Months	10.0M	270m^2		* Foundation with plinth beams
	Pitumpe						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Slipform walls /Cement sand
							block walls
57.0	School Library Building	2014	24Months	1.2M	120m^2		* Foundation with plinth beams
	Development Project in			per each	per each		6" x 6" Pre-stressed columns
	Hambanthota (50 buildings)						*6" stabilized cement soil blocks
							*Concrete Door & window frames

58.0	Construction of Vidatha Resource	2014	6 Months	7.09M	192m^2		* Foundation with plinth beams
	Centre Glenbindunuwewa						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
59.0	Circuit Bunglow for the Dept. of	2014	5Months	2.5 M	88 m^2		* Foundation with plinth beams
	Wild life Conservation at Galoya						*6"x6" pre-stressed columns
	National Park						*6" stabilized cement soil blocks
							NERD Slab
							*Concrete Door & window frames
							*Pre-cast Stair case including
							Pre cast stringer beams with pre
							stressed concrete steps
60.0	Construction of Vidatha Resource	2015	6 Months	7.09M	205m^2		* Foundation with plinth beams
	Centre Madampe						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof

61.0	Construction of Building for Medical	2015	9Months	13.02 M	312 m^2		* Foundation with plinth beams
	Centre Dodangoda MOH						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement soil block walls
62.0	Construction of Clinical Building	2015	9Months	22.6 M	491.74 m^2		* Foundation with plinth beams
	for Kothalawala, Rathmalana						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
							*Cement sand block walls
63.0	Construction of Vidatha	2015	5 Months	5 M	197 m^2		* Foundation with plinth beams
	Resource Centre - Kahawatta						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof

64.0	Construction of Vidatha Resource	2015	6 Months	9.8 M	197m^2		* Foundation with plinth beams
	Centre - Sainthamarudhu						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC perlins for
							roof
65.0	Construction of Vidatha Resource	2015	8 Months	9.2 M	192m^2		* Foundation with plinth beams
	Centre - Nallur						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
66.0	Construction of Vidatha Resource	2015	8 Months	9.5 M	192m^2		* Foundation with plinth beams
	Centre - Sandilipay						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
67.0	Construction of Vidatha Resource	2015	12 Months	7.3 M	192m^2		* Foundation with plinth beams
	Centre - Musali						* 6" x 6" Pre-stressed Columns

							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof
68.0	Construction of Vidatha Resource	2016	8 Months	8.0 M	201.85 m^2		* Foundation with plinth beams
	Centre Padukka						* 6" x 6" Pre-stressed Columns
							* NERDC composite floor slab
							* Concrete Door window frames
							* Ferro cement Canopies
							* PSC Trusses & PSC purlins for
							roof

Appendix 03

Cost Calculation

Cost calculations for 1 m^2

<u>50mm thick floor slab with G10 50mm x 50mm wire mesh with decorated panel Concrete 1:1 1/2 :3 (3/4") components from NERDC licence yards</u>

Consider 24'-0"x 24'-0" - 576 ft2	96	Ft3	0.96	Cbs
Materials				
Concrete				
Cement	Bags	22.08	1095	24,177.60
Sand	Cbs	0.4	16000	6,400.00
Metal	Cbs	0.79	8000	6,320.00
Wastages 10%				3,689.76
Basket & water		Allow		100
2"x2" welded mesh (G10)	ft2	633.6	50	31,680.00
5" pre-stressed beams	ft	240	175	42,000.00
2' x 4' Decorated panels	Nos	72	635	45,720.00
Transport		Allow		1,000.00
material for filling the gap				1,000.00
Total material				162,087.36
Labour (Laying of purlins, mixing & placing)				
Skilled Mason	Day	4	2000	8,000.00
Unskilled Labour	Day	20	1500	30,000.00
Labour (Filling the gap between slab & beam & finishing)				
Skilled Mason	Day	2	2000	4,000.00
Unskilled Labour	Day	2	1500	3,000.00
Allow 5% for scaffolding				2,250.00
Total labour				47,250.00
Rate per 576 ft2				209,337.36
Rate per 51.84m2				209,337.36
Rate per 1 ft2				363.43
Rate per 1 m2				4038.14
Add 20% over heads & profit				72.69
Add 20% over heads & profit				807.63
Total cost per 1 ft2				436.12
Say for 1 ft2			Rs.	436
Say for 1 m2			Rs.	4846

50mm thick floor slab with G10 50mm x 50mm wire mesh with suspended form work Concrete 1:1 1/2 :3 (3/4") components from NERDC licence yards

Consider 24'-0"x 24'-0" - 576 ft2	96	Ft3	0.96	Cbs
Materials				
Concrete				
Cement	Bags	22.08	1095	24,177.60
Sand	Cbs	0.4	16000	6,451.20
Metal	Cbs	0.79	8000	6,297.60
Wastages 10%				3,692.64
Basket & water		Allow		100
2"x2" welded mesh (G10)	ft2	633.6	50	31,680.00
5" pre-stressed beams	ft	240	175	42,000.00
Transport		Allow		1,000.00
material for filling the gap				1,000.00
Binding wire		Allow		50
Props		Allow		500
Total material				116,949.04
Form Work				
2"x 2" class ii timber	ft	336	60	20,160.00
8"x4" plywood sheet	ft	288	50	14,400.00
wire nails & screws etc.				200
Skilled Labour : Cutting & fixing	Day	2	2000	4,000.00
Unskilled Labour : Cutting & fixing	Day	2	1500	3,000.00
Shuttering 2 times used				20,880.00
Labour (Laying of purlins, mixing & placing)				
Skilled Mason	Day	4	2000	8,000.00
Unskilled Labour	Day	25	1500	37,500.00
Labour (Filling the gap between slab & beam & finishing)				
Skilled Mason	Day	2	2000	4,000.00
Unskilled Labour	Day	2	1500	3,000.00
Allow 5% for scaffolding				2,625.00
Total labour				55,125.00
Rate per 576 ft2				192,954.04
Rate per 51.84m2				192,954.04
Rate per 1 ft2				334.99

Rate per 1 m2				3722.11
Add 20% over heads & profit				67
Add 20% over heads & profit				744.42
Total cost per 1 ft2				401.99
Total cost per 1 m2				4466.53
Say			Rs.	400
Say for 1 m2				4465

Appendix 04

Cube Test Results

Compressive strength test results on Grade 40 concrete cubs

NERD Centre	Materials Test Records	Page No: 1 of 1	
Compressive Strength test on Concrete Cube			
Test Method	BS-1881	Invoice No:	
Casting Date	24.04.2019.	Testing Date	12.06.2019.

Reference of Requestion	-
Sample Description	Compressive Strength test on Concrete Cube
Sample Details	-
Instrument	Compressive Testing Machine.

Client	Mr.P.N.S.Amaradasa.
Address	NERDC
Name of Structure	
Remarks	Concrete Grade 40

Serial No of Cube		1	2	
Age of Cube.	(No of Days)	48	48	
Length of Cube.	mm	150	150	
Sectional Area	(mm^2)	22500	22500	
Height of Cube.	mm	150	150	
Weight of cube	Kgs	7.850	8.050	
Maximum Load	(KN)	890.9	963.5	
Compressive Strength.	**(N/mm^2)**	**39.60**	**42.82**	
Average Compressive Strength.				**41.21**
Mini. Average Required.	**(N/mm^2)**			

	Name	Designation	Signature	Date
Tested By	D. H. E. Kalyanasiri	Technical Officer.		
Checked By	Eng I.P.Batuwita	Research Engineer		
Approved By	Eng. J.A.C. Charishanthi.	Director / Civil		

Note: The test results reported herein related to the specimens submitted to NERDC and do not certify the quality of a product in general.

Compressive strength test results on Grade 30 concrete cubs

NERD Centre	Materials Test Records		Page No	1 of 1
Compressive Strength test on Concrete Cube				
Test Method	BS-1881	Invoice No		
Casting Date	16.06.2019	Testing Date	16.07.2019	

Reference of Requisition	
Sample Definition	Compressive Strength test on Concrete Cube
Sample Details	
Instrument	Compressive Testing Machine
Client	Mr P N S Amarasekara
Address	NERDC
Name of Structure	Casting of 6' - 0" x 6' - 0" NERD Slab with sheet ..., Casting of 6' - 0" x 5' - 0" 2" thick slab 1Nos
Remarks	Concrete Grade ~~40~~ 30

Serial No of Cube		1	2	3	4	5	6
Age of Cube	(No of Days)	30	30	30	30	30	30
Length of Cube	mm	150	150	150	150	150	150
Sectional Area	(mm²)	22500	22500	22500	22500	22500	22500
Height of Cube	mm	150	150	150	150	150	150
Weight of cube	Kgs	7.900	7.980	8.000	8.100	7.850	8.500
Maximum Load	(KN)	445.1	647.5	780.5	876.8	615.6	747.8
Compressive Strength.	**(N/mm²)**	**15.96**	**28.78**	**34.69**	**38.97**	**27.36**	**33.24**
Average Compressive Strength.		**30.51**					
Min. Average Required.	**(N/mm²)**						

	Name	Designation	Signature	Date
Tested By	D. H. F. Kalyanasiri	Technical Officer	[signature]	07/09/2019
Checked By	Eng I P Balawela	Research Engineer		
Approved By	Eng. J.A.C. Chandrasiri	Director / Civil		

Note: The test results reported herein related to the specimens submitted to NERDC and do not certify the quality of a product in general.