Evaluation and Comparison of Strengthening Methods to Deliver a Safe, Efficient and Economical Solution

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ABSTRACT

The challenges posed in the choice selection of strengthening methods to strengthen old existing buildings which might be exposed to external loads, poor concrete grade, poor construction and review in codes has been of concern recently. Through the various guidelines for building evaluation, strengthening and with innovative structural codes for design, building assessment and strengthening are carried out using newly developed technologies worldwide.

Basically, there are two major categories of strengthening; local and global methods. Local method is focused at the element level on structural members which are deficient and need improvement to perform better. This method includes adding composites, concrete or steel on the surface of a structural member. They are all effective but also have their disadvantages, while the global method acts on the structural level. Its application will lead to obtaining the behavior of the entire structure. This method consists of addition of steel bracings, shear walls and infill walls. These methods equally have their effectiveness and disadvantages.

In this study, strengthening methods were discussed considering its advantages and disadvantages where some application procedures were as well highlighted. The procedures for building assessment were also discussed. A coded decision selection program for strengthening methods were constructed which will help for selecting the best option from strengthening methods. Coded strengthening programs for fiber reinforced polymer (FRP) were also prepared and it gave the same result with the referenced FRP strengthening. Two case studies were carried out in other to compare the result with the coded decision selection tool. The cases studies were modeled and designed with structural software such as STA4CAD, CSi COL and Engissol structural software. The selection tool and the case studies gave the same result for the strengthening option. Both have shown that shear wall strengthening is the best option for strengthening the two investigated buildings. The economic evaluation of the materials used in different strengthening method studied, has shown that, shear wall is the least in the cost of strengthening. Therefore, shear wall strengthening method is cheaper, efficient and has significantly contributed to the overall strengthening and improvement of the performance of the considered building.

Keywords: Strengthening, Performance, Global Strategy, Local Strategy, Ductility.

ÖZ

Değişen dış yükler, yetersiz beton sınıfı, kötü işçilik ve şartnamelerde yapılan değişikliklere bağlı olarak mevcut binaların değişik yöntemlerle güçlendirilmesi ve buna bağlı sorunlar günümüzde önemli bir yer tutmaktadır. Buna bağlı olarak, binaların performanslarının değerlendirilmesi ve güçlendirilmesi için birçok yeni geliştirilen teknolojiler dünya çapında kullanılmaktadır.

Temel olarak güçlendirme iki ana kategoriye göre sınıflandırılabilir; lokal ve global yöntemler. Lokal yöntemde eleman bazındaki yetersizliklerin giderilmesi yeterli olacaktır. Bu yöntem, bir yapı elemanı yüzeyine kompozit malzemeler, beton veya çelik eklenmesini kapsamaktadır. Bu metodlar ilgili yapı elemanlarının performansını artırmada etkin olmalarına karşın bazı dezavantajları da bulunmaktadır. Global yöntemde ise yapısal düzeyde sistemin iyileştirilmesi gerekmektedir. Bu yöntem çelik çaprazların, perde ve dolgu duvarların eklenmesine dayanır. Bu yöntemlerin de avantajları yanında dezavantajları da mevcuttur.

Bu çalışmada, güçlendirme yöntemleri bazı uygulama detayları ile avantajları ve dezavantajları ışığında tartışılmıştır. Ayrıca, bina değerlendirme prosedürü de incelenmiştir. Uygulamalarda en uygun güçlendirme yöntemini seçecek bir bilgisayar yazılımı hazırlanmıştır. Çalışmada, karbon lifli dokuma ile güçlendirme uygulamaları için bir bilgisayar yazılımı düzenlenerek literaürdeki çözümlerle karşılaştırılmıştır. Seçilen iki mevcut binaya performans analizinden sonra STA4CAD, CSİ COL ve Engissol programları yardımı ile çeşitli güçlendirme yöntemleri uygulanmıştır. Sonuç olarak en ekonomik yöntem araştırıldığı zaman bunun perde kullanılarak yapılan güçlendirme olduğu saptanmıştır. Güçlendirme yöntemini seçmek için hazırlanan bilgisayar programı ile de aynı sonuca varılmıştır.

Anahtar Kelimeler: Güçlendirme, Performans, Global Yönem, Lokal Yöntem, Süneklik.

DEDICATION

I dedicate this page to my lovely mother Mrs B. E Okakpu.

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I owe quit a lot to my family who allowed me to travel all the way from Nigeria to North Cyprus and supported me all throughout my studies. I would like to dedicate this study to them as an indication of their significance in this study as well as in my life.

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$A_{f,req}$	The required area of FRP
A_{frp}	The area of FRP
A_{fv}	The area of FRP shear reinforcement
Ag	The gross-sectional area
Ast1	The area of tension steel
Ast2	The area of compression steel
b	The width of beam
b_f	The beam flange width
Cs	The compression side
C_g	The dead load
C_q	The live load
C_s	The earth pressure
C_e	The seismic notation
C_w	The wind
C_t	The thermal notation
d	The effective depth of the beam
d_{fe}	The effective depth of FRP shear reinforcement
E_c	The modulus of elasticity
E_{cc}	The long term elasticity of steel
E_s	The modulus of elasticity of steel
F_{frp}	Tensile stress of fiber reinforced polymer
E_{fk}	The elastic modulus of fiber reinforced polymer
f_{ck}	The compressive strength of concrete

f'_c	The characteristic cylinder compressive strength
f'_{cc}	The apparent confined concrete strength
f_p	The confining pressure by FRP
f_y	The characteristic strength of steel
f_s	The tensile stress in steel reinforcement
h	The height of the beam
k_1	The multipliers
L_e	The effective bond length of FRP strip
L_0	The effective bond length of one ply of FRP
M_u	The redesigned moment
M_1	The initial moment condition of the beam
NA	The neutral axis
n_f	The number of FRP
P_u	The nominal axial capacity
s_f	The spacing of FRP
T_{fp}	The tensile force in FRP
T_s	The tension side
t_j	The jacket thickness
t_f	The FRP thickness
V_c	The design shear resistance provided by the concrete
V_f	The shear contribution of the FRP
V_r	Shear capacity resistance
V_u	The maximum factored shear force
X_u	The distance of the neutral axis
x_d	The distance from the neutral axis

ε_j	The maximum allowable strain of FRP
$\emptyset$	The reinforcement diameter
Ψ_f	The partial safety factor for FRP material
γ_{mc}	The partial safety factor for concrete
γ_{mE}	The partial safety factor for the modulus of elasticity of FRP
γ_{ms}	The partial safety factor for steel

LIST OF ABBREVIATIONS

ACI	American Concrete Institute
AFRP	Aramid Fiber Reinforced Polymer
ASTM	American Society for Testing and Materials
CFRP	Carbon Fiber Reinforced Polymer
CPWD	Central Public Works Department, Government of India
CC	Collapse Case
CP	Collapse Prevention
CSA	Canadian Standards Association
GFRP	Glass Fiber Reinforce Polymer
FRP	Fiber Reinforced Polymer
I	Immediate Occupancy
LS	Life Safety
NCHRP	National Cooperation highway research program
TR55	Concrete Society Technical Report 55

Chapter 1

INTRODUCTION

Structures which are not safe and weak in service are thought to be demolished and a new one is usually erected. Nowadays, it is preferable to strengthen old existing buildings instead of demolishing and setting up a new one. Engineers who assess old existing buildings either for increased live loads, change of use of the building, new design codes, revision in loading standard etc. have tried to obtain a safe, efficiency methods to strengthen the structures considering the best in economy. Such methods are the use of fiber reinforced polymer jacketing, steel jacketing, steel plating, concrete jacketing, steel bracing and shear walls. Therefore, a method should be decided which will be used to strengthen buildings to a required standard. In choosing the best method, one should consider its effects on the structure, the availability, the disturbance to the free use of the structure, its advantages and disadvantages.

In order to be able to choose a better option successfully, there should be a guide or tool where the strengthening options are collectively put according to their effects on the structure, advantages and disadvantages as well as the current use of the structure.

1.1 Previous Work Done

A great number of researches have been done to ascertain the various types of strengthening options and their effectiveness on the structures strengthened. Most of these works has been done separately. It is either the use of FRP in strengthening beams or columns, or the use of steel plates. Few researchers have tried to compare methods together while some have come about the combination of two or more methods named as integrated method, all to devising a means to have a best option for a particular strengthening work.

In comparing strengthening methods, Jinbo Li, et al (2009) in their studies, affirmed that steel jacket strengthening is not appropriate for a marine environment. The rate of corrosion of steel will be very high. Hence, it may require coating and this will increase the cost of the strengthening scheme. Abdullah and Katsuki (2003) in their experiment found that, application of grout between steel jackets and concrete structural member has no significant effect to increasing performance of that structural member if that member does not have a significant damages or cracks on them. Therefore, it will only add to the cost of strengthening. Also, Eunsoo and Kim (2005) in their experimental study on the use of rectangular steel jackets, deducted that building at the early sixties, with inadequate seismic resistance can be strengthened by steel jacketing, especially for columns that lack flexural strength. According to Riyad and Jirsa (1999) steel jacketing is proper for columns designed for gravity loads and with no knowledge of earthquake design codes. On the contrary, Adhikary et al (2006) through their studies affirmed that, steel plate strengthening performs better on beams than steel jackets strengthening, especially when there are externally anchored stirrups. Rodriguez and Park (1994) in their experiment for columns strengthened with concrete jacket and with no jacket as a control, showed a great improvement in ductility and high strength to other columns unstrengthened. Thus, they concluded that, this strengthening method was successful but labour intensive. Leng and Teng (2008) on comparing steel and FRP uses in strengthening aforesaid that FRP does not possess the ductility that steel have. Therefore, this may limit the ductility behavior of reinforced concrete member strengthened with FRP wrap. Amoury and Ghobarah (2005) in their studies fixed that FRP strengthening does not significantly alter the dynamic response of structural frames but changes the damage location and pattern in the frame. They concluded that, it does not affect the initial stiffness of structural members but improves the strength of the member. Parvin and Wang (2002) in their studies deducted that FRP strengthening could be used to delay the degradation of stiffness of reinforced concrete columns. Amoury and Ghobarah (2005) also stated that FRP strengthening can be used to account for missing transverse reinforcement at joints. Central Public Works Department (2007), in their work stated also that, missing transverse reinforcement can be solved in structural frames by strengthening with shear walls. Although, the use of shear wall strengthening may cause building use disturbance, Hasan et al (2011), in their studies found that, application of shear wall externally is effective in increasing the strength of building and also possess no building disturbance if applied during building use. It is also a good strengthening application for buildings which are symmetrical. However, Maheri (1997) from his experiments on the use of steel bracing concluded that steel bracing will be a good supplement to shear wall strengthening in concrete framed building in seismic areas. Badoux and Jirsa (1990) in their studies equally found that, steel bracing is good for frames with weak short columns, it provides minimal disturbance to users during construction time. They also deducted that steel bracing could face aesthesis problem when used to strengthen some building, its application is labour-intensive and expensive to maintain. Amoury and Ghobara (2005) in their studies found that steel bracing is very effective in increasing the frame stiffness. However, its disadvantage is the non-ductile failure pattern such as joint shear failure that may occur at some locations within the frame. To control this defect, they also proposed the use of FRP and steel bracing as a hybrid method. Their experiment carried out on hybrid method of FRP and steel bracing, has shown an effective improvement in joint shear failure; improved stiffness; reduces maximum inter-storey drift of frames compared when it is only FRP or steel bracing. Jinbo et al (2009) in their experiment on hybrid method for FRP and steel jacket fixed that the combination is effective for structural members that have corroded internal reinforcement. It gives better result than when strengthening with only FRP or steel jacket.

Although, to compare strengthening methods is very difficult, therefore, a guide should be made which should help engineers to decide the best option to choose after analysis and building assessment has been done however, satisfying the problem requirement by the building in need of strengthening.

Chintha Perera et al (2006) in their work has created map which they have used to decide options for a particular strengthening problem. In their work, structures which have same problems where suggested to have the same strengthening option for their upgrade. For example, if in the previous work need an addition of new floor, and the members like columns where strengthened with FRP jacket, it now means that whenever there is such a case in any building, this method will now be the same option to be implemented on the new strengthening project.

However, this guide may not be acceptable in other region or country or even for another building that has the same problem since the building type is not considered and the seismicity of the region is not accounted. Equally, the general performance of the building is not considered. Therefore, these may cause miss interpretations for a new user with a new project, even though the building will look alike, the building problem requirement must be accounted for, as the required strengthening option is utilized.

1.2 Objectives and Scopes

A number of strengthening option is available but one need to have a critical understanding of these methods before their applications. Therefore, a guide or tool which contains the strengthening methods will be constructed. After the consideration of their uses, advantages and disadvantages, an engineer will consider appropriately the best option in any geographical area with respect to economy and efficiency.

1.3 Reasons for the Objective

In order to avoid misconception in the use of strengthening methods, a selection tool is constructed. This will help any engineer or contractor to be able to choose effectively without doubt the options to use in his strengthening projects.

1.4 Work Done to Achieve the Objective

Each available method was investigated accordingly, the advantages and disadvantages where obtained. Some projects applications and strengthening method targets were also obtained. The methods were divided into three forms in order of their effect on building structures generally. They were also arranged according to buildings that are symmetrical and asymmetrical; building with irregularities and without irregularities. The irregularities are considered coupled with the seismicity of the environment of which the structure is located and hence, a strengthening method selection tool is formed. This however will be used to assess the building performance result obtained, after the performance check has been carried out on buildings that require strengthening with structural software such as STA4-CAD. And the result of the assessment will determine the strengthening option suitable for the building.

1.5 Result

A selection tool has been produced which is effective to decide a strengthening option, by considering the efficiency and economy. Its application through the case studies shows its effectiveness for deciding a best strengthening method.

1.6 Organisation

This study is comprises of eight chapters. Chapter 1 is the introduction which contains the aim and scope of the thesis and as well the research method and the organisation of the thesis chapters. Chapter 2, introduces more discussion through different strengthening methods. Chapter 3, presents an over view in the assessment of building before strengthening and reasons for strengthening. Chapter 4, introduces coded decision selection program. It also discusses the strategies of strengthening methods, sited some projects executed and the methods used and finally an idea to construction of the strengthening coded decision selection program. Chapter 5, provides the FRP Mathematica coded programs for finding the required number of fiber reinforced polymer and also the thickness of the FRP required. In Chapter 6, a real life case study problems and solution with computer model were presented. While in Chapter 7, the cost effectiveness of the strengthening methods and their economical values were discussed. Finally, Chapter 8 presents the conclusions and some recommendations for engineers and future researchers.

Chapter 2

STRENGTHENING METHODS

2.1 Introduction

Strengthening is the process of increasing the load caring capacity of a structure. It could be bridges, buildings or wooding/metal frames. Increasing the life span of a previously existing structure is also done by this means. Therefore, instead of demolishing a building because it is incapable of carrying new loads, it is best to evaluate so as to see if strengthening will be the best option and as well investigate the economy that is involved, by comparing if a new structure be constructed or the existing old structure be strengthened. The next question is to what extent an existing building will be strengthened. It will not be expected that after evaluating old existing building, it will have the same performance with a newly constructed building. If an old building has lived up to 50% of its life span, it should be strengthened to resist at least 70% as per the new design standards (CPWD, 2007). Expected design life span of most buildings depends on the quality of construction and its building type and usage. For a typical reinforced concrete building, the design life could be 50 to 60 years.

2.2 Reasons for Strengthening

Most structures need to be strengthened when their performance is not satisfactory. This has been a problem in civil construction industry. It could be as a result of change in design codes and hence a newer codes and limit to be implemented in order to satisfy the required capacity checks. In addition, there may be application mistakes during the construction of the structure. This could arise as a result of the skills implemented in the construction site. Problems may later arise, which could be a poor concrete grade, some missing transverse reinforcement, longer spans and lower sizes of some structural members.

Another reason for strengthening is the change of the use of the structure (increment of the live load). This will lead to an increase in the load that will be carried by the structure. For example, a building which was initially used as an office converted to a library, hence, there is a tendency of increased live loads on the building. Therefore, this will require the building to undergo strengthening by adding strength and ductility needed to carry the additional load.

Furthermore, when some parts of the structure have started wearing, there is a tendency that the building will lose its ability to carry load. This can be restored by firstly repairing and shortly followed by strengthening. Also, there may be a need to strengthen structures due to high vulnerability to seismic motions.

Therefore, the aim of strengthening is to reduce the effect of the internal forces as a result of flexure, shear, torsional and axial. This can be achieved when the magnitude of their forces are reduced to bare minimum or by establishing additional structural member in order to resist them. Thus, care should be taking when strengthening building in order to see that the best method for strengthening is used and for which the total cost including maintenance cost does not exceed the cost of raising a new building and safer as the economy is considered.

2.3 Categories of Strengthening

There are two main approach for strengthening; namely, active and passive strengthening (Peter Emmons et al 1998).

In active strengthening approach, the strengthening materials are added to the existing structural member to resist both live and superimposed loads, and present dead loads all together. While in passive strengthening approach, the strengthening materials added, resist only the future loads. The added materials become active as soon as the existing structural members strengthened have partly deformed. Figure 2.1 below shows structural member and added materials.

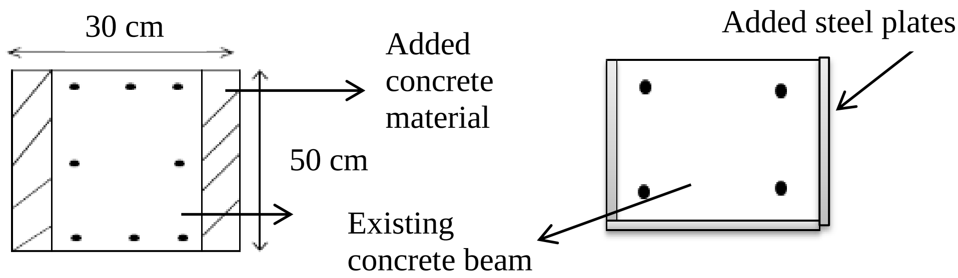


Figure 2.1: Beams with different cross section added materials

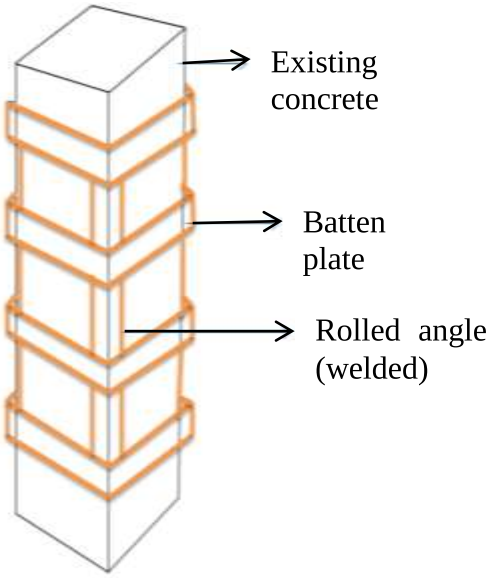
2.4 Methods of Strengthening

There are different methods of strengthening for which will be restricted to the general methods such as steel jacketing, steel plating, strengthening with FRP, concrete jacketing, steel bracing and application of shear wall. Therefore, there are challenges of which method to apply and which will enhance the serviceability and give maximum strength to a structure especially when addressing some limitations which include; building operations, budget and constructability.

2.4.1 Steel Jacketing

Steel jacketing is one of the methods of increasing the capacity of a structural member. This could be achieved by encasing metal plates around a structural member as in Figure 2.2 (a). It is an effective method for mostly columns that have deficiency in reinforcement (Priestly and Seible, 1991). It can be used for both circular and rectangular columns. But the use of this confinement on circular columns achieves the best result since the stresses are distributed evenly. In order to confine the rectangular column, the edges need to be smoothened by curving it to a circular form of radius ranging from 4 mm to 25 mm.

Also, an elliptical jacket can be used to confine rectangular columns where the gap between the jacket and the columns as in Figure 2.2 (b) is filled with grout.



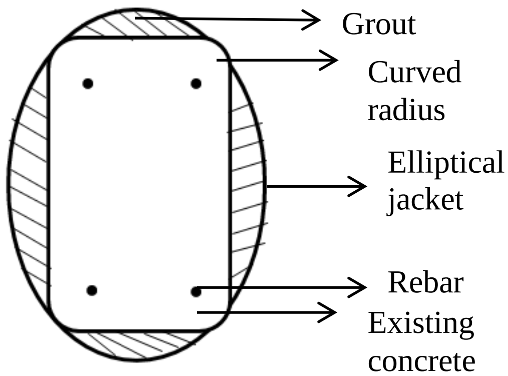


Figure 2.2 (a): Column with steel jacket   Figure 2.2 (b): Elliptical steel jacket

The steel jacketing of column can be achieved by welding together two semi-circular shells of steel of about 25.4 mm larger than the column radius. Recent test performed on this method has shown that it will improve the performance of the structural member either at hinge or splice region for columns. Therefore, to strengthen a column, the jacket thickness needs to provide confinement for flexure at plastic hinge region. It should be a function for the maximum ductility of that hinge.

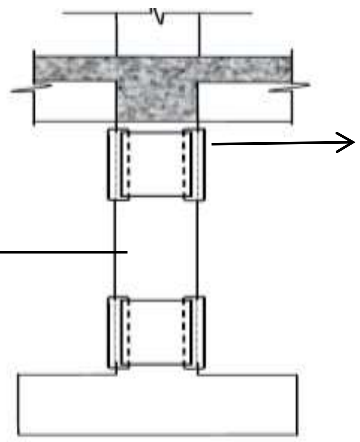


Figure 2.3: Steel jacketing at the hinge region of the column (CPWD, 2007)

A test carried out by Priestly et al (1996) has shown that confinement needed to provide slippage of bars in a lap splice is to ensure that radial dilation strain is less than 0.0015.

2.4.1.1 Advantages and Disadvantages of Steel Jacketing

It is a good strengthening approach for columns that are deficient of transvers reinforcement. Also columns which were designed with little or no consideration of earthquake code design can be strengthened by this method. However, it is not suitable for corroded reinforced concrete (Jinbo et al 2009). It is also damaged by marine environment and de-icing salts.

2.4.2 Addition of Steel Plate

This process increases the cross-sectional area and stiffness of existing members to be plated. Evidently, it is important to check the structure if it will be able to support nominal dead load and a proportion of live load before applying steel plating. This is to prevent collapse due to accidental loss of the bonded plate. Rehabcon (2004) suggested a minimum plate thickness of 4 mm to prevent shape distortion during site execution. Recent results have shown that thin plating is more effective than thick narrow plating. Thin plates fail mainly in flexure for beams. As the width of plating steel decreases, the longitudinal shear stresses increases. The normal stresses and shear stresses occur at the steel ends. The use of bonded anchor plates on the tensile plates is also effective as it produces yielding of the tensile plates and hence the full theoretical strength is achieved 36% than the un-plated beams and columns. When the plating is anchored, it increases the ductility of the beam structure. On the application of bolts, the ductility decreases as the thickness of the steel plate increases.

To provide anchorage to the steel as in Figure 2.4 (b), plated beams will require extra work and this will therefore add to the cost of the plate bonding method.

Its advantage is that, it controls flexural deformations and also crack widths in beams. Under service load for ultimate conditions, there is an increased load carrying capacity of the member.

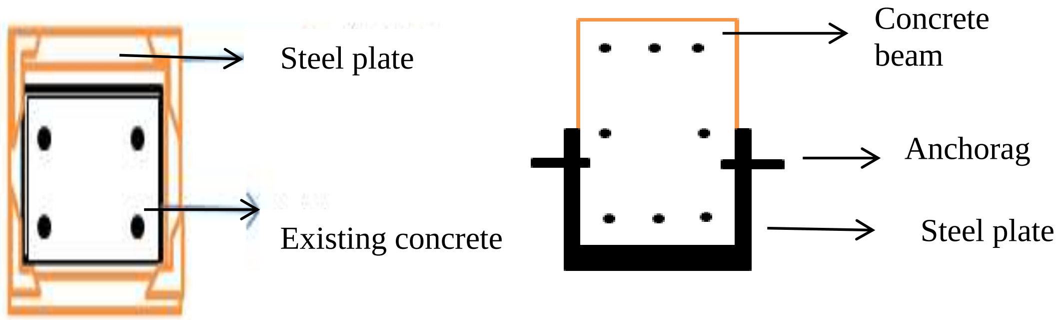


Figure 2.4 (a): Section of steel plated beam (b): Beam with steel plate at tension side

Gaps are also provided at the top of the column when steel plate is applied so that the plate offers passive strengthening to the column as in Figure 2.5 (a) and also not to offer axial enhancement. To strengthen flange beams in compression, the three side of the beam can be plated as seen in Figure 2.5 (d). Figure 2.5 (c) is the side view of three side plated beams.

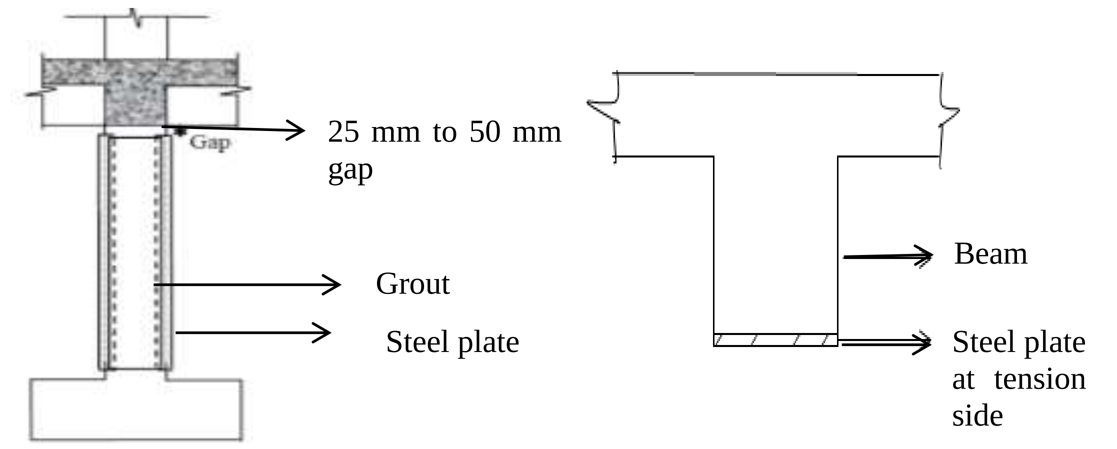


Figure 2.5 (a): Column steel plating (CPWD 2007) (b): Steel plate at tension side of beam

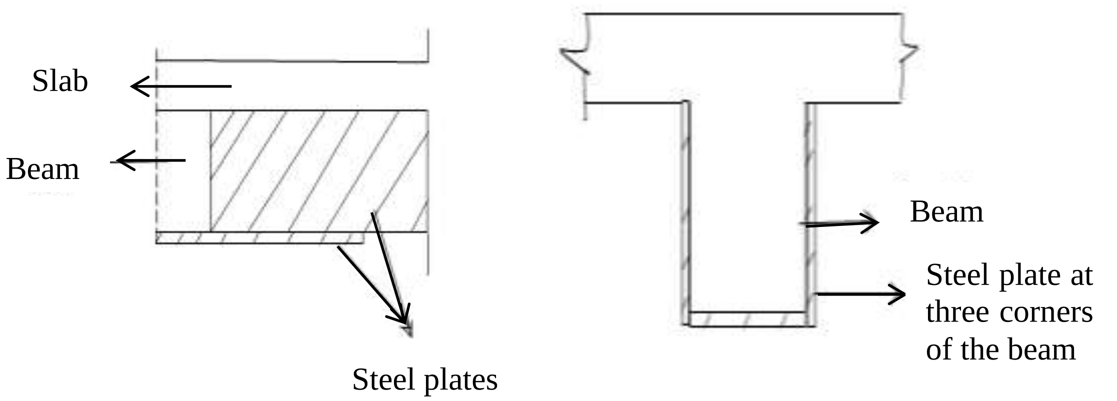


Figure 2.5: (c) Side view of steel plated beam (d) Cross section of three side plated flange beam.

2.4.2.1 Advantages and Disadvantages

The exposure of steel plate to atmospheric weather will cause corrosion of the bonded plate and this will eventually loose strength. Steel plates are difficult to shape so as to fit irregular and complex shapes. The weight of the plates makes it difficult to transport and handle on site. It also reduces space in a place of limited access. It will also add to the dead weight of the structure after the installation.

During and after construction, struts or support are needed. This may cause an infringement into the access of the area when the structure is in use during the time of construction. The struts are expensive as more of it will be needed which will affect its cost of application. The length of steels cannot be transported with original length from manufacturer to the site work; therefore, it will be delivered in smaller length; hence, joints will be necessary. As a result, butt joints will be used to join them together and this needs a different design. If plates are loaded in compression, buckling may occur, causing the plates to experience or become detached. However, it improves or is more efficient in shear strengthening than to steel jacket since the latter is made intermittently on structural members.

2.4.3 Concrete Section Enlargement

This involves placing additional reinforced concrete to an existing structural member. A member can be increased by casting an overlay or by jacketing form. This method is very efficient to increasing the shear capacity of columns and beams. It can be achieved when new stirrups are applied and are covered with new concrete layers. Also, the flexural strength enhancement in beam is obtained by jacketing the beam.

The main disadvantage of this method is that, there is an increase in the concrete member size after the application of concrete reinforced with steels where there is a need to construct a new formwork.

For slabs, the load carrying capacity and stiffness is increased. The typical enlargement is approximately 5-8 cm for slabs. The most commonly used technique in section enlargement is the use of concrete jacketing. Also, column jacketing can be used to improve a weak column and strong beam. This method unlike other methods such as steel jacketing does not have a specialized work demand. Therefore, its simplicity makes it possible for most construction companies to adapt to it. Because the increased stiffness of the structure is uniformly distributed, it is preferable compared to steel bracing and shear walls. The later methods will involve the extra strengthening of the foundation or execution of new foundations. In order to strengthen beam members by this means, the following steps should be considered; Remove concrete cover, roughen the beam surface, and clean the steel reinforcement bars and coat with an appropriate material that would hinder corrosion. Also, holes should be created at 15-25 cm on the whole span and width of the beam as in Figure 2.6.

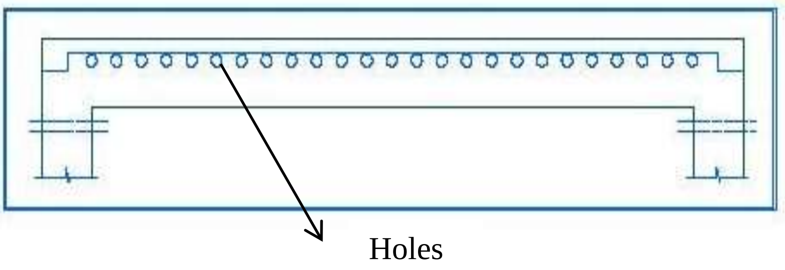


Figure 2.6 Beam section enlargement (The Constructor, Civil Engineering Home, 2013)

The holes should be filled with a low viscosity cement mortar. Steel connectors should be installed for fastening the new stirrups. The steel connectors should be installed into the columns so as to fasten the steel bars added to the beam. Close the added stirrups with steel wires and install new steel into these stirrups. The concrete surface should be coated with appropriate epoxy material which will guarantee the bond between the old and new concrete before pouring the concrete. Finally, a low shrinkage concrete should be casted onto the surface as detailed in Figure 2.7 step 6 with six steps.

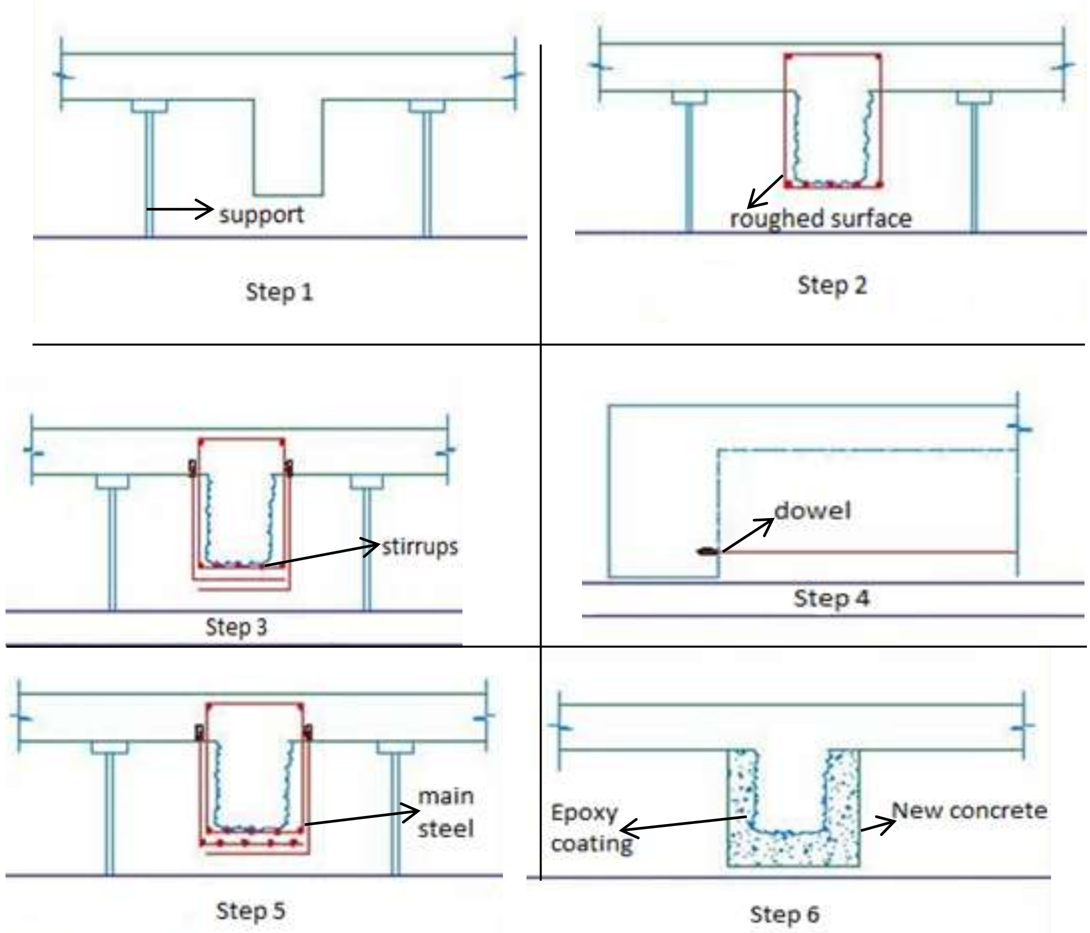


Figure 2.7: Steps in section enlargement of beam (The Constructor, Civil Engineering Home, 2013)

For column jacketing; the procedure can be assessed according to the different aspects namely; anchoring, slab crossing of the added longitudinal reinforcement, preparation of surface interface, spacing of added stirrups, temporal shoring of the structure and addition of new concrete.

2.4.3.1 Anchoring

The steel rebars in the concrete jacket can be anchored to the original footings as seen in Figure 2.8 (a). Also, it can be bonded to the RC footings or slabs for flange beams as shown in Figure 2.8 (b) by epoxy resins. The holes drilled on the footings should be appropriately cleaned before adding the reinforcement to avoid slippage (Julio, 2001). This can be done with a vacuum cleaner.

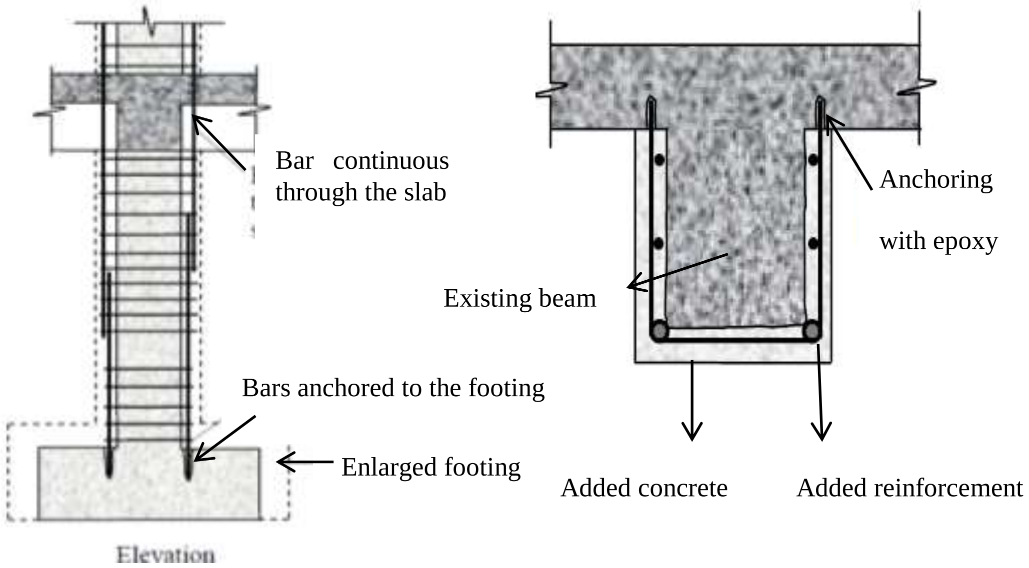


Figure 2.8: (a) Concrete jacketing of column (b) Concrete jacketing of beam (CPWD 2007)

2.4.3.2 Crossing the Slab

If there will be a continuity of the RC jacketing and the floors of the building, holes must be created for reinforcement passage. It must be provided at the corners, so as not to interrupt the middle bars as in Figure 2.8 (a). In this way, increased column shear strength and ductility is obtained. The experiment carried out by Hayashi et al (1980) on how mortar reinforced with welded wire fabric increases the shear strength and ductility of an existing RC column for different test specimen was conducted. But the most important part is that, the samples that were not strengthened deteriorated at an early stage. The strength samples showed decrease in load capacity and the tensile reinforcement yielded before maximum load. This was agreed that the technique increases shear strength and ductility, and protects the column from brittle shear failures.

2.4.3.3 Interface Surface Treatment

This can be done by hand chipping, sand-blasting, jack-hammering, electric hammering, water demolition, iron brushing etc. This is however increasing the roughness of the interface surface. Of all the methods, sand-blasting is the best roughening technique to be used (Julio, 2001) while the use of pneumatic hammering causes micro-cracking of the substrate (Hindo, 1990). In fact, it can cause weakness on the joints. Addition of steel connectors increases the longitudinal shear strength considering slipping. Interface roughness or to use any kind of bonding agent may not be necessary for an undamaged column (Julio, 2001). There is also a high risk of debonding of the reinforced concrete jacket for columns that are short.

2.4.3.4 Spacing of Added Stirrups

The uniform performance of jacketed RC columns can be obtained if a higher percentage of transverse reinforcement is considered by the jacket concrete (Gomes and Appleton, 1994; 1992). Therefore, it was recommended that half the spacing of the initial column transverse reinforcement be used for the transverse internal reinforcement of the concrete jacket as in Figure (2.9).

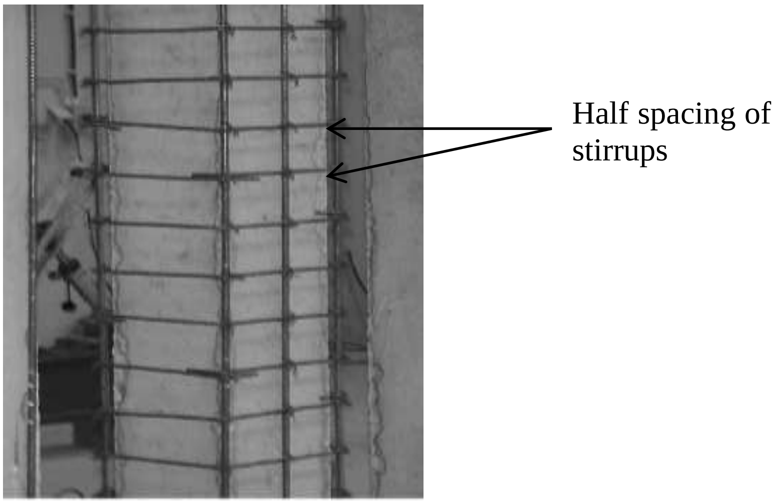


Figure 2.9: Longitudinal section of column, Julio et al (2003)

2.4.3.5 Temporary Shoring of the Structure

During concrete jacket application, if the jacket and the original column with the composites are to resist the load together, the load should be unloaded from the column by using hydraulic jacks. This is referred to as temporary shoring of the structure. The aim is to transfer the load installed on the column to this shoring structure.

2.4.3.6 Added Concrete

The aggregates dimension for the added concrete is almost 2 mm as a result of lack of space in the jacket, hence a self-compacting concrete is frequently used. Because the thickness of the jacket is small of about 100 mm, a high-strength concrete is usually used. The added concrete (Figure 2.10) is treated with silica fume which makes it to be high-durability concrete and the performance is very high. If the original body is too old, it is advisable to use a non-shrinkage concrete for the added concrete. Previous tests performed by Julio et al (2003) on columns strengthened by jacketing, showed that, the performance of the column was improved. No jacket debonding occurred in any of the models because the surface application was treated properly with self-compacting concrete/ high performance concrete.

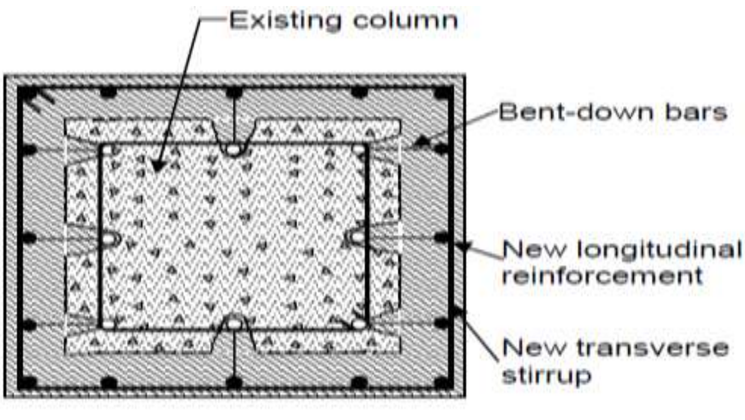


Figure 2.10: Cross section of concrete jacketing (Durgesh, 2004)

On the other hand, column foundations can be strengthened when there is an additional load to carry. However, this will be done by widening and constructing a concrete jacket on the existing footings as in Figure 2.11. To carry out this technique, an isolated foundation can be strengthened by firstly excavating around the footings, cleaning and roughening the footing surfaces. The next step is to install dowels at 25-30 cm spacing. The steel bars should be fastened with steel wires. The footing surfaces will be coated with bonding agent to increase the bonding between the old concrete and the new concrete. A non-shrinking concrete material will be poured to form a complete jacketing.

(a) Excavating around the footing

(b) Hardening the surface and installing the dowels

(c) Installing the main steel

(d) Completing the jacket

Figure 2.11: Foundation strengthening procedure. (The Constructor, Civil Engineering Home, 2013)

2.4.3.7 Significance of Concrete Section Enlargement

With concrete jacketing, increase in strength and ductility is achieved. The studies by Alcocer and Jirsa, (1993) on reinforced concrete structures strengthened by this method have shown an improvement in the strong beam-weak column to a strong column-weak beam concept.

In summary; the durability of the structural member strengthened by this method is improved and to compare with the corrosion and fire protection needs of other methods such as steel being exposed and or where epoxy resins are used. There is a needless for improving the roughness of the interface surface for jacketing except for the situation of short reinforced concrete columns. Here, sand blasting could be used in this process. Steel connectors can be applied especially when there is a short RC column to improve the strength and stiffness level under cyclic loading. During application of temporal shoring, consideration should be made in a way that the added reinforced concrete jacket will resist part of the total load instead of part of load increments. The minimum thickness of the added jacket should be 100 mm. When the size is enlarged, free available usable space becomes less and hence, huge dead mass is added.

This method requires adequate dowelling to the existing column. The longitudinal bars should be anchored to the foundation and should also pass through the slab. It also requires drilling of holes in the existing slabs, columns footings and beams. When adding the stirrups, the half of spacings of the original transverse reinforcement should be considered so as to obtain a monolithic behavior, especially under cyclic loading. The added concrete should be a non-shrinkage and with a tendering of self-compacting, high-strength and high-durability concrete. There is an increase in size, weight and stiffness of the strengthened member. Placement of ties at beam column joint is not practically feasible. The rate of reinforced concrete jacket implementation is slow.

2.4.3.8 Problems of Section Enlargement

The newly cast concrete will shrink as it cures while the original section will stay dimensionally the same. As the two sections are bonded and mechanically interconnected, the newly casted concrete will be prevented from shrinking and therefore tensional stresses will be developed. When the stresses are much, the newly casted concrete section may end up cracking or debonding from the existing concrete. There may be corrosion of new reinforcing bars and dowels placed in close contact with existing concrete that may be undergoing corrosive processes.

2.4.4 Shortening the Span

When simple-span beam is overstressed due to bending, such beam can be strengthened by shortening its design span. This can be done by erecting additional column not very close to the previous existing columns. In order to carry out this operation, columns will require footings. This means that the floor slabs will be removed in order to execute such operation. This is considerably expensive.

However, it can also be shortened by applying steel beams or diagonal braces extending from the bases of the existing columns to some points at the bottom of the beam (Figure 2.12). Its advantage is that it does not require additional footings.

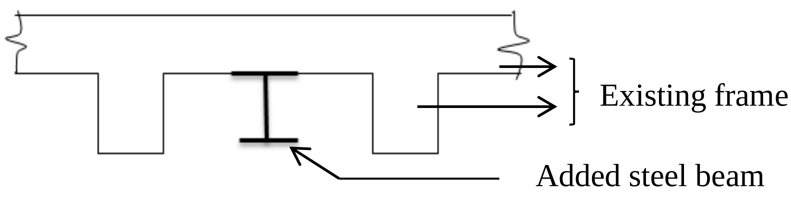


Figure 2.12: Slab span shortening

2.4.5 Shear Wall

Shear walls are concrete walls designed to resist shear and lateral forces that cause damage during earthquake. Nowadays, building codes mandate the use of shear wall to make structural buildings safer and more stable in the earthquake regions. Use of shear wall strengthening is also aesthetically pleasing on reinforce concrete building. It is normally constructed as heavily braced panels. They can be seen as brace line walls in some regions in structural plan. The new walls are connected to the adjoining frame by drilled-in dowels in a relatively straight forward fashion. Its foundations are also doweled into the existing column footings as shown in Figure 2.13. The wall shrinkage can be accommodated if the wall can be stop short some distance of about 2 in (50.8 mm) from the existing concrete at the top of shear wall, the space will now be filled with non-shrink grout.

Adding shear wall does not cause any major changes in the interior layout of the structure. Dangerous soft-storey condition at the lower levels of the building is also corrected. This was paramount in the execution of shear wall on the historical hotel at Utah in Salt Lake City, constructed in the early 1900. In order to achieve this, the wall was made relatively thin. The shear capacity was improved by using 28-day compressive strength of 5000 psi (34474 Kpa). Due to congested reinforcement, concrete mix with 3/8 in (9.5 mm) coarse aggregate was specified.

One disadvantage is the complication involved in the shear-wall foundation. The column footings will hinder the execution of wall foundation since it is normally placed between columns. Therefore, the existing columns will be shored, the footings removed and replaced with footings of a configuration to suit the space for shear-wall footings.

The foundations are also to be strengthened before adding shear wall to the building and when this building is incapable of resisting the overturning moment induced by the shear wall. The shear wall reinforcement foundations and dimensions can be obtained using code specified structural analysis utilizing area seismic hazard, soil condition, geometry, structure type and as built structural properties.

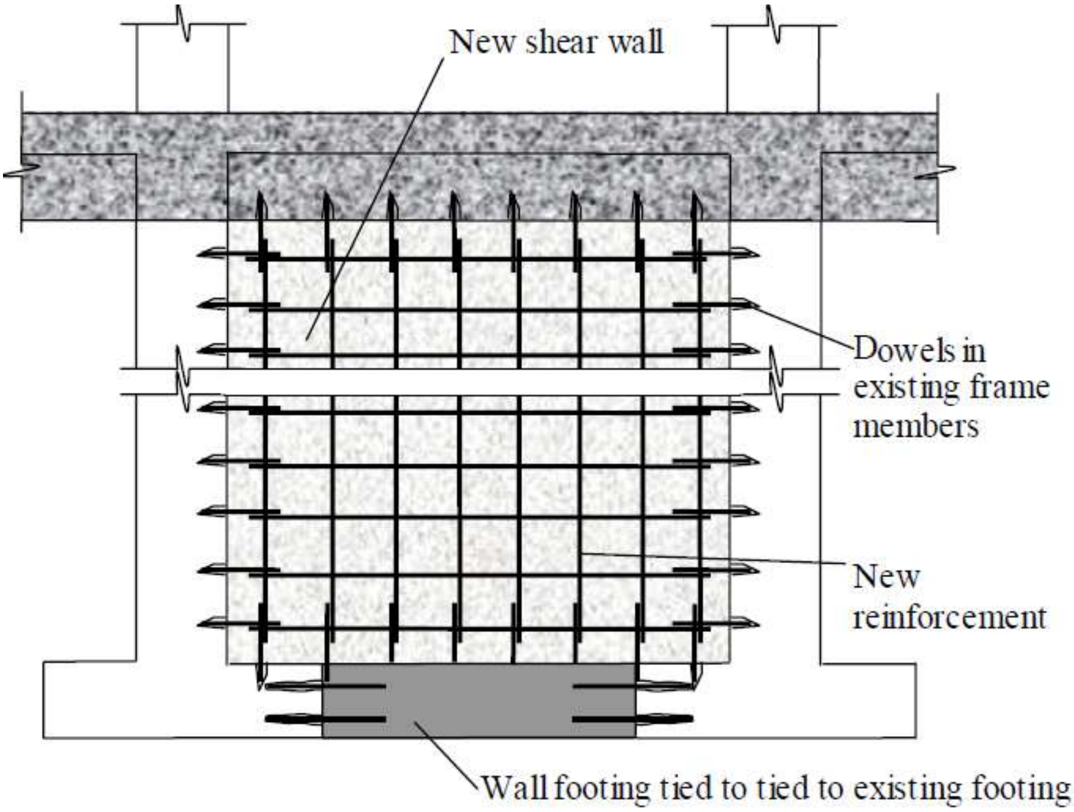


Figure 2.13: Shear wall (Durgesh C. Rai, 2004**)**

The arrangement of shear wall is placed in the concrete frame in such a way that the distance between the center of mass and center of rigidity will not be far from each other (Figure 2.14). When the shear wall is placed wrongly, it will cause a turning effect on the building and it will no longer be safe for such building. The heavy lines represent the shear walls placed in the following frames (Figure 12.14)

Figure 2.14: (a) Not recommended placement (b) Recommended shear placement

2.4.6 Steel Bracing

Buildings which were designed without the consideration of seismic codes in the seismic regions can be upgraded with this technique. It is good in controlling lateral loads and resisting earthquake loads in multistoried structural buildings. Most frames which were under-reinforced or inadequate in reinforcement in seismic zone can be readily corrected by steel bracing technique. It is economical, easy to erect, occupies less space and has flexibility to design for meeting the required strength and stiffness.

The method has proved to be a better option catering to economic considerations and immediate shelter problems instead of replacement of the entire buildings. It is equally efficient since the diagonals work in axial stress and therefore call for minimum member sizes in providing stiffness and strength against horizontal shear. Through the addition of the bracing system, load could be transferred out of the frame and into the braces, bypassing columns which could be weak to carry load while increasing strength. Ferraioli et al, (2006) in their experiment suggested the use of steel bracing for reinforced concrete frames with inadequate lateral resistance.

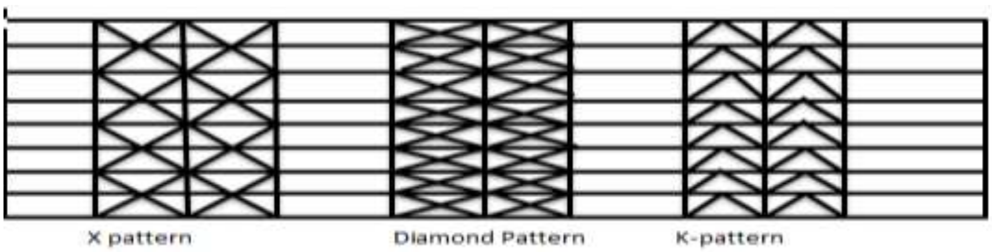


Figure 2.15: Various pattern of bracing

2.4.6.1 Types of Bracing

There are two configuration of bracing methods, concentric bracing system and eccentric bracing system. Concentric bracing system increases the lateral stiffness of frames, decrease the natural frequency and lateral drifts (Figure 2.15, X-pattern). It is pertinent to know also that increase in stiffness may attract a large inertia force due to earthquake. Bending moments and shear forces are reduced while there is an increase in axial compression in the columns in which it is connected. But since columns are very strong in compression, it will not pose any problem.

Eccentric bracings reduce the lateral stiffness of the system and will also improve the energy dissipation capacity (Figure 2.15, K-pattern). Since this connection affects beams eccentrically, the lateral stiffness of the system now depends upon the flexural stiffness of the beams and columns, and hence reduction of the lateral stiffness of the frame.

The test carried out on different types of bracing has shown that X-pattern bracing gave the best result to reduced lateral displacement. Bracing can also be applied by indirect or direct method. In indirect method, steel is usually encased in and attached to the surface of the column or beam before the bracing is applied as detailed in Figure 2.16 (a). This is usually needed on the building to increase the in-plane shear resistance of the frame. When beams or columns need improvement by increasing shear resistance, this technique can be applied for the shear enhancement. However, this procedure is not economical and very costly.

Another disadvantage of this method is that there will be a diverse effect in the dynamic interaction during earthquake between the concrete and steel frames. Direct method involves direct connection of the steel bracing on the concrete frame. This method is more economical and easy to apply. In this approach, the dynamic interaction of bracing system with concrete frame is minimal.

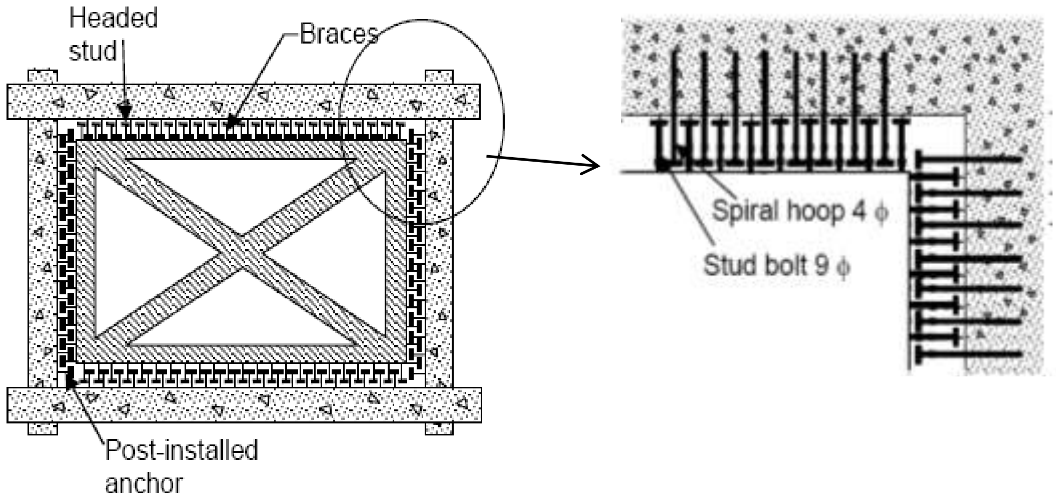


Figure 2.16 (a) Prefabricated bracing (b) Detailing of the corner view (Durgesh**,** 2004**)**

2.4.7 Application of Fiber Reinforced Polymer (FRP)

In the past decades, FRP application has proven to be good in upgrading and strengthening of existing structures Amoury and Ghobara (2007). There has been many research and publication on the use of FRP for strengthening. These have resulted after the first test on FRP bonding on a beam strengthening which was carried out in Switzerland in 1984. Its high strength to its weight ratio makes it a better choice compare to steel plating and other strengthening methods. However, its behaviour when on long term usage and its high cost could lead to scarcity in some regions. There are guidelines for strengthening using FRP by ACI 400.2R (2002), British Concrete Society, TR55 (2000). Its role in strengthening structures is growing at a faster rate as its construction speed, application on structures without the disturbance of the current functionality of the structure undergoing strengthening. It is a composite material containing generally high strength carbon glass fibers or aramid in a polymeric matrix. One more thing to consider is the epoxy material used to bind the FRP to the concrete surface. Hamid and Mohammad, (1991) suggested that rubber-toughened epoxies are good candidates for FRP application to concrete surface.

2.4.7.1 Characteristics of FRP

The fiber reinforced polymer (FRP) is mostly of glass, carbon and aramid fiber. Glass fiber reinforced polymer has low modulus of elasticity and its stiffness is lower than that of steel plating. It has poor resistance to alkalis and can be controlled with the application of resins. It also suffers creep rupture under sustained load at a lower levels at which the material supports instantaneously. Therefore, it is best to resist applied load such as earthquake force. Aramids fibers have low compressive strength. Therefore, its application is limited to tension members. It is as costly as Carbon Fibers Polymer. FRP generally provides added strength for about 50% for a reasonable upper limit. Different FRP with different tensile strengths are detailed in Table (2.1).

Table: 2.1 Characteristics of different types of FRP (Gupta, 2004)

Serial No.	**FRP (Types)**	**Age of fiber %**	**Density in kg/m^3**	**Tensile strength kg/cm^2**
1	GFRP	50 to 80	1600 - 2000	4000 - 18000
2	CFRP	65 to 75	1600 - 1900	12000 - 22500
3	AFRP	60 to 70	1050 - 1250	10000 - 18000

Therefore, it is important for an engineer to know what type of FRP to use to avoid a sudden failure of any member strengthened with FRP fiber. FRP is not as ductile as the other types such as steel plates. One important aspect of this method is its speed and ease at which it is implemented.

FRP has many uses and application in strengthening industry. It can either be applied at the tension part of beams to provide flexure, or on the other side of the beam applied for shear strength. It can as well be used to wrap columns in seismic region to improve ductility as a result of the confinement of the concrete. The selection of this method should be based on stiffness, durability or strength needed in a particular application. Selection of the resins should be made depending on the environment for which the FRP will be exposed and choice of method of FRP application. It is normally applied or attached on the surface of concrete member in such a way that, it will be transverse to the plane of the member and will resist transverse shear force resultant and lateral forces in column or wall.

2.4.7.2 Method of FRP Applications

When FRP is considered as a means of strengthening, it is desirous to decide the method of FRP application on the concrete member. This will help to reduce or solve problems with respect to debonding. A member may be in need of shear strengthening or flexural strengthening, axial strengthening or the combination of any two. In solving debonding issues, it is good to wrap the member by U wrap (Figure 2.17 a)

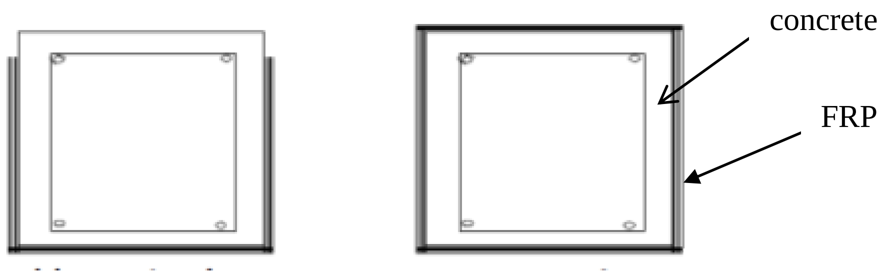


Figure 2.17: (a) U strip and (b) total wrap of FRP on beams.

Choosing to do so, the corners of the beam and columns which are rectangular should be rounded to prevent sharp corner edges that causes stress concentrations and a premature failure of the FRP wrap (Figure 2.18).

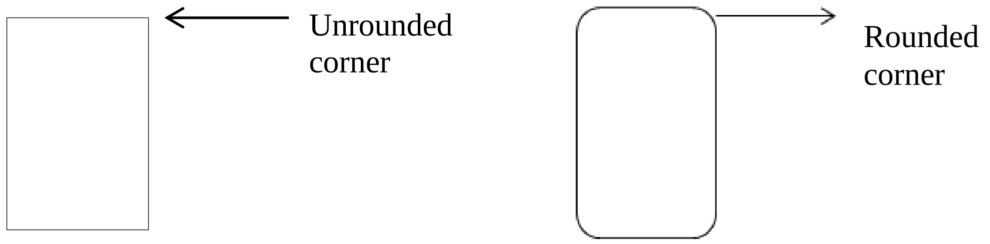


Figure 2.18: Rounded corner of a column

2.4.7.3 Shear Strengthening with FRP

Here, the FRP is applied in such a way as to reduce the tension in the concrete caused by shear forces. Increasing the shear strength of the concrete member will help to avoid brittle failure of the member. This can be achieved by providing strips of FRP placed on the member surface and is properly glued. The widths and spacing can be determined by design calculations. For shear strengthening, FRP can be applied as U shape, sides or complete wrap depending on the desired strength. Overlapping is necessary at the compression side of the beam.

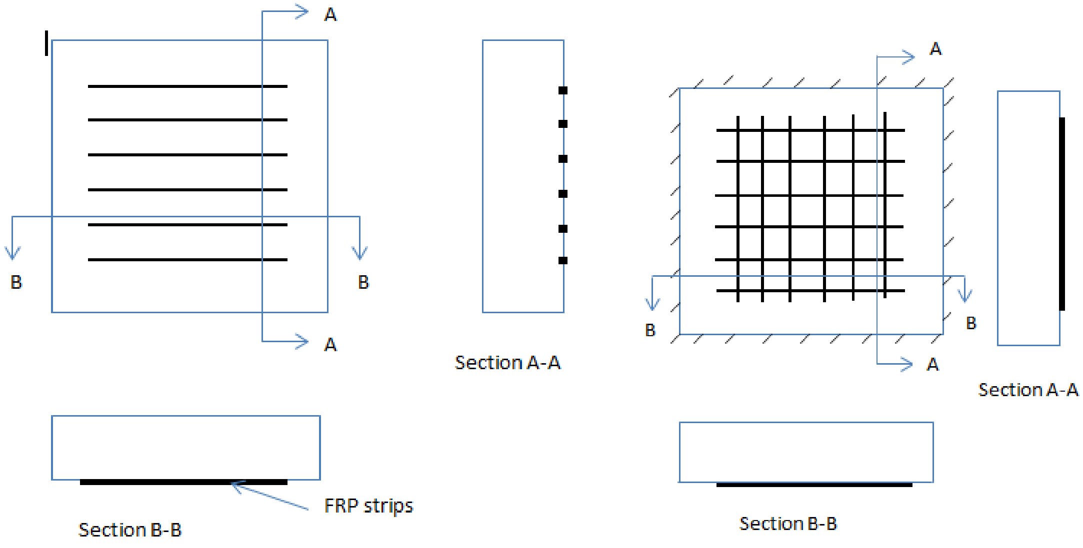


Figure 2.19 (a): FRP slab strengthening (1- way) (b) FRP slab strengthening (2-ways)

2.4.7.4 Flexural Strengthening with FRP

This involves the application of FRP on the tension sides of beams or slabs (Figure 2.19). It serves as additional external reinforcement. On the application of the FRP on the tensional side, the neutral axis is shifted and at this point, the structure is strengthened to its desired extent.

Mechanical anchorage can be used to hold the applied FRP to avoid debonding. Application of flexural strengthening increases the strength of the member but lesser in ductility.

2.4.7.5 Axial Strengthening Using FRP

This is obtained when the arrangement of FRP is in such a way that, its main fiber direction is circumferential to the column and also perpendicular to the structural member's longitudinal axis. The increment in capacity could be on axial or flexural or both. In this case, the FRP is wrapped all around the column in other to control such failures as in Figure 2.20(a).

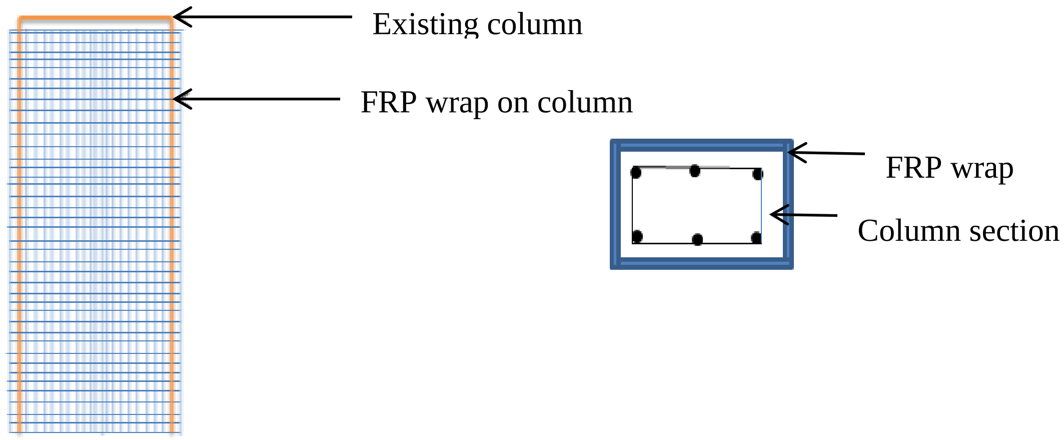


Figure 2.20 (a) Column FRP wrap (b) Cross sectioning of Figure (a)

The axial load capacity of column is presented by ACI 440.2R-02 and is compatible with ACI 318-99. Axial forces on column can influence the shear capacity and the lateral displacement capacity. This type of strengthening also provides shear strengthening to the member so long as it is oriented perpendicular to the member axis. Saadatmanesh et al (1994) suggested that confining effect reduces when the wrapping is made intermittently on the member. Although it is generally preferred, but however, it is difficult to place in the field. It gives room for moisture to migrate from concrete to air. A continues wrap also has the effect of the degradation of interface. It may lead to difficulties with inspection since the surface of the concrete is not visible. FRP confining increases the contribution of the concrete in the internal force equilibrium rather than increasing the contribution of steel reinforcing as in case of FRP flexural or shear strength. It is advisable to make the column a circular type since the pressures provided by the jacketing are uniform around the circumference of the column.

2.4.7.6 Procedure of FRP Application

The surface of the member is brushed to remove dust and loosed cements. If the member needs repair such as cracks on the concrete, it will be filled with cement or mortar. The corners of the column if it is a rectangular type will be rounded off at the edges with a specified rounding radius of about 4 mm. The next step is the application of saturant. Here, the fiber wrap is wetted. Therefore the fiber is then wrapped on the beam or column carefully making sure that, no air is trapped underneath or within. After this process, it is again wetted with one more layer of saturated FRP ensuring that the fiber is fully saturated.

2.4.7.7 Advantages of using FRP Strengthening

It has a good corrosion resistance. It is also very effective for column confinement. During the application of FRP, minimal disturbance is experienced on the normal functioning of the structure. Construction joints are prevented since it can be applied on the site in rolls. No drilling is needed when strengthening with FRP since the holes may create a point of starting failure in the future. Institutions such as NCHRP, CPWD etc. recommended the use of FRP composites for concrete structural strengthening.

Designing FRP is mainly based on these assumptions; the FRP jacket has an elastic stress-strain relationship to failure point. The shear deformation within adhesive layer is neglected. There is no slip at the interface of FRP and concrete. The concrete tensile strength is negligible.

2.4.7.8 Disadvantages of Using FRP Strengthening

It is less ductile, therefore, members strengthened with this method is not known or accountable for ductility. Due to its irregular plastic behavior, it is likely that its behavior may not be uniform. It is attacked by ultra violet rays and this reduces the strength of the FRP. It is very costly. It is not cost effective for small size projects. The cost of FRP in strengthening depends largely on the cause and requirement for a building and its structural members. These defects can be found by evaluation and assessment of structures before carrying out strengthening scheme.

Chapter 3

BUILDING ASSESSMENT

3.1 Introduction

In order to carry out structural strengthening, it is important that the building is first assessed to determine its possible defects and other structural failure problems. This is paramount when there is an addition of live loads and dead loads on the structure. Example is due to the addition of floors to medium or high rise buildings. Some times after the assessment, the building may have no problem, but due to the fact that there were no allowance for accidental overload or improper alteration, could make the building not capable and hence desires strengthening.

Depending on the type of structure, the building assessment can be done considering the condition of the building; a preliminary investigation which involves the assessment of available original drawings and information about the geotechnical condition of the building surrounding and also a detailed investigation which will involve carrying out of different test on the building. To standardize the assessment process, there is a publication by ASCE 11-90 (Guideline for Structural Condition Assessment of Existing Buildings). This standard has set up a multilevel approach to carrying out building assessments. However, if there is enough information to assess the safety of the building at this stage, there will be little or no need for detailed investigation which however is costly and time consuming, Anand and Ankash, (2007).

3.2 Reasons for Assessment

- The construction may be of poor quality.
- There has been an addition or modifications in the use of the building and this has given rise to instability of the building.
- The building was initially designed for gravity load without consideration of lateral loads such as earthquake and wind.
- In the seismic region, the building could be previously designed but earthquake resistant design was not considered.
- The physical condition of the building has deteriorated with time.

At initial stage, there is a preliminary assessment which involves the investigation of the existing construction documents, site inspection, initial analysis on the structure and arrival at first initial conclusions and recommendations. Upon the assessment result, a second level which usually will involve a detailed investigation may or not be needed.

3.3 Building Assessment Procedure

The assessment will commence as long as there is a construction document. The document should indicate the material properties; design loads, construction details and member layouts. Knowing this information, the load capacity of the frame can theoretically or analytically be determined as well as the safety against the provisions in building codes. Test for concrete strength may be required at this time. When all these information are not found, the detailed investigation should be carried out.

3.3.1 Field Investigation

This procedure follows after the review of the existing document. The drawing will contain the design loading and the member sizes and material properties information. At this period, the visual inspection on the structure could be carried out to ascertain if all the parameters in the drawings are properly constructed in the same way within the site. This may include the member sizes corresponding to the drawing, whether there are some major modifications or not and sign of overload and members deteriorations. The building can be assessed visually by observing the interior structural outfit to external. It is good to start from the top floor to the lower floor and check all rooms in a clockwise manner (Eric C. Freund and Gary L.Olsen, 1985). At this stage, if the drawing document is missing, the spans, spacing and structural members can be measured. Also, there should be check for any visible structural damages such as spalling concrete or cracking. The general quality of the construction will also be noted as well as the condition of the soil and the foundation. Observed deviations will be noted in the present conditions. However, if there were differences with what is contained in the construction document to the existing site, or unavailability of this document, a more detailed investigation should be carried out.

3.5.2 Detailed Investigation

This is implemented when the required information that is to be carried out for preliminary investigation is not available. Therefore, a detailed investigation will be carried out for the entire structural system. Within this investigation, structural members are to be measured including the dimensions of the structural elements. Also, the properties of structural materials such as the steel reinforcements, concrete and bricks in the physical structural members will be ascertained by conducting a non-destructive testing in the site. The samples collected in the site will be tested in the laboratory. Information about the soil profiles will be obtained by geotechnical investigations. However, all these information will be used to evaluate the condition of the building and put up suitable strengthening measures.

3.6 Assessment of Reinforced Concrete in-situ Quality Test

As soon as the weak zones are identified in a structural system, the in-situ test for the quality of the material should be carried out. However, there are different types of test which were developed and standardized according to the different properties of concrete. The type of test to be carried out will depend on the aim of the test to be done. Such aims are; determination of concrete strength, concrete quality and durability and corrosion of embedded steel. Among these parameters, concrete strength is the most important for evaluating safety of structural system against loading. Low concrete strength can also result when there is a poor construction and or supervision. The widely used test to determine strength of concrete are;

3.6.1 Non-destructive Test (NDT)

This procedure is based on indirect measurement for concrete strength by measuring the dynamic elastic modulus and surface hardness. There are periodically adapted NDT methods for evaluation of concrete strength principles.

3.6.1.1 Rebound Hammer

This is used to measure surface hardness of the concrete. It is allowed to strike on the surface of concrete and the rebound distance given as R-values is determined from calibration curves (Figure 3.1). This determines weak concrete cover; while low rebounds indicate weak concrete where the weak concrete could be as a result of corrosion of internal reinforcements.

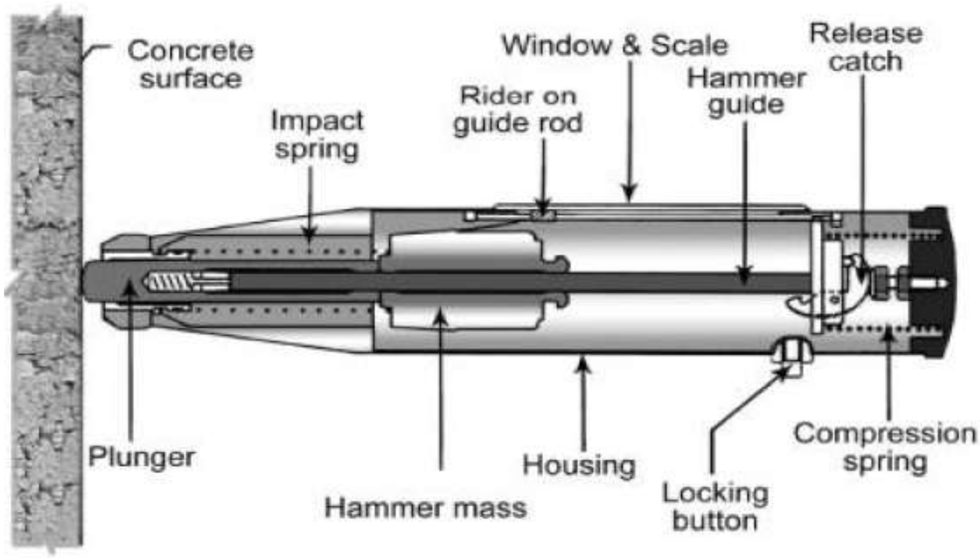


Figure 3.1: Rebound Hammer (Anand and Ankush 2007)

3.6.1.2 Ultrasonic Pulse Velocity

This technique uses the ultrasonic pulse waves induced on materials and the time of propagation or rebound is measured (Figure 3.2). The rebound is determined by the quality of concrete. It is also affected by modulus of elasticity and concrete strength. With this technique, honey combing and compaction in concrete can be detected.

Figure 3.2: UPV Testing equipment (Proceq SA, 2013)

3.6.1.3 Penetration Resistance Test

This is a partially destructive test. Here, bolt is forced into a concrete with a standard explosive cartridge as shown in Figure 3.3. The testing procedure equally has been standardized by ASTM C803. The amount of penetration depth, details of failure of concrete can be measured to determine the strength of concrete.

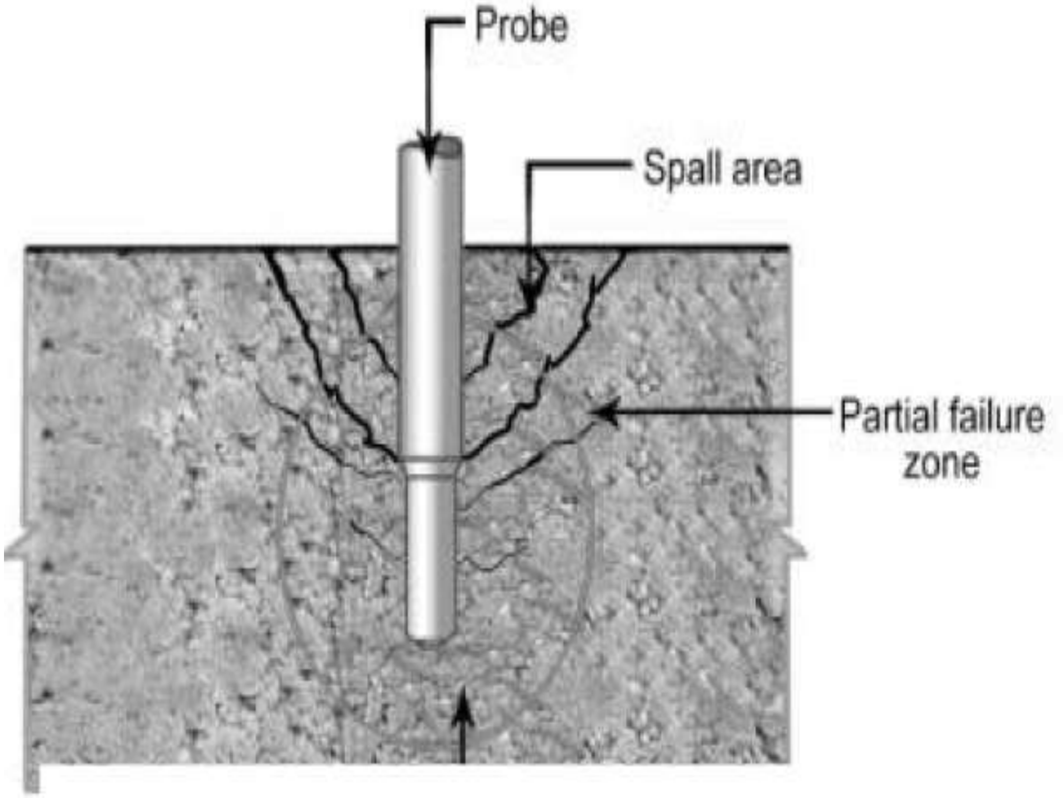


Figure 3.3 Penetration resistant test (ACI 228.1R-03)

3.6.1.4 Pull-out Test

In this technique, the force needed to pull out bolt or closely related device embedded in a concrete can be measured and correlated with the strength of the concrete. The bolts may be inserted during the time of casting or with epoxy glue as shown in Figure 3.4(a). There are other versions of this method such as CAPO test (Figure 3.4b).

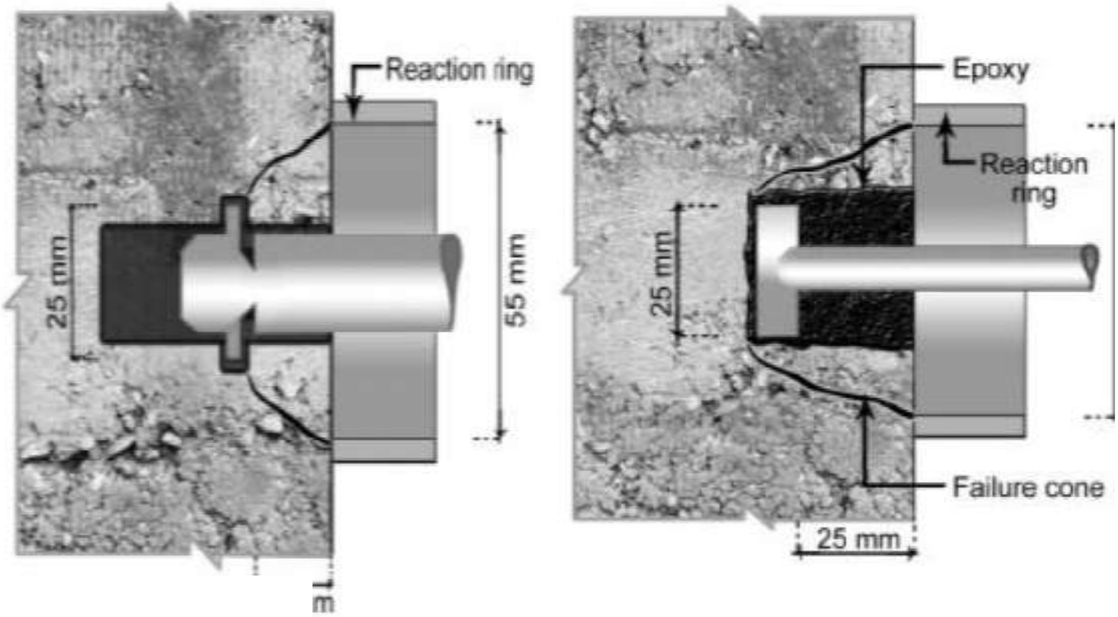


Figure 3.4 (a): Pull-out Test (b):CAPO test (ACI 228.1R-03)

3.6.1.5 Pull-off Method

This is based on in-situ tensile strength of concrete. The details of test are as seen in Figure 3.5 below.

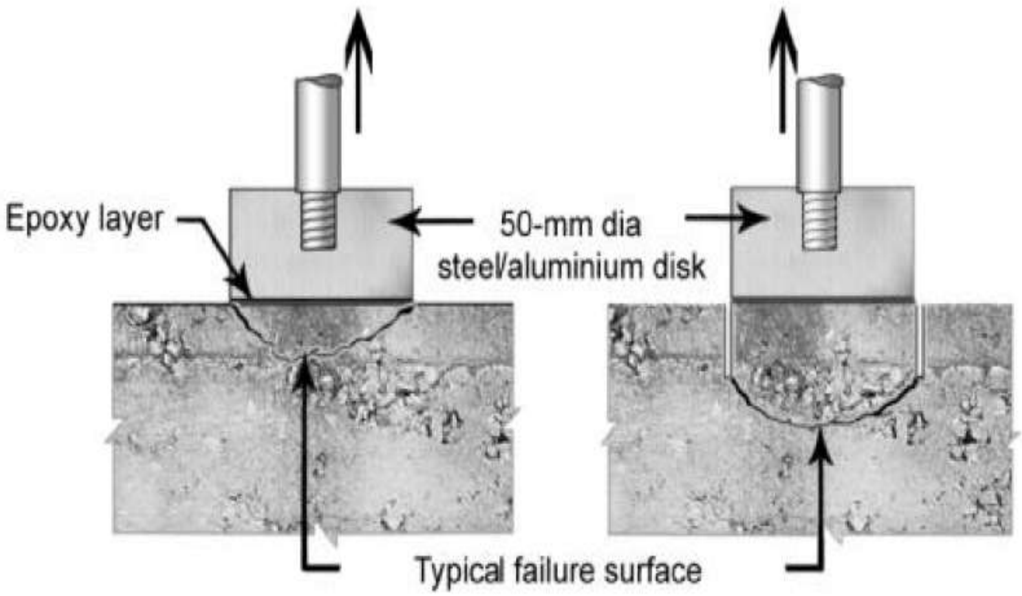


Figure 3.5: Pull-off Test (Anand and Ankush 2007)

3.6.1.6 Location of Steel Reinforcement

This is a non-destructive test for finding the locations of steel reinforcement and also the available thickness of concrete cover. The instrument used to carrying out this technique is Cover meters/Pachometers. It works with the principle that steel are attracted by magnetic field. Another instrument used to find the location of steel in concrete is X-ray. It is also used to find the quantity of available reinforcement steel in concrete.

3.7 Analytical Evaluation

The materials and information gathered during preliminary investigation is used to determine safe load-carrying capacity of the building or the building members. In this analysis, values of moments and forces existing in the structural system are found. Then these values will be used to predict how the structural system will respond to existing load effects. Such analysis done by elastic method provides reasonable estimated values of important load effects. A second part analysis helps to determine the behavior of structures. By doing this, the concrete and steel will be assumed to behave in a linearly elastic manner. Therefore, the strength of the building will be determined considering the principles of strength design as applied in ACI 318. It provides the bases for nominal strength for building members. As the analysis is ongoing, some characteristics of the structure is modeled such as the effect of soil on the column base; the responses of the slab systems for a two way loading; torsional effect on building members. After these evaluations, the analysis will either show that the building has adequate safety according to the requirements by building codes; that the design strength is less to the required factor load but may be greater to the service load; that the design strength is lesser than the service load under the applicable building codes. In the first scenario, the design strength exceeded the required factor load. In the second condition, the building is not adequate but may be permitted for usage if the applicable load does not exceed the computed strength. The final scenario shows that the building is eventually in a bad condition. The owner of the building needs to be notified, and hence a restriction of the use of the building until a remedial work has been taken place.

3.8.2 Load testing

This procedure or technique is the last resort for evaluating structures when the drawings and other information obtained from the preliminary investigation are not available. It is mainly suitable for testing concrete structures. The test described in the previous section can be used to evaluate the concrete characteristics. Load testing is common for deteriorated framing and buildings which are theoretically over-stressed by the proposed loading. When carrying out this test, the frames are subjected to a particular loading and its behavior is monitored. It can be applied on frames by means of sand bags, concrete blocks on the frame. The aim is to produce uniform loading on floors. Hydraulic jacks and air pressure is also used for this purpose. The last two methods have the advantage of being unloaded very fast from frames when failure is envisaged but it is very expensive. A shoring can be constructed to help safeguard against failure. The area to be tested should be found or determined by a well experience engineer who will also supervise the whole process. The procedure of carrying out load test on concrete structures is contained in ACI 318. The deflections on the building members are measured after every load increment and 24 hours after the test. The total load according to ACI 318 should not be less than

$$0.85\ (1.4D + 1.7L) \tag{3.1}$$

where D and L are the intensity of dead load and live load respectively. The magnitude of concentrated load for 2000 lb test specified by building codes can be determined by equation (3.2);

$$0.85\ (2000)\ (1.7) = 2890\ \text{lb} \tag{3.2}$$

This is accepted by ACI 318 as long as there is no evidence of failure. Also, the deflection value of a structural member should not exceed the value in equation (3.3)

$$\frac{{l_t}^2}{20{,}000h} \tag{3.3}$$

Where l_t and h are the span and overall thickness of that member respectively. When the two conditions are not met, the code allow for a repetition of the procedure after seventy-two hours. A second test may be acceptable if the balanced maximum deflection will not exceed 20 % of the maximum deflection during the second test taking from the beginning of the second test. Cracks that occur during the load test should be investigated.

3.8.3 Evaluation Report

The information as obtained from evaluation is arranged which will be used by engineers or contractors to decide the strengthening option to select. The report could contain the description of the building, the required design loading if needed and performance criteria, the assessment and evaluation processes descriptions, the results and conclusion and finally the recommendations. From these results, the engineer can answer the question in strengthening selection tool, but must also understand which strengthening strategy method is desired for the building according to the problem requirements of that building being investigated. Figure 3.6 details the flowchart for building assessment.

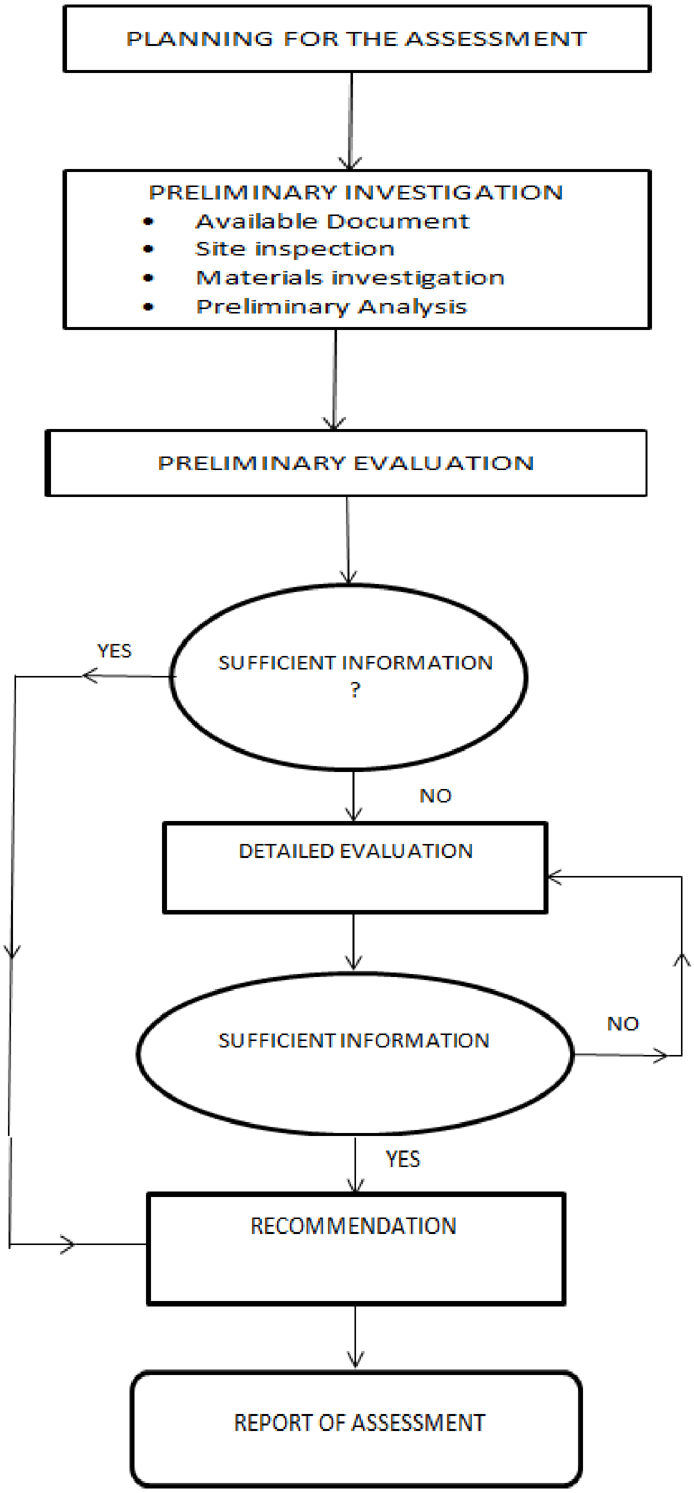


Figure 3.6: Flowchart for building assessment

Chapter 4

CODED DECISION SELECTION PROGRAM FOR STRENGTHENING METHODS

4.1 Introduction

The coded decision selection program is a guide prepared with an excel file. The aim of this program is to guide strengthening engineers to selecting the best suitable strengthening method according to the problem requirement of the building. It compares and assesses a problem input to come about a suitable strengthening technique. A user is required to answer the questions on the guide having at hand the evaluation result from the preliminary or detailed investigations. In answering the questions in the guide, selecting ‘No’ from the answer dropdown menu box means that such constraint is not encountered in the building during the evaluation stage while selecting ‘Yes’ will immediately alert strengthening methods which have been encoded for such constraints to have values at the relevance count. The highest percentage is deemed the best suitable method.

However, a user should also know from the point of view the irregularities in the building which is about to be strengthen, the seismic region the building belongs to and as well as understanding the strengthening strategies suitable to apply before carrying out these schemes.

4.2 Strengthening Strategies

Strengthening can be grouped into three strategies namely; global, local and integrated strengthening methods.

4.3 Global Strengthening Strategy

This is a condition that involves the complete check on the performance of the whole building. The building also may be severely deficient under the design seismic forces. It takes control for lateral strength and stiffness of the building. Therefore, when the general performance of the building is the target, it is required that this method will be considered, (CPWD, 2007), (Xilin Lu, 2010). Examples of strengthening methods under this strategy are steel bracing, shear wall and infill walls.

4.3.1 Problems Solved with Global Strengthening Strategies

Irregularities in the structural configuration, increased forces on columns located at the corners of the building. Torsional irregularity caused by building plan asymmetry. Buildings with roof top, overhead water tanks could result to asymmetry of the structure. The discontinuity of columns at ground storey; an example of in-plane discontinuity is floating column. Curved buildings may result to asymmetry of the building. Strength irregularity could occur when there is a reduction in lateral strength from the top of the storey down to the ground storey. At this point, such storey will result to weak storey. For example, an open ground storey.

4.4 Local Strengthening Strategy

In this method, the target is the strength and ductility of a building member. It does not always affect the lateral strength of the building or the total structural performance for a building, (CPWD 2007), (Xilin Lu 2010). The capacity of members such as the slab, beam and the column can be strengthened individually by this means. The problem could be seen as a result of improper design, poor construction and poor quality materials. Hence, there will be individual failure of the building members either by shear failure or flexural failure of beams, columns and walls, beam-column joints, slab-beam or slab column connections. Examples of strengthening in this strategy are concrete jacketing, steel jacketing, steel plating and fiber reinforced polymer strengthening applications.

4.5 Integrated Strengthening Strategy

This is the combination of different methods of strengthening to achieve a desired demand. This could be the combination of two local strategies to achieve a specified demand or the combination of a global strategy and local strategy. Example is the combination of FRP wrap and steel bracing. It is effectively used to control joint shear failure, stiffness and also reduce maximum inter-storey drift of frames structures (Amoury, 2005).

Another combination is FRP and Steel Jacket. This can be used for structural members that have some deteriorated reinforcement or corroded internal reinforcement (Jinbo et al, 2009).

Other integrated methods are; shear wall and column jacketing, shear wall and steel beam, bracing and column jacketing, bracing and steel beam, steel beam and column jacketing. These combinations however, have given a better result compare to application of a single method of various options.

4.6 Organisation of the Decision Selection Tool

The guide is arranged according to seismic zones of the building. While using this program, buildings which are under earthquake region are seismic or low seismic while non-earthquake regions are termed non-seismic. Entering 'Yes' means the building is located in these zones, while 'No' indicates not located in that zone. The tool is also prepared considering the advantages and disadvantages of strengthening methods as detailed in Table 4.1. Also the type of strengthening in terms of general performance is considered. When global strengthening strategy is entered 'Yes' the tool will assess and compare the problem requirement of the building in terms of total performance of the building. However, if 'No' is the case, it will be assessed as a local strengthening strategy. In Figure 4.1, the building irregularities considered is the projecting elements, the balcony or cantilevers, open floor, floating column, plaza type building and mass irregularities. Also the availability of materials is used to decide if the strengthening materials are readily common for any strengthening project.

The building use disturbance accounts for the resistance to use of the building during any strengthening application. If the time required for the strengthening will hinder the activities of the user of the building, a 'Yes' will be selected and this automatically deselects that method or else, 'No' will be appropriate.

However, in Table 4.1, a further discussion of advantages and disadvantages of the strengthening methods, through some applied projects with different strengthening method as well as the characteristics of the methods are highlighted.

4.7 Projects and Executed Strengthening Methods

Table 4.1: Strengthening methods, Characteristics and Application Targets

Strengthening options	**Steel jacketing**
Applied projects	(1) Orange and Pomona freeway connectors 1991, United States by California Transportation Department. Kassaye (2003). (2) Garanti Bank, Nicosia.
Characteristics	• The strength of the jacket depends on the welding condition between the split jackets. • Limited enhancement to flexural strength. • It improves flexural and shear strength. • It requires no grout for application for a new existing building without degradation. • It improves stiffness, ductility and axial load carrying capacity.
Application targets	• For passive confinement. • To strengthen members with deficiency in transverse reinforcement. • For members with low concrete grade.
Application areas	• Columns. • Beams.
Demerits	• Not suitable for corroded RC at marine environment. • Because it is not anchored to the foundation, it does not provide maximum flexural confinement. • Not good for axial load enhancement when there are gaps at the beginning and end application on a member (25mm to 50mm). • It may add weight to the structure.

Strengthening option	**Steel Plating**
Applied projects	(1) Orange and Pomona freeway connectors 1991, united states by California Transportation Department. Kassaye (2003).
Characteristics	• For flexural and shear strength improvement. • It can be bonded to the surface with epoxy and anchors. • No significant improvement with the application of steel plate with very low concrete grade. • Failure mode when applied with adhesive is premature debonding. With bolts is ductile failure.

Application target	• Applied to side surface to improve shear strength and tension surface to improve flexural strength.
Application Areas	• Columns. • Beams. • Slabs.
Demerits	• It is not safe to apply on a deteriorated concrete member. • There are difficulties in joining two plates by butt-welded joint in the site.

Strengthening options	**Section enlargement**
Applied projects	• US Ski resort; Column and beam enlargement. • Government office building in Washington DC parking structure; the beam suffers an overloading and was enlarged to restore the desired capacity. Concrete Repair Bulletin (2009)
Characteristics	• It is a cheap method. • It involves concrete jacketing of members. • In harshly weather, there could be a possible corrosion of internal reinforcement. • Load capacity and stiffness can be increased.
Application target	• Increase stiffness of structures. • Improve weak column strong beam. • To correct mistake of transverse reinforcement.
Application Areas	• Columns, beams, slabs. • Walls. • Foundations.
Demerits	• Holes created for anchoring is not proper for members that are small in size or poor concrete grade. • There could be disturbance of the use of the building during strengthening.
Strengthening options	**Shear walls**
Applied projects	• Naval station building in Guam Island; reason was to enhance the capacity of the floor diaphragm. Newman (2001) pg 677-678.
	• Historic hotel Utah in Salt Lake City; dangerous soft storey was corrected here with this method. Miller and Reaveley (1996).
Characteristics	• Good application for symmetrical building. • Correct discontinuity in building structures. • It is effective in buildings with flat slab. • It is good for resisting vertical and lateral loads.

Application target	• To strengthen buildings with inadequate amount of shear walls on both sides. • To strengthen walls with inadequate thickness and reinforcement. • Increase lateral stiffness. • Solve soft storey/ weak storey problem. • Staggered column buildings.
Application Areas	• Beams and Walls.
Demerits	• Additional weight to the structure and requires a lot of work. • May lead to building use disturbance during the time application. • It will require additional construction of foundations.
Strengthening options	**Steel bracing**
Applied projects	• School building in Sendal, Japan (1987); reasons was because of several short columns, to improve lateral strength and stiffness. • Building in Mexico City (1990), to carry high overturning forces, strengthen and stiffen the building. Badoux and Jirsa (1990).
Characteristics	• It yields better result in flexible frame. • It gives minimal building use disturbance to the users during construction time. • It is a viable alternative to shear walls in concrete framed building in seismic region. • Its application will barely cause a small mass increase and minimal cost. • It is applied to frames with inadequate lateral resistance. • It could be applied to unsymmetrical building since it will not add much weight to the structure. • It is suitable for plaza type building.
Application target	• For frames with weak short columns. • Increase frame stiffness by providing alternate stiff lateral load resisting system. • To control overturning forces by applying externally to the building.
Application Areas	• Walls between columns and beams. • At weak storeys.
Demerits	• It has non-ductile failure pattern. • It could be liable to corrosion if not protected. • It may be difficult to construct.
Strengthening options	**Fiber reinforced polymer**

Applied projects	• Kings college hospital London, Uk (1996). An extra floor was required and therefore additional live load capacity was increased. • Shiriya-Zaki lighthouse historic building, Japan. Len and Teng (2008).
Characteristics	• It improves the ductility and energy absorption capacity of RC columns. • Its application delays the degradation of stiffness of reinforced concrete columns. • It does not alter the dynamic response of frame; rather, it changes the damage location and patter in the frame. • It has high strength to weight ratio compare to other strengthening materials. • Aesthetic appeal, light weight and non-corrosive.
Application target	• To eliminate non-ductile failure mode. Example is shear failure. • To account for missing transverse reinforcement at joints. • It is used for flexural, axial confinement and shear upgrade and equally cracks control.
Application Areas	• Columns • Beams, slabs, walls. • Wooden beam, column. • Masonry.
Demerits	• It has high cost. • It has brittle behavior. • Inadequate fire resistant. • It does not possess the ductility that steels have. • Its brittleness could limit the ductility behavior of RC member strengthened by this method.

After successful study of the projects and the method of strengthening, with the advantages and disadvantage of each method, the strengthening selection tool was thus constructed. It does not require any code for selection; it is general for all regions for selecting strengthening method. Its requirement is to answer the questions correctly after obtaining the result from the building assessment. The selection tool flow chart is detailed in Figure 4.1 Table 4.2 details the hybrid method. It involves combination of two different strengthening methods to achieve a higher performance for the strengthened building. Figure 4.2 details the selection tool.

Strengthening method		Hybrid method		
Names of methods	**Application project**	**Characteristics**	**Application on target**	**Application area**
Steel Jacket/FRP Jacket	Change of use of school to Library. Strengthening of beam which supports a roof due to error in capacity calculation.	It is less expensive. It is aesthetically pleasing.	Improves bearing capacity.	Slabs with openings. Ribs slabs. Members with corroded internal reinforcement.
Shear wall/ column Jacketing	Building with low size columns	It gives better performance to the individual method	Controls lateral stiffness and weak column.	Columns and between columns.
Shear wall/ steel beam.	To be considered	Gives better performance compare to the separate methods.	Controls lateral stiffness and reduce longer span.	Between columns and on beams.

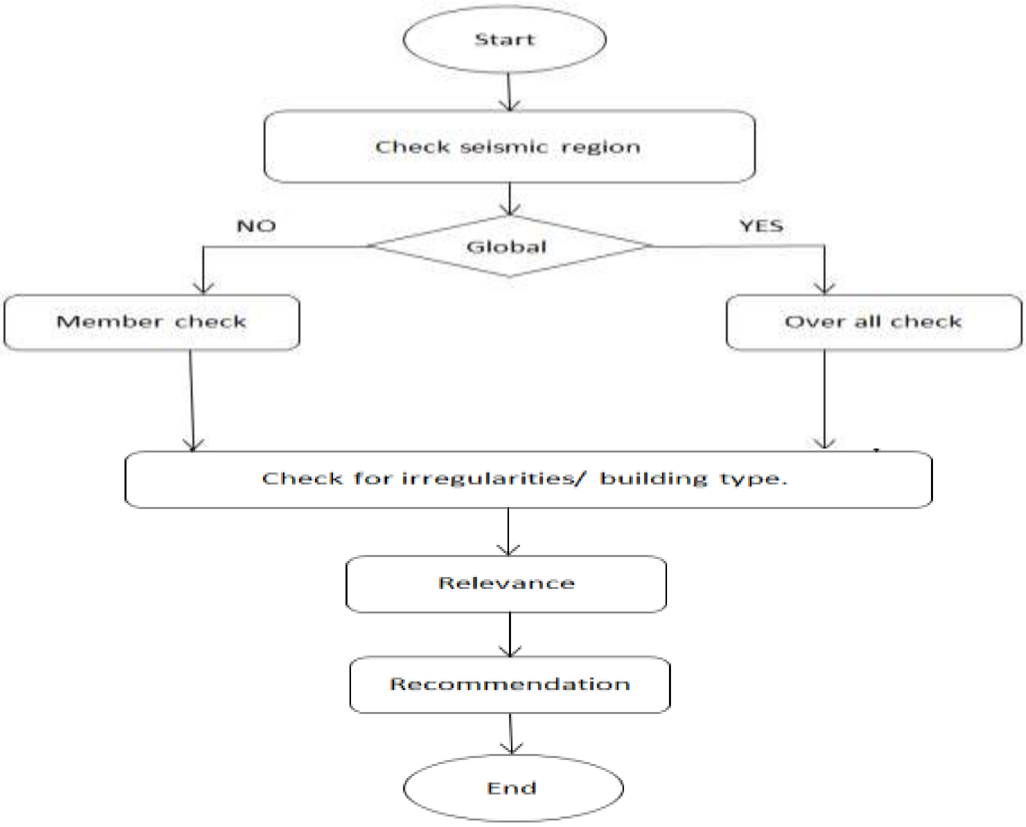


Figure 4.1: Decision selection tool flowchart

Seismicity	
Non seismic	No
Low seismic	Yes
Seismic	No
Strengthening strategy	
Global strategy	Yes
Local stagtegy	
Column	
low dimensions	No
degraded column	No
missing transverse rebars	No
low concrete grade	Yes
short column	No
strong beam/weak column	Yes
corroded internal rebar	No
Joints	
missing transverse rebars	Yes
Beam	
low concrete grade	Yes
low dimensions	No
missing transverse rebars	No
over stressed beam	Yes
Slab	
low thickness	Yes
inadequate rebar	No
long span	No
Foundation	
low dimensions	No
weak soil	No
Building irregularity	
Regular	Yes
Irregular	
Unsymmetric	
projecting elements	No
open floor	Yes
floating/staggered column	No
plaza type building	No
mass irregularity	Yes

Strengthening options	Shear walls	Steel bracing	Concrete jacketing	FRP jacket	Steel plating	Steel jacketing	FRP wrap	FRP / steel bracing	FRP / steel jacket	span shortening	FRP strip	soil improvement	Shear wall/ column
Relevance	0.43	0.43	0.57	0.29	0.29	0.43	0.29	0.14	0.29	0.14	0.29	0	0.14
Availability	Yes	Yes	Yes	Yes	No	No	Yes	No	No	Yes	Yes	Yes	Yes
Building use disturbance	Yes	No	Yes	No	Yes	Yes	No	Yes	No	Yes	No	No	No
Proposition	0	0.43	0	0.29	0	0	0.29	0	0	0	0.29	0	0.14
Strengthening Choices	Shear walls	Steel bracing	Concrete jacketing	FRP jacket	Steel plating	Steel jacketing	FRP wrap	FRP / steel bracing	FRP / steel jacket	span shortening	FRP strip	soil improvement	Shear wall/ column

Figure 4.2 Decision selection tool

Chapter 5

CODED PROGRAMS FOR FRP STRENGTHENING

5.1 Introduction

With the aid of Mathematica version 8.0, programs were prepared for the calculation and checks for shear and flexural stresses for beams and axial load capacity for columns. It is important to know that these programs allow us to determine the corresponding thickness and area of FRP to be used for strengthening structural members. It is based on limit-state design principle which has provided an acceptable safety check for serviceability and ultimate-limit states. There are various guidelines such as fib Task Group 9.3 (2001), Concrete Society (2004), ACI code (2005), ACI committee 440 (2002) for design and estimating the required number and thickness of FRP to strengthen beams and columns. In this study, an ACI code (2005) guideline is used for beam strengthening (Perumalsamy et al, 2009). However, the design principle by Priestley et al (1996) for steel jacketing was adopted for column strengthening with FRP.

5.2 Fundamental Assumptions

- The reinforcement ratio should satisfy both maximum and minimum requirements according to codes utilized.
- The distribution of strain across the thickness is linear.
- The concrete maximum strain at failure is 0.003.

- The steel maximum stress is f_y.
- The distribution of stress of a concrete is assumed rectangular with an average stress of $0.85f'_c$ and a depth of a.
- For the depth of stress block, a= β ×c where c is the depth of neutral axis.
- β=0.85 if $f'_c \leq$ 4000 psi.

  0.65 if $f'_c \geq$ 8000 psi,

  $0.85 - 0.05(\frac{f'_c - 4000}{1000})$ if 4000 < f'_c < 8000 psi.

5.3 Failure modes of FRP

The failure of FRP can be related to debonding, FRP rupture, and concrete crushing. Failure due to rupture could occur when there is a balanced condition. This is a situation or the point at which the maximum compressive tensile strain in FRP and strain in concrete, reach their ultimate values at the same time. ACI 318 (2002) laid stresses on a failure mode of reinforced concrete beams based on the yield of steel reinforcement in tension.

For flexural strengthening, the designed moment is mostly greater than the balance moment of resistance and therefore the failure occurs as concrete crushing. Hence, area of FRP will be determined from the strain-stress behavior of concrete and steel reinforcement. In order to avoid the failure by debonding of FRP, there should be a proper anchorage at the ends.

5.4 Beam Flexural Strengthening Analysis

The depth of the neutral axis as detailed in Figure 5.1 can be obtained by force equilibrium;

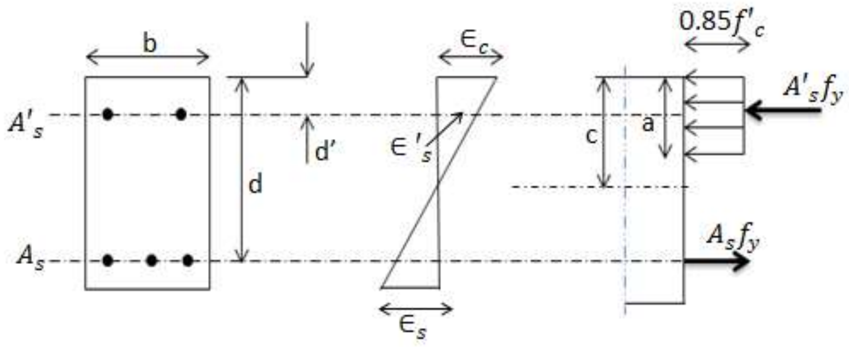


Figure 5.1: Stress and strain distribution for elastic rectangular section

$$0.85f'_c ba = A_s f_y \tag{5.1}$$

$$a = \beta_1 c \tag{5.2}$$

where β_1 is taken as 0.85. For rectangular beams with compression reinforcements, the neutral axis can be obtained by;

$$0.85f'_c ba + A'_s f'_s = A_s f_y \tag{5.3}$$

Stress in compression steel can be found with equation 5.4. But however, it will be solve by trial and error adjustment.

$$\mathcal{E}'_s = 0.003\ (\frac{c-d'}{c}) \tag{5.4}$$

where f'_c, A_s, f_y, f'_s are compression strength, area of steel, yield strength of steel and yield strength at compression side respectively. The yield strength at the compression side f'_s can be obtained with equation (5.5)

$$f'_s = E_s \mathcal{E}'_s \tag{5.5}$$

where E_s is the elastic modulus of steel reinforcement. This value can be used to recompute a using equation (5.3). For flanged beam section with tension reinforcement only, a can be found using

$$0.85f'_c bh_f + 0.85f'_c b_w\left(a - h_f\right) = A_s f_y \tag{5.6}$$

However, if there is a compression reinforcement, then the equation will be

$$0.85f'_c bh_f + 0.85f'_c b_w\left(a - h_f\right) + A'_s f_y = A_s f_y \tag{5.7}$$

Finding the nominal moment capacity for rectangular section without compression reinforcement, use equation (5.8)

$$M_n = 0.85f'_c ba(d - \frac{a}{2}) \quad (5.8)$$

For rectangular beams with compression reinforcement;

$$M_n = 0.85f'_c ba\left(d - \frac{a}{2}\right) + A'_s f'_s (d - d') \quad (5.9)$$

The factored computed should be less or equal to $\emptyset M_n$ where $\emptyset$ is 0.9 for flexure. Therefore, the factored load is calculated as;

$$w_u = 1.2w_D + 1.6w_L \quad (5.10)$$

where w_D and w_L are intensities of dead load and live load respectively. The minimum reinforcement ratio can thus be found with equations below;

$$\frac{A_s}{b_w d} \quad (5.11)$$

$$\frac{200}{f_y} \quad (5.12)$$

$$\frac{3\sqrt{f'_c}}{f_y} \quad (5.13)$$

Where d and b_w are the depth and web thickness of a beam, then the reinforcement ratio will be satisfactory if the check in this order is true;

$$\frac{A_s}{b_w d} > \frac{3\sqrt{f'_c}}{f_y} > \frac{200}{f_y} \quad (5.14)$$

Then, the reinforcement ratio is satisfactory as long as equation (5.14) is true while equations 5.11 and 5.12 and 5.13 must be greater than equation 5.15.

$$0.75[\frac{b_w}{b}\left(\frac{0.85\beta_1 f'_c}{f_y}\right)\left(\frac{87000}{87000+f_y}\right) + \frac{0.85f'_c\,(b-b_w)h_f}{f_y bd}] \quad (5.15)$$

5.4.1 Analysis of Strengthened Beam

The analysis is the same with the previous analysis for unstrengthen beam. However, the only difference is the addition of the tensile force from the FRP.

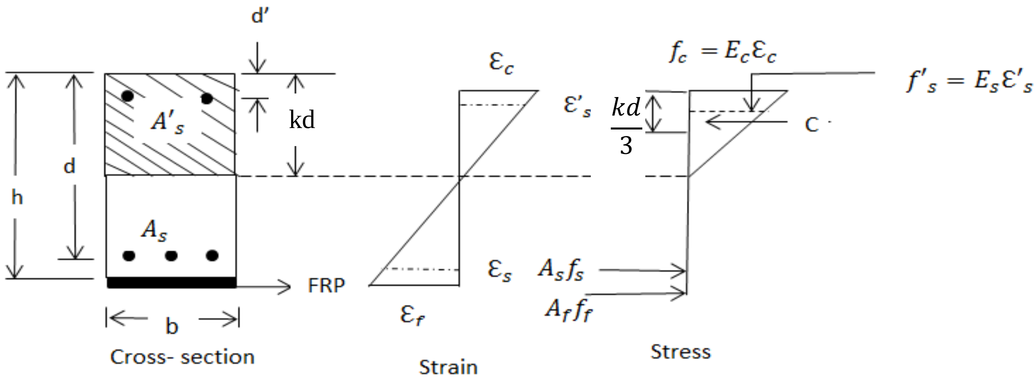


Figure 5.2: Strain and stress distribution for strengthened reinforced concrete beam

The added FRP plate provides more strength to the concrete beam section. Thus, the application of this FRP as detailed in Figure 5.2 will lead to a change in the neutral axis in the beam. This neutral axis can be determined with the equation below;

$$\frac{bkd^2}{2} + (n-1)A'_s(kd - d') = nA_s\,(d - kd) + n_f A_f(h - kd) \tag{5.16}$$

Where n_f is found by

$$n_f = \frac{E_f}{E_c} \tag{5.17}$$

$n_f A_f$ is the tensional force contributed by FRP, and then area of FRP can thus be calculated by

$$A_f = \frac{M_u - \emptyset M_{ni}}{\phi\,(E_f 0.85\varepsilon_{fu} \times 0.9h)} \qquad 5.18)$$

where E_f is the modulus of elasticity of FRP while A_f is the area of FRP, M_u is the designed ultimate moment while $\emptyset M_{ni}$ is the initial moment before strengthening. Cracked moment of inertia can thus be computed with equation (5.19)

$$I_{cr} = \frac{bkd^3}{2} + nA_s\,(d - kd)^2 + (n-1)A'_s\,(kd - d') + n_f A_f\,(h - kd)^2 \tag{5.19}$$

However, the stresses caused by the moment after strengthening on the concrete, steel and the composite can be determined by the equations below while Figure 5.3 is the flowchart for use of FRP flexural strengthening for rectangular beam. The chart shows only the application of FRP. The checks for stresses on FRP materials should be done after finding the area and thickness of FRP materials required for the flexural strengthening enhancement.

$$f_c = \frac{M}{I_{cr}} kd \tag{5.20}$$

$$f_s = \frac{M}{I_{cr}} (d - kd)n \tag{5.21}$$

$$f'_s = \frac{M}{I_{cr}} (kd - d')n \tag{5.22}$$

$$f_f = \frac{M}{I_{cr}} (h - kd)n_f \tag{5.23}$$

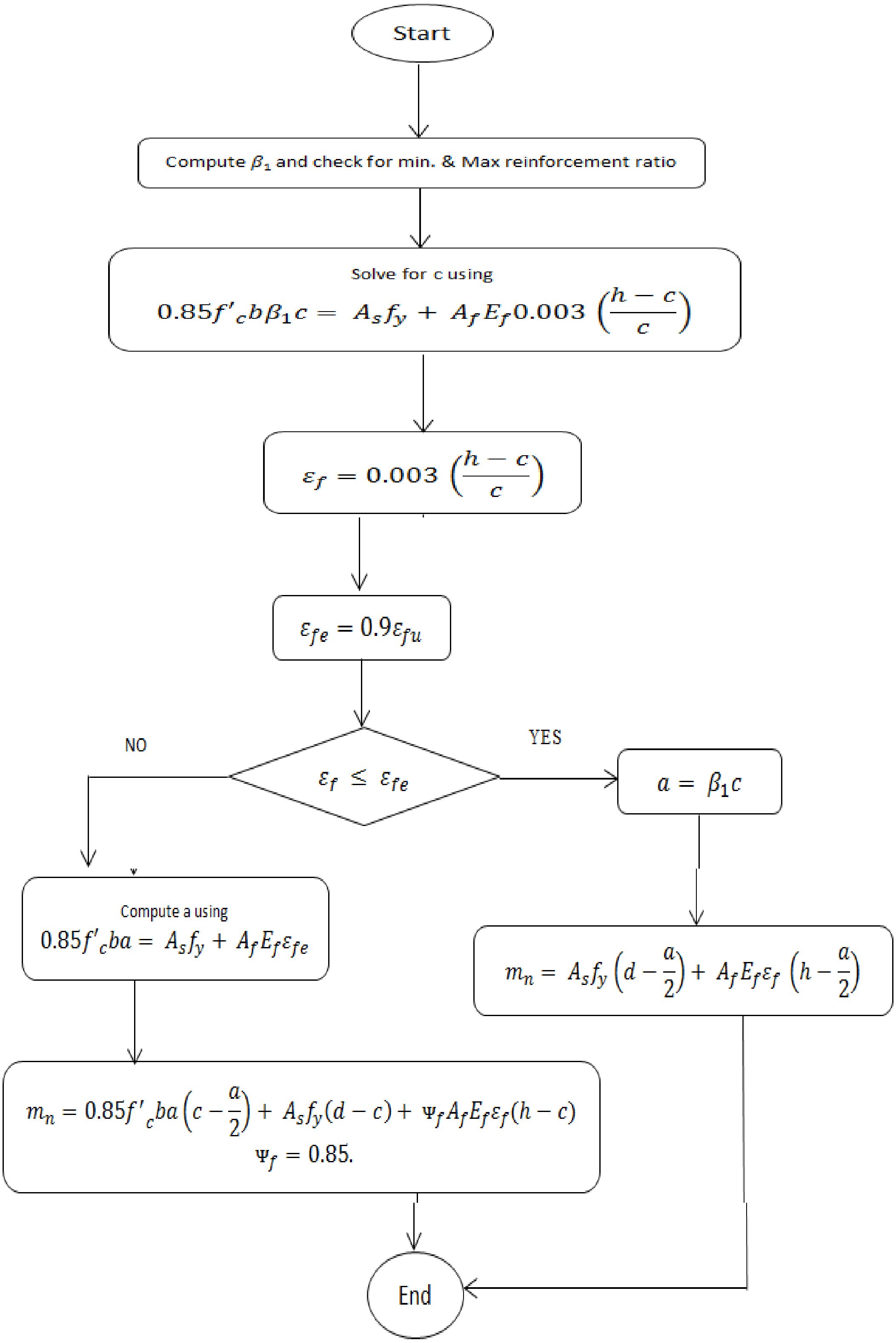


Figure 5.3: Flowchart for Beam Strengthening

5.5 Column Strengthening

Calculation on column confinement are done based on that, column are not exposed to buckling. Also columns are completely wrapped with FRP. There is no slip between the confinement and the concrete. The compressed concrete fibres at the cross-section are subject to equal longitudinal strains. Also the confined area by internal tie reinforcement for circular column strengthening is approximately assumed to be equal to the core cross-section.

With the principle of strength of materials, the confining pressure induces by FRP on columns is a function of composites thickness and stiffness and thus calculated by equation (5.24);

$$F_p = \frac{2t_j f_f}{D} \tag{5.24}$$

where D is the diameter of the column. While the force generated by the layer is found by;

$$f_f = n_f t_f E_f \mathcal{E}_f \tag{5.25}$$

The effective fracture strain of composite at failure $\mathcal{E}_{fe}$, is maximum at 0.004 and $0.75\mathcal{E}_{fu}$.

At failure, the confining pressure is found by;

$$f_{cp} = \frac{E_f \mathcal{E}_{fe} \rho_f}{2} \tag{5.26}$$

where the parameters are as same in beam design. Based on experimental result (ACI 440, 2002) compressive strength at this confining pressure is calculated by;

$$f'_{cc} = f'_c[2.25\sqrt{1 + 7.9\frac{f_{cp}}{f'c}} - 2\frac{f_{cp}}{f'c} - 1.25] \tag{5.27}$$

Shear corresponding to this peak stress is found by;

$$\mathcal{E}'_{cc} = \mathcal{E}'_c[\frac{6f'_{cc}}{f'_c} - 5] \tag{5.28}$$

where f'_c and $\mathcal{E}'_c$ are peak compressive stress and the corresponding strain for unconfined concrete. When $\mathcal{E}'_c$ is not given, one can estimate it by;

$\varepsilon'_c = 1.71\frac{f'_c}{E_c}$ (5.29)

A reduction factor Ψ_f is recommended for concrete contribution of 0.95. The nominal capacity of a column with spiral reinforcement can be computed using the following equations;

$$Pu = 0.85[0.85f'_{cc}\ \Psi_f(Ag - A_{st}) + A_{st}f_y] \quad (5.30)$$

$$Pu = 0.80[0.85f'_{cc}\ \Psi_f(Ag - A_{st}) + A_{st}f_y] \quad (5.31)$$

is for columns with lateral ties, where Ag is the cross-sectional area of the concrete, A_{st} is the area of longitudinal reinforcement, f_f is the force generated by the composite, E_f is the modulus of elasticity of the fiber, t_j is the equivalent thickness of fibers, ε_f is the hoop strain of the fiber while n_f is the number of layers of FRP. Figure 5.4 details the flowchart for FRP use for column strengthening.

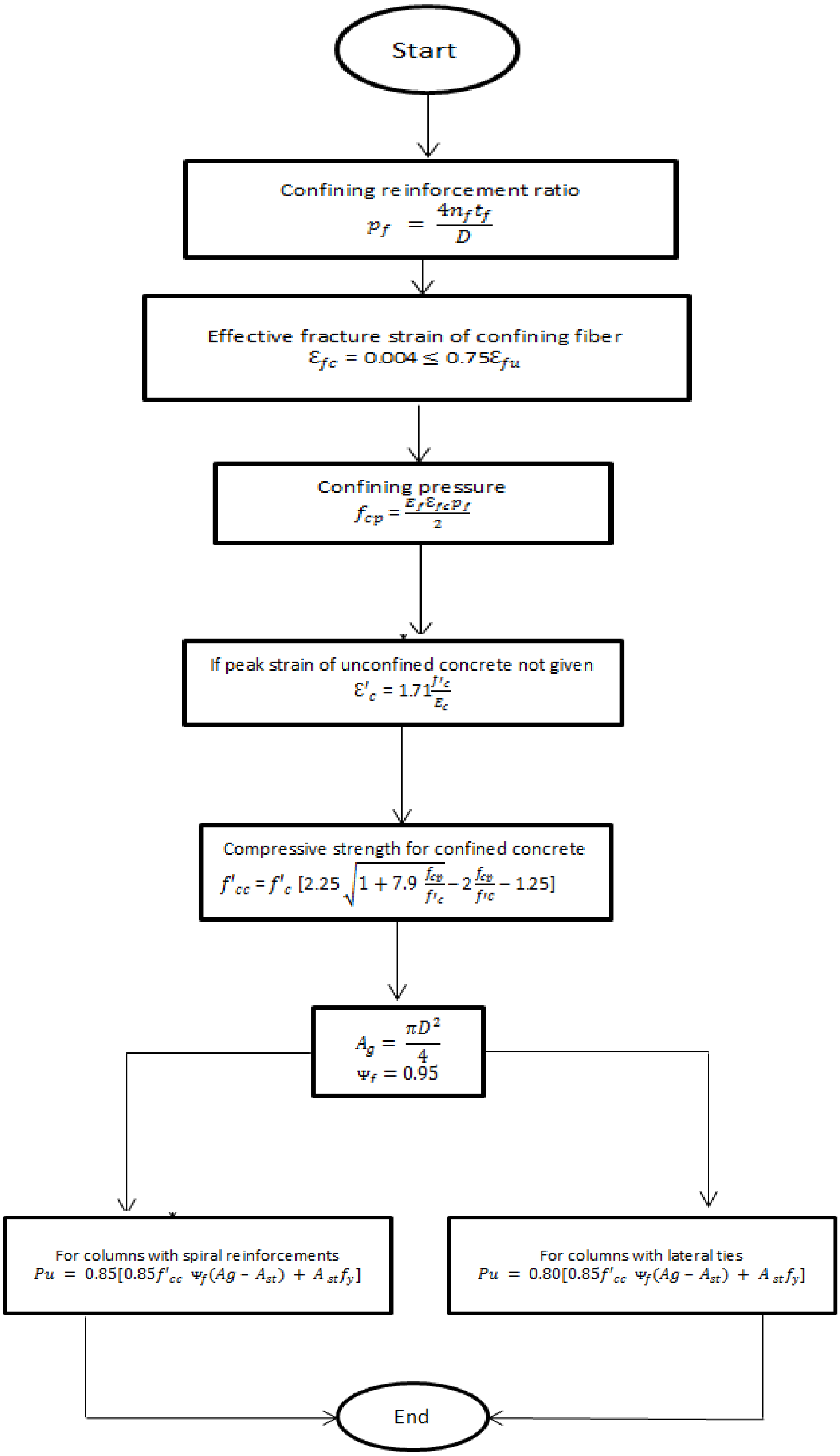


Figure 5.4: Flowchart for column strengthening

5.6 Beam Shear Design Example

This reference taking from Newman (2001) will be compared with the FRP coded program. It was originally designed to carry two point loads from mechanical equipment spaced at 6 ft apart. New equipment was installed of the same magnitude, but now is spaced 3 ft apart as detailed in Figures 5.5. Table 5.1 contains the design parameters while Table 5.2 details the design calculation. Figures (5.7-5.9) is the coded program.

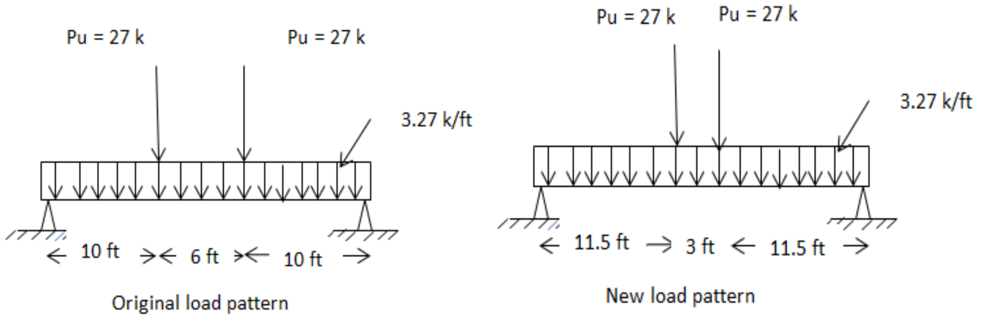


Figure 5.5: Original load part and new load part

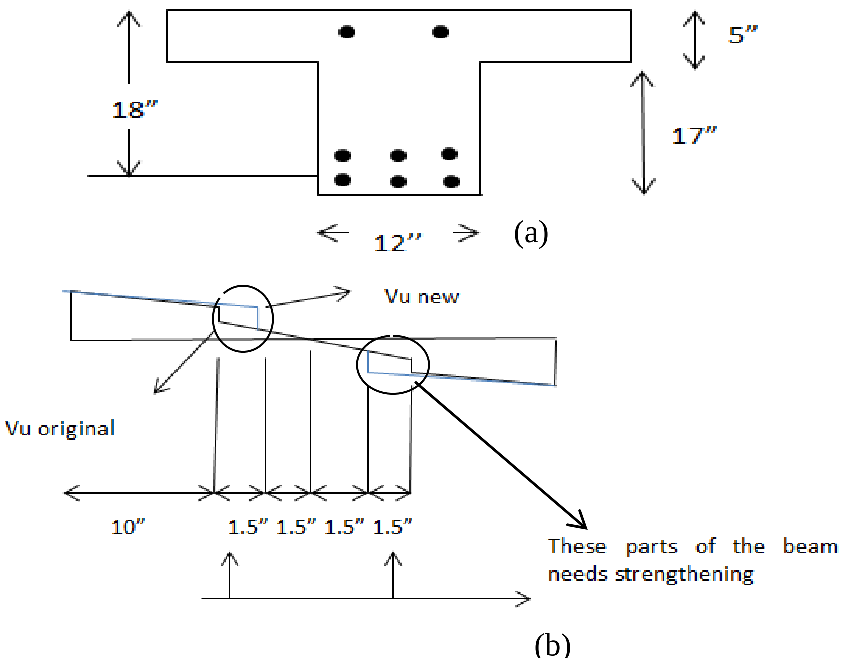


Figure 5.6: (a) Cross section of beam (b) shear required shown by the circled regions on the beam

Table 5.1 Beam material properties

Concrete	**Steel**	**FRP**
f'_c = 4000 psi	F_y = 60 ksi	f_{fu} = 550 ksi
		ε_{fu} = 0.017

Table 5.2: Referenced shear capacity for Beam

To assess the current condition; $V_c = 2\sqrt{f'_c} b_w d = 2\sqrt{4000}$ x12x18 27322 lb = 27.3 kips. To check if the stirrup in the new position of load is enough; Vu initial $> \frac{\phi V_c}{2}$. $\frac{(\phi Vc)}{2} = \frac{(0.85 \times 27.3)}{2}$; 11.6 kips > Vu initial = 9.81 kips. In the new Vu which is 36.8 kips > $\frac{1}{2}$ (ϕVc), hence, additional shear reinforcement shall be provided. Finding the shear contribution that must be provided by the FRP Vu = ϕ(V_c+ 0.85V_f) 36.81 kips = 0.85 (27.3 kips + 0.85V_f) V_f,reqd = 18.8 kips. Selecting the type of FRP to use for this strengthening, MBrace CF 130 for shear strengthening and 20 in wide will be used. Its properties are detailed in the Table 5.1 above. Determining the effective bond length L_0, for effective bond length of one ply of FRP is 2 in. However, for MBrace CF 130, the effective length for one strip will be;

$L_e = \frac{1}{\sqrt{n}} L_0$ = 2 in,

So for one ply, n = 1.

To determine the reduction factor on the ultimate strength of the sheet;

k_1 (Multiplier on effective bond length for concrete strength)

$(\frac{f'_c}{4000})^{\frac{2}{3}} = 1.$

The depth of FRP shear reinforcement d_f is the depth to tension steel minus the slab thickness;

d_f = d - h_s

18 – 5 = 13 in.

Calculate the effective bond length of FRP shear reinforcement d_{fe}.

$d_{fe} = d_f - L_e$

13 – 2 = 11 in.

For multiplier k_2

$\frac{d_{fe}}{d_f}$ = 11/13 = 0.846.

Calculate R (reduction factor)

$\frac{k_1 k_2 L_e}{486 \varepsilon_{fu}}$ = 0.213.

The stress level in the fiber can be at ultimate load

$f_{fe} = R f_{fu}$,

0.213 x 550 = 116.9 ksi.

Determine the shear contribution of the FRP.

$V_f = \frac{AfvFfe\ (sin\ \beta + cos\ \beta)\ df}{s_f} \leq 4\sqrt{f'_c}\ b_w$ d.

A_{fv} = total area of one strip, β = 90º (orientation angle with respect to the longitudinal axis of the beam). Also S_f is the spacing of the strip taken as 12 in.

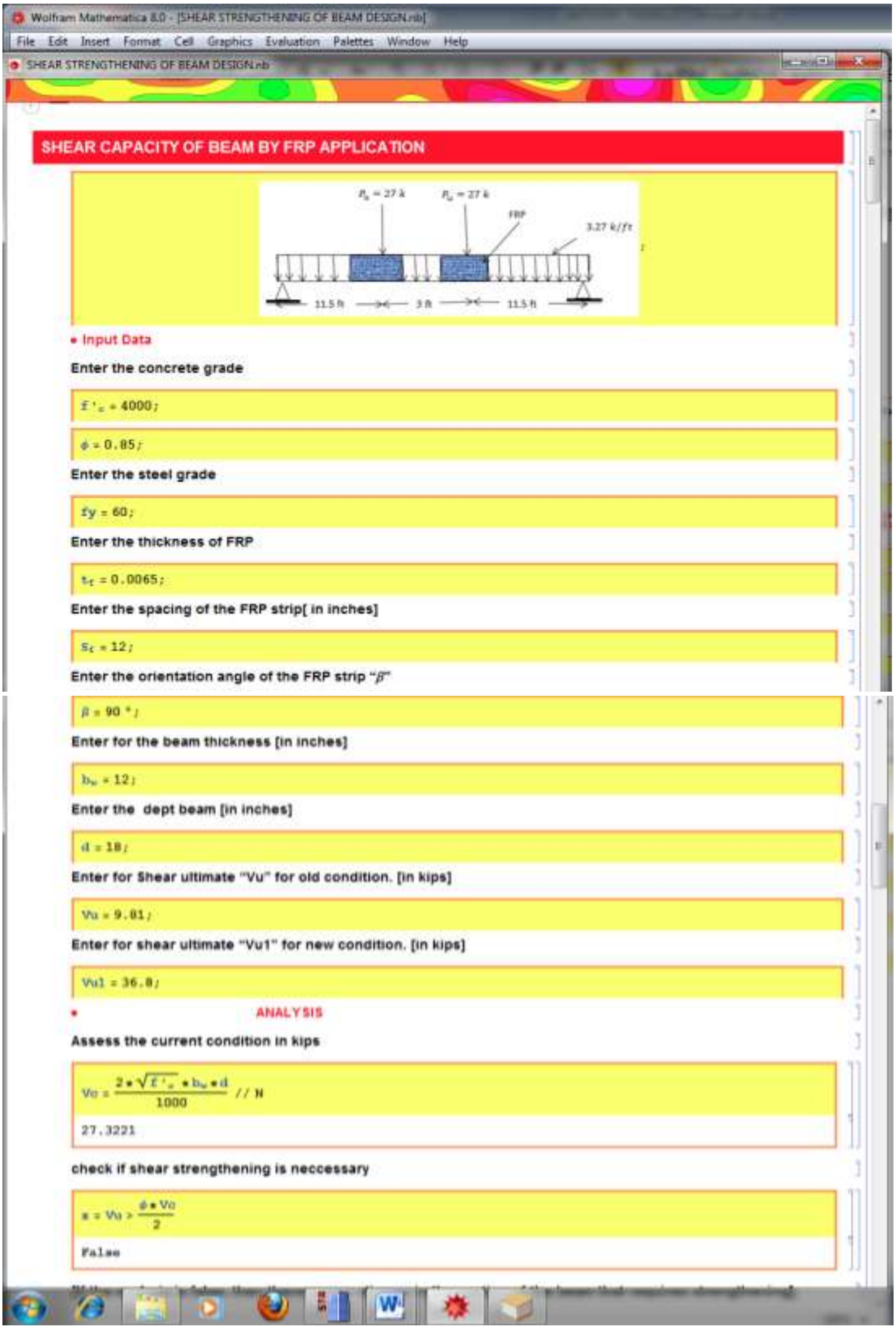
$$V_f = \frac{2 \text{ x } 1 \text{ x } 0.0065 \text{ in x } 20 \text{ in x } 116.9 \text{ ksi x } (1+0) \text{x } 13 \text{ in}}{12 \text{ in}} \leq 4\sqrt{4000\ psi} \text{ x12 in x 18 in}$$

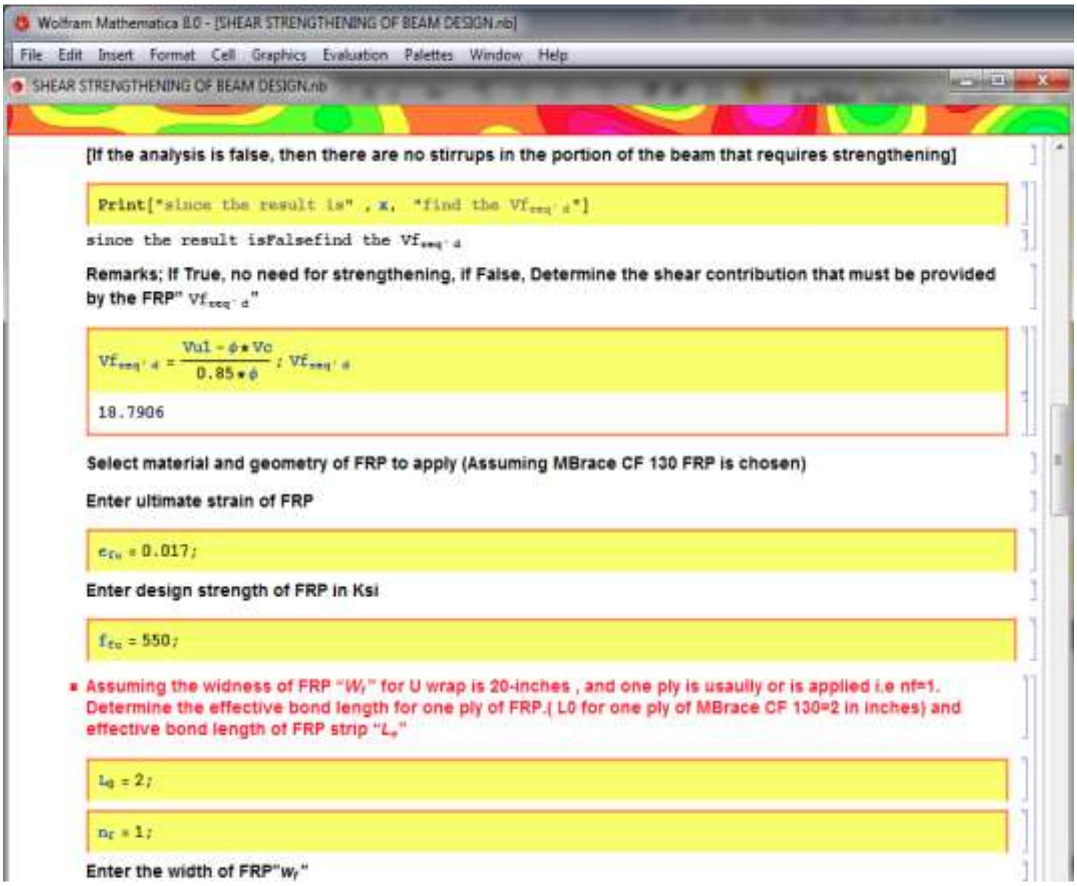
32.9 kips < 54.6 kips.

V_f = 32.9 kips > $V_{f,req'd}$ = 18.8 kips.

Hence one ply is sufficient.

5.7 Coded beam design





Wolfram Mathematica 8.0 - [SHEAR STRENGTHENING OF BEAM DESIGN.nb]

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SHEAR STRENGTHENING OF BEAM DESIGN.nb

[If the analysis is false, then there are no stirrups in the portion of the beam that requires strengthening]

```
Print["since the result is" , x, "find the Vf_req'd"]
since the result isFalsefind the Vf_req'd
```

Remarks; If True, no need for strengthening, if False, Determine the shear contribution that must be provided by the FRP" $Vf_{req'd}$"

$$Vf_{req'd} = \frac{Vul - \phi * Vc}{0.85 * \phi}; Vf_{req'd}$$

18.7906

Select material and geometry of FRP to apply (Assuming MBrace CF 130 FRP is chosen)

Enter ultimate strain of FRP

$e_{fu} = 0.017;$

Enter design strength of FRP in Ksi

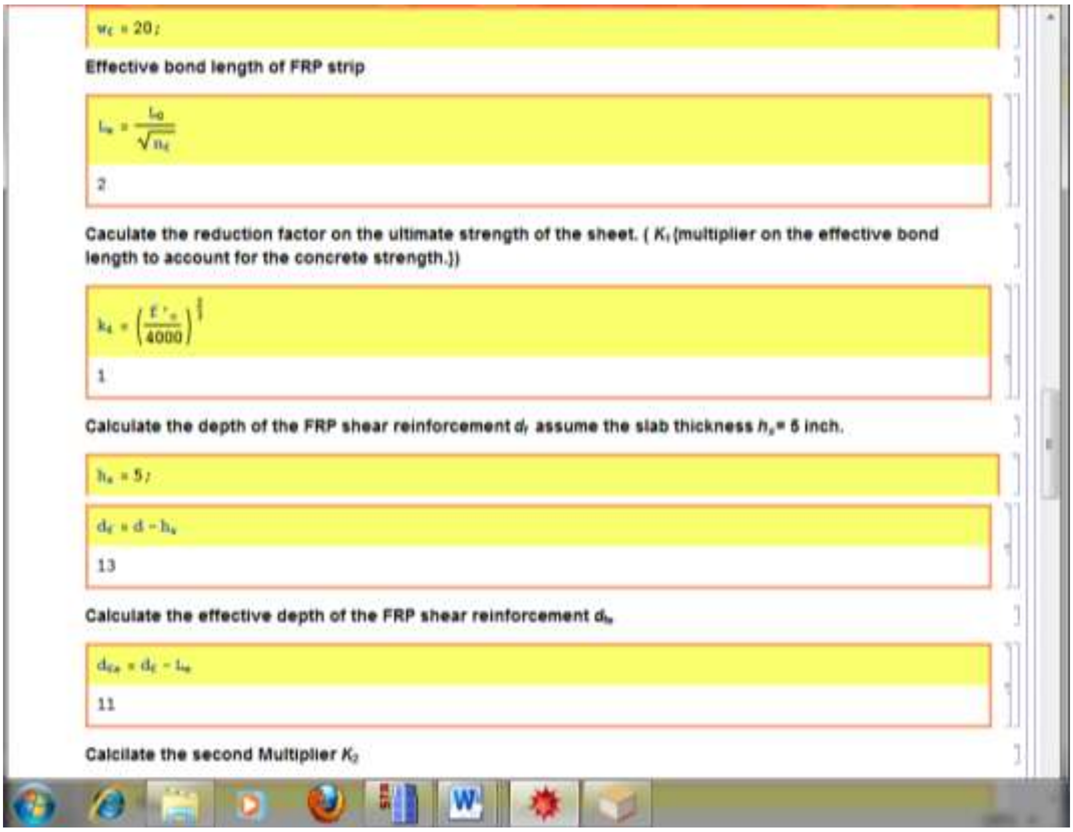
$f_{fu} = 550;$

- Assuming the widness of FRP "W_f" for U wrap is 20-inches , and one ply is usaully or is applied i.e nf=1. Determine the effective bond length for one ply of FRP.(L0 for one ply of MBrace CF 130=2 in inches) and effective bond length of FRP strip "L_e"

$L_0 = 2;$

$n_f = 1;$

Enter the width of FRP"w_f"



$w_f = 20;$

Effective bond length of FRP strip

$$L_e = \frac{L_0}{\sqrt{n_f}}$$

2

Caculate the reduction factor on the ultimate strength of the sheet. (K_1 (multiplier on the effective bond length to account for the concrete strength.))

$$k_1 = \left(\frac{f'_c}{4000}\right)^{\frac{2}{3}}$$

1

Calculate the depth of the FRP shear reinforcement d_f assume the slab thickness h_s= 5 inch.

$h_s = 5;$

$d_f = d - h_s$

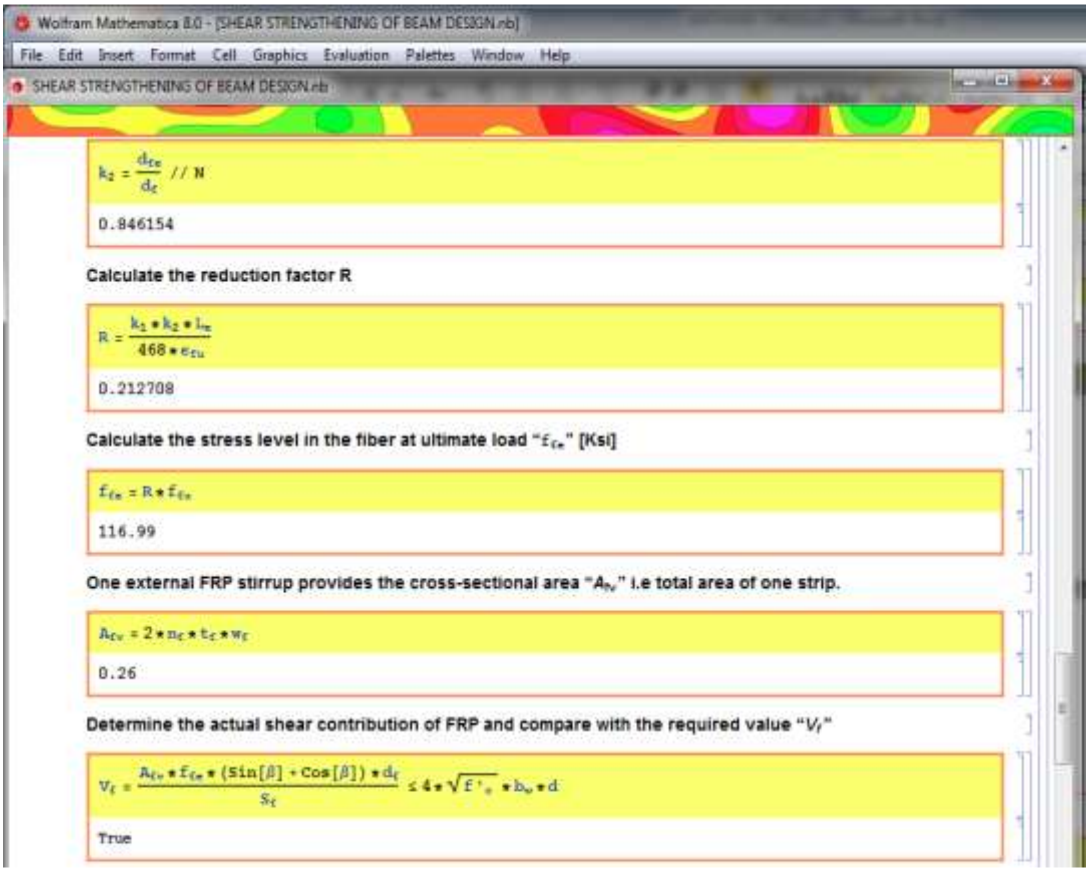
13

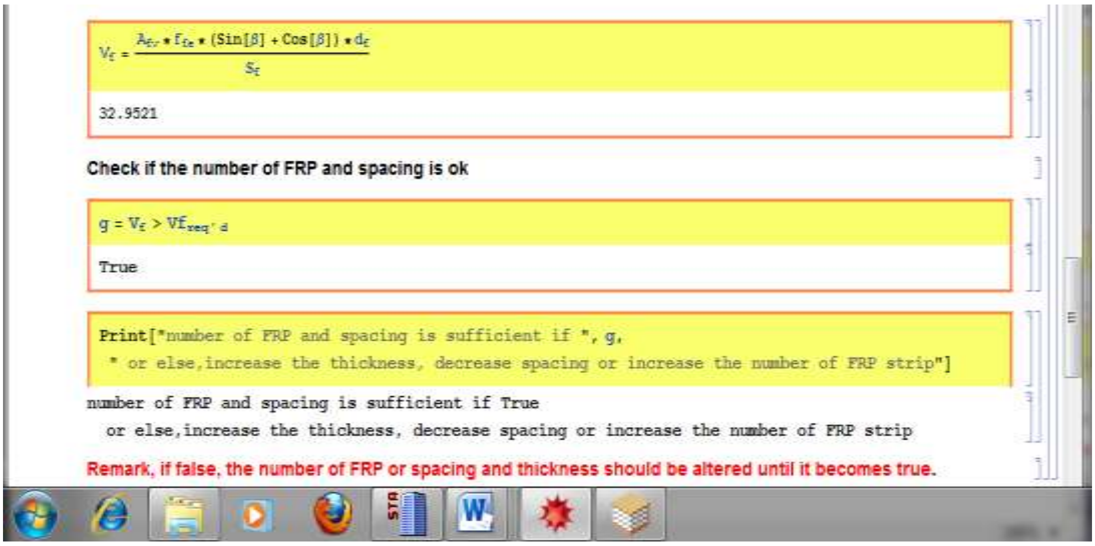
Calculate the effective depth of the FRP shear reinforcement d_e

$d_{fe} = d_f - L_e$

11

Calcilate the second Multiplier K_2





Figures 5.7 to 5.9: FRP Coded Shear Strengthening

5.8 Flexural Strengthening Example

A second reference from Perumalsamy et al (2009) the flexural capacity of beam can be strengthened. The properties of FRP and beam are detailed in Table 5.3 and 5.4.

Table 5.3: Beam design properties

Concrete	**Steel reinforcement**	**FRP**
f'_c = 4000 psi	f_y = 60000 psi	E_f = 33 x 10^6 psi
$d = 16.5\ in, d' = 2.5\ in$	E_s = 29 x 10^6 psi	ε_{fu} = 0.017
ε_c = 0.003	A_s = 3.6 in^2 (6 - # 7 bars)	t_f = 0.0065 in
β = 0.85, $\emptyset = 0.90$	A'_s = 0.88 in^2 (2 - # 6 bars)	Length of beam span = 24 ft

Table 5.4: Current loads and upgraded load

Current loads	**Upgraded loads**
W_{LL} = 650 lb/ft	W_{LL} = 1100 lb/ft
W_{DL} = 1450 lb/ft	W_{DL} = 1450 lb/ft
W_{WL} = 2100 lb/ft	W_{WL} = 2550 lb/ft
W_{UL} = 3135 lb/ft	W_{UL} = 3900 lb/ft

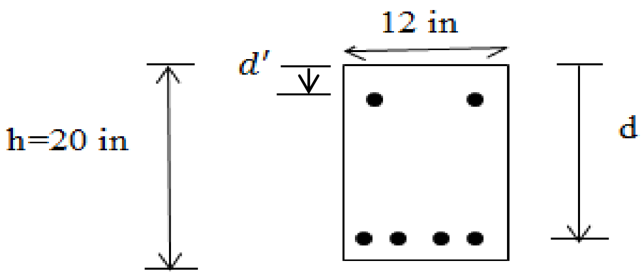


Figure 5.10: Section of beam for strengthening

Table 5.5: Hand Calculation for flexural beam strengthening

Neglecting the contributions of compression steel, the neutral axis can thus be found as;
a = $\frac{A_s f_y}{0.85 f'_c b} = 5.30\ in.$
Hence, c = $\frac{a}{\beta} = \frac{5.30}{0.85} = 6.23\ in.$

To find strain at compression level;

$\mathcal{E}'_s$ = 0.003 $((c - d')/d) =$ 0.0018.

Strain at the compression side $f'_s = 0.0018 \times 29 \times 10^6 = 52066\, psi.$

As compression steel force is added, the position of neutral axis changes and a decrease in f'_s

If f'_s is taken to be 45000 psi, the next iteration for new neutral axis is;

$$a = \frac{3.6 \times 60000 - 0.88 \times 45000}{0.85 \times 4000 \times 12} = 4.32\, in.$$

Hence, new $c = 5.09\, in.$

Therefore, f'_s will now become;

$$29 \times 10^6 \times 0.003 \left(\frac{5.09 - 2.5}{5.09}\right) = 44209\, psi.$$

Taking f'_s = 44600 psi,

$$a = \frac{3.6 \times 60000 - 0.88 \times 44600}{0.85 \times 4000 \times 12} = 4.33\, in.$$

Checks for reinforcement minimum ratio;

$$\frac{200}{f_y} = 0.0033.$$

$$\frac{\sqrt[3]{f'_c}}{f_y} = 0.00316.$$

$\frac{A_s}{bd} = 0.018$. Hence, 0.018 > 0.0033 > 0.00316, the value is OK. Therefore the minimum reinforcement check is satisfactory. Now checking for maximum reinforcement ratio;

$$0.75 \times \beta \left(\frac{0.85 f'_c}{f_y}\right) \frac{87000}{87000 + f_y} = 0.0213.$$

$$\frac{A_s}{bd} - \frac{A'_s}{bd}\left(\frac{f'_s}{f_y}\right) = 0.0151.$$

Comparing the two results; 0.0151 < 0.0213 is true. Therefore the maximum reinforcement is ok.

The existing moment on the beam is calculated as;

$$\emptyset M_{ni} = 0.9\left[0.85 f'_c ba\left(d - \frac{a}{2}\right) + A'_s f'_s\,(d - d')\right] = 2773755\ lb.$$

For the revised loads,

$$M_u = \frac{W_{UL}L^2}{8}, \qquad \frac{3900 \times 24^2 \times 12}{8} = 3369.60\ lb.$$

This is the ultimate moment of the upgraded load. To calculate the required area of FRP for the additional moment;

$$A_f = \frac{M_u - \emptyset M_{ni}}{\emptyset(E_f 0.8\mathcal{E}_{fu} \times 0.9h)} = 0.084\ in.$$

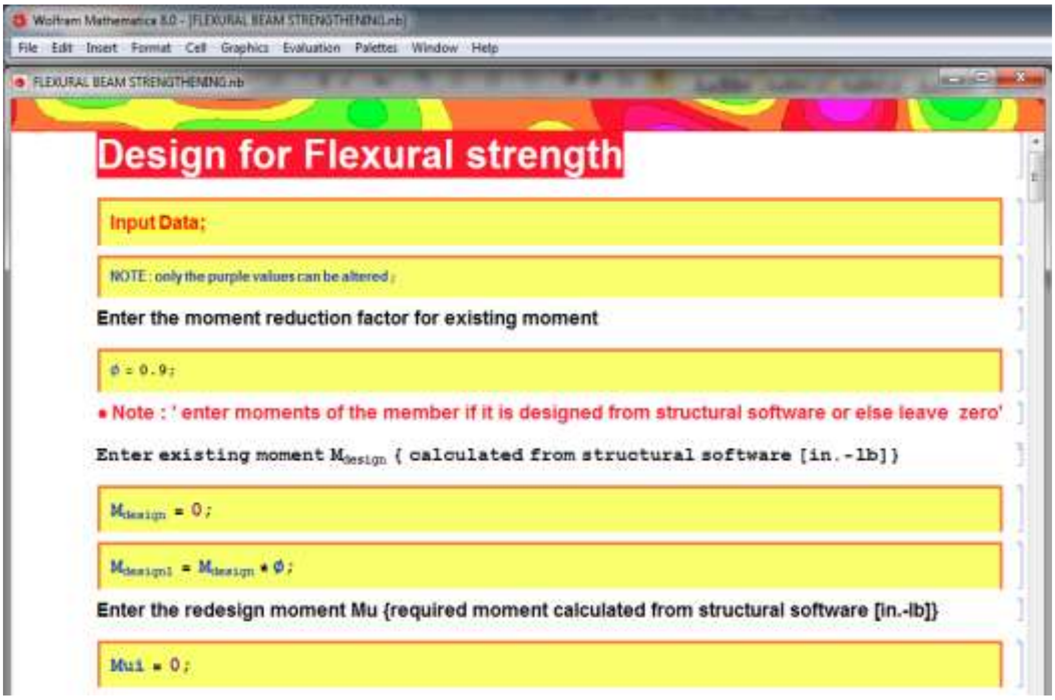
The width of fiber can be calculated considering the available thickness of FRP. Hence, for FRP with a thickness of $0.0065\ in$, the width will be;

$$\frac{0.084}{0.0065} = 12.9\ in.$$

However, since the sheet comes in $20\,in$ width, 3 layers of $10\,in$ are recommended.

Hence, $A_f = 3 \times 10 \times 0.0065 = 0.195\ in^2$.

5.9 Coded FRP Flexural Strengthening



Wolfram Mathematica 8.0 - [FLEXURAL BEAM STRENGTHENING.nb]

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FLEXURAL BEAM STRENGTHENING.nb

```
Sp = 24;
```

Enter the value of current load [lb/ft]

```
WuLL1 = 3135;
```

Enter the value of upgraded load [lb/ft]

```
WuLL2 = 3900;
```

Enter the compressive strength of concrete [psi]

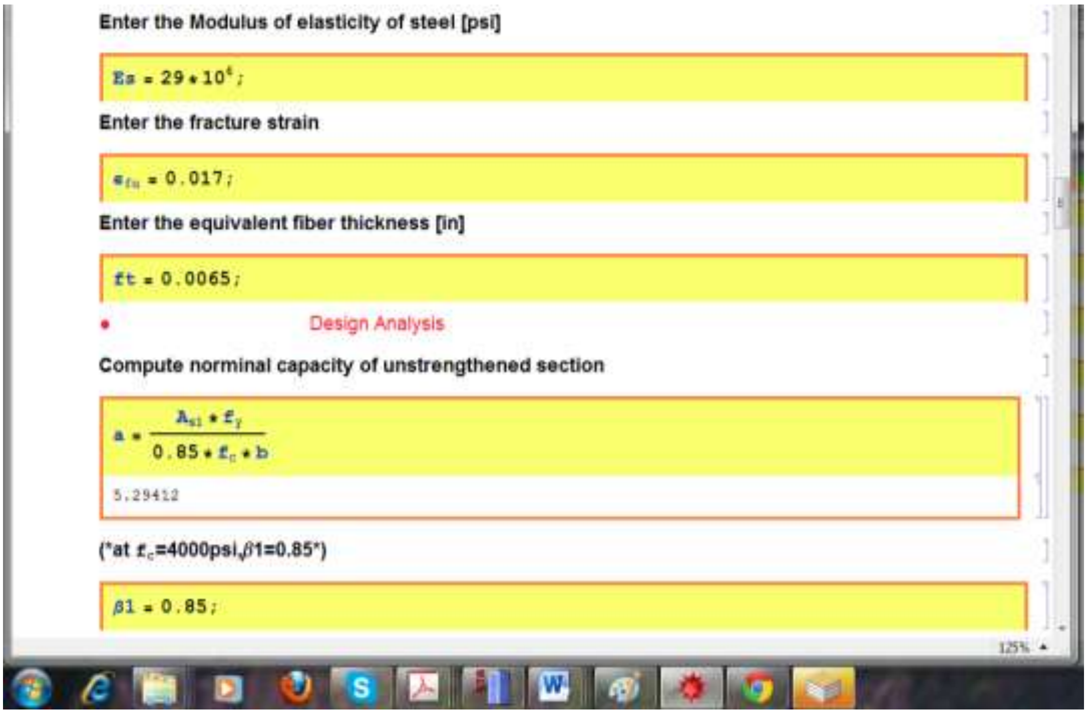
$f_c = 4000;$

Enter the yield strenght of steel [psi]

$f_y = 60000;$

Enter the modulus of elasticity of carbon fibers [psi]

```
Ef = 33000000;
```



Enter the Modulus of elasticity of steel [psi]

$Es = 29 * 10^6;$

Enter the fracture strain

$\epsilon_{fu} = 0.017;$

Enter the equivalent fiber thickness [in]

```
ft = 0.0065;
```

- Design Analysis

Compute norminal capacity of unstrengthened section

$$a = \frac{A_{s1} * f_y}{0.85 * f_c * b}$$

5.29412

(*at f_c=4000psi,β1=0.85*)

$\beta1 = 0.85;$

125%

Wolfram Mathematica 8.0 - [FLEXURAL BEAM STRENGTHENING.nb]

```
c = a / β1
6.22837
```

Compute strain "εs"at compression level

```
εs = 0.003 (1 / c (c - d2))
0.00179583
```

```
fs = εs * Es
52079.2
```

[ADDITION OF COMPRESSION STEEL FORCES DECREASES THE DEPT OF THE NEUTRAL AXIS AND fs]

[ASSUME NEW fs by increasing to fs1]

```
fs1 = 45000;
```

Run second iteration to find new decreased neutral axis

```
a1 = (As1 * fy - As2 * fs1) / (β1 * fc * b)
4.32353
```

```
c1 = a1 / β1
5.08651
```

[Compute for new fs, now as fs2]

```
fs2 = Es * 0.003 ((c1 - d2) / c1)
44239.8
```

```
Print["The stress in compression steel is between ", fs2 , and,
 fs1]
The stress in compression steel is between 44239.8and45000
```

(*Check for minimum reinforcement ratio,Assume fs3=44600*)

```
fs3 = 44600;
```

```
a = (As1 * fy - As2 * fs3) / (0.85 * fc * b)
4.33216
```

Check for minimum reinforcement ratio:

$k_i = \frac{200}{f_y};$

$k_{ii} = \frac{3 * \sqrt{f_c}}{f_y};$

$k_{iii} = \frac{A_{s1}}{b * d_1};$

$k_{iii} > k_i > k_{ii}$

True

Check for maximum reinforcement ratio:

$qi = 0.75 * \beta 1 \; ((0.85 * f_c) / f_y) * \left(\frac{87000}{87000 + f_y}\right)$

0.0213801

$qii = \frac{A_{s1}}{b * d_1} - \frac{A_{s2}}{b * d_1} \left(\frac{f_{s3}}{f_y}\right)$

0.0148781

$qii < qi$

True

$Mnii = 0.9 \left(0.85 * f_c * b * a1 * \left(d_1 - \frac{a1}{2}\right) + A_{s2} * f_{s3} * (d_1 - d_2)\right);$

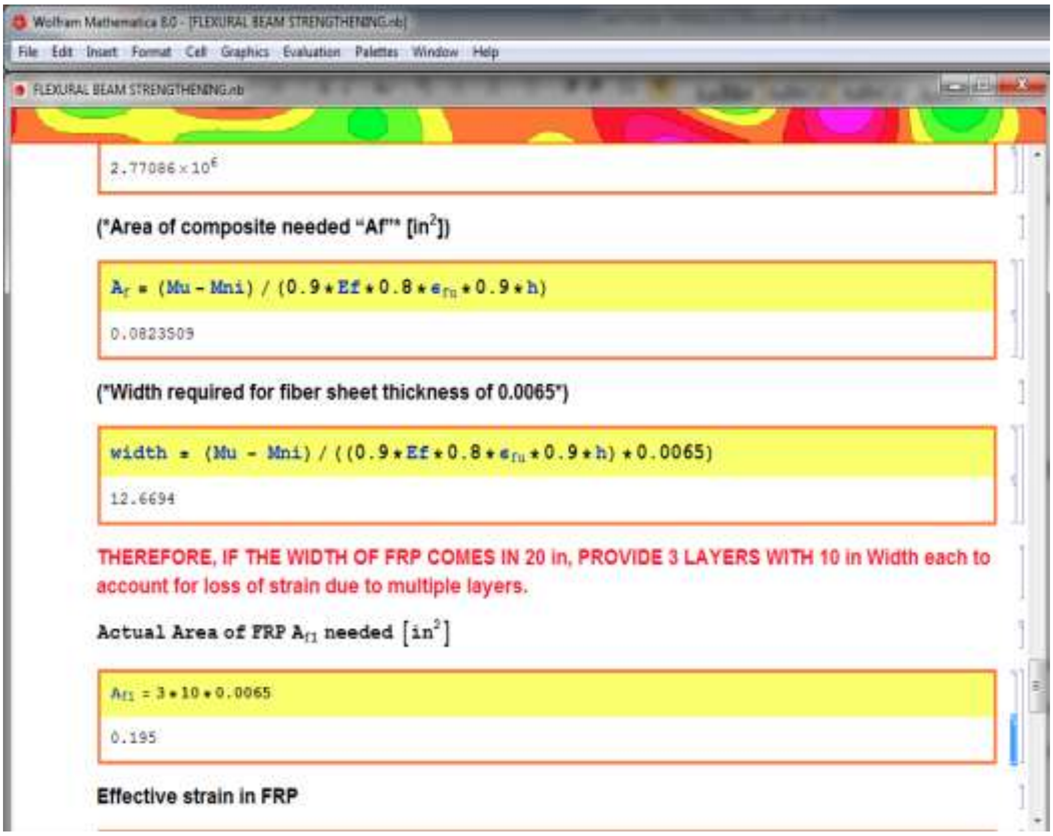
(*For the revised loads*)

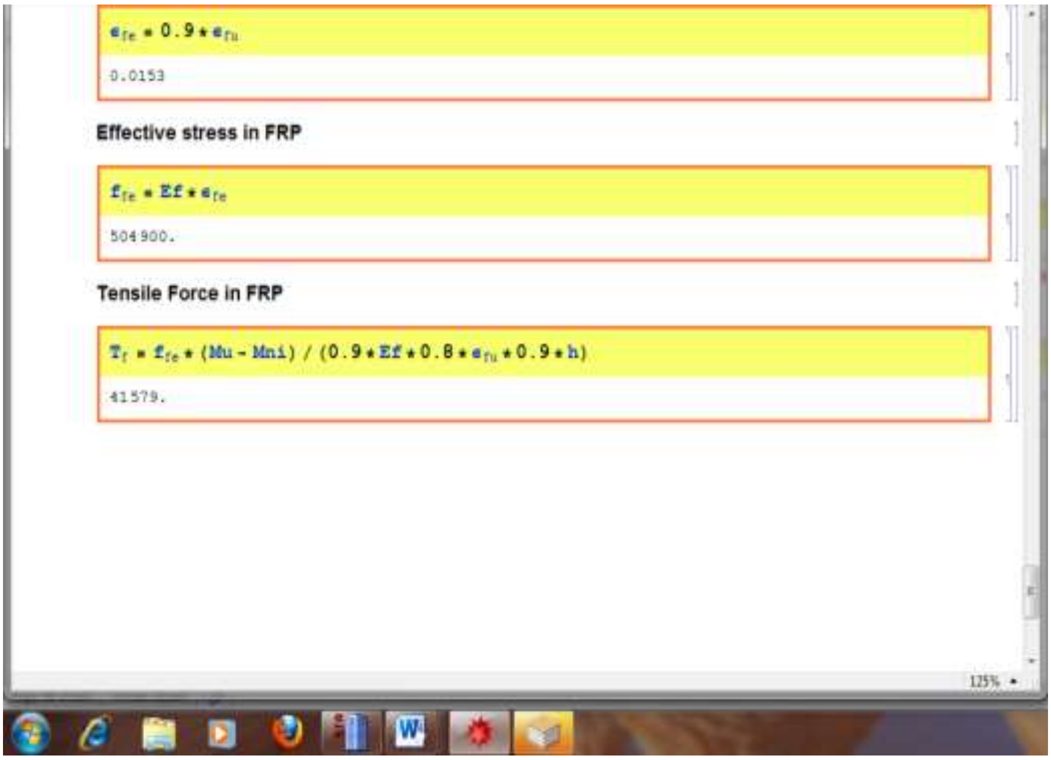
$Muii = \frac{1}{8} WuLL2 * Sp^2 * b;$

Mu = Max[Muii, Mui]

3369600

Mni = Max[$M_{designi}$, Mnii]





Figures 5.11 to 5.15: FRP Coded Program for Flexural Strengthening

5.10 Column Design Example

This reference is taken from Perumalsamy et al, (2009) where the axial capacity of 16 in diameter column with 10#7 bars and reinforcement with #3 bars are designed. The hand calculation will be compared with the design of the computer program (Figures 5.16 to 5.18). It consists of three layers of carbon confinement. Table 5.6 details the parameters for the strengthening while Table (5.7) shows the design calculation procedure.

Table 5.6: FRP properties for column design

Concrete	**Steel**	**FRP**
f_c = 5000 psi	f_y = 60,000 psi	$\mathcal{E}_f$ = 33 *10^6 psi
16 in diameter	A_{st} = 6 in^2	$\mathcal{E}_{fu}$ = 0.0167
		t_f = 0.0065 in

Table 5.7: Column hand design calculation

Confinement fiber reinforcement ratio;

$$p_f = \frac{4n_f t_f}{D} = \frac{4 \times 3 \times 0.0065}{16} = 0.0049.$$

The effective strain of confining fibers should be equal to 0.004 or less of 0.75$\mathcal{E}_{fu}$;

$\mathcal{E}_{fc}$ = 0.004 ≤ 0.75 × 0.0167 = 0.0117.

For the confining pressure;

$$f_{cp} = \frac{E_f \mathcal{E}_{fc} p_f}{2} = \frac{33 \times 10^6 \times 0.004 \times 0.0049}{2} = 323.4\, psi.$$

Find the compressive strength of concrete;

$$f'_{cc} = f'_c\, [2.25 \sqrt{1 + 7.9\, \frac{f_{cp}}{f'_c}} - 2\, \frac{f_{cp}}{f'c} - 1.25]$$

$$= 5000\, [2.25 \sqrt{1 + \frac{7.9 \times 323.4}{5000}} - \frac{2 \times 323.4}{5000} - 1.25] = 6932\, psi.$$

Cross sectional area of concrete;

$$Ag = \frac{\pi\ 16^2}{4} = 200.96\ in^2.$$

The nominal axial capacity of the column can be obtained as follows;

$Pu\ =\ 0.85\ [0.85\ f'_{cc}\ \Psi_f(Ag - A_{st})\ +\ A_{st}f_y]$

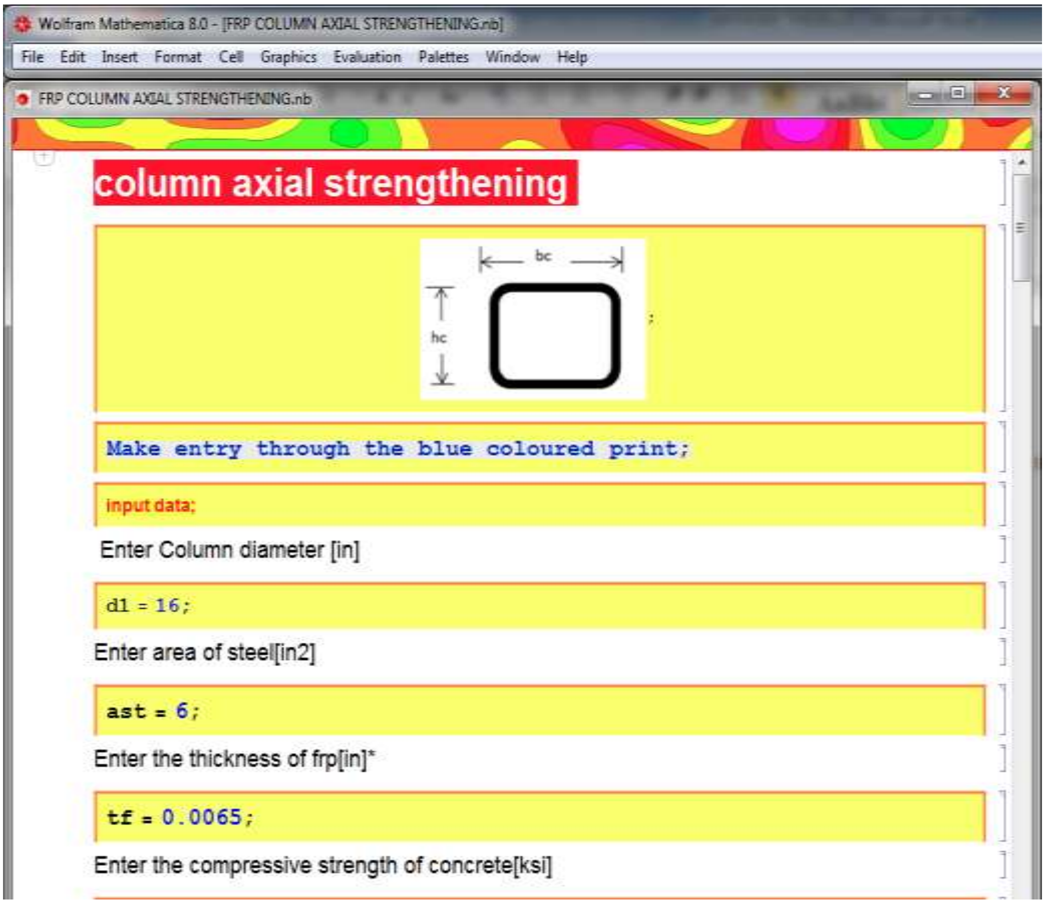
0.85 [0.85x 6932 x 0.95 (200.96 – 6.0) + 6.0 x 60,000]

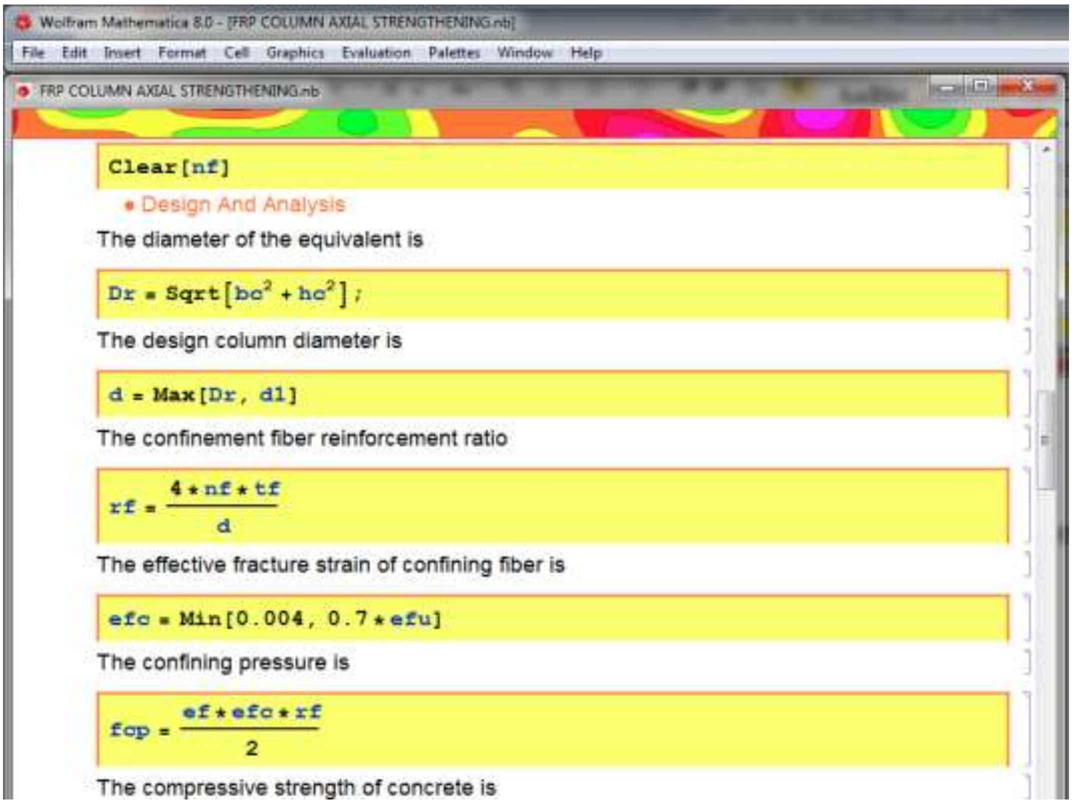
1,233,610 lb

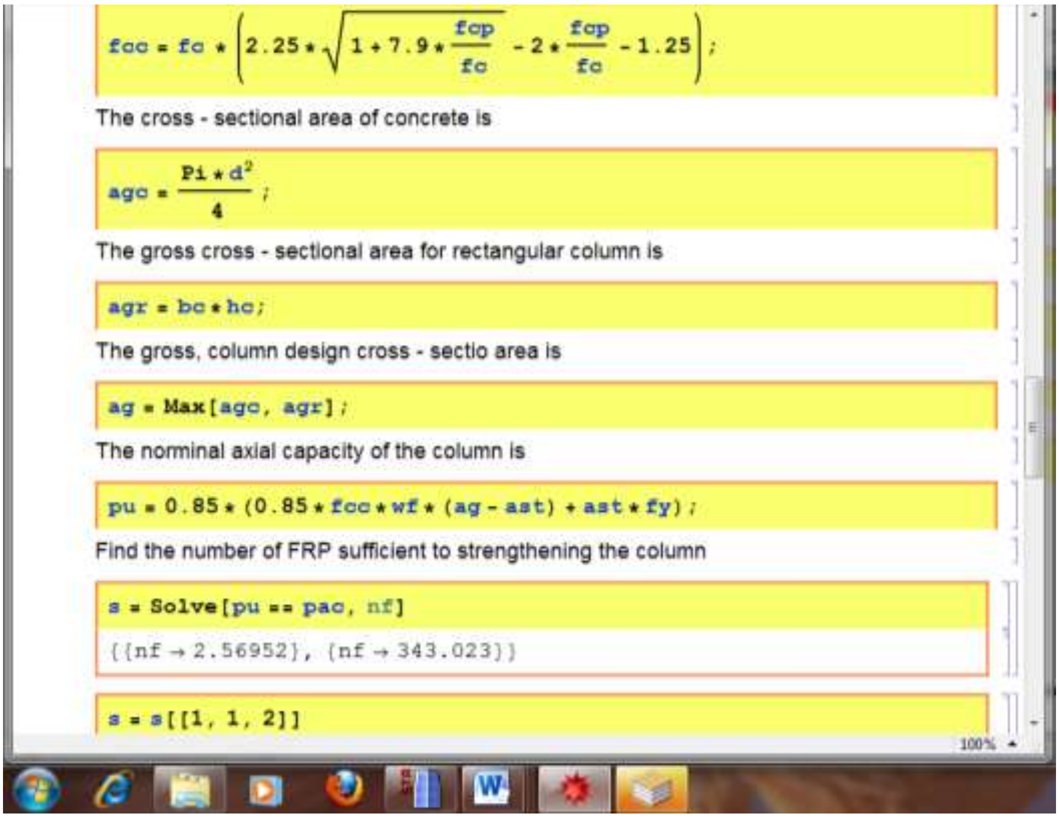
1233 kips.

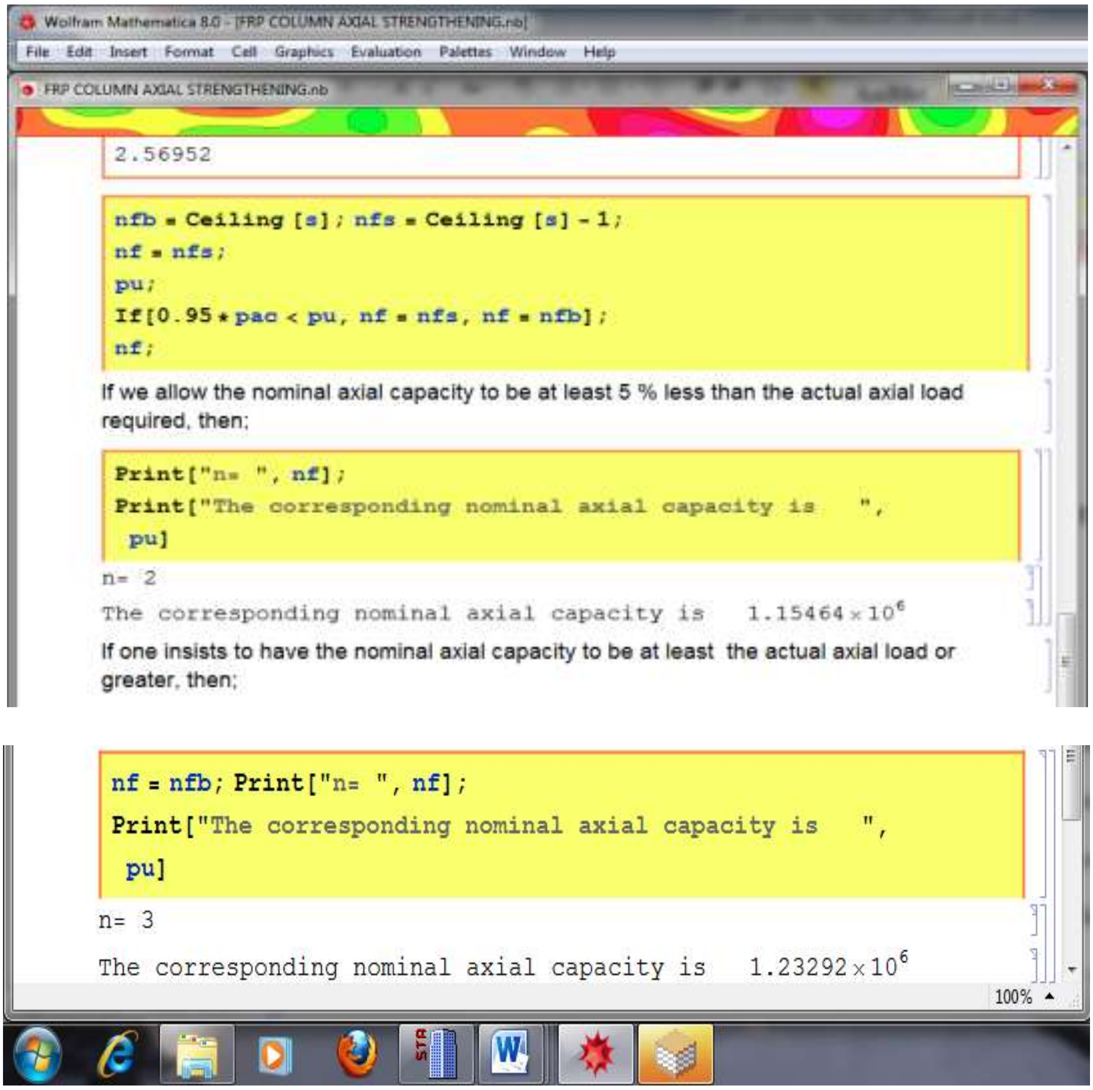
Note; it is important to know that 3 layers and thickness of 0.0065 was able to give the calculated nominal axial capacity.

5.9 FRP Coded Program for Column Axial Enhancement




Wolfram Mathematica 8.0 - [FRP COLUMN AXIAL STRENGTHENING.nb]
File Edit Insert Format Cell Graphics Evaluation Palettes Window Help
FRP COLUMN AXIAL STRENGTHENING.nb
Clear[nf]
• Design And Analysis
The diameter of the equivalent is
Dr = Sqrt[bc^2 + hc^2];
The design column diameter is
d = Max[Dr, d1]
The confinement fiber reinforcement ratio
rf = 4*nf*tf / d
The effective fracture strain of confining fiber is
efc = Min[0.004, 0.7*efu]
The confining pressure is
fcp = ef*efc*rf / 2
The compressive strength of concrete is


fcc = fc*(2.25*Sqrt[1 + 7.9*fcp/fc] - 2*fcp/fc - 1.25);
The cross - sectional area of concrete is
agc = Pi*d^2 / 4;
The gross cross - sectional area for rectangular column is
agr = bc*hc;
The gross, column design cross - sectio area is
ag = Max[agc, agr];
The nominal axial capacity of the column is
pu = 0.85*(0.85*fcc*wf*(ag - ast) + ast*fy);
Find the number of FRP sufficient to strengthening the column
s = Solve[pu == pac, nf]
{{nf → 2.56952}, {nf → 343.023}}
s = s[[1, 1, 2]]
100%



Figures 5.16 to 5.18: FRP Coded Program for Column Axial Strengthening

5.10 Discussion of Results

The result from the FRP program gave the same result as in reference calculation except for flexural strengthening. The width of FRP is 12.9 in and 12.66 in for reference and coded program respectively. The shear contributed by the FRP on the beam is 32.9 kips for both calculations while for column, the same number of FRP produced 1.233 kips of nominal capacity. However, the program is faster; could also find the required FRP layers to give the same nominal capacity in revised calculation and also makes checks for stresses on the materials used.

Chapter 6

CASE STUDIES

6.1 Introduction

In this chapter, there will be an investigation of two different building as case studies. In the first case study, one storey is added to the building. It is investigated to find out if the building will require strengthening because of the added load. In the second case study, the building is an old existing building. It is investigated to find out if it will require strengthening as per new building codes. The results will be applied on the decision selection tool to check its efficiency to choice selection of strengthening methods. The different strengthening method applied for strengthening in case II will be compared according to the economy and efficiency to the building.

6.2 Case Study I

The building is a three storey school building. It has four parts and is connected together at the expansion joints. One additional storey is added as detailed in Figure 6.1. Investigation is carried out with regards to the building information available.

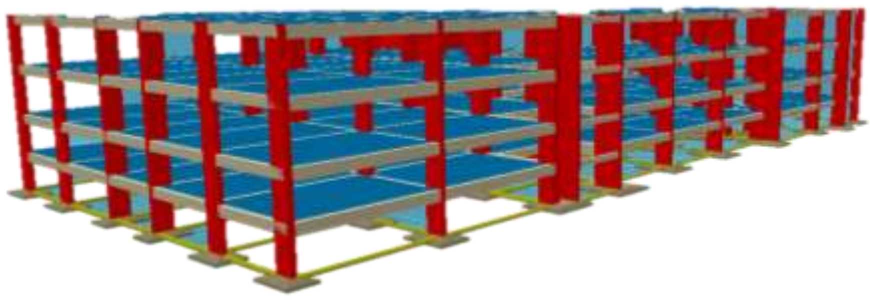


Figure 6.1 School building with extra floor (Sta-4Cad drawing model)

The building has open floor at the ground floor. The height of each storey differs and this causes mass irregularity. Figure 6.2 (a) shows the side views of the building while Figure 6.2 (b) shows the 3D displacement view of the building.

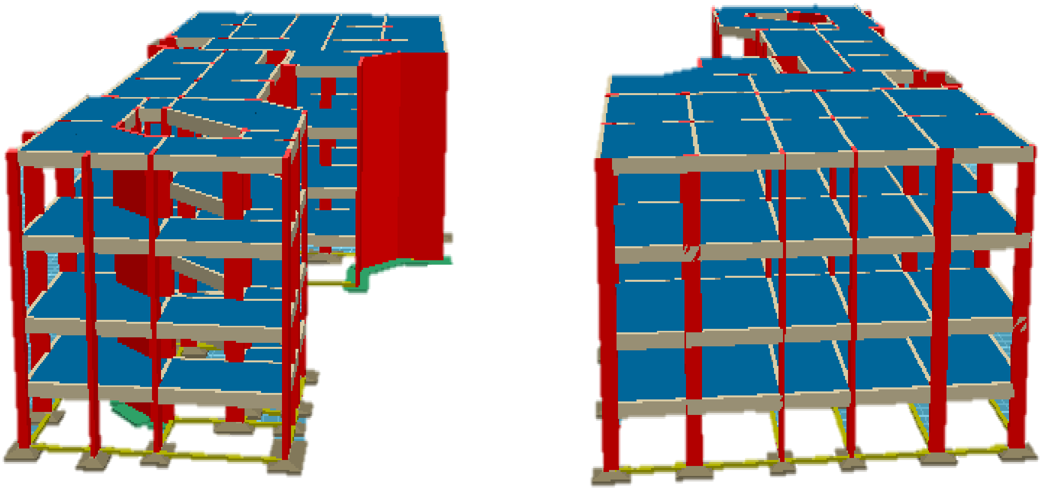


Figure 6.2 :(a) Side views of the four storey building for Case study I

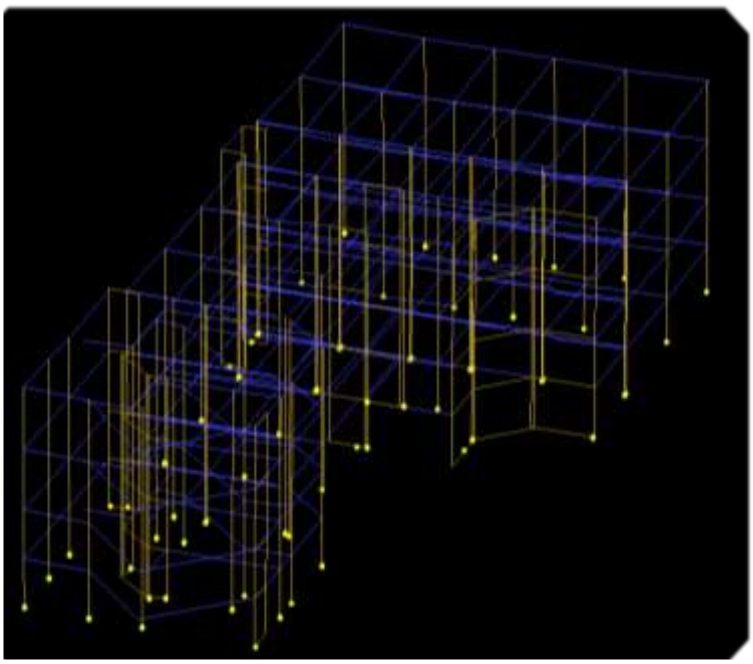


Figure 6.2 :(b) 3D displacement view for Case study I

Figure 6.3 shows the structural plan of the building for case study I. All the building floors have the same column and beam sizes, from the first floor to the last floor.

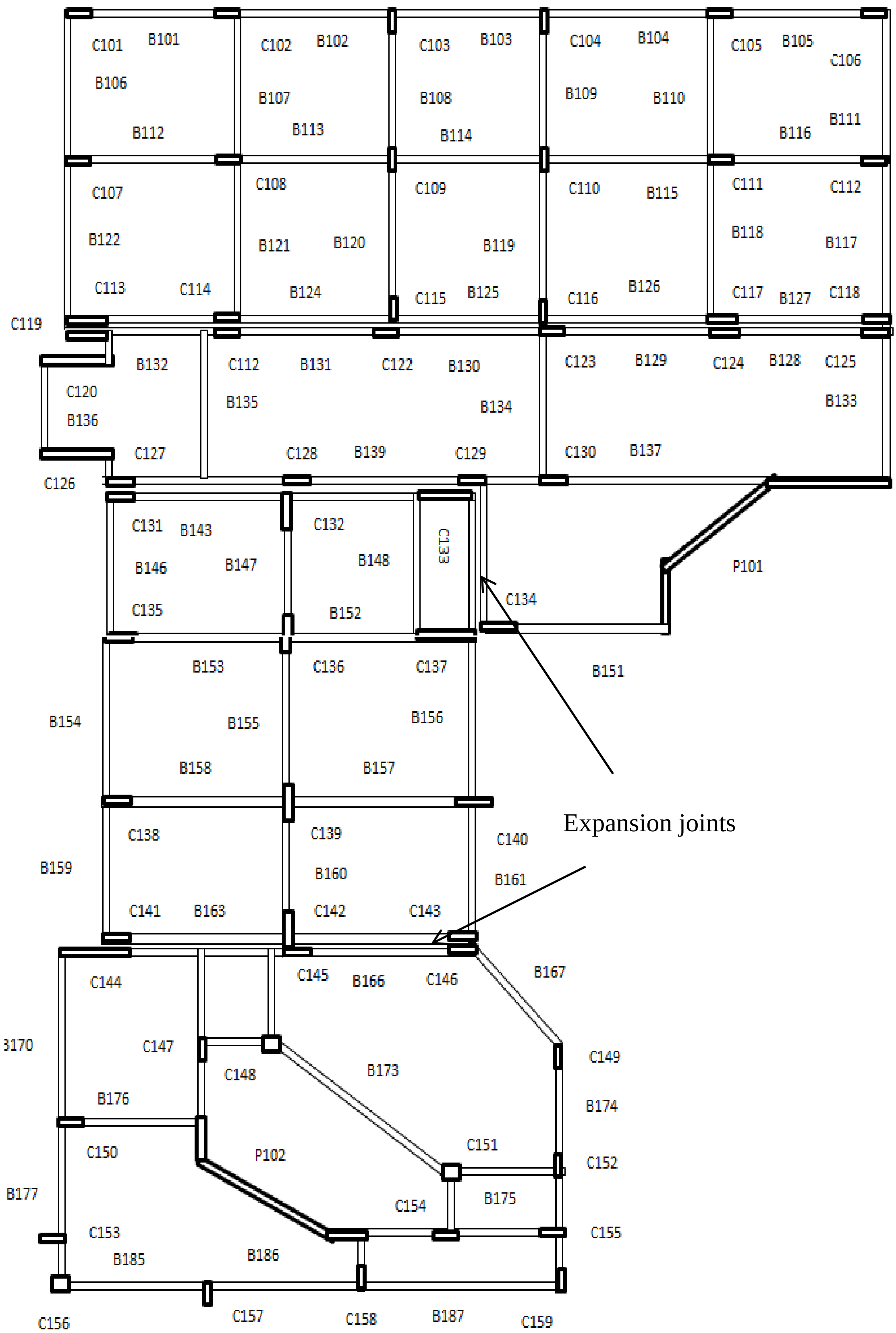


Figure 6.3: Structural plan of the building, Case study I

6.3 Building Information

Because of the availability of the building drawing prepared in 1997 which contains the quantity and properties of construction material, a preliminary investigation will be just enough to carry out the building assessment for capacity check for extra new load due to the addition of one extra floor. A visual inspection carried out has shown no change in the physical modification of the building as contained in the drawings. Table 6.1 details the general building information.

Table 6.1: General building information for case study I

Storey number	4	
Storey height	$14.8\ m$	$48.56\ ft$
Concrete strength	$225\ kg/cm^2$	3200.3 psi
Steel grade	$4200\ kg/cm^2$	59738.04 psi
Slab thickness	$17\ cm$	6.69 in
E modulus of steel	$213500\ kg/cm^2$	3036683.8 psi
Columns **Panels**	$60\ cm\ by\ 60\ cm, 100\ cm\ by\ 30\ cm,$ $120\ cm\ by\ 30\ cm, 30\ cm\ by\ 230\ cm, 30\ cm\ by\ 450\ cm.$	
Beams	$30\ cm\ by\ 70\ cm, 30\ cm\ by\ 80\ cm.$	
Foundation	$290\ cm\ by\ 325\ cm, 370\ cm\ by\ 210\ cm, 200\ cm\ by\ 200\ cm$	
Tie beams	$30\ cm\ by\ 20\ cm$	

The building has the same sizes of column and beams for all the floors.

Table 6.2: Reinforcements and stirrups for case study I

Name	**Stirrups**	**Reinforcements**		
Column	1Ø8/20-10	14Ø18	18Ø12	16Ø18
Beam	Ø8/15,Ø8/25	3Ø12 bent	2Ø12 bottom	3Ø12top bars

Table 6.2 presents the various reinforcements contained in the columns and beams and the stirrups. For columns, the reinforcement differs according to the sizes of the columns. This is shown by the differences in reinforcement area for column. The foundation type is single foundation. It is tied with foundation beams.

6.4 Building Systems Models

The building is modeled and designed with STA4-CAD structural design software to obtain the initial condition of the building. The analysis was performed with ACI code 2005 and for seismic performance with the 2007 Turkish Earthquake Code. The software utilizes load combination as detailed in Table 6.3 where G and Q are dead and live load respectively. Table 6.4 contains the building model parameters for the analysis.

Table 6.3: Loading combination utilized by the software for case I & II

Number	**Load explanation**	**Load cases**
1	(G + G + G + G)	1.2G(1) + 1.6Q(2)
2	(Q + Q + Q + Q)	1.2G(1) + 1.6Q(3)
3	(0 + Q + 0 + Q)	1.2G(1) + 1.6Q(4)
4	(Q + 0 + Q + 0)	1.2G(1) + 1.6Q(5)
5	(Q + Q +0 + Q)	1.2G(1) + 1.6Q(6)
6	(0+ Q + Q + 0)	1.2G(1) + 1.6Q(7)
7	(0 + 0 + Q + Q)	G(1) + Q(2) + E(9)
8	X-Seismic + 5%	G(1) + Q(2) – E(9)
9	X-Seismic – 5%	G(1) + Q(2) + E(10)
10	Y-Seismic + 5%	G(1) + Q(2) – E(10)
11	Y-Seismic – 5%	G(1) + Q(2) + E(11)

12	X-Seismic Capacity -	G(1) + Q(2) – E(11)
13	X-Seismic Capacity +	G(1) + Q(2) + E(12)
14	Y-Seismic Capacity -	G(1) + Q(2) – E(12)
15	Y-Seismic Capacity +	

Table 6.4: Building parameters for Case study I

Building importance factor: I	0.5
T_A T_B	0.15 s. 0.4 s.
Structural behavior factor: R	4
Seismic zone coefficient: A_0	0.3
Live load reduction factor: n	0.60
Assumed allowable soil pressure	30 t/m^2

6.5 Evaluation of Result

The same concrete strength, reinforcement steel grade and modulus of elasticity when initially constructed, were used to model the building as detailed in Table 6.1. Table 6.5 presents the performance result of the building before the addition of new loads and the performance result of the building after the addition of one extra floor where V_r is the shear capacity, x and y are the horizontal and vertical directions respectively.

Table 6.5: Capacity result before addition of new load

Storey number	**V_r (x-direction)**	**V_r (y-direction)**
Storey 1	1286.50 t	1083.96 t
Storey 2	1488.27 t	1197.21 t

Storey 3	1563.74 t	1170.01 t
Beam damage ratio	0.0 < 30%	
Building performance	Life safety	
Period	0.42 s.	

In each of the storey as seen in (Table 6.5) has the shear capacity for x-directions and y-directions for all storey levels. The general performance of the building is life safety. The result will be compared with the result found when there is additional floor as detailed in Table 6.6.

Table 6.6: Capacity result after addition of a new floor for case study I

Storey number	V_r (x-direction)	V_r (y-direction)
Storey 1	1172.81 t	1022.26 t
Storey 2	1299.99 t	1094.95 t
Storey 3	1342.41 t	1179.12 t
Storey 4	1563.73 t	1168.92 t
Beam damage ratio	2.2 < 30 %	
Building performance	Life safety	
Period	0.63 s.	

6.5.1 Discussion of the Result

Although the second analysis in Table 6.6 gave the same general building performance of life safety with the first analysis in Table 6.5, however, there is a beam percentage damage of 2.2 in the four storey building performance. There is also shear capacity reduction on the floors mainly on the x-direction. Four beams; beam 145, beam 245, beam 345 and beam 445 became brittle and will need strengthening to restore to ductility level. Also, storey drift check is not satisfactory for storey two, storey three and storey four. However, all these problems found will be examined and applied on the decision selection tool.

6.5.2 Evaluation with Strengthening Option Selection Tool

The aim of this analysis with the above mentioned tool is to obtain the best strengthening method from the list of methods contained in the selection tool, according to the problem requirement of the building. Figure 6.4 presents the evaluation of the school building. It can be observed in Figure 6.4 that, some parameters were selected 'yes' while some were 'no'. Because of the high percentage decrease in shear capacity along x-axis for each storey floors, global strategy is answered 'yes'. Figure 6.5 contains the final result of evaluation of the choice selection tool. In the relevance count, shear wall has the highest value among all other strengthening method contained in the tool. Therefore, the choice selection tool has selected shear wall as the best option according to the problem requirement of the school building.

Nevertheless, the availability of the materials and the building use disturbance for each method is also considered. This is paramount when considering the locations of the project and the period of carrying out the strengthening. Application of shear wall during summer holidays will add no disturbance to the building use during the period of strengthening the building.

However, the result as per shear wall will be used to strengthen the building and with other strengthening method and will be compared to see if the best selected method will be the same as the strengthening choice decision selection tool.

Seismicity	
Non seismic	No
Low seismic	No
Seismic	Yes
Strengthening strategy	
Global strategy	Yes
Local strategy	
Column	
low dimensions	No
degraded column	No
missing transverse rebars	No
low concrete grade	No
short column	No
strong beam/weak column	No
corroded internal rebar	No
Joints	
missing transverse rebars	No
Beam	
low concrete grade	No
low dimensions	No
missing transverse rebars	No
over stressed beam	No
Slab	
low thickness	No
inadequate rebar	No
long span	No
Foundation	
low dimensions	No
weak soil	No

Figure 6.4: Selective criteria used for the school building

Building irregularity	
Regular	Yes
Irregular	
Unsymmetric	
projecting elements	No
open floor	Yes
Floating column	No
plaza type building	No
mass irregularity	Yes

Strengthening options	Shear walls	Steel bracing	Concrete jacketing	FRP jacket	Steel plating	Steel jacketing	FRP wrap	FRP / steel bracing	FRP / steel jacket	span shortening	FRP strip	soil improvement	Shear wall/ column jacketing
Relevance	1	0.7	0.67	0.33	0.33	0.33	0.33	0.67	0.67	0.33	0.33	0	0.33
Availability	Yes	Yes	Yes	Yes	No	No	Yes	No	No	Yes	Yes	Yes	Yes
Building use disturbance	No	No	Yes	No	Yes	Yes	No	Yes	No	Yes	No	No	No
Proposition	1	0.67	0	0.33	0	0	0.33	0	0	0	0.33	0	0.33
Strengthening Choices	Shear walls	Steel bracing	Concrete jacketing	FRP jacket	Steel plating	Steel jacketing	FRP wrap	FRP / steel bracing	FRP / steel jacket	span shortening	FRP strip	soil improvement	Shear wall/ column jacketing

Figure 6.5: Result of choice decision selection tool

6.5.3 Strengthening with Shear Wall for Case study I

The structural software STA-4CAD is utilized to strengthen the building by applying shear walls of 7.87 in (20 cm) at the corners of the frame. Figure 6.6 shows the plan of the building (same for all floors) and placement of shear walls at the corners and along x-axis of the building. The lighter double lines signify beams, the tick lines are the columns and panels while the shear walls are in yellow colour. Figure 6.7 shows the shear wall placement in 3D view.

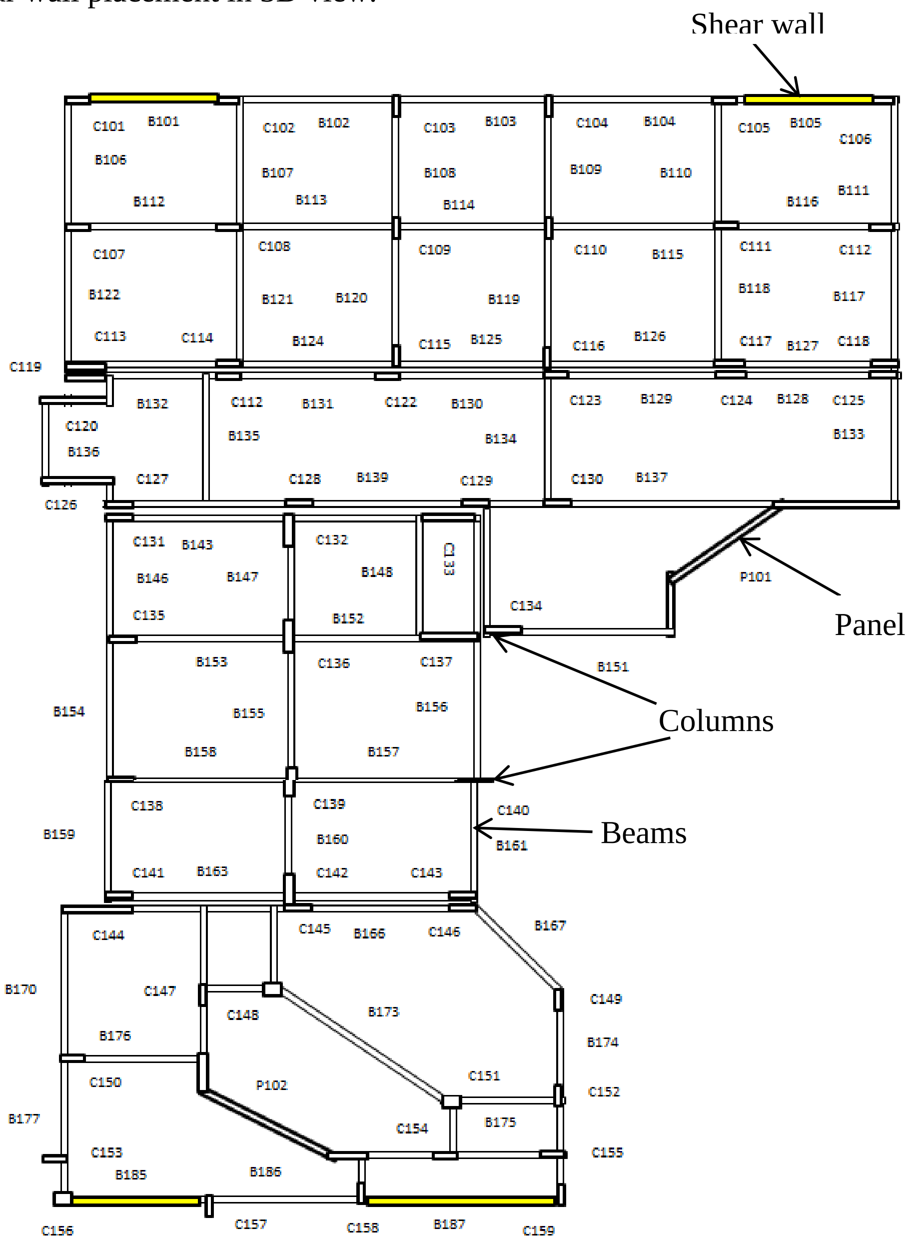


Figure 6.6: Shear wall placement in the plan of the building for Case study I

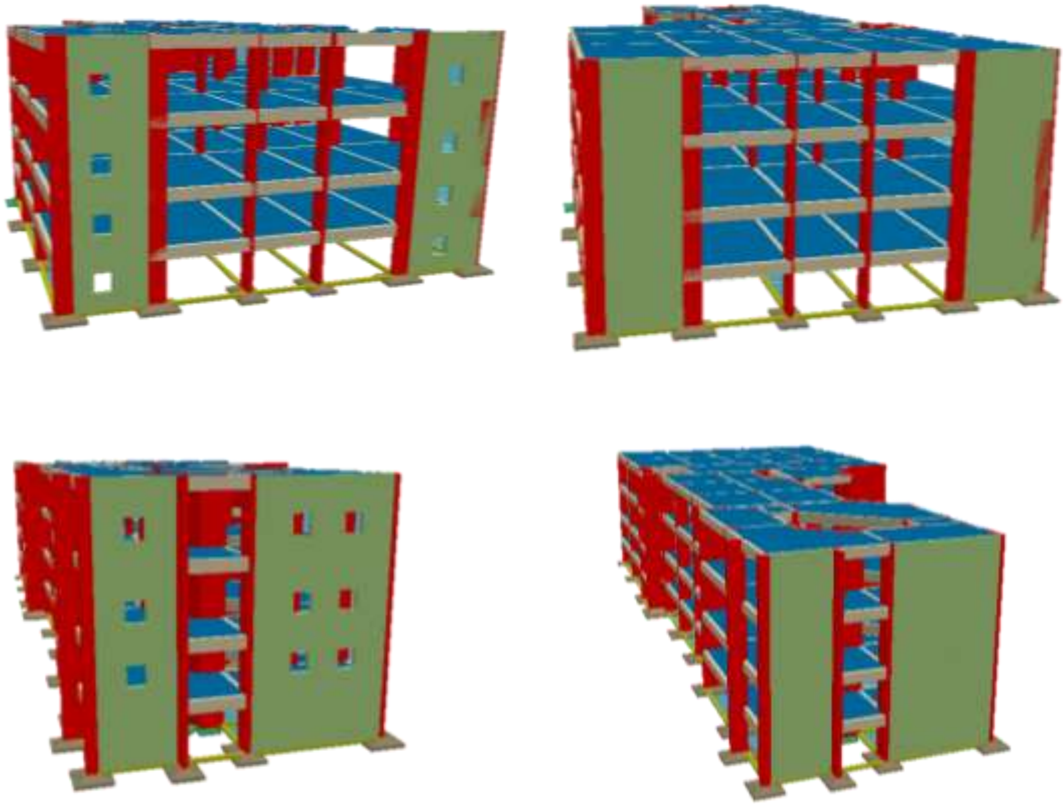


Figure 6.7: (a) Shear walls with openings (b) shear walls with no openings

Table 6.7 Storey shear capacities for shear walls with opening and no opening

Storey no	**Shear wall with openings**		**Shear walls with no openings**	
#	**X-direction**	**Y-direction**	**X-direction**	**Y-direction**
1	2172.34 t	1391.09 t	2305.72 t	1096.38 t
2	2633.37 t	1225.05 t	2787.24 t	1193.73 t
3	3076.84 t	1208.64 t	3316.50 t	1212.55 t
4	4293.01 t	1107.81 t	4683.98 t	1387.15 t

Shear walls with opening and without opening are shown in Figure 6.7 and is compared to strengthen the school building. Table 6.7 details the improved shear capacity for each story level. The aim of shear wall opening and without opening strengthening applications study is to compare their efficiency to improving the capacity of the building. These openings are for windows and doors. The result obtained (Table 6.7) shows that, shear walls without openings improved the performance more than shear walls with openings. Both shear walls were able to solve the beam damage percent of two completely and the problems of storey drift as well. However, the shear wall with opening were more efficient at y-direction for storey one and two while at upper stories, shear wall without opening were more efficient both in x-direction and y-direction (Table 6.7).

6.5.4 Section Enlargement for Case Study I

Beams (B145, B245, B345, and B445) which required strengthening were strengthened by concrete jacketing. This was modeled and designed with structural software STA-4CAD. The lengths of the beam are 1.2 meters for all the four beams. The moment capacity of each beam need to be improved and each beam needs to be restored to ductility level. Iteratively, the capacity of the beam was improved when the additional concrete is 3.94 in (10 cm) that is 1.97 in (5 cm) for each sides of the beam (Figure 6.8).

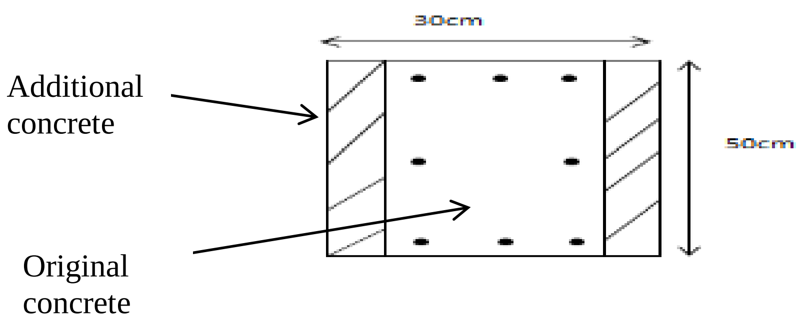


Figure 6.8: Section Enlargement of beam 145,245,345,445

However the application of this method could not solve the beam damage percent. Although it was able to restore the failure mode of the beam from brittle mode to ductile mode but other problems of the building such as the storey drift could not be solved by this method.

6.5.5 Strengthening by Steel Plating

Steel plate is also considered and used to strengthening the beams which requires strengthening. Engissol structural software (Figure 6.9) is used to model and design the beams for strengthening. The parameters used in modeling are as detailed in Table 6.8.

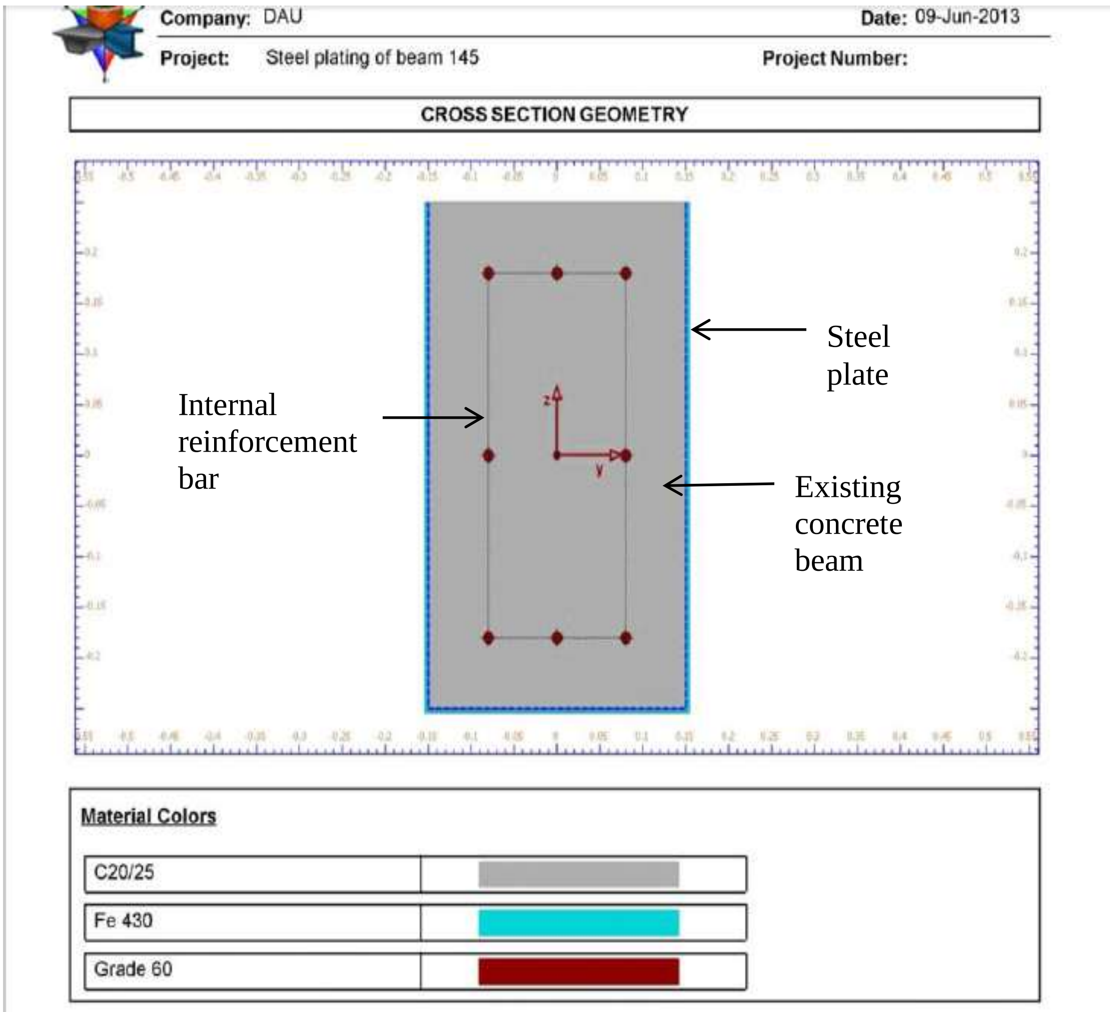


Figure 6.9: B145 Modeling with Engissol design structural software

Table 6.8: Parameters for beam plate design using Engissol

Name of the material	(Fe 430 & Fe510) Hot rolled steel
Elasticity modulus	210 GPa
Maximum strain	0.2
Minimum strain	-0.2
Minimum allowable reinforcement ratio	0.01 (default as per code)
Maximum allowable reinforcement ratio	0.08 (default as per code)
Partial safety factor	1.15
Phi- Tension controlled	0.9
Phi-Compression controlled	0.65

The plates are applied to the most critical side of the beams. European steel hot rolled with hardening (Fe430) is used for this plating. By continues iterations for the thickness of the steel plate, it was found that 5 mm was sufficient to increase the capacity to these beams.

Practically, the two plated side may not be important. Its work is to help anchoring the tension plates and to positioning it well on the beams. The height may not necessary be as high as seen in the software from economic consideration.

6.5.6 Strengthening the Beam with FRP for Case study I

The beam's performance level is immediate occupancy, but they are brittle and need to be improved. FRP is good for increasing the ductility of structural members. The number and thickness of FRP is found by flexural strengthening since the beams are satisfactory in shear check from the analysis result in the STA-4CAD structural design software. However, the FRP flexural strengthening coded program prepared is used to calculate the required number and thickness of the FRP needed. The modulus of elasticity and fracture strain of the FRP is 33000000 psi and 0.017 respectively. The values of the thickness are 0.2 mm at the compression zone, 0.58, 0.68, and 0.50 mm (0.008 in, 0.023 in, 0.027 in, and 0.020 in) at tension zone for B145, B245, B345, and B445 respectively as detailed in Figure 6.10 (a) and (b).

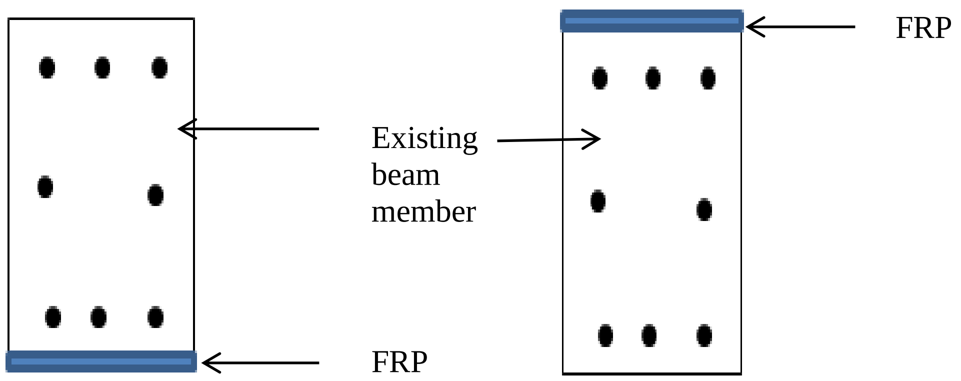


Figure 6.10: (a) FRP tension zone plating (b) FRP compression zone plating

However, shear wall strengthening from every point of view is the best strengthening method in terms of efficiency to enhance the capacity of the school building studied. FRP, steel plates or concrete jacket applied, could only improve the member capacity, but not general performance of the building. Therefore, using the local strengthening methods for this purpose will not be appropriate and will be costly if it is the only option to use for the case study I.

6.6 Second Case Study II

The building is a four storey old existing building with a basement floor as detailed in Figure 6.11. It is a residential apartment and was designed and constructed 1970 with no consideration of Earthquake codes design. Figure 6.12 details the 3D displacement view of the frame structure given in Figure 6.11.

Figure 6.11: Four storey frame structure

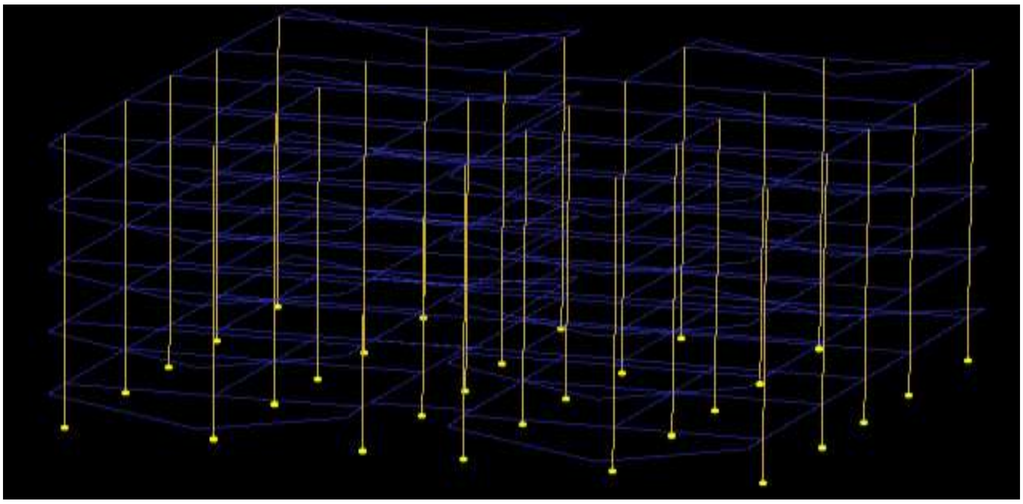


Figure 6.12: 3D displacement view for Case study II

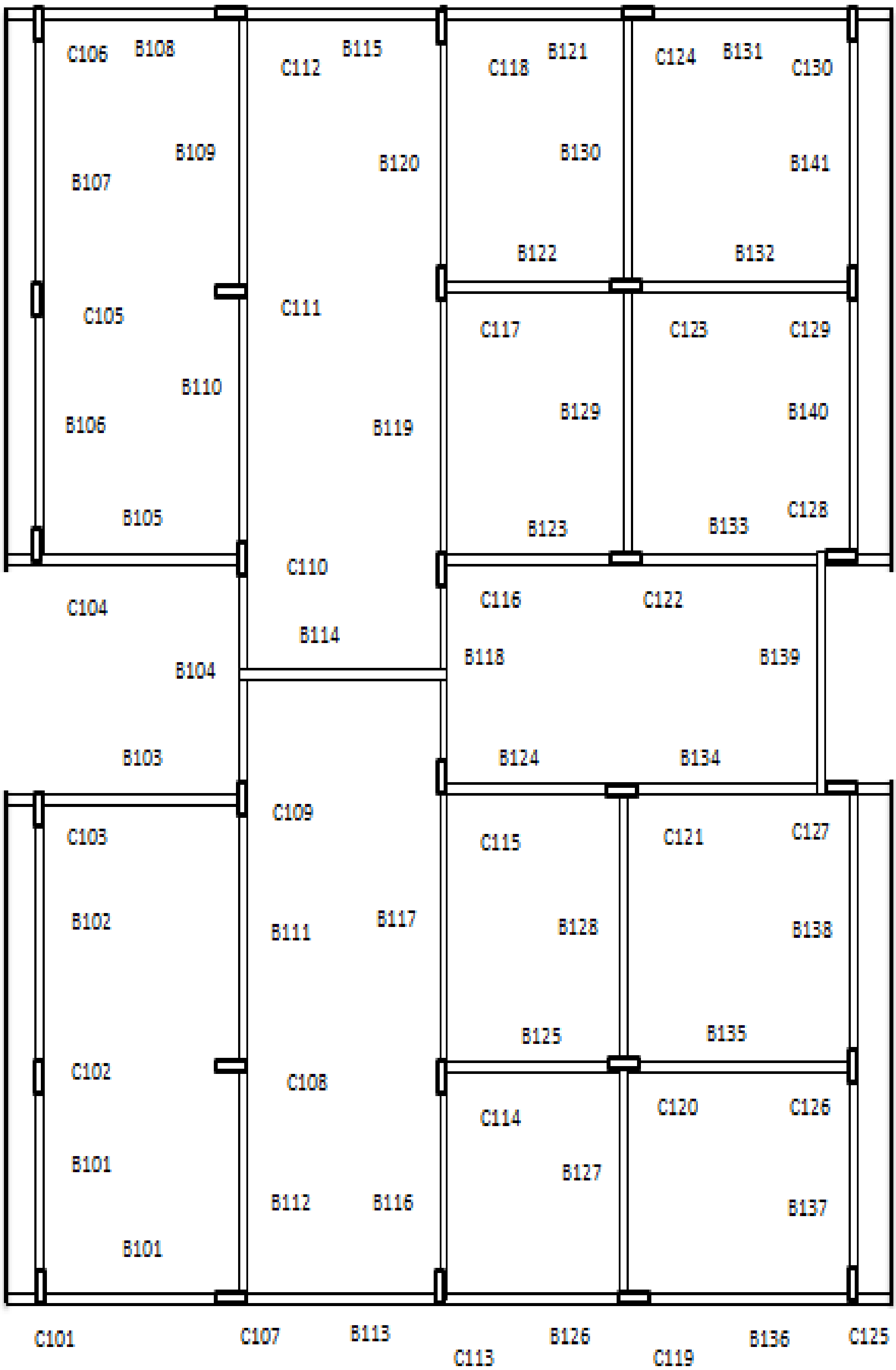


Figure 6.13: Building plan for floor 1, 2, 3, 4 for Case study II

Figure 6.13 details the structural plan of the building. It is the same for every storey of the building.

6.6.1 Building Information

Because of the availability of the building drawing prepared in 1970 which contains the quantity and properties of construction material, a preliminary investigation will be just enough to carry out the building assessment for capacity check and performance of the building. A visual inspection carried out has shown no change in the physical modification of the building as contained in the drawings. Table 6.9 details the general building information while Table 6.10 details the reinforcements of beams and columns.

Table 6.9: General building information for Case study II

Storey number	4	
Storey height	$14.5\ m$	$47.57\ ft$
Concrete strength	$160\ kg/cm^2$	2275.68 psi
Steel grade	$2200\ kg/cm^2$	31290.60 psi
Slab thickness	$20\ cm$	7.87 in
E-modulus	$275323\ kg/cm^2$	3915919.03 psi
Beams	20 cm by 50 cm	
Column type-A		
Basement floor	20 cm by 60 cm	
Floors 1, 2, 3, 4	20 cm by 50 cm	
Foundation size	195 cm by 195 cm	
Column type-B		
Basement floor	20 cm by 50 cm	

Floor 1, 2, 3, 4	20 cm by 40 cm
Foundation size	160 cm by 160 cm
Tie beam size	20 cm by 40 cm

Table 6.10: Reinforcements and stirrups for Case study II

Name	**Column A-type**	**Column B-type**	**Stirrup**
Basement floor	6Ø18	6Ø16	Ø6/17
Floor 1	6Ø16	6Ø14	
Floor 2, 3, 4	6Ø14	6Ø14	
Name	**Beam**		
Top bar	2Ø12		
Bottom bar	4Ø16		

6.6.2 Building System Models

The building is model and design with STA4-CAD structural design software to find the initial condition of the building. The analysis was performed with ACI code 2005 and for seismic performance with the 2007 Turkish Earthquake Code. The software utilizes load combination as in first case study, detailed in Table 6.3 where G and Q are dead and live load respectively. Table 6.11 details the building model parameters for the analysis.

Table 6.11: Building parameter models of Case study II

Building importance factor: I	1.0
$\boldsymbol{T_A}$	0.15 s.
$\boldsymbol{T_B}$	0.4 s.
Structural behavior factor: R	4
Seismic zone coefficient: $\boldsymbol{A_0}$	0.3

Live load reduction factor: n	0.60
Assumed allowable soil pressure	20 t/m^2

6.6.3 Evaluation of Result

The same concrete strength, reinforcement steel grade and modulus of elasticity when initially constructed were used to design and model the building. Table 6.12 presents the performance result of the building.

Table 6.12: Shear capacity result for the performance analysis for Case study II

Storey number	V_r (x-direction)	V_r (y-direction)
Basement floor	123.04 t	81.13 t
Floor 1	112.59 t	74.56 t
Floor 2	105.26 t	71.63 t
Floor 3	89.83 t	66.41 t
Floor 4	72.21 t	55.01 t
Beam damage ratio	86.4 %	
Column damage ratio	54.1%	
Building performance	Collapse case	
Period	0.82 s	

6.6.4 Discussion of the Result

Table 6.13 details the performance of the building. The general performance of the building is collapse case with beam collapse prevention damage ratio of 86.4% and column with plastic hinge V_c ratio of 54.1% which were greater than 20% and 30% respectively for maximum damage limit for building standards for earthquake code utilized by STA-4CAD. Hence, the damage capacities were very high, and the general performance is low. However, a great number of columns were weak, there were beams with low moment resistance at the joints, and the stiffness of the building is at stake. However, all these problems found will be examined and applied on the decision selection tool.

6.6.5 Evaluation with Strengthening Option Selection Tool

The aim of this analysis with the above mentioned tool is to obtain the best strengthening method from the list of methods contained in the selection tool according to the problem requirement of the building. Figure 6.14 presents the evaluation of the residential apartment. As observed in Figure 6.14, some parameters were selected ‘yes’ while some were ‘no’. Because of the high percentage damage in building members, global strategy is answered ‘yes’. Figure 6.15 details the final result of evaluation of the choice selection tool. In the relevance count, steel bracing has the highest value among all other strengthening method contained here while shear wall is the second with FRP and steel jacketing as a hybrid method. Therefore, the choice selection tool has selected these methods as the best option according to the problem requirement of the building. Nevertheless, the availability of the materials and the building use disturbance for each method is also considered. This is paramount when considering the locations of the project and the period of carrying out the strengthening work. However, during the strengthening material market survey, materials for use of shear wall strengthening is readily obtainable compared to steel bracing materials, use of steels for jacketing and FRP materials. However, shear wall will be considered more economical. Therefore, these methods will be applied and compare to see if the best method will be the same as the strengthening choice decision selection tool, considering the economy of strengthening materials.

Seismicity	
Non seismic	No
Low seismic	No
Seismic	Yes
Strengthening strategy	
Global strategy	Yes
Local strategy	
Column	
low dimensions	No
degraded column	No
missing transverse rebars	No
low concrete grade	Yes
short column	No
strong beam/weak column	Yes
corroded internal rebar	No
Joints	
missing transverse rebars	No
Beam	
low concrete grade	Yes
low dimensions	No
missing transverse rebars	No
over stressed beam	No
Slab	
low thickness	No
inadequate rebar	No
long span	No
Foundation	
low dimensions	No
weak soil	No

Figure 6.14: Evaluation answers from decision selection tool for Case study II

Building irregularity	
Regular	Yes
Irregular	
Unsymmetric	
Projecting elements	No
Open floor	No
Floating/staggered column	No
plaza type building	No
Mass irregularity	No

Strengthening options	Shear walls	Steel bracing	Concrete jacketing	FRP jacket	Steel plating	Steel jacketing	FRP wrap	FRP / steel bracing	FRP / steel jacket	span shortening	FRP strip	soil improvement	Shear wall/ column jacketing
Relevance	0.5	0.67	0.3	0.33	0.2	0.3	0.2	0.5	0.5	0.2	0.2	0	0.3
Availability	Yes	No	Yes	Yes	No	No	Yes	No	No	Yes	Yes	Yes	Yes
Building use disturbance	No	No	No	No	Yes	Yes	No	Yes	No	Yes	No	No	No
Proposition	0.5	0	0.3	0.3	0	0	0.2	0	0	0	0.2	0	0.3
Strengthening Choices	Shear walls	Steel bracing	Concrete jacketing	FRP jacket	Steel plating	Steel jacketing	FRP wrap	FRP / steel bracing	FRP / steel jacket	span shortening	FRP strip	soil improvement	Shear wall/ column jacketing

Figure 6.15 Result of the evaluation for Case study II

6.7 Local Method

The different strengthening methods whose target is to increase the building member capacity will be applied to strengthen the building apartment. The economic evaluations and cost of materials for these strengthening methods will be considered in the following pages. Such strengthening methods are the concrete jacket, FRP wrap on columns and plating on beams, and steel plating.

6.7.1 Concrete Jacket

The concrete jacket was used to strengthen the columns starting from the collapse case performance. It was observed that while adding concrete jacket on the columns, other columns with better performance tend to be decreased in their performance. Hence, they will require strengthening and this will increase the number of columns to be strengthened. The size of the jackets is 100 mm concrete cover with reinforcements of 14 mm diameter. The shape of the jacket is L shape and U shape as detailed in Figure 6.16. All the jacket load option is passive strengthening. Table 6.13 details the performance result after column jackting.

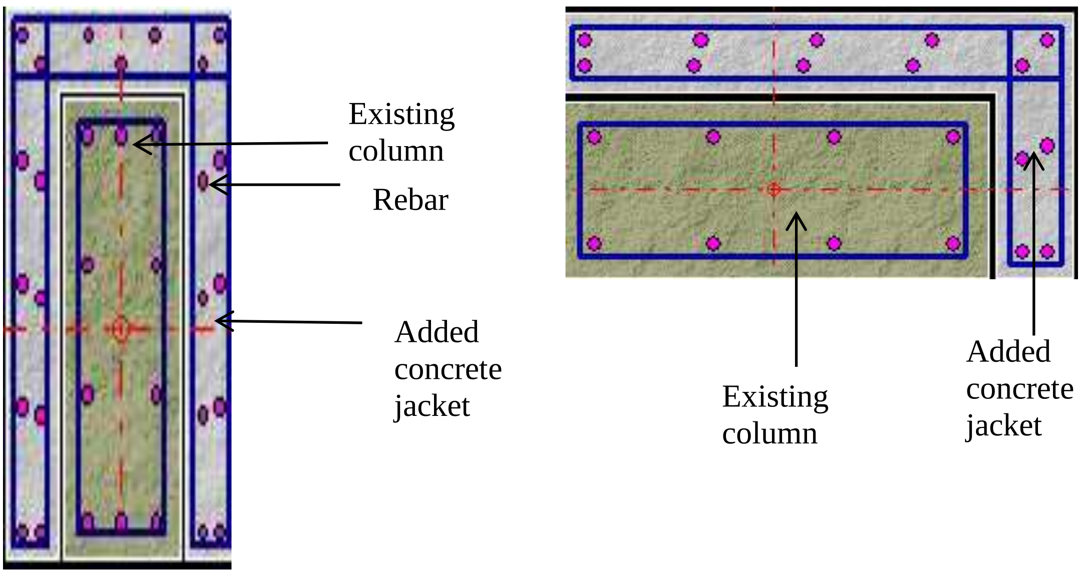


Figure 6.16: (a) U shape concrete jacket (b) L shape concrete jacket

Table 6.13: Performance result with Concrete Jacket for Case study II

Storey number	V_r **(x-direction)**	V_r **(y-direction)**
Basement floor	163.28 t	110.00 t
Storey 1	187.86 t	140.86 t
Storey 2	195.98 t	151.06 t
Storey 3	205.28 t	170.88 t
Storey 4	216.44 t	211.46 t
Beam damage ratio	82.6 %	
Column damage ratio	14.2 %	
Building performance	Collapse case	
Period	0.76 s	

After the application of concrete jacket using STA-4CAD structural software, the result obtained and detailed in Table 6.13 shows that concrete jacket method is not sufficient enough to eliminate the building damage percentage. Although it improved the performance of some columns individually, it improved general column performance from 54.1 % to 14.2 %. On further application of the jacket to eliminate the remaining damages, the members tends to become brittle, hence, the procedure was terminated. The cost of the material utilized will be provided in the following pages.

6.7.2 FRP Application for Case study II

With the application of FRP to building for strengthening purpose, it improves the ductility of building members to a better standard. FRP composites is used to strengthen the beams and the columns. The properties of the fiber material are contained in Table 6.14. The coded FRP programs prepared for beams and column is utilized to calculate the number of FRP and thickness required by the beams and the columns.

Table 6.14: Typical Properties of FRP

Weight of FRP SikaWra	$920\ g/m^2$	
Tensile strength of fiber	$2250\ N/mm^2$	326334.91 psi
E– modulus of fiber	$70000\ N/mm^2$	10152641.64 psi
Sika CarboDur Plates		
Tensile strength of fiber	$2400\ N/mm^2$	348090.57 psi
E – modulus of Fiber	$210000\ N/mm^2$	30457924.92 psi

The thickness obtained is average of 0.25 mm for column and beams with varying number of FRP layers ranging from 4 to 7 for columns. These layers were applied as a complete wrap on the columns. Before the application of the FRP, the surfaces of the columns is treated and spread with epoxy before wrap with the FRP. This takes care of the axial and shear strength enhancement of the columns. The quantity of materials and cost effectiveness will be treated in the next chapter.

6.7.3 Steel Plating Strengthening

Steel plate is also applied as a next option to strengthen the building members. The column is modeled and strengthened with CSI col structural software (Figure 6.16). The material used is hot rolled steel, f_y is 275 N/mm^2 while the E-modulus is 2×10^8. Iteratively the thickness of steel found ranges from 2 mm to 3 mm The code used is ACI 318 02. The four corners are rounded to radius of 4 mm as detailed in Figure 6.18. Before the application of the plate; the columns were treated with epoxy. The plate is best applied over the entire surface to account for both flexural and axial strength. This is also applicable to beams strengthening with steel plate. For only axial enhancement, spaces are allowed at intervals, and the corners are welded together.

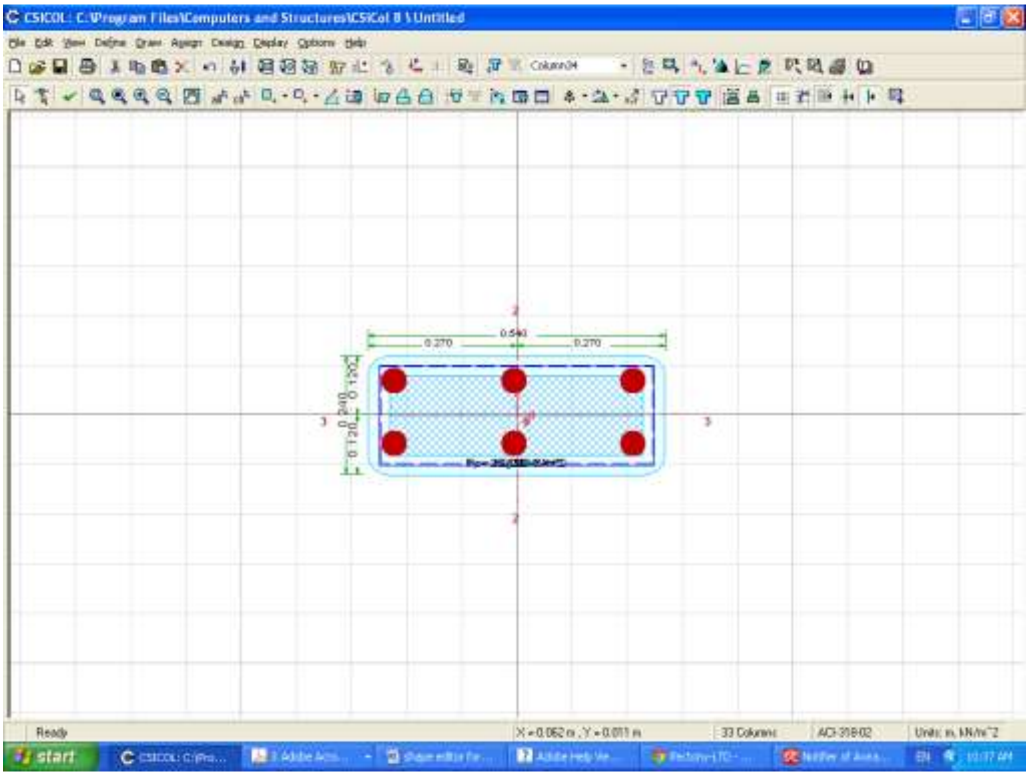


Figure 6.17: Modeled column with steel plate in CSI col design software

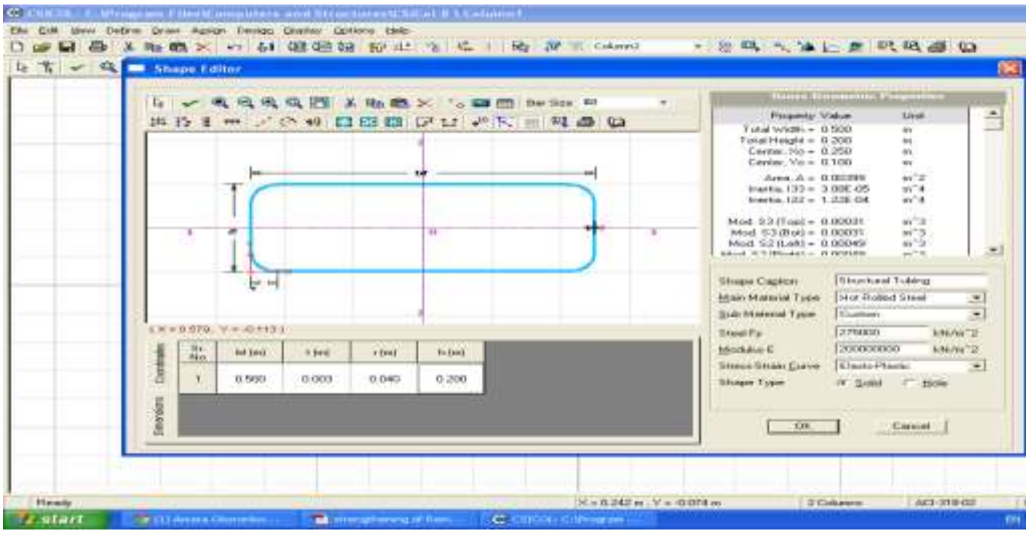


Figure 6.18: Shape curvature of the steel plate using CSI col software

6.3.3 Global method Application for Case study II

Among the global strengthening strategies, shear wall is used to strengthen the building. Although, steel bracing will offer efficient strength to this building at global level with little or no foundation strengthening as recommended by the strengthening selection tool, but because materials for application of shear wall is most available at this time and equally the consideration of the aesthetics of the building is very important, therefore, the use of shear wall strengthening was selected among the global methods to strengthen this building.

However, before the application of the shear wall, the architectural plan is first studied to ensure that after its application, the user will not find it difficult to adapt considering its formal shape and openings available on the walls and the partitions as well. Also, the center of mass and the rigidity of the building are important factors considered before the placement of shear wall as detailed in Figure 6.19. This will help to check torsional effect and ensure that the rigidity and mass centers are maintained closely to each other. If it is not properly placed, it will be difficult to achieve a good building performance. Even though it may be achieved, majority of the building members will become brittle, especially the columns of the ground floor. This is not desired because at sudden impact of external load such as wind or earthquake, the columns will yield and consequently will collapse.

Figure 6.19 shows the structural plan of the building in case study II. The building is symmetrical and with the center of rigidity and mass close to each other at the center of building.

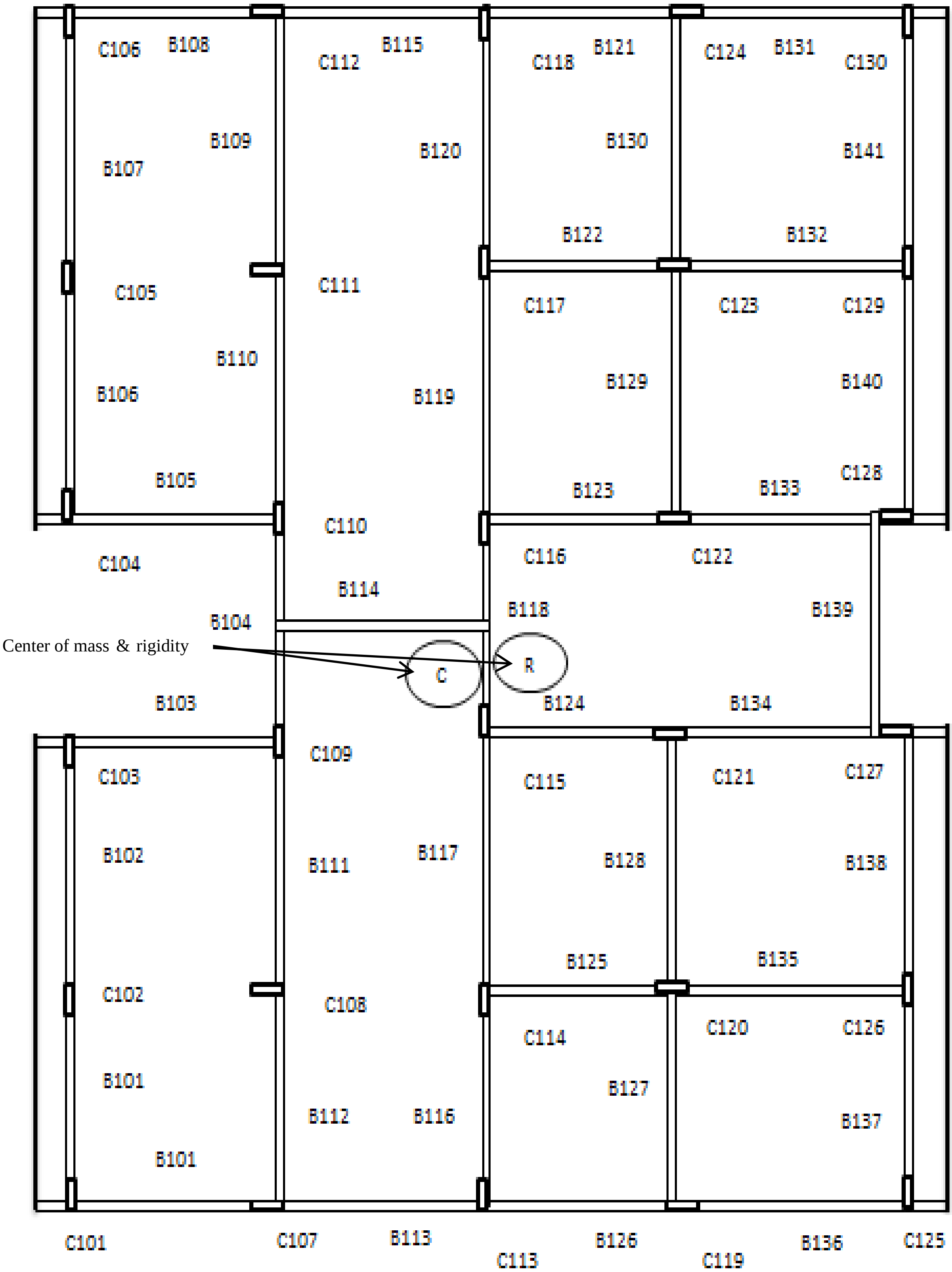


Figure 6.19: Structural plan and center of mass and rigidity

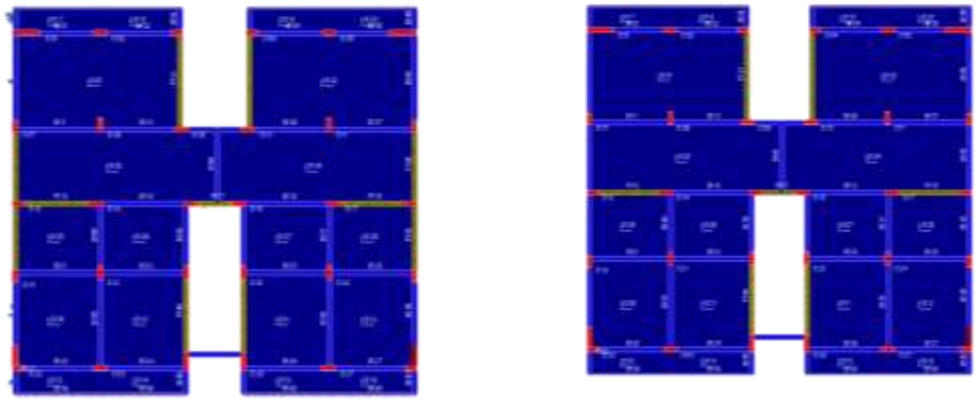


Figure 6.20: (a) Shear walls arrangement (b) Shear walls arrangement

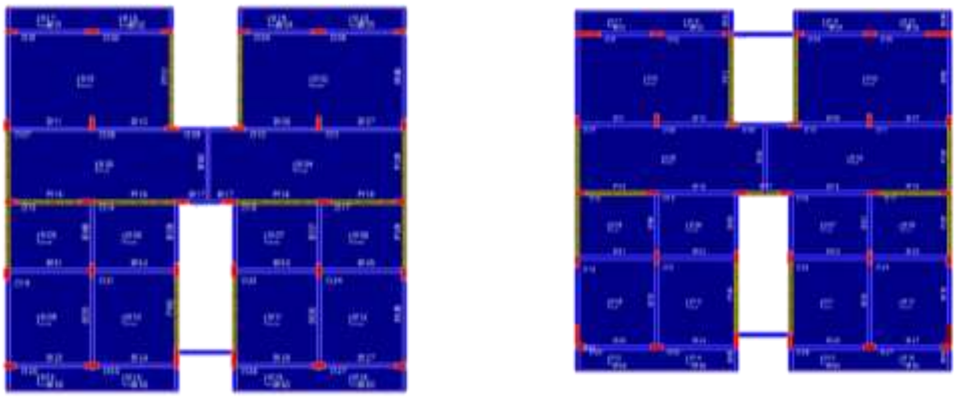


Figure 6.20: (c) Shear walls arrangement (d) Shear walls arrangement

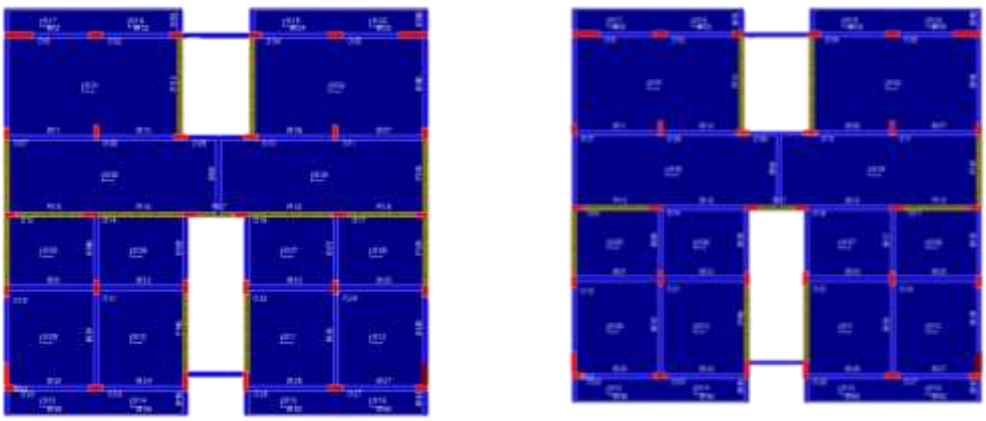


Figure 6.20 (e) Shear walls arrangement (f) Shear walls arrangement

Figures 6.20 a-f shows the arrangement of the shear walls on the structural plan. In Figure 6.20(a) the shear walls applied is not sufficient to improve the performance of the building, architectural plan is considered. There are also additional columns at the corners to help stiffen the structure. In Figure 6.20(b), the configuration of shear wall is not sufficient to improve the capacity of the building. The beams percentage damage is high. In Figure 6.20(c), the performance is life safety. The shear walls have openings like doors and windows according to the architectural plan. In Figure 6.20(d), the performance is collapse prevention with beam damage ration of 42.9%, column damage ration of 0.1% and roof Storey V_c ratio of 0.1%. In Figure 6.20(e), the application of the shear walls gave immediate occupancy for the building performance. Though, the shear walls were applied without architectural design plan, and therefore the doors and windows were not considered. In Figure 6.20(f), doors and windows were not considered too. They previous positions of doors were eliminated. However, the performance result for the shear walls in Figures 6.20 (a) to 6.20 (f) are detailed in Table 6.15.

Table 6.15: Performance of the shear walls applied on the building Case study II

Figure 6.20	**Beams % damage**	**Column % damage**	**Performance**	**Period**	**Roof damage V_c ratio**
a	20 %	0 %	CP	0.35 s	none
b	45 %	0.1 %	CP	0.58 s	0.1 %
c	5.3 %	0 %	LS	0.23 s	none
d	14.3 %	0.1 %	CP	0.35 s	none
e	0 %	0 %	IO	0.23 s	none
f	28.6 %	0 %	CP	0.37 s	none

However, the application of shear walls in Figure 6.20 (c) and Figure 6.20 (e) produced a better performance globally. In Figure 6.20 (c), the architectural plan was considered, the positions of the windows and the doors were considered while applying strengthening as detailed in Figure 6.21. Although its performance is life safety, it is preferred to Figure 6.20 (e) with immediate occupancy because the architectural plan was not considered (Figure 6.22). Hence, shear wall pattern in Figure 6.21 was considered for the strengthening, and then the four columns at the corners will be enlarged to avoid failure from the corners of the building.

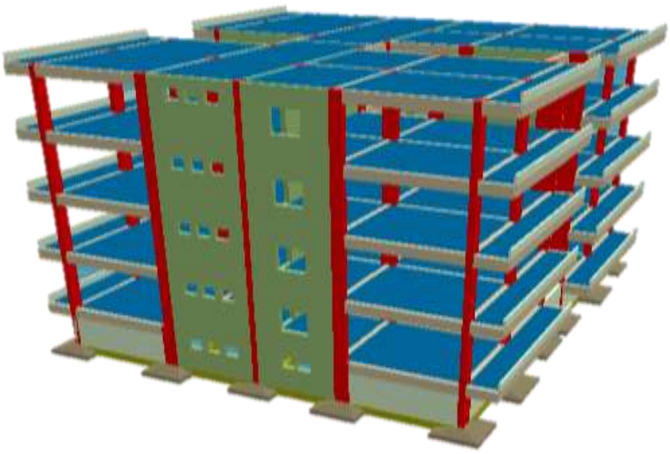


Figure 6.21: Frame structure strengthened with shear wall Figure 6.20 (c)

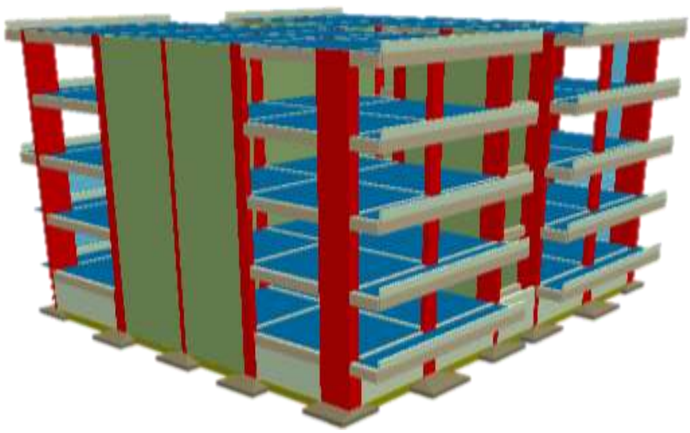


Figure 6.22: Shear wall arrangement for Figure 6.20 (e)

Chapter 7

ECONOMIC EVALUTION OF DIFFERENT STRENGTHENING METHODS

7.1 Introduction

The choice of selection of strengthening methods can be deployed considering the total cost of a particular strengthening scheme or technical feasibility. The costs of strengthening methods are site specific depending on the working conditions, site locations and access including technical specification. In this chapter, the cost of materials for different strengthening methods will be discussed. However, only the approximate direct material cost comparison is carried out. For better comparison, Case Study II will be investigated.

7.2 Strengthening with FRP

With FRP coded programs, the number of FRP and thickness is obtained as in case study II for the beams and columns discussed in the previous chapter. Table 7.1 shows the number and thickness of FRP obtained for columns strengthened by axial confinement with FRP wrap. The composite fabric used is Sika Wrap (Hex-100G) with a weight of 920 g/m^2 and tensile strength of 2250 N/mm^2. The epoxy adhesive used was Sikadur 330 of flexural modulus 70000 N/mm^2. The adhesive strength on concrete is greater than 4 N/mm^2 at concrete failure.

Table 7.1: FRP Column Design for Case Study II

Column	a (m)	b (m)	Length of column (m)	Thickness of FRP (mm)	Area (m^2) of column section	Number of FRP
313	0.2	0.4	2.8	0.04	0.08	5
314	0.5	0.2	2.8	0.0457	0.1	5
315	0.5	0.2	2.8	0.0483	0.1	5
316	0.5	0.2	2.8	0.0483	0.1	5
317	0.5	0.2	2.8	0.0508	0.1	4
318	0.4	0.2	2.8	0.0559	0.08	4
322	0.2	0.5	2.8	0.0559	0.1	4
323	0.2	0.5	2.8	0.0635	0.1	4
330	0.5	0.2	2.8	0.0686	0.1	3
329	0.2	0.5	2.8	0.0711	0.1	3
328	0.2	0.5	2.8	0.0711	0.1	3
327	0.5	0.2	2.8	0.0711	0.1	3
413	0.4	0.2	2.8	0.0432	0.08	3
415	0.5	0.2	2.8	0.0508	0.1	5
416	0.5	0.2	2.8	0.0508	0.1	5
418	0.4	0.2	2.8	0.0559	0.08	4
430	0.5	0.2	2.8	0.0533	0.1	4
427	0.5	0.2	2.8	0.0533	0.1	4
529	0.2	0.5	2.8	0.0508	0.1	4
528	0.5	0.2	2.8	0.0508	0.1	4
527	0.5	0.2	2.8	0.0559	0.1	4

7.3 Cost of FRP Materials

To obtain the cost of the FRP material, equation 7.1 is used.

$$[L \times W \ + 0.2] \ \times \ H \qquad (7.1)$$

Where L is the length of the column measured in meters, W is the width in meters and H is the height of column also measured in meters. 0.2 is added to account for overlapping of the FRP wrap application. Where this value will be multiplied by the number of FRP obtained from FRP coded design program and the unit cost of FRP for one unit area measured in square meters to obtain the cost of using FRP strengthening on the building member.

The epoxy cost is calculated by the ratio 60:40 based on the quantity of FRP used in g/m^2. Therefore, at every 100 g of FRP sheet, 66 g of epoxy is required. However, the average cost will be 15% less of the total cost for the total area required (Fyfe Co. LLC, 2013).

The price of FRP materials are as follows; Sikader 330 is €119.8 per 6 grams; the Sika wrap 300c is €43.18 for one unit size of FRP which is 50 cm by 100 cm. Hence, the final cost for column strengthening with FRP is given in table 7.2.

Table 7.2: Total cost of material for strengthening with FRP on column

Total area (m^2)	Cost of total area	15 % deduction	Total epoxy	Cost of Epoxy	Total cost
102.816	€8,879.2	€7,547.2	125,042 g	€2,496,672	€2,504,219

7.3.1 Beam strengthening with FRP

Table 7.3 shows the required thickness of FRP obtained by flexural strengthening of beam. The FRP is applied at the tension sides denoted by ‘T’ and compression side denoted by ‘C’ as seen in the Table 7.3 to account for the deficiency of moment capacity at these areas. However, the cost is also obtained by multiplication of the total area with unit price per area covered. This time, there is no overlap like in FRP column calculations. Table 7.4 presents the cost of using FRP to strengthen beams.

Table 7.3: Beam FRP result for case study II (Continued to next pages)

Beam	**Required moment (kNm)**	**Moment after strengthening with FRP (kNm)**	**FRP thickness (mm)**	**Reinforcement type or pattern**
B101	45	46.6	0.3	C
B102	43.6	44.3	0.15	C
B113	41.2	49.6	0.35	C
B102	43.6	47.4	1.3	T
B112	33	33.1	1.01	T
B110	43.3	45.2	0.22	C
B109	38.5	38.6	1.21	T
B108	43.2	45.9	1.21	T
B107	41.7	43.3	0.7	T
B106	41.2	43.5	0.31	C
B105	45	48.9	1.3	T
B104	43.7	45	0.3	C
B115	51.4	54.2	0.35	C
B116	51	56.6	0.35	C
B117	39.9	42.8	1.4	T
B118	51.2	52.2	0.32	C
B119	51.2	53.8	1.4	T
B140	51.2	51.7	0.36	C
B138	39.5	40.5	0.32	C
B135	39.5	40.5	0.33	C
B137	51.3	53.3	0.45	C
B125	34.8	35.6	0.86	T
B128	35.6	36	1.1	T
B127	43.9	44.4	1.33	T
B136	57.9	59.3	1.5	T
B134	38.6	38.8	1.22	T

B133	38.6	38.8	1.22	T
B132	34.9	35.6	0.87	T
B124	42	42.6	1.2	T
B139	57.9	60.1	1.5	T
B123	43.6	43.8	0.32	C
B122	35.6	36	1.1	T
B201	54.5	54.8	0.34	C
B202	53.1	53.6	1.4	T
B204	53	53.6	0.34	C
B205	54.6	57.4	1.5	T
B206	43	44.2	0.32	C
B203	37.5	38	1.1	T
B212	37.5	38.1	1.1	T
B213	43.1	43.6	0.31	C
B211	42	44.2	0.18	C
B210	51.28	51.7	0.33	C
B209	45.3	47.1	1.43	T
B208	51.1	52.8	1.38	T
B207	41.9	43.3	0.7	T
B220	33.6	33.7	1.16	T
B219	56.3	57.6	1.5	T
B218	58.4	58.9	1.5	T
B217	46	48.8	0.37	C
B216	58.5	59.8	0.37	C
B215	56.4	56.7	0.36	C
B214	33.6	33.8	1.23	T
B221	35.6	36.7	0.31	C
B240	58.2	59.2	0.41	C
B232	36	36.1	0.9	T
B238	47	48	0.37	C
B235	46.9	48	0.37	C
B230	35.6	37.4	0.23	C
B237	58.2	59.3	0.41	C
B225	35.6	36.5	0.9	T
B229	35.8	37.6	0.31	C
B228	38.7	39.9	0.29	C
B227	54	54.1	1.54	T
B236	63.9	64	1.6	T
B226	50.6	51.5	0.35	C
B234	49.1	49.5	1.5	T
B233	49.1	49.5	1.5	T

B224	50.3	50.6	1.41	T
B239	63.9	64	1.6	T
B223	53.6	54.8	0.38	C
B222	38.8	39.9	0.29	C
B301	49.9	50.3	0.31	C
B302	48.1	49.2	1.28	T
B313	40.2	41.4	0.3	C
B312	35.6	35.7	1.02	T
B309	41.2	42.5	1.3	T
B304	48	48.9	0.31	C
B305	49.9	49.9	1.26	T
B306	40.1	41.4	0.3	C
B307	40.2	41.8	0.65	T
B308	48.7	49.4	1.28	T
B309	41.1	42	0.31	C
B310	48.7	49.1	0.31	C
B314	30.5	31.6	1.1	T
B315	50.8	51.7	0.33	C
B316	55.8	56.9	0.35	C
B317	40.9	40.9	1.33	T
B318	55.8	55.8	1.41	T
B319	50.9	51.6	1.35	T
B320	30.5	31.3	1.09	T
B321	31.5	31.8	0.27	C
B340	29.4	29.6	1	T
B338	42.1	44.8	0.35	C
B335	42.1	44.8	0.35	C
B330	32.8	35.5	0.22	C
B337	50.6	52	0.37	C
B329	31.7	31.8	0.27	C
B328	36	36.5	1.12	T
B327	47.6	4.85	1.4	T
B336	59.2	59.8	1.5	T
B326	45.7	46.9	0.32	C
B334	44.6	45.6	1.4	T
B333	44.6	45.6	1.4	T
B324	45.6	46.4	1.3	T
B339	59.3	59.9	1.5	T
B323	47.3	48.4	0.34	C
B322	36.3	38.5	0.28	C
B402	40	40.2	1.2	T
B405	42.4	42.7	1.04	T

B408	43.7	43.9	1.1	T
B415	41.4	41.7	0.26	C
B416	50.2	51.4	0.31	C
B418	49.9	51.8	1.29	T
B419	41.1	42.8	1.1	T
B437	39.9	40.7	0.29	C
B440	39.9	40.7	0.29	C
B424	37.8	38.9	1.08	T
B427	38.3	38.4	1.11	T
B433	36.3	36.5	1.44	T
B434	36.3	36.5	1.44	T
B436	51	51.6	1.28	T
B439	50.9	51.6	1.28	T
B539	41.9	42.1	0.95	T
B536	41.9	42.1	0.95	T

Table 7.4: Cost of FRP Strengthening on Beam for Case Study II

Total area m^2	**Cost of total area**	**15% deduction**	**Cost of Epoxy**	**Total cost**
134.09	€11,580.01	€9,843.01	€3,248,097.5	€3,257,940.5

7.4 Strengthening with Steel Plate

The result of the plating quantity required for column strengthening will be used to calculate the direct cost of material for steel plate strengthening case study II. Table 7.5 contains the thickness and area of plate for each column as obtained from CSI col. The steel used is hot rolled sheet S275JR. The price is obtained from Norex product catalog price list web site where one unit price per metric ton is 546 USD as of today.

Table 7.5: Column Steel Plate result for Case Study II (continued to next page)

Columns	**Column size (cm)**	**a (mm)**	**b (mm)**	**Steel thickness (mm)**	**Length of column (mm)**	**Area of steel plate (mm^2)**
313	20/40	200	400	2	2800	3360000
314	50/20	500	200	3	2800	3920000
315	50/20	500	200	3	2800	3920000
316	50/20	500	200	3	2800	3920000
317	50/20	500	200	3	2800	3920000

318	40/20	400	200	2	2800	3360000
322	20/50	200	500	3	2800	3920000
323	20/50	200	500	3	2800	3920000
330	50/20	500	200	3	2800	3920000
329	20/50	200	500	3	2800	3920000
328	20/50	200	500	3	2800	3920000
327	50/20	500	200	3	2800	3920000
413	40/20	400	200	2	2800	3360000
415	50/20	500	200	2	2800	3920000
416	50/20	500	200	2	2800	3920000
418	40/20	400	200	2	2800	3360000
430	50/20	500	200	2	2800	3920000
427	50/20	500	200	2	2800	3920000
529	20/50	200	500	2	2800	3920000
528	50/20	500	200	2	2800	3920000
527	50/20	500	200	2	2800	3920000
					total Area =	80080000

Table 7.5 details the total area of steel plates on columns. Thickness of 4 mm is considered for the unit price given in Table 7.6. This was considered according to REHABCON suggestion for using a plate minimum of 4 mm to avoid plate distortion on the site. Although, this will increase the cost of the steel plating by approximately 40 % but it will equally increase the efficiency of steel plating.

Table 7.6: Cost of column steel plating

Total area to be plated (mm^2)	**Area of one unit plate (mm^2)**	**Number of sheet required**	**Cost of plate per tonne**	**Weight per a sheet (tonne)**	**Total cost of sheets**
80080000	1,862,500	43	$546	0.515	$12,088.36

The value is obtained by finding the total area required to be plated and obtain the number of plates to use in the strengthening, and multiply with the available unit price per weight of the sheet.

7.4.1 Beam Strengthening with Steel Plate for Case Study II

Table 7.7 contains the thickness of plate for each beam and the total area required by every beam for plating. The 'T' denotes plating only on the tension side of the beam while 'C' denotes plating for improvement of moment deficient about the compression zone.

Table 7.7: The required steel plate thickness for beams case study II (Continued)

Beam	Thickness of Plate	Width	Height	Length	Reinforcement type T=tension, C= compression.	Total Area mm^2
	mm	mm	mm	mm		
B101	0.9	200	500	3700	C	4440000
B102	0.35	200	500	3450	C	4140000
B113	0.9	200	500	4200	C	5040000
B112	1.6	200	500	4200	T	840000
B110	0.6	200	500	3700	C	4440000
B109	2	200	500	2700	T	540000
B108	1.9	200	500	4200	T	840000
B107	0.9	200	500	3700	T	740000
B106	0.9	200	500	4200	C	5040000
B105	2	200	500	3700	T	740000
B104	0.9	200	500	3450	C	4140000
B115	1.1	200	500	3700	C	4440000
B116	1	200	500	3700	C	4440000
B117	2.4	200	500	2700	T	540000
B118	1	200	500	3700	C	4440000
B119	2.4	200	500	3700	T	740000
B140	1.2	200	500	3000	C	3600000
B138	1	200	500	3000	C	3600000
B135	1	200	500	3000	C	3600000
B137	1.2	200	500	3000	C	3600000
B125	1.2	200	500	3700	T	740000
B128	1.7	200	500	4150	T	830000
B127	2.4	200	500	3700	T	740000
B136	2.7	200	500	4150	T	830000
B134	2	200	500	3550	T	710000
B133	2	200	500	3550	T	710000
B132	1.2	200	500	3700	T	740000
B124	1.9	200	500	3700	T	740000
B139	2.6	200	500	4150	T	830000

B123	1	200	500	3700	C	4440000
B122	1.7	200	500	4150	T	830000
B201	1.1	200	500	3700	C	4440000
B202	2.45	200	500	3450	T	690000
B204	1.1	200	500	3450	C	4140000
B205	2.5	200	500	3700	T	740000
B206	1	200	500	4200	C	5040000
B203	1.8	200	500	4200	T	840000
B212	1.8	200	500	4200	T	840000
B213	1	200	500	4200	C	5040000
B211	0.4	200	500	3700	C	4440000
B210	1.1	200	500	3700	C	4440000
B209	2.5	200	500	2700	T	540000
B208	2.4	200	500	3700	T	740000
B207	0.85	200	500	3700	T	740000
B220	1.9	200	500	3200	T	640000
B219	2.7	200	500	3700	T	740000
B218	2.7	200	500	3700	T	740000
B217	1.2	200	500	2700	C	3240000
B216	1.2	200	500	3700	C	4440000
B215	1.2	200	500	3700	C	4440000
B214	2.1	200	500	3200	T	640000
B221	1	200	500	3000	C	3600000
B240	1.41	200	500	3000	C	3600000
B232	1.4	200	500	3700	T	740000
B238	1.2	200	500	3000	C	3600000
B235	1.2	200	500	3000	C	3600000
B230	0.6	200	500	3700	C	4440000
B237	1.42	200	500	3000	C	3600000
B225	1.3	200	500	3700	T	740000
B229	0.9	200	500	3000	C	1.5E+11
B228	0.9	200	500	4150	C	2.08E+11
B227	2.9	200	500	3700	T	740000
B236	3.1	200	500	4150	T	830000
B226	1.15	200	500	3700	C	4440000
B234	2.7	200	500	3550	T	710000
B233	2.7	200	500	3550	T	710000
B224	2.6	200	500	3700	T	740000
B239	3	200	500	4150	T	830000
B223	1.25	200	500	3700	C	4440000
B222	0.9	200	500	4150	C	4980000

B301	1	200	500	3700	C	4440000
B302	2.1	200	500	3450	T	690000
B313	0.9	200	500	4200	C	5040000
B312	1.6	200	500	4200	T	840000
B309	2.1	200	500	2700	T	540000
B304	1	200	500	3450	C	4140000
B305	2.1	200	500	3700	T	740000
B306	0.9	200	500	4200	C	5040000
B307	0.8	200	500	3700	T	740000
B308	2.1	200	500	3700	T	740000
B310	1	200	500	3700	C	4440000
B314	1.8	200	500	3200	T	640000
B315	1.1	200	500	3700	C	4440000
B316	1.1	200	500	3700	C	4440000
B317	2.3	200	500	2700	T	540000
B318	2.5	200	500	3700	T	740000
B319	2.3	200	500	3700	T	740000
B320	1.8	200	500	3200	T	640000
B321	0.8	200	500	3000	C	3600000
B340	1.5	200	500	3000	T	600000
B338	1.1	200	500	3000	C	3600000
B335	1.1	200	500	3000	C	3600000
B330	0.5	200	500	3700	C	4440000
B337	1.2	200	500	3000	C	3600000
B329	0.8	200	500	3000	C	3600000
B328	1.8	200	500	4150	T	830000
B327	2.4	200	500	3700	T	740000
B336	2.7	200	500	4150	T	830000
B326	1	200	500	3700	C	4440000
B334	2.4	200	500	3550	T	710000
B333	2.4	200	500	3550	T	710000
B324	2.2	200	500	3700	T	740000
B339	2.7	200	500	4150	T	830000
B323	1.1	200	500	3700	C	4440000
B322	0.8	200	500	4150	C	4980000
B402	1.5	200	500	3450	T	690000
B405	1.6	200	500	3700	T	740000
B408	1.8	200	500	3700	T	740000
B415	0.75	200	500	3700	C	4440000
B416	0.95	200	500	3700	C	4440000
B418	2.2	200	500	3700	T	740000

B419	1.7	200	500	3700	T	740000
B437	0.9	200	500	3000	C	3600000
B440	0.9	200	500	3000	C	3600000
B424	1.7	200	500	3700	T	740000
B427	1.8	200	500	3700	T	740000
B433	1.8	200	500	3550	T	710000
B434	1.8	200	500	3550	T	710000
B436	2.11	200	500	4150	T	830000
B439	2.11	200	500	4150	T	830000
B539	1.39	200	500	4150	T	830000
B536	1.39	200	500	4150	T	830000

Table 7.8 contains the total cost of steel plate required to strengthen the beam. The result is found by multiplying the total area found or to be plated by the available cost for one sheet per weight.

Table 7.8: Cost of strengthening beam with steel plate design for case study II

Price per weight	**Total Area (mm^2)**	**Area of one sheet (mm^2)**	**No of sheet Req.**	**Total cost**
$280.95	357,769,780,000.0	1,862,500.0	192,091.2	$53,968,010.6

7.5 Concrete Jacket Strengthening for Case Study II

The columns where strengthened with concrete cover of 100 mm with 14 mm longitudinal reinforcement. The jackets were mainly of L shapes and U shape as shown in previous sections. Table 7.9 (a) below shows the quantity of materials used and their form works while Table 7.9 (b) contains the cost of materials for the concrete jacket on columns. There is no concrete jacketing application on beams because while the concrete jacket is being place on the columns, most of the structural members became brittle, therefore, the strengthening was terminated. Hence, the cost of jacket available here are only concrete jacket applied just before the building members started showing a brittle failure mode. The summary of the calculation for cost of materials for concrete jacket is thus detailed in Table7.10.

Table 7.9: (a) Building Concrete/form Quantity

Storey numbers	Concrete (m^3)	R.C. Form (m^2)
1. Storey Beams	0	0
1. Storey Columns	1.8	42.4
1. Storey Total	1.8	42.4
2. Storey Beams	0	0
2. Storey Columns	8.1	195.8
2. Storey Total	8.1	195.8
3. Storey Beams	0	0
3. Storey Columns	7	169
3. Storey Total	7	169
4. Storey Beams	0	0
4. Storey Columns	6.5	158.2
4. Storey Total	6.5	158.2
5. Storey Beams	0	0
5. Storey Columns	6.1	147.4
5. Storey Total	6.1	147.4

BUILDING WALL QUANTITY (m^2)	Mortars
Storey number	
basement	217.4
1. Storey	195.8
2. Storey	169.08
3. Storey	158.28
4. Storey	147.48
5. Storey	0

Table 7.9 (b) Cost of concrete jacket strengthening for case study II

Reinforcement details			
Storey No	**Ø 8 kg**	**Ø 14 kg**	**TOTAL kg**
FOUNDATION	0	0	0
1. Storey Beam	0	0	0
1. Storey Column	171	455	626
1. Storey Total	171	455	626
2. Storey Beam	0	0	0
2. Storey Column	767	2089	2856
2. Storey Total	767	2089	2856
3. Storey Beam	0	0	0
3. Storey Column	663	1818	2481
3. Storey Total	663	1818	2481

4. Storey Beam	0	0	0
4. Storey Column	619	1689	2309
4. Storey Total	619	1689	2309
5. Storey Beam	0	0	0
5. Storey Column	575	1561	2136
5. Storey Total	575	1561	2136
TOTAL	2797	7614	10411

Table 7.10: Summary of concrete jacket for case study II

Summary of strengthening **Unit price description**	**Unit price**	**Quantity**	**Total**
C20 factory concrete	114	29.7 m^3	$3,387.6
Plain surface concrete Form	11	713.1 m^2	$7,843.7
8-12 mm reinforcement	700	2.8 *t*	$1,957.9
14-50 mm reinforcement	700	7.6 *t*	$5,329.8
		Total	$18,519.00

These results do not include the transport cost of the materials, sales taxation and monthly finance charge.

7.6 Strengthening with Shear Wall for Case Study II

The quantity of materials used in strengthening with shear wall for case study II is detailed in Table 7.11. This table also shows the quantity of form work used for each floor of the building and the mortars as well. Table 7.12 shows the quantity of reinforcement utilized during the strengthening for each level of the floor of the building while table 7.13 is the actual cost of each material for the strengthening.

Table 7.11: Concrete/form cost for shear wall for case study II (continued)

Storey	**Concrete (m^3**	**R.C. Form m^2**
1. Storey Beams	0	0
1. Storey Columns	16.1	141.3

1. Storey Total	16.1	141.3
2. Storey Beams	0.3	2.6
2. Storey Columns	23.4	209.1
2. Storey Total	23.7	211.7
2. Storey Beams	0	0
3. Storey Columns	23.4	209.1
3. Storey Total	23.4	209.1
3. Storey Beams	0	0
4. Storey Columns	23.4	209.1
4. Storey Total	23.4	209.1
4. Storey Beams	0	0
5. Storey Columns	23.4	209.1
5. Storey Total	23.4	209.1

Building wall quantity (m^2)

Storey number	**Mortars**
Basement	303.56
1. Storey	211.77
2. Storey	209.17
3. Storey	209.17
4. Storey	209.17
5. Storey	0

Table 7.12: Quantity of rebar used in shear wall strengthening for case study II

Building Rebar			
Story Number	**Ø 8 (kg)**	**Ø 12 (kg)**	**TOTAL (kg)**
FOUNDATION	0	0	0
1.storey Beam	0	0	0
1.storey Column	593	576	1170
1.story Total	593	576	1170
2.storey Beam	13	21	35
2.storey Column	833	769	1602
2.storey Total	846	790	1637
3.storey Beam	0	0	0
3.storey Column	833	769	1602
3.storey Total	833	769	1602
4.storey Beam	0	0	0
4.storey Column	833	769	1602
4.storey Total	833	769	1602
5.Storey Beam	0	0	0
5.Storey Column	833	769	1602
5.Storey Total	833	769	1602
TOTAL	3940	3675	7615

Table 7.13: Summary of shear wall strengthening for case study II

Unit price description	**Unit price (USD)**	**Quantity**	**Total (USD)**
C20 factory concrete/ m^3	114	110.1 m^3	$12,552.0
Surface concrete form / m^2	11	980.7 m^2	$10,787.3
8-12 mm rebar (ton)	700	7.6 t	$5,330.7

The taxes and monthly finance charge is not included in this result.

7.7 Summary of Cost of Materials of different Strengthening Methods Considered for Case Study II

The cost of the various materials for strengthening can vary from country to country, region to region as was determined over cost effective. The labour cost and the taxes will also have some effects on the cost of executing any method for the purpose of strengthening. Table 7.14 contains the total cost of materials for executing the strengthening methods discoursed in the previous chapters.

Table 7.14 Summary of cost of each method for case study II

Strengthening methods	Cost of beam strengthening (USD)	Cost of column strengthening (USD)	Total cost (USD)
FRP	4,353,333.40	3,346,193.96	7,699,527.36
Steel plating	53,968,010.57	12,080.85	53,980,091.42
Concrete jacket***	______________	18,519.00	18,519.00
Shear wall	_______________	____________	28,670.00

The result as obtained from Table 7.14 above showed that, the cost of strengthening materials for shear wall is minimal compared to other strengthening methods. The cost of epoxy resins has probably contributed to the high cost of the FRP strengthening. The cost obtained from concrete jacket was gotten from strengthening only column. Due to the additional weight of the concrete added to the structure during strengthening, some of the structural members which were safe became brittle, continuous strengthening of the structural members could only make the building loose its ductility. Therefore, the cost obtained in concrete jacket will be higher if one persisted with carrying on with the strengthening while the structural members develop brittle failure mode. Hence, the strengthening scheme was terminated after concrete jacket strengthening proved to be inefficient for this building and the cost obtained will not be considered for comparing with other strengthening methods. Also the cost of steel plating is high. The quality of steel used is a good corrosion resistant; there is no need for further coating. Secondly, the thickness of the steel contributed to the high cost of the strengthening scheme. 4 mm thickness steel is used as suggested by REHABCON (2004). If the actual thickness is considered, the cost is likely to be 40% less of the total cost of the steel used. Therefore, shear wall is preferred in terms of efficiency and economy among other methods studied.

Chapter 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Based on the comparison of the strengthening method in this study, it can be seen that it is important to evaluate buildings firstly before carrying out strengthening work. This will help to find the extent of strengthening required by any building. Buildings may collapse after strengthening as a result of wrong selection of strengthening method. Hence, there is a need for proper evaluation of the building and selection of suitable strengthening method. In the case study I, the strengthening needed appears to be a local strengthening strategy because the additional load affected or caused some damages to few beams. But by proper investigation for performance of the building, there was a reduction in the shear capacity of the building in the x-horizontal directions. There is equally an inter-storey drift from second storey to fourth storey of the building. Shear wall strengthening was able to improve the performance and solve the problems with the shear capacity, beam damage and inter-storey drift.

In the case study II, over 75 % of the members of the building needed performance improvement. Although, building members capacity were improved while executing different strengthening methods studied, however, the cost of materials for carrying out strengthening for some strengthening methods became shortsighted. Such as FRP strengthening and steel plate strengthening. If there will be a further investigation for cost of a new building, the cost might be the same or higher than the cost of raising a new building. And hence, will not be feasible to carry out strengthening if such methods are the last option. Shear wall; a global strengthening strategy method have also shown to be the best both in efficiency and economy.

The use of FRP coded programs became very important especially when there are too many building members to strengthen. It is faster than hand calculation.

The decision selection tool has shown to be effective in deciding option for strengthening methods according to their building problem requirements. It approximately gave the same strengthening option “shear wall” as the best method when compared with other posibilities. However, it will be achieved if the questions from the selection tool are answered correctly. Nevertheless, it will guide engineers who may not know the advantages and disadvantages of strengthening methods. It will be used effectively to decide their choice of selection in strengthening methods.

The cost analysis has also shown that the cost of using local methods to achieve the aim of global strengthening scheme is too high compare to when a global method is used instead. This is also the case when it is used vise-versa. In this case, shear wall also became less in the cost of strengthening to improve the performance of the building.

The choice of material to strengthen building may contribute to the high cost of the strengthening. High quality materials will cost more to lower materials. Availability of materials can course change in material price. Materials which are not readily available tend to be higher in cost for strengthening work. Therefore, in choosing a method for strengthening, the availability of material should be considered. This is paramount in Case study II, where the selection of steel plate strengthening method was used.

Finally, no strengthening method is better than the other; however, their appropriate should be determined by considering building problem requirements.

8.2 Recommendations for Future Studies

All structures strengthened are able to improve in their performance because the strengthening methods and the materials used are of good quality. Most strengthening methods require a connecting element to hold the new materials and the existing structural member requiring strengthening together as an anchorage. These elements should carefully be designed to obtain a good performance. Therefore, since most of the strengthening work requires connecting element, these element should equally be investigated for performance check.

Since old existing buildings constructed without considering earthquake code design may require strengthening at global level, it will be a further advantage to compare strengthening methods under global strengthening strategy such as steel bracing, shear wall, infill wall to have more economical solution.

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