EFFECTIVENESS OF STEEL CONCRETE COMPOSITE BEAM OVER RC BEAM FOR LONG SPAN STRUCTURES

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CERTIFICATE OF APPROVAL

The thesis titled **"Effectiveness of Steel Concrete Composite Beam Over RC Beam For Long Span Structures"**, submitted by Mehedi Bin Sharif, Student No.: 0413042325 P and Session: April 2013 has been accepted as satisfactory in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Structural) on 29th December, 2019.

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Except for the contents where specific references have been made to the work of others, the studies embodied in this thesis are the result of investigation carried out by the author. No part of this thesis has been submitted to any other University or other educational establishment for a Degree, Diploma or other qualification (except for publication).

Mehedi Bin Sharif.

To

My Family

ACKNOWLEDGEMENT

In the name of Allah, the most Gracious and the most Merciful

The author sincerely expresses his deepest gratitude to the Almighty.

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ABSTRACT

In this study effectiveness of steel concrete composite beams for long span structures has been investigated. The aim was to provide a choice to the design community for selecting proper framing system while designing long span structure. Performance of composite and RC beams has been studied based on strength and serviceability parameters. Numerical analysis has been done to understand the behavior of composite and RC beam.

Three different spans have been considered in the study. Six (6) different Finite Element (FE) models were prepared in ETABS to analyze, design and select optimized beam sections for 60ft, 80ft and 100ft spans based on strength requirement. A three storied structure was selected for analysis and design. Similar framing system and loadings were considered for both RC and composite beams. 50% composite action was considered while designing the composite beams. For designing the composite beams, pinned connection was considered at beam ends. However, deflection of composite beams was calculated for fully restrained, partially restrained and pinned connections at the ends. Selected beams were then analyzed to check the serviceability performance. Short and long term deflections were calculated for both Composite and RC beams. Numerical simulations were conducted on RC and Composite beams using ABAQUS. Beams selected in the first step were loaded at one third points of the span. Load was increased up to failure of concrete or steel to obtain the moment vs. displacement curves. Moment vs. deflection curves were prepared from the results obtained from analysis in ABAQUS. The ultimate moment capacity and corresponding vertical displacements for composite and RC beams were compared for the selected span lengths.

The results of current study showed that considering strength, serviceability and ductility criteria for long span floor systems, steel concrete composite beams performed better as compared to RC beams. Composite beams showed higher flexural capacity and improved ductility for the selected span lengths. Similar depth of composite and RC beams was selected for a particular span length. Flexural capacity of composite beam was found to be 30% to 60% higher than that of RC beam. This increase in capacity is affected by the end connections and partial composite action. Short term deflection of RC beam was observed to be slightly lower than that of composite beam under service load condition. However, long term deflection of RC beam was found to be about 4 to 5 times higher as compared to composite beam. The serviceability criteria in design can be easily satisfied with a shallower Composite beam as compared to RC beam. The finite element analysis showed that the deflection of the composite beam at the ultimate capacity point is 150% to 250% higher as compared to the RC beam ensuring improved ductility of composite beams.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 Background

Long span floor systems are used when large column free space is required. It offers greater unobstructed movable space and better visibility for users. This type of structure is often needed in commercial buildings, factories, warehouses, airport hangers, agricultural facilities, stadiums, seminar hall, auditoriums etc. Several structural systems are available for constructing long span floor systems. Steel plate girder, steel truss, steel concrete composite beam, RC beam, pre-stressed concrete beam, arch structure, gable frame, tensile structure etc. are popular options for long span. However, for multistoried buildings mostly used horizontal framing members are reinforced concrete (RC) beams or steel concrete composite girders. RC and composite beams have their own advantages and disadvantages. Concrete is good in compression but it has inherent weakness against tension. On the other hand, steel is good in tension but exhibits some stability problems in compression. But composite beams utilize the beneficiary effects of steel and concrete together. A comparative study is required to identify and quantify the effectiveness of RC and composite beams for floor framing members in a long span system.

Steel Concrete Composite members are structural elements in which steel and concrete act together through mechanical interlock, friction and adhesion. They are designed to maximize the efficiency and benefits of the two component materials. A composite beam consists of structural steel, concrete, profiled steel deck and shear connectors. Figure 1.1 shows the typical components of a composite beam. The combined behavior of concrete and the steel beams makes it a very efficient framing element. This is why it can be an excellent choice for long span floor systems. To meet the serviceability requirements of long span floor system, large depth RC beams are required. Concrete has inherent weakness against tension, creep and shrinkage which adversely affect the performance of RC beam in the long run. Steel concrete composite beam can be a viable solution to this problem. This study investigates the performance of RC and composite beams for long span structure. Strength, serviceability and ductility of RC and composite beams is analyzed and compared for three long spans.

Steel concrete composite members are designed and constructed to work as a single beam. Steel members (beams) are installed at first. After installation of steel beams, steel decks are placed on them. Shear connectors are installed on top flange of structural steel members by welding through the steel deck. Concrete is then cast on the steel deck. After hardening of concrete, the supporting steel and the concrete on steel deck acts as a single unit which is called composite beam. Figure 1.2 shows a typical composite floor at construction stage. The shear connectors ensure proper bonding between the steel beams and concrete deck slab by providing proper resistance to the horizontal shear flow between the steel beams and overlaying concrete deck slab. Hot rolled or fabricated steel I sections are the most popular members used for composite beam construction. Headed shear studs and channel sections are common types of shear connector. To gain different percentage of composite actions between the steel and concrete, number of installed shear connectors along the beam is varied.

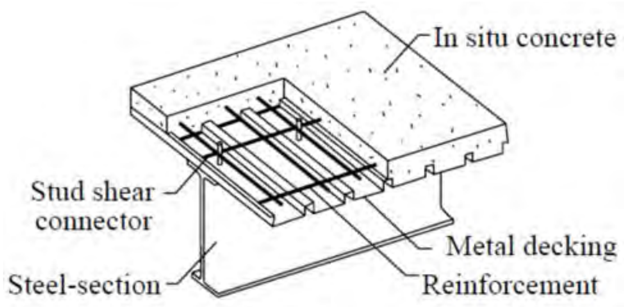


Figure 1.1 Composite beam with concrete slab on metal deck (Uy and Liew 2002)

Figure 1.2 Typical example of composite floor construction (Rackham *et al.* 2009)

1.2 Objectives and Scope of the Study

The main objectives of this research can be summarized as follows:

a) To analyze and design a long span floor system using steel concrete composite members as well as with RC members.
b) To study the effect of end restraints and composite action on the design of the beam. Also the study the effect of composite action on the beam.
c) To compare strength and deflection of the composite beam with RC beam.
d) To study the effectiveness of composite beam over RC beam for various span length (18m, 24m and 30m).

Finite element-based software ETABS has been used to design the RC and composite beams. Similar framing system and loading conditions has been considered for both RC and composite beams. Composite beam has been designed with released end conditions. However, serviceability check has been performed for three different end conditions; simply supported, partially restrained and fully restrained condition. Finally, nonlinear finite element analysis was performed for both composite and RC beams using ABAQUS (HKS 2014), a finite element based software to investigate the flexural behavior of the designed beams subjected to standard two point loading condition. This study only considers the effects of gravity loads on the behavior of the selected beams for long span structure. Performance of the beams under dynamic loading and vibration in not considered in current study.

1.3 Organization of the thesis

This thesis is divided into five chapters.

An introduction to the study is presented in Chapter 1. It includes the background, requirement, objectives and the scope of the study. A brief description of composite beam components and their function have been discussed.

Chapter 2 presents a brief review on the literature related to composite beams. Some experimental and analytical research works carried out on composite beams have been explained briefly in this chapter. Historical use of composite beams has been discussed here. Also, several forms of composite beams have been presented.

Chapter 3 describes detailed methodology of the finite element model for RC and composite beams. The selected materials, framing systems, analysis and design theory, calculation methods, general assumptions have been discussed.

Chapter 4 contains the results obtained from the analysis and design performed in FEM software. The comparison between the results of RC and Composite Beam has been discussed in this chapter.

Chapter 5 presents the summary of the results and draws conclusion based on the results found. Suggestions regarding the design of long span structure have been provided. Recommendations for further study on the topics have been made.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Every building or structure has its own functional requirements. Usable and open space in a structure has become an expensive commodity. It is often required by clients or architects to build structures with lower number of columns. That is when long span floor systems are required. For economy in structure design, it is very important to choose the correct type of framing and construction material. Among a number of options, steel concrete composite beams and RC beams are most widely used structural elements. In this study, effectiveness of these two framing elements will be compared.

In this chapter, history of composite beams will be discussed briefly. Also, the benefits of using composite beams will be highlighted. Several types of composite beams are being used in practice. An overview will be given on various types of composite beams. Basics for computing the strength of composite beams will be discussed briefly. Some experimental and analytical research works performed on composite beams will be discussed at the end of this chapter.

2.2 History and Structural Applications

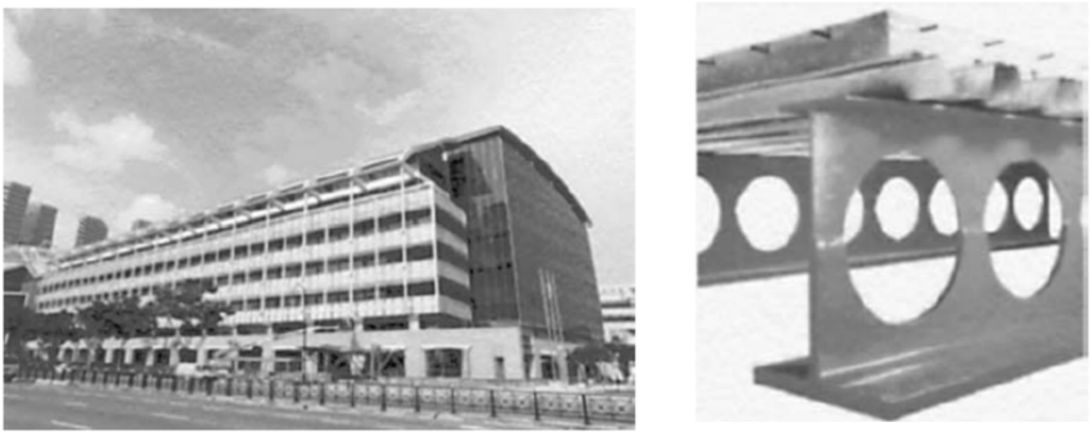
Steel-concrete composite beams are the earliest form of the composite construction method. Recent concept of composite construction was used both in building and bridges in the U.S over a century ago (Uy and Liew 2002). Composite bridges and buildings have been used in Canada since the early 1930s (Chien and Ritchie). Initial form of composite construction involved the use of steel and concrete for flexural members. But without the use of shear connectors there was an issue of longitudinal slip between steel members and concrete slab. To overcome this problem a patent was developed by an American Engineer. Shear connectors were introduced at the top flange of a universal steel section to prevent longitudinal slip. The development of fully composite systems comprising of steel and concrete began. Initially concrete encased steel sections were developed in order to overcome the fire related problem in steel construction. Tubular composite sections were developed because of their beneficial effects of effective formwork and reduced construction time and costs.

Traditionally composite slabs have mostly been used in steel framed office buildings. But they have also been used for industrial buildings, stadiums, cinemas, schools etc.

Grosvenor Place in Sydney is an example which is shown in Figure 2.1. Composite beams were used in this building. The structural system of this building consisted of an elliptical core and radial steel beams. These beams spanned up to 15 m and rested on perimeter steel frames. These beams were designed to be composite to meet the strength and serviceability requirement. Beam ends were connected to the elliptical core by semi rigid connections. Slabs were designed to be one-way slabs which consisted of profiled sheeting that spanned between composite beams (Uy and Liew 2002).

Forrest place, Perth is another multistory building where composite columns, beams and slabs were used. Republic Plaza, which is one of the tallest buildings in Singapore, uses an efficient structural system consisting of both RCC and steel concrete composite frames. Another example of composite construction is One Raffles Link, Singapore; which is shown in Figure 2.4a. This is an eight-story building with wide-span column-free space, where 1300mm deep cell form beams were used to support the composite floor slabs. The girders had a span of 18m. There were 900mm diameter circular web openings spaced at 1350mm centers. The cell form beam was preferred because it is lightweight and permits the passing of all building services through the beam web. The whole structure was designed and detailed as pin connected.

Figure 2.1 Composite Steel-Concrete Floor; Grosvenor Place, Sydney
(Uy and Liew 2002)



(a) (b)

Figure 2.2 a) One Raffles Link; Singapore, b) Cell form Beam (Uy and Liew 2002)

2.3 Benefits of Composite Beams

There are several benefits of using the composite action of steel and concrete.

a) Composite beams utilize the inherent compressive strength of the concrete and tensile strength of the steel. In a composite beam, most of the time concrete remains in compression. An ideal combination of strength can be achieved using different size of steel and concrete.

b) Composite systems are lighter compared to concrete construction. As a result, foundation cost is low.

c) Site erection and installation is easier. Steel decks provide a safe working platform for the construction process. Decks are easy to install and they eliminate the need of formwork.

d) Composite beams are stronger compared to RCC or bare steel beams. Concrete slab provides stiffness to resist deflection and reduce floor vibration. Steel members help to overcome the cracking problems associated with the use of concrete in the tension region of the beam.

e) Steel deck works as positive reinforcement for the RCC slab above deck. Minimum amount of reinforcing steel is required.

f) Properly designed steel deck provides adequate lateral resistance for the beams. Composite slab act as a diaphragm for wind load in the complete structure.

g) Composite action allows the use of shallower steel beams resulting in reduction of storey height and increased number of floors (Rackham *et al.* 2009).

h) Construction time for steel concrete composite floor systems is lower compared to other types of construction.

2.4 Types of Composite Slab and Beam

Steel and concrete are combined together by using shear connectors on the top flange of the steel members. These shear connectors join the concrete with the steel beams. Concrete provides a fire resisting encasement for the steel member. Following methods can be used to form the floor slab (Uy and Liew 2002).

a) A flat soffit reinforced concrete slab.
b) Precast concrete planks with cast in situ concrete topping.
c) Precast concrete slab with in situ grouting at the joints
d) A metal steel deck with concrete, either composite or non-composite

Most common arrangement for composite floor systems is formed by connecting hot rolled or built up steel beams with formed steel deck and concrete slab. The metal deck typically spans between the steel members. It also serves as a working platform for concreting. For long span structures composite beams are excellent solutions. Some practical options for long span construction are discussed here based on "The Civil Engineering Handbook, Second Edition".

Beams with Web openings

To form a standard castellated beam, hot rolled beams are cut along a zigzag line through the web. Top and bottom halves of the beams are then displaced to form a castellation. Castellated beams have limited shear capacity. It is best to use them as long span secondary beams where loads are low. Horizontal stiffeners may be needed to strengthen the web openings. Maximum height of the opening should be limited to 70% of the beam depth. Castellated beams are suitable where small sized mechanical duct work is needed.

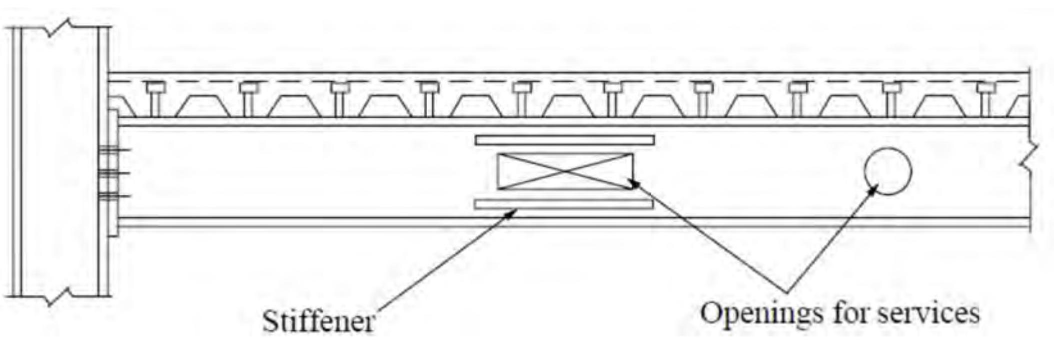


Figure 2.3 Composite beam with web opening and horizontal reinforcement (Uy and Liew 2002).

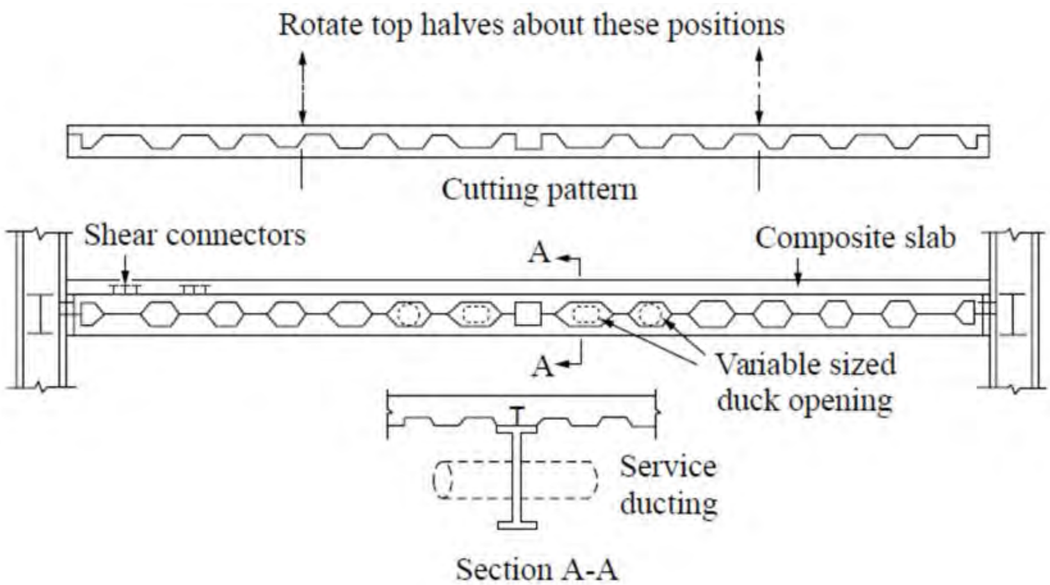


Figure 2.4 Composite castellated beam (Uy and Liew 2002).

Tapered Composite Beam

Tapered beams are economical because they can be designed as per the shear and moment requirement along the beam span. A simply supported tapered beam will have larger depth at the middle and shallower depth at the end of the beam. On the other hand, a rigidly connected beam will have heavier sections at the end. Web stiffeners are required where taper slope exceeds approximately 6 degrees. They are also required where web slenderness ratio is too high.

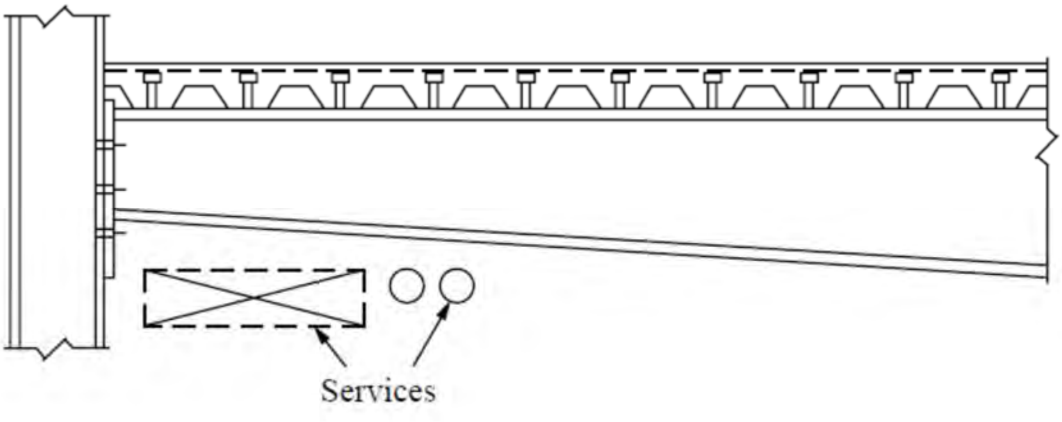


Figure 2.5 Tapered Composite beam (Uy and Liew 2002).

Haunched Beam

Haunched beams are designed as continuous beams and formed by introducing a rigid moment connection between the beams and supporting columns. The haunch connection offers end restraints to the beam and consequently reduces the midspan moment and deflection. Haunched ends may lead to a reduction in beam depth upto 30% and deflection up to 50%. The haunch may be designed to develop the required design moment. This end moment can be large than the plastic moment resistance of the beam. The critical section of a haunced beam shifts to the tip of the haunch. The depth of haunch is selected based on the required moment at the beam column connections. Haunched composite beams are usually used in the case where the beams frame directly to the major axis of the column. So the columns must be designed for the moments that are transferred to the columns from the beam. As a result, heavier columns and more complex connections are required compared to the condition where beams are designed considering simple supports at the ends.

Haunched connections develop rigid frame action which resists lateral loads effectively without the need for vertical bracing. Haunced beams offer higher strength and stiffness which makes it suitable for large span structures. However, haunched connections behave differently under positive and negative moments as the connection is asymmetric about the axis of bending moment.

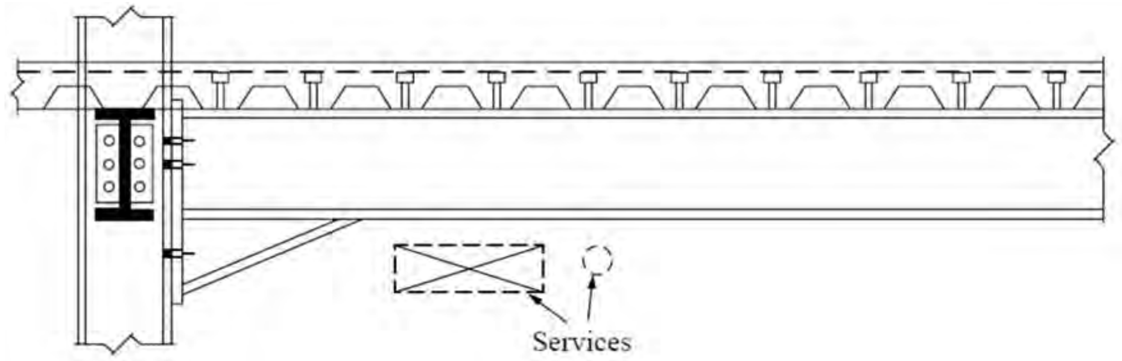


Figure 2.6 Haunched Composite Beam (Uy and Liew 2002).

Parallel Beam System

Parallel Beam system is made of some secondary parallel beams running between two main beams. The main beams are supported by columns and can be continuous. Secondary beams can also be continuous over the main beam. This reduces the number of joints at each supporting beam. Depth of secondary beams can be lower as they are designed to act compositely with the floor slab. The main beam can be made composite or non-composite. This type of beam system is suitable where large service ducts are needed to be accommodated.

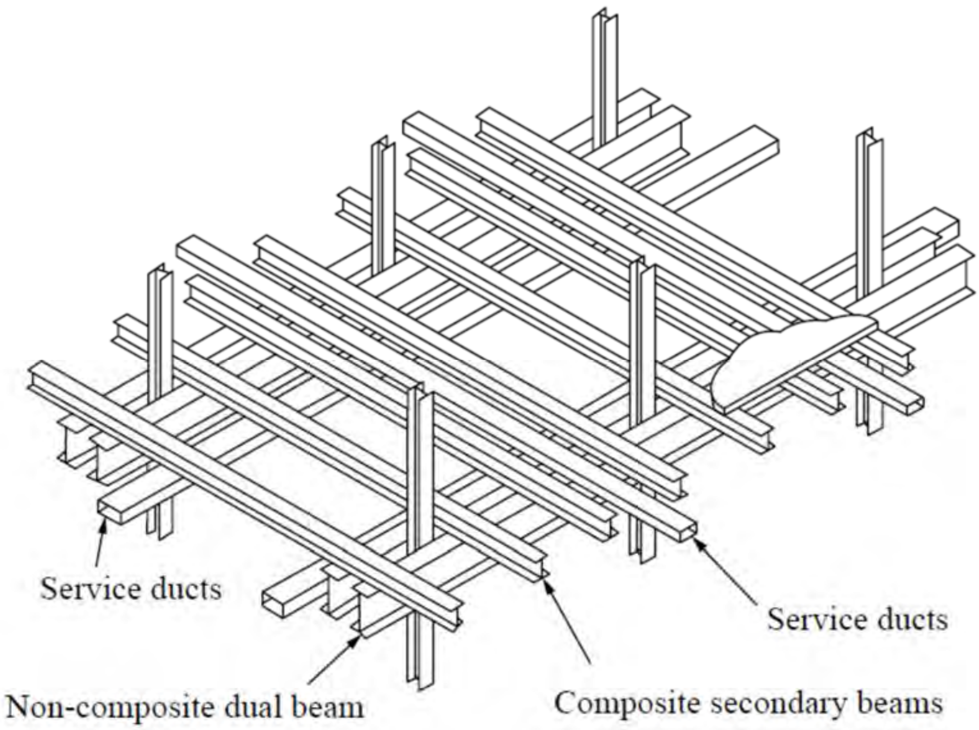


Figure 2.7 Parallel Composite Beams (Uy and Liew 2002).

Composite Truss

Fabrication cost of trusses are higher than traditional beams. But composite truss systems are cost effective and suitable for large span structures. Large service ducts can pass through trusses very easily. The capacity of composite truss is controlled by the yielding of the bottom chord, crushing of the concrete slab, failure of the shear connectors, buckling of the top chord during construction, buckling of web members and instability issues during and after construction. The bottom chord is designed to yield before the crushing of concrete to ensure ductility.

Stub Girder System

Short beam stubs are welded to the top flange of a continuous and heavier bottom girder member to form a stub girder system. These stubs are connected to the top concrete slabs by the shear connectors. Secondary beams and service ducts can pass through the openings formed by the beam stubs.

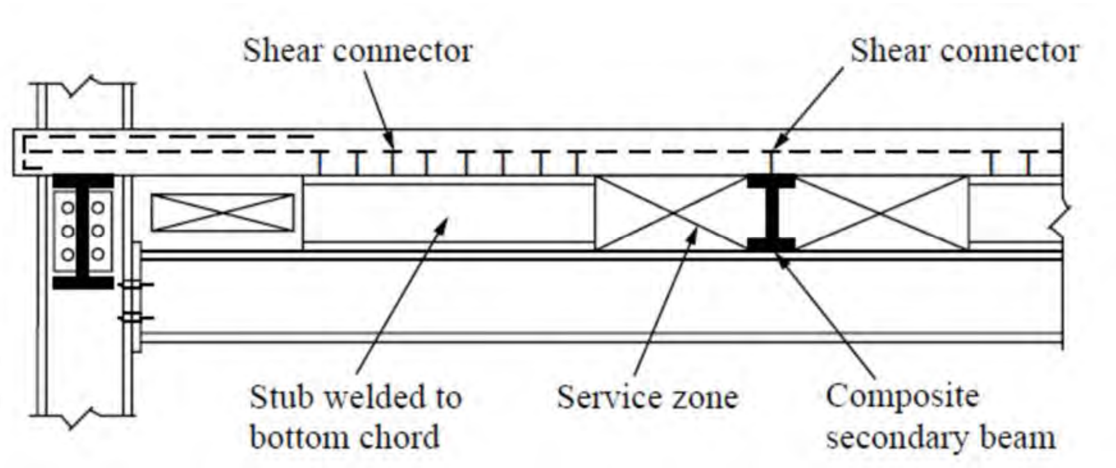
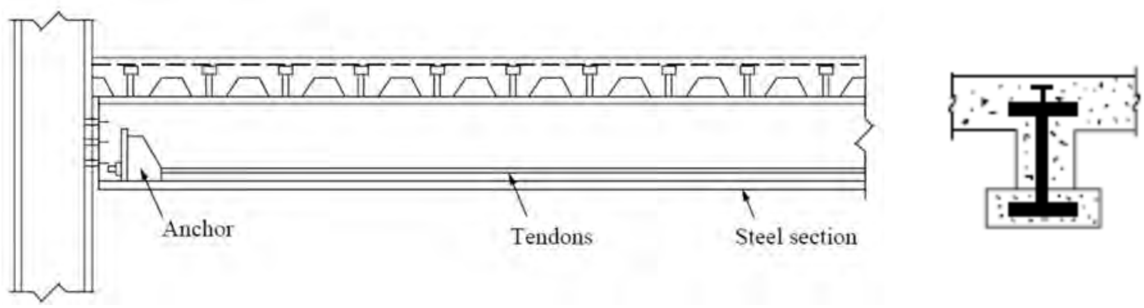


Figure 2.8 Stub Girder System (Uy and Liew 2002).

Prestressed Composite Beams

Prestressing is done by applying a camber to the steel beam and subsequent preloading in the against the bending curvature. Shear connectors ensures the connection between the steel members and the concrete slab. Here concrete remains in compression and the full steel sections works as tension members. As the full beam is encased by concrete, these steel beams are good at resisting fire corrosion.



(a) (b)

Figure 2.9 a) Prestressing of composite beam, b) Prestressed Beam (Uy and Liew 2002).

One of the main components of composite beam is the shear connectors. They prevent slip between the concrete and steel. Various types of shear connectors are used in practice. High strength bolts, steel angles or channels and headed shear studs are some commonly used shear studs. Headed shear stud is the most popular among these.

2.5 Strength of Composite Beams

Designs of composite beams require consideration for both steel and concrete behavior. Strength of composite sections is calculated based on either of the two following two approaches (AISC 2010).

a) Strain compatibility method: It provides a general calculation method.
b) Plastic stress distribution approach: It provides a simple and convenient calculation method for the most common design situations.

Plastic stress distribution method is based on the assumption of linear strain across the cross section and elasto-plastic behavior. It assumes that the concrete has reached its crushing strength in compression at a strain of 0.003 and a corresponding stress of 0.85f'c on a rectangular stress block and that the steel has exceeded its yield strain, typically taken as Fy/Es. Strength of the shear connectors control the interaction between the concrete slab and the steel beam. When composite beam is loaded in positive moment;

full interaction is assumed when shear connector strength exceeds either the tensile yield strength of steel section or the compressive strength of the concrete slab. When composite beam is loaded in negative moment, full interaction is assumed when shear connector strength exceeds the tensile yield strength of the longitudinal reinforcing bars in the slab or the compressive strength of the steel section. When full interaction is not present then the beam is said to be partially composite.

Plastic stress distribution method for all cross sections or for all design situations. Generalized strain-compatibility approach is followed when plastic stress distribution method is not applicable. This approach allows the use of any reasonable strain-stress model for the steel and concrete.

2.5.1 Percent of Composite Action

There is no minimum requirement for the amount of shear connectors i.e. the percent of composite action. A minimum of 25% composite action is often recommended by the U.S. design aids. Two issue arise with the use of low degree of partial composite action (AISC 2010). First, less than 50% composite actin requires large rotations to reach the available flexural strength of the member. This results in very limited ductility after the nominal strength is reached. Second, low composite actins result in an early departure from elastic behavior in both the beam and the studs.

For this study minimum 50% composite action has been ensured. To ensure that the composite beam can reach its plastic moment capacity, web depth-to-thickness ratio was kept below $3.76\sqrt{(E/F_y)}$. In absence of web buckling research on composite beams, the same ratio of bares steel beam is conservatively applied to composite beams.

2.5.2 Material Limitations

For calculating the strength of composite beams, an upper limit of 10 ksi (70Mpa) is imposed on concrete. For lightweight concrete upper limit of compressive strength is 6 ksi (42 Mpa). To ensure the use of good quality concrete a lower limit of 3 ksi (21 Mpa) is imposed on both normal and lightweight concrete. The use of higher strength is permitted if appropriate testing and analyses are carried out.

2.6 Research on Composite Beams and its Components

Many experimental, analytical and numerical research works were carried out on Composite beams to understand their behavior. Some of the investigations performed by previous researchers are briefly discussed below.

2.6.1 Experimental Investigations

Experimental researches were carried out on Composite beams by several research teams (Baran and Topkaya 2014, Pavlović *et al.* 2013, Aida Mazoz *et al.* 2013) to investigate the behavior of beams and shear studs. Several tests were performed on Composite beams with various types of beams and shear studs. Their behavior was recorded. Summary of these experiments are described below:

Baran and Topkaya (2014)

An experiment was performed to investigate the flexural behavior of partially composite beams. Channel type shear connectors were used for composite action. Four full scale steel concrete composite beams were compared with a simple steel beam. Objective of the study was to identify the variation of strength and stiffness properties for various degrees of composite actions. AISC recommendations were followed for assessment of the influence of partial composite actions on flexural behavior. The experiment revealed that the stiffness and strength of composite beams with channel type shear connectors are acceptably close with calculated values. It also revealed that steel beams had significantly lower stiffness and strength compared to the fully composite beams.

Pavlović *et al.* (2013)

Pavlovic et al. performed an investigation to compare between the use of high strength bolts and shear studs as shear connectors. Bolted and welded shear connectors are tested. Basic shear connector properties i.e. shear resistance, stiffness, ductility and failure modes was compared. The research was done in order to improve competiveness of prefabricated composite structures. To understand the difference between the headed shear studs and the bolted shear connectors, push out tests were performed according to EN1994-1-1, using 4 M16-grade 8.8 bolts with embedded nuts. Same layout was followed for the bolted and headed shear studs. Finite element models were also prepared to compare the results with experimental findings. The tests revealed that the bolted shear connectors with single embedded nuts achieved approximately 95% of shear resistance against static loads. Stiffness of the bolted shear connectors we less due to the slip in hole. Also bolted shear connectors with single embedded nut in full depth concrete showed brittle behavior.

Mazoz *et al.* (2013)

This paper presents the results of 24 push out test specimens with a new type of shear connector. The connector was called I shape connector. The experiment was done the investigate the effect of four parameters on the ultimate load capacity. They are a) height of the I shape connector, b) length of the I shape connector, c) compressive strength of concrete and d) the number of transverse reinforcing bars. Failure modes and the load slip behavior were mainly focused. The experimental results were compared with the design equations to predict the ultimate load capacity of I shape shear connectors. From the push out tests, two basic types of failure modes were observed; i.e. shearing of the connector and crushing-cracking of the concrete slab. Shear failure of connector was observed when lower steel grade, and smaller length connectors were used with higher strength concrete. On the other hand, concrete crushing-cracking occurred when higher steel grade and lower strength concrete slab was used. The slip values obtained from the push out tests were greater than 6mm, so the I shape connectors can be considered as ductile as per Eurocode.

2.6.2 Numerical Investigations

Several numerical investigations have been done to analyze the performance of composite beams and its components. Some of them are briefly discussed below:

Jeyarajan *et al.* (2015)

This paper investigates the progressive collapse behavior of steel concrete composite buildings subject to ground blast explosion using nonlinear dynamic analysis and conventional alternative path approach. The analysis model was prepared in ABAQUS. Steel beams were modelled as a two-node linear beam element. Interaction between beam and slab was defined by tie constraint to represent the composite action between the concrete slab and steel beam. Partial composite action was not considered. Local buckling of beam was not considered. A simplified composite slab model was used to avoid geometrical complication and to reduce the computational time required for analysis the 3-D large scale framework. The profile metal deck was represented by rebars in a longitudinal direction based on equivalent area of the respective web and flange plates of the metal deck. Profile concrete is converted into an equivalent uniform concrete section. The concrete section was modelled using a four-node homogeneous shell element. Rebar of deck concrete was defined using rebar definition through the ABAQUS library. A slab model with an equivalent second moment area was compared against the proposed slab, which was based on an equivalent area of steel and concrete. It was observed that the effect on global response of frame is not significant since the slab is being modelled with a steel beam.

Katwal *et al.* (2015)

This paper has presented a simplified numerical model for composite beams with trapezoidal steel decking. The model has been implemented in ABAQUS. Detailed finite element modelling using shell and solid elements can predict the behavior of composite beams precisely. But this method is time consuming. Katwal *et al.* (2015) used shell elements to simulate concrete slab, rebar elements to represent steel sheeting and connector elements to simulate the shear studs. The behavior of shear stud was incorporated in the model by defining shear force versus slip relationship. The predicted results were compared with test results from other experiments and from results from detailed FE models using solid elements. From the comparisons it was concluded that simplified model can predict the behavior of composite with reasonable accuracy. The predictions for both deflection and beam end slip have excellent correlation with the test results of composite beams.

Begum *et al.* (2013)

This paper presents a comparison between the costs involved for composite construction and pure concrete construction in Bangladeshi context. One of the aims was to provide a brief description to various components of steel concrete framing system for buildings. Also a cost effectiveness of steel-concrete composite frames over traditional reinforced concrete frames for buildings structures was to be investigated. A typical commercial building with a floor area of 7720 sft was selected for the study. Design and estimation was done for similar floor patterns and for variable storey heights like 6 storey, 12 storey, 18 storey and 24 storey. Costing was estimated for two type of framing systems based on Bangladesh standard. A cost versus number of storey curve was developed. It was found that RCC framing system is cheaper for low-rise buildings. For buildings with number of stories greater than 15, composite construction becomes economic than RCC construction.

2.7 Summary

Several studies have been done on the behavior of composite beams and its components. However, very few studies have been done to compare the behavior of Composite and RC beams. For long span floor systems, RC beam design demand increases. To meet the serviceability requirements, higher depth is required. Composite can be a viable solution for these limitations. In this research, effectiveness of composite beams has been studied for long span structures based on strength and serviceability requirements.

CHAPTER 3

METHODOLOGY

3.1 Introduction

The primary objective of this study is to compare the effectiveness of composite beams over RC beams for long span structures. Composite beams are formed in various ways. In current study the combination of structural steel, concrete filled steel deck and shear studs is considered as composite beam. In composite beams shear studs prevent the longitudinal slip which enables the steel beam and the overlaying deck slab to act as a unit. On the other hand, reinforced concrete (RC) beams are composed of concrete and embedded steel reinforcement. In this investigation, composite and RC beams with three long spans (18m, 24m and 30m) are designed for similar loading conditions and framing system. AISC 360-10 code is used to design the composite beam and ACI 318-08 is used to design the RC beam. FEM based software ETABS is used to analyze and design the beams. Finally nonlinear finite element simulation was performed using ABAQUS software to investigate the ultimate flexural capacity and ductility of reinforced concrete beams as well as steel concrete composite beams designed for the various span length selected in this study. Detail description of the finite element simulations along with the design parameters used in current study is presented in this chapter.

3.2 Framing System

A three storied building with large column free space was selected for current study. The typical floor plan of the building is shown in Figure 3.1. This building consists of a single bay in X-direction and two bays in Y–direction. The floor span in X-direction is longer than the floor span in Y-direction and was varied from 18 m to 30 m with an intermediate value of 24 m. On the other hand, the floor span in the Y-direction was kept constant at a value of 6 m. The story height of the building was fixed at 3.75 m. Fixed support was considered at column base. The floor slab of the building is initially analyzed and designed with steel concrete composite floor system. Later on the floor was designed with conventional beam supported RC slab. The effectiveness of these two floor systems for variable span length was then studied with respect to serviceability and strength criteria for design. Moreover, the selected floor beams are modeled in ABAQUS, a nonlinear finite element-based software, to explore their flexural capacity and deflection at failure subjected to standard two-point loading.

3.3.1 Composite Floor System

The composite floor system selected for the three storied building is shown in Figure 3.1 and 3.2. The floor framing consists of primary beams (main Girders) and secondary beams (SB). There are two spans in the transverse(Y) direction and single span in the longitudinal (X) direction. CC1 and CC2 are composite columns. SB1 and SB2 are sub beams. Beam SB1 spans between Girder 1 and Girder 2 and SB2 spans between the composite columns. Girder 1 spans between two composite columns CC1. Girder 2 spans between CC2. Decks are spanning in the X direction. Loads from Deck slab are being transferred to the sub beams. Sub beams are taking the loads to the Girders. The Girders are then transferring the loads to the columns. And the columns are eventually taking the loads to the foundations. All the sub beams and the girders are designed as composite members. CC1 and CC2 are fully encased composite columns. Steel I section was fully encased in RCC rectangular section to form the composite column.

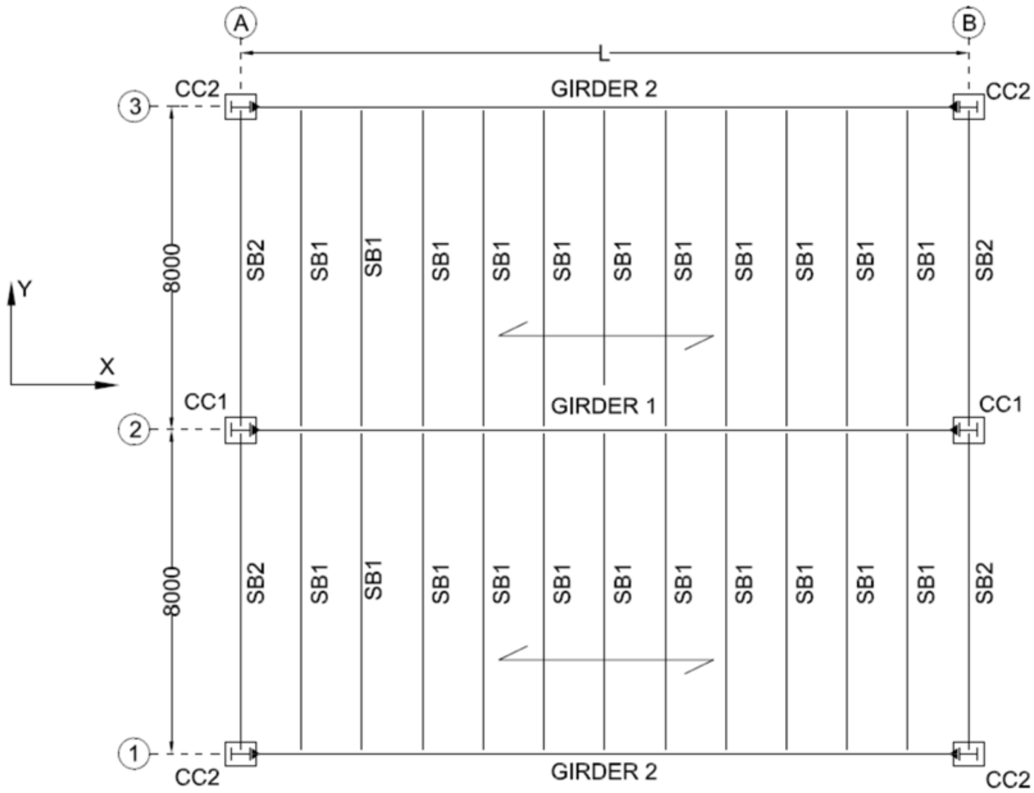


Figure 3.1 Floor Plan of Composite System

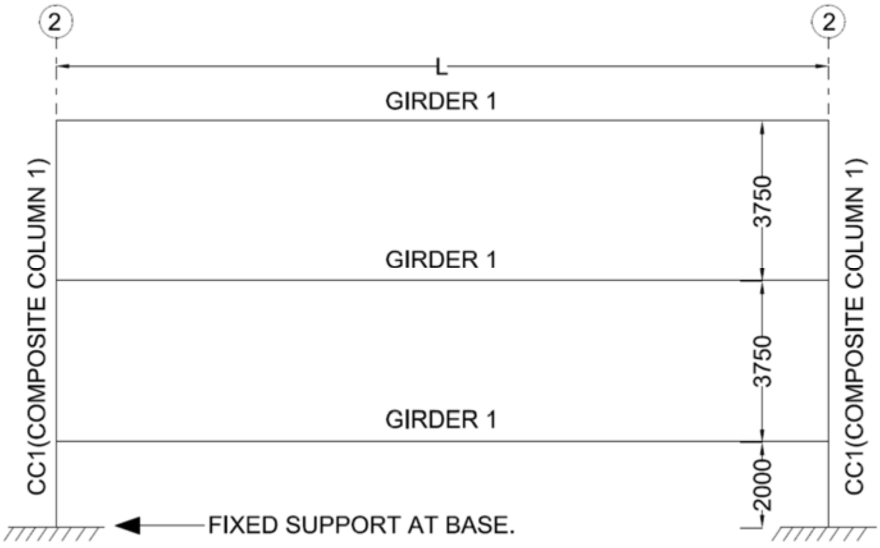


Figure 3.2 Elevation at Grid 2 (Composite framing)

Girder 1 and Girder 2 are 8m apart from one another. So, the span of SB1 and SB2 are 8m. This span will remain constant for all analysis. Secondary beams are spaced at 1.5 m center to center. The spacing between Girder 1 and Girder 2 are kept constant at 8 m. Therefore, the secondary beams in all analysis cases are similar. The size of the secondary beams is also similar for the analysis cases considered.

3.3.2 Analysis and Design Conditions for Composite Beam

In the floor system shown in Figure 3.1 the interior primary beam (Girder 1) is analyzed and designed for three different spans: 18 m, 24 m and 30 m. It is possible to design a composite beam for different percent of composite actions. For this study composite girders will be primarily designed for 50% and 100% composite actions. Moreover, the serviceability criteria for the composite beam will be assessed for three different end conditions: fully restrained, partially restrained and no restraint conditions. First the strength and serviceability of composite beams will be compared for different percentage of composite actions. Finally, the strength and deflection behavior of the composite beam will be compared with that of RCC beam with similar span length.

3.3.3 Selection Criteria of Deck Slab (As per AISC 360-10)

Flexural strength of composite beams is evaluated from the combined effect of steel beam section and formed steel deck filled with concrete. AISC 360-10 imposes some requirements for formed steel deck. In this study the deck slab was selected to meet the criteria provided by AISC. The requirements as per AISC 360-10 are presented below and are shown in Figure 3.3,

a) The nominal rib height cannot be greater than 75mm or 3 inches. The average width of concrete rib or haunch, w_r, cannot be less than 50 mm or 2 inch. For calculation, w_r cannot be taken more than the minimum clear width near the top of the steel deck. The concrete needs to be connected to the steel beam with welded steel headed stud anchors. Stud diameter is to be 19mm (3/4 inch) or less. The steel headed stud anchors needs to be welded either through the deck or directly to the steel cross section. Steel headed stud anchors, after installation, is not to extend 38mm (1 ½ inch) above the top of the steel deck and there needs to be at least 13mm (1/2 inch) of specified concrete cover above the top of the steel headed stud anchors.

b) The slab thickness above the steel deck cannot be less than 2inch or 50mm.

c) Steel deck needs to be anchored to all supporting members. Maximum spacing of the anchor cannot exceed 18 inch or 460 mm. Such anchorage needs to be provided by steel headed stud anchors, a combination of steel headed stud anchors and arc spot welds, or other devices specified by the contract documents.

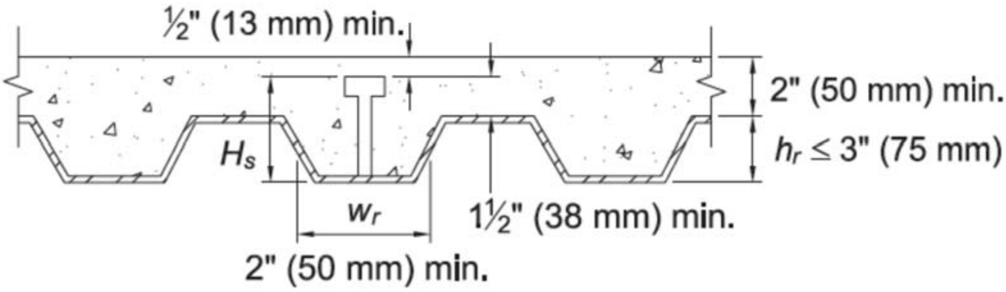


Figure 3.3 Deck Slab Dimensional Requirements (AISC 2010)

For this study a deck was selected which satisfies all the requirements of AISC 360-10. Dimensional properties of the selected deck system are as follows:

Nominal height of Rib, h_r= 63.5mm (2.5 inch)
Width of concrete rib at bottom, w_{rb}= 125 mm (4.92 inch)
Width of concrete rib at top, w_{rt}= 175 mm (6.89 inch)
Diameter of headed stud= 19mm (3/4 inch)

Height of stud, H_s= 127 mm

Shear stud tensile strength, F_u= 400 MPa

Slab thickness above the steel deck, t_c= 114.3 mm [For 2 Hour fire rating]

Extension of steel stud above the top of steel deck= 63.5mm

Concrete cover above the top of steel headed stud anchors= 50 mm

Specified Concrete Compressive strength of deck slab, f'c= 27.5 MPa

Yield strength of Steel Deck, fy= 345 MPa

The dimensional properties of the selected deck slab are shown in Figure 3.4.

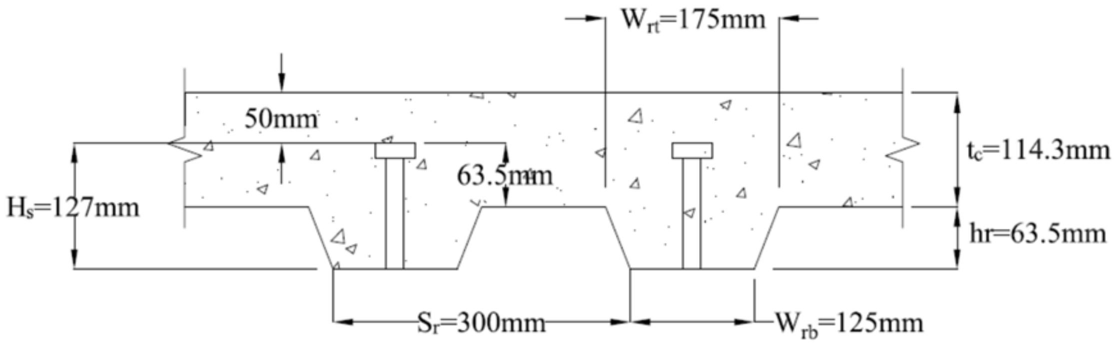


Figure 3.4 Geometric Properties of the Composite Deck Slab

3.3.4 Selection Criteria for Connections

The composite framing consists of steel concrete composite columns, composite secondary beams, main girders and formed steel deck with RCC slab. In composite floor system usually shear connections are designed between secondary beam to main beam and between secondary beam to column connections. This is due to the simplicity and reduced cost of shear connections as compared to moment connections. However, partial or full moment restraint connections are generally used between main beam to column connections to reduce the positive moment demand at the midspan of the main beam. The connection types selected in current study are described in the following sections.

Connection between the sub beams and the girders

In the composite floor system selected in current study the deck ribs are considered to be perpendicular to the sub beams. So primarily the sub beams are receiving loads from the deck. They are transferring the loads to the girders. For the analysis, connections at the end of the sub beams were considered to be at released conditions. So these connections will transfer only shear force to the girders. A fixed connection between the beams and girders would have transferred moments to the girders. This would create minor axis moment for the girders which will induce torsional stress in them. This is not a desirable condition for steel design. So the sub beams were considered to be simply supported on the girders.

Connection between the girder and columns

Girders are the main beams for the framing system. They are receiving the loads carried by sub beams. So the girders will be heavily loaded. Moreover, they will be analyzed and designed for 3 long spans, i.e., 18m, 24m and 30m. It is important to select a suitable connection at the end of these beams. If the girders were considered to have fixed connections at the ends, there will be significantly high negative moments at the joints. This will result in relatively lower positive moment at the girder mid span. At the end of the girders, beam top will be in tension and the bottom will be in compression. In this case the concrete in the deck will be in tension. Concrete is good in compression but weak in tension. So, the concrete filled deck slab will not be effective in this region and beam will need to be designed as steel only member. Also, when the girders have fully fixed end connections, positive moment at the mid span will be low. At mid span concrete remains in compression and can contribute to the strength. However, as the positive moment is low, composite capacity of the beams are not mobilized. So for fixed end girder connections, negative end moment governs and composite beam capacity remains ineffective.

On the other hand, if the girders are considered to be simply supported then all the moments accumulate at the mid span. As the ends are fully released, no moments are transferred to the columns. So the moment capacity required at the mid span will be high. This will result in high deflection at mid span. Higher section size will be required. Therefore, it can be concluded that neither the fully fixed nor the fully released end connections is perfectly suitable at the end of the girders. Each has advantages and disadvantages. To take care of the situation a third option is chosen. The connection between the girder and the columns were considered to be partially restrained moment connection. At first a viable and simple method will be established to analyze the girder considering partially restrained moment connection at the ends.

3.3.5 Flexible Moment Connection

As per ANSI/AISC 360-10, Section 6b, two types of moment connections are permitted to be used in steel and composite structures. The connections are described below:

a) Fully Restrained (FR) Moment Connections: A fully restrained moment connection transfers moment with a negligible rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. Fully restrained connection has sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states.
b) Partially Restrained (PR) Moment Connections: Partially restrained (PR) moment connections transfer moments, but the rotation between connected members is not negligible. In the analysis of the structure, the force-deformation response characteristics of the connection will need to be included. The response characteristics of a PR connection shall be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness and deformation capacity at the strength limit states.

Partially Restrained (PR) moment connections are also known as Flexible Moment Connections (FMC). Geschwindner and Disque (2005) proposed an approach to design unbraced frames with flexible moment connection. Practically there are no connections which are fully rigid or purely released. Generally, when a connection resists at least 90 percent of the beam fixed end moments, then the connection is defined as Fully Restrained or Rigid connection. The connection which allows enough rotation at beam ends and resists no more than 20 percent of the fixed end moment; is referred to be simple connection. Any connection that is capable of resisting a moment between these limits, and permits some rotation at beam ends is called semi rigid. Both the semi rigid and the simple connections are termed as Partially Restrained connection. Simple connections are special case of PR moment connections (Geschwindner and Disque 2005).

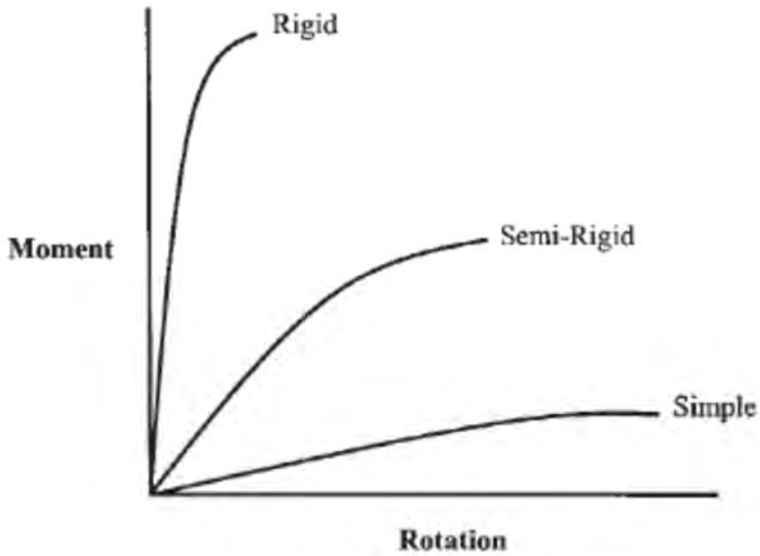


Figure 3.5 Typical moment-rotation curve for three connection types
(Geschwindner and Disque 2005)

Figure 3.5 shows the typical moment rotation curve for three types of connections discussed above. Rigid connection undergoes least rotation while rotation of simple connection is highest.

It is important to understand the behavior of the connections because they will dictate the behavior of frames. Based on the stiffness of connections, moment will be distributed between beams and columns.

General Slope-Deflection Equation for beams carrying gravity load is:

$$M = \frac{WL^2}{12} - \frac{2EI}{L}\theta \qquad (3.1)$$

Where,

M= Beam end moment

Θ= Beam end rotation

From this equation it is visible that, when rotation at beam end is zero (as in rigid connection), moment at the end is highest. When rotation is large then end moment reduces. End moment will depend on the amount of rotation. This relation results in the graph shown in Figure 3.6.

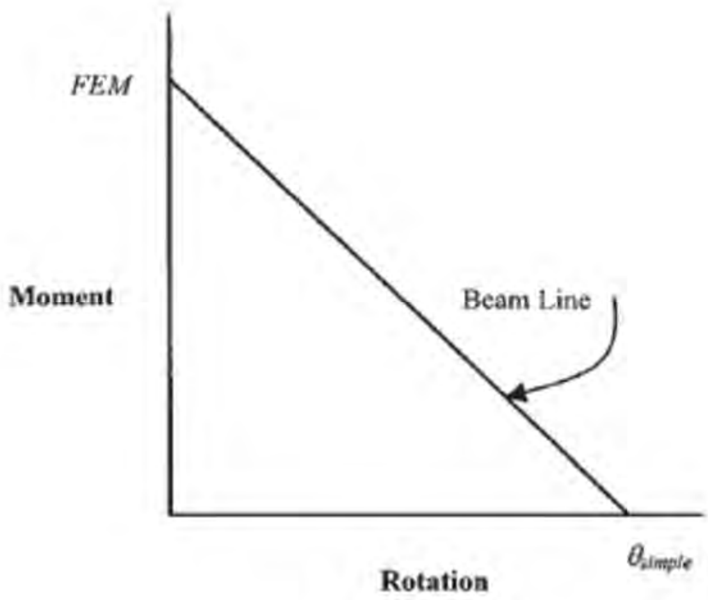


Figure 3.6 Moment Diagram for vertically loaded beams

(Geschwindner and Disque 2005)

By superimposing Figure 3.5 and 3.6 an equilibrium point "a", is found which is shown in Figure 3.7. Actual connection capacity curve is nonlinear and the equilibrium point is difficult to obtain. But a simple solution can be found if the connection is modeled as a straight line. In this case a connection model will have a slope of K = M/ Θ. This straight line intersects the beam line at the same point as the actual connection curve.

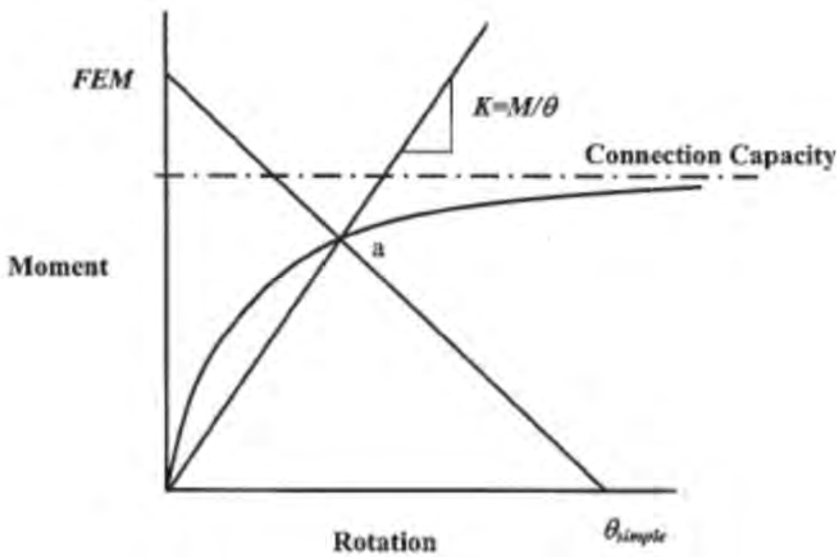


Figure 3.7 Beam line and connection capacity curve equilibrium

(Geschwindner and Disque 2005)

The response of a beam end and mid span moments with respect to the beam/connection stiffness is shown using a curve in Figure 3.8.

In Figure 3.8, u is the Stiffness Ratio (EI/KL) of beam and connection.

Higher value of u represents weaker connections at the end of the beams.

From the curves of this figure, it is observable that, with the increase in stiffness ratio, beam end moment decreases and the mid span moment increases.

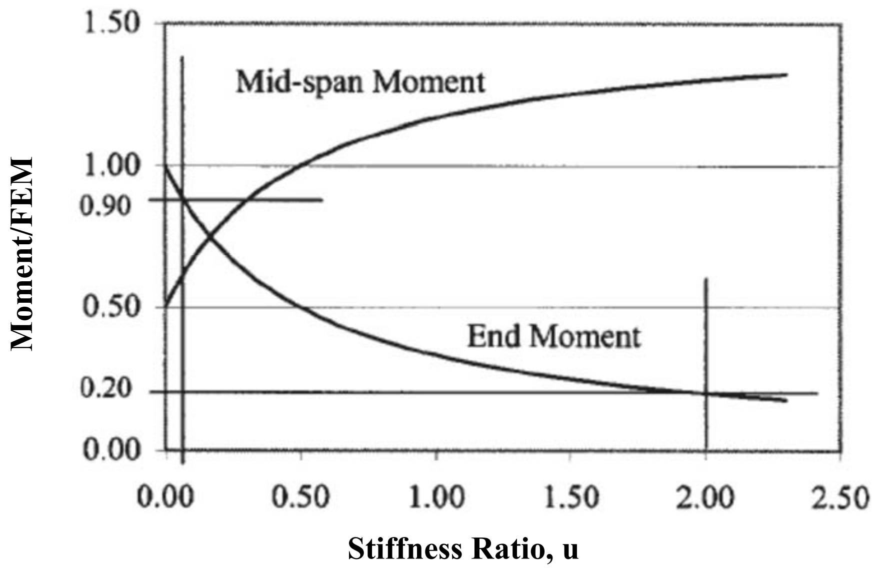


Figure 3.8 Response of Beam as a function of connection stiffness (Geschwindner and Disque 2005)

It is easily understandable that actual moment versus rotation curve is very difficult to obtain. Also, the curve will differ from one another for different type of connections. Even for the same type of connection, moment-rotation curve will vary depending on the connection plate thickness. There is always a possibility of error while calculating the beam moment for flexible moment connection. A simplified approach to define the behavior of flexible moment connections accounting the worst-case scenario for beam design moments has been proposed by Geschwindner and Disque (2005).

This approach is based on two simplified assumptions for implementation of Flexible Moment Connection (FMC) at beam ends:

a) The beams will be designed as simply supported beams. They will be designed for gravity loads only.
b) The connections will be designed for lateral loads only.

3.3.6 Moment-Rotation Curve

It is important to select a moment rotation curve for analysis using flexible moment connections. Following steps were followed to calculate the proper connection stiffness with required fixity.

- At first connection stiffness for fully fixed condition, K was calculated from the equation K= 4EI/L.

  Where,

  K= Connection stiffness for fully fixed end condition

  E= Modulus of elasticity of the beam material

  I= Moment of inertia of the beam to be analyzed

  L= Length of the beam
- Analyses were performed using fully fixed end connections. Fully Fixed end moments, M_{FF} were recorded
- Desired moments at the Partially Fixed beam ends, M_{PF} were calculated
- Ratio of the desired moment M_{PF} and Fixed end moment M_{FF} was calculated.

  $n = M_{PF} / M_{FF}$
- Reduction factor (R.F) for connection stiffness was calculated

  R. F= n / (1-n)
- Connection stiffness, K_{PF} of the partially fixed or flexible moment connection was calculated from the following equation:

  $K_{PF} = K \times R.F$

3.4 RC Floor Systems

The floor system consisting of RCC frames was kept almost similar to that of composite floor system. RC1 and RC2 are reinforced concrete columns. Only line diagrams are shown for the beams. RC BM1 and RC BM2 are secondary beams. The secondary beams are spanning between the main girders. RC Girder1 is the center girder and RC Girder2 is the edge girder. The Girders are supported by column RC1 and RC2. Reinforced concrete slab will be used to carry the floor loads. For RCC framing the secondary beams are spaced at 3m center to center. Height of the RCC structure will be same as the composite structure. Span of the RC secondary beam is 8m. The framing plan and elevation for RC structure is shown in Figure 3.9 and Figure 3.10.

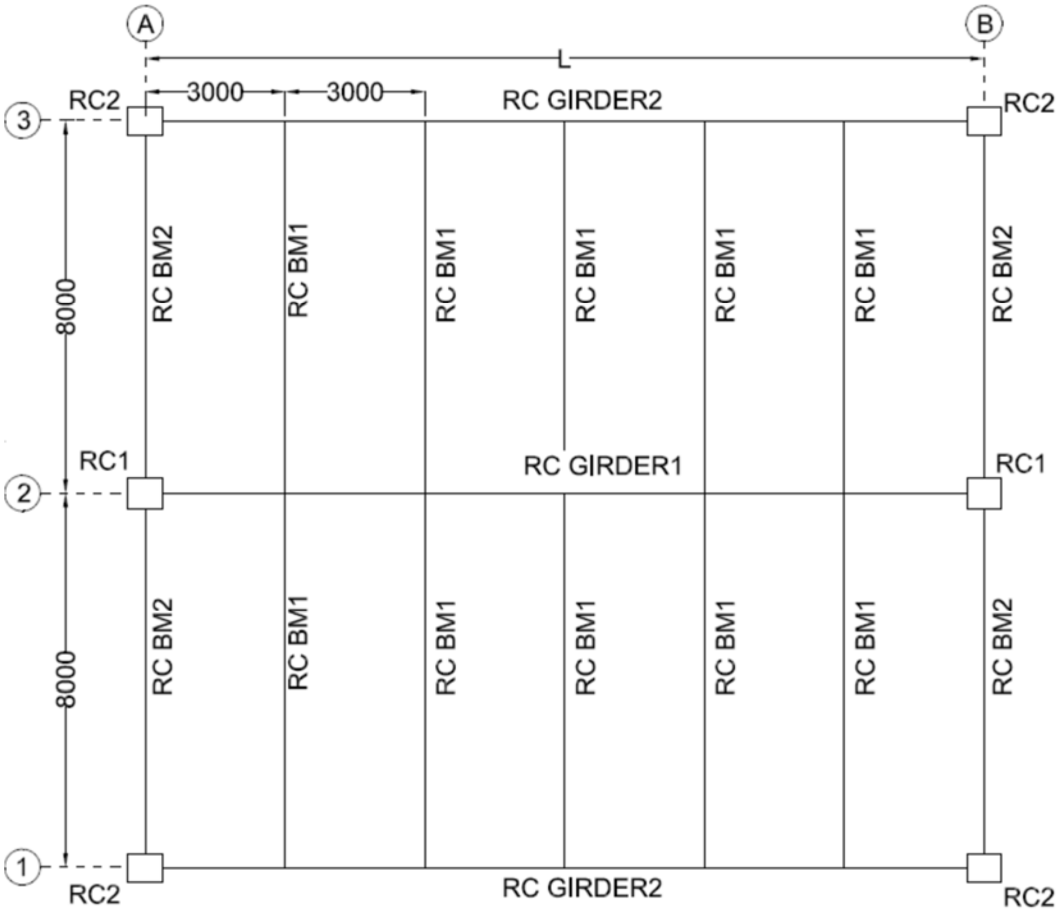


Figure 3.9 Floor framing plan for RC structure

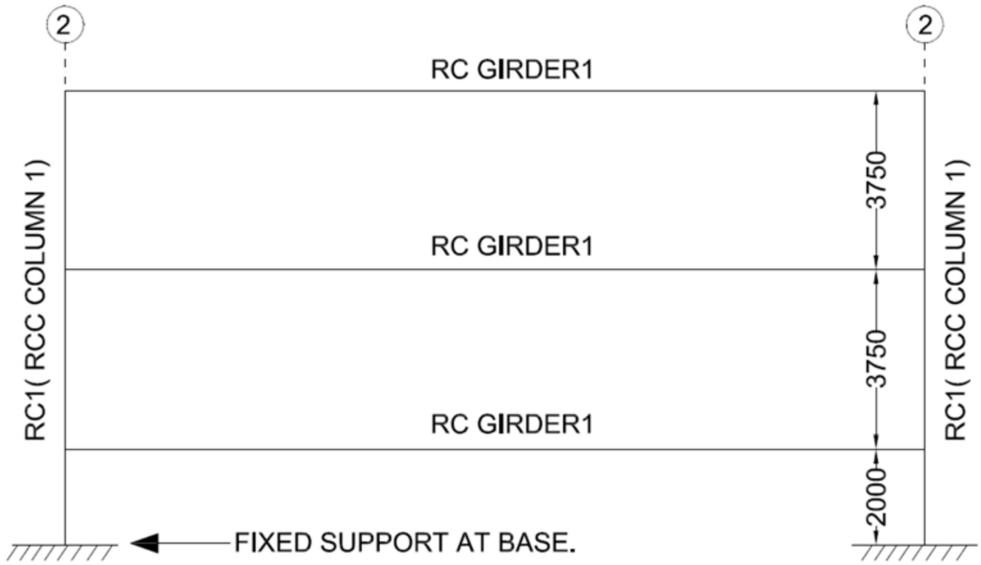


Figure 3.10 Elevation along Grid line 2 (RC framing).

Length of the girders (L) will vary throughout the analysis process. The girders will be designed for different spans and the results will be compared to the results obtained from the composite girders. RC beams, slab and girders will be monolithically cast. So they will act together when loads are applied.

The ration between the span and spacing of secondary beam is 8/3= 2.66 which is greater than 2. So the RCC slab will act as one way. Thickness of RCC slab was selected to be 125mm (5 inch). This thickness satisfies the minimum thickness requirement (L/24) for one-way continuous condition.

3.5 Foundation System

For both RC and composite framing systems, fixed supports were considered at the base of the columns. For the composite framing system, fixed supports will ensure stability against lateral loads particularly in the transverse direction of the framing

3.6 Material Properties

For the analysis and design of Composite and RCC structure the following properties have been considered.

3.6.1 Composite Structure

Concrete compressive strength for deck slab, f'c= 27.5 MPa
Yield stress of structural steel and formed deck, fy= 345 MPa
Ultimate stress of structural steel fu= 448 MPa
Tensile strength of shear stud= 400 MPa
Modulus of elasticity of structural steel Es= 200 GPa
Modulus of elasticity of concrete Ec= 24.8 GPa

3.6.2 RC Structure

Concrete compressive strength for deck slab, f'c= 27.5 MPa
Yield stress of reinforcing steel, fy= 415 MPa
Modulus of elasticity of concrete Ec= 24.8 GPa
Modulus of elasticity of reinforcing steel= 200 GPa

3.7 Member Stiffness

Stiffness is a very important property of a member. Distribution of forces and moments depends on the stiffness of members. Stiffness of a member may vary for different load ranges. Code provided approach was followed to calculate the stiffness of the members.

3.7.1 Stiffness of RC members

Different moment of inertia was considered for RCC members depending on the purpose of calculation. Concrete is good in compression. But it easily cracks under tension. The tensile stress may be the result of direct tension or flexure. Before application of loads concrete members remains untracked. But after application of loads; concrete cracks and the moment of inertia of concrete member changes.

As per section 10.10.4 of ACI 318-08, for strength design of RCC members, the stiffness EI should represent the stiffness of the members prior to failure. At ultimate loads RCC members crack and their stiffness decreases. Section 10.10.4.1 suggests the following values for effective moment of inertia (I_{eff}) for strength design purpose:

Columns……………………….0.70Ig
Walls (Uncracked)……………0.70Ig
Walls (Cracked)……………….0.35Ig
Beams………………………….0.35Ig
Flat plates and flat slabs……….0.25Ig

Where, Ig is the gross moment of inertia of corresponding members.

For deflection calculation it is important to know the effective moment of inertia at service loads. For all concrete members there is a certain cracking moment M_{cr}, up to which the sections remain un-cracked. When service load moment or applied moment M_a exceeds the cracking moment than the section cracks and the inertia of the section reduces. The effective moment of inertia depends on the degree of cracking. Figure 3.11 shows the variation of effective moment of inertia with moment ratio M_a/M_{cr}.It is observed that for a value of $M_a=M_{cr}$, the effective moment of inertia Ie, of beam is equal to I_{ut}. Here, Iut is the moment of inertia of uncracked transformed section.

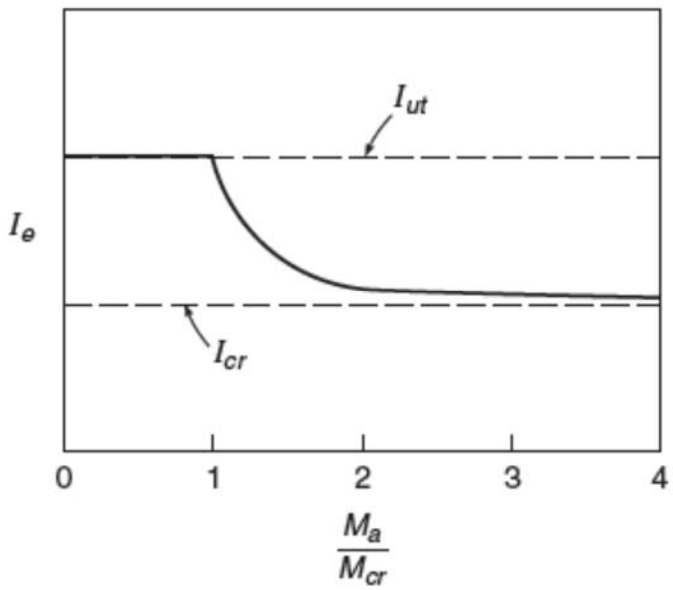


Figure 3.11 Variation of beam Ie with moment ratio (Nilson *et. al* 2009)

With the increase of applied moment, effective moment of inertia decreases. When the service load moment M_a is about 3 times the cracking moment, effective moment of inertia decreases to I_{cr}. Here Icr is the moment of inertia cracked transformed section.
When applied moment is low i.e. $M_a < M_{cr}$, then immediate deflection of RCC beams can be calculated from the following equation:

$$\Delta = \frac{f}{E_c I_{ut}} \tag{3.2}$$

Where f is a function of load, span and support arrangement (Nilson *et. al* 2009). E_c is the modulus of elasticity of concrete and I_{ut} is the moment of inertia of untracked transformed section.

Equation 3.2 is valid only for a very small range of load. Typically, for a beam service load Ma ranges 1.5 times to 3 times of cracking moment, M_{cr}. At higher loads, flexural tension cracks are formed in the beams. It reduces the moment of inertia of the RC beams. For higher range of load, deflection or RC beams can be calculated from the equation below:

$$\Delta_{ic} = \frac{f}{E_c I_e} \tag{3.3}$$

Where,

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_{ut} + [1 - \left(\frac{M_{cr}}{M_a}\right)^3] I_{cr} \leq I_{ut} \quad (3.4)$$

Here,

M_{cr}= Cracking moment of RC beam, which can be calculated from the following equation:

$$M_{cr} = \frac{f_r \, I_{ut}}{y_t} \quad (3.5)$$

Icr= Moment of inertia of cracked transformed section of RC beam.

E_c is the modulus of elasticity of concrete which can be calculated from the following equation provided by ACI (2008):

Ec= 57000√f'c Psi

As long as the applied moment is smaller than the cracking moment, deflection is proportional to the moments. At large moments the effective moment of inertia become progressively smaller. So deflection increases with the increase in moment. The relationship between the applied moment, beam uncracked or cracked stiffness with the dead and live load deflection can be shown in a graphical method in Figure 3.12.

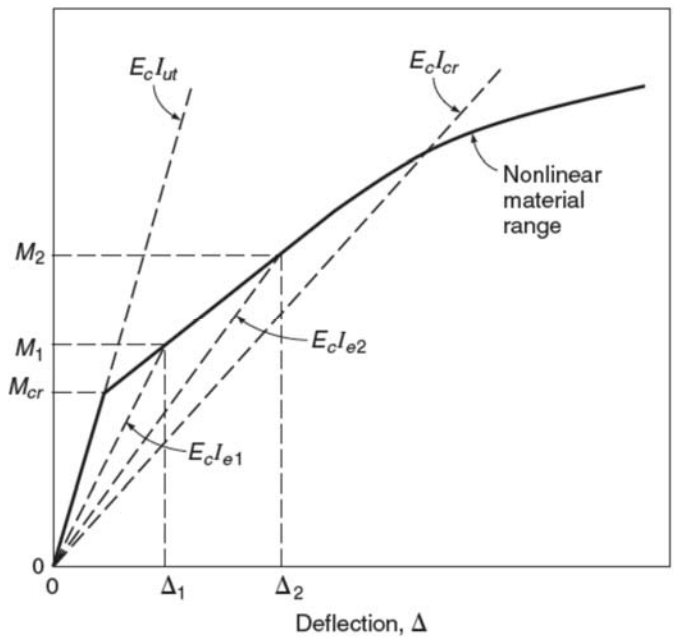


Figure 3.12 Deflection of Reinforced Concrete Beam (Nilson *et. al* 2009)

Most RC beams are continuous over spans or rigidly connected at the ends. So for RC beams some rotational restraint is available at the ends. As per section 9.5.2.4 of ACI 318-08, a simple average of values obtained from Equation 3.4 for the critical positive and negative moment sections can be used to calculate effective moment of inertia of the RC beams. This produces the equation below:

$$I_e = 0.50 I_{em} + 0.25(I_{e1} + I_{e2}) \qquad (3.6)$$

ACI committee 435 showed that, in case of prismatic members, improved results can be obtained by using the following equation:

$$I_e = 0.70 I_{em} + 0.15(I_{e1} + I_{e2}) \qquad (3.7)$$

In equation 3.6 and 3.7;
I_{em} is the effective moment of inertia for the mid span section and I_{e1} and I_{e2} are those for the negative moment sections at the end of the beams. I_{em}, I_{e1} and I_{e2} will be calculated using equation 3.4. A basic problem for continuous span is the uncertainty of calculating the moment on the beam. Deflection depends on the moment diagram and on the flexural rigidity EI of each member of the frame. Again the flexural rigidity depends on the degree of cracking. Cracking depends on the moments. So it is a circular process. The process is iterative and very time consuming. So an approximate approach is required.

For service load analyses of structure ACI (2008) suggests to use moment of inertia that represents the degree of cracking in concrete members at different load levels. However, in section 10.10.4 of ACI 318-08, a relatively simple method was given to calculate the member's stiffness for service load analyses in absence of the accurate estimation of degree of cracking. For service load analysis or deflection calculation ACI (2008) permits to use 1.43 times the moment of inertia used for strength design.

So for deflection calculation the following moment of inertia was used:
For Columns……………I_e= 0.70 Ig x 1.43= 1.0 I_g
For Beams………………I_e= 0.35 Ig x 1.43= 0.5 I_g

This simplified consideration resembles with figure 3.11. When the applied moment is about 2 to 3 times of the cracking moment, the effective moment of inertia of RC beams reduces down to almost half of the uncracked section. Also it is easy to use and produces a very close result which can be found by performing rigorous calculation for the effective RC beam stiffness.

3.7.2 Stiffness of Composite Beam

Most of the times, it is impractical to calculate the accurate stiffness of composite beams. Comparison to short-term deflection tests indicates that the effective moment of inertia, I_{eff} is 15 to 30 percent lower than the calculated value $I_{equiv.}$, based on linear elastic theory. For realistic deflection calculations, I_{eff} should be taken as $0.75I_{equiv.}$ (AISC 2010).

The elastic moment of inertia ($I_{equiv.}$) of composite beam can be calculated as follows:

$$I_{equiv} = I_s + \sqrt{\left(\sum Q_n / C_f\right)}\,(I_{tr} - I_s) \tag{3.8}$$

Where,

I_s= moment of inertia for the structural steel section; in.^{4} (mm^4)

I_{tr}= moment of inertia for the fully composite uncracked transformed section; in.^{4} (mm^4)

ΣQ_n= strength of steel anchors between the point of maximum positive moment and the point of zero moment to either side; kips (N)

C_f= compression force in concrete slab for fully composite beam; smaller of A_sF_y and $0.85f_c'A_c$, kips (N)

A_c= area of concrete slab within the effective width; in.^{2} (mm^2)

Alternatively, a lower bound moment of inertia, I_{LB}. can be used; which can be calculated as follows:

$$I_{LB} = I_s + A_s(Y_{ENA} - d_3)^2 + (\Sigma Q_n / Fy)(2d_3 + d_1 - Y_{ENA})^2 \tag{3.9}$$

Where,

A_s= area of steel cross section, in.2 (mm2)

d_1 = distance from the compression force in the concrete to the top of the steel section, in. (mm)

d_3 = distance from the resultant steel tension force for full section tension yield to the top of the steel, in. (mm)

ILB = lower bound moment of inertia, in.4 (mm4)

I_s= moment of inertia for the structural steel section, in.4 (mm4)

ΣQ_n= sum of the nominal strengths of steel anchors between the point of maximum positive moment and the point of zero moment to either side, kips (kN)

$$Y_{ENA} = [A_s d_3 + \left(\frac{\sum Q_n}{Fy}\right)(2d_3 + d_1)]/[A_s + \left(\frac{\sum Q_n}{Fy}\right)] \text{, in. (mm)} \tag{3.10}$$

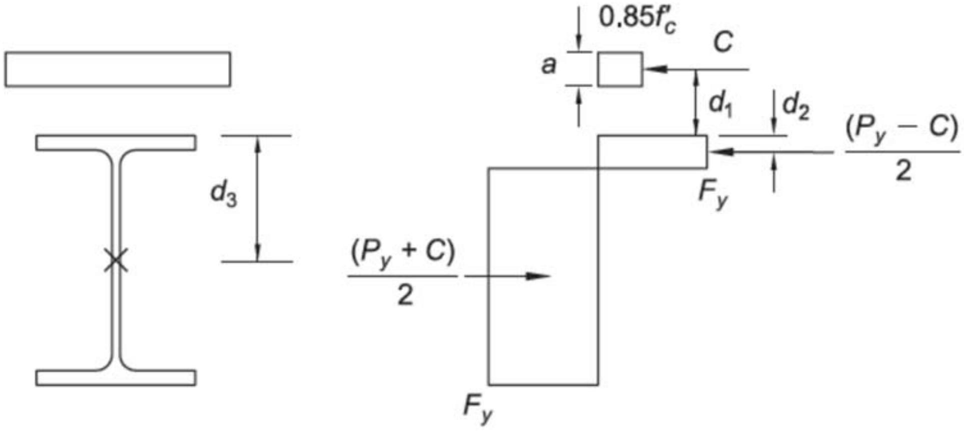


Figure 3.13 Plastic stress distributions for positive moment in composite beam

(AISC 2010)

For Continuous beams, the stiffness can be calculated from the weighted average of moments of inertia in the positive and negative moment regions. The equation is given below:

$$I_t = aI_{pos} + bI_{neg} \tag{3.11}$$

Where,

I_{pos} = Effective moment of inertia for positive moment, in^4 (mm^4)

I_{neg} = Effective moment of inertia for negative moment, in^4 (mm^4)

The effective moment of inertia is based on the cracked transformed section considering the degree of composite action. For continuous beams subjected to only gravity loads, the value of a and b can be taken as 0.6 and 0.4 respectively. When composite beams are part of the lateral force resisting system, the value of a and b can be taken as 0.5 for calculation related to lateral drift (AISC 2010).

The effective section modulus, S_{eff}, referred to the tension flange of the steel section for a partially composite beam, may be approximated by the following equation,

$$S_{eff} = S_s + \sqrt{\left(\sum Q_n / C_f\right)}(S_{tr} - S_s) \tag{3.12}$$

Where,

S_s= Section modulus for the structural steel section, referred to the tension flange, in.^{3} (mm^3)

S_{tr}= Section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3} (mm^3)

3.7.3 Local Buckling

For a steel beam to reach its plastic moment capacity, the flange and web of the beam must be compact. AISC has provided with some limiting values for checking the compactness of beam flange and web. For a doubly symmetric I shaped built up member the compactness requirements are as follows:

For Compact Compression Flange

$$\frac{b_f}{2t_f} \leq 0.38\sqrt{\frac{E}{F_y}}\ (= 9.15)$$

For Compact Web

$$\frac{h}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}}\ (= 90.55)$$

Where,

b_f= Total width of Beam Flange

t_f= Thickness of Flange

h= Depth of Beam Web

t_w= Thickness of Beam Web

E= Modulus of Elasticity of Steel

Fy= Yield Stress of Steel.

3.8 Design Loads and load combination

The following loads were considered for the design of Composite and RC beam.

Dead Loads:

Self-weight of the structure.

Floor finish 30 psf

Partition wall 20 psf

Miscellaneous dead load 10 psf

Live load 100 psf

Load Combination= 1.2 Total dead load + 1.6 Live load

3.9 Design Method of RC Beam

It is important to understand the behavior of structure under loads. The actual behavior of a RC beam is complicated. To simplify the joint action of RCC and steel some assumptions are needed to be made.

3.9.1 Fundamental Considerations

The fundamental propositions on which the mechanics of reinforced concrete is based are as follows (Nilson *et. al* 2009):

i) The internal forces, such as bending moments, shear forces, and normal and shear stresses, at any section of a member are in equilibrium with the effects of the external loads at that section.

ii) The strain in an embedded reinforcing bar (unit extension or compression) is the same as that of the surrounding concrete. It is assumed that perfect bonding exists between concrete and steel at the interface. So, no slip can occur between the two materials.

iii) Cross sections that were plane prior to loading continue to be plane in the member under load. This assumption is not absolutely true when the members are loaded close to failure. But the deviations are usually minor.

iv) Tensile strength of concrete is only a small fraction of its compressive strength. So the part of the concrete member which is in tension is usually cracked and cannot resist any tension stress.

v) The theory is based on the actual stress-strain relationship and strength properties the two constituent materials or some reasonable equivalent simplification thereof.

3.9.2 Design Steps of RC Beams

FEM based software ETABS version 2016 was used for analysis and design of RCC and Composite structures. As proposed by ACI 10.2; the program uses a simplified rectangular stress block for the design of RC beams which is shown in Figure 3.14.

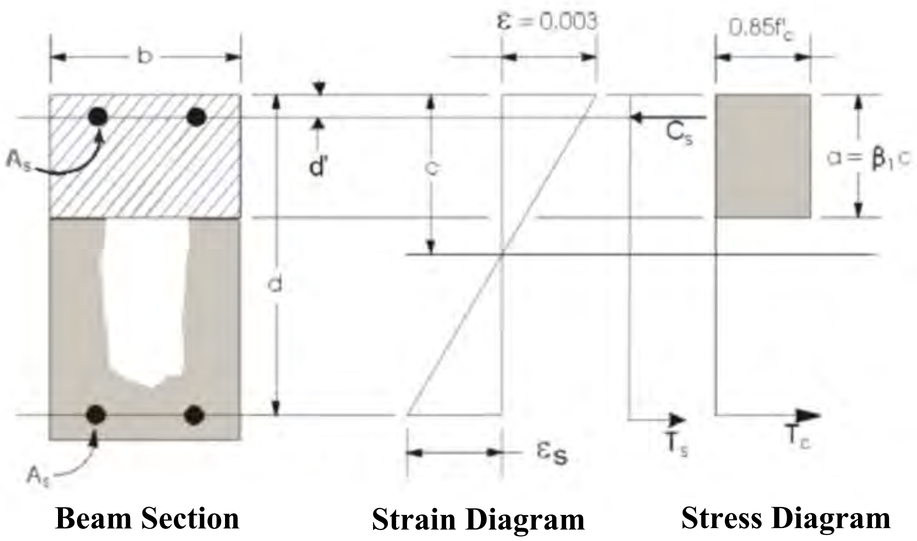


Figure 3.14 Stress-strain diagram for the RC rectangular beam design (ETABS CFD Manual 2016)

As per ACI 10.3.4 the net tensile strain of the reinforcing steel is considered to be minimum 0.005 for tension-controlled condition. When design moment is larger than the moment capacity of RC beam for this design condition, the area of compression steel is calculated. Concrete compressive strain is considered to be 0.003. After calculating the design moment ETABS follows the steps described below for design of RC beams.
The depth of compression block “a” is calculated using the following equation:

$$a = d - \sqrt{d^2 - \frac{2|M_u|}{0.85 f'_c \Phi b}} \tag{3.13}$$

Where,
M_u = Ultimate design moment
f'_c = Concrete compressive strength
d = Effective depth of beam

b= Width of beam

Φ= 0.9 for tension-controlled section.

The maximum depth of the compression zone, c_{max} , is calculated from the following equation:

$$c_{max} = \frac{\varepsilon_{c\,max}}{\varepsilon_{c,max} + \varepsilon_{s,min}} d \tag{3.14}$$

Where,

$\varepsilon_{c,max} = 0.003$

$\varepsilon_{s,min} = 0.005$

The maximum allowable depth of the rectangular compression block can be obtained by,

$a_{max} = \beta_1 c_{max}$

Where β_1 is calculated from the equation below:

$$\beta_1 = 0.85 - 0.05\left(\frac{f_c' - 4000}{1000}\right), \qquad 0.65 \le \beta_1 \le 0.85$$

If a <= a_{max} , then area of tensile reinforcement is calculated from the following equation:

$$A_s = \frac{M_u}{\Phi f_y \left(d - \frac{a}{2}\right)} \tag{3.15}$$

The area of steel calculated from equation 3.15 is positive reinforcement. It will be placed at the bottom of the beam.

If a >= a_{max} , then only bottom reinforcement of beam is not enough to take care of the design moment. In this case compression reinforcement will be required. The area of compression reinforcement is calculated through the steps described below.

At first, the compressive force developed in concrete alone is given by,

$$C = 0.85\, f'_c b a_{max} \tag{3.16}$$

The moment resisted by concrete compression and tensile steel is calculated from the following equation:

$$M_u = C\left(d - \frac{a_{max}}{2}\right)\Phi \tag{3.17}$$

So the moment resisted by the compression steel and additional steel is,

$M_{us} = M_u - M_{uc}$

Required compression steel A's is calculated from the equation below:

$$A'_s = \frac{M_u}{(f'_s - 0.85f'_c)(d - d')\Phi} \tag{3.18}$$

Where,

$$f'_s = E_{s\ \varepsilon_{c\max}} \left[\frac{c_{max} - d'}{c_{max}}\right] \le f_y \tag{3.19}$$

The total reinforcement required at the bottom of the beam will be $A_s = A_{s1} + A_{s2}$

Where,

A_{s1} is the required compression steel for balancing compression in steel.

A_{s2} is the reinforcement required for balancing the compression steel provided at the top of the beam.

As1 and As2 can be calculated from the equations below:

$$A_{s1} = \frac{M_u}{\Phi f_y\left(d - \frac{a_{max}}{2}\right)} \tag{3.20}$$

$$A_{s2} = \frac{M_u}{\Phi f_y(d - d')} \tag{3.21}$$

3.9.3 Deflection Calculation of RC Beam

Deflection of RC beams can be classified into two basic categories. Short term deflection and long-term deflection. Short term deflection consists of deflection due to sustained loads i.e., dead loads and deflection due to live loads. It is important to use appropriate stiffness of the member while calculating the deflection. Stiffness depends on the moment of inertia of a member. Depending on applied loads and resulting moments, the moment of inertia changes. For different loading concrete beams undergo different degrees of cracking. The amount of cracking dictates the moment of inertia of a RC beam for certain loading condition. When the applied moment exceeds the cracking moment of a beam, the beam cracks. So the moment of inertia reduces. For calculating deflection, this reduced moment of inertia will need to be used. As described in ACI 10.10.4 the moment on inertia of a RC beam can be taken as half of the gross moment of inertia while calculating the deflection under service loads. So for calculation of deflection due to dead load and live load, the effective moment of inertia $I_e = 0.5\ I_g$ was used.

Long term deflection of RC beams occurs due to creep and shrinkage. To calculate the long-term deflection, the deflection due to sustained loads is multiplied by a factor λ_Δ. So if the deflection due to sustained loads is Δ_1, then long term deflection Δ_{lt} can be calculated from the equation below:

$$\Delta_{lt} = \lambda_\Delta\, \Delta_1 \qquad (3.22)$$

The value of λ_Δ depends on a number of factors. Higher strength concrete undergoes reduced creeps. Reinforcement in the compression zone of beams provides some beneficial effects against the deflection of beam. Based on long term tests, the following equation was recommended to calculate the value of λ_Δ,

$$\lambda_\Delta = \frac{\mu\xi}{1 + 50\mu\rho'} \qquad (3.23)$$

Where,

$$\mu = 1.4 - f'_c/10{,}000$$

$$0.4 \leq \mu \leq 1.0$$

And,

ρ' = As'/ bd = percentage of compression reinforcement

ζ is a time dependent variable. It is taken from the graph below:

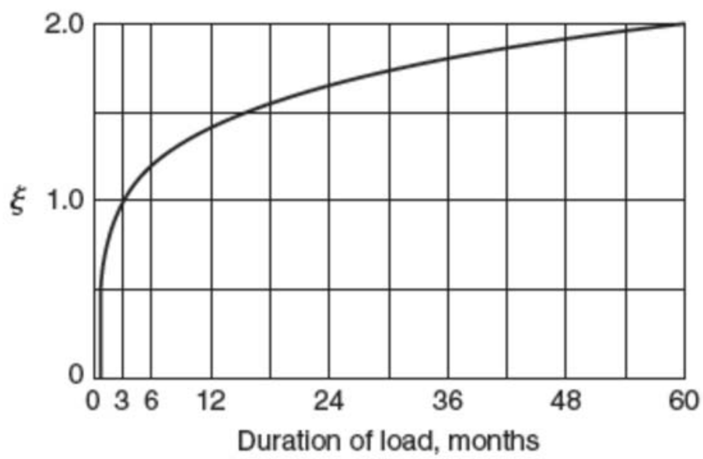


Figure 3.15 Variation of ζ for long term deflection (Nilson *et. al* 2009)

After calculating $\lambda_\Delta$from equation 3.23, long term deflection due to sustained loads can be calculated from equation 3.22.

If deflection due to live load is Δ_2 then total deflection Δ becomes,

$$\Delta = \Delta_1 + \Delta_{lt} + \Delta_2 \tag{3.24}$$

3.10 Design Method of Composite Beam

Composite beam gains its strength from the combined effect of concrete and steel. In this study we are investigating the behavior of composite beams with formed steel deck, filled with concrete. To understand the behavior of a composite beam and to calculate the flexural strength of the beam, it is necessary to be familiar with certain terms.

3.10.1 Effective Width of Concrete Slab

The effective width of concrete slab varies based on different codes. As per AISC 360-10 chapter I, effective width of the concrete slab of the composite beam will be the sum of the effective widths for each side of the beam centerline. The effective width on each side will not exceed the following:

a) One-eighth of the beam span, center to center of supports;
b) One-half the distance to the centerline of the adjacent beam; or
c) The distance to the edge of the slab.

3.10.2 Strength of Composite Beams During Construction

Composite beams start acting as composite when the concrete has gained adequate strength. So during the construction the steel beams will need to be temporarily shored. If temporary shored are not used, then the steel beam will need to take the loads alone prior to the concrete attaining 75% of its compressive strength.

3.10.3 Limitations for Concrete Filled Steel Deck

The selection criteria or limitations for the selection of concrete filled steel deck were discussed in section 3.3.2 of this chapter. AISC 360-10 chapter I was followed for this investigation.

3.10.4 Effective Concrete

As per AISC 360-10; effectiveness of the concrete below the top of the steel deck is dependent on the orientation of the deck ribs. If the deck rib is perpendicular to the steel beam, then the concrete below the top of the steel deck is neglected. This portion of the concrete will not be considered during the calculation of the composite section properties. It will also be ignored while calculating the area of concrete A_c.

On the other hand, if the orientation of deck rib is parallel to the steel beam, then the concrete below the top of steel deck is not ignored. It will be included in the composite section properties.

3.10.5 Load transfer between the steel beam and concrete slab

Composite beams act as a unit consisting of concrete and steel beam. But to ensure this behavior, it is necessary to prevent the slip between the concrete slab and the steel beam. Steel headed studs or steel channel anchors are used as shear connectors to ensure proper resistance against slip. The entire horizontal shear at the interface of steel beam and concrete slab is assumed to be transferred by the steel headed studs or channel anchors. For positive flexure zone the concrete remains under flexural compression. The nominal shear force, V' between the steel beam and the concrete will be governed by concrete crushing, tensile yielding of the steel section and the shear strength of the steel headed stud or steel channel anchors. Value of V' will be the lowest value calculated from the equations below:

a) Concrete Crushing

$$V' = 0.85f'_c A_c$$

b) Tensile yielding of the steel section

$$V' = F_y A_s$$

c) Shear strength of steel headed stud or steel channel anchors

$$V' = \sum Q_n$$

Where,

Ac= Area of concrete slab within effective width, in^2 (mm^2)

As= Area of steel cross section, in^2 (mm^2)

$\sum Q_n$=Sum of nominal shear strength of steel headed stud or steel channel anchors between the point of maximum positive moment and the point of zero moment, kips (N)

3.10.6 Positive Flexural Strength of Composite Beams

The positive flexural strength of a composite beam may be controlled by the strength of the steel section, the concrete slab or the steel anchors. If the web of the steel section is slender and a large portion of the web is in compression, then web buckling may limit flexural strength of the beam. Local web buckling does not reduce the plastic strength of a bare steel beam if the beam depth to web ratio remains under the limit 3.76√ (E/Fy). There is no research available on composite beams regarding web buckling. So AISC conservatively imposes the same limit for composite beams.

The nominal plastic moment resistance of a composite section in positive bending is calculated from equation 3.25 below. Corresponding terms were explained in Figure 3.13.

$$M_n = C(d_1 + d_2) + P_y(d_3 - d_2) \tag{3.25}$$

Where,

M_n is the nominal positive flexural strength.

P_y= Tensile strength of the steel section = F_yA_s kips (N)
d_1= distance from the centroid of the compression force, *C*, in the concrete to the top of the steel section, in. (mm)
d_2 = distance from the centroid of the compression force in the steel section to the top of the steel section, in. (mm). For the case of no compression in the steel section, d_2 = 0.
d_3 = distance from *Py* to the top of the steel section, in. (mm)
C = Compression force in the concrete slab which is the smallest of;
$C = A_{sw}F_y + 2A_{sf}F_y$
$C = 0.85f'_cA_c$
$C = \sum Q_n$

Where,

f'_c =specified compressive strength of concrete, ksi (MPa)
A_c = area of concrete slab within effective width, in.^{2} (mm^2)
A_s = area of steel cross section, in.2 (mm2)
A_{sw} = area of steel web, in.2 (mm2)
A_{sf} = area of steel flange, in.2 (mm2)
F_y = minimum specified yield stress of steel, ksi (MPa)
ΣQ_n = sum of nominal strengths of steel headed stud anchors between the point of maximum positive moment and the point of zero moment to either side, kips (N)

3.10.7 Negative Flexural Strength of Composite Beams

AISC 360-10 provides two alternatives to calculate the available negative flexural strength of composite beams. The negative flexural strength can be determined for the steel section alone following the requirements of simple steel only beams. Alternatively, the available flexural strength of the composite beam can be calculated from the plastic stress distribution on the composite section. But in that case the following limitations will need to be met:

a) The steel beam is compact and is adequately braced.
b) Steel headed stud or steel channel anchors connect the slab to the steel beam in the negative moment region.
c) The slab reinforcement parallel to the steel beam, within the effective width of the slab is properly developed.

In this study, the negative flexural strength of the composite beam was calculated from the steel beam alone. The concrete slab is subjected to tension at negative moment zone. As a result, the concrete slab cracks under negative moment. So for negative moment zone, composite action was ignored.

3.10.8 Design Flexural Strength and Allowable Strength

After calculating the nominal flexural strength of the composite beams, the design flexural strength (LRFD) can be obtained by multiplying the nominal moment M_n with a reduction factor Φ_b. The allowable moment capacity (ASD) can be obtained by dividing the nominal moment with a factor of safety Ω_b.

Where,

$\Phi_b = 0.90$ for design as per LRFD method and,

$\Omega_b = 1.67$ for design as per ASD method.

3.10.9 Design Steps for Composite Beams

In section 3.3.4 it was explained that; in this study, the composite beams were designed with flexible moment connections at their ends. In this case the beam was designed as a simply supported member. The steps below were followed to select and design a composite beam for the applied loads:

Step 1: At first design moment was calculated from the applied loads. In this step the beam end connections were considered to be released i.e. the beam was considered to be simply supported.

Step 2: A preliminary beam size was assumed from thumb rule. This preliminary beam was checked against applied loads. Three different checks are done by ETABS while designing a composite beam.

a) Strength check for construction load: If the beam is not shored during construction, then the steel beam will need to carry the loads during construction. Construction loads include self-weight of the steel member and deck and the weight of wet concrete.
b) Strength check for final loads: After the concrete has hardened and gained its strength the beam starts to act as composite. At this stage, other loads will be applied. So the composite beam is checked for finally applied loads.
c) Deflection check for final loads: When the selected member passes the strength check, it is then subjected to check for deflection.

Step 3: After the beam has been finally selected, the connections are designed. It has been discussed earlier that the end connections of the composite beams will be flexible moment connections. Geschwindner and Disque (2005) suggested in their paper, to design the flexible moment connections for lateral loads only. For this approach it is assumed that the leeward column and beam connection will be hinged. In this study, a single span frame has been considered. So at each storey, there are 2 connections between the beam and column. These connections will be so designed, that any one of them will be able to resist the total lateral load moment. To induce lateral load moment, earthquake load was applied in the longitudinal direction of the frame. The beam ends were considered fully fixed this time. At this condition beam end moment M_{eq} was recorded. As per BNBC 2006, earthquake load amplification factor for steel structure is 1.5. As stated earlier, only one connection will take the total lateral load moment. So the minimum connection design moment will be 2 x 1.5 x M_{eq}. The connections and the columns will be designed for this moment at first.

RC beams are monolithically casted with the columns. So the connection between a RC beam and column are always fixed. The objective of this study is to compare the performance of RC and composite beams. Designing the composite beam end connections only for lateral loads yields higher deflection under gravity loads. So for this study stronger connections were selected to reduce the deflection of the composite beams.

Two different conditions were considered for calculating the deflection of the composite beams. They are:

a) Fully released condition
b) End connections designed for 25 percent of the plastic moment capacity of bare steel beam.

The composite beam end connection was also designed with 50 percent of the plastic moment capacity of bare steel beam. However, for this stronger connection, beam end moment became higher and mid-span moment became lower. So, negative bending moment of the beam end governed the design. This is not a desired condition for composite beam. So the end connection was designed for up to 25 percent of the plastic moment capacity.

3.10.10 Deflection of composite beams

Like RCC beam, composite beam deflection can be subdivided into immediate and long-term deflection. Immediate deflection is the result of applying dead and live load on the beam. This deflection occurs right after the loads are applied. Long term deflection can be classified as shrinkage and creep deflection. The concrete slab over the steel beam tends to shrink and undergoes through creep over time. This causes the beam to deflect further with time.

Immediate deflection of composite beams

Deflection of composite beam is calculated for two stages: Construction stage and operating or final stage. At construction stage if the beam is not shored, deflection can be significant. During construction the beam acts as a bare steel member. Deflection is calculated for self-weight and wet concrete weight. This deflection is called pre-composite deflection. In this case the moment of inertia of the steel beam alone is used.

After concrete has gained at least 75 percent of its desired strength, the beam starts to act as composite. While in operating condition, additional loads are applied to the composite beam. So the beam deflection is calculated for the additional dead and live loads. This deflection is called post-composite deflection. Composite moment of inertia is used for calculating this deflection. The total immediate deflection of the composite beam is the sum of pre-composite and post-composite deflection.

$\Delta_{comp} = \Delta_{precomp.} + \Delta_{postcomp.}$ (3.26)

Where,

Δ_{comp}= Immediate deflection of composite beam

$\Delta_{precomp.}$= Pre-composite deflection of steel beam

$\Delta_{postcomp.}$= Post-composite deflection of composite beam

Long term deflection of composite beams

Long term deflection of composite beams occurs from the shrinkage and creep of overlaying RCC slab on steel deck. AISC does not provide any direct guideline for the computation of long-term deflection of composite beams. It refers to a simplified model to calculate the deflection due to shrinkage of concrete (Figure 3.16),

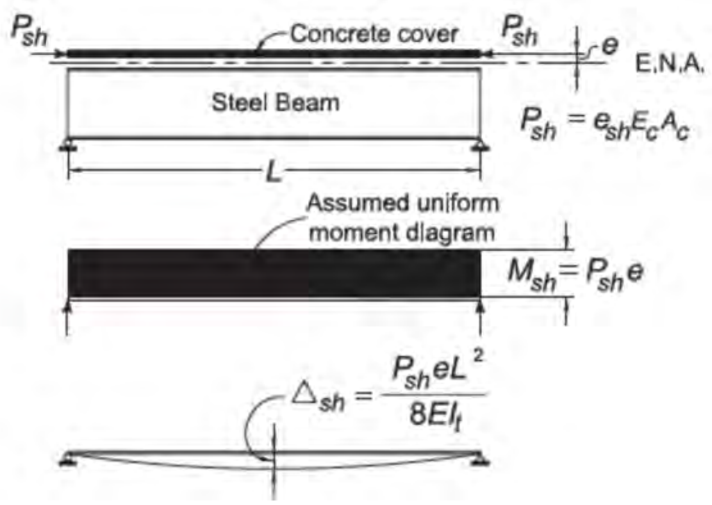


Figure 3.16 Calculation of shrinkage effects (AISC 2010)

In this model, effect of shrinkage is taken as an equivalent set of end moments given by the shrinkage force times the eccentricity between the center of the slab and the elastic neutral axis. The shrinkage strain for this calculation may be taken as 0.02%.

The long-term deformation of composite beams due to creep is usually small. However, this can be quantified using a similar model which is used for shrinkage deflection Chien and Richie (2005) proposed a method to calculate the deflection of composite beam due to creep. They suggested reducing the transformed moment of inertia of the composite beam to account for the creep in concrete. To implement this, a reduction in modulus of elasticity of concrete was proposed. So, $E_{c(reduced)} = E_c/2.5$. For overburden sustained load deflection was calculated for both the reduced and actual transformed moment of inertia. Deflection due to creep was calculated from the following equation:

$$\Delta_{cr} = \Delta_r - \Delta_a \tag{3.27}$$

Where,

Δ_{cr} = Deflection of composite beam due to creep

Δ_r = Deflection due to sustained load for reduced transformed moment of inertia

Δ_a = Deflection due to sustained load for actual transformed moment of inertia

Total deflection of composite beam can be calculated from the following equation:

$$\Delta_{c.total} = \Delta_{comp} + \Delta_{sh} + \Delta_{cr} \tag{3.28}$$

3.11 Camber of Beams

Cambering is in an effective and widely used procedure for reducing the deflection of a beam. Specially for steel beams it is a very popular method. In this method the beam is pre bent to adjust the deflection due to construction load.

If a composite beam is not shored during construction, it will deflect under self-weight of steel and concrete. For this study approximately 80% of the pre-composite deflection of composite beam was adjusted by cambering. Camber can be done in a steel member by two methods. Heat cambering and cold cambering. Minimum camber for composite beams was considered to be 0.75 inch.

It is very difficult to camber a RCC beam in real life. However, to match with the deflection calculation of composite beam, camber was included in the calculation for RCC beams.

3.12 Nonlinear Finite Element Analysis

One of the objectives of this study is to compare the ductility of RC and Composite beams. For this purpose, both of these beams were analyzed in ABAQUS, a nonlinear finite element based software. Three dimensional nonlinear finite element models for composite as well RC beams were developed and analyzed under two point loading condition. The flexural behavior of the beams were evaluated and compared.

3.12.1 Description of the Nonlinear FEM Models

For analysis in ABAQUS, previously designed sections in ETABS were chosen. For RC beams, similar amount of longitudinal and transverse reinforcement was used. For composite beams, similar steel section and effective slab width was used. For designing the beams 3 different span length were considered. For analysis in ABAQUS similar span length was chosen. RC and composite beams were designed for different end conditions. However, for analysis in ABAQUS, both RC and composite beams were considered to be simply supported. Both RC and composite beams were subjected to two points loading. Loads were applied vertically at one third distance from two supports of the beams (Figure 3.17).

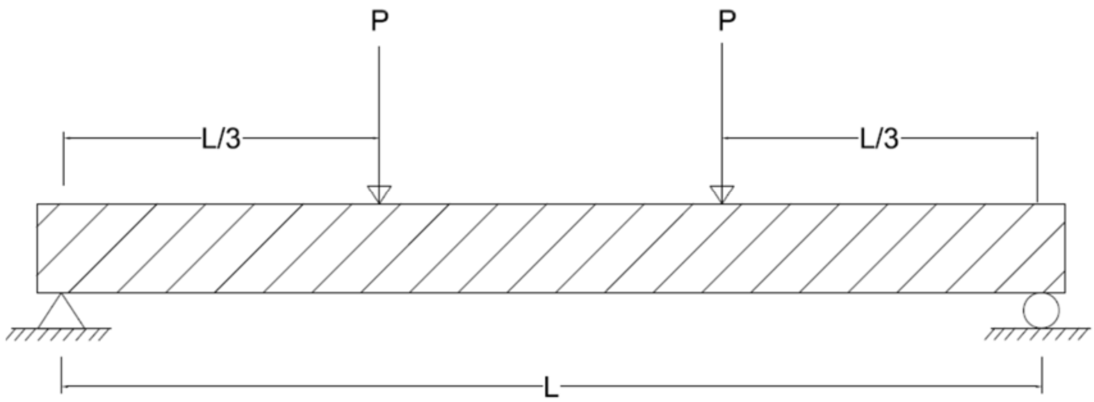
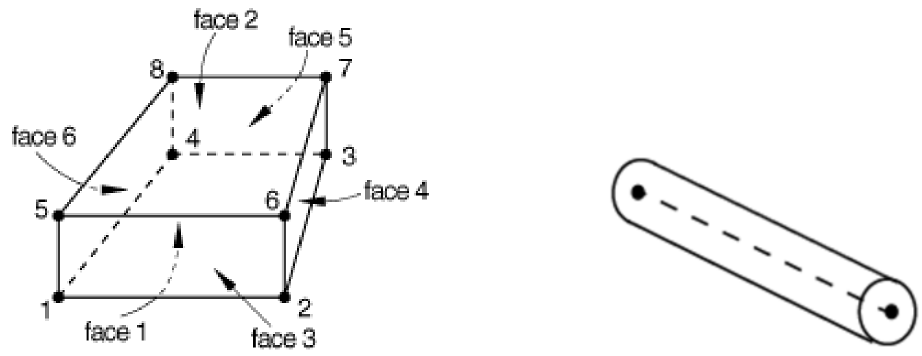


Figure 3.17 Loading condition of the beams in finite element model

Element Selection and Mesh Information for RC Beams

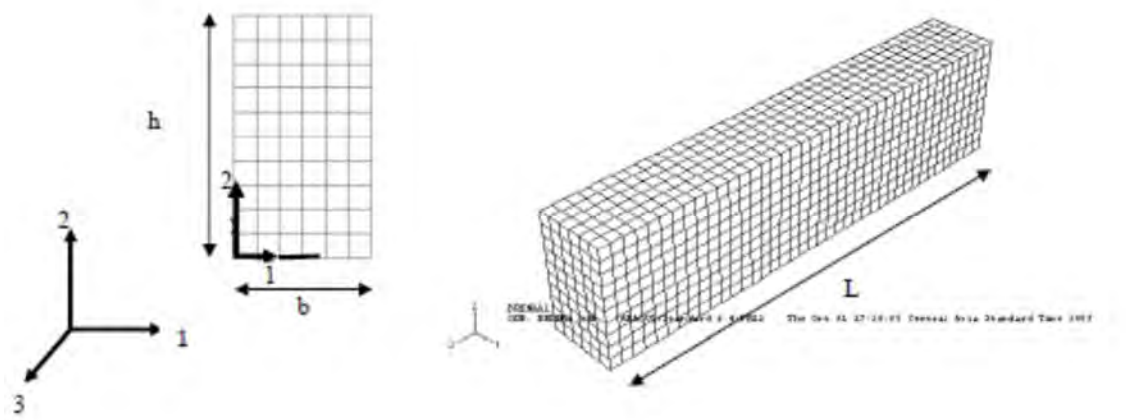
Solid element was used in ABAQUS to represent the RC beam. Element C3D8R was used to model the concrete beam. The C3D8R element is an eight-node reduced integration brick element. Each of the 8 nodes has three translational degrees of freedom. Element type T3D2 was used to model the longitudinal and transverse reinforcing bars. The T3D2 element is a three dimensional 2-node truss elements. In reality concrete and reinforcing bars act as bonded elements. To ensure proper bonding in ABAQUS, the reinforcing bars were defined as “embedded” element in the concrete. This definition effectively represents the bonding behavior of reinforcements with the surrounding concrete. Elements used in ABAQUS for RC beams are shown in Figure 3.18. A typical RC beam analysis model is shown in Figure 3.19.



a) Element C3D8R (8-node Solid) b) Element T3D2 (2-node Truss)

Figure 3.18 Finite Elements used in ABAQUS Simulation

The size of solid element mesh was 100mm x 100mm. For the reinforcing bars, 100 mm mesh length was used.



(b) Cross-Section **(a) 3D view**

Figure 3.19 Cross section and 3D view of RC beam with mesh

End Boundary Conditions for RC Beams

In the case of designing the RC beams, ends were considered to be fixed with RC columns. This means fixed supports were considered However, in ABAQUS model, simply supported beams subjected to two point loading (Figure 3.17) was considered to investigate the pure flexural behavior of the designed beams. To represent the simply supported boundary conditions the support points at the beam ends were hinged (by restraining the translation of the support nodes in longitudinal and transverse directions.

At the point of applied loads, nodes were modeled with rigid body elements. This was done for proper distribution of loads and to prevent local distortion of nodes.

Element Selection and Mesh Information for Composite Beams

In this study the composite beam was modeled using the method proposed by Katwal et al. (2015). The steel beam was modeled using beam element B31. Shell elements with reduced integration S4R was used to represent the deck slab. The composite profiled slab was modeled using a simplified method proposed by Jeyarajan et al. (2015). The method is shown in Figure 3.20.

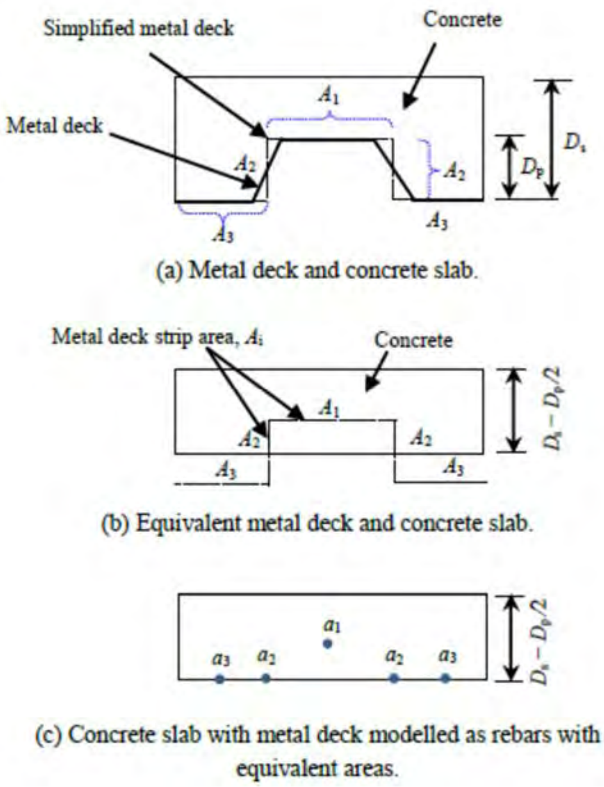


Figure 3.20 Simplified model for composite slab proposed by Jeyarajan et al. (2015)

In this method, deck slab was converted into an equivalent concrete slab with uniform thickness of D_s - $D_p/2$.

Where,
D_s= Slab depth
D_p= Height of the steel decking

The steel decking is simplified as shown in Figure 3.20 (a). The corresponding steel deck strip areas A1, A2 and A3 were calculated from the product of steel of deck thickness and corresponding strip length. The deck strips are modeled as rebars as shown in Figure 3.20 (c). The equivalent area of rebar a_i is determined from the following equations,

$a_1 = A_1$

$a_2 = A_2$ and,

$a_3 = A_3 \times D_s^2 / [(D_s - D_p/2)^2]$

A four-node homogeneous shell element with reduced integration, S4R was used to model the slab. Rebars were defined using REBAR LAYER option available in ABAQUS. Slip between the concrete and metal deck was neglected. Tie constraint was used to represent interaction between the composite slab and steel beam. At the end of the composite beams, vertical and horizontal restraint was applied.

Mesh size of 150x150mm was used for both the beam and shell elements.

3.12.2 Material Properties Used for ABAQUS Model

Concrete compressive strength was considered to be 27.5 MPa. Yield strength of rebar and structural steel was considered to be 415 MPa and 345 MPa respectively. For modeling the exact behavior of steel, true stress-strain curve was used. Damage plasticity model in ABAQUS was used to simulate the nonlinear material behavior of concrete.

Steel

Steel is a ductile material. While loaded, steel experiences large inelastic strain beyond the yield point. For modeling the exact material behavior of steel, it is necessary to use the true stress and logarithmic strain graph. This is a bilinear curve which is also called strain hardening curve. It shows the variation of true stress with plastic strain.

For a isotropic material, if nominal stress strain data for uniaxial test is available then a simple conversion (Lubliner 1990) to true stress and logarithmic plastic strain is possible using the following equations:

$$\sigma_{true} = \sigma_{nom} (1 + \varepsilon_{nom}) \qquad (3.29)$$

$$\varepsilon^{pl}_{ln} = \ln (1 + \varepsilon_{nom}) - (\sigma_{true} / E) \qquad (3.30)$$

Here, E is Young's modulus of the material

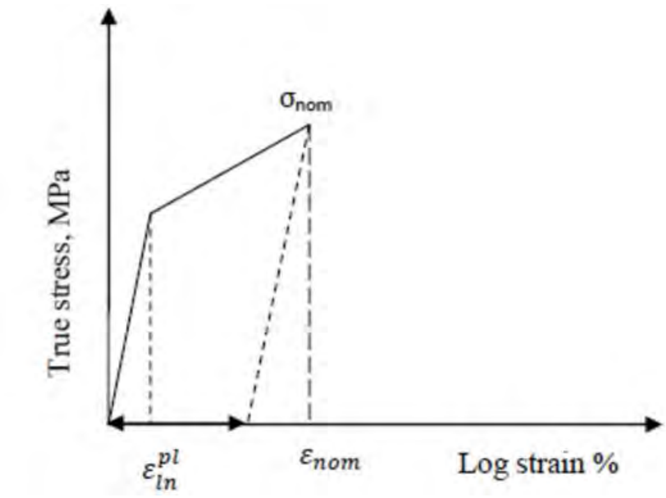


Figure 3.21 Stress-Strain curve for steel

Poisson's ratio of steel was considered to be 0.3. Modulus of elasticity of steel is constant, which is 200 GPa. ABAQUS also needs corresponding plastic strain values at the starting point of yield and ultimate stress.

Concrete in Compression

To simulate the behavior of concrete material in RC beams, available damage plasticity model in ABAQUS was used. Two types of plasticity damage based material stress-strain behavior were used. One is for compression hardening and the other is for tension stiffening. The damage plasticity model uses a non-associated plastic flow rule to describe the plastic strain increments. A value of 15 degrees was defined for the dilation angle of concrete. Dilation angle is used to identify the plastic strain direction relative to the gradient of the yield surface. The stress-strain curve of concrete in compression (Figure 3.22) was defined by the model proposed by Carriera and Chu (1985).

Carriera and Chu (1985) model was used to define the complete stress-strain relationship of concrete beyond the elastic limit. The following equations were used:

$$\frac{f_c}{f'_c} = \frac{\beta\left(\frac{\varepsilon}{\varepsilon'_c}\right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon'_c}\right)} \tag{3.31}$$

$$\beta = \left[\frac{f_{cu}}{32.4}\right]^3 + 1.55 \tag{3.32}$$

The compressive behavior of concrete up to linear elastic portion was defined using the modulus of elasticity in compression. Elastic limit for normal strength concrete is assumed to be 30% of its compressive strength. The effective stress-plastic strain curve for plastic region, was developed using the stress-strain function proposed by Carriera and Chu (1985) in uniaxial compression.

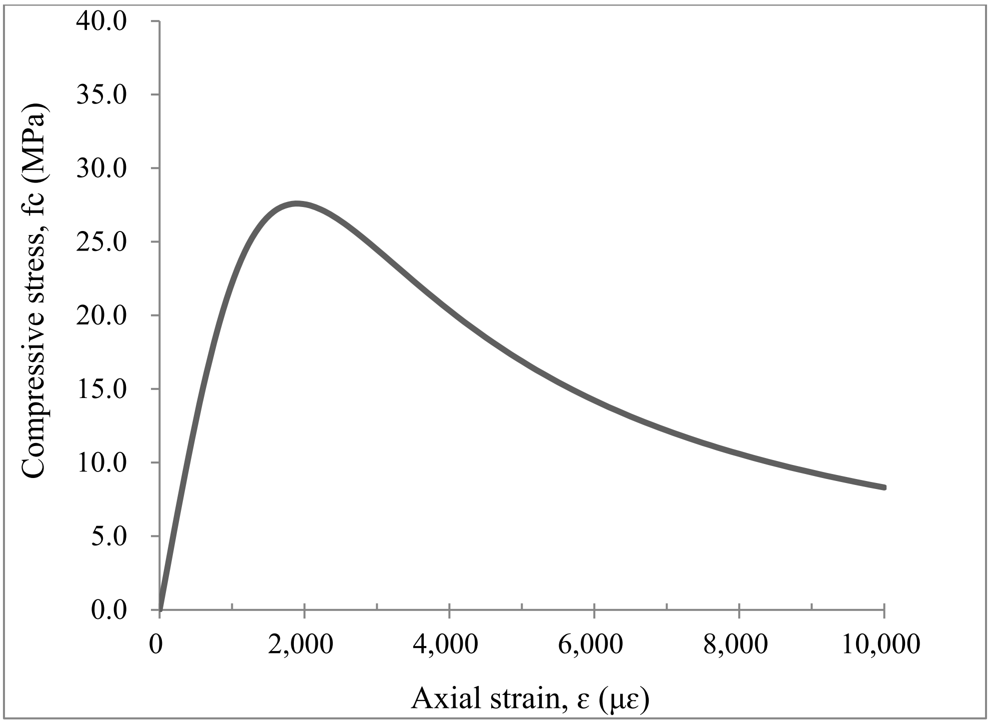


Figure 3.22 Stress-strain curve for concrete in uniaxial compression

Concrete in Tension

Concrete tension properties for damage model are defined in two stages: The linear Elastic portion up to the tensile strength and the nonlinear post peak portion or tension stiffening portion. The linear part is represented using the Modulus of Elasticity of concrete and the uniaxial tensile strength of concrete. Uniaxial tensile strength of concrete f'_t was set to be 10% if the uniaxial compressive strength, $f'c$, for the normal strength concrete as observed by Marzouk and Chen (1995). To generate the tension stress-strain diagrams for nominal strength concrete the following equation was used, which was proposed by Carreira and Chu (1985).

$$\mathrm{f_c} = \mathrm{f'_t}\left[\frac{\beta\left(\frac{\varepsilon}{\varepsilon_{tu}}\right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_{tu}}\right)\left(\frac{\varepsilon}{\varepsilon_{tu}}\right)^{\beta}}\right] \tag{3.33}$$

Value of β was calculated from the equation proposed by Carreira and Chu (1985). Figure 3.23 represents the concrete tensile stress-strain curve used for analysis in ABAQUS.

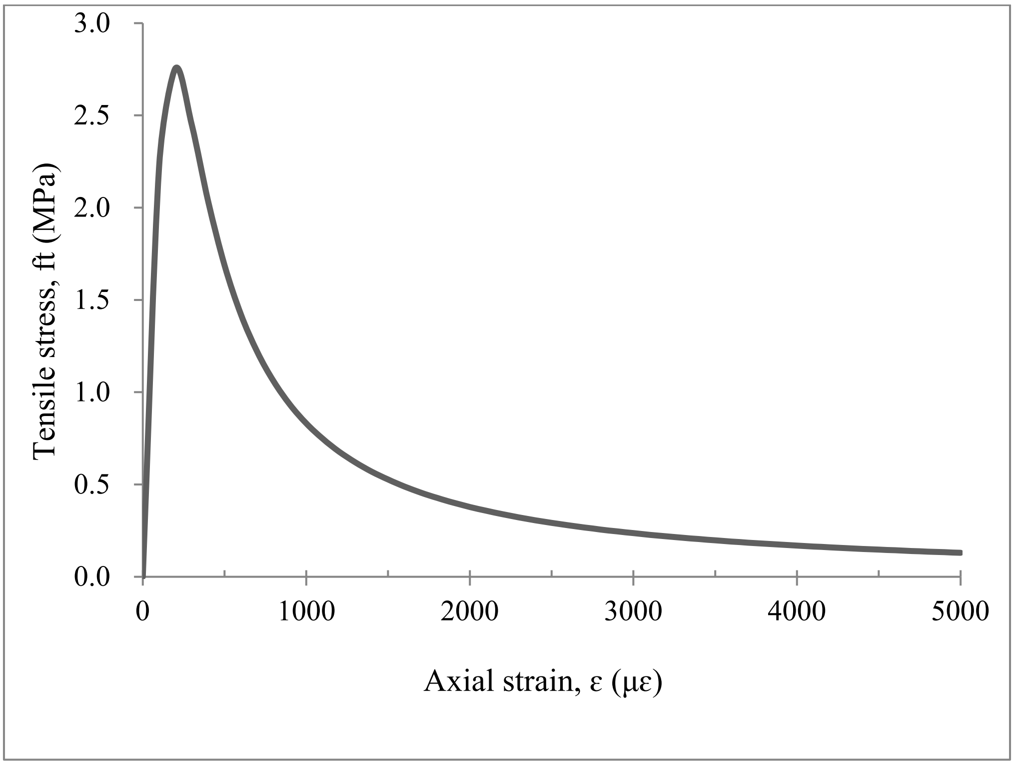


Figure 3.23 Stress-Strain curve for concrete in tension

3.12.3 Load Application and Solution Strategy

In the finite element analysis both RC and composite beams were subjected to two point loading with a view to obtain their load versus displacement curve and failure behavior under pure flexure. The beams were considered to be simply supported. The load was applied through displacement control technique at the one third point of the beam as shown in Figure 3.17. Modified Newton Raphson solution strategy was implemented to trace the load versus displacement response of the simulated beams.

CHAPTER-4

RESULTS AND DISCUSSION

4.1 Introduction

The objective of this study was to compare the performance of RC and composite beams for long span structures. For this purpose, three RC beams and three composite beams were designed following the guidelines provided by ACI (2008) and AISC (2010). Design sections of the beams were selected based on the serviceability and strength criteria as specified in the relevant codes. Several trials were made to obtain the optimum design section for both RC and composite beams. However, similar beam depth was considered for the design to make the comparison realistic. The results of the study are presented in this chapter. Initially, the selected size of RC and composite members for three different span length (18m, 24m and 30m) is presented followed by the linear and nonlinear behavior obtained from the finite element analyses. From the linear elastic analysis performed using ETABS (2016) the design capacities and deflections for both floor systems are reported and compared with each other. Moreover, the moment versus vertical deflection curves, ultimate capacities and corresponding deflections for the designed beams obtained from the nonlinear finite element analysis (as performed using ABAQUS) (HKS 2014) are presented and compared. Finally, a cost comparison is made for the two floor systems (RC and composite) for various span lengths in the context of Bangladesh.

4.2 Design Section for RC Beams

ACI (2008) recommendations were followed to design the RC beams. The rebar area and number of bars were selected in a way, so that the finally designed beams meet all the criteria of code. Both strength and constructability issues were considered. Three spans were considered for the analysis and design. Three different beam sizes were selected for these spans. Table 4.1 shows the selected beams for RC structures.

Table 4.1- Selected RC beams for different spans

Span (m)	Design Code	Beam depth (mm)	Beam width (mm)	Deflection Limit [L/240] (mm)
18		900	600	75
24	ACI 318-08	1200	750	100
30		1500	750	125

Figure 4.1 shows the general detail of an RC beam. It shows the beam size and general reinforcing. Also it shows the required reinforcement at different locations of the beam. For clarity, only the qualitative reinforcement detail has been shown in this figure. For these beam design, specified concrete strength f'c was considered to be 27.5 Mpa. Minimum yield strength of reinforcing steel, Fy= 415 MPa. Modulus of Elasticity of concrete was 24.8 GPa. During rebar placement, it was not possible to provide the exact amount of steel in the beams. So provided reinforcement in the finally selected concrete beam is slightly higher than that required by actual calculation. Minimum clearance requirements (provided by ACI) for bars were maintained while selecting the size and number of bars.

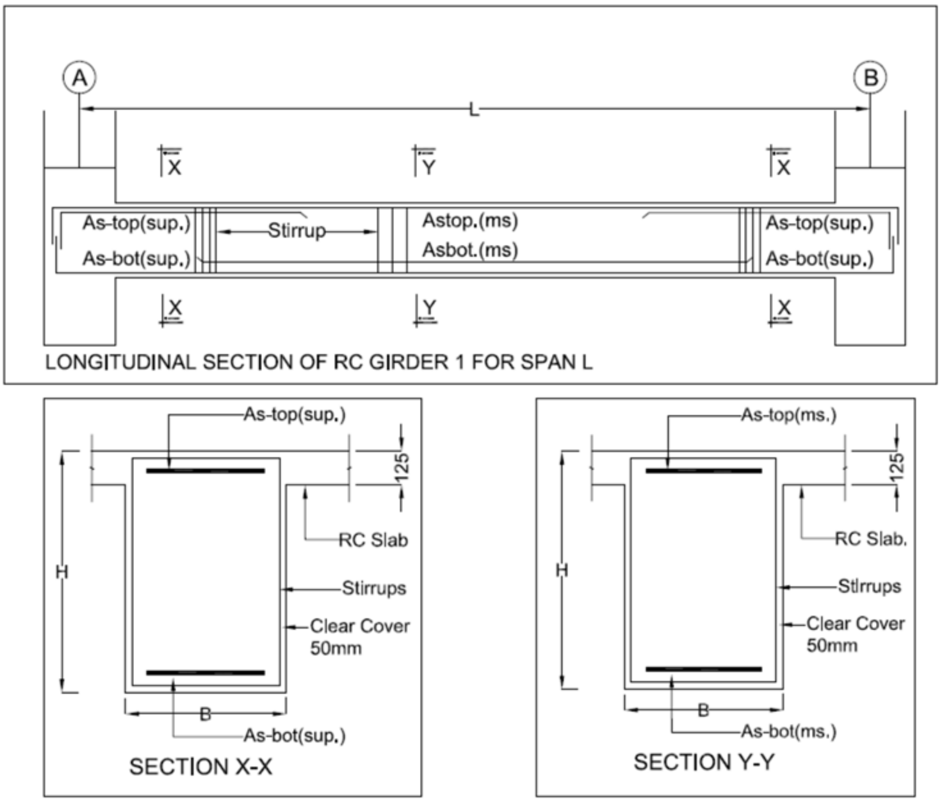


Figure 4.1 General Detail of RC beam for Span L

In Figure 4.1,

H = Depth of RC beam

B= Width of RC beam

Typical Slab Thickness for all span of girder =125 mm (5 inch)

Clear cover = 50 mm (2 inch)

As-top(ms.) = Area of reinforcement at top of beam for mid-span

As-bot(ms.) = Area of reinforcement at bottom of beam for mid-span

As-top(sup.) = Area of reinforcement at top of beam near support

As-bot(sup.) = Area of reinforcement at bottom of beam near support

Number and legs of stirrups were provided as required by the analysis and design.

While selecting the number of bars to allocate the required area of reinforcement, it was ensured that code required minimum clear spacing between beam longitudinal bars were maintained. Required reinforcement areas for RC beams are listed in Table 4.2, 4.3 and 4.4. Figure 4.2, 4.3 and 4.4 show the reinforcement details for 18M, 24M and 30M span RC beams. Both longitudinal and transverse rebar are shown in the figures. Two cross sections X-X and Y-Y are shown for clarity.

Percentage of reinforcement in RC beams for different spans can be obtained from table 4.2, 4.3 and 4.4. It is observed that the required reinforcement and percentage is high for all the RC beams specially at the top of beams near supports. Large diameter bars are used to accommodate the required reinforcement area. It ensures code specified spacing between the Longitudinal bars.

Table 4.2: Area of Reinforcement at different locations of beams (18m Span)

Beam Span	18m		
Beam Size	900 mm x 600 mm (36 inch x 24 inch)		
Reinforcement Location	Required Reinforcement (mm^2)	Provided Reinforcement (mm^2)	Reinforcement Percentage %
As-bot (midspan)	10612	10938	2.025
As-top (midspan)	3350	4938	0.914
As-bot (support)	7356	7812	1.446
As-top (support)	14769	14812	2.743

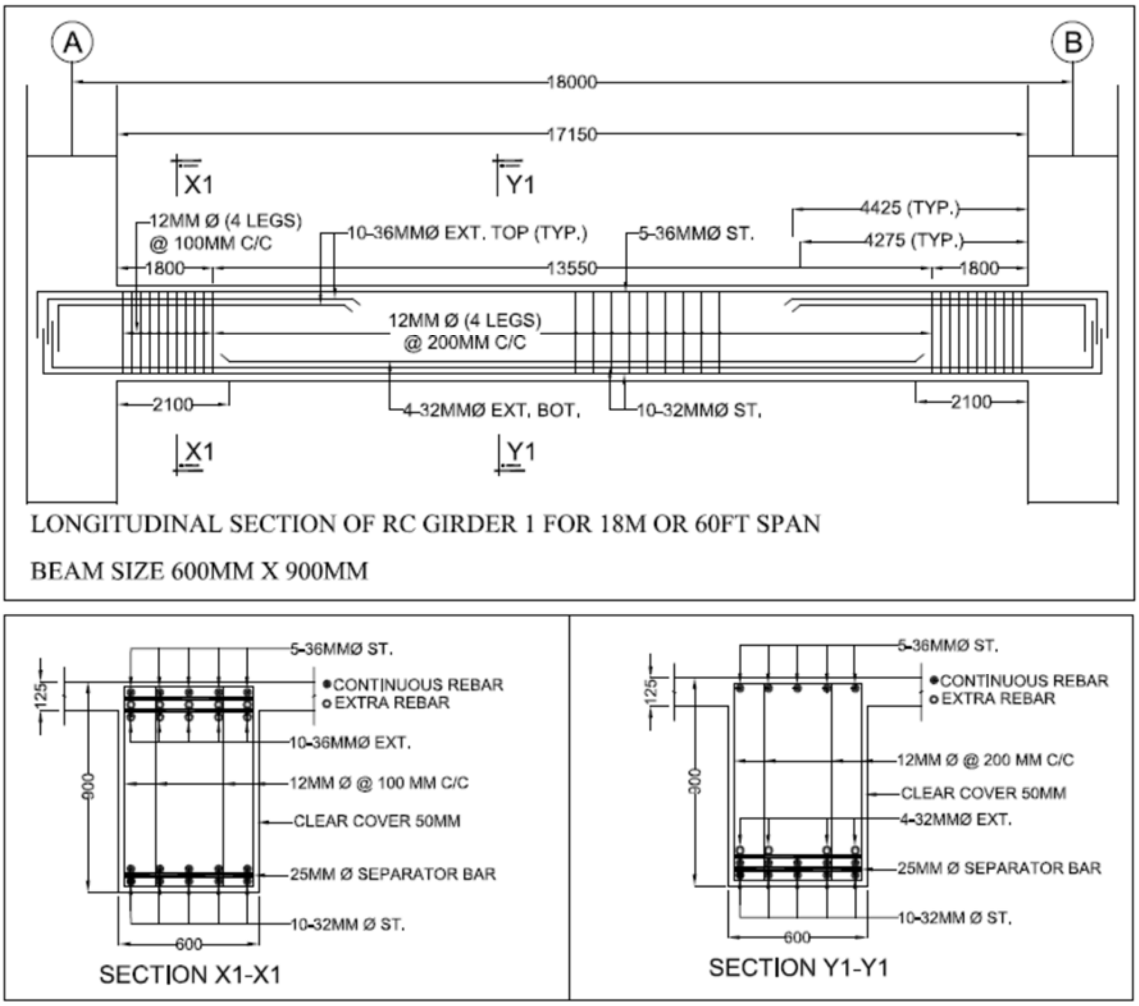


Figure 4.2 Reinforcement detail of RC beam for 18m (60ft) span

Table 4.3: Area of Reinforcement at different locations of beams (24m Span)

Beam Span	24m		
Beam Size	1200 mm x 750 mm (48 inch x 30 inch)		
Reinforcement Location	Required Reinforcement (mm^2)	Provided Reinforcement (mm^2)	Reinforcement Percentage %
As-bot (midspan)	15612	16406	1.823
As-top (midspan)	4800	4800	0.533
As-bot (support)	8612	9375	1.042
As-top (support)	20663	20738	2.304

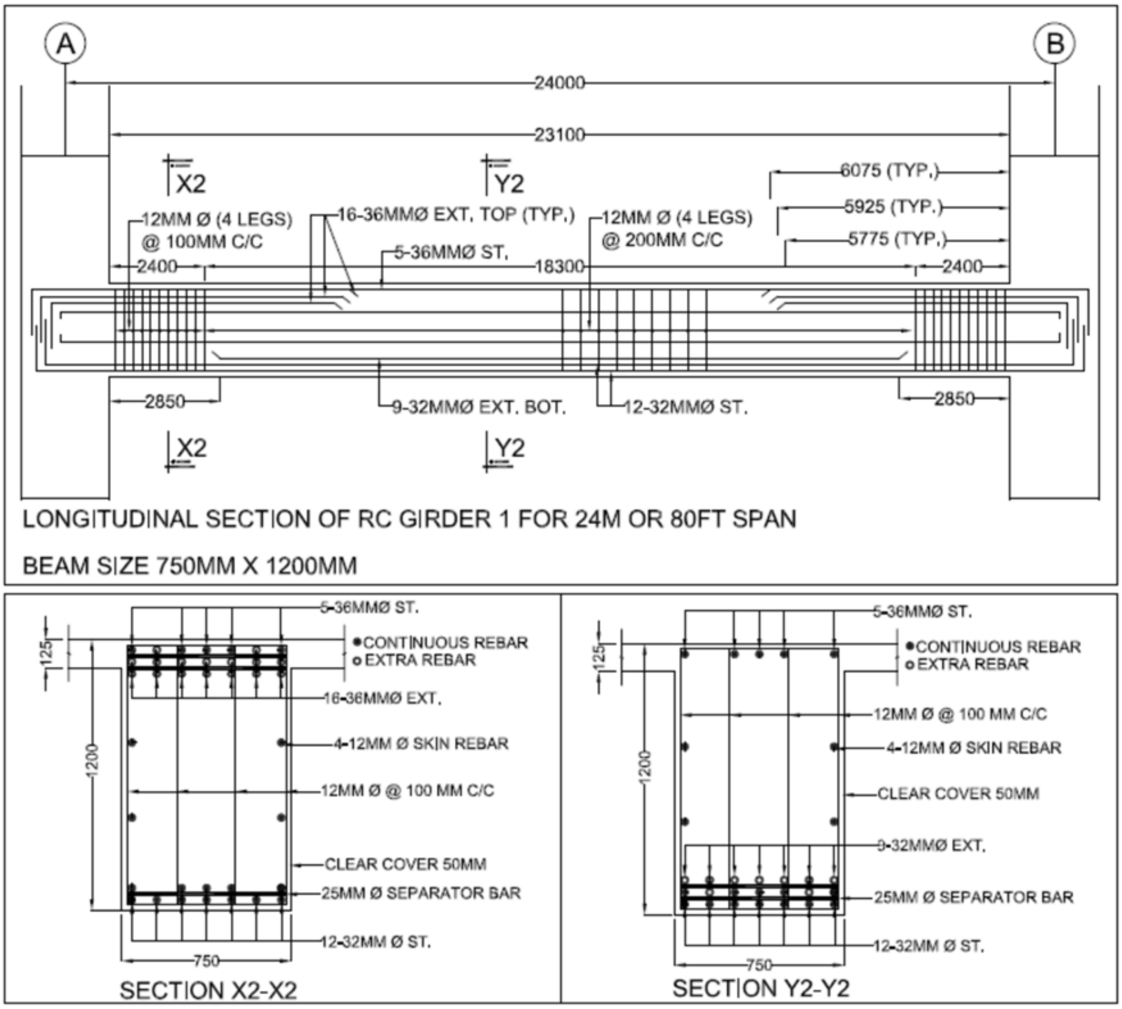


Figure 4.3 Reinforcement detail of RC beam for 24m (80ft) span.

Table 4.4: Area of Reinforcement at different locations of beams (30m Span)

Beam Span	30m		
Beam Size	1500 mm x 750 mm (60 inch x 30 inch)		
Reinforcement Location	Required Reinforcement (mm^2)	Provided Reinforcement (mm^2)	Reinforcement Percentage %
As-bot (midspan)	15794	15800	1.404
As-top (midspan)	5481	5625	0.5
As-bot (support)	8525	9875	0.878
As-top (support)	24213	25313	2.25

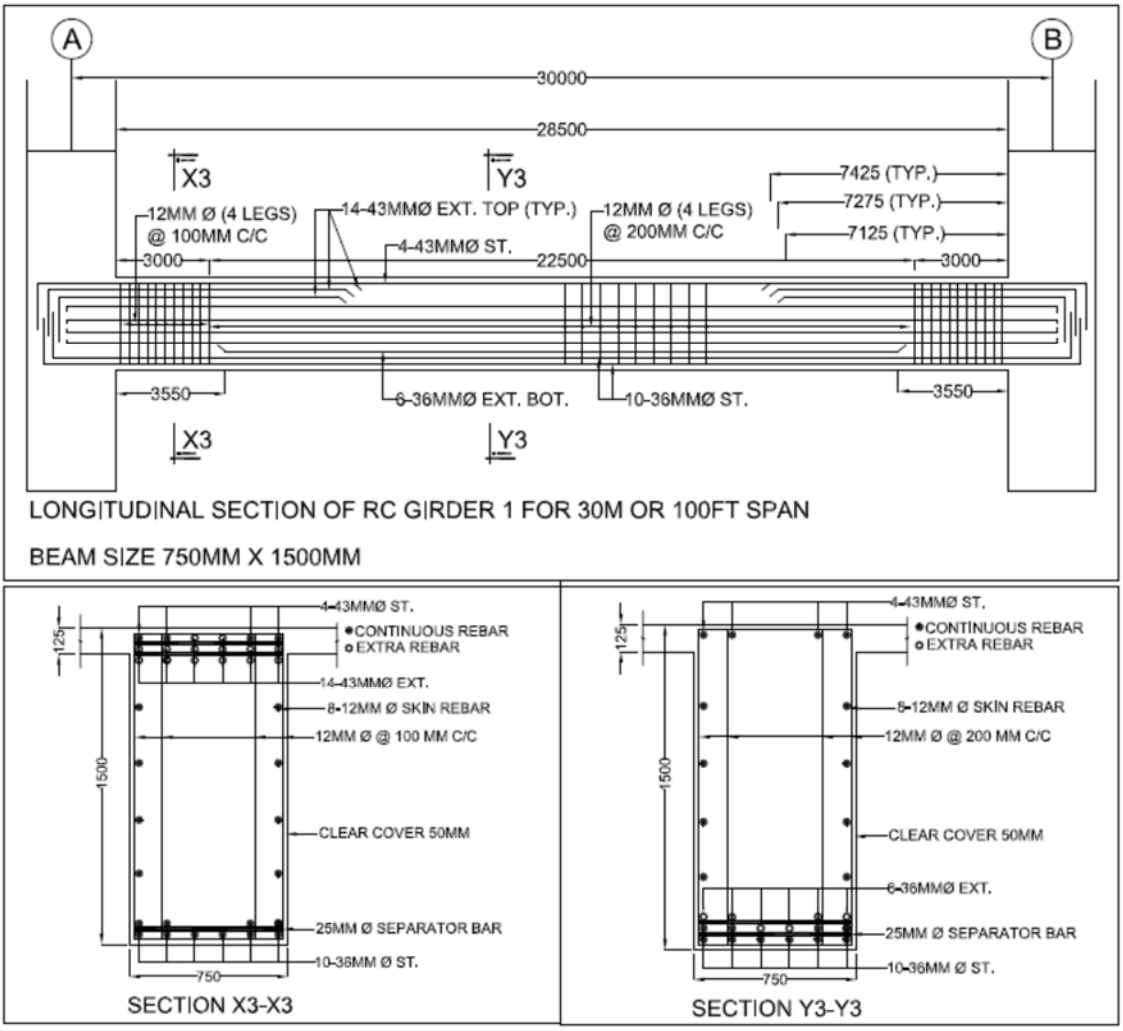


Figure 4.4 Reinforcement detail of RC beam for 30m (100ft) span.

4.3 Design Section for Composite Beams

For the RC beams, slab and sub beams were kept same throughout the analysis process. The deck slab and the secondary beams were kept same for all three spans of composite beam. The aim of the analysis and design process was to select an optimum steel beam that will satisfy the strength and serviceability requirement of the composite framing.

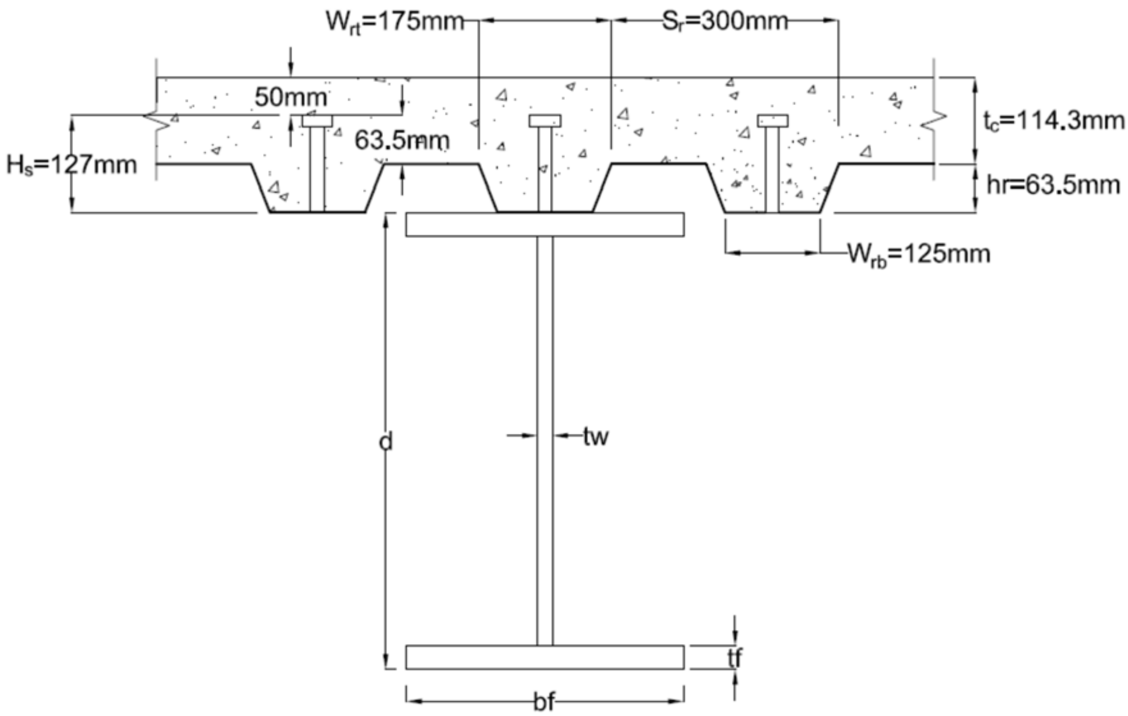


Figure 4.5A Typical Detail of Composite beam

Figure 4.5A shows a typical detail of composite beam. Steel beam section size and thickness are shown in the figure. Shear stud placement details are illustrated in Figure 4.5B and Figure 4.5C.

Here,

d = Depth of steel beam

b_f = Width of steel beam

t_f = Flange thickness of steel beam

t_w = Web thickness of steel beam

For different spans; depth, width and thickness of steel beam varies. Selected beam sizes for different spans are listed in Table 4.5A. Shear stud specifications are listed in Table 4.5B.

Table 4.5A- Selected Composite beams for different spans

Span (m)	d (mm)	b_f (mm)	t_f (mm)	t_w (mm)
18	900	350	30	12
24	1200	400	30	16
30	1500	475	32	18

Table 4.5B- Shear Stud Details

Span "L" (m)	Number of Shear Studs		Spacing X (mm)	
	50% Composite Action	100% Composite Action	50% Composite Action	100% Composite Action
18	126	260	140	140
24	172	360	135	135
30	228	470	130	130

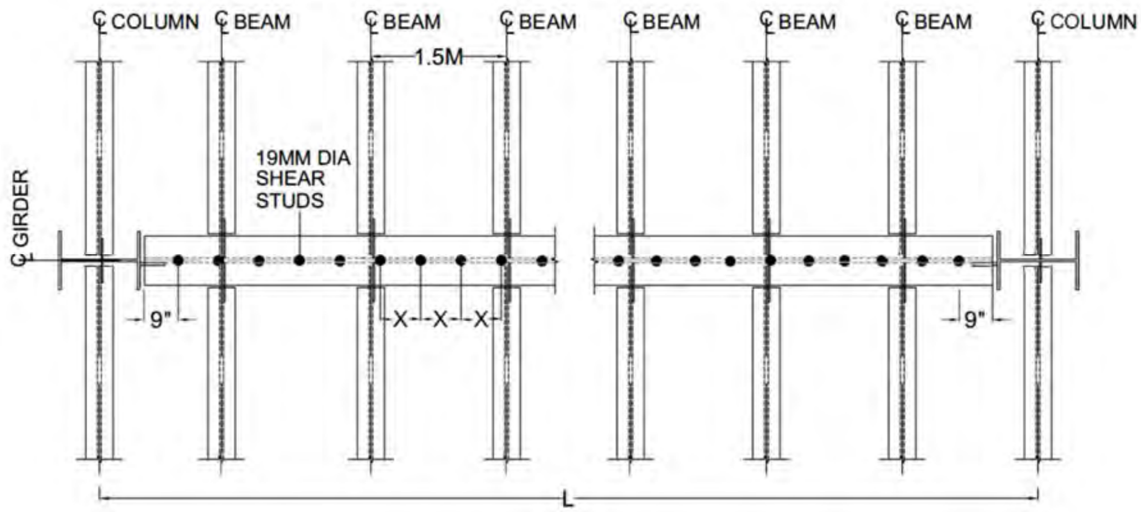


Figure 4.5B Typical Layout of Shear Studs for Composite Beam (Single Stud)

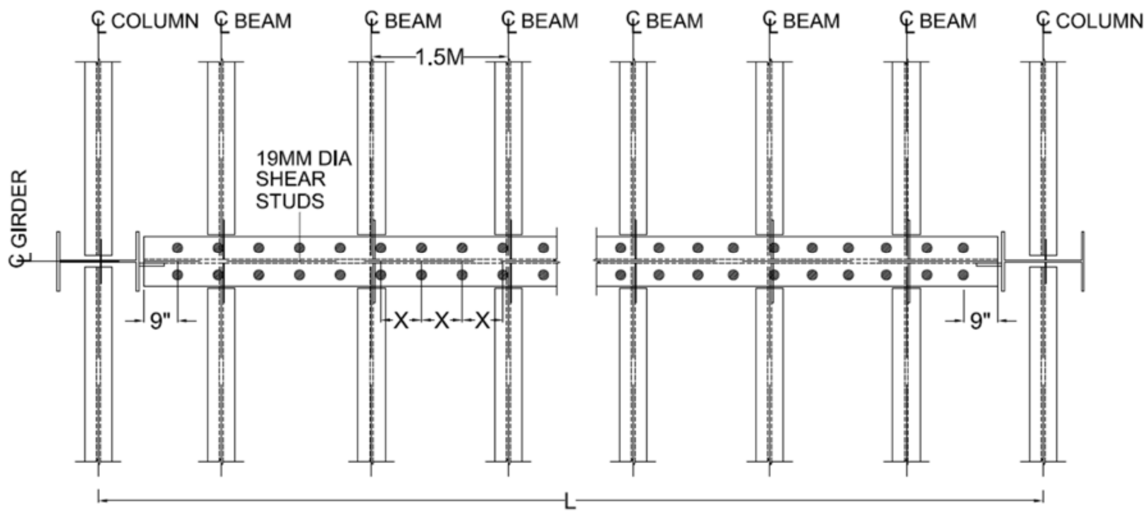


Figure 4.5C Typical Layout of Shear Studs for Composite Beam (Double Stud)

4.4 Deflection of Beams

Deflection of RC and composite beams were calculated for similar type of loading. As stated in chapter 3, immediate and long-term deflections were calculated. Immediate deflection includes deflections due to self-weight, additional dead load, live load and long-term deflection includes deflections due to creep and shrinkage.

4.4.1 Deflection of RC Beams

Deflection of RC beams for different spans are listed in table 4.6. The following notations have been used for deflection types.

DL= Deflection due to self-weight

SDL= Deflection due to super imposed dead load

LL= Deflection due to live load

I.DEF= Immediate deflection

LT.D= Long term deflection i.e. deflection due to creep and shrinkage

CAMBER= Pre-bending of beams to reduce self-weight deflection

TOTAL= Total deflection including long term effect – Camber

Table 4.6- RC beam deflection table

Span	**Depth**	**DL**	**SDL**	**LL**	**DL+SDL**	λ	CR+SH	Camber	Total (mm)
m (ft)	mm	**1**	**2**	**3**	**4**	**5**	**6=(4X5)**	**7**	**(1-7+2+3+6)**
18 (59)	900	31	14	23	45	1.37	62	25	105
24 (79)	1200	40	16	26	56	1.57	87	31	137
30 (98)	1500	52	18	30	70	1.60	112	38	174

In table 4.6 total deflection was calculated by subtracting the camber value from the sum of immediate and long-term deflection. Total, immediate and long-term deflection of RC beams is summarized in table 4.7. Results are graphically presented in Figure 4.6. The lower most line in the graph represents the immediate deflection values for 18m, 24m and 30m spans. The line in the middle is for long term deflection and the line at the top represents the total deflection i.e., the sum of immediate and long-term deflection.

Table 4.7- RC beams deflection summary

Span m (ft)	Total Deflection (mm)	Immediate Deflection, mm (%)	Long Term Deflection, mm (%)
18 (59)	105	43 (*41%*)	62 (*59%*)
24(79)	137	50 (*36%*)	87 (*64%*)
30 (98)	174	62 (*36%*)	112 (*64%*)

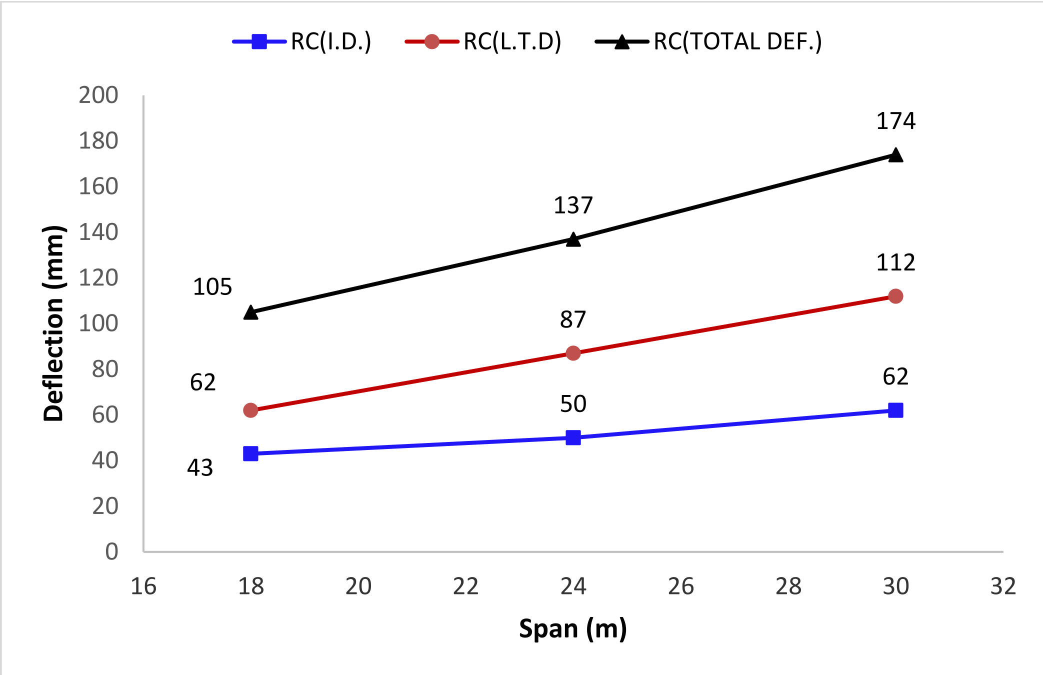


Figure 4.6 Deflection of RC beams for different spans

From Table 4.7 it can be observed that, for all three spans, long term deflection is higher (about 62% higher on an average) than immediate deflection. The reason behind this is the creep and shrinkage of concrete with time.

Long-term deflection of RC beam is approximately 1.5 to 2.0 times the immediate deflection. Effect of long-term deflection increases with the increase of span. Percent contribution of long-term deflection in the total deflection is higher for 24M and 30M span compared to 18M span. For 18M span contribution of immediate and long-term deflection in the total deflection is 41% and 59% respectively. For 24M and 30M span contribution of immediate and long-term deflection in the total deflection is 36% and 64%, respectively.

It can be concluded that with the increase of span, deflection of RC beam increases with a higher gradient. It is difficult to meet serviceability criteria for long term deflection for long span RC beams.

4.4.2 Deflection of Composite Beams

Deflections of composite beams were calculated for three types of end connections. Fully released or simple connection, partially restrained or flexible moment connection and fully restrained or fixed end connection. For partially restrained connection, fixity corresponding to 25% plastic moment capacity of steel beam was considered at the end of steel beams. For the purpose of comparing the deflection values of RC and composite beams, 50 Percent composite actions were considered for all type of connections. Table 4.8, Table 4.9 and Table 4.10 contains the deflection values for composite beams with 50% composite action. Table 4.8 shows the deflection for fully released end connections, Table 4.9 and Table 4.10 contain deflection (mm) values for partially fixed and fully fixed connection respectively. In case of RC beams, combined deflection for shrinkage and creep was calculated. But for composite beams, creep and shrinkage deflection were calculated and shown separately in the tables. For graphical presentation, creep and shrinkage deflections are shown together as long term deflection.

Table 4.8- Composite beam deflection table (For fully released end condition)

Span	Depth	DL	SDL	LL	DL+SDL	SH DEF	CR DEF	Camber	Total (mm)
m (ft)	mm	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**(1-7+2+3+5+6)**
18 (59)	900	40	18	30	58	11	3	32	70
24 (79)	1200	61	27	45	88	15	5	44	108
30 (98)	1500	76	34	56	109	18	6	57	133

Table 4.9- Composite beam deflection table (For partially fixed end condition)

Span	Depth	DL	SDL	LL	DL+SDL	SH DEF	CR DEF	Camber	Total (mm)
m (ft)	mm	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**(1-7+2+3+5+6)**
18 (59)	900	26	12	19	37	11	2	19	50
24 (79)	1200	39	18	30	57	15	3	32	73
30 (98)	1500	50	22	39	73	18	4	38	95

Table 4.10- Composite beam deflection table (For fully fixed end condition)

Span	Depth	DL	SDL	LL	DL+SDL	SH DEF	CR DEF	Camber	Total (mm)
m (ft)	mm	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**(1-7+2+3+5+6)**
18 (59)	900	10	5	14	15	11	1	9	32
24 (79)	1200	16	6	20	22	15	1	13	45
30 (98)	1500	20	9	24	29	18	2	15	57

Total, immediate and long-term deflection of composite beams are summarized in Table 4.11, 4.12 and 4.13. Deflections are graphically shown in figure 4.7, 4.8 and 4.9.

Table 4.11- Composite beams deflection summary (Released end connections)

Span m (ft)	Total Deflection (mm)	Immediate Deflection, mm (%)	Long Term Deflection, mm (%)
18 (59)	70	56 (*80%*)	14 (*20%*)
24(79)	108	89 (*82%*)	19 (*18%*)
30 (98)	133	109 (*82%*)	24 (*18%*)

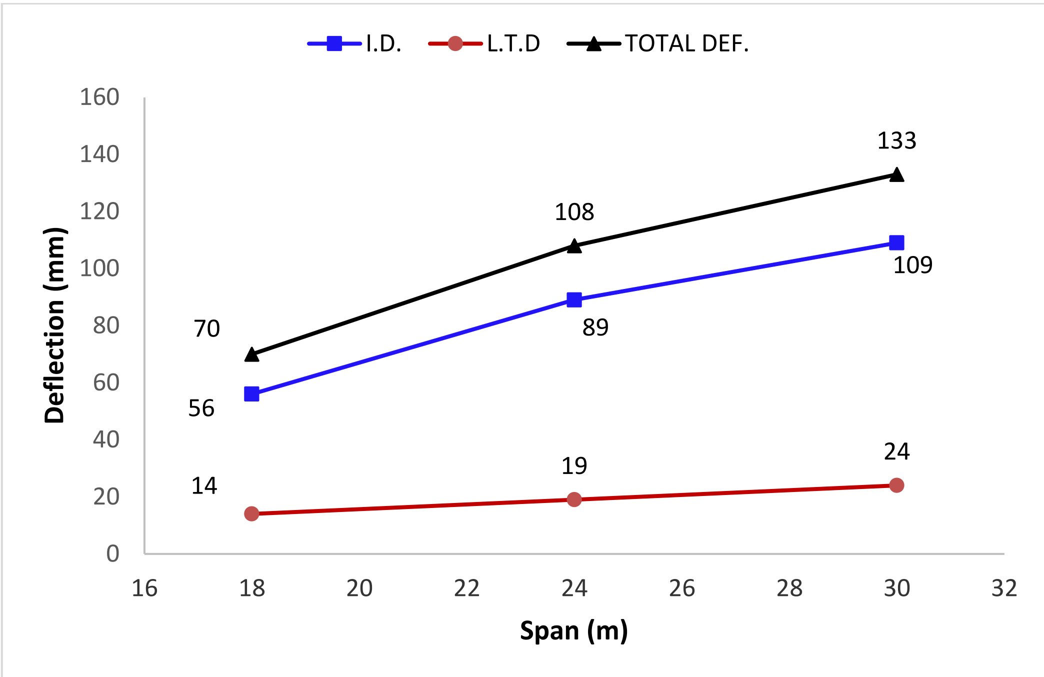


Figure 4.7 Deflection of Composite beam for different spans (End Released Condition)

Table 4.12- Composite beams deflection summary (Partially fixed end connections)

Span m (ft)	Total Deflection (mm)	Immediate Deflection, mm (%)	Long Term Deflection, mm (%)
18 (59)	50	37 (*75%*)	13 (*25%*)
24(79)	73	55 (*75%*)	18 (*25%*)
30 (98)	95	73 (*77%*)	22 (*23%*)

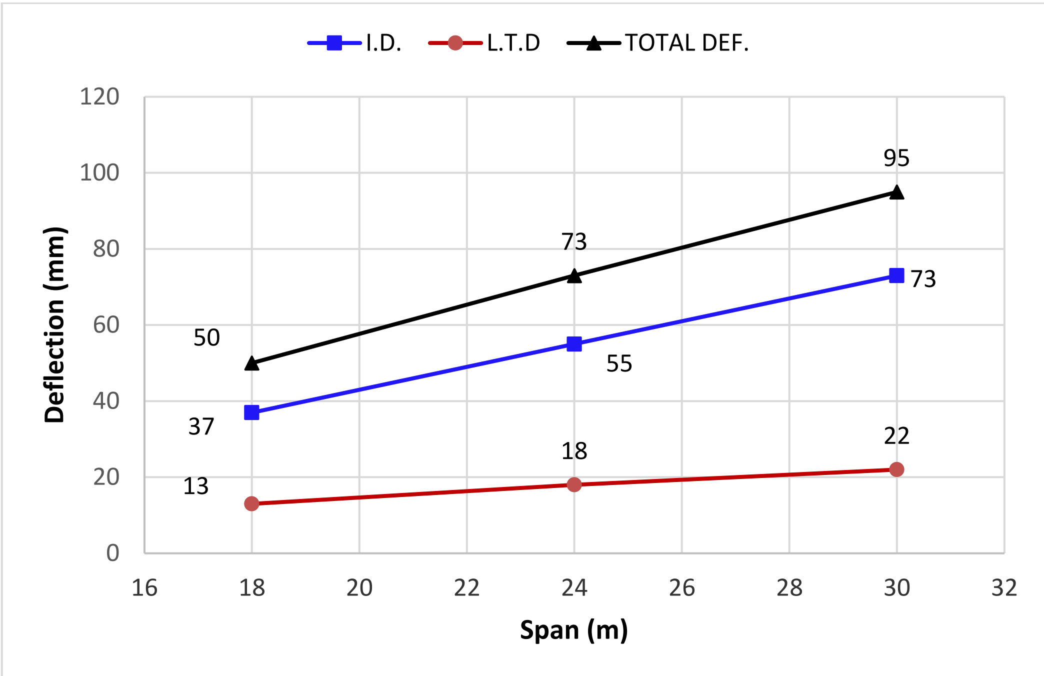


Figure 4.8 Deflection of Composite beam for different span (Partially Fixed Condition)

Table 4.13 Composite beams deflection summary (Fully Fixed end connections)

Span m (ft)	Total Deflection (mm)	Immediate Deflection, mm (%)	Long Term Deflection, mm (%)
18 (59)	32	21 (*65%*)	11 (*35%*)
24(79)	45	29 (*65%*)	16 (*35%*)
30 (98)	57	37 (*65%*)	20 (*35%*)

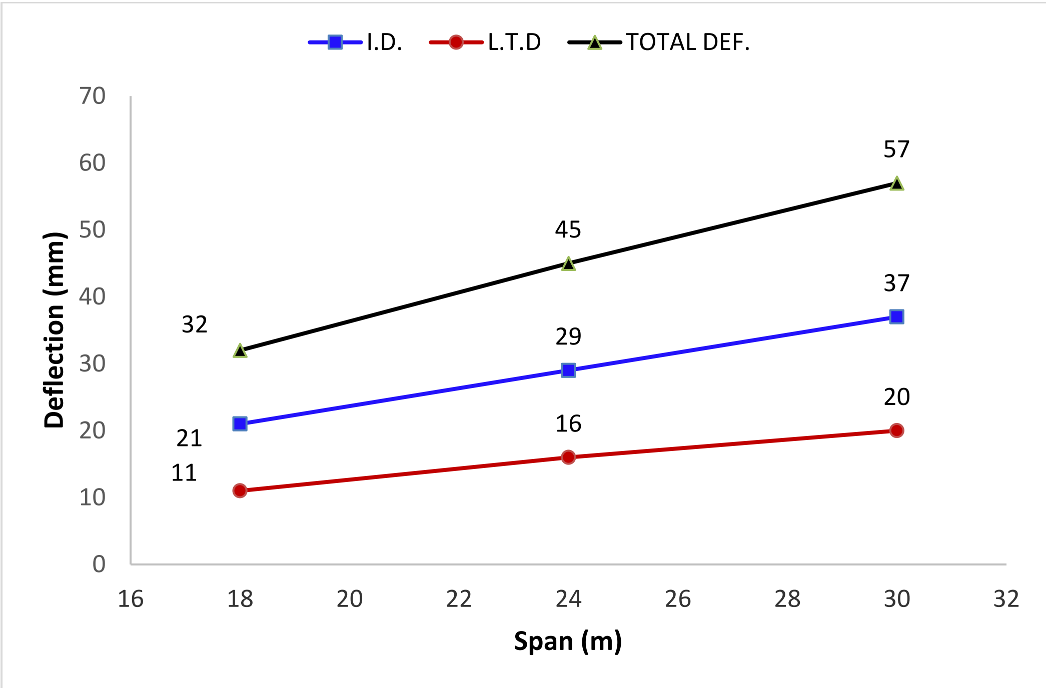


Figure 4.9 Deflection of Composite beam for different spans (Fixed end Condition)

Composite beam deflection analysis

Simply supported composite beams have no rotational restraint. Therefore, deflections of these beams are highest as compared to fixed and partially restraint connections. About 80% of the total deflection is contributed by immediate deflection and approximately 20% is contributed by long-term deflection. Beams with partially restrained connections at ends exhibit almost similar pattern. Percentage of immediate and long-term deflection is approximately 75% and 25% respectively. For fully fixed end connections immediate deflection is 65% and long-term deflection is 35%.

Immediate deflection is dominant for composite beams for all types of end connections. For long span, long-term deflection is not a problem for composite beams since stiffness of composite beam is mostly obtained from the steel beam. Creep and shrinkage with time is negligible for steel.

4.4.3.1 Composite Beams-Effect of Connection Fixity

To compare the performance of composite beam end connections, tables are prepared showing total deflections. Table 4.14 contains total deflection for three types of end connections. Figure 4.10 graphically represents the total deflection for three spans.

Table 4.14- Composite beam total deflection:

Span	Deflection (End Fully Released)	Deflection (End Partially Restrained)	Deflection (End Fully Fixed)
m(ft)	(mm)	(mm)	(mm)
18 (59)	70	50	32
24(79)	108	73	45
30 (98)	133	95	57

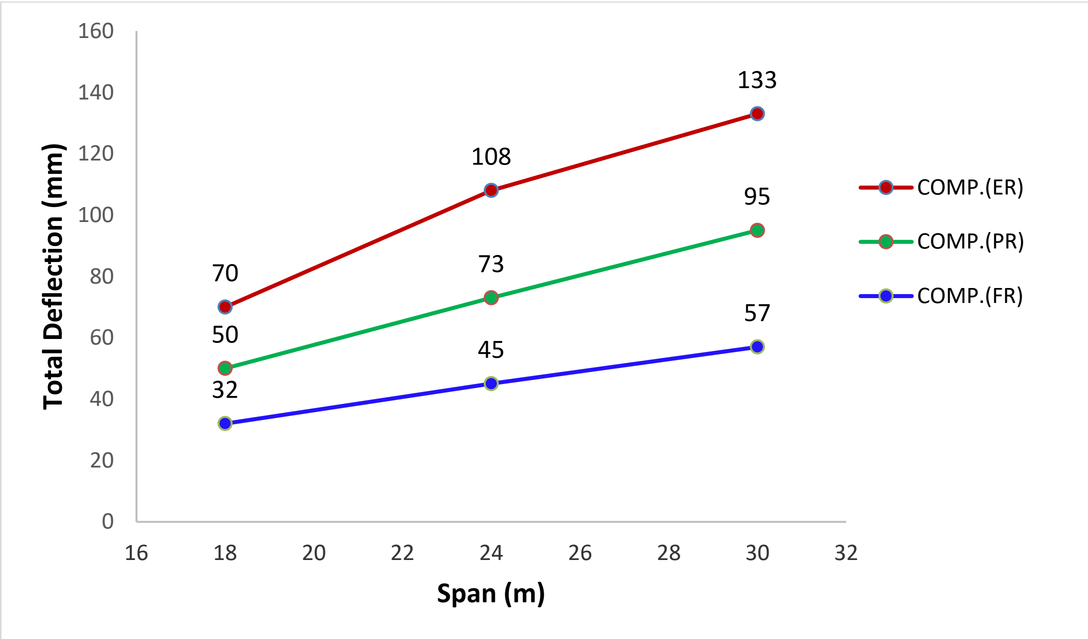


Figure 4.10 Deflection of Composite beam for different spans.

Table 4.15- Comparison between fully released and fully fixed end connection

Span	Deflection (End Fully Released)	Deflection (End Fully Fixed)	*% Difference*
m(ft)	(mm)	(mm)	
18 (59)	70	32	*119 %*
24(79)	108	45	*140 %*
30 (98)	133	57	*133 %*

Table 4.16- Comparison between partially fixed and fully fixed end connection

Span	Deflection (End Partially Restrained)	Deflection (End Fully Fixed)	% *Difference*
m(ft)	(mm)	(mm)	
18 (59)	50	32	*56 %*
24(79)	73	45	*62 %*
30 (98)	95	57	*67 %*

Table 4.17- Comparison between partially fixed and fully released end connection

Span	Deflection (End Partially Restrained)	Deflection (End Fully Released)	% *Difference*
m(ft)	(mm)	(mm)	
18 (59)	50	70	*40 %*
24(79)	73	108	*48 %*
30 (98)	95	133	*40 %*

Table 4.15, 4.16 and 4.17, demonstrate the comparison between the deflections obtained for various end conditions and various span lengths selected in current study. As shown in Table 4.15, deflection of simply supported beams are 119%, 140% and 133% higher for 18M, 24M and 30M span respectively as compared to fixed end (fully restrained) beams,. Beams with partially restrained end connections show 56%, 62% and 67% higher deflection (Table 4.16) than beams with fixed end connections. Beams with fully released (simply supported) end connections show 40%, 48% and 40% higher deflection (Table 4.17) than beams with partially restrained end connections. Considering serviceability criteria, fixed end beams perform better. For all spans, composite beams with fixed end connections exhibit lowest amount of deflection.

Fully restrained connections have high rotational fixity. That is why these connections work better against deflection. However, fixed end connections attract large amount of negative moments at beam ends. This end moment creates tension at beam top surface. Due to tension at beam top, deck concrete cracks. Consequently, the effectiveness of deck concrete in composite action diminishes. The steel beam is then required to be designed as a bare steel beam for this negative moment region. Since, composite action of beam becomes ineffective a higher steel section is required to meet the design demand. Figure 4.11, 4.12 and 4.13 show the negative (end) moment and positive (mid-span) moment for three spans. For all three spans, negative end moment (KN-m) is very significant and approximately 60% higher than mid-span moment. These negative moments govern the design, which is not a desired condition for composite beam. Besides for larger end moment of beams, stronger columns will be required. Overall cost of the building increases. Fully released end connections exhibit very high deflection. This is not preferable considering serviceability issues in design.

Partially restrained connections are neither very stiff nor very flexible. Negative end moment is moderate for these connections. For this study 25% of steel beam plastic moment capacity was used to calculate the fixity of the connections. It has adequate strength and rotational stiffness which works effectively to reduce deflection. Also it allows enough rotation of beam to activate the composite action of steel beam and concrete filled deck. Considering strength and serviceability criteria, partially restrained end connection is the best option for composite beam design.

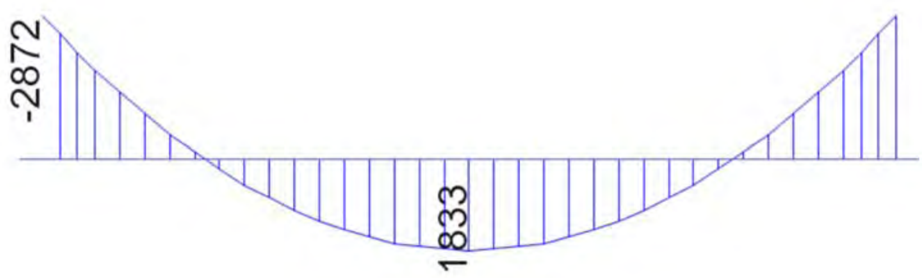


Figure 4.11 18m Beam moment (KN-m) diagram for 1.2DL + 1.6LL combination

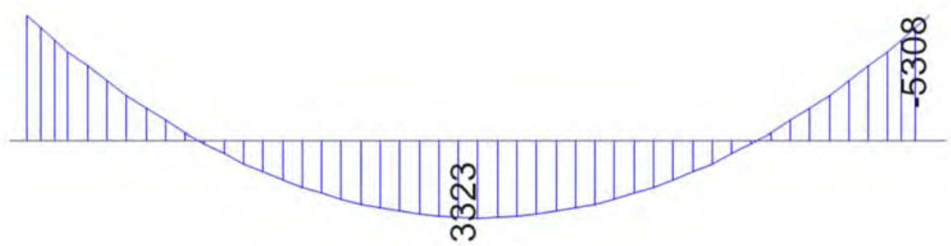


Figure 4.12 24m Beam moment (KN-m) diagram for 1.2DL + 1.6LL combination

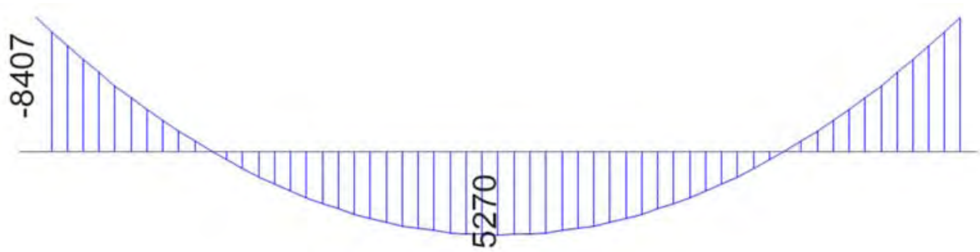


Figure 4.13 30m Beam moment (KN-m) diagram for 1.2DL + 1.6LL combination

4.4.3.2 Composite Beams-Effect of Percentage of Composite Action

To compare the performance of composite beam for different composite actions, beam capacity was calculated for partial and full composite action i.e. 50% and 100% composite actions. Also total deflections were calculated for those two conditions. Table 4.18 contains the number of required shear studs for different composite actions. Table 4.19 shows the nominal moment capacity for different composite actions. Total deflection for different composite actions is shown in Table 4.20 and Table 4.21. Bar chart is shown in Figure 4.14 and Figure 4.15 to compare the serviceability of fully composite beam and partially composite beam.

Table 4.18- Composite beam-Required Shear studs

Beam Span	Shear Studs 50% Composite Action	Shear Studs 100% Composite Action
m(ft)	(pieces)	(pieces)
18 (59)	126	252
24(79)	172	344
30 (98)	228	456

Table 4.19- Composite beam nominal moment capacity

Beam Span	Nominal Moment Capacity (50% composite action)	Nominal Moment Capacity (100% composite action)	Difference (%)
m(ft)	(KN-m)	(KN-m)	
18 (59)	5578	6182	10.8 %
24(79)	9752	10572	8.41 %
30 (98)	15855	16921	6.70 %

From Table 4.18 it can be understood that for full (100%) composite action of composite beams, required number of studs is at least twice compared to the required number of shear studs for partial (50%) composite action. From Table 4.19 it can be derived that, the increase of moment capacity with the increase of composite action is not very significant. When the composite action is increased from 50% to 100% the increment of nominal moment capacity of selected beams is low, ranging from 6% to 11% for the selected beams of this study.

Table 4.20- Comparison between 50% and 100% composite actions of beam (Fully released end condition)

Span	Deflection (Allowable)	Deflection (50% Composite)	Deflection (100% Composite)	*% Difference*
m(ft)	(mm)	(mm)	(mm)	
18 (59)	75	70	56	*25 %*
24(79)	100	108	88	*23 %*
30 (98)	125	133	108	*23 %*

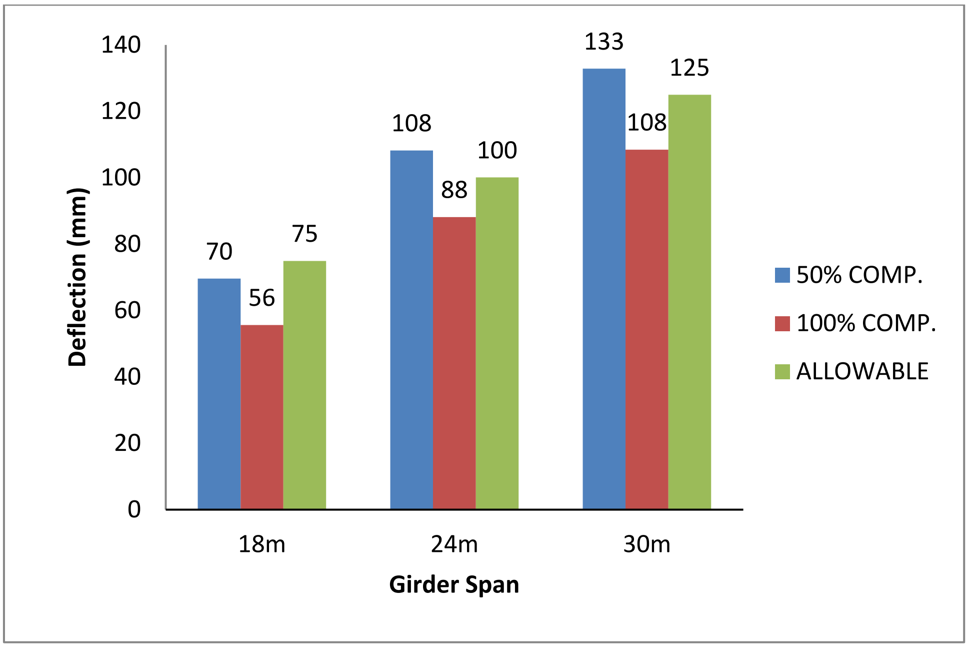


Figure 4.14 Comparison between deflection of Full and Partially Composite beam

(Fully Released End Connection)

Table 4.21- Comparison between 50% and 100% composite actions of beam (Partially restrained end condition)

Span	Deflection (Allowable)	Deflection (50% Composite)	Deflection (100% Composite)	*% Difference*
m(ft)	(mm)	(mm)	(mm)	
18 (59)	75	50	41	*22 %*
24(79)	100	73	60	*22 %*
30 (98)	125	96	79	*22 %*

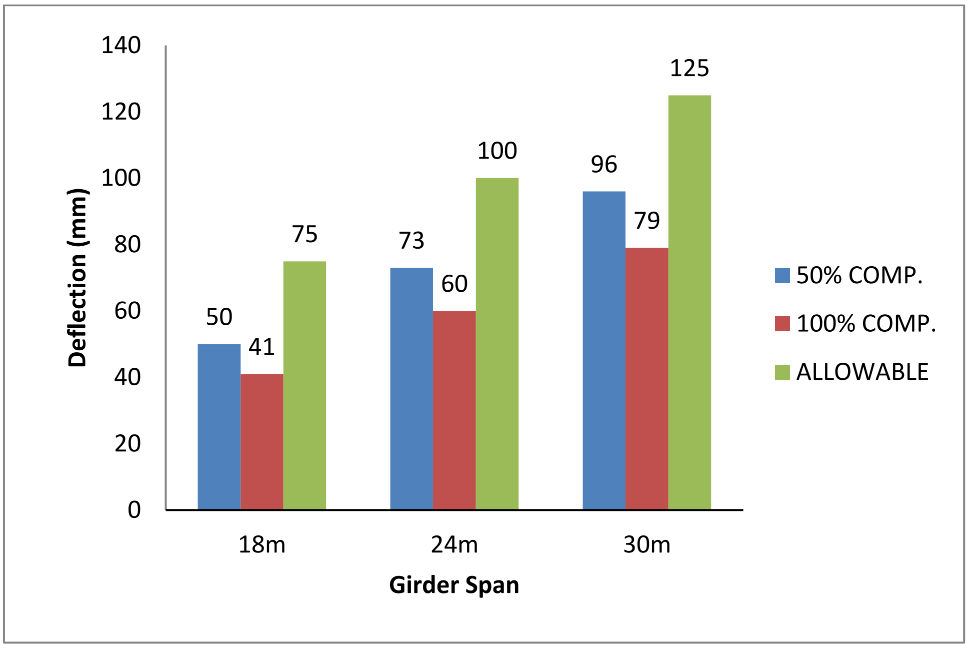


Figure 4.15 Comparison between deflection of Full and Partially Composite beam

(Partially Restrained End Connection)

Table 4.20 and Figure 4.14 illustrate that for fully released end connections deflections of partially composite beams are slightly over the allowable limit. On the other hand deflections of fully composite beams are within the allowable limit with a small margin. Deflection values for partially restrained end connection are listed in Table 4.21 and graphically represented in Figure 4.15. For partially restrained end connections, deflections of both partially composite and fully composite beams are within allowable limits for different spans. The difference between the deflection of partial and full composite beams is about 25% for both type of end connections.

It was observed that for the selected beams of this study fully composite action of beams does not provide much beneficial effects to the strength and serviceability. On the other hand, the number of shear studs to attain full composite action is double. It can be said that for the beams of this study, fully composite action is not very economical solution to design with. In the next sections of this study partially (50%) composite beams have been considered for comparison between the results.

4.4.4 Serviceability Comparison Between RC and Composite Beams

Total deflection of RC and composite beams are summarized in Table 4.22 and Figure 4.16.

Table 4.22- RC and composite beam total deflection table

Span	Deflection RC beams	Deflection (End Fully Released)	Deflection (End Partially Restrained)	Deflection (End Fully Fixed)
m(ft)	(mm)	(mm)	(mm)	(mm)
18 (59)	105	70	50	32
24(79)	137	108	73	45
30 (98)	174	133	95	57

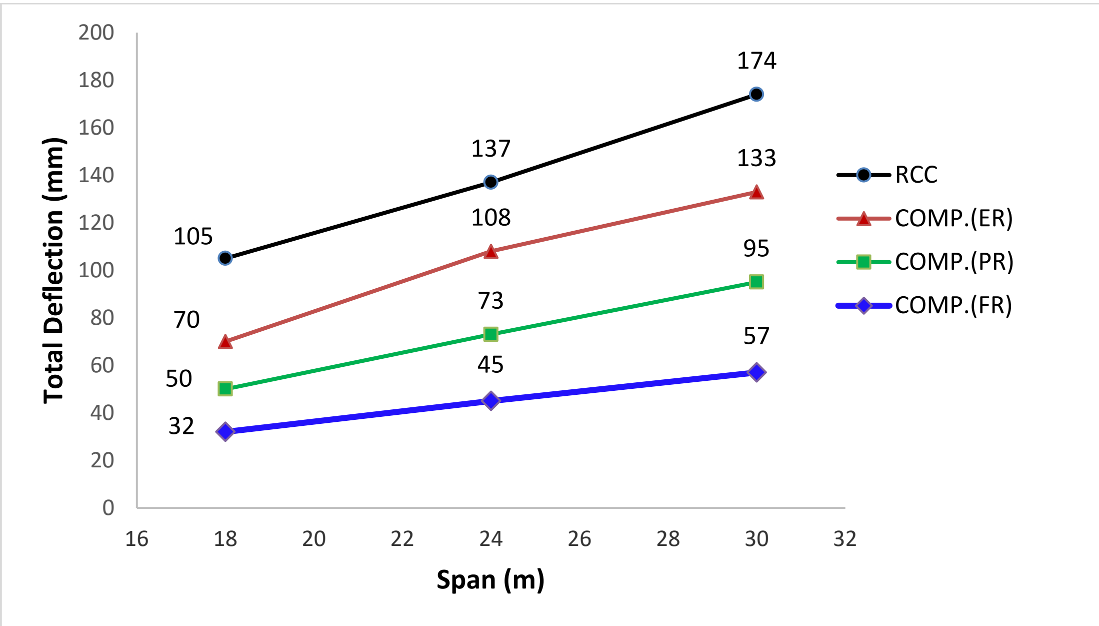


Figure 4.16 Total deflection of RC and Composite beams

Total deflections of RC beams are higher than that of composite beams for all spans and for all types of end connections. The reason behind this is the long-term creep and shrinkage effect of RC beams. This long-term effect increases with the increase of span. Figure 4.17 and Figure 4.18 show the immediate and long term deflection of RC and composite beams respectively for various span lengths. It is apparent that there is large difference between long term performance of RC and composite beam.

Performance of RC and composite beams has been numerically compared through Table 4.23 to Table 4.28. Both instantaneous and long-term deflections have been analyzed.

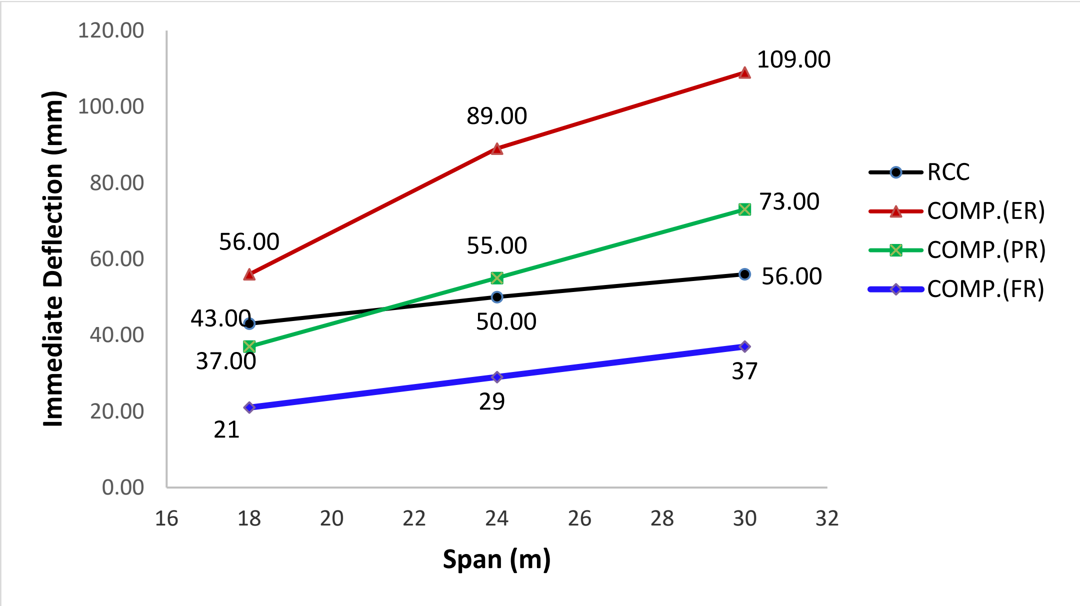


Figure 4.17 Immediate deflections of RC and Composite beams

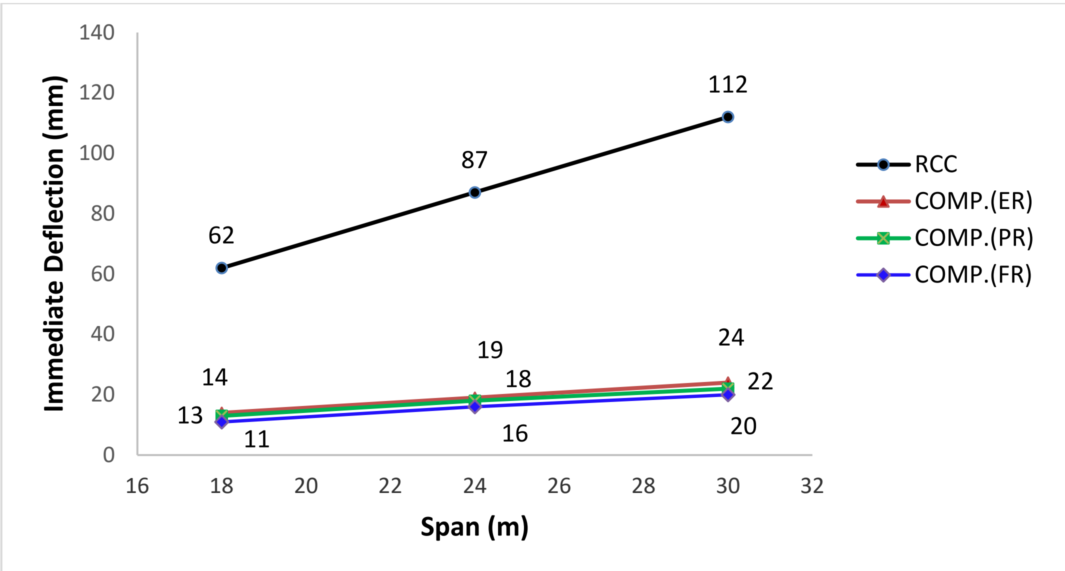


Figure 4.18 Long-term deflections of RC and Composite beams

Table 4.23- RC vs. Composite (End Released) Beam Immediate Deflection

Beam Span	RC(mm)	Composite (mm)	% Difference	Remarks
18m	45	56	-29 %	*Def. comp > Def. rc*
24m	50	89	-78 %	*Def. comp > Def. rc*
30m	56	109	-95 %	*Def. comp > Def. rc*

Table 4.24- RC vs. Composite Beam (Partially Restrained) Immediate Deflection

Beam Span	RC(mm)	Composite (mm)	% Difference	Remarks
18m	45	37	15 %	*Def. rc > Def. comp*
24m	50	55	-12 %	*Def. comp > Def. rc*
30m	56	73	-32 %	*Def. comp > Def. rc*

Table 4.25- RC vs. Composite (Fully Restrained) Beam Immediate Deflection

Beam Span	RC(mm)	Composite (mm)	% Difference	Remarks
18m	45	21	107 %	*Def. rc > Def. comp*
24m	50	29	69 %	*Def. rc > Def. comp*
30m	56	37	49 %	*Def. rc > Def. comp*

Table 4.26- RC vs. Composite (End Released) Beam Long-Term Deflection

Beam Span	RC(mm)	Composite (mm)	% Difference	Remarks
18m	62	14	444 %	*Def. rc > Def. comp*
24m	87	19	446 %	*Def. rc > Def. comp*
30m	112	24	466 %	*Def. rc > Def. comp*

Table 4.27- RC vs. Composite (Partially Released) Beam Long-Term Deflection

Beam Span	RC(mm)	Composite (mm)	% Difference	Remarks
18m	62	13	488 %	*Def. rc > Def. comp*
24m	87	18	491 %	*Def. rc > Def. comp*
30m	112	22	509 %	*Def. rc > Def. comp*

Table 4.28- RC vs. Composite Beam (Fully Restrained) long-term Deflection

Beam Span	RC(mm)	Composite (mm)	% Difference	Remarks
18m	62	11	542 %	*Def. rc > Def. comp*
24m	87	16	546 %	*Def. rc > Def. comp*
30m	112	20	568 %	*Def. rc > Def. comp*

Immediate deflection of RC beam is lower for most of the cases as compared to composite beams except for for beams with fixed end connections. However, long-term deflection of RC beam is found to be much higher than that of composite beams.

While comparing between partially restrained composite beam and RC beam, it is found that long-term deflection of RC beam is 388%, 391% and 409% higher for 18M, 24M and

30M spans respectively. It proves that, with the increase of span, long term effect has increased. Similar pattern is observed for released and fixed end connections. It is apparent that for long span and during the design life, performance of composite beam is much better compared to RC beams.

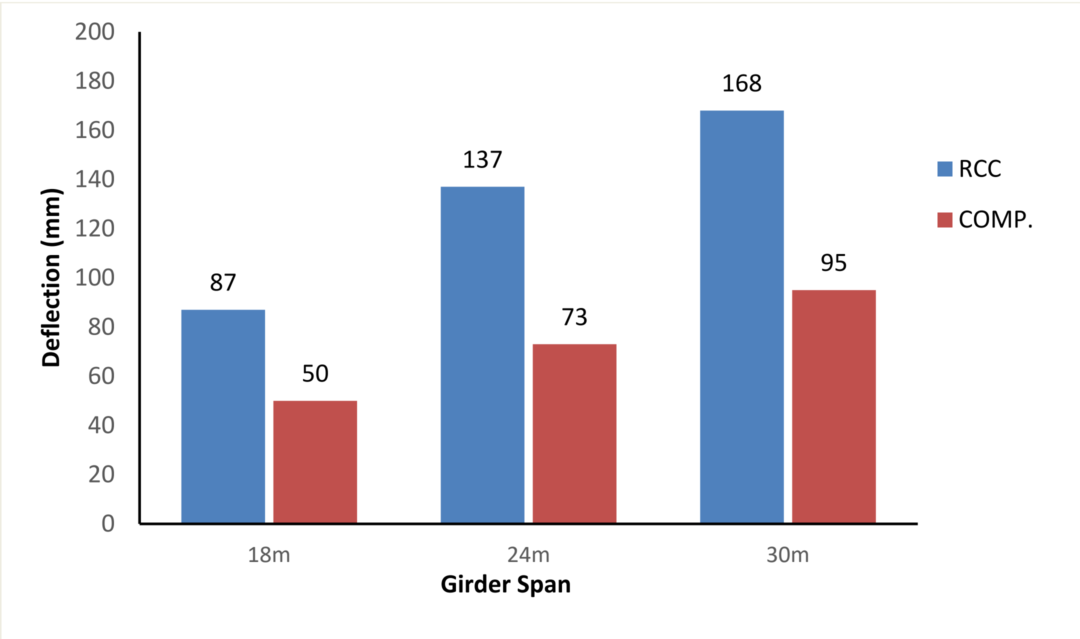


Figure 4.19 Comparison between RC and Partially Restrained Composite beam

4.4.5 RC versus Composite Beam-Performance Analysis

Considering strength, stiffness and effectiveness, partially restrained connection is the most suitable solution for composite beams. A bar chart is presented in Figure 4.19 to compare the serviceability performance of RC beam and partially restrained composite beam which illustrates the effectiveness of composite beams over RC beams. Considering performance, it can be concluded that partially restrained composite beam is the most effective solution for long span structures.

4.5 Ultimate Flexural Capacity

Nonlinear finite element analysis for both the RC and composite beams were performed using ABAQUS (HKS 2-14) to explore the ultimate flexural capacity and corresponding vertical deflection at midspan under pure flexure. Beams were simple supported and loaded at two points. Loads were applied at one third distance from each end supports. Figure 4.20, 4.21 and 4.22 shows the ultimate moment vs deflection curves for both RC and composite beams for 18M, 24M and 30M span respectively.

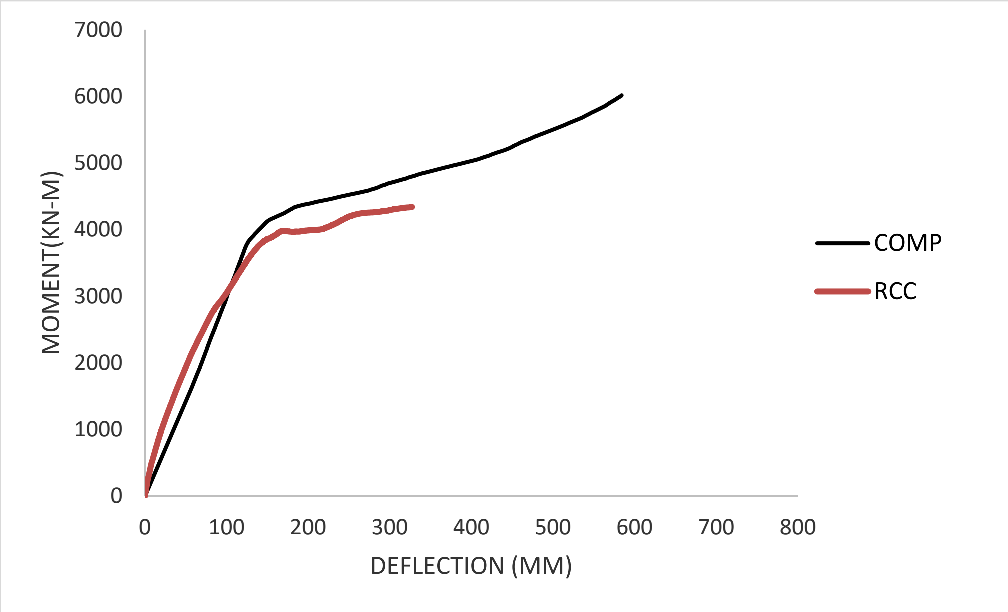


Figure 4.20 Flexural Capacity vs. Deflection curve for 18m RC and Composite beam.

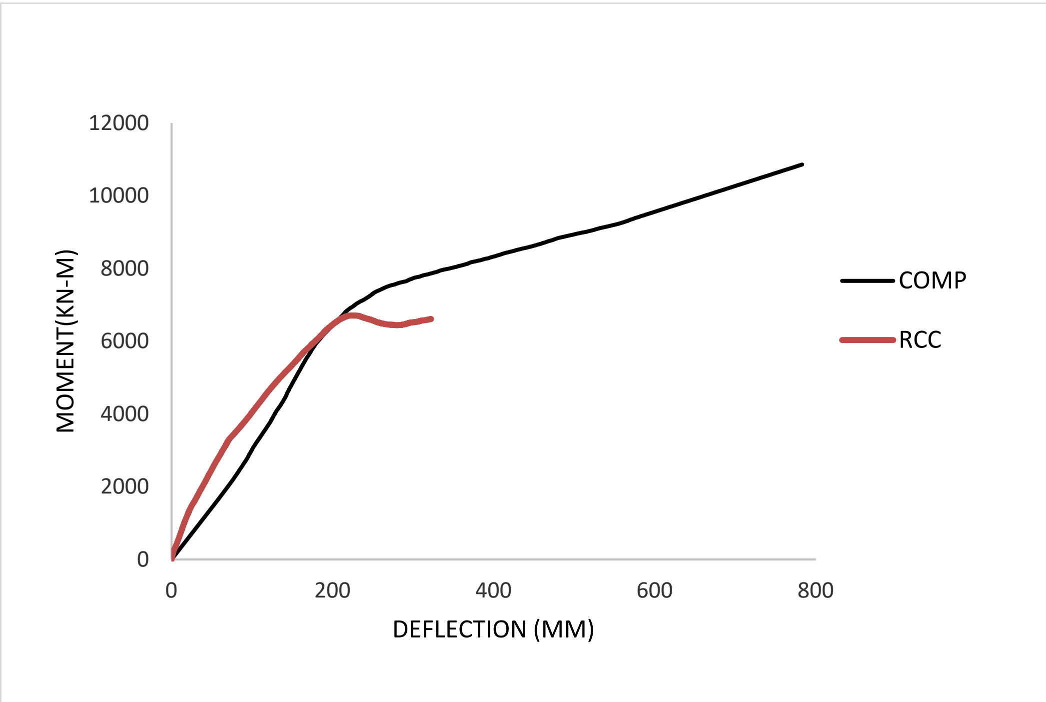


Figure 4.21 Flexural Capacity vs. Deflection curve for 24M RC and Composite beam.

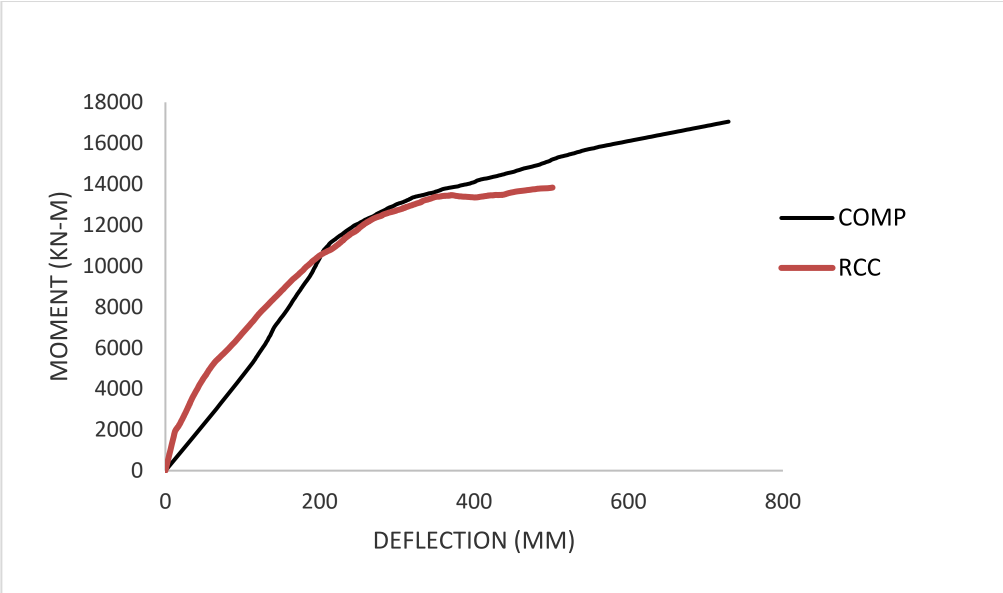


Figure 4.22 Flexural Capacity vs. Deflection curve for 30M RC and Composite beam.

Table 4.29 summarizes the flexural capacity of RC and composite beams for different spans. In Figure 4.21 flexural capacity of RC and composite beams for different spans has been represented using bar chart.

Table 4.29- Flexural Capacity of RC and Composite Beam

Span (m)	Ultimate Moment (KN-m)		% Increase	Remarks
	COMP.	RC		
18	6300	3970	59	Comp.>RC
24	11000	7310	50	Comp.>RC
30	17000	12700	34	Comp.>RC

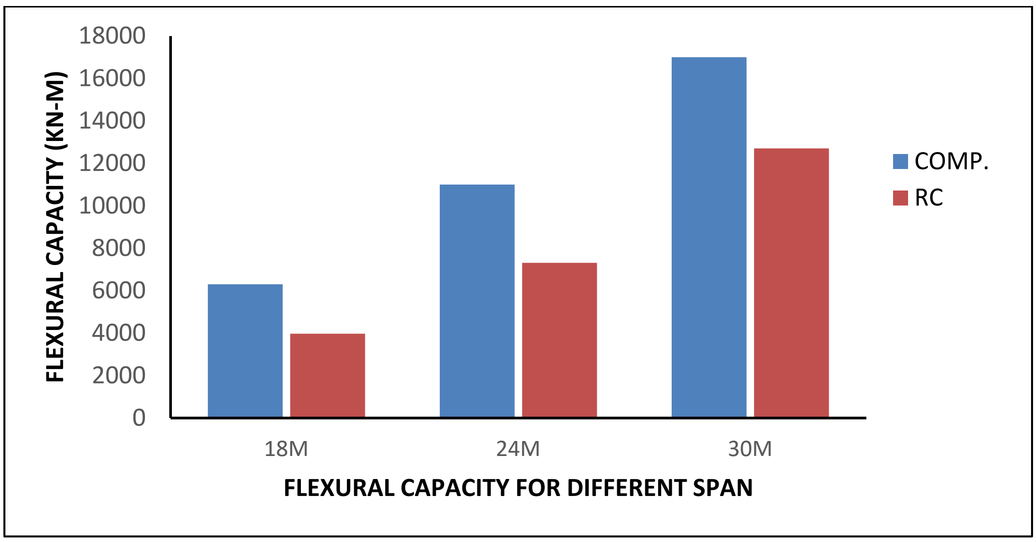


Figure 4.23 Flexural Capacity of RC and composite beams for different span.

From **Table 4.29** and Figure 4.23, it can be observed that ultimate moment capacity of composite beam is higher than that of RC beam.

Table 4.30 shows the deflection values of RC and composite beams at ultimate moment. It is observed that deflections of composite beams are significantly higher than RC beams at ultimate moment point. This indicates that composite beams undergo significant amount of deflection before failure.

Table 4.30- Ultimate Deflection of RC and Composite Beam

Span (m)	Ultimate Deflection (mm)		% Difference	Remarks
	COMP.	RC		
18	581	165	152%	Comp.>RC
24	782	225	247%	Comp.>RC
30	730	298	145%	Comp.>RC

4.6 Cost Comparison

Throughout the previous sections of this chapter, it has been established that composite beams are more effective considering strength and serviceability criteria. In this section a cost comparison has been presented between RC and composite beam. Analysis and design was performed for a 3 storied structure. However, costing has been prepared for a single floor of RC and composite framing.

4.6.1 Costing Analysis

Table 4.31 and Table 4.32 presents the costing of RC framing and composite framing respectively. Both total cost and unit area cost has been shown in different columns. This costing represents the amount required to construct a single floor only. Foundation cost was not included. Comparison between the cost of RC and Composite beams has been shown in Table 4.33. It can be observed that cost of superstructure construction for composite framing is 18%, 14% an 17% higher for 18M, 24M and 30M span respectively.

Table 4.31- RC Beam Costing Summary

Beam Span (m)	Width of Floor (m)	Total Cost (BDT)	Area per floor (sft)	Costing per sft (BDT)
18	16	2,696,775	3098	*870*
24	16	4,183,486	4131	*1012*
30	16	5,942, 267	5164	*1150*

Table 4.32- Composite Beam Costing Summary

Beam Span (m)	Width of Floor (m)	Total Cost (BDT)	Area per floor (sft)	Costing per sft (BDT)
18	16	3,181,149	3098	1027
24	16	4,763,612	4131	*1153*
30	16	6,951, 350	5164	*1346*

Table 4.33- Costing Comparison for Superstructure

Beam Span (m)	Cost per sft (BDT)		% Difference	Remarks
	RC BEAM	COMP. BEAM		
18	870	1027	18%	Comp. > RC
24	1012	1153	14%	Comp. > RC
30	1150	1346	17%	Comp. > RC

Table 4.34 summarizes the foundation loads (Dead Load + Live Load). Foundation load for RC framing system is significantly higher than that of composite framing. Foundation load of RC construction is approximately 28%, 36% and 47% higher for 18M, 24M and 30M span respectively. Foundation construction becomes costlier for RC structures with the increase of span length.

Table 4.34- Foundation Loads (Considering 3 Storied Structures)

Beam Span (m)	Foundation Load (KIP)		% Difference	Remakrs
	RC Beam	COMP. Beam		
18	3142	2458	28%	Comp. < RC
24	4432	3254	36%	Comp. < RC
30	6020	4108	47%	Comp. < RC

4.6.2 Costing Summary

Construction cost of super structure is approximately 15%-20% higher for composite framing system. On the other hand, foundation cost is about 30%-50% higher for RC framing system. For longer span, foundation cost of RC framing is higher. Therefore, it can be concluded that, for long span structure, total construction cost of RC and composite beam is approximately similar.

4.7 Construction Time

In the comparison of cost for the two floor systems construction and erection time of the building structure was not considered. Construction time of RC structure will be definitely higher than the construction time of composite framed building. During construction of RC structure, huge amount of time is required for temporary propping, shuttering, rebar cutting, bending, placing and hardening of concrete. Also it is very difficult to prepare the beam formwork to achieve desired camber. On the other hand, steel deck works as a formwork for concrete casting and provides with a nice working platform for the workers. Composite steel beams can be designed for both shored and unshored construction. So temporary propping can be avoided. Cambering can be done in a shop by applying brute force or by temperature. Composite construction is much faster than RC construction, which adds value to the effectiveness of composite beams over RC beams.

4.8 Summary

It can be summarized that, for long span structures composite beams are more effective as compared to RC beams. Composite beam showed better flexural capacity and ductility than the RC beam for the selected span lengths in current study. When it comes to serviceability, RC beams are good for short time period. However, during the lifetime of a structure, RC beams undergo a significant amount of deflection. This makes composite beam a better choice for long term use. So, considering strength, ductility and serviceability criteria; composite beams are more effective compared to RC beams for long span structures.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The aim of this research was to study the comparative behavior of steel concrete composite beams and RC beams from strength as well as serviceability consideration for long span floor systems. The results of this study will help the engineers to select an effective floor system for long span structures. Effectiveness of composite beam over RC beam has been established by analyzing and designing RC and composite beams for similar loading conditions using three different spans. 3D finite element models were used for analysis. Within the limited scope of the study the following conclusions can be drawn:

1. Larger size of RC beams is required for long span structures. For the sake of comparison, similar depth for RC and Composite beams were selected. However, the selected RC beam cross section were wider than that of composite beams.
2. Ultimate moment capacity of optimized composite beams was found to be higher than that of RC beams. Flexural capacity of composite beam was approximately 30% to 60% higher than that of RC beam.
3. Increasing the composite action from 50% to 100% resulted in an increase in the moment capacity by 6% to 10% and reduction in deflection by about 22% to 25%. Therefore, benefits of increasing the composite action do not have significant impact on strength and serviceability composite beams.
4. End connection plays an important role for the design of composite beams. Partially fixed or flexible moment connections are most effective for composite beams. Fully fixed end connections attract large negative moments at beam ends which makes the composite action ineffective. Flexible moment connections produce a favorable and economic moment distribution between beam ends and mid span. Simply supported beams show large deflections. Partially fixed connection reduces overall deflection of the beam. In this study the beam end connections were selected to resist 25% of the plastic moment capacity of bare

steel beam. Partially restrained connections reduced the deflection of composite beams by approximately 40% compared to fully released end connections.

5. Total Deflection of composite beams was found to be lower than that of RC beams, for both (Released and partially fixed) connection types. Total deflection composite beams with partially fixed end connections were approximately 50% of the deflection of the RC beams.
6. Long term deflections of the RC beams were observed to be almost 1.5 to 2 times larger than its immediate deflection. Long term deflection of RC beam was about 5 times larger than the deflection of partially restrained composite beams. Increase in the long-term deflection of RC beams were found to be higher as the span length increases.
7. Long term deflections of selected composite beams were only one third of the immediate deflection.
8. Design and selection of a composite beam is a flexible process. There are several controlling mechanisms such as percent of composite action, end connections, plate thickness, cambering, shored/unshored construction etc. So it is possible to limit the size of the beam to an extent. This way the height of building storey can be controlled.
9. Finally, this study showed that considering the strength, serviceability and ductility criteria, partial composite beams with flexible end connections is more effective as compared to RC beams for long span floor systems.

5.2 Recommendations:

For future investigation on the performance of composite and RC beams the following recommendations are made:

i. Efficiency of RC and composite beams against vibration can be investigated.
ii. The effect of lateral loads on RC and composite beams can be compared.
iii. For composite beams, effect of composite action on beam design can be investigated with larger number of sample beams to find out the most suitable design condition.
iv. For analysis in ABAQUS full composite action of composite beams was considered. In future performance of partially composite beams can be investigated.
v. Long term performance of composite beams can be experimentally investigated for long span structures.

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Appendix- A

A.1 Nominal Moment Capacity of Composite Beam

The nominal moment capacity of steel concrete composite beam is calculated according to the process described in Chapter I (and its commentary section) of AISC 360-10. Nominal moment capacity of the selected composite beam of 18m span is manually calculated in the following sections.

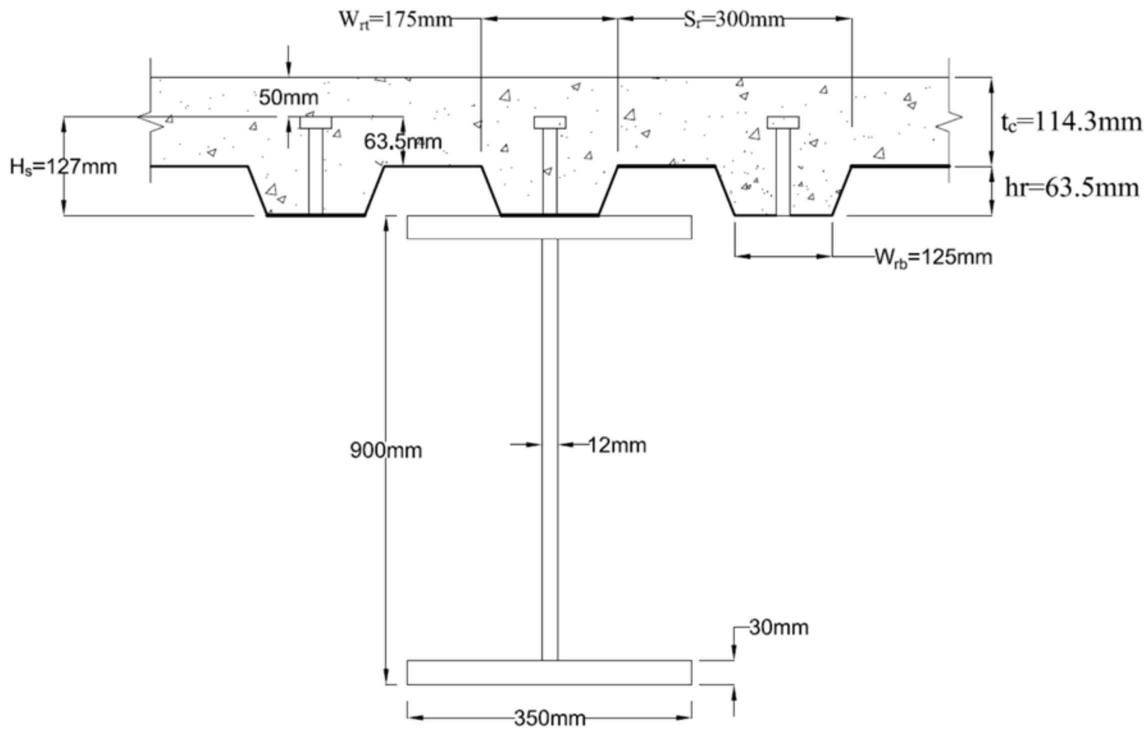

Figure A.1 Selected Composite Beam for 18m Span

A.2 Given Data

Beam Span, L=18m

Distance between the adjacent beams = 8m

Distance to the edge of the slab = 8m

Concrete Compressive Strength, f'_c= 27.5 MPa

Steel Yield Stress, F_y= 345 MPa

Steel Beam Cross Sectional Area, A_s= 31080 mm^2

Steel Stud Diameter d_{sa}= 19mm

Steel Stud Tensile Strength, F_u= 400 MPa

A.3.1 Nominal Moment Capacity Calculation For 100% Composite Action

Step 1.1: Calculation of Effective width of Concrete Slab (b_{eff})

b_{eff} =Minimum of:

2x 1/8 of Beam Span = 2x 1/8 x 18m = 4.5m= 4500mm [Governs]

2x ½ x Distance between the adjacent beams = 2x ½ x 8m = 8m= 8000mm

2x Distance to the edge of the slab = 2 x 8m = 16m= 16000mm

So, Effective Width of Concrete Slab, b_{eff} = 4500mm

Step 1.2: Calculation of effective area of Concrete Slab (A_c)

Deck is running parallel to Steel beam. Considering deck profile is 50% void and 50% concrete filled;

$A_c = b_{eff}$ x 114.3 + (b_{eff} /2) x 63.5 = 4500 x 114.3+ (4500/2) x 63.5 = 657225 mm^2

Step 1.3: Limit States Calculation

a) Concrete Crushing

C= 0.85 f'_c x A_c

= 0.85 x 27.5 x 657225 = 15362634 N= 15362 KN

b) Steel Yielding

$P_y= A_s F_y$

= 31080 x 345 = 10722600 N= 10723 KN [*Governs*]

c) Shear Transfer

Considering Full (100%) composite action,

C= ΣQ_n = Minimum of [15362; 10723] = 10723 KN

Step 1.4: Location of Plastic Neutral Axis

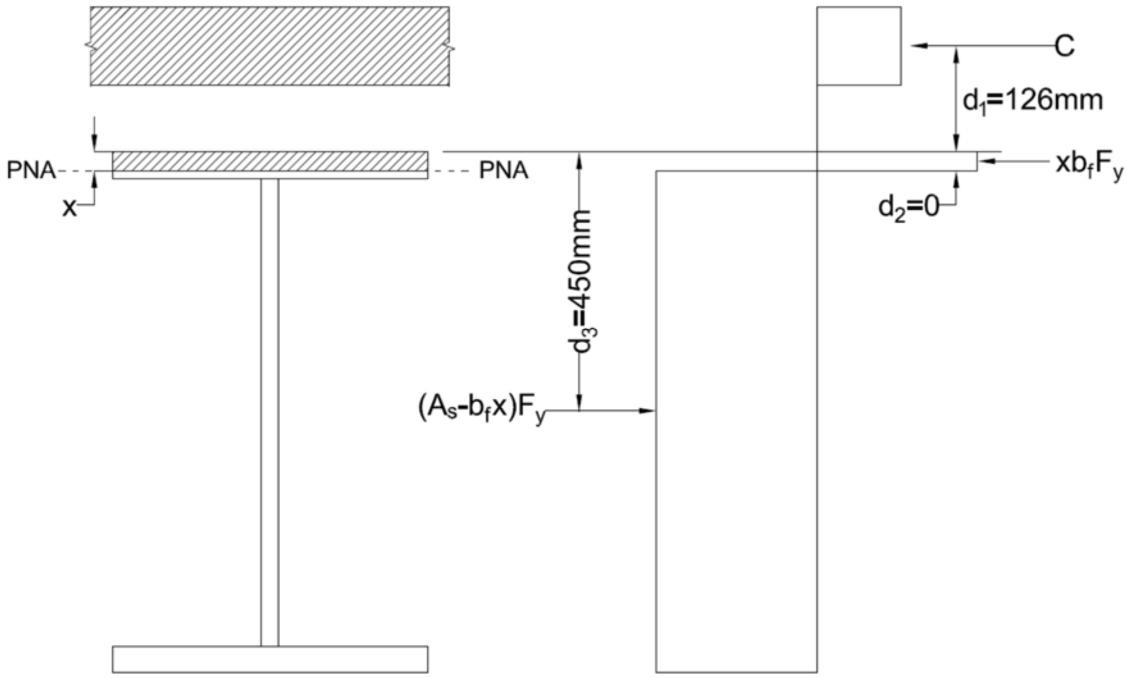

Figure A.2 Plastic Neutral Axis Location (100% Composite Action)

Assuming the trial PNA location is within the top flange of the steel beam,

$\Sigma F_{\text{above PNA}} = \Sigma F_{\text{below PNA}}$

$C + xb_f F_y = (A_s - b_f x) F_y$

Solving for x:

$x = (A_s F_y - C) / 2b_f F_y$

$x = (10723-10723) / 2b_f F_y$

$x = 0 <= t_f$ [PNA is right at the top of beam flange]

As, PNA is right at the top face of Steel Section;

There will be no compression in Steel Section.

So, $d_2 = 0$

Step 1.5: Nominal Moment Capacity Calculation (100% Composite Action)

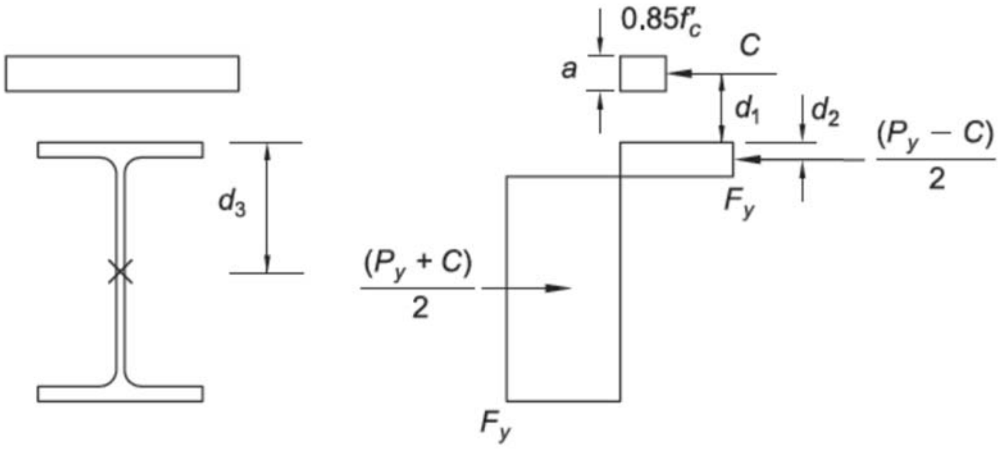

Figure A.3 Plastic stress distributions for positive moment in composite beam

(AISC 2010)

$M_n = C\,(d_1+d_2) + P_y\,(d_3-d_2)$

$a = C/\,(0.85f'_c b)$

a= 10723/ (0.85 x 27.5 x 4.5)

= 102mm < 127 mm [Above top of Deck]

$d_1 = t_{slab} - a/2$

d_1=177- 102/2

=126mm

$d_2 = x/2$

=0/2 = 0

$d_3 = d/2$

= 900/2

= 450 mm

$P_y = A_s F_y$

= 10723 KN

So, $M_n = C (d_1+d_2) + P_y (d_3-d_2)$

= 10723 (126+0) +10723 (450-0)

= 6176448 KN-mm

= 6176 KN-m

Step 1.6: Number of Shear Studs

$Q_n = 0.5 A_{sa} (f'_c E_c)^{1/2} <= R_g R_p A_{sa} F_u$

$A_{sa} = \pi d^2_{sa} /4$

$=284 \text{ mm}^2$

f'_c = 27.5 MPa

$E_c = W_c^{1.5} (f'_c)^{1/2}$

= $(150 \text{ lb/ft}^3)^{1.5}$ x $(4 \text{ ksi})^{1/2}$

= 3675 ksi

= 25.34 KN/mm^2

R_g= 1.0 Stud anchors welded directly to the steel shape within the slab haunch

R_p= 0.75 Stud anchors welded directly to the steel shape

R_u= 400 MPA= 400 N/mm^2

$Q_n = 0.5 A_{sa} (f'_c E_c)^{1/2} <= R_g R_p A_{sa} F_u$

Q_n = (0.5) (284) $(27.5 \text{ x } 25340)^{1/2}$ <= (1.0) (0.75) (284) (400)

=118538 N > 85200 N

Use Q_n = 85200 N= 85.2 KN

Number of shear studs in between the maximum positive moment and point of zero moment = C/ Q_n = 10723/85.2 = 126

Total Number of Steel Anchors in Beams for 100% Composite Action= 126x2 = 252

Nominal Moment Capacity (100% Composite Action) = 6176 KN-m,

Minimum Number of Shear Studs= 252.

A.3.2 Nominal Moment Capacity Calculation For 50% Composite Action

Step 2.1: Calculation of Effective width of Concrete Slab (b_{eff})

b_{eff} =Minimum of:

2x 1/8 of Beam Span = 2x 1/8 x 18m = 4.5m= 4500mm [Governs]

2x ½ x Distance between the adjacent beams = 2x ½ x 8m = 8m= 8000mm

2x Distance to the edge of the slab = 2 x 8m = 16m= 16000mm

So, Effective Width of Concrete Slab, b_{eff} = 4500mm

Step 2.2: Calculation of effective area of Concrete Slab (A_c)

Deck is running parallel to Steel beam. Considering deck profile is 50% void and 50% concrete filled;

$A_c = b_{eff} \times 114.3 + (b_{eff}/2) \times 63.5 = 4500 \times 114.3 + (4500/2) \times 63.5 = 657225\ mm^2$

Step 2.3: Limit States Calculation

a) Concrete Crushing

$C = 0.85\ f'_c \times A_c$

$= 0.85 \times 27.5 \times 657225 = 15362634$ N= 15362 KN

b) Steel Yielding

$P_y = A_s F_y$

$= 31080 \times 345 = 10722600$ N= 10723 KN [*Governs*]

c) Shear Transfer

Considering Partial (50%) composite action,

$C = \Sigma Q_n$ = 50% of Minimum of [15362; 10723] = 5361 KN

Step 2.4: Location of Plastic Neutral Axis

Assuming the trial PNA location is within the top flange of the steel beam,

$\Sigma F_{\text{above PNA}} = \Sigma F_{\text{below PNA}}$

$C + xb_f F_y = (A_s - b_f x) F_y$

Solving for x:

$x = (A_s F_y - C) / 2b_f F_y$

x= (10723-5361) / (2 x 0.35 x 345000)

x= 22mm <= t_f [PNA in beam top flange]

Step 2.5: Nominal Moment Capacity Calculation

$M_n = C (d_1+d_2) + P_y (d_3-d_2)$

$a = C/ (0.85f'_c b)$

a= 5361/ (0.85 x 27.5 x 4.5)

= 51mm < 127 mm [Above top of Deck]

$d_1 = t_{slab} - a/2$

d_1=177- 51/2

= 151.5mm

$d_2 = x/2$

=22/2 = 11mm

$d_3 = d/2$

= 900/2

= 450 mm

$P_y = A_s F_y$

= 10723 KN

So, $M_n = C(d_1+d_2) + P_y(d_3-d_2)$

= 5361 (151.5+11) +10723 (450-11)

= 5578559 KN-mm

= 5578 KN-m

Step 2.6: Number of Shear Studs

$Q_n = 0.5\, A_{sa}\, (f'_c\, E_c)^{1/2} <= R_g R_p A_{sa} F_u$

$A_{sa} = \pi d^2_{sa}/4$

$=284\ mm^2$

f'_c = 27.5 MPa

$E_c = W_c^{1.5} (f'_c)^{1/2}$

= $(150\ lb/ft^3)^{1.5}$ x $(4\ ksi)^{1/2}$

= 3675 ksi

= 25.34 KN/mm^2

R_g= 1.0 Stud anchors welded directly to the steel shape within the slab haunch

R_p= 0.75 Stud anchors welded directly to the steel shape

R_u= 400 MPA= 400 N/mm^2

$Q_n = 0.5\, A_{sa}\, (f'_c\, E_c)^{1/2} <= R_g R_p A_{sa} F_u$

Q_n = (0.5) (284) $(27.5 \times 25340)^{1/2}$ <= (1.0) (0.75) (284) (400)

=118538 N > 85200 N

Use Q_n = 85200 N= 85.2 KN

Number of shear studs in between the maximum positive moment and point of zero moment = C/ Q_n = 5361/85.2 = 63

Total Number of Steel Anchors in Beams for 50% Composite Action= 63x2 = 126

Nominal Moment Capacity (50% Composite Action) = 5578 KN-m,

Minimum Number of Shear Studs= 126.

A.4 Calculation Summary:

Nominal moment capacity of 18m span composite beam for partial and full composite action has been calculated by manual calculation. The beam was also designed using ETABS. The results are summarized in table A.1. It is observed that manual calculation and ETABS provide similar results.

Table A.1- Nominal Moment Capacity of 18m Span Composite Beam

Percentage of Composite Action	Number of Shear Studs	Nominal Moment Capacity, M_n (Manual Calculation) (KN-m)	Design Moment Capacity, ΦM_n (Manual Calculation) (KN-m)	Design Moment Capacity, ΦM_n (From ETABS) (KN-m)	% Difference
50%	126	5578	5020	5020	0%
100%	252	6176	5558	5564	0.1%

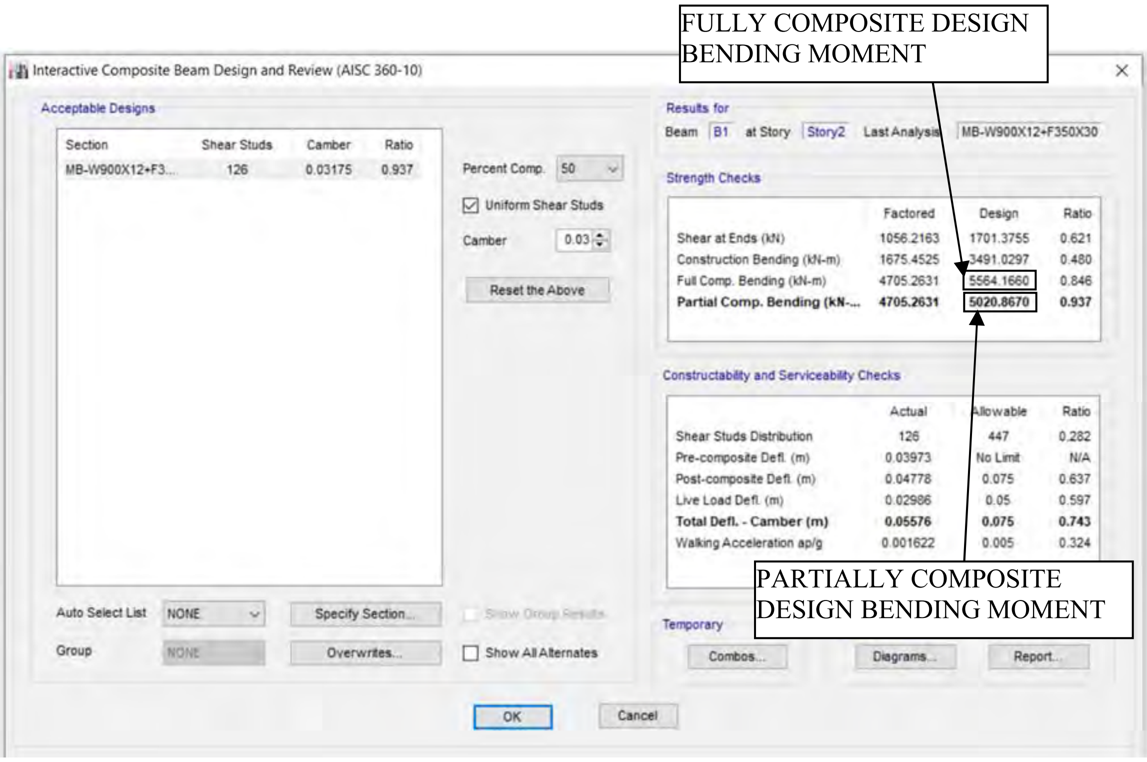

Figure A.4 Composite Beam Design Results from ETABS