COMPARATIVE STUDY ON REINFORCED CONCRETE AND STEEL FRAMED BUILDINGS WITH VARIOUS FLOOR SYSTEMS

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A THESIS

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CERTIFICATE OF APPROVAL

The thesis titled **"Comparative Study on Reinforced Concrete and Steel Framed Buildings with Various Floor Systems"** submitted by Tarique Nazmus Sadat, Roll No.: 0409042304(P), Session: April 2009 has been accepted as satisfactory in partial fulfillment of the requirement for the degree of Master in Engineering in Civil and Structural Engineering on 31 March, 2014.

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DECLARATION

It is hereby declared that the works contained in this thesis is the research work carried out by the author under the supervision of Dr. Shafiul Bari, Professor of the Department of Civil Engineering, BUET, Dhaka. Neither this thesis nor any part thereof has been submitted elsewhere for the award of any degree or diploma except for publications.

Tarique Nazmus Sadat

Dedicated

To

My Beloved Parents

ACKNOWLEDGEMENT

Alhamdulillah. All praises to Allah (SWT) who bestowed His Mercy upon me to complete this work successfully.

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The Author

ABSTRACT

Reinforced concrete and steel framed structures with various floor systems are being used for multistory buildings. So there are alternative options for designers to decide structural system for a particular building. A comparative study on RC and steel framed buildings with various floor systems is necessary to evaluate better structural system regarding overall economy, structural performance, construction time etc. This study will be helpful in deciding structural system for multistory industrial buildings. Past studies on multistory industrial building structures of Bangladesh are not much enough. This limits our ability in making decision about structural system of industrial buildings in Bangladesh.

To conduct the intended research work, architectural layout plan of a six story garments factory is prepared. Following the layout plan, RC structure with flat plate and beam-slab floor system is formed. According to same plan, steel structure with non-composite and composite floor system is also formed. Structural modeling and analysis have been performed by STAAD.Pro. Loads are assigned following BNBC 1993. From analytical results, RC structures are designed following ACI Building Code 2008. Steel structures are designed following AISC LRFD 2010. Comparisons of structural behavior and cost analysis have been performed for the four types of structural system.

The research outcome shows that building cost increases 1.6% for RC flat plate system, 1.8% for steel composite system and about 9% for steel non-composite system compared to RC beam-slab system. When floor system of steel structure is designed as composite then structural steel weight savings is about 18% for typical grid of 7.62m×7.62m. Composite system brings significant economic benefit for steel structure; for example structural steel cost decreases 18-19%, total structural cost decreases 9-11% and finally total building cost decreases by 6-8%. Compared to steel structure, foundation cost increases 22% for RC beam-slab system and 24% for RC flat plate system.

TABLE OF CONTENTS

TITLE **PAGE**

Chapter 3: METHODOLOGY

Chapter 4: ANALYSIS OF RESULTS AND DISCUSSION

Chapter 5: CONCLUSIONS AND RECOMMENDATIONS

LIST OF TABLES

TABLE NO. **PAGE**

LIST OF FIGURES

FIGURE NO. **PAGE**

Chapter 1
INTRODUCTION

1.1 General

Worldwide different types of RC and steel structures with various floor systems are being used for multistory buildings. In the past, masonry structures were widely used for building construction. Day by day technology has developed. Later, steel structural systems were started for multistory buildings. With the introduction of reinforced concrete, RC structural systems started for multistory building construction. RC floor system supported on steel beam was historically designed as non composite. With the advent of welding, it became practical to provide mechanical shear connectors to consider composite action. Due to failure of many multi-storied and low-rise RC and masonry buildings due to earthquake, structural engineers are looking for the alternative methods of construction. Use of composite or hybrid material is of particular interest. Bare steel structure is sensitive to fire. Now a day, different fire proofing system has developed significantly. In Bangladesh, mostly masonry and RC structures were being used. During last decade, steel structural systems are being popular. So, alternative structural systems are gradually developing to compete with RC structural systems. Now a day, use of masonry structure is very limited. RC structure is dominating and steel structure is entering gradually for multistory building structures in Bangladesh. So, comparative study is required to identify most effective structural system for a particular building.

1.2 Background

Reinforced concrete rigid frame and shear walled-frame structure with different floor systems such as two-way slab supported on beam, flat plate etc. are being used widely for last few decades in Bangladesh. Now a day, to cut short the construction time, steel structure is getting popularity for multistory industrial and commercial buildings. Eccentric and concentric braced steel frame with steel girder-beam floor system topping RC slab on corrugated steel deck is being used. RC slabs are being connected with steel beam and girder by the help of mechanical shear connectors. So, steel floor system may be designed as composite or non composite. Usually, when composite action is considered then significant economic benefit may be achieved. Steel columns also may be designed as composite, using

APPENDICES

Chapter 1
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RC encased or concrete in-filled steel tube systems. So there are alternative options for the designers to select structural form and floor system for a particular building. Especially for industrial multistory building, there is a construction time limit. So, investors show interest for steel structure due to quick constructionability. But sometimes investors and designers are confused about construction cost of steel structure compared to RC structure. Some designers have preconception that steel structures are highly expensive compared to RC structures and they show less interest for steel structure.

In the past, research were conducted to evaluate and compare structural behavior, overall structural performance, construction time and economy of RC, steel and composite structural systems for multistory buildings (Panchal and Patodi 2010, Johnson 2004, Rackham et al. 2009, Dabhade et al. 2009, Panchel and Marathe 2011). Those works suggest that steel structure with composite floor system brings considerable economy i.e. 20 to 60 percent structural steel weight savings is possible. Also 40 to 60 percent construction time savings for steel structure is possible compared to RC structure. Steel structure with composite floor system is 6 to 10 percent cost effective over RC structure. However, such information for structural systems of typical multistory industrial and commercial buildings in Bangladesh is not known exactly. This limits our ability in making decision about structural systems of upcoming multistory buildings in Bangladesh. This justifies the necessity of conducting comparative research and study on RC and steel framed buildings with various floor systems to evaluate the most effective structural system with emphasis on overall economy, structural performance, construction time etc. This study will be helpful in deciding effective structural system for multistory commercial and industrial buildings in Bangladesh.

1.3 Objectives

1. To perform structural analysis and design of a reinforced concrete framed six storied industrial building with various floor systems.
2. To conduct structural analysis of the same building using steel framing with composite and non-composite floor systems.
3. To compare the structural behavior and cost analysis of the buildings.

1.4 Scope of Work

Architectural layout plan of a six story garments factory has been prepared. The typical floor height is 3.35 meter and column spacing is 7.62 meter in both directions. Following the plan, RC shear walled-rigid frame structure with two-way slab supported on beam and flat plate floor system is formed. Using the same plan, eccentric braced steel frame with steel girder-beam floor system topping RC slab on corrugated steel deck is formed as composite and non-composite. To activate composite action, mechanical shear connectors have been used. Then the building is designed and estimated for these four types of structural system including foundation. Total building cost including foundation, plumbing, sanitary, wall, floor finishing etc. has been prepared excluding electro mechanical cost for the selected four types of structure. Cost of fire proofing spray is considered separately. For steel structure, columns are non-composite for both cases i.e. steel I-section. Static analysis is performed. Duration of construction time is not considered. Span and height is constant. Construction cost, structural behavior and other related matters are observed to evaluate the better structural system for the selected garments factory building.

1.5 Outline of Methodology

To conduct the intended research work, architectural layout plan of a six story factory building i.e. garments factory has been prepared. Following the plan, RC structure with beam supported two-way slab floor and flat plate floor system is formed. Again following same plan, steel structure with steel girder-beam floor topping RC slab on corrugated steel deck is formed as composite and non-composite. Steel columns are non-composite i.e. steel I-sections for both cases. Then three dimensional structural modeling and static analysis have been performed by STADD.Pro for the four types of structural system. Loads are assigned as per BNBC 1993. Load combinations are generated regarding BNBC 1993, AISC LRFD Specifications 1993 and ACI Building code 2008. From analytical results, RC structure is designed using USD method as per ACI Building Code 2008. Steel structure is designed following AISC LRFD Specifications 2010. Welding, connections, anchor bolts, base plates, nut-bolts, shear connectors etc. are also designed following AISC LRFD Specifications 2010. Complete construction cost excluding electro-mechanical cost has been prepared. Comparison of construction cost, structural behavior and other related matters have been prepared to evaluate better/ most effective structural system for the building used for this research.

1.6 Organization of the Thesis

The thesis consists of five chapters. Chapter 1 presents an introduction to the study. It includes the research background, objectives, scope of work and outline of methodology.

Chapter 2 deals with literature review. It illustrates the structural systems, load considerations, load combinations, connection systems, foundation systems, serviceability criteri, materials and specifications used for structural steel design, estimating and costing procedure etc.

Chapter 3 deals with complete methodology of the research work. It illustrates architectural planning and structural formation of selected building, load calculations, structural modeling, structural analysis, design, estimating and preparation of tables for different design results.

Chapter 4 deals with analysis and discussion of all data obtained from the design program performed in chapter 3 to compare, evaluate and draw findings and conclusions of the research work.

Chapter 5 deals with conclusions and recommendations of the research work.

Chapter 2
LITERATURE REVIEW

2.1 Introduction

Different types of RC and steel structure with various floor systems are commonly used for multistory buildings. In designing multistory building structure, thorough knowledge is required about different structural systems, load considerations, load combinations, foundation systems, serviceability requirements, construction materials and specifications, connection types etc. These are illustrated in this chapter in brief.

2.2 Structural Forms of Multistory Buildings

In the structural modeling and design process of a multistory building, a thorough knowledge of multi-story building structural components and their modes of behavior is a prerequisite to devising an appropriate load resisting system. Such a system must be efficient, economic and should minimize the structural penalty for height and span while maximizing the satisfaction of the basic serviceability requirements. Some conventional structural forms are described here in brief.

Rigid Frame Structure

Rigid frame structures consist of columns and girders jointed by moment resistant connections. The lateral stiffness of a rigid-frame bent depends on the bending stiffness of the columns, girders, and connections in the plane of the bent. If used as the only source of lateral resistance in a building, in its typical 6 to 9 meter bay size, rigid framing is economic only for buildings up to about 25 stories. Rigid-frame construction is ideally suited for reinforced concrete buildings because of the inherent rigidity of reinforced concrete joints. The rigid-frame form is also used for steel frame buildings, but moment resistant connections in steel tends to be costly. The sizes of the columns and girders at any level of a rigid frame are directly influenced by the magnitude of the external shear at that level, and they therefore increase towards the base. Consequently, the design of the floor framing cannot be repetitive as it is in some braced frames. A further result is that sometimes it is not possible in the lowest stories to accommodate the required depth of girder within the normal ceiling space. Gravity loading also is resisted by the rigid frame action. Negative moments are induced in the girders adjacent to the columns causing the mid-span positive moments to be significantly less than in a simply supported span. In structures in which gravity loads dictate the design, economics in member sizes that arise from this effect tend to be offset by the higher cost of the rigid joints. While rigid frames of a typical scale that serve alone to resist lateral loading have an economic height limit of about 25 stories, smaller scale rigid frames in the form of a perimeter tube, or typically scaled rigid frames in combination with shear walls or braced bends can be economic up to much greater heights (Smith and Coull 1991).

Wall-Frame Structure

When shear walls are combined with rigid frames as shown in Figure 2.1, the walls which tend to deflect in a flexural configuration, and the frames, which tend to deflect in a shear mode, are constrained to adopt a common deflected shape by the horizontal rigidity of the girders and slabs. As a consequence, the walls and frames interact horizontally, especially at the top, to produce a stiffer and stronger structure. An additional, less well known feature of the wall-frame structure is that, in a carefully “tuned” structure, the shear in the frame can be made approximately uniform over the height, allowing the floor framing to be repetitive. Although the wall-frame structure is usually perceived as a concrete structural form, with shear walls and concrete frames, a steel counterpart using braced frames and steel rigid frames offers similar benefits of horizontal interaction. The braced frames behave with an overall flexural tendency to interact with the shear mode of the rigid frames (Smith and Coull 1991).

Flat-Plate and Flat-Slab Structure

The flat-plate structure is the simplest and most logical of all structural forms in that it consists of uniform slabs, of 125-200 millimeter thickness, connected rigidly to supporting columns. The system, which is essentially of reinforced concrete, is very economical in having a flat soffit requiring the most uncomplicated form work and, because the soffit can be used as the ceiling, in creating a minimum possible floor depth. Under lateral loading the behavior of a flat-plate structure is similar to that of a rigid frame, that is, its lateral resistance depends on the flexural stiffness of the components and their connections, with the slabs corresponding to the girders of the rigid frame. The flat plate structure is economical for spans of up to 7.6 meter, above which drop panels can be added to create a flat-slab structure for spans of up to 11.6 meter. Buildings that depend entirely for their lateral resistance on flat-plate or flat-slab action are economical up to about 25 stories (Smith and Coull 1991).

Braced Frame Structures

In braced frames the lateral resistance of the structure is provided by diagonal members that, together with the girders, form the “web” of the vertical truss, with the columns acting as the “chords”. Because the horizontal shear on the building is resisted by the horizontal components of the axial tensile or compressive action in the web members, bracing systems are highly efficient in resisting lateral loads. Bracing is generally as an exclusively steel system because the diagonals are inevitably subjected to tension for one or the other directions of lateral loading. Concrete bracing of the double diagonal form is sometimes used, however, with each diagonal designed as a compression member to carry the full external shear. The efficiency of bracing, in being able to produce a laterally very stiff structure for a minimum of additional material, makes it an economical structural form for any height of building, up to the very tallest. External large scale, extending over many stories and bays has been used to produce not only highly efficient structures, but aesthetically attractive buildings (Smith and Coull 1991).

Eccentric Braced Frame Structures

Concentric braced frames are excellent from strength and stiffness considerations and are therefore used widely either by themselves or in conjunction with moment frames when the lateral loads are caused by wind. However, they are of questionable value in seismic regions because of their poor inelastic behavior. Moment-resistant frames possess considerable energy dissipation characteristics but are relatively flexible when sized from strength considerations alone. Eccentric bracing is a unique structural system that attempts to combine the strength and stiffness of a braced frame with the inelastic behavior and energy dissipation characteristics of a moment frame. The system is called eccentric because deliberate eccentricities are employed between beam-to-column and beam-to brace connections as shown in Figure 2.2. The eccentric beam element acts as a fuse by limiting large forces from entering and causing buckling of braces. The eccentric segment of the beam, called the link, under goes flexural or shear yielding prior to formation of plastic hinges in the other bending members and well before buckling of any compression members. Thus the system maintains stability even under large inelastic deformations. The required stiffness during wind or minor earthquakes is maintained because no plastic hinges are formed under this loads and all behavior is elastic. Although the deformation is larger than in a concentrically braced frame because of bending and shear deformation of the "fuse," its contribution to deflection is not significant because of the relatively small length of the fuse. Thus the elastic stiffness of the eccentrically braced frame can be considered the same as the concentrically braced frame for all practical purposes. The ductile behavior is highly desirable when the structure is called upon to absorb energy such as when subjected to strong ground motions (Taranath 1998).

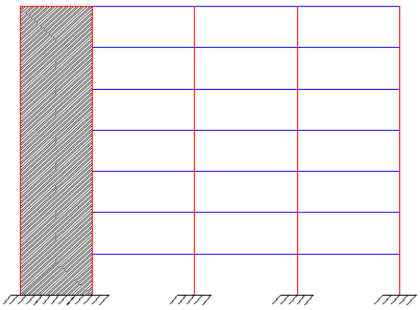


Figure 2.1 Wall-frame structure

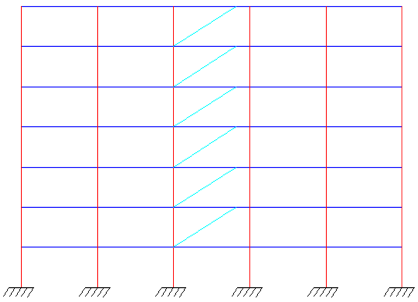


Figure 2.2 Eccentric braced frame

2.3 Floor Systems of Multistory Buildings

An appropriate floor system is an important factor in the overall economy of the building. Some of the factors that influence the choice of floor system are architectural. Other factors affecting the choice of floor system are related to its intended structural performance, such as whether it is to participate in the lateral load-resisting system, and to its construction, for example, whether there is urgency in the speed of erection.

2.3.1 Reinforced Concrete Floor Systems

Different types of RC floor systems are being used for building construction. Some typically used floor systems are described below in brief.

One-Way Slab on Beams or Walls

A solid slab of up to 200 millimeter thick shown in Figure 2.3, spanning continuously over walls or beams up to 7.3 meter apart, provides a floor system requiring simple formwork, with simple reinforcement. The system is heavy and inefficient in its use of both concrete and reinforcement. It is appropriate for use in cross-wall and cross-frame residential high-rise construction and when construction in a number of uninterrupted continuous spans lends itself to pre-stressing (Smith and Coull 1991).

One-Way Pan Joists and Beams

A thin, mesh-reinforced slab shown in Figure 2.4 sits on closely spaced cast-in-place joists spanning between major beams which transfer the loads to the columns. The slab may be as thin as 65 millimeter while the joists are from 150 to 500 millimeter in depth and spaced from 500 to 750 millimeter centers. The compositely acting slab and joists form in effect a set of closely spaced T-beams, capable of large, up to 12 meter, spans. The joists are formed between reusable pans that are positioned to set the regular width of the joist, as well as any special widths (Smith and Coull 1991).

One-Way Slab on Beams and Girders

A one-way slab shown in Figure 2.5 spans between beams at a relatively close spacing while the beams are supported by girders that transfer the load to the columns. The short spanning may be thin, from 75 to 150 millimeter thick, while the system is capable of providing long spans of up to 14 meter. The principal merits of the system are its long span capability and its compatibility with a two-way lateral load resisting rigid-frame structure (Smith and Coull 1991).

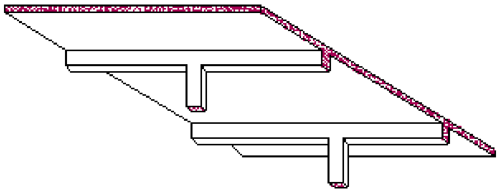


Figure 2.3 One-way slabs on beams or walls

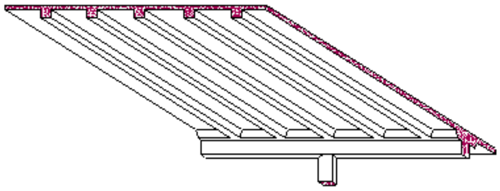


Figure 2.4 One-way pan joists and beams

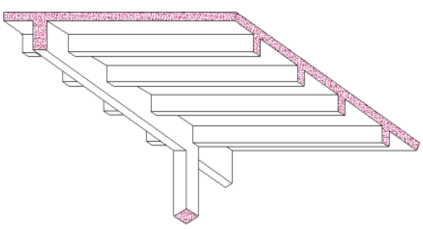


Figure 2.5 One-way slab on beams and girders

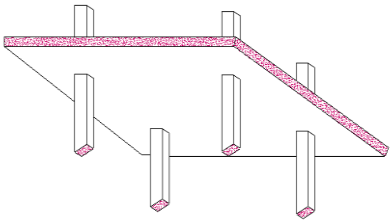


Figure 2.6 Two-way flat plate

Two-Way Flat Plate

A uniform thick, two-way reinforced slab shown in Figure 2.6 is supported directly by columns or individual short walls. It can span up to 8 meter in the ordinary reinforced form and up to 11 meter when post tensioned. Because of its simplicity, it is the most economical floor system in terms of form work and reinforcement. Its uniform thickness allows considerable freedom in the location of the supporting columns and walls and, with the possibility of using the clear soffit as a ceiling, it results in minimum story height (Smith and Coull 1991).

Two-Way Flat Slab

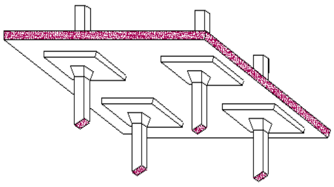
The flat slab shown in Figure 2.7 differs from the flat plate in having capitals and/or drop panels at the tops of the columns. The capitals increase the shear capacity, while the drop panels increase both the shear and negative moment capacities at the supports, where the maximum values occur. The flat slab is therefore more appropriate than the plate for heavier loading and longer spans and, in similar situations, would require less concrete and reinforcement. It is most suitably used in square, or near-to-square, arrangements (Smith and Coull 1991).

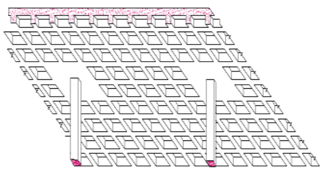
Waffle Flat Slabs

A slab shown in Figure 2.8 is supported by a square grid of closely spaced joists with filler panels over the columns. The slab and joists are poured integrally over square. Domed forms that are omitted around the columns to create the filler panels. The forms, which are of sizes up to 750 millimeter square and up to 500 millimeter deep, provide a geometrically interesting soffit, which is often left without further finish as the ceiling (Smith and Coull 1991).

Two-Way Slab and Beam

The slab shown in Figure 2.9 spans two ways between orthogonal sets of beams that transfer the load to the columns or walls. The two-way system allows a thinner slab and is economical in concrete and reinforcement. It is also compatible with a lateral load-resisting rigid-frame structure. The maximum length-to-width ratio for a slab to be effective in two directions is approximately 2.





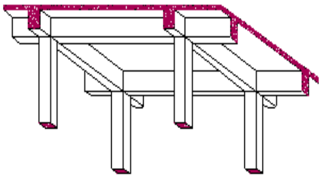


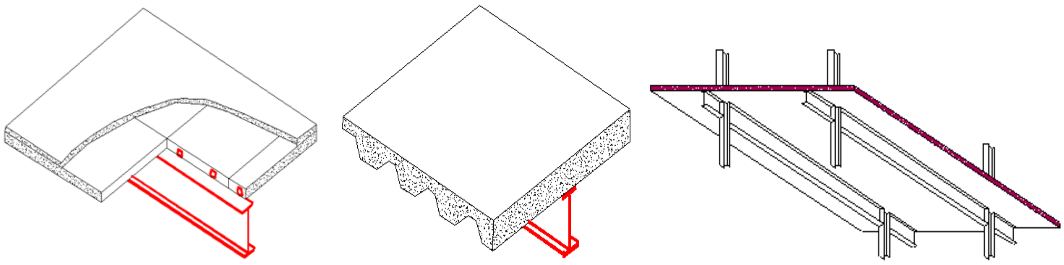
Figure 2.7 Two-way flat slab **Figure 2.8** Waffle slabs **Figure 2.9** Two-way slab and beam

2.3.2 Non-Composite Floor Systems of Steel Framing

The steel-framed floor system is characterized by a reinforced concrete slab supported on a steel form work consisting variously of joists, beams, and girders that transfer the gravity loading to the columns. The slab component is usually one-way with either a cast-in-place solid reinforced concrete slab from 100 to 175 millimeter thick, or a concrete on metal deck slab with a variety of possible section shapes and a minimum slab thickness from 65 millimeter, or a slab of precast units laid on steel beams and covered by a thin concrete topping. A major consideration in the weight and cost of a steel frame building is the weight of the slab. A floor arrangement with shorter spanning, thinner slabs is desirable. Longer span, closer spaced beams supporting a short-spanning slab is a typical arrangement meeting these arrangements (Smith and Coull 1991). The following types of steel framing are categorized according to the spanning arrangement of the supporting steel frame work.

One-Way Beam System

A rectangular grid of columns supports sets of parallel longer span beams at a relatively close spacing, with the slab spanning the shorter spans transversely to the beams. In cross-frame structures, the beams at partition lines may be deepened to participate in lateral load resisting rigid frames or braced bents. One way beam systems are shown in Figure 2.10.



(a) Precast units **(b)** One-way slab on metal deck **(c)** One-way beam system in steel

Figure 2.10 one-way beam systems

Two-Way Beam System

In buildings in which columns are required to be farther apart in both directions, a two-way frame system of girders and beams is often used, with the slab spanning between the beams. To minimize the total structural depth of the floor frame, the heavily loaded girders are aligned with the shorter span and the relatively lightly loaded secondary beams with the longer span. Two way beam system is shown in Figure 2.11.

Three-Way Beam System

In buildings in which the columns have to be very widely spaced to allow large column-free areas, a three-way beam system may be necessary. A deep lattice girder may form the primary component with beams or open web joists forming the secondary and tertiary systems. In each case the system is arranged to provide relatively short spans for the supported concrete slab. Three way beam system is shown in Figure 2.12.

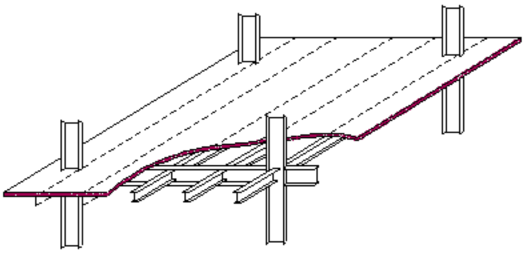


Figure 2.11 Two-way beam system

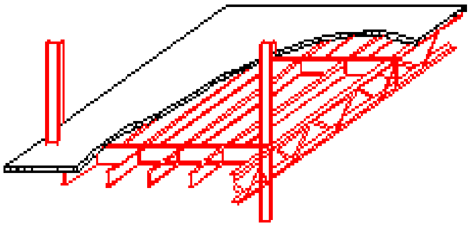
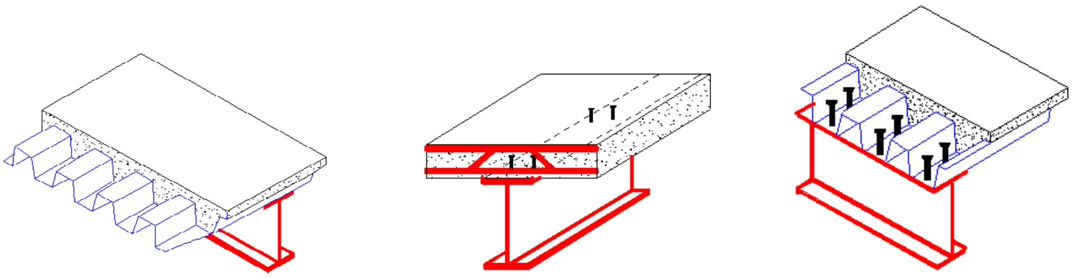


Figure 2.12 Three-way beam system

2.3.3 Composite Floor Systems of Steel Framing

The use of steel members to support a concrete floor slab offers the possibility of composite construction in which the steel members are joined to the slab by shear connectors so that the slab serves as a compression flange. In one simple and constructionally convenient slab system, steel decking, which in often used to act merely as rapidly erected permanent framework for a bar-reinforced slab, serves also as the reinforcement as the concrete slab in a composite role, using thicker wall sections with indentations or protrusions for shear connectors. Slabs may also be designed to act compositely with the supporting beams by the more usual forms of stud, angle, or channel shear connectors, so that the slab alone spans the short distance between the beams while the compositely acting slab and beam provide the supporting system. The further combination of a concrete slab on metal decking with shear connectors welded through to the supporting beam or truss is an efficient floor system. Composite floor system is shown in Figure 2.13.



(a) Steel deck composite slab **(b)** Composite frame **(c)** Composite frame and steel deck

Figure 2.13 Composite floor system

2.4 Load Considerations

Dead load and live load are gravity loads act vertically. Earthquake and wind load acts horizontally. These loads are basic loads to be considered for building design. BNBC 1993 is followed for load considerations.

Dead Loads

Dead load is the vertical load due to the weight of permanent structural and non structural components of a building such as walls, floors, ceilings, permanent partitions and fixed service equipments etc. Dead load for a structural member shall be assessed based on the forces due to:

- weight of the member itself,
- weight of all materials of construction incorporated into the building to be supported permanently by the member,
- weight of permanent partitions,
- weight of fixed service equipments, and
- net effect of pre-stressing.

When partition walls are indicated on the plans, their weight shall be considered as dead load acting as concentrated line loads in their actual positions on the floor. The loads due to anticipated partition walls, which are not indicated on the plans, shall be treated as live loads.

Weight of fixed service equipment and other permanent machinery, such as electrical feeders and other machinery, heating, ventilating and air-conditioning systems, lifts and escalators, plumbing stakes and risers etc. shall be included as dead load whenever such equipments are supported by structural members.

Live Loads

Live load is the load superimposed by the use or occupancy of the building not including the environmental loads such as wind load, rain load, earthquake load or dead load. The live loads used for the structural design of floors, roof and the supporting members shall be the greatest applied loads arising from the intended use or occupancy of the building, or from the stacking of materials and the use of equipment and propping during construction, but shall not be less than the minimum design live loads set out by the provisions of any standard code. For live load considerations BNBC 1993 is followed.

Pre-composite construction live load for composite floor system is 25 psf (1.2 kN/m^2) according to AISC Design Examples, Version 14 (AISC 2011).

When partitions, not indicated on the plans, are anticipated to be placed on the floors, their weight shall be included as an additional live load acting as concentrated line loads in an arrangement producing the most severe effect on the floor, unless it can be shown that a more favorable arrangement of the partitions shall prevail during the future use of the floor. In the case of light partitions, wherein the total weight per meter run is not greater than 5.5 kN, a uniformly distributed live load may be applied on the floor in lieu of the concentrated line loads. Such uniform live load per square meter shall be at least 33% of the weight per meter run of the partitions, subject to a minimum of 1.2 kN/m^2 (BNBC 1993).

Live loads on regular purpose roofs shall be the greatest applied loads produced during use by movable objects such as planters and people, and those induced during maintenance by workers, equipments and materials but shall not be less than those specified in BNBC 1993.

Reduction of live load is permitted for primary structural members supporting floor or roof, including beam, girder, truss, flat slab, flat plate, column, pier, footing and the like. Where applicable, the reduced live load on a primary structural member shall be obtained by multiplying the corresponding unreduced uniformly distributed live load with an appropriate live load reduction factor as per BNBC 1993.

Lateral Loads

Lateral loads are wind load and earth quake load. The minimum design wind load on buildings and components thereof shall be determined based on the velocity of the wind, the shape and size of the building and the terrain exposure condition of the site as set forth by the provisions of BNBC 1993**.**

Minimum design earthquake forces for buildings, structures or components thereof shall be determined in accordance with the provisions of BNBC 1993. For primary framing systems of buildings or structures, the design seismic lateral forces shall be calculated by the equivalent static force method. Overall design of buildings and structures to resist seismic ground motion and other forces shall comply with the applicable design requirements.

2.5 Load Combinations

Buildings, foundations and structural members shall be investigated for adequate strength to resist the most unfavorable effect resulting from the various combinations of loads provided in this section. The most unfavorable effect of loads may also occur when one or more of the contributing loads are absent or act in the reverse direction. Loads such as F, H, or S shall be considered in design when their effects are significant. Floor live loads shall not be considered where their inclusion result in lower stresses in the member under consideration. The most unfavorable effects from both wind and earthquake loads shall be considered where appropriate, but they need not be assumed to act simultaneously.

2.5.1 BNBC Load Combinations

Load combinations of ASD and USD methods for RC and steel structure are stated here as per BNBC 1993. These combinations are followed for the assigned design.

ASD combinations

Provisions of this section shall apply to all construction permitting their use in proportioning structural members by allowable stress design method. When this method is used in designing structural members, all loads listed herein shall be considered to act in the following combinations. The combination that produces the most unfavorable effect shall be used.

1. D
2. D+L
3. D+S
4. D+(W or E)
5. 0.9D+(W or E)
6. D+(H or F)
7. D+L+(H or F)

8. D+S+L
9. D+S+(W or E)
10. D+L+(W or E)
11. D+L+(H or F)+(W or E)
12. D+S+L+(H or F)+(W or E)

The maximum permissible increase in the allowable stresses of all materials and soil bearing capacities for working (or allowable) stress design method, when load combinations 7 through 11 mentioned above is used, shall be 33%.

USD combinations

When strength design method is used, structural members and foundations shall be designed to have strength not less than that required to resist the most unfavorable effect of the combinations of factored loads are listed below.

For RC and masonry structures**:**

1. 1.4D
2. 1.4D+1.7L
3. 1.4D+1.4S
4. 0.9D+ 1.3(W or 1.1E)
5. 0.9D+1.7(H or F)
6. 1.4D+1.7L+1.7(H or F)
7. 0.75[1.4 D+1.4S+1.7L]
8. 0.75[1.4 D+1.4S+1.7(W or 1.1E)]
9. 0.75[1.4D+1.7L+1.7W]
10. 0.75[1.4D+1.7L+1.7(H or F)+1.7(W or 1.1E)]
11. 0.75[1.4D+1.4S+1.7L+1.7(H or F)+1.7(W or 1.1E)]
12. 1.4(D+L+E)

For steel structures**:**

1. 1.4D
2. 1.2D+1.6L_f +0.5(L_r or P)
3. 1.2D+1.6(L_r or P) +(0.5L_f or 0.8W)
4. 1.2D+ 1.3W+0.5 L_f +0.5(L_r or P)

5. $1.2D+1.5E+0.5\ L_f$
6. $0.9D+(1.3W$ or $1.5E)$

Where D=dead load, E=earthquake load, F=loads due to fluids, H=loads due to weight and lateral pressure of soil and water in soil, $L=L_f + (L_r$ or $P)$, L_f=live loads due to intended use and occupancy, L_r=roof live loads, P= loads due to initial rain water ponding, S=self-straining forces and W=wind load.

2.5.2 AISC LRFD Load Combinations

Load combinations as per AISC LRFD 1993 for design of steel structures are stated here. These combinations are used for the assigned design. The combinations are as follows.

$1.4D$

$1.2D+1.6L+0.5(L_r$ or S or $R)$

$1.2D+1.6(L_r$ or S or $R)+(\ 0.5L$ or $0.8W)$

$1.2D+1.3W+0.5L+0.5(L_r$ or S or $R)$

$1.2D\pm1.0E+0.5L+0.2S$

$0.9D\pm(1.3W$ or $1.0E)$

Where D=dead load, L=live load, L_r=roof live load, W=wind load, S=snow load, E=earthquake load and R=rain water or ice load.

2.5.3 ACI Load Combinations

Load combinations for USD method as per ACI 2008 for design of RC structures are stated here. These combinations are used for the assigned design. The combinations are as follows.

$1.2D+1.6L$

$1.4(D+F)$

$1.2(D+F+T)\ +1.6(L+H)+0.5(L_r$ or S or $R)$

$1.2D+1.6(L_r$ or S or $R)+(1.0L$ or $0.8W)$

$1.2D+1.6W+1.0L+0.5(L_r$ or S or $R)$

$0.9D+1.6W+1.6H$

$1.2D+1.0E+1.0L+0.2S$

0.9D+1.0E+1.6H

Where, D=Dead load, E=Earthquake load, F=Fluid pressure, H=Weight or pressure from soil, L=Live load, L_r=Roof live load, W=Wind load, S=Snow load, R=Rain water load, T=Cumulative effects of temperature, creep, shrinkage, and differential settlement.

2.6 Foundation Systems

Before foundation design of a building, appropriate geotechnical investigation of the proposed project site is required. Performing field investigation, necessary geotechnical data and sample collection, laboratory test, preparation of geotechnical investigation report with necessary technical information and recommendations are required for foundation design. Depending on sub-soil condition and calculated load for foundation design, type of foundation becomes deep or shallow. If the bearing capacity of sub-soil in reasonable to bear the load from foundation then shallow foundations may be selected, otherwise deep foundation may be required.

Shallow Foundations

Spread footing (square or rectangular), combined footing, trapezoidal footing, continuous footing, strap footing, grid foundation, mat or raft foundation etc. are most common shallow foundations. When bearing capacity is good enough and design load is medium then spread footing is most suitable. But when bearing capacity is medium then mat foundation, grid foundation, continuous footing etc. may be appropriate.

Deep Foundations

In case of heavy design load and low bearing capacity for a particular soil condition, deep foundation may be required. R.C.C. cast in situ pile foundation, R.C.C. pre-cast pile foundation, caisson foundation, well foundation, drilled pier, concrete filled steel tube pile foundation, hollow steel tube pile are most common deep foundations. In Bangladesh, R.C.C. cast in situ pile is widely used. Pre-cast piles are also being used in many projects.

2.7 Connection Systems

Connection system of structure is very important for structural stability, response to load, structural behavior and load transfer mechanism. Connection may be rigid, semi-rigid and hinge depending on restraint condition. Steel and RC structural connection is discussed below.

2.7.1 Connection of Steel Structure

Simple shear connection and moment connection are commonly used for steel structure. Moment connection may be partially or fully restrained.

2.7.1.1 Simple Connections

A simple connection transmits a negligible moment across the connection. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure. Inelastic rotation of the connection is permitted (AISC Specification, 2005).

2.7.1.2 Moment Connections

A moment connection transmits moment across the connection. Two types of moment connections, fully restrained and partially restrained, are permitted, as specified below.

Fully-Restrained (FR) or Rigid Moment Connections

A fully-restrained (FR) 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. An FR connection shall have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states (AISC Specification, 2005).

Partially-Restrained (PR) or Semi-Rigid Moment Connections

Partially-restrained (PR) moment connections or semi-rigid 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 shall 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 (AISC Specification, 2005).

Semi-rigid connections, as the name implies, are those with rotational characteristics intermediate in degree between fully rigid and simple connections. These connections offer known rotational restraint at the beam ends resulting in significant reduction in mid-span gravity moments. Although several specifications such as the AISC, the British, and the Australian codes permit semi-rigid connections it has rarely been used because of the difficulty in predicting the rather complex response of these connections. However, reasonable success has been by another type of partially rigid connection which AISC designates as Type 2 wind connection, with similar provisions found in the British and Australian codes (Taranath 1998).

Although the AISC specification permits the designer to take advantage of reduction in the mid-span moment of a beam with semi-rigid connections, in practice this procedure has not found wide acceptance primarily because of lack of reliable analytical techniques. The type 2 wind connection, which basically ignores the beam restraint for gravity loads, has found relatively greater acceptance. Wind connection is designed to resist wind moment (Taranath 1998).

Partially restrained or semi-rigid framing occurs when rotational restraint is approximately between 20% and 90% of that necessary to prevent relative angle change. This means that with semi-rigid framing the moment transmitted across the joint is neither zero (or a small amount) as in simple framing, nor is it the full continuity as assumed in elastic rigid-frame analysis. In LRFD the use of PR connection "depends on the evidence of predictable proportion of full end restraint." In ASD, the design of semi-rigid connections requires a "dependable and known moment capacity intermediate in degree between the rigidity of fully restraint and flexibility of unrestrained connection. Semi-rigid connections are not used in structures when plastic analysis is used in design, and are not commonly used in allowable stress design because of the difficulty in obtaining the moment-rotation relationship for a given connection (Salmon and Johnson 1995). However, with great availability of high-strength steels required in designing this type of connection, Salmon and Johnson (1995) believe that use of semi-rigid connections will increase.

Analysis of frames that incorporate Type 2 wind and semi-rigid (Type 3) connections must include considerations of**:**

- Connection ductility.
- Evaluation of the drift characteristics of frames with less than fully rigid connections.
- Effect of partial restraints on column and frame stability.

2.7.2 Plastic Hinge for Steel Structure

Redistribution of the moments occurs during loading beyond the elastic range in usual statically indeterminate situations; that is, the bending moment diagram after a plastic hinge has occurred will no longer be proportional to the elastic bending moment diagram. Once the plastic moment strength M_p has been reached, the section can offer no additional resistance to rotation, behaving as a hinge but with constant resistance M_p, a condition known as a plastic hinge. In general, any combination of three hinges, real or plastic, in a span will result in a collapse mechanism (Salmon and Johnson 1995).

2.7.3 RC Structural Joint

RC member connections are inherently fully restrained or rigid connection. All the connections like column-beam connections, beam-girder connections, beam-slab connections etc. are usually cast monolithically by concrete.

2.7.4 RC Expansion Joint

Spacing of a functional (contraction and expansion) joints depends upon a great number of factors: shrinkage properties of the concrete, type of exposure to temperature and humidity, resistance to movement (restraint), thickness of members, amount of reinforcement, structural function of the member, external loads, soil conditions, structural configurations, and other conditions (Fintel 1986). Many of these factors are exclusive variables, sometimes difficult to establish. As a consequence, both experience and opinion on joint spacing vary greatly.

In reinforced concrete elements, joint spacing and reinforcement are interrelated variables, and the choice of one should be related to the other. As yet, however, a reliable relationship between the two quantities does not appear to have been established. Sufficient steel must be included to control cracking between the joints. If the joint spacing is increased, the reinforcement must be increased correspondingly to control cracking over the longer distance. There is a considerable divergence of opinion on spacing of movement joints (expansion and contraction) with recommendations for expansion joints varying from 30 to 60 meter, while for contraction joints, they vary from a few meter up to 25 meter.

Spacing of expansion joints in buildings is a controversial issue. There is a great divergence of opinion concerning the importance of expansion joints in concrete construction. Some experts recommended joint spacing as low as 9 meter while others consider expansion joints entirely unnecessary. Joint spacing of roughly 45 to 60 meter for concrete structures seems to be typical ranges recommended by various authorities. Divergent viewpoints are reflected both in private practice and in building codes. The existence of such opposing opinions, which, obviously, cannot be equally valid as a consideration in a single structure, is nonetheless understandable, since it is based on divergence of previous experience.

Those who advocate the complete omission of joints in concrete construction state that drying shrinkage is greater than the expansion caused by a $38^{o}C$ increase in temperature; therefore, any temperature increase will tend to close up shrinkage cracks, and there will be practically no compressive stress in the concrete due to thermal expansion. In a 1940 report of a joint committee (AIA, AISC, ACI, AREA, PCA) it was suggested that, in localities, with large temperature ranges, expansion joints should be provided every 61 meter. In middle climates, 92 meter was suggested.

In the 1940s a distinct trend started toward the elimination of expansion joints in long buildings. This trend is continuous into the present time. Even in locations with large temperature ranges, buildings up to 122 and 153 meter have been constructed without expansion joints, and seemingly the performance has been satisfactory. The following are examples of such buildings: The General Accounting Office Building, Washington, D.C.

The 1972 "Minimum Property Standards-Manual of Acceptable Practices" by the FHA recommends that spacing of expansion joints for buildings not exceeded the values given in Table 2.1.

Table 2.1 Spacing of expansion joint

Types of building	Outside temperature variations	Maximum joint spacing(meter)
Heated	Up to 21^{o}C	183
	Above 21^{o}C	122-153
Unheated	Up to 21^{o}C	92
	Above 21^{o}C	61

2.8 Serviceability Criteria

Serviceability is defined in the AISC Specification as "a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage". Although serviceability issues have always been a design consideration, changes in codes and materials have added importance to these matters. The shift to a limit-states basis for design is one example. Since 1986, both the AISC LRFD and AISC ASD Specifications have been based upon the limit-states design approach in which two categories of limit states are recognized: strength limit states and serviceability limit states. Strength limit states control the safety of the structure and must be met. Serviceability limit states define the functional performance of the structure and should be met. The distinction between the two categories centers on the consequences of exceeding the limit state. The consequence of exceeding a strength limit may be buckling, instability, yielding, fracture, etc. These consequences are the direct response of the structure or element to load. In general, serviceability issues are different in that they involve the response of people and objects to the behavior of the structure under load (West et al. 2003). For example, the occupants may feel uncomfortable if there are unacceptable deformations, drifts, or vibrations.

Whether or not a structure or element has passed a limit state is a matter of judgment. In the case of strength limits, the judgment is technical and the rules are established by building codes and design specifications. In the case of serviceability limits, the judgments are frequently non-technical. They involve the perceptions and expectations of building owners and occupants. Serviceability limits have, in general, not been codified, in part because the appropriate or desirable limits often vary from application to application. As such, they are more a part of the contractual agreements with the owner than life-safety related. Thus, it is proper that they remain a matter of contractual agreement and not specified in the building codes (West et al. 2003).

In a perfect world the distinction between strength and serviceability would disappear. There would be no problems or failures of any kind. In the real world all design methods are based upon a finite, but very small probability of exceedance. Because of the non-catastrophic consequences of exceeding a serviceability limit state, a higher probability of exceedance is allowed by current practice than for strength limit states. The foregoing is not intended to say that serviceability concerns are unimportant. In fact, the opposite is true. By having few codified standards, the designer is left to resolve these issues in consultation with the owner to determine the appropriate or desired requirements (West et al. 2003).

Serviceability problems cost more money to correct than would be spent preventing the problem in the design phase. Perhaps serviceability discussions with the owner should address the trade-off between the initial cost of the potential level of design vs. the potential mitigation costs associated with a more relaxed design. Such a comparison is only possible because serviceability events are by definition not safety related. The Metal Building Manufactures Association (MBMA) in its Common Industry Practices (MBMA, 2002) states that the customer or his or her agent must identify for the metal building engineer any and all criteria so that the metal building can be designed to be “suitable for its specific conditions of use and compatible with other materials used in the Metal Building System.” Nevertheless, it also points out the requirement for the active involvement of the customer in the design stage of a structure and the need for informed discussion of standards and levels of building performance. Likewise the AISC Code of Standard Practice (AISC, 2000) states that in those instances where the fabricator has both design and fabrication responsibility, the owner must provide the “performance criteria for the structural steel frame.” Numerous serviceability design criteria exist, but they are spread diversely through codes, journal articles, technical committee reports, manufacturer’s literature, office standards and the preferences of individual engineers (West et al. 2003).

2.8.1 Story Drift and Top Deflection

Lateral deflection or drift is the magnitude of displacement at the top of a building relative to its base. The ratio of the total lateral deflection to the building height, or the story deflection to the story height, is referred to as the deflection index. In the absence of code limitations in the past, buildings were designed for wind loads with arbitrary values of drift, ranging from about **1/300** to **1/600**, depending on the judgment of the engineer. Deflections based on drift limitation of about **1/300** used several decades ago were computed assuming the wind forces to be resisted by the structural frame alone. In reality, the heavy masonry partitions and exterior cladding common to buildings of that period considerably increased the lateral stiffness of such structures. In contrast, in most buildings that have been constructed in recent years, the frame alone resists the lateral forces. The dry-wall interior partition and the light curtain-wall exterior contribute little to the lateral resistance of modern buildings (Fintel 1986).

Up to 1983, only the Uniform Building Code, BOCA, and the National Building Code of Canada, among North American model building codes, specify a maximum value of the deflection index **1/500**, corresponding to the design wind loading. Also, ACI Committee 435 recommends a drift limit of **1/500**. At that time, many engineering offices, owing to competitive pressures, have somewhat relaxed the drift criterion by allowing an overall drift of slightly over **H/500** with the maximum drift in any one story not to exceed **H/400**. Also, in cases where wind tunnel studies indicate wind forces in the building to be smaller than those specified in the code, designers take the liberty of applying the **H/500** criterion to the smaller (wind tunnel) wind forces. Most of the modern tall reinforced concrete buildings containing shear walls have computed deflections ranging between **H/800** and **H/1200** due to the inherent rigidity of the shear wall-frame interaction (Fintel 1986).

Sound engineering judgment is required when deciding on the drift index limit to be imposed. However, for conventional structures, the preferred acceptable range is 0.0015 to 0.003 (that is, approximately 1/650 to 1/350). It does not necessarily follow that the dynamic comfort criteria will also be satisfactory. Generally, lower values should be used for hotels or apartment buildings than for office building, since noise and movement tend to be more disturbing in the former. Consideration may be given to whether the stiffening effects of any internal partitions, in-fills, or claddings are included in the deflection calculations. In addition to static deflection calculations, the question of the dynamic response, involving the lateral acceleration, amplitude, and period of oscillation, may also have to be considered (Smith and Coull 1991).

Lateral deflections must be limited to prevent second-order P-Delta effects due to gravity loading being of such a magnitude as to precipitate collapse. In terms of the serviceability limit states, deflections must first be maintained at a sufficiently low level to allow the proper functioning of nonstructural components such as elevators and doors; second, to avoid distress in the structure, to prevent excessive cracking and consequent loss of stiffness, and to avoid any redistribution of load to non-load-bearing partitions, infills, cladding, or glazing; and third, the structure must be sufficiently stiff to prevent dynamic motions becoming large enough to cause discomfort to occupants, prevent delicate work being undertaken, or affect sensitive equipment. In extreme circumstances, it may be necessary to add dampers, which may be of the passive or active type (Smith and Coull 1991).

As per BNBC 1993, story drift, Δ shall be limited as follows,

(i) $\Delta \leq 0.04h/R \leq 0.005h$ for $T < 0.7$ second.

(ii) $\Delta \leq 0.03h/R \leq 0.004h$ for $T \geq 0.7$ second.

(iii) $\Delta \leq 0.0025h$ for unreinforced masonry structure.

Where, h=height of the building or structure, T=fundamental period of vibration in seconds of the structure for the direction under consideration and R=response modification factor (shall be applicable only when earthquake forces are present)

.

The drift limits may be exceeded where it can be demonstrated that greater drift can be tolerated by both structural and nonstructural elements without affecting life safety.

2.8.2 Vertical Deflection

In general, “deflections” refers to how much a material can bend and flex over the course of its lifetime as part of a building components (West et al. 2003). According to AISC Design Examples, Version 14 (AISC 2011), pre-composite deflections of steel girders and beams must be smaller than equal to L/360 or 1 inch (25 mm) which is smaller. Composite deflections of steel girders and beams also must be smaller than equal to L/360 or 1 inch (25 mm) considering 50% live load which is smaller. Allowable limit for deflection adopted from IBC is shown in Table 2.2.

Table 2.2 Deflection limits, adopted from IBC

Building component		Live load deflection	Snow or Wind load deflection	Dead load+ Live load deflection
Roof member	Supporting plastered ceiling	L /360	L /360	L /240
	Supporting non-plastered ceiling	L /240	L /240	L /180
	Not supporting ceiling	L /180	L /180	L /120
Floor members		L /360	-	L /240

Where, L=Length of member

2.8.3 Floor Vibrations and Dynamic Response

For floor serviceability, stiffness and resonance are dominant considerations in the design of steel floor structures and footbridges. The first known stiffness criterion appeared nearly 170 years ago. Tredgold (1828) wrote that girders overlong spans should be "made deep to avoid the inconvenience of not being able to move on the floor without shaking everything in the room". Traditionally, soldiers "break step" when marching across bridges to avoid large, potentially dangerous, resonant vibration. A traditional stiffness criterion for steel floors limits the live load deflection of beams or girders supporting "plastered ceilings" to span/360. This limitation, along with restricting member span to depth rations to 24 or less, have been widely applied to steel framed floor systems in an attempt to control vibrations, but with limited success. Resonance has been ignored in the design of floors and footbridges until recently. Approximately 30 years ago, problems arose with vibrations induced by walking on steel-joist supported floors that satisfied traditional stiffness criteria. Since that time much has been learned about the loading function due to walking and the potential for resonance (Murray et al. 1997).

More recently, rhythmic activities, such as aerobics and high-impact dancing, have caused serious floor vibration problems due to resonance. A number of analytical procedures have been developed which allow a structural designer to assess the floor structure for occupant comfort for a specific activity and for suitability for sensitive equipment. Generally, these analytical tools require the calculation of the first natural frequency of the floor system and the maximum amplitude of acceleration, velocity or displacement for a reference excitation. An estimate of damping in the floor is also required in some instances. A human comfort scale or sensitive equipment criterion is then used to determine whether the floor system meets serviceability requirements. Some of the analytical tools in corporate limits on acceleration into a single design formula whose parameters are estimated by the designer (Murray et al. 1997).

Human Response to Floor Motion

Human response to floor motion is a very complex phenomenon, involving the magnitude of the motion, the environment surrounding the sensor, and the human sensor. A continuous motion (steady-state) can be more annoying than motion caused by an infrequent impact (transient). The threshold of perception of floor motion in a busy workplace can be higher than in a quiet apartment. The reaction of a senior citizen living on the fiftieth floor can be considerably different from that of a young adult living on the second floor of an apartment complex, if both are subjected to the same motion. The reaction of people who feel vibration depends very strongly on what they are doing. People in offices or residences do not like "distinctly perceptible" vibration (peak acceleration of about 0.5 percent of the acceleration of gravity, g), whereas people taking part in an activity will accept vibrations approximately 10 times greater (5 percent g or more). People dining beside a dance floor, lifting weights beside an aerobics gym, or standing in a shopping mall, will accept something in between (about 1.5 percent g). Sensitivity within each occupancy also varies with duration of vibration and remoteness of source. The above limits are for vibration frequencies between 4Hz and 8Hz. Outside this frequency range, people accept higher vibration accelerations (Murray et al. 1997).

Human Response to Acceleration

Considerable research in the last 20 years has been conducted on the subject of determining perception threshold values for acceleration caused by building motion (Chen and Robertson, 1972; Khan and Parmelee, 1971 and ASCE, 1981). Much of this work has also attempted to formulate design guidelines for tolerance thresholds to be used in the design of tall and slender buildings. Some of the earliest attempts to quantify the problem were performed by Chang (Chang, 1967 and Chang, 1972) who proposed peak acceleration limits for different comfort levels that were extrapolated from data in the aircraft industry. Chang's proposed limits are stated in Table 2.3.

Table 2.3 Human response to acceleration

Peak Acceleration	Comfort Limit
< 0.5% g	Not Perceptible
0.5% to 1.5% g	Threshold of Perceptibility
1.5% to 5.0% g	Annoying
5% to 15.0% g	Very Annoying
> 15% g	Intolerable

Building Drift and Motion Perception

Engineers designing tall buildings have long recognized the need for controlling annoying vibrations to protect the psychological well being of the occupants. Prior to the advent of wind tunnel studies this need was addressed using rule-of-thumb drift ratios of approximately 1/400 to 1/600 and code specified loads. Recent research (Islam, Ellingwood and Corotis, 1990), based on measurement of wind forces in the wind tunnel, has clearly shown that adherence to commonly accepted lateral drift criteria, per se, does not explicitly ensure satisfactory performance with regard to motion perception. The results of one such study (Islam, Ellingwood and Corotis, 1990) for two square buildings having height/width ratios of 6/1 and 8/1 where each is designed to varying drift ratios. At drift ratios of 1/400 and 1/500 neither building conforms to acceptable standards for acceleration limits. The reason that drift ratios by themselves do not adequately control motion perception is because they only address stiffness and do not recognize the important contribution of mass and damping, which together with stiffness, are the predominant parameters affecting acceleration in tall buildings components (West et al. 2003).

Wind Induced Motion

If a tall flexible structure is subjected to lateral or torsional deflections under the action of fluctuating wind loads, the resulting oscillatory movements can induce a wide range of responses in the building's occupants, ranging from mild discomfort to acute nausea. Motions that have psychological or physiological effects on the occupants may thus result in an otherwise acceptable structure becoming an undesirable or even un-rentable building. There are as yet no universally accepted international standards for comfort criteria, although they are under consideration, and engineers must base their design criteria on an assessment of published data. It is generally agreed that acceleration is the predominant parameter in determining human response to vibration, but other factors such as period, amplitude, body orientation, visual and acoustic cues, and even past experience can be influential (Smith and Coull 1991)

Dynamic wind pressure sinusoidal or narrow-band random vibration motions of the building, which will generally oscillate in both along-wind and cross-wind directions, and possibly rotate about a vertical axis. The magnitudes of the three displacement components will depend on the velocity distribution and direction of the wind, and on the shape, mass, and stiffness properties of the structure. In certain cases, the effects of cross-wind motions of the structure may be greater than those due to the along-wind motions (Smith and Coull 1991).

2.8.4 Fire Resistance

An important objective of building codes and regulations is to provide a fire-resistive built environment. Thus, building fire safety regulations contain numerous provisions including directives for the minimum number of exits, the maximum travel distances to exits, minimum exit widths, fire compartment requirements, fire detection and suppression mandates, and the protection of structural members in buildings.

Although structural steel offers the advantage of being noncombustible, the effective yield strength and modulus of elasticity reduce at elevated temperatures. The yield strength of structural steel maintains at least 85 percent of its normal value up to temperatures of approximately 800°F (427°C). The strength continues to diminish as temperatures increase and at temperatures in the range of 1300 °F (704 °C), the yield strength may be only 20 percent of the maximum value. The modulus of elasticity also diminishes at elevated temperatures. Thus, both strength and stiffness decrease with increases in temperature. Measures can be taken to minimize or eliminate adverse effects. An obvious approach is to eliminate the heat source by extinguishing the fire or by generating an alert so that an extinguishing action can be initiated. Extinguishing systems such as sprinklers and smoke and heat detection devices are responses to this approach and are classified as active fire protection systems (Ruddy et al. 2003).

Another approach to improving the fire safety of a steel structure is to delay the rate of temperature increase to the steel to provide time for evacuation of the environment, to allow combustibles to be exhausted without structural consequence, and /or to increase the time for extinguishing the fire. This approach, which involves insulating the steel or providing a heat sink, is classified as a passive fire protection system. Such a system is using Spray-Applied Fire Resistive Material (SFRM). The typical approach to satisfying the passive protection system objective is prescriptive. Buildings are classified according to use and occupancy by the building code. For each occupancy classification there are height and area limitations that are dependent upon the level of fire resistance provided. For instance, building providing for business uses may have a height and floor area requiring building elements to be noncombustible and have a fire resistance rating of 2 hours. Then a tested floor assembly that provides a 2-hour fire resistance rating is identified and, as necessary, adjustments to the specifics of the tested assembly are made to match the actual construction. Thus, the required level of fire resistance is provided based on tests and extrapolation of test results. Improving the fire resistance of steel-framed structures using a passive system is only one of the strategies for providing fire-safe structures. Improvements in fire safety are most effective when used in conjunction with active systems (Ruddy et al. 2003).

An alternative approach to fire-safe construction is performance based. Under this option, calculations are prepared to predict a level of performance of the structure in a fire environment. Extensive research is progressing toward a thorough understanding of the behavior of steel-framed structures when exposed to fire, and an increase in the use of alternative design methods is inevitable (Ruddy et al. 2003).

2.8.5 Camber

Camber may or may not be a solution to a serviceability issue. In most instances, the amount of total movement is of concern rather than the relative movement from the specified floor elevation, in which case camber is not an appropriate solution. There are, however, situations where camber is appropriate, such as in places where it is possible to sight down the underside of exposed framing.

Camber tolerances are established in the AISC Code of Standard Practice as follows "For beams that are equal to or less than 50 ft in length, the variation shall be equal to or less than minus zero/plus $^{1}/2$ in. For beams that are greater than50 ft in length, the variation shall be equal to or less than minus zero/plus $^{1}/2$ in. plus $^{1}/8$ in. for each 10 ft or fraction thereof in excess of 50 ft in length." These tolerances are set with the worthy goal of ensuring positive camber, but it should be noted that there is a bias toward over cambering. The AISC Code of Standard Practice states "For the purpose of inspection, camber shall be measured in the Fabricator's shop in the unstressed condition."

This requirement is further amplified which states: "Inspection of shop work by the inspector shall be performed in the fabricator's shop to the fullest extent possible." Again states: "Rejection of material or workmanship that is not in conformance with the contract documents shall be permitted at any time during the progress of the work." The inspection of camber is an exception to this general principle. Unlike other physical characteristics of a fabricated beam or girder, such as yield strength, dimensions, welds, etc., the camber in a beam can change as the member is handled, shipped, unloaded and raised into position. The Code commentary provides the following explanation of this phenomenon. Camber can vary from that induced in the shop due to factors that include:

a. The release of stresses in members over time and in varying applications,

b. The effects of the dead weight of the member,

c. The restraint caused by the end connections in the erected state, and

d. The effects of additional dead load that may ultimately be intended to be applied, if any.

Because of the unique nature of camber in beams and the limits on the inspection for conformity to the project requirements for camber, it is incumbent on the specifier to recognize these limits and prepare the construction documents accordingly. It is common practice not to camber beams when the indicated camber is $^{3}/4$ in. or less. The AISC Code of Standard Practice provides that if no camber is specified, horizontal members are to be fabricated and erect beams with "incidental" camber upward. The AISC Code also provides that beams received by the Fabricator with 75 percent of the specified camber require no further cambering. Because of the provisions, it should be expected that all framing members should have at least some upward camber at the initiation of concreting operations. However, given the limits presented there will be instances of downward deflection below level during concreting. To control the excessive accumulation of concrete in the deflected bay Ruddy (1986), quoting Fisher/West, recommends that the total accumulated deflection in a bay due to dead load be limited to L/360, not to exceed 1 in. The foregoing discussion on determining and specifying camber is intended to impress upon the designer of the framing to be judicious in determining cambers and to be pro-active in communicating the basis of the camber determinations (West et al. 2003).

2.8.6 Other Serviceability Requirements

Other important serviceability requirements are expansion and contraction, connection slip, corrosion, fatigue, ponding of rain water on roof, durability, column shortening, long term deflection etc. for steel structure.

Expansion and Contraction

Expansion and contraction is discussed to a limited extent. The goal is to discuss those aspects of primary and secondary steel framing behavior as they impact non-structural building components. For many types of low-rise commercial and light industrial projects, expansion and contraction in a limited context are rarely an issue. This does not mean that the topic of expansion and contraction is unimportant and, of course, the opposite is true. For large and/or tall structures, careful consideration is required to accommodate absolute and relative expansion and contraction of the framing and the non-structural components (West et al. 2003).

Connection Slip

The various drift and deflection limits include the movements due to connection slip. Where connection slip, or especially the effect of accumulated connection slip in addition to flexural and/or axial deformations, will produce movements in excess of the recommended guidelines, slip-critical joints should be considered. Slip-critical joints are also required in specific instances. It should be noted that joints made with snug-tightened or pre-tensioned bolts in standard holes will not generally result in serviceability problems for individual members or low-rise frames components (West et al. 2003). Careful consideration should be given to other situations components.

Corrosion

Corrosion, if left unattended, can lead to impairment of structural capacity. Corrosion is also a serviceability concern as it relates to the performance of non-structural elements maintenance. The primary concerns are the control or elimination of staining of architectural surfaces and prevention of rust formation, especially inside assemblies where it can induce stresses due to the expansive nature of the oxidation process components (West et al. 2003). Again, the solutions are proper detailing and maintenance.

Fatigue

Fatigue criteria applies to members and connections subject to high cycle loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the limit state of fatigue.

The provisions for fatigue apply to stresses calculated on the basis of service loads. The maximum permitted stress due to un-factored loads is $0.66F_y$. Stress range is defined as the magnitude of the change in stress due to the application or removal of the service live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation. In the case of complete-joint-penetration butt welds, the maximum design stress range applies only to welds with internal soundness meeting the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1. No evaluation of fatigue resistance is required if the live load stress range is less than the threshold stress range, F_{TH}. No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 20,000.The cyclic load resistance determined by the provisions of Appendix 3 of AISC specification, 2005 is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions. The cyclic load resistance determined by the provisions of Appendix 3 of AISC specification, 2005 is applicable only to structures subject to temperatures not exceeding 300◦F(150◦C). The engineer of record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections.

Ponding of Water on Roof

When members of a flat roof system deflect, a bowl-shaped volume is created which is capable of retaining water. As water begins to accumulate, deflection increases to provide an increased volumetric capacity. This cyclical process continues until either (i) the succeeding deflection increments become smaller and equilibrium is reached; or (ii) succeeding deflection increments are increasing, the system becomes unstable, and collapse occurs. This retention of water which results solely from the deflection of flat roof framing is what is referred to as ponding. From a serviceability standpoint, this ponding of water is a major reason for splitting of roof membranes, resulting in costly replacement of both the membrane and the insulation (Salmon and Johnson 1995).

To prevent ponding of water accumulated on flat roofs, the 1963 AISC Specification required supporting members to satisfy the limitation, $L/d \leq 600/f_b$. Where, f_b is the computed service load bending stress in ksi, L is length of beam and d is depth of beam. Using this equation, it would correspond roughly to a deflection limitation, L/240 on a simply supported span. Avoidance of ponding is much more complex than indicated by the above limitation.

Long Term Deflection

In very tall concrete buildings, the cumulative vertical movements due to creep and shrinkage may be sufficiently large to cause distress in nonstructural elements, and to induce significant structural actions in the horizontal elements, especially in the upper region of the building. The differential movements due to creep and shrinkage must be considered structurally and accommodated as far as possible in the architectural details at the design stage. In buildings with partially or fully exposed exterior columns, significant temperature differences may occur between exterior and interior columns, and any restraint to their relative deformations will induce stresses in the members concerned (Smith and Coull 1991).

2.9 Seismic Performance of Steel Structure

Seismic performance of steel structure depends on proper seismic detailing of joints, members, welding etc.

Seismic Welding Issues

In high-seismic applications, the requirements in the building code differ from other loading conditions in that it is assumed that portions of the building’s seismic load resisting system

(SLRS) will undergo controlled inelastic response when subjected to major seismic events. Welds and welded connections that are part of the SLRS connect members that are subject to yield-level stresses and plastic deformations during such events. In order to resist the imposed loads, welded connections must be designed, detailed, fabricated, and inspected to more rigorous standards than are required for statically loaded buildings. The weld metal property requirements are also different (Miller 2006).

High-seismic framing systems generally have the highest demands concentrated at the ends of beams and braces, right near the point of the connections. Thus, connections are often in or near the most severely stressed portions of a structure. Inelastic deformations are not typically expected to be concentrated in the welds themselves, but welds are often near the base metal in which such strains are located. In order for the expected inelastic deformations to occur, the welded connections must be strong enough to resist the applied stresses without fracture, and the base metal must be capable of deforming to accommodate the straining (Miller 2006).

The welded connections in high-seismic applications must be strong, ductile, and fracture resistant. Strength and ductility are primarily addressed through the selection of the welding filler metals and control of the procedures used to deposit the metal. Such criteria are not significantly different than the requirements for low-seismic applications. In high seismic applications, because of the potential consequences of connection fracture, as well as the demands placed on the connections, the welded connections are treated differently with respect to fracture resistance. Three factors determine the ability of a connection to resist brittle fracture: the applied stresses; the presence (or lack) of cracks, notches, and other stress concentrations; and the fracture toughness of the material. The applied stresses in the connection are inherently linked to the configuration of the connection. In general terms, two approaches have been used in seismic design to reduce the applied stresses in the connection: the connection can be strengthened (by the use of reinforcing ribs, gussets, cover plates, etc.), or the demand on the connection can be reduced (such as through the use of reduced beam sections, often called “dog bones”). These factors are not directly weld related but have a direct effect on the localized stresses in the weld and ductility demands on the weld (Miller 2006).

The other two factors (stress concentrations and material fracture toughness) are specifically welding related. The first variable consists of two different issues: cracks and stress concentrations. For connection fracture resistance, welds and heat-affected zones must be free of cracks and crack like discontinuities; that is, planar and near-planar flaws. To avoid cracks, specifications like the AWS D1.8 Structural Welding Code—Seismic Supplement emphasize hydrogen control. The AISC Seismic Provisions call for specific post welding nondestructive testing (NDT) to detect any cracking that might have occurred during or after welding. Lamellar tearing can be similarly detected. Incomplete fusion, some slag inclusions, and planar discontinuities, may have a crack-like effect on fracture resistance. Good welding procedures and welder workmanship limit the production of such discontinuities, and effective NDT is used to detect remaining planar flaws (Miller 2006).

Stress concentrations occur in a variety of forms, including notches and gouges from flame cutting, weld toes, left in-place weld tabs, and weld discontinuities such as undercut, under fill, and porosity. These stress concentrations are generally not planar, but volumetric and, as such, are typically less severe than cracks. However, depending on the exact geometry of the discontinuity, the local stress levels, and the orientation of the stress concentration to the stress field, the effect can range from inconsequential to severe. The AISC Seismic Provisions and the AISC Prequalified Connection Standard, as well as AWS D1.8, prescribe limits for such stress concentrations in the connections of structures subject to seismic loading. Steel backing left in-place in T-joints of moment connections can create a crack-like planar discontinuity that constitutes a major stress concentration.

AISC Seismic Provisions for Structural Steel Buildings

The AISC Seismic Provisions were developed to augment the AISC Specification, adding provisions deemed necessary for high-seismic applications, which require capability to dissipate energy through controlled inelastic deformations in major seismic events. Members and connections in the seismic load resisting system (SLRS), including the welds that join various members, are subject to the special requirements contained in the AISC Seismic Provisions. The AISC Seismic Provisions contain a variety of welding-related requirements.

AWS D1.8 Structural Welding Code—Seismic Supplement

AWS D1.8 contains the additional provisions intended to be applied to joints or members that are designed to resist yield level stresses or strains during design earthquakes. Just as the AISC Seismic Provisions augment the AISC Specification, so AWS D1.8 supplements AWS

D1.1. When AWS D1.8 is specified, all the provisions of AWS D1.1 still apply, unless modified or superseded by AWS D1.8. In AWS D1.8, it is assumed that the structure has been designed in accordance with the AISC Seismic Provisions.

Performance of Moment End-Plate Connections for Seismic Loading

Cyclic loading of moment end-plate connections was first studied by Popov and Tsai (1989). Since that time a number of studies have been conducted worldwide. Two studies that used design procedures are Meng and Murray (1997) and Sumner, et al. (2000). Meng and Murray (1997) conducted a series of tests using the four-bolt extended, un-stiffened connection. The connections were designed using the yield-line and modified Kennedy procedures that include prying force effects in the bolt design. The test specimens were designed such that the connection was stronger than the connected beam. Each specimen was subjected to the Applied Technology Council (ATC-24) protocol loading (ATC 1992). Even though bolt forces decreased from the fully tightened level (in some tests, even to zero) as the testing progressed, failure occurred in the beam for every test. If weld access holes were not used, robust hysteresis loops were obtained. In all the specimens tested with weld access holes, flange fracture at the weld access hole occurred a few cycles into the inelastic regime of the ATC-24 protocol. Subsequent finite element analysis showed that the presence of a weld access hole significantly increases flange strain adjacent to the hole. Meng and Murray recommended that weld access holes not be used in moment end-plate connections.

As part of the SAC Joint Venture, Sumner, et al. (2000) conducted beam-to-column tests using the SAC Protocol (1997). Their test matrix included the four-bolt extended, un-stiffened end-plate connection. For each end-plate geometry, two tests were performed**:** one with the connection design to develop 110 percent of the nominal plastic moment strength of the beam (strong plate connection) the other with the connection designed to develop 80 percent of the plastic moment strength of the beam (weak plate connection). It was found that the four-bolt extended, un-stiffened end-plate connection can be designed and detailed to be suitable for seismic loading. A design procedure, very similar to the procedure contained in AISC Design Guide 16, was then developed. The procedure is found in the Federal Emergency Management Agency (FEMA) Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings (2000).

2.10 Estimating and Costing

Estimating and costing is done by using schedule of item rate prepared by using standard procedure and present market rate of material, labor and other related matters. For RC structural works and other civil and sanitary works, schedule of rate for different items is prepared using present market rate of material, labor and other related matters following PWD item rate analysis procedure. For steel structural works, schedule of rate for different items of steel structure is prepared by analyzing item rates as per present market rate of material, labor and other related matters following guide lines of PWD rate analysis procedure and present practice of different steel structure fabrication company in Bangladesh.

2.11 Materials and Specifications for steel structure

In designing steel and composite structural members, hot-rolled and built-up sections are used. In the shop, using steel plates of different thickness and strength, different sizes of I-sections may be fabricated by cutting and wielding of steel plates as per design requirements. Different sizes of hot-rolled I-sections of different strength are available and may be selected as per design requirements. There are steel plates of different thickness, strength and properties. Also different types of fastener materials i.e. nut bolts, stud anchors, anchor bolts, corrugated steel decking sheets of different strength and properties are available. For joining plates in the shop and field, different wielding process and electrodes of different strength and properties are used. All the materials usually used for fabrication and erection of steel structural members are discussed in brief with necessary specifications.

2.11.1 Structural Steel

Different types of structural steels such as carbon steel, high-strength low-alloy steel, alloy steel etc. are widely used for steel structure as described below in brief with properties.

Carbon Steel

Carbon steels are divided into four categories based on the percentage of carbon**:** low carbon (less than 0.15%); mild carbon (0.15-0.29%); medium carbon (0.3-0.59%); and high carbon (0.60-1.70%). Structural carbon steels are in the mild carbon category; a steel such as A36 has maximum carbon varying from 0.25 to 0.29% depending on thickness. Structural carbon steels exhibit definite yield points. Increased carbon percent raises the yield stress but reduces ductility, making welding more difficult.

High-Strength Low-Alloy Steel

This category includes steels having yield stresses from 40 to 70 ksi, exhibiting well defined yield point. The addition to carbon steels of small amounts of alloy elements such as chromium, columbium, copper, manganese, molybdenum, nickel, phosphorus, vanadium, or zirconium, improves some of the mechanical properties. Where as carbon steel gain their strength by increasing carbon content, the alloy elements create increased strength from a fine rather than course microstructure obtained during cooling of the steel. High- strength low-alloy steels are used in the as-rolled or normalized conditions; i.e., no heat treatment is used. The high-strength low-alloy steels are A242, A441, A572, A588, A606, A607, A618, and A709, Grades 50 and 50W.

Alloy Steels

Low-alloy steels may be quenched and tempered to obtain yield strengths of 80 to 110 ksi. Yield strength is usually defined as the stress at 0.2% offset strain, since these steels do not exhibit a well-defined yield point. These steels are weldable with proper procedures, and ordinarily require no additional heat treatment after they have been welded.

Corrosion Resistance and Weathering Steel

Corrosion resistance may be improved by the addition of copper as an alloy element. However copper-bearing carbon steel is too expensive for general use. High-strength low-alloy steels have several times the corrosion resistance of structural carbon steel, with or without the addition of copper. The high-strength low-alloy steels do not pit as severely as carbon steels and the rust that forms becomes a protective coating to prevent further deterioration. With certain alloy elements the high-strength low-alloy steel will develop an oxide protective coating that is pleasing in appearance and is described as follows: “It is a very dense corrosion –actually a deeply colored brown, red, purple……It has a texture and color which cannot be reproduced artificially—a character only nature can give, as with stone, marble, and granite.” When steels are to be unpainted and left exposed they are called weathering steels. ASTM **A588** is generally used for weathering steel in buildings and **A709** Grades 50W and 100W for weathering steel in bridges. The extra expense resulting from fabrication and erection is offset by the elimination of painting at intervals during the life of structure.

Properties of Steel

Properties of common structural steel used in building and bridge construction is given in Table 2.4 with strength, ASTM designation and use.

Table 2.4 Properties of steels used for buildings and bridges

ASTM designation	Minimum yield stress, F_y (ksi)	Tensile strength, F_u (ksi)	Common use
A36	32	58-80	General structural purposes; bolted and welded, mainly for buildings.
	36	58-80	
A572 Grade 42	42	60	Structural shapes, plates, sheet piling, and bars for bolted and welded building; welded bridges in Grades 42 and 50 only; essentially superseded by A709, Grade 50
Grade 50	**50**	65	
Grade 60	60	75	
Grade 65	65	80	

Fatigue Strength of Structural Steel

The AISC specifications prescribe no fatigue effect for fewer than 20000 cycles, which is approximately two applications a day for 25 years. Since most loadings in buildings are in that category, fatigue is generally not considered. The exceptions are crane-runway girders and structures supporting machinery. Fatigue is always considered in the design of highway bridges, which are expected to have in excess of 100,000 cycles of loading.

2.11.2 Welding

Welding is the process of connecting metal. Welding is widely used for structural steel plate joining process.

Basic Process

Welding is the process of jointing materials (usually metal) by heating them to suitable temperatures such that the materials coalesce into one material. There may or may not be pressure, and there may or may not be fillet material applied. Arc wielding is the general term for many process that use electrical energy in the form of an electric arc to generate the heat necessary for wielding. Shielded Metal Arc Welding (SMAW) and submerged Arc Welding (SAW) are widely used conventional welding process. Other conventional welding processes are Gas Metal Arc Welding (GMAW), Flux Cored Arc Welding (FCAW), Electrogas Welding (EGW), Electroslag Welding (ESW), Stud Welding etc.

Shielded metal arc welding is one of the oldest, simplest, and perhaps most versatile types for welding structural steel. The SMAW process is often referred to as the manual stick electrode process. Heating is accomplished by means of an electric arc between a coated electrode and the materials being jointed. The coated electrode is consumed as the metal is transferred from the electrode to the base material during the welding process. The electrode wire becomes filler material and the coating is converted partly into a shielding gas, partly into slag, and some part is absorbed by the weld metal. The transfer of metal from electrode to the work being welded is induced by molecular attraction and surface tension, without application of pressure. The shielding of the arc prevents atmospheric contamination of the molten metal in the arc stream and in the arc pool. It prevents nitrogen and oxygen from being picked up and forming nitrides and oxides which may cause embrittlement. The electrode material is specified under various American Welding Society specifications. The designation of electrode material such as E60XX or E70XX indicate 60 ksi and 70 ksi, respectively, for tensile strength.

In the SAW process the arc is not visible because it is covered by a blanket of granular, fusible material. The bare metal electrode is consumable in that it is deposited as filler material. The end of the electrode is kept continuously shielded by the molten flux over which is deposited a layer of unfused flux in its granular condition. The granular flux, which is a special feature of this method, is usually laid automatically along the seam ahead of the advancing electrode, and provides a cover that allows the weld to be made without spatter, sparks, or smoke. This flux material protects the weld pool against the atmosphere, serves to clean the weld metal, and modifies the chemical composition of the weld metal. Welds made by the submerged arc process have uniform high quality; exhibiting good ductility, high impact strength, high density, and good corrosion resistance. Mechanical properties of the weld are consistently as good as the base material. The combinations of bare-rod electrodes and granular flux are classified under AWS. They are designed FXXX-EXXX where the first X following the F is the first digit of the tensile strength (i.e.,7 for 70 ksi).

Types of Welds

The weldability of a steel is a measure of the ease of producing a crack-free and sound structural joint. Groove weld, fillet weld, plug weld and slot weld are common types of weld.

The principal use of groove welds is to connect structural members that are aligned in the same plane. Since groove welds are usually intended to transmit the full load of the members they join, the weld should have the same strength as the pieces joined. Such a groove weld is known as a complete joint penetration groove weld. When joints are designed so that groove welds do not extend completely through the thickness of the pieces being joined, such welds are referred to as partial joint penetration groove welds.

Fillet welds owing to their overall economy, ease of fabrication, and adaptability are the most widely used. They generally require less precision in the “fitting up” because of the overlapping of pieces, whereas the groove weld requires careful alignment with specified gap (root opening) between pieces. The fillet weld is particularly advantageous to welding in the field or in realigning members or connections that were fabricated within accepted tolerances but which may not fit as accurately as desired. In addition, the edges of pieces being joined seldom need special preparation such as beveling or squaring since the edge conditions resulting from flame cutting or from shear cutting procedures are generally adequate.

Slot and plug welds may be used exclusively in a connection, or they may be used in combination with fillet welds. Principal use for plug or slot welds is to transmit shear in a lap joint when the size of welds are also useful in preventing overlapping parts from buckling.

Minimum Size of Fillet Welds

Minimum size of fillet welds is shown in Table 2.5. Though designed size of fillet weld may be smaller, the AISC Specifications 2005 specify minimum size of fillet weld which must be followed during welding design.

Table 2.5 Minimum size of fillet welds

Material thickness of thinner part jointed, in.(mm)	Minimum size of fillet weld,[a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)
[a] Leg dimension of fillet welds. Single pass welds must be used.	

Maximum Fillet Weld Size along Edges

Maximum fillet weld size along edges to be designed according to the requirements given below.

- Along edges of material less than ¼ inch (6.4mm) thick, the maximum size is equal to the thickness of the material.
- Along edges of material ¼ inch (6.4 mm) or more in thickness, the maximum size shall be 1/16 inch (1.6 mm) less than the thickness of the material, unless the weld is especially designated on the drawings to be built out to obtain full throat thickness.

Effective Areas of Fillet Welds

Assuming the fillet to have equal legs of nominal size **a**, the effective throat t_e is 0.707a. The effective throat dimensions for fillet welds made by the submerged arc (SAW) process as modified to account for the inherently superior quality of such welds,

- For fillet welds the leg size equal to or less than 3/8 inch (9.5 mm), the effective throat dimension shall be taken as equal to the leg size a.

- For fillet welds larger than 3/8 inch (9.5 mm), the effective throat dimension shall be taken as the theoretical throat dimension plus 0.11 inch (2.8 mm) i.e. 0.707a+0.11 for symmetric welds.

Load and Resistance Factor Design of Welds

Load and Resistance Factor Design of Welds as per 1993 AISC Specification for Steel Buildings is described here in brief.

For groove welds, the design strength per unit length of complete penetration groove welds depends on the type of stress that is applied.

- Tension and compression normal to effective area, and tension and compression parallel to axis of weld: $ØR_{nw}=0.9t_eF_y$ for base material………………………...(2.1)
  $ØR_{nw}=0.9t_eF_{yw}$ for weld metal………………………………………………...(2.2)
- Shear on effective area:
  $ØR_{nw}=0.9t_e(0.6F_y)$ for base metal……………………………………………...(2.3)
  $ØR_{nw}=0.8t_e(0.6F_{EXX})$ for weld metal…………………………………………..(2.4)

For fillet welds, the design strength per unit length of a fillet is based on the shear resistance through the throat of the weld, as follows.

$ØR_{nw}=0.75t_e(0.6F_{EXX})$ for fillet weld……………………………………………..(2.5)

But not greater than the shear rupture strength of the adjacent base metal.

$ØR_{nw}=0.75t(0.6F_u)$ for base metal ………………………………………………(2.6)

Where, t_e = effective throat dimension.

t = thickness of base material along which weld is placed.

F_u = tensile strength of base metal.

F_{EXX} = tensile strength of electrode material.

R_{nw} = the nominal strength per unit length of weld, but not to exceed the nominal strength per unit length of adjacent base material.

Ø = strength reduction factor.

F_y= yield strength of base metal.

F_{yw} = yield strength of weld metal.

2.11.3 Structural Fasteners

Every structure is an assemblage of individual parts or members that must be fastened together, usually at the member ends. Welding is one method and already discussed. The other method is to use fastener, such as rivets or bolts. Here only high strength bolt is discussed. High strength bolts have replaced rivets.

2.11.3.1 High-Strength Bolts

The two basic types of high-strength bolts are designated as ASTM A325 and A490. These bolts are heavy hexagon-head bolts, used with heavy semi-finished hexagon nuts. A325 bolts are of heat-treated medium carbon steel having an approximate yield strength of 81 to 92 ksi depending on diameter. A490 bolts are also heat-treated but are of alloy steel having an approximate yield strength of 115 to 130 ksi depending on diameter. A449 bolts are occasionally used when diameters over $\frac{1}{2}$ inch up to 3 inch are needed, and also for anchor bolts and threaded rods. High strength bolts range in diameter from $\frac{1}{2}$ to $1\frac{1}{2}$ inch (3 inch for A449). The most common diameters used in building construction are $\frac{3}{4}$ inch and $\frac{7}{8}$ inch, whereas the most common sizes in bridge design are $\frac{7}{8}$ inch and 1 inch.

High strength bolts are usually tightened to develop a specified tensile stress in them, which results in a predictable clamping force on the joint. The actual transfer of service loads through a joint is, therefore, due to the friction developed in the pieces being joined. Joints containing high-strength bolts are designed either as slip-critical (formally called friction-type), where high slip resistance at service load is desired; or as bearing-type, where high slip resistance at service load is unnecessary. Tensile strength of the bolt material is 120 ksi for A325 bolts and 150 ksi for A490 bolts.

2.11.3.2 Load and Resistance Factor Design of Fasteners

Load and Resistance Factor Design of Fasteners as per 1993 AISC Specification for Steel Buildings is described here in brief.

Design Shear Strength (No Threads in Shear Plane)

Design shear strength, $ØR_n$=0.75(0.5F_u^b)mA_b……………………………………….(2.7)

Where, Ø = 0.75, the standard Ø value for shear.

R_n= the nominal strength.

F_u^b= tensile strength of the bolt material (120 ksi for A325 bolts; 150 ksi for A490 bolts).

m = the number of shear planes participating (usually 1 for single shear or 2 for double shear).

A_b= gross cross-sectional area across the unthreaded shank of the bolt.

Design Shear Strength (Threads in Shear Plane)

Design shear strength, $ØR_n$=0.75(0.4F_u^b)mA_b………………………………………….(2.8)

Design Tension Strength

Design tension strength, $ØR_n$=0.75(0.75F_u^b)A_b…………………………………………...(2.9)

Design Bearing Strength

(i) Usual conditions based on the deformation limit state. This applies for all holes except long-slotted holes perpendicular to the line of force, where end distance L_e is at least 1.5 times the bolt diameter d, the center-to-center spacing s is at least 3d, and there are two or more bolts in the line of force.

Design bearing strength, $ØR_n$= Ø(2.4dtF_u)…………………………………………….(2.10)

Where, Ø = 0.75

d= nominal diameter of bolt at unthreaded area.

t = thickness of part against which bolt bears.

F_u= tensile strength of connected part against which bolt bears.

L_e = distance along line of force from the edge of the connected part to the center of a standard hole or the center of a short and long-slotted hole perpendicular to the line of force.

R_n= the nominal strength.

(ii) Deformation limit state for long-slotted holes perpendicular to the line of force, where end distance L_e is at least 1.5 times the bolt diameter d, the center-to-center spacing s is at least 3d, and there are two or more bolts in the line of force.

Design bearing strength, $ØR_n = Ø(2dtF_u)$…………………………………………...(2.11)

Where, Ø = 0.75

(iii) Strength limit state for the bolt nearest the edge:

Design bearing strength, $ØR_n = ØL_etF_u$……………………………………...........(2.12)

Where, Ø = 0.75

(iv) Strength limit state when hole elongation exceeding 0.25 inch and hole "ovalization" can be tolerated:

Design bearing strength, $ØR_n = Ø(3dtF_u)$…………………………………………….(2.13)

Where, Ø = 0.75

Minimum Spacing of Bolts in Line of Transmitted Force

Minimum spacing $\geq P/(ØF_ut) + d/2$ ……………………………………………..(2.14)

Where, Ø = 0.75

P= factored load acting on one bolt.

t = thickness of plate material.

F_u= tensile strength of plate material.

d = diameter of the bolt.

The minimum spacing of bolts in a line is preferably 3 bolt diameters and shall not be less than $2\frac{2}{3}$ diameters.

Minimum End Distance in Direction of Transmitted Force

Minimum end distances must be at least 1.5 diameters. When higher bearing strengths are used then the minimum end distance , as follows, $L_e \geq P/(ØF_ut)$………………………(2.15)

Where, Ø = 0.75

P= factored load per bolt.

t = thickness of plate material.

F_u= tensile strength of the plate material.

Maximum Edge Distance

The maximum distance from the center of a bolt to the nearest edge is 12t, where t is the thickness of the connected part, and this edge distance may not exceed 6 inch. The purpose of this requirement is to prevent corrosion resulting from moisture entering the joint. The two contact surfaces of a joint may not be perfectly flat, and the clamping action will be lower when the bolts are far apart (or far from an edge).

Maximum Spacing of Connector

The maximum longitudinal spacing of connectors between elements in continuous contact when the elements consist of a plate and a shape or two plates, is given by,

(i) For painted members or unpainted members not subject to corrosion,

$$S \leq 24t \leq 12 \text{ inch.}$$

(ii) For unpainted members of weathering steel subject to atmospheric corrosion,

$$S \leq 14t \leq 7 \text{ inch.}$$

Where t is the thickness of the thinner element.

Design of Slip Critical Connections

The design of slip-critical connections requires full consideration of the strength limit states. The strength of the fastener in shear, bearing, and direct tension must be investigated. Sufficient strength must be provided to resist factored loads. In addition, the service load that must be transferred by friction without slip must not exceed maximum acceptable value.

Design slip resistance $\text{Ø}R_{slr} = \text{Ø}1.13\mu T_i m$……………………………………………….(2.16)

Where, R_{slr} = nominal slip resistance per bolt at factored loads.

m = number of slip (shear) planes.

T_i = minimum fastener initial tension.

μ= mean slip coefficient, as applicable, or as established by test.

= 0.33 for class A surface condition.

=0.50 for class B surface condition.

=0.40 for class C surface condition.

P= factored load per bolt.

t = thickness of plate material.

F_u= tensile strength of the plate material.

Ø = 1.0 for standard holes.

=0.85 for oversize and short-slotted holes.

=0.70 for long-slotted holes transverse to load.

=0.60 for long-slotted holes parallel to load.

2.11.4 Anchor Rod

The preferred specification for anchor rods is ASTM F1554, with Grade 36 being the most common strength level used. The availability of other grades should be confirmed prior to specification. ASTM F1554 Grade 55 anchor rods are used when there are large tension forces due to moment connections or uplift from overturning. ASTM F1554 Grade 105 is a special high strength rod grade and generally should be used only when it is not possible to develop the required strength using larger Grade 36 or Grade 55 rods. Unless otherwise specified, anchor rods will be supplied with unified coarse (UNC) threads with a Class 2a tolerance, as permitted in ASTM F1554. While ASTM F1554 permits standard hex nuts, all nuts for anchor rods, especially those used in base plates with large oversize holes, should be furnished as heavy hex nuts, preferably ASTM A563 Grade A or DH for Grade 105. ASTM F1554 anchor rods are required to be color coded to allow easy identification in the field. The color codes are as follows: Grade 36-Blue, Grade 55-Yellow, Grade 105-Red. In practice, Grade 36 is considered the default grade and often is not color coded. Table 2.6 shows tensile strength of different ASTM designated anchor rod materials.

Table 2.6 Anchor rod materials

Materials ASTM	F1554			A449			A36	A307	A354 Gr BD	
	Gr36	Gr55	Gr105							
Tensile strength, F_u (ksi)	58	75	125	120	105	90	58	58	150	140

The ASTM specification allows F1554 anchor rods to be supplied either straight (threaded with nut for anchorage), bent or headed. Rods up to approximately 1 in. in diameter are sometimes supplied with heads hot forged similar to a structural bolt. Thereafter, it is more common that the rods will be threaded and nutted. Hooked type anchor rods have been extensively used in the past. However, hooked rods have a very limited pullout strength compared with that of headed rods or threaded rods with a nut for anchorage. Therefore, current recommended practice is to use headed rods or threaded rods with a nut for anchorage.

The addition of plate washers or other similar devices does not increase the pullout strength of the anchor rod and can create construction problems by interfering with reinforcing steel placement or concrete consolidation under the plate. Thus, it is recommended that the anchorage device be limited to either a heavy hex nut or a head on the rod. As an exception, the addition of plate washers may be of use when high-strength anchor rods are used or when concrete blow out could occur. In these cases, calculations should be made to determine if an increase in the bearing area is necessary. Additionally, it should be confirmed that the plate size specified will work with the reinforcing steel and concrete placement requirements. ASTM F1554 Grade 55 anchor rods can be ordered with a supplementary requirement, which limits the carbon equivalent content to a maximum of 45%, to provide weldability when needed. Adding this supplement is helpful should welding become required for fixes in the field. Grade36 is typically weldable without supplement (Fisher and Kloiber 2006).

There are also two supplemental provisions available for Grades 55 and 105 regarding Charpy V-Notch (CVN) toughness. These provide for CVN testing of 15 ft-lbs at either 40 °F or at -20 °F. Note, however, that anchor rods typically have sufficient fracture toughness without these supplemental specifications. Additional fracture toughness is expensive and generally does not make much difference in the time to failure for anchor rods subjected to fatigue loading. Although fracture toughness may correspond to a greater crack length at the time of failure (because cracks grow at an exponential rate) 95% of the fatigue life of the anchor rod is consumed when the crack size is less than a few millimeters. This is also the reason it is not cost effective to perform ultrasonic testing or other nondestructive tests on anchor rods to look for fatigue cracks. There is only a small window between the time cracks are large enough to detect and small enough to not cause fracture. Thus, it generally is more cost effective to design additional redundancy into the anchor rods rather than specifying supplemental CVN properties.

Galvanized anchor rods are often used when the column base-plate assembly is exposed and subject to corrosion. Either the hot-dip galvanizing process (ASTM 153) or the mechanical galvanizing process (ASTM B695) is allowed in ASTM F1554; however, all threaded components of the fastener assembly must be galvanized by the same process. Mixing of rods galvanized by one process and nuts by another may result in an unworkable assembly. It is recommended that galvanized anchor rods and nuts be purchased from the same supplier and shipped preassembled. Because this is not an ASTM requirement, this should be specified on the contract documents. Note also that galvanizing increases friction between the nut and the rod and even though the nuts are over tapped, special lubrication may be required. ASTM A449, A36 and A307 specifications are listed in Table for comparison purposes, because some suppliers are more familiar with these specifications. Note that ASTM F1554 grades match up closely with many aspects of these older material specifications. Note also that these older material specifications contain almost none of the anchor rod specific requirements found in ASTM F1554.

2.11.5 Shear Connector

The horizontal shear that develops between the concrete and the steel beam during loading must be resisted so that the slip will be restrained. A fully composite section will have no slip at the concrete-steel interface. Although some bond may develop between the steel and concrete, it is not sufficiently predictable to provide the required interface shear strength. Neither can friction between the concrete slab and the steel beam develop such strength. Instead, mechanical shear connectors are required, except for the totally concrete-encased steel beam. The only connectors specifically provided for in the AISC specifications are stud connectors and channel connectors. Currently, nearly all shear connectors are headed studs. Ideally, to obtain a fully composite section, the shear connectors should be stiff enough to provide the complete interaction (i.e., no slip at the interface). Nominal strength of stud shear connector is given in Table 2.7.

Specifications of stud shear connectors are given below as per AISC LRFD 2010. These specifications are used for design.

- ASTM A108 stud anchor's tensile strength, F_u=65 ksi.
- Use steel headed stud anchors ¾ inch or less in diameter.
- Concrete strength to be, 3ksi $\leq f_c' \leq$10 ksi.
- Steel headed stud anchors, after installation, shall extend not less than 1.5 inch above the top of the steel deck.
- Minimum stud anchor length to be equal to (rib height+1.5").
- Minimum length of stud anchors = 4d_{sa} where, d_{sa}= stud anchor diameter.

- There shall be at least ½ inch of specified concrete cover above the top of the headed stud anchor.
- Burn off length of stud anchor may be used 3/8".
- Steel headed stud anchor diameter to be smaller than or equal to 2.5 times flange thickness of beam.
- Slab thickness above steel deck must be ≥ 2 inch.
- Maximum spacing of stud anchor=$8t_{s,eff}$.
- Minimum spacing of stud anchor=$6d_{sa}$.
- Minimum transverse spacing between anchor pairs=$4d_{sa}$.
- Minimum distance of anchor to slab free edge along shear force=8 inch.

Table 2.7 Nominal strength Q_n (ksi) for stud shear connector

Connector	Concrete strength f_c' (ksi)		
	3	3.5	4
½” dia. x 2” headed stud	9.4	10.5	11.6
5/8” dia. x 2.5” headed stud	14.6	16.4	18.1
3/4” dia. x 3” headed stud	21	23.6	26.1
7/8” dia. x 3.5” headed stud	28.6	32.1	35.5

2.11.6 Formed Steel Deck

Floor and roof slabs incorporating cold-formed steel deck panels, which serve both as form and reinforcement for the concrete placed over them, are widely used in buildings where the main framing is either of steel or composite construction. There are many manufacturers of the steel deck used for composite slabs. Most have developed their own cross-section shapes and details. The steel sheet from which the panels are made ranges in thickness from about 0.024 to 0.060 inch. Such composite slabs have a number of advantages.

(i) The steel deck, easily and quickly laid on the steel floor beams, serves as a working platform to support construction activity and to carry the freshly poured concrete. This eliminates the need for temporary false work and forms.

(ii) The steel deck, with proper attention to details, can serve as the main tensile reinforcement for the slab.

(iii) If parts or all of the deck panels are formed into closed cells, these cells can serve as ducts for electric and communication cables, or for heating and air conditioning ducts.

Specifications of formed steel deck are given below as per AISC LRFD 2010. These specifications are used for design.

- Rib height of steel deck must not be greater than 3 inch.
- Rib width of steel decking must be greater than or equal to 2 inch.
- Steel deck shall be anchored to all supporting members at a spacing not to exceed 18 inch.

2.11.7 Hot-Rolled, Built-Up and Cold-Formed Sections

The common hot-rolled shapes are the angle, the tube, the channel, and the I. The I is available in two classifications. The most widely used is W shape. The other, once called the American Standard Beam, is called the S shapes. Miscellaneous column and beam shapes used for lightweight construction are rolled by a few mills. Rolled section properties which are to be used in structural design calculations are presented in the AISC Manuals. Structural shapes are identified by a letter designator which indicates the particular cross section. Typical indicators are:

W =Wide flange beam.

M =Miscellaneous beam.

S =American standard beam.

The letter designators are followed by numbers which identify the particular section, for example, W18 × 50. The first numeral indicates the depth of the section and the second its weight in pounds per foot.

Wide-flange shapes can be produced by passing an assembly of two flange plates and a web plate through sub-merged arc welding machines which simultaneously weld both flanges to one side of the web. The section is turned over to weld the flanges to the other side.

Cold-formed steel shapes are formed in rolls or brakes from sheet or strip steel. Because of the great variety which can be produced, shapes of this type, unlike hot-rolled shapes, have not been standardized. Although a number of fabricators have developed their lines of members, the designer may device special shapes for particular jobs. While shapes up to thickness of ½ and even ¾ inch can be formed, cold-formed steel construction is usually restricted to thickness ranging from 0.012 inch to 0.224 inch.

2.11.8 Theory of Design of Steel Structure

Different components of steel structure such as column, beam, plate girder, hybrid girder, composite beam, composite girder etc. may be designed as per AISC Specifications. Steel structural members may be designed as non-composite or composite using LRFD method as per AISC Specification 2010. Welding, connections, anchor bolts, base plates, nut-bolts, stud anchors etc. may also be designed as per AISC Specification 2010. The design steps and procedures of different steel and composite structural members, base plate with anchor bolts, end plate rigid connection, simple shear connection, continuous connections etc. are stated in Appendix-A in brief.

2.12 Concluding Remarks

To perform the intended research work, design of RC and steel framed six storied building with four types of floor system is required. The structural form, floor system, load considerations, load combinations, serviceability criteria, foundation system, connection system, materials and specifications, seismic performance, estimating and costing procedure discussed in this chapter directly or indirectly helped the research work. This chapter will be helpful in making a full understanding in brief about different considerations of multistory building design.

Chapter 3
METHODOLOGY

3.1 Introduction

The primary objective of this chapter is to perform analysis and design of a six storied industrial building as RC and steel structure with various floor systems. Finally, comparison of construction cost and structural behavior of the building structures are required to evaluate better structural system.

To achieve this objective, complete architectural design of a six story garments factory building has been prepared regarding the present and future context of Bangladesh. Following the architectural plan, RC structural systems with beam-slab and flat plate floor have been formed. Again following same plan, steel structural systems with non-composite and composite floor have been formed. Then structural modeling and analysis have been performed by STADD.Pro for the selected four types of structural system. Loads are assigned as per BNBC 1993. From analytical results, RC structural members are designed following ACI Building Code 2008. Steel structural members, joints etc. are designed as non-composite and composite following AISC LRFD Specification for Structural Steel Buildings 2010. Complete construction cost including foundation has been prepared for all the four types of structural system. Other information, related to structural behavior, is obtained.

Here, illustration of architectural planning, formation of structural systems, load calculations, structural modeling, analysis, design, estimating, costing and observation of structural behavior (lateral drift, top deflection, vertical deflection, base shear etc.) have been performed for the intended research program.

3.2 Architectural Design

Complete architectural design of a six storied industrial building is prepared as shown in Figure 3.1. The typical floor plan of this building is functionally solved as per present demand and future trend of export oriented garments industry in Bangladesh. The typical floors comprise of 6 nos. stair cases, 4 nos. lift cores, 4 nos. female toilet blocks, 4 nos. male toilet blocks, 2 nos. lobbies, 4 nos. office block, 4 nos. warehouse block and a central large production area. The exterior and interior walls are 125 mm thick brick wall. Lift core walls are 250 mm thick brick/RC wall. Sufficient windows, doors and collapsible gates are shown. 4 nos. over head water tanks are considered with total capacity of 182000 liters. Roof top stair blocks and lift machine rooms are considered.

The length of the building is 92 meter and width is 56.45 meter. Total area per floor is 4747 square meter. Typical floor height is 3.35 meter. In the prepared architectural plan, column spacing is mainly 7.62 meter at both directions. At staircases, the column spacing is 5 meter in one direction. Within lift cores, different column spacing is used to accommodate with the functional plan.

3.3 Structural Form with Floor System

Following architectural design of the building, RC structural system is formed with beam-slab and flat plate floor. Again for the same building, steel structural system is formed with non-composite and composite floor.

3.3.1 RC Structural Forms with Floor Systems

Following the architectural plan of the six storied building, RC structural system is formed with beam supported slab and flat plate floor as shown in Figure 3.2.

Structural system is considered as intermediate moment resisting rigid frame with shear walls at lift cores as shown in Figure 3.3. Floor slab is assumed as rigid in plane which acts as diaphragm to transfer lateral load horizontally to shear walls. All columns are interconnected by grade beams at finished ground level shown in Figure 3.4. Foundations are initially assumed as shallow foundation.

Figure 3.1 Typical architectural plan of the selected building

(a) RC beam-slab floor system

(b) RC flat plate floor system

Figure 3.2 Floor systems for RC structure

Figure 3.3 Rigid frame for RC structure

Figure 3.4 Grade beam layout for RC and steel structure

(a) Without steel deck

(b) With steel deck

Figure 3.5 Steel beam topping RC slab

3.3.2 Steel Structural Forms with Floor Systems

According to same architectural plan of Figure 3.1, steel structural system is formed with non-composite floor (NCF) and composite floor (CF). In case of steel structure with CF, RC slab with or without steel deck is connected to supporting steel girder and beam by sufficient shear connectors. Shear connectors make the beam composite by resisting the horizontal shear which develops during bending. But for steel structure with NCF, minimum numbers of shear connectors are used and composite action is neglected. For both NCF and CF systems, columns are same i.e. steel I-sections are used.

For sub-structure, initially shallow foundations are considered with RC pedestals which are interconnected by grade beams at finished ground level. Super structure columns, floor beams, girders etc. are built-up steel I-sections. RC slab with or without cold formed steel deck is supported on steel framed floor system. This RC floor slab is connected with supporting steel beams or girders with the help of mechanical shear connectors as shown in Figure 3.5.

Structural form is taken as eccentrically braced semi-rigid steel frame as shown in Figure 3.7. The floor system is taken as one way RC slab supported on two way steel beam system with or without steel deck as shown in Figure 3.6. Floor slab is assumed as rigid in plane and acts as diaphragm to transfer lateral load horizontally to braced panel. Connections of girders with columns and connections of secondary beams with girders are considered as partially restrained (semi-rigid) connection with appropriate ductility. Bracing ends are pin joints. Column joints are fully restrained. Girders and beams are capable to reach plastic strength collapse mechanism where plastic hinge rotation is necessary. Semi-rigid connections require a dependable and known moment capacity.

Figure 3.6 Floor system of steel framing

Figure 3.7 Sami-rigid frames with and without bracing for steel structure

3.4 Design Loads

Both gravity loads (dead load and live load) and lateral loads (wind load and earthquake load) are considered to design the selected building for four types of structural system. Design loads are considered and calculated following BNBC 1993 and given in details in Appendix B.

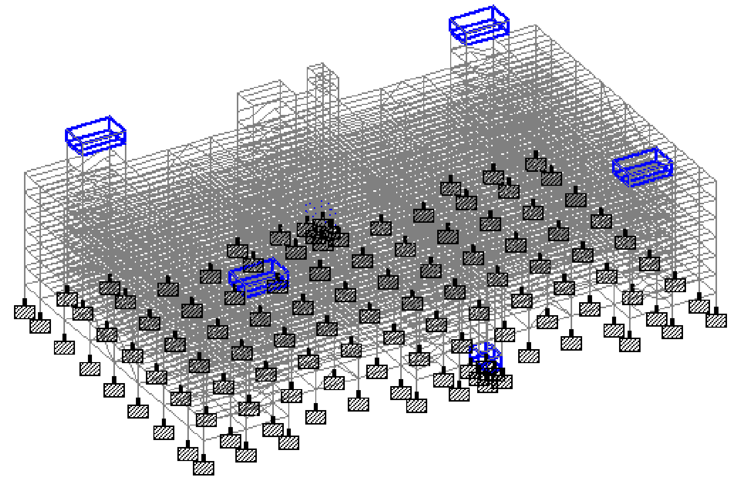
3.5 Structural Modeling and Analysis

This section deals with structural modeling, assigning member properties, assigning basic loads, generation of load combinations and structural analysis of the selected four types of structure for the intended research work.

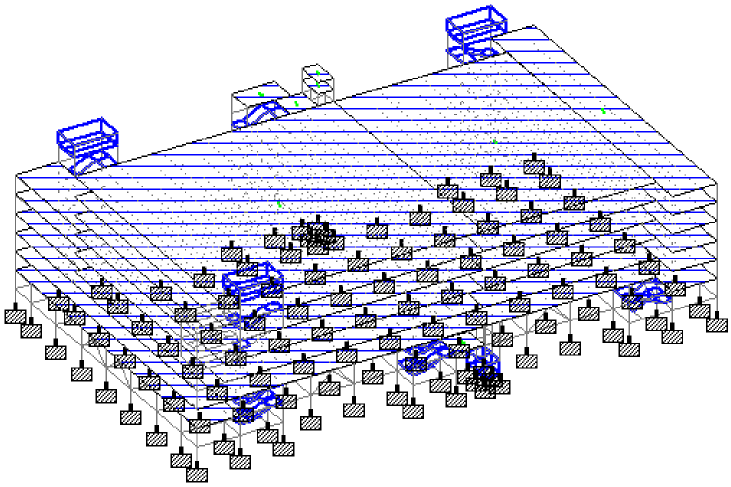
3.5.1 Generation of Model

Following architectural design and selected four types of structural system from section 3.2 and 3.3; three dimensional structural models are generated as RC beam-slab, RC flat plate, steel NCF and steel CF system by STAAD.Pro.

Model for steel (NCF and CF) structure comprises of RC pedestal and grade beam for sub-structure with fixed support. Super structure comprises of steel column, girder and secondary beam with proper orientation. Diagonal bracings are generated for lateral load resisting system. Stair case, lift core, water tank, lift machine room etc. are also generated as realistic as possible. RC slab on steel deck, connected with steel girder and beam by stud anchor, is generated with appropriate properties. Structural form is generated as moment resisting semi-rigid frame with eccentric braced panel as shown in Figure 3.8. Connections of girders with columns and connections of secondary beams with girders are assigned as partially restrained (semi-rigid). Bracing ends are hinge joint. Column joints are fully restrained. Girders and beams are capable to reach plastic strength collapse mechanism where plastic hinge rotation is necessary. Under-ground lift core is generated by surface elements.



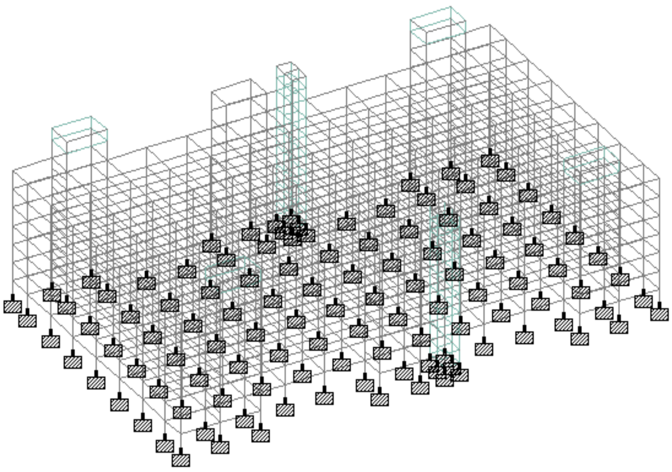
(a) 3D model of steel structure (NCF system)



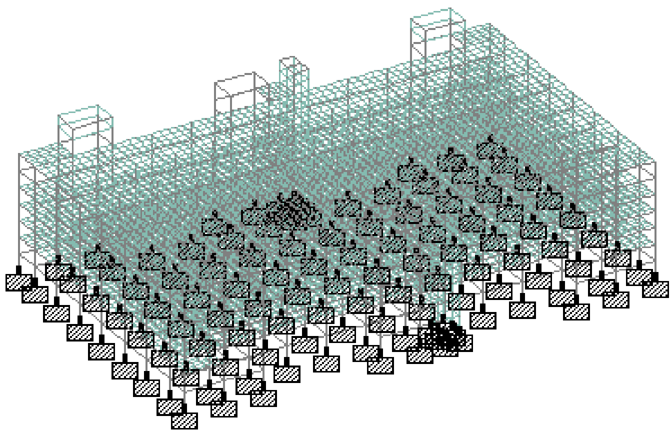
(b) 3D model of steel structure (CF system)

Figure 3.8 STAAD.Pro models for steel structures

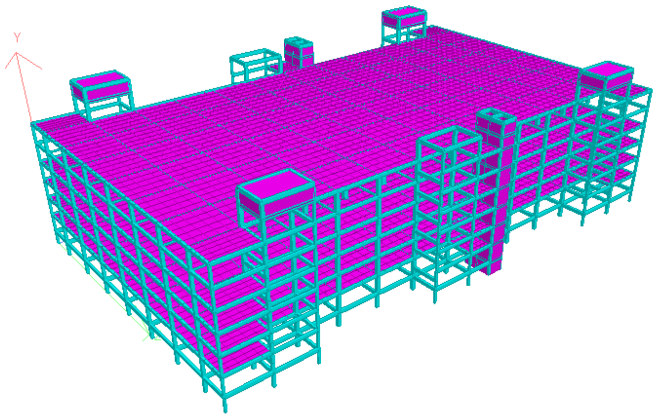
Model for RC structure with beam-slab floor system comprises of column, grade beam, floor beam, lift core shear wall, stair case, water tank, lift machine room etc. with fixed support and rigid connections. Model for RC structure with flat plate floor system comprises of column, grade beam, flat plate, edge beam, lift core shear wall, stair case, water tank, lift machine room etc. with fixed support and rigid connections. Flat plate is generated by plate elements. Structural form is generated as moment resisting rigid frame with shear walls as shown in Figure 3.9.



(a) 3D model of RC beam-slab system



3D model



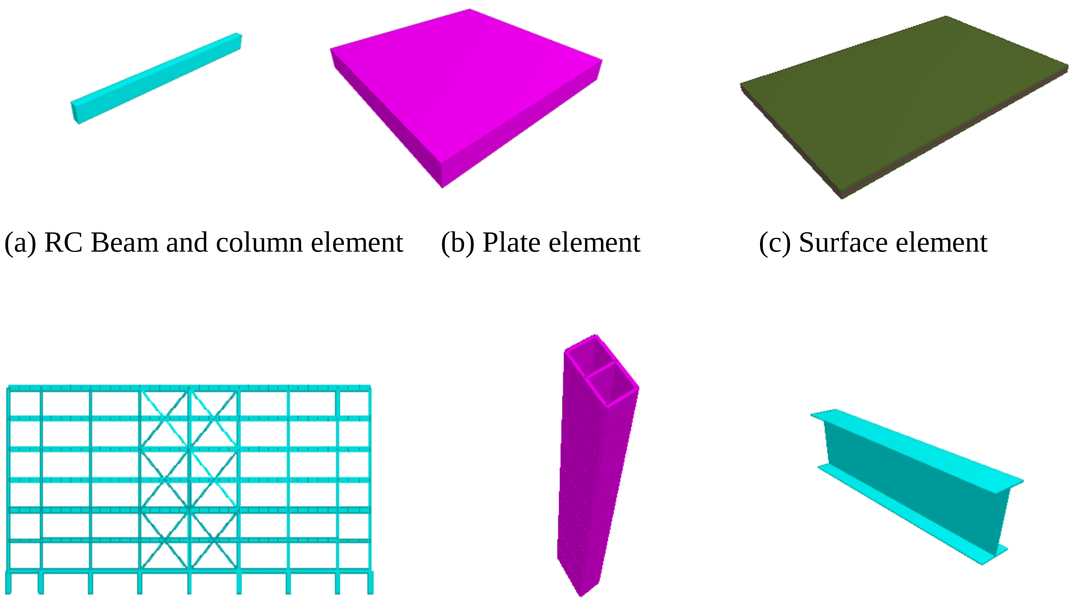
Rendered 3D model

(b) 3D model of RC flat plate system

Figure 3.9 STAAD.Pro models for RC structures

3.5.2 Assigning Member Properties

Member properties are primarily assigned based on preliminary analysis and design for beam elements, surface and plate elements shown in Figure 3.10. Finally member properties are corrected as per final design and checked by STAAD.Pro whether the final design is correct or not.



(a) RC Beam and column element (b) Plate element (c) Surface element

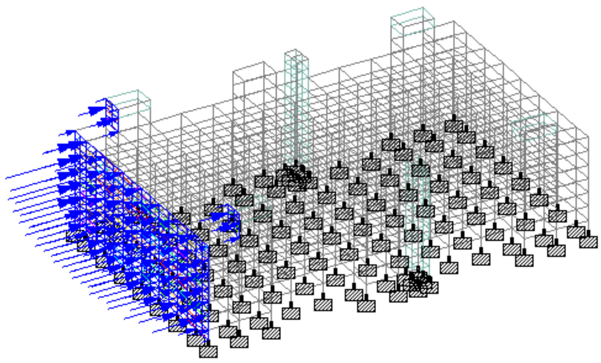
(d) Steel bracing-panel (e) RC lift core shear wall (f) Steel column and beam element

Figure 3.10 Some elements from STAAD.Pro models

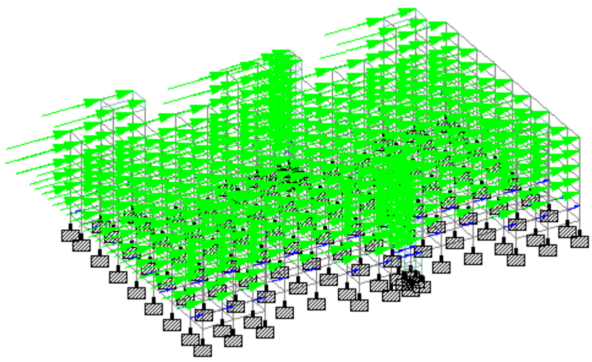
3.5.3 Assigning Basic Loads

Following calculated loads from Appendix B, both gravity loads and lateral loads are assigned in all the four models generated above. Two types of basic wind load is first generated as type-1 and type-2 by taking design wind pressure at different height calculated in section B.1.2.1. Type-1 wind load is perpendicular to length and type-2 wind load is perpendicular to width of building. Intensity of type-1 is greater than type-2. Wind loads are generated as surface load. Earthquake loads are generated as nodal loads taking the calculated point loads at different heights in section B.1.2.2 for earthquake load considerations. Nodal loads are assigned at column beam connections at story level. Live loads and dead loads are assigned according to available data of section B.1.1. Basic loads assigned are listed below. Some basic loads assigned in STAAD.Pro models are shown in Figure 3.11.

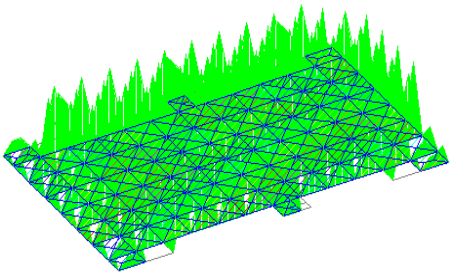
1. D=dead loads
2. L=live loads
3. WX+ = wind loads towards X direction (along building length).
4. WX- = wind loads opposite to X direction.
5. WZ+ = wind loads towards Z direction (along building width).
6. WZ- = wind loads opposite to Z direction.
7. EX+ = earthquake loads towards X direction.
8. EX- = earthquake loads opposite to X direction.
9. EZ+ = earthquake loads towards Z direction.
10. EZ- = earthquake loads opposite to Z direction.



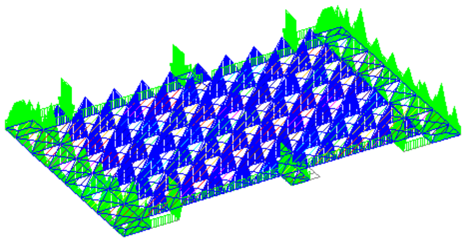
(a) Wind load WX+



(b) Earthquake load EX+



(c) Live load at typical floor



(d) Dead load at typical floor

Figure 3.11 Some basic loads from STAAD.Pro models

3.5.4 Generation of Load Combinations

Load combinations are generated using assigned basic loads. BNBC 1993 is followed for ASD load combinations of steel and RC structure. AISC LRFD Specification 1993 is followed for LRFD load combinations of steel structure. ACI Building code 2008 is followed for USD load combinations of RC structure. Generated load combinations are given in Table 3.13.

Table 3.1 Generated load combinations in STAAD.Pro models

For steel structure (LRFD method)	For RC structure (USD method)	For steel and RC structure (ASD method)
1.4D	1.4D	D
1.2D + 1.6L	1.2D + 1.6L	D + L
1.2D+0.5L+1.3WX+	1.2D+L+1.6WX+	0.75(D+L+WX+)
1.2D+0.5L+1.3WX-	1.2D+L+1.6WX-	0.75(D+L+WX-)
1.2D+0.5L+1.3WZ+	1.2D+L+1.6WZ+	0.75(D+L+WZ+)
1.2D+0.5L+1.3WZ-	1.2D+L+1.6WZ-	0.75(D+L+WZ-)
1.2D+0.5L+EX+	1.2D+L+EX+	0.75(D+L+EX+)
1.2D+0.5L+EX-	1.2D+L+EX-	0.75(D+L+EX-)
1.2D+0.5L+EZ+	1.2D+L+EZ+	0.75(D+L+EZ+)
1.2D+0.5L+EZ-	1.2D+L+EZ-	0.75(D+L+EZ-)
0.9D+1.3WX+	0.9D+1.6WX+	0.75(D+WX+)
0.9D+1.3WX-	0.9D+1.6WX-	0.75(D+WX-)
0.9D+1.3WZ+	0.9D+1.6WZ+	0.75(D+WZ+)
0.9D+1.3WZ-	0.9D+1.6WZ-	0.75(D+WZ-)
0.9D+EX+	0.9D+EX+	0.75(D+EX+)
0.9D+EX-	0.9D+EX-	0.75(D+EX-)
0.9D+EZ+	0.9D+EZ+	0.75(D+EZ+)
0.9D+EZ-	0.9D+EZ-	0.75(D+EZ-)
		0.75(0.9D+WX+)
		0.75(0.9D+WX-)
		0.75(0.9D+WZ+)
		0.75(0.9D+WZ-)
		0.75(0.9D+EX+)
		0.75(0.9D+EX-)
		0.75(0.9D+EZ+)
		0.75(0.9D+EZ-)

3.5.5 Structural Analysis

After completion of generation of load combinations; the structural models, member properties, basic loads and load combinations are checked thoroughly. After that, static analysis is performed and analysis results are preserved for structural design.

3.6 Design of Structural Components

Using structural analysis results, all the members of the four types of building structure are designed using standard codes and methods.

3.6.1 Design of Steel Structure

Steel structural members are designed as non-composite and composite following AISC LRFD Specification for Structural Steel Buildings. Welding, connections, anchor bolts, base plates, nut-bolts, shear connectors etc. are also designed following AISC LRFD Specification for Structural Steel Buildings. Design procedures of steel structural components are illustrated in Appendix A. Following specifications of materials are used in designing steel members and joints**:**

- Built-up section used.
- ASTM A572 grade 50 steel plate with F_y =345MPa (50 ksi) and F_u = 447 MPa (65 ksi) is used.
- Bearing type connection considered.
- ASTM A325 bolts with F_u= 825 MPa (120 ksi) and F_t = 619 MPa (90 ksi) used.
- Anchor rod tensile strength F_u=399 MPa (58 ksi) and yield strength F_y= 248 MPa (36 ksi).
- Concrete strength, f_c**'** = 21 MPa (3ksi) for slab and 25 MPa (3.5 ksi) for pedestal.
- For welding, E70XX electrode used with F_{EXX}= 481 MPa (70 ksi) for SMAW welding.
- For steel decking, rib height h_r= 50 mm, rib width W_r = 150 mm, stud anchor above steel deck top surface = 38 mm is considered.
- ASTM A108 stud anchor with F_u= 447 MPa (65 ksi) is considered.

3.6.1.1 Design of Steel members

Steel secondary beams and girders are designed as non-composite and composite sections. Steel columns and bracings are designed as non-composite sections. Design results of steel members are given in Table 3.2, 3.3, 3.4, 3.5 and 3.6. Necessary drawings are given in Appendix-D.

Table 3.2 Steel secondary beam sections

Secondary beam notations	Ultimate negative moment (kN-m)	Ultimate positive moment (kN-m)	Ultimate shear (kN)	Designed non-composite sections	Pre-com. ultimate neg. moment (kN-m)	Pre-com. ultimate pos. moment (kN-m)	Com. ultimate neg. moment (kN-m)	Com. ultimate pos. moment (kN-m)	Designed composite sections
SB_1	82	84	80	100(6) 315(4)-S19*	26	35	54	121	90(5) 250(4)-S30
SB_2	91	123	93	100(6) 385(5)-S19	26	35	62	138	90(5) 290(4)-S32
SB_3	80	92	76	100(6) 335(4)-S19	26	35	54	106	Same as SB_1
SB_4	0	141	80	100(6) 420(5)-S19	0	62	0	141	90(5) 300(4)-S22
SB_5	45	30	49	100(6) 260(4)-S12	4	7	26	60	90(5) 160(4)-S26
GSB_1	81	85	80	Same as SB_1	23	25	54	115	Same as SB_1
GSB_2	52	43	45	140(6) 200(4)-S19	N/A	N/A	N/A	N/A	N/A
GSB_3	48	57	76	Same as SB_5	10	20	26	69	Same as SB_5
GSB_4	65	62	58	100(5) 300(4)-S19	23	24	41	88	90(5) 200(4)-S28
LB	104	79	116	125(6) 375(4)-S7	7	7	7	7	Same as SB_1 where applicable

Note**:** 16 mm dia. stud anchors are considered for all secondary beams.

*100(6)315(4)-S19 = flange width (flange thickness) total depth (web thickness)-stud nos. All dimensions of steel I-sections are in millimeter.

Table 3.3 Steel girder sections

Girder notations	Ultimate negative moment (kN-m)	Ultimate positive moment (kN-m)	Ultimate shear (kN)	Designed non-composite sections	Pre-com. ultimate neg. moment (kN-m)	Pre-comp. ultimate pos. moment (kN-m)	Com. ultimate negative moment (kN-m)	Composite ultimate pos. moment (kN-m)	Designed composite sections
G_1	407	416	365	175(10) 635(5)-S19	127	127	350	456	145(8) 510(5.5)-S46
G_2	213	218	200	145(8) 500(5)-S19	54	54	160	271	125(8) 400(5)-S34
G_3	172	174	160	140(7) 450(5)-S19	54	54	136	231	125(7) 350(5)-S31
G_4	358	357	320	150(10) 610(5)-S19	115	115	308	407	140(8) 480(5.5)-S44
G_5	132	89	111	125(8) 375(5)-S13	26	27	88	132	120(6) 250(5)-S24
G_6	282	117	218	150(10) 510(5)-S13	54	27	136	206	125(6) 350(5)-S30
G_7	209	138	222	145(8) 500(5)-S13	46	46	136	206	Same as G_6

Note: Transverse stiffener size for girders= 60mm (5mm) and stud anchor dia.=20 mm.
Transverse stiffener spacing for G_1= 300mm,600mm,900mm.
Transverse stiffener spacing for G_2, G_6 and G_7= 600mm,900mm.
Transverse stiffener spacing for G_3= 900mm.
Transverse stiffener spacing for G_4= 350mm,600mm,900mm.
Transverse stiffener spacing for G_5= not required.

Table 3.4 Steel secondary beam sections over roof level

Secondary beam notations	Ultimate negative moment (kN-m)	Ultimate positive moment (kN-m)	Ultimate shear (kN)	Designed non-composite sections	Pre-composite ultimate negative moment(kN-m)	Pre-composite ultimate positive moment (kN-m)	Com. ultimate neg. moment (kN-m)	Composite ultimate positive moment (kN-m)	Designed composite sections
LB	39	39	36	75(5) 225(4)-S7	7	7	39	39	75(5) 150(4)-S10
SRB	0	94	40	100(6) 350(4)-S19	0	52	0	94	90(5) 225(4)-S26
WTB	0	61	36	100(6) 250(4)-S19	0	52	0	61	75(5) 150(4)-S26

Note**:** 16 mm dia. stud anchors are considered for all secondary beams.

Table 3.5 Diagonal bracing sections

Bracing notations	Ultimate axial load (kN)	Designed steel section
DB1	356	200(6)200(4)
DB2	240	175(6)175(4)
DB3	227	175(6)175(4)

Table 3.6 Designed steel column sections

Column notations	Ground floor sections	1st floor sections	2nd floor sections	3rd floor sections	4th and 5th floor sections
C_1	330(20) 350(8)	300(18) 350(8)	275(16) 350(6)	275(12) 350(5)	225(10) 350(5)
C_2	350(20) 350(8)	320(18) 350(8)	300(16) 350(6)	275(12) 350(6)	225(10) 350(5)
C_3	330(18) 350(8)	300(16) 350(8)	275(16) 350(6)	250(12) 350(5)	225(10) 350(5)
C_4	250(12) 350(6)	250(10) 350(5)	225(10) 350(5)	225(8) 350(5)	225(8) 350(5)
C_5	275(14) 350(6)	260(12) 350(5)	250(10) 350(5)	225(10) 350(5)	225(8) 350(5)
C_6	300(18) 350(8)	280(18) 350(7)	275(16) 350(6)	250(15) 350(5)	225(13) 350(5)
C_7	225(10) 350(5)	225(10) 350(5)	225(10) 350(5)	225(10) 350(5)	225(10) 350(5)

3.6.1.2 Design of Connections

Column base plates, girder end plate connections, secondary beam simple shear connections, secondary beam continuous connections, column joints, moment connections of beam with column web, bracing end hinge connections are designed following design procedure of Appendix A.

Base Plates and Anchor Bolts

Base plates and anchor bolts are designed following AISC LRFD Specification and given in Table 3.7. Connection of steel column and RC pedestal by base plate and anchor bolts are moment connection.

Table 3.7 Base plates and anchor bolts schedule

Base plate notations	Base plate size, (mm × mm)	Base plate thickness (mm)	Pedestal size, (mm × mm)	Anchor bolt nos. and dia.(mm)	Anchor bolt length (mm)
BP_1	600 ×510	40	700 × 625	4-20	305
BP_2	600 ×540	40	700 × 650	4-20	305
BP_3	560× 490	35	650 ×600	4-20	305
BP_4	500 ×350	22	600 ×450	4-20	305
BP_5	500 × 355	24	600 ×450	4-20	305
BP_6	550 ×450	33	650×550	4-20	305
BP_7	500 ×350	22	600 ×450	4-20	305

Girder End Plate Connections

Girder joint with column flange is designed as 4E extended end plate moment connections following AISC LRFD Specification. A typical detailing of girder end plate is shown in Figure 3.12. Summary of girder end plate connections are given in Table 3.8 and 3.9.

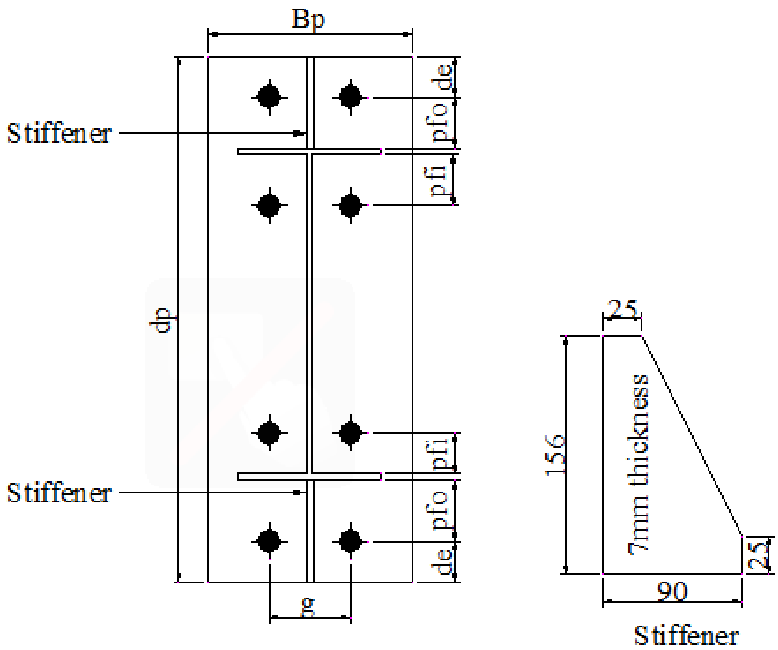


Figure 3.12 Typical end plate

Table 3.8 Girder end plate joint schedule (NCF system)

End plate notations	End plate size = width × depth ×thickness (mm × mm × mm)	Total bolt nos. and dia.(mm)	Column stiffener thickness (mm)	Column flange min^{m} thickness (mm)
EP_1	200 ×815×17	8-22	10	13
EP_2	200×680 ×16	8-20	8	13
EP_3	200×630×16	8-20	7	13
EP_4	200×790×17	8-22	10	13
EP_5	200×555×16	8-20	8	13
EP_6	200×690×17	8-22	10	13
EP_7	200×680×16	8-20	8	13

Note**:** d_e=40 mm, P_{fo}=50 mm, P_{fi}=50 mm, g=140 mm, stiffener size=90mm ×156 mm×7mm.

Table 3.9 Girder end plate joint schedule (CF system)

End plate notations	End plate size = width × depth ×thickness (mm × mm × mm)	Total bolt nos. and dia.(mm)	Column stiffener thickness (mm)	Column flange min^{m} thickness (mm)
EP_1	200 ×690×17	8-22	8	13
EP_2	200×580 ×16	8-20	8	13
EP_3	200×530×16	8-20	7	13
EP_4	200×660×17	8-22	8	13
EP_5	200×430×16	8-20	6	13
EP_6	200×530×17	8-22	6	13
EP_7	200×530×16	8-20	6	13

Note:d_e=40mm, P_{fo}=50mm, P_{fi}=50mm, g =140mm, stiffener size=90mm ×156 mm ×7 mm.

Simple Shear Connections

This type of connection is only designed for discontinuous end of both composite and non-composite secondary beam for all floors shown in Figure 3.13 details.

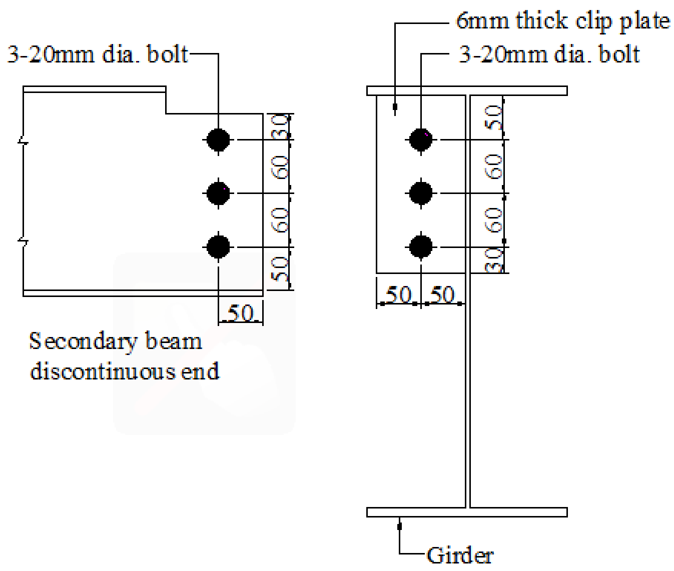


Figure 3.13 Simple shear connection

Continuous Connections

This type of connection is only designed for continuous end of both composite and non-composite secondary beam for all floors shown in Figure 3.14 details. This type of connection only adds cover plate arrangement with simple shear connection of Figure 3.13.

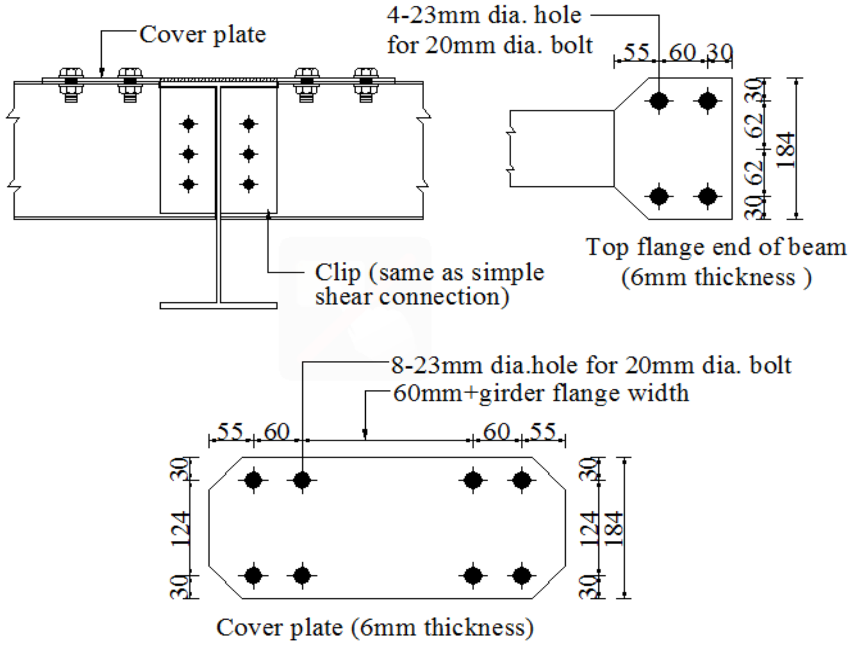


Figure 3.14 Continuous connection

Column Joints

This type of connection is only designed for column joint at 3rd floor level for all columns and at roof top level where required as shown in Figure 3.15 details.

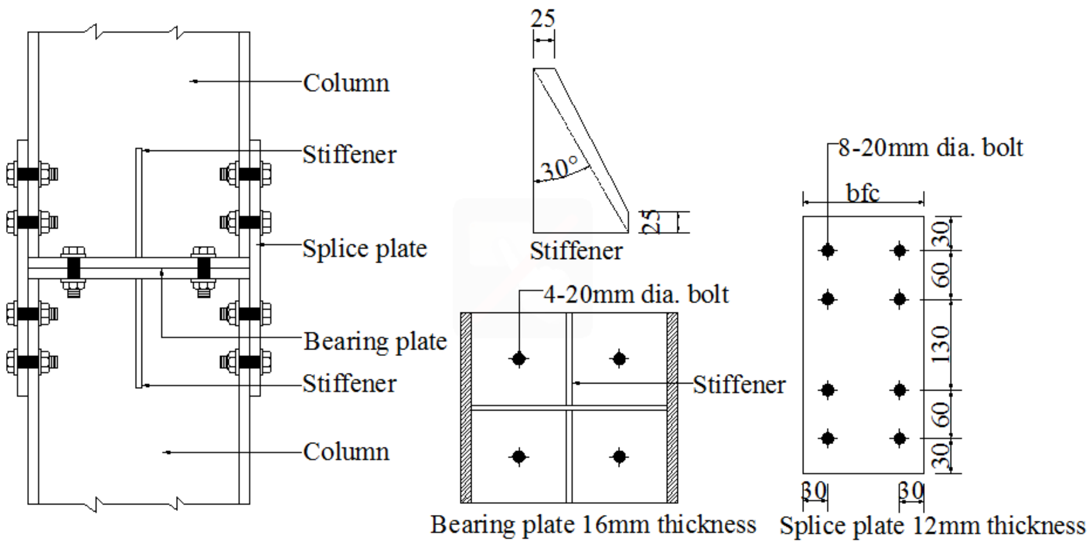


Figure 3.15 Column joint

Beam Moment Connection with Column Web

This type of connection is only designed for secondary beam moment connection with column web at grid as shown in Figure 3.16 details.

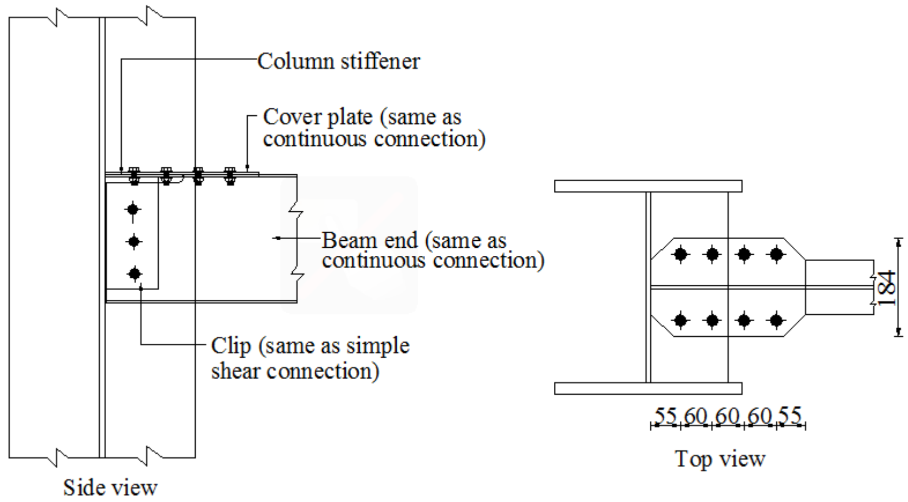


Figure 3.16 Moment connection of beam with column web

Bracing End Hinge Connections

This type of connection is designed for diagonal bracing end joint with beam or column as shown in Figure 3.17 details.

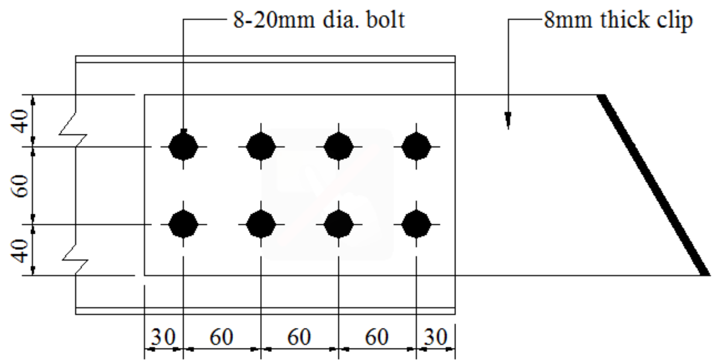


Figure 3.17 Connection of diagonal bracing

3.6.1.3 Design of Sub-Structure and Floor Slab

Sub structure of steel framed building comprises of RC footing, pedestal, grade beam, lift core wall, water tank, slab on grade etc. Floor slab with or without steel deck is designed as one-way RC slab. Following USD method as per ACI Building Code2008, these RC members are designed. After completion of design, necessary drawing sheets have been prepared. Specifications used for design of these RC members are as follows**:**

- Allowable bearing capacity of soil= 168 kN/m^2 (3.5 ksf) assumed.
- Concrete strength, f_c' =21 MPa (3ksi) considered for all concrete work except pedestal.
- Concrete strength, f_c' = 24 MPa (3.5ksi) considered for pedestal.
- Yield strength of reinforcing bar f_y=415 MPa (60 ksi) considered.

3.6.2 Design of RC structure

Two types of RC framed building structure (two-way slab supported on beam and two-way flat plate supported on column directly) are designed using the structural analysis results. RC structure with slab-beam floor system comprises of RC footing, column, grade beam, lift core shear wall, water tank, slab on grade, two-way floor slab, floor beam, stair case etc. RC structure with flat plate floor system comprises of RC footing, column, grade beam, lift core shear wall, water tank, slab on grade, flat plate, edge beam, stair case etc.

Following USD method as per ACI Building Code2008, RC structures have been designed. After completion of design, necessary drawing sheets have been prepared. Specifications used for design of RC structures are as follows**:**

- Allowable bearing capacity of soil= 168 kN/m^2 (3.5 ksf) assumed.
- Concrete strength, f_c'=21 MPa (3ksi) considered for all concrete work except column.
- Concrete strength, f_c' = 24 MPa (3.5ksi) considered for column.
- Yield strength of reinforcing bar f_y=415 MPa (60 ksi) considered.

3.7 Stress Ratio for steel structures

After designing all the steel structural members, the section properties of STAAD.Pro models for steel structures have been corrected. After that, final analysis is performed and stress ratios from STAAD.Pro models are obtained which are given in Table 3.10, 3.11, 3.12, 3.13, 3.14 and 3.15.

Table 3.10 Stress ratio for columns (steel NCF system)

Column	Stress ratio at ground floor	Stress ratio at 1st floor	Stress ratio at 2nd floor	Stress ratio at 3rd floor	Stress ratio at 4th floor	Stress ratio at 5th floor
C_1	0.98-1.05	0.97-1.02	0.96-1.03	0.90-1.04	0.80-1.06	0.34-0.96
C_2	0.99-1.01	0.99-1.00	0.95-0.98	0.96-0.98	0.87-0.94	0.37-0.51
C_3	1.01-1.09	1.02-1.03	0.92-1.06	0.99-1.05	0.81-1.04	0.37-0.91
C_4	0.81-1.00	0.95-1.04	0.79-1.09	0.82-1.06	0.58-1.05	0.38-0.88
C_5	0.89-1.05	0.97-1.02	1.01-1.04	0.87-1.05	0.67-0.98	0.48-0.68
C_6	0.80-0.94	0.82-1.03	0.82-1.03	0.82-1.02	0.82-1.00	0.60-0.71
C_7	0.67-0.95	0.54-0.98	0.46-0.86	0.41-0.78	0.36-0.69	0.30-0.57

Table 3.11 Stress ratio for columns (steel CF system)

Column	Stress ratio at ground floor	Stress ratio at 1st floor	Stress ratio at 2nd floor	Stress ratio at 3rd floor	Stress ratio at 4th floor	Stress ratio at 5th floor
C_1	0.97-1.04	0.96-1.02	0.95-1.03	0.90-1.04	0.80-1.02	0.33-1.00
C_2	0.99-1.01	0.98-1.01	0.95-0.98	0.95-0.98	0.87-0.94	0.37-0.50
C_3	0.97-1.02	1.02-1.03	0.92-1.05	0.98-1.06	0.81-1.04	0.36-0.91
C_4	0.75-1.05	0.67-1.01	0.61-1.08	0.56-1.06	0.44-1.05	0.32-0.86
C_5	0.90-0.96	0.98-1.07	1.01-1.02	0.87-1.05	0.85-0.98	0.49-0.68
C_6	0.80-0.97	0.82-1.03	0.81-1.03	0.81-1.02	0.82-1.00	0.59-0.71
C_7	0.72-0.99	0.61-0.99	0.52-0.86	0.45-0.78	0.40-0.67	0.29-0.57

Table 3.12 Stress ratio for secondary beams

Secondary beams	Stress ratio of non-composite beams (when act as composite)	Stress ratio of composite beams
SB_1	0.37-0.41	0.54-0.62
SB_2	0.41-0.43	0.70-0.74
SB_3	0.40-0.41	0.54-0.62
SB_4	0.43	0.80-0.81
SB_5	0.16-0.23	0.58-0.59
G-SB_1	0.37-0.41	0.54-0.62
G-SB_2	0.65-0.75	N/A
G-SB_3	0.16-0.23	0.58-0.59
G-SB_4	0.34-0.36	0.51-0.54
LB	0.14-1.46	0.28-0.65

Table 3.13 Stress ratio for girders

Girders	Stress ratio of non-composite girders (when act as composite)	Stress ratio of composite girders
G_1	0.13-0.51	0.16-0.81
G_2	0.07-0.41	0.04-0.57
G_3	0.09-0.41	0.06-0.56
G_4	0.12-0.43	0.16-0.59
G_5	0.17-0.22	0.32-0.39
G_6	0.63	0.72
G_7	0.50-0.94	0.94-1.01

Table 3.14 Stress ratio of non-composite secondary beams (when act as non-composite)

Secondary beams	Stress ratio
SB_1	0.87-0.97
SB_2	0.98-0.99
SB_3	0.95-0.96
SB_4	0.99
SB_5	0.85-0.86
$G\text{-}SB_1$	0.81-1.02
$G\text{-}SB_2$	0.82-0.88
$G\text{-}SB_3$	0.70-0.73
$G\text{-}SB_4$	0.81-0.88
LB	0.57-1.02

Table 3.15 Stress ratio of non-composite girders (when act as non-composite)

Girders	Stress ratio
G_1	0.77-1.07

G_2	0.19-0.93
G_3	0.16-0.90
G_4	0.96-1.02
G_5	0.60-0.66
G_6	0.93
G_7	0.61-0.93

3.8 Lateral Drifts and Vertical Deflections

After completing design of all of the building members, the member properties of all the four STAAD.Pro models have been corrected. Then after final analysis lateral drifts, top deflections and vertical deflections are observed and recorded for discussion. Lateral drifts and top deflections for all the four structures are shown in Table 3.16.

Table 3.16 Lateral Drifts and Top Deflections

Type of structures	Story drift at 1^{st} slab level, mm	Story drift at 2^{nd} slab level, mm	Story drift at 3^{rd} slab level, mm	Story drift at 4^{th} slab level, mm	Story drift at 5^{th} slab level, mm	Story drift at 6^{th} slab level (top deflection), mm
Steel (NCF)	6 (H/929)	14 (H/646)	22 (H/571)	29 (H/541)	37 (H/519)	42 (H/536)
Steel (CF)	7 (H/797)	16 (H/566)	26 (H/479)	36 (H/447)	45 (H/428)	51 (H/440)
RC (beam-slab)	3 (H/2057)	5 (H/1820)	8 (H/1575)	11 (H/1471)	14 (H/140)	16 (H/1409)
RC (flat plate)	4 (H/1649)	7 (H/1304)	11 (H/1100)	16 (H/1012)	20 (H/972)	23 (H/973)

Note**:** H= Story height from base.

Vertical deflections of floor beams and girders for steel structures are given in Table 3.17, 3.18, 3.19 and 3.20. Pre-composite deflections of steel girders and beams must be smaller than equal to L/360 or 1 inch (25 mm) which is smaller. Composite deflections of steel girders and beams also must be smaller than equal to L/360 or 1 inch (25 mm) considering 50% live load which is smaller. Vertical deflections of floor beams and flat plates for RC structures are given in Table 3.21 and 3.22.

. **Table 3.17** Vertical deflections of steel secondary beams

Secondary beam notations	Pre-composite deflections of non-composite beams (mm)	Composite deflections of non-composite beams (mm)	Pre-composite deflections of composite beams (mm)	Composite deflections of composite beams (mm)
SB_1	5	3-10	10	6-18
SB_2	7	6-7	17	12-14
SB_3	5	1	11	2
SB_4	9	2	25	5
SB_5	6	4-6	18	6
G-SB_1	5	3-10	10	6-18
G-SB_2	2-18	1	N/A	N/A
G-SB_3	3	4-6	4-8	5
G-SB_4	7	2	17	4
LB	1	2-4	1	1

Table 3.18 Vertical deflections of steel girders

Girder notations	Pre-composite deflections of non-composite girders (mm)	Composite deflections of non-composite girders (mm)	Pre-composite deflections of composite girders (mm)	Composite deflections of composite girders (mm)
G_1	3-5	2-5	4-9	4-10
G_2	1-4	2-6	1-7	4-10
G_3	1-6	1-3	5-9	2-6
G_4	4-6	2-4	5-10	3-8
G_5	2-3	2-3	4-6	4-6
G_6	3	3	5	4
G_7	1-2	`1-4	1-4	1-8

. **Table 3.19** Vertical deflections of steel NC beams (without composite action)

Secondary beam notations	Deflections during concreting (mm)	Deflections due to live load (mm)
SB_1	9-16	19-31
SB_2	9-11	24-25
SB_3	13-15	4
SB_4	11	9
SB_5	6	12
$G\text{-}SB_1$	6-8	4-26
$G\text{-}SB_2$	1-2	1
$G\text{-}SB_3$	2-3	3-6
$G\text{-}SB_4$	10	14-19
LB	1	1-2

. **Table 3.20** Vertical deflections of steel NC girders (without composite action)

Girder notations	Deflections during concreting (mm)	Deflections due to live load (mm)
G_1	3-5	6-17
G_2	1-4	2-14
G_3	1-6	2-10
G_4	4-6	9-16
G_5	2-3	2-3
G_6	Stair Landing Beam	Stair Landing Beam
G_7	1-2	1-4

Table 3.21 Vertical deflections of floor beams (RC beam-slab system)

Beam notations	Dead load deflections (mm)	live load deflections (mm)
B_1	3-4	2-3
B_2	2-5	1-2
B_3	2-4	1
B_4	1-3	1
B_5	1	1
B_6	2-4	1
B_7	2-4	1-3
B_8	3-4	1-3
B_9	3	3
B_{10}	2-4	1-2

Table 3.22 Vertical deflections of RC flat plate system

Flat plate types	Dead load deflections (mm)	live load deflections (mm)
Middle panel	6	6
Side panel	7	6
Corner panel	11	2

3.9 Estimating and Costing

After completion of structural design and drawing; estimating and costing are required for all the four types of structure. At first, item rate analysis and schedule of unit rate have been prepared. After that, complete estimation and costing are completed for two types of steel (NCF and CF system) and two types of RC (beam-slab and flat plate system) structure.

3.9.1 Rate Analysis and Schedule of Item Rate

For RC structural works and other civil and sanitary works, schedule of rate for different item has been prepared following PWD item rate analysis procedure as per present market rate of material, labor and other related costs.

For steel structural works, schedule of rate for different item of steel structure has been prepared by analyzing item rates as per present market rate of material, labor and other related costs following guide lines of PWD rate analysis procedure and present practice of different steel structure fabrication companies. Schedule of item rate is given in Appendix-C.

3.9.2 Estimating and Costing for Steel Structure

Estimating and costing have been prepared for two types of steel structure (NCF and CF system) using schedule of item rate. Now the summary of costing is shown in Table 3.23, 3.24, 3.25, 3.26 and 3.27.

Table 3.23 Foundation cost up to plinth (steel NCF and CF system)

Type of works	Total cost (lac BDT)
1.Structural works	282
2.Other civil works	115
Total foundation cost up to plinth	397

Table 3.24 Typical floor cost of steel NCF system with deck (ground to 4th floor slab)

Type of work	Sub component	Item name	Estimated quantity	Estimated cost (lac BDT)	Total cost (lac BDT)
1.Structural works	Structural steel	Secondary beams	63723 kg	215	286
		Girders	34320 kg		
		Columns	23694 kg		
		Bracings	2425 kg		
	Joints	Joint plates	9141 kg		
		Nut bolts	2826 kg		
		Stud anchor	1691 kg		
	Deck	Decking sheet	33541 kg		
		Decking screw	64 kg		
	RC slab			71	
2.Civil works					130
3. Sanitary works					12
Total cost per floor					428

Table 3.25 Typical floor cost of steel CF system with deck (ground to 4^{th} floor slab)

Type of work	Sub component	Item name	Estimated quantity	Estimated cost (lac BDT)	Total cost (lac BDT)
1.Structural works	Structural steel	Secondary beams	47784 kg	184	255
		Girders	25901 kg		
		Columns	23694 kg		
		Bracings	2425 kg		
	Joints	Joint plates	7813 kg		
		Nut bolts	2826 kg		
		Stud anchor	2955 kg		
	Deck	Decking sheet	33541 kg		
		Decking screw	64 kg		
	RC slab			71	
2.Civil works					130
3. Sanitary works					12
Total cost per floor					397

Table 3.26 Top floor cost of steel NCF system with deck

Type of work	Sub component	Item name	Estimated quantity	Estimated cost (lac BDT)	Total cost (lac BDT)
1.Structural works	Structural steel	Secondary beams	67163 kg	238	321
		Girders	38924 kg		
		Columns	29107 kg		
		Bracings	2425 kg		
	Joints	Joint plates	9141 kg		
		Nut bolts	3278 kg		
		Stud anchor	1846 kg		
	Deck	Decking sheet	36442 kg		
		Decking screw	69 kg		
	RC slab			83	
2.Civil works					135
3. Sanitary works					12
4. Water tanks					41
Total cost per floor					509

Table 3.27 Top floor cost of steel CF system with deck

Type of work	Sub component	Item name	Estimated quantity	Estimated cost (lac BDT)	Total cost (lac BDT)
1.Structural works	Structural steel	Secondary beams	50039 kg	204	287
		Girders	28641 kg		
		Columns	29107 kg		
		Bracings	2425 kg		
	Joints	Joint plates	9682 kg		
		Nut bolts	3278kg		
		Stud anchor	1846 kg		
	Deck	Decking sheet	36442 kg		
		Decking screw	69 kg		
	RC slab			83	
2.Civil works					135
3. Sanitary works					12
4. Water tanks					40
Total cost per floor					474

3.9.3 Estimating and Costing for RC Structure

Estimating and costing have been prepared for two types of RC structure (beam-slab and flat plate floor system) using schedule of item rate. Now the summary of costing is shown in Table 3.28, 3.29, 3.30, 3.31, 3.32 and 3.33.

Table 3.28 Foundation cost up to plinth (RC beam-slab system)

Type of works	Total cost (lac BDT)
1.Structural works	367
2.Other civil works	114
Total foundation cost up to plinth	481

Table 3.29 Foundation cost up to plinth (RC flat plate system)

Type of works	Total cost (lac BDT)
1.Structural works	376
2.Other civil works	114
Total foundation cost up to plinth	490

Table 3.30 Typical floor cost of RC beam-slab system (ground to 4th floor slab)

Type of works	Total cost (lac BDT)
1.Structural works	234
2. Civil works	130
3. Sanitary works	12
Total cost per floor	376

Table 3.31 Typical floor cost of RC flat plate system (ground to 4th floor slab)

Type of works	Total cost (lac BDT)
1.Structural works	240
2. Civil works	130
3. Sanitary works	12
Total cost per floor	382

Table 3.32 Top floor cost of RC beam-slab system

Type of works	Total cost (lac BDT)
1.Structural works	253
2. Civil works	135
3. Sanitary works	12
4. Water tanks	40
Top floor total cost	440

Table 3.33 Top floor cost of RC flat plate system

Type of works	Total cost (lac BDT)
1.Structural works	259
2. Civil works	135
3. Sanitary works	12
4. Water tanks	40
Top floor total cost	446

3.10 Concluding Remarks

In this chapter structural analysis, design, estimating and costing of the selected six story garments factory building have been completed using two types of steel structure (NCF and CF system) and two types of RC structure (beam-slab and flat plate floor system). Summary of all data for the four types of structure is now available which has been analyzed in the next chapter.

Chapter 4
ANALYSIS OF RESULTS AND DISCUSSION

4.1 Introduction

Comparison, analysis and discussion have been performed using all data obtained from load calculation, structural modeling and analysis, design, estimating and costing of the selected four types of structural system for the same building.

4.2 Structural Steel Weight Comparison

Structural steel weight comparison of steel structure for non-composite floor (NCF) and composite floor (CF) system is shown in Table 4.1. Graphical presentation of structural steel weight comparison is also shown in Figure 4.1, 4.2 and 4.3.

Table 4.1 Comparison of structural steel weight

Type of steel structure	Typical floor secondary beam weight (kg/m^2)	Typical floor girder weight (kg/m^2)	Typical floor column weight (kg/m^2)	Typical floor bracing weight (kg/m^2)	Weight of typical floor system only (kg/m^2)	Weight of typical floor with column (kg/m^2)	Total weight of building (kg/m^2)
NCF system	13.42	7.23	5	0.51	23.51	29	29.57
CF system	10.06	5.45	5	0.51	18.36	23.86	24.32
Comment	25% wt. savings for composite action.	24 % wt. savings for composite action.	Remain unchanged.	Remain unchanged.	22 % wt. savings for composite action.	18 % wt. savings for composite action.	18 % wt. savings for composite action.

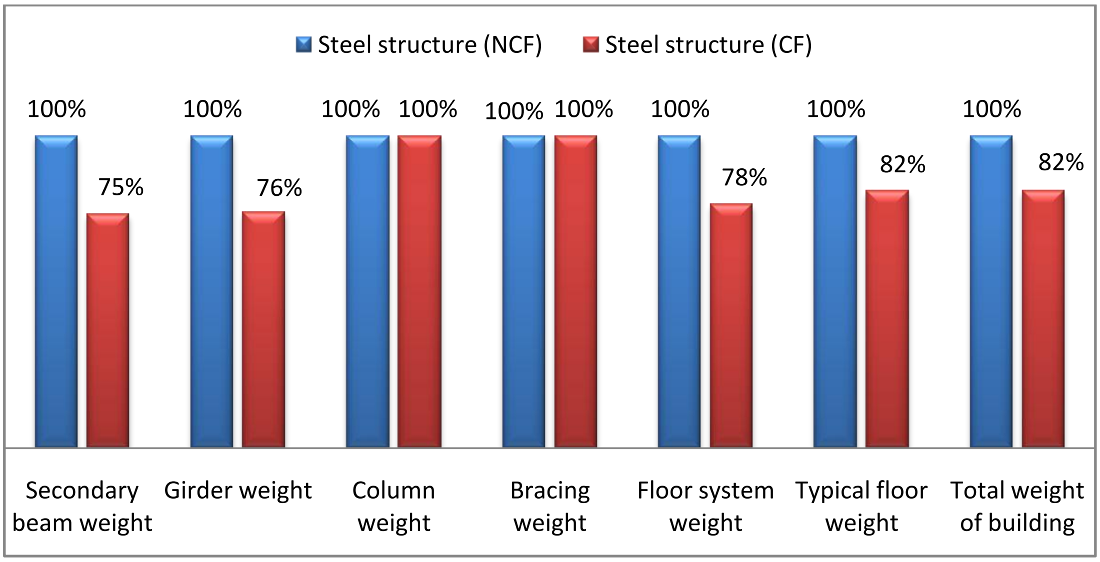


Figure 4.1 Comparison of structural steel weight

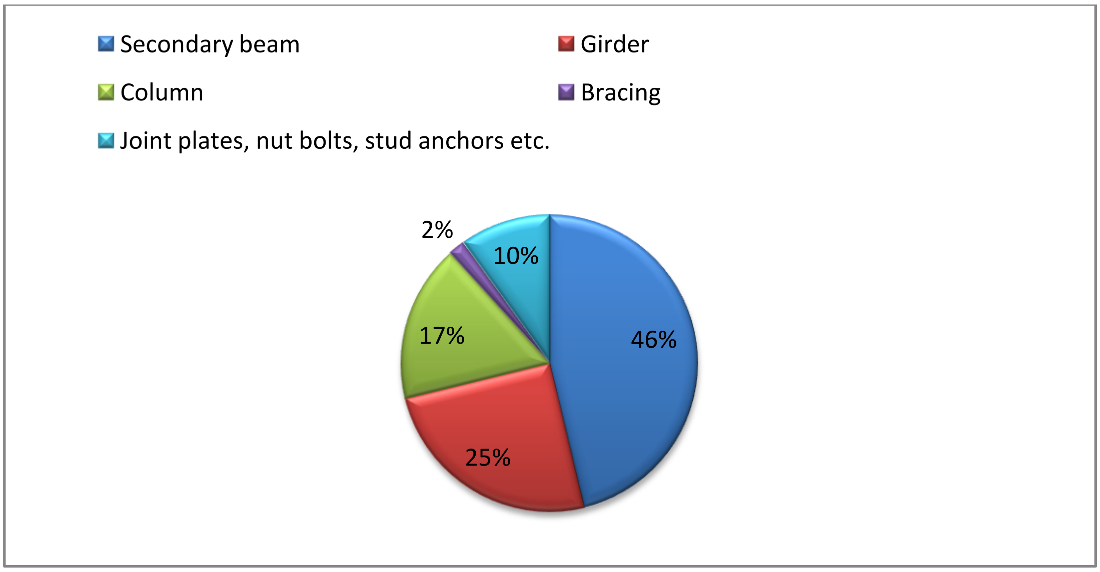


Figure 4.2 Structural steel weight analysis for non-composite floor system

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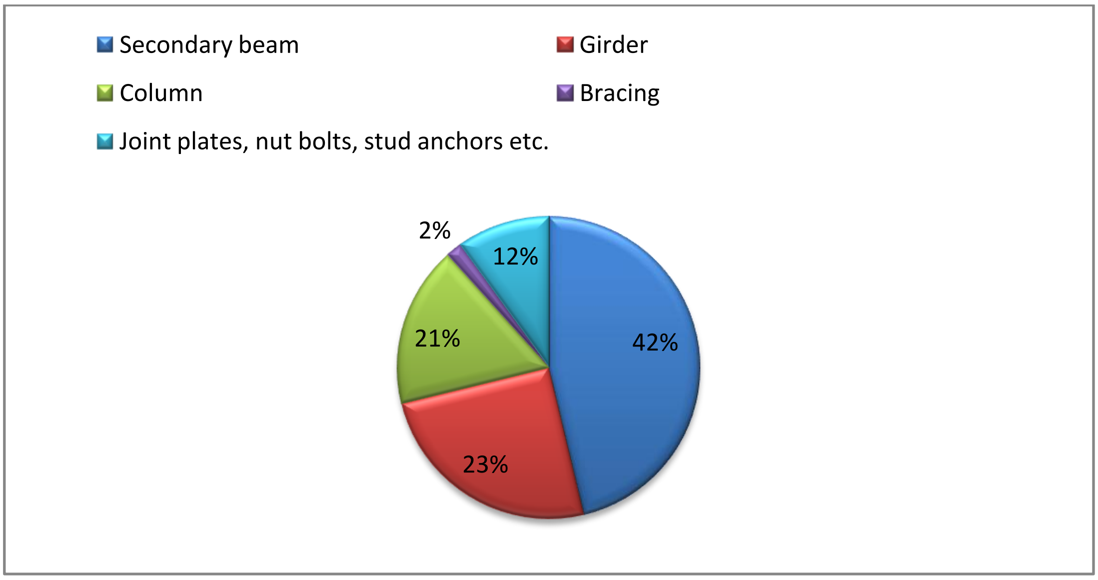

Figure 4.3 Structural steel weight analysis for composite floor system

From the analysis and comparison given in Table 4.1 and Figure 4.1, 4.2 and 4.3 the major findings are as follows**:**

- When floor system of steel structure is designed as composite then secondary beam weight decreases about 25% and girder weight decreases about 24%. Column and bracing weights remain same.
- Due to composite action, weight savings for floor system (only beam and girder) is about 22%. Weight savings for typical floor (including column, bracing etc.) is about 18%. Finally net weight savings for total building is about 18%.
- Weight of secondary steel I-beam is about 42-46% of total structural steel weight. So designers should be careful during floor system planning to ensure cost effective spacing and span of secondary beam.
- Structural steel weight per square meter is 29.57 kilogram for non-composite system and 24.32 kilogram for composite system.

4.3 Cost Comparison of Steel Structures

Construction cost comparison of steel structure (with and without steel deck) for non-composite floor (NCF) and composite floor (CF) system is shown in Table 4.2. Graphical presentation of cost comparison is also shown in Figure 4.4, 4.5, 4.6, 4.7 and 4.8.

Table 4.2 Cost comparison of steel structure

Type of steel structure		NCF with deck	NCF without deck	CF with deck	CF without deck
Foundation cost up to plinth (lac BDT)	Structural cost	282	282	282	282
	Total cost	397	397	397	397
Typical floor cost (lac BDT)	Structural steel cost	171	171	139	139
	Structural cost	286	289	255	258
	Total cost	428	.431	397	400
Super structure cost (lac BDT)	Structural steel cost	1048	1048	851	851
	Structural cost	1750	1768	1561	1579
	Total cost	2649	2668	2460	2463
Total project cost (lac BDT)	Total structural steel cost	1048	1048	851	851
	Total structural cost	2032	2051	1843	1862
	Total project cost	3046	3065	2857	2860
Total project cost with fire proof spray (lac BDT)	Total structural steel cost	1591	1545	1302	1262
	Total structural cost	2575	2547	2294	2272
	Total project cost	3589	3561	3308	3271

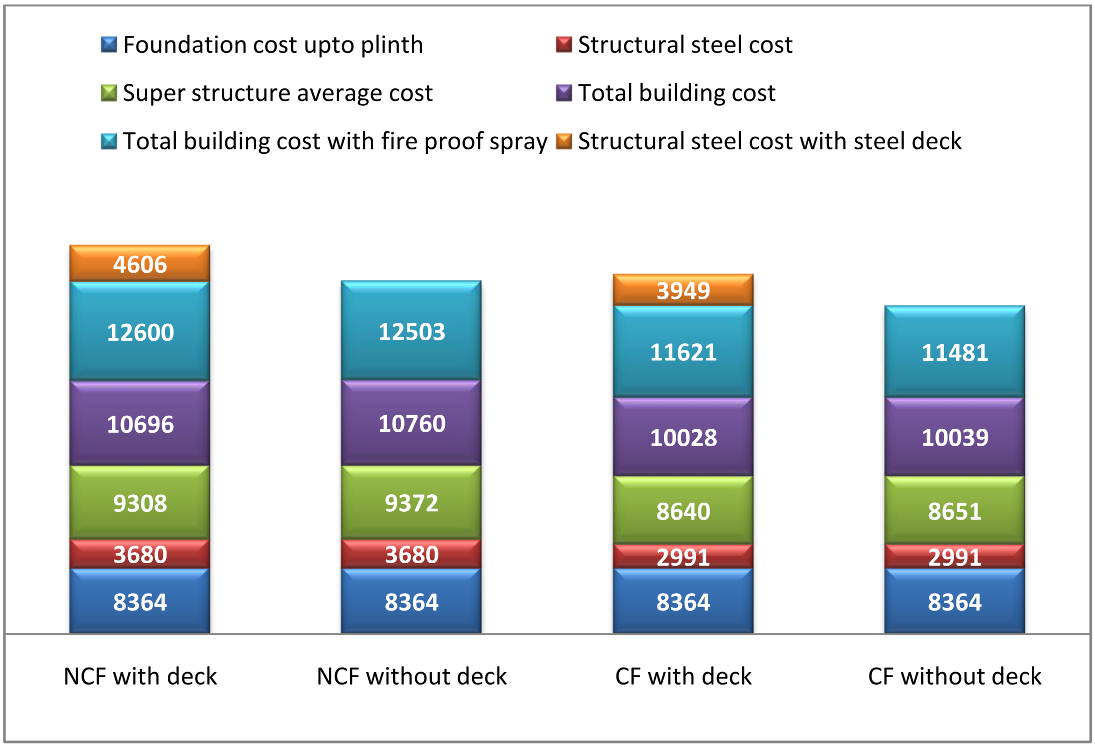

Figure 4.4 Cost of steel structure (BDT per square meter)

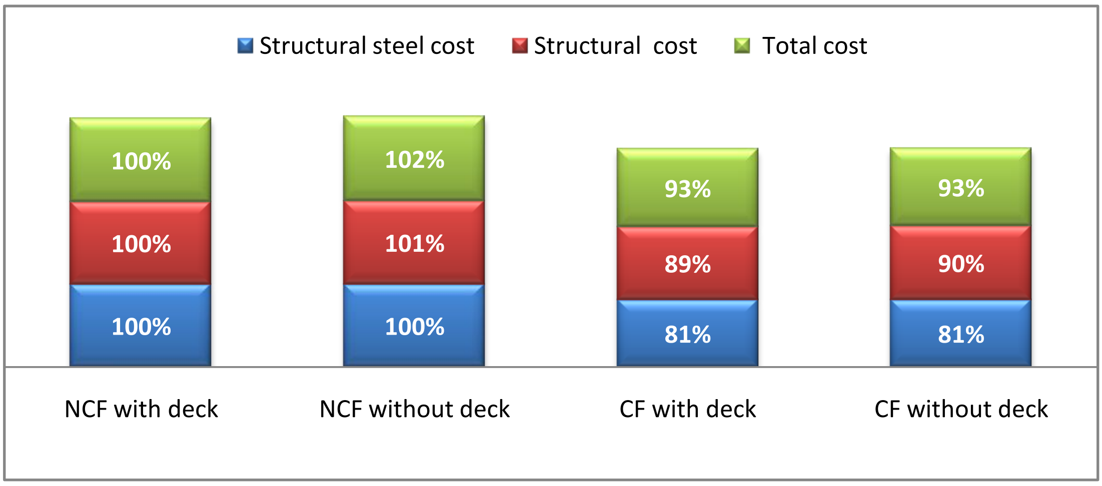

Figure 4.5 Super structure cost comparison of steel structure

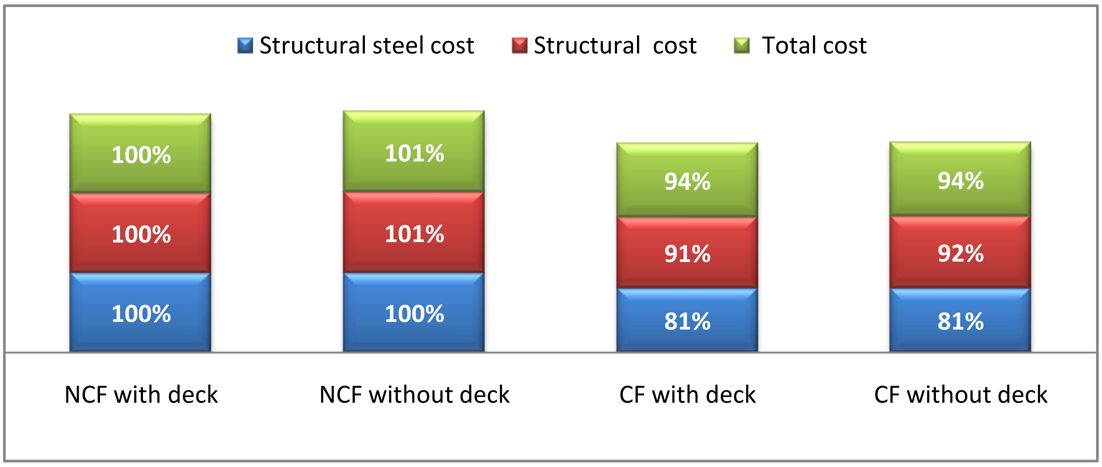

Figure 4.6 Total building cost comparison of steel structure

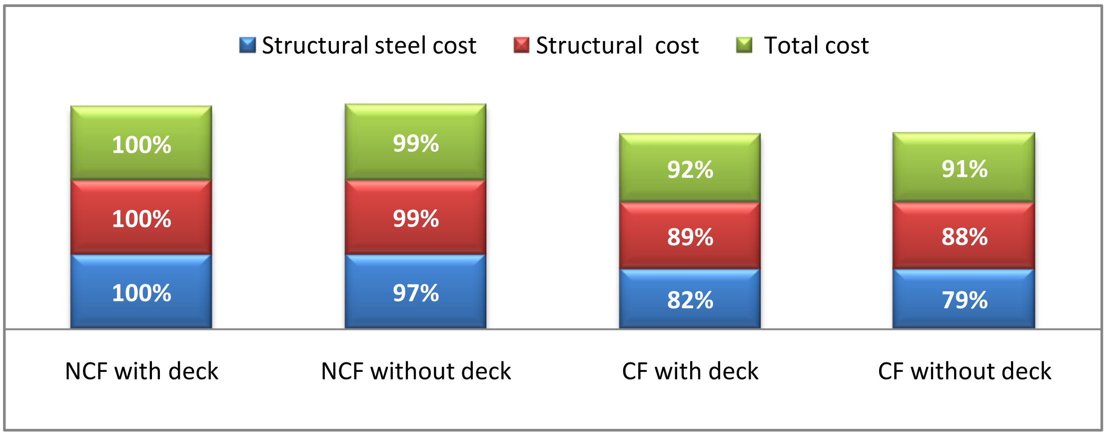

Figure 4.7 Total building cost comparison of steel structure (with fire proof spray)

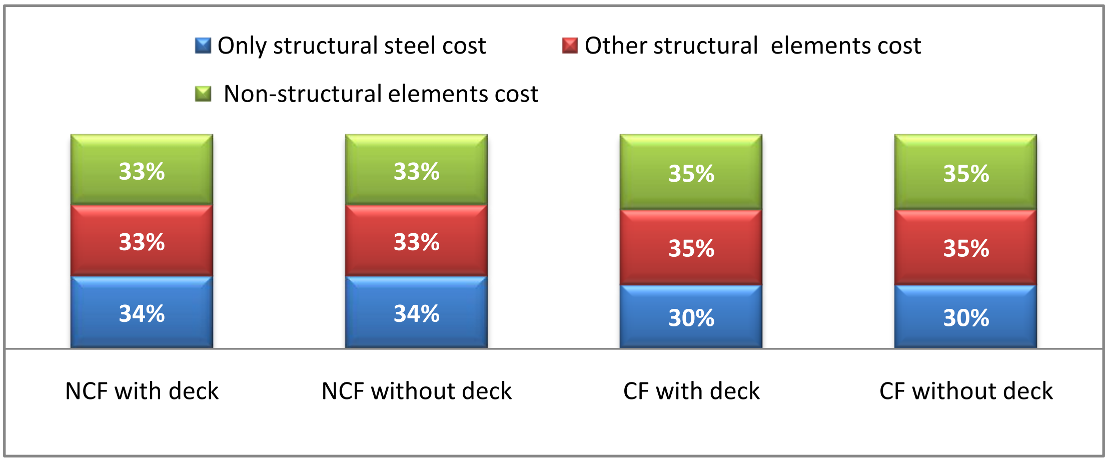

Figure 4.8 Distribution of total building cost for steel structure

From the analysis and comparison shown in Table 4.2 and Figure 4.4, 4.5, 4.6, 4.7 and 4.8 the major findings are as follows**:**

- Composite system brings significant economic benefit for steel structure which is described below.

  Super structure cost: Only structural steel cost savings is about 19%, total structural cost savings is about 11% and finally total cost savings is about 7-9%.

  Total building cost: Only structural steel cost savings is about 19%, total structural cost savings is about 9% and finally total building cost savings is about 6-7%.

  Total building cost with fire proof spray: Only structural steel cost savings is about 18%, total structural cost savings is about 11% and finally total building cost savings is about 8%.
- Analyzing total project cost, only structural steel cost is 30-34%, other structural cost is 33-35% and non-structural cost is 33-35% i.e. only structural cost is about two third. So designers should care about economic and safe structural design.
- If fire proof spray is used for steel structure then total building cost increases about 14-18%.
- Cost of structural steel is BDT 2991-3680 per square meter and cost of structural steel with steel deck is BDT 3949-4606 per square meter.

4.4 Cost Comparison of Steel and RC Structures

Construction cost comparison of steel structure (NCF and CF system with and without steel deck) and RC structure (slab-beam and flat plate floor system) is shown in Table 4.3. Graphical presentation of cost comparison is also shown in Figure 4.9, 4.10, 4.11, 4.12, 4.13 and 4.14.

Table 4.3 Cost comparison of steel and RC structure

Type of structure	Foundation cost up to plinth (lac BDT)		Super structure cost (lac BDT)		Total building cost (lac BDT)		Total building cost with fire proof spray (lac BDT)	
	Structural cost	Total cost	Structural cost	Total cost	Structural cost	Total cost	Structural cost	Total cost
Steel (NCF with deck)	283	397	1750	2650	2032	3047	2575	3589
Steel (NCF without deck)	283	397	1768	2668	2051	3065	2547	3561
Steel (CF with deck)	283	397	1561	2460	1843	2858	2294	3308
Steel (CF without deck)	282	397	1579	2463	1862	2861	2272	3271
RC (beam-slab)	367	482	1425	2324	1792	2806	N/A	N/A
RC (flat plate)	376	491	1461	2360	1837	2851	N/A	N/A

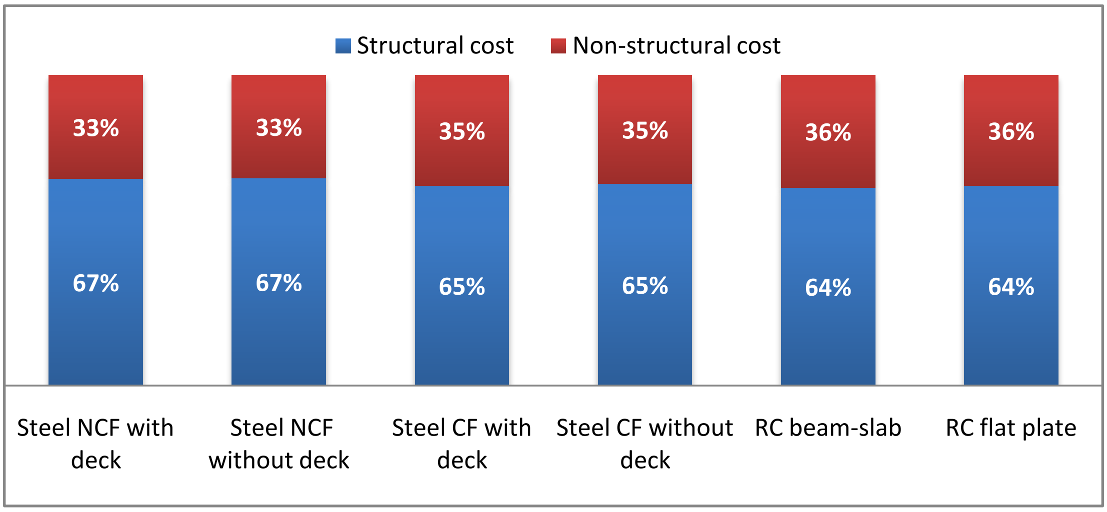

Figure 4.9 Distribution of total building cost of steel and RC structures

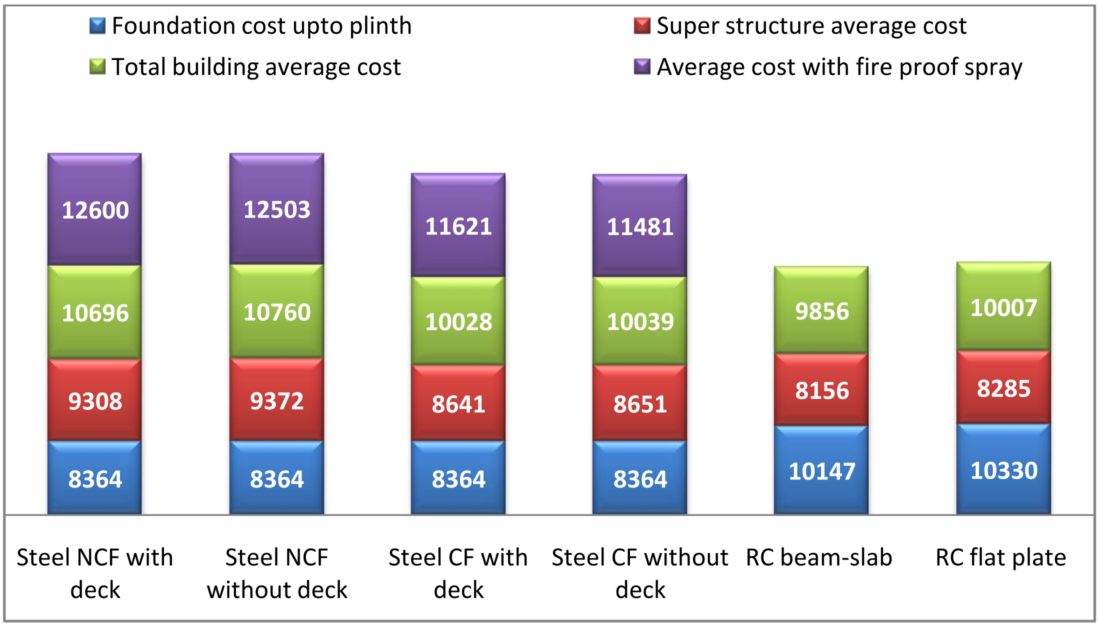

Figure 4.10 Cost of steel and RC structure (BDT per square meter)

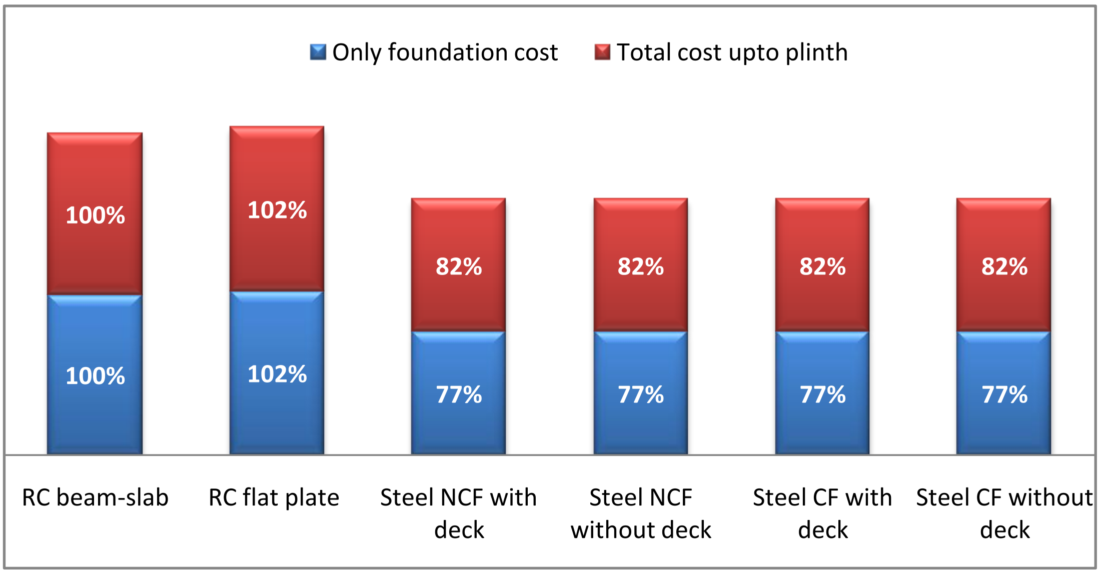

Figure 4.11 Comparison of foundation cost up to plinth

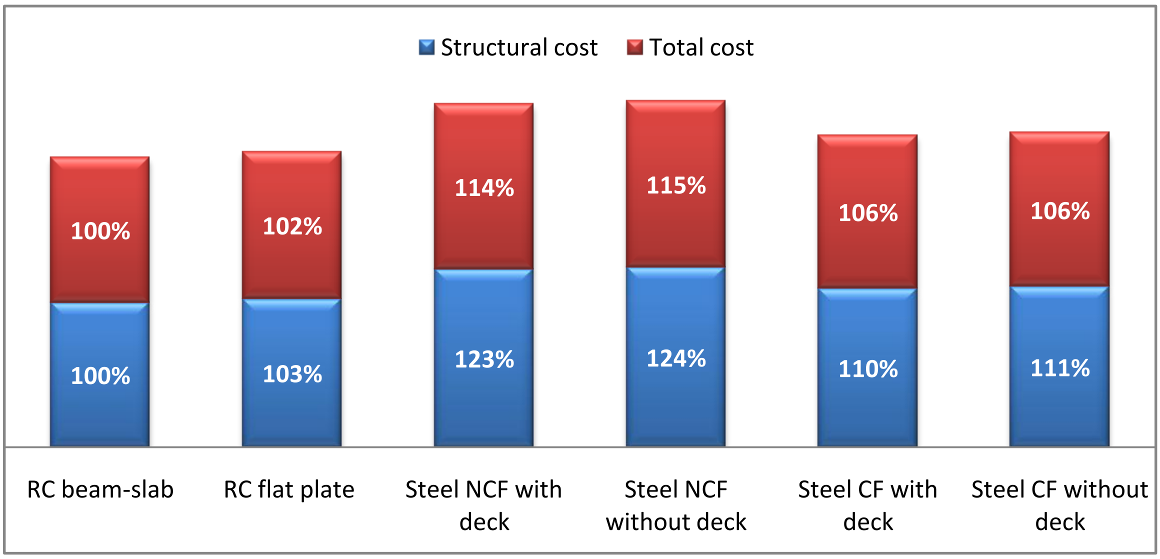

Figure 4.12 Super structure cost comparison

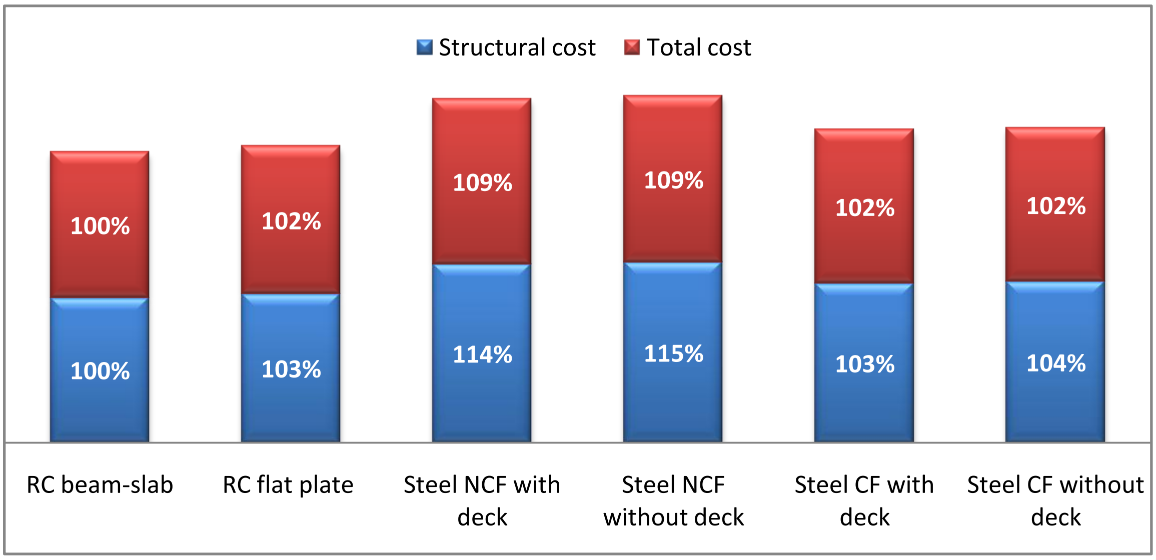

Figure 4.13 Total building cost comparison

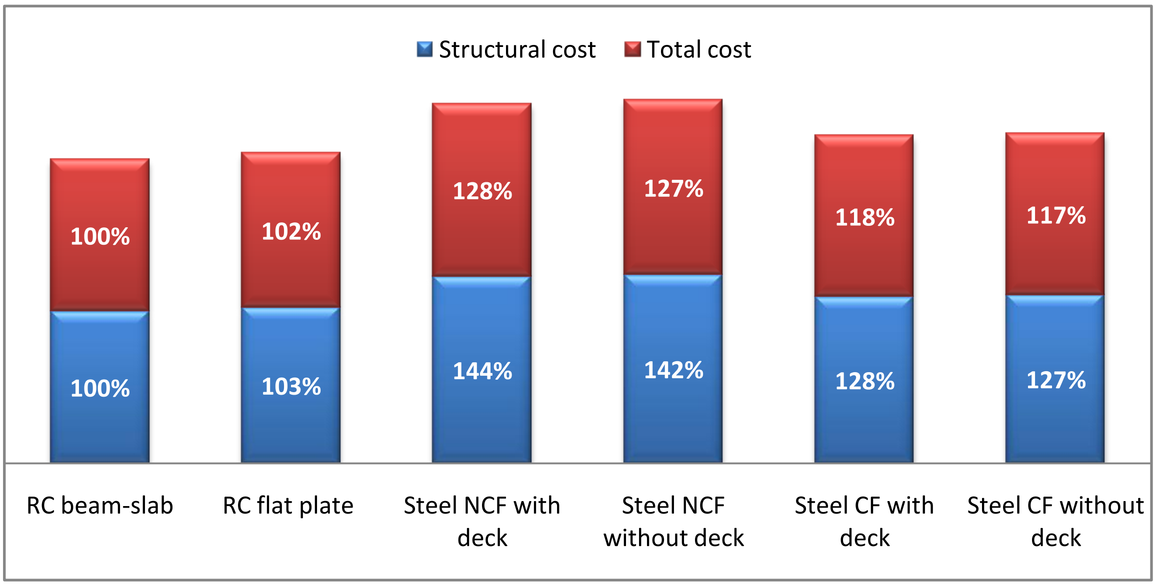

Figure 4.14 Total building cost comparison with fire proof spray at steel structure

From the analysis and comparison shown in Table 4.3 and Figure 4.9, 4.10, 4.11, 4.12, 4.13 and 4.14 the major findings are as follows**:**

- For the selected building, only structural cost is about 64-67% for both steel and RC structural system. So structural designers should care about economic and safe structural design.
- Construction cost of steel structure is BDT 10028-10760 per square meter and when fire proof spray is added then it becomes BDT 11481-12600 per square meter. Again construction cost for RC structure is BDT 9856-10007 per square meter.
- Compared to steel structure, foundation cost up to plinth of RC structure increases about 22% for beam-slab floor system and about 24% increases for flat plate floor system. So initial investment for RC structure is more.
- RC flat plate system is about 2% more costly than RC beam-slab system.
- Compared to RC systems, super structure total cost increases about 12-15% for steel non-composite system and about 4-6% increases for steel composite system. Only super structure structural cost increases about 19-23% for steel non-composite system and about 7-11% increases for steel composite system compared to RC system.
- Compared to RC buildings, steel building total cost increases about 7-9% for non-composite and about 0-2% for composite system. Total structural cost of steel building increases about 11-15% for non-composite and about 0-4% for composite system compared to RC structures.

- Compared to RC buildings, steel building total cost increases about 25-28% for non-composite and about 15-18% for composite system when fire proof spray is added. In this case only structural cost increases about 38-44% for steel non-composite system and about 23-28% increases for steel composite system compared to RC system.
- Only total structural cost increases about 3% for flat plate, about 3-4% for steel CF system and about 13-15% for steel NCF system compared to RC beam-slab system.
- Compared to RC beam-slab system, only total structural cost increases about 27-28% for steel CF system and about 42-44% for steel NCF system when fire proof spray is added.

4.5 Comparison of Seismic Load

After calculation of seismic load, significant variation is observed. Comparison of seismic load and related parameters are shown in Table 4.4. Graphical presentation of seismic load comparison is also shown in Figure 4.15.

Table 4.4 Comparison of seismic load

Item	Steel NCF with deck	Steel NCF without deck	Steel CF with deck	Steel CF without deck	RC beam-slab floor	RC flat plate floor
Seismic dead load	174354 kN	191377 kN	174354 kN	191377 kN	283447 kN	292975 kN
Total base shear	4585 kN	5033 kN	4585 kN	5033 kN	10170 kN	10512 kN
Period	0.7885 sec.	0.7885 sec.	0.7885 sec.	0.7885 sec.	0.6935 sec.	0.6935 sec.
C_t	0.083	0.083	0.083	0.083	0.073	0.073
Response modification factor, R	10	10	10	10	8	8
Additional top force, F_t	253 kN	278 kN	253 kN	278 kN	0	0

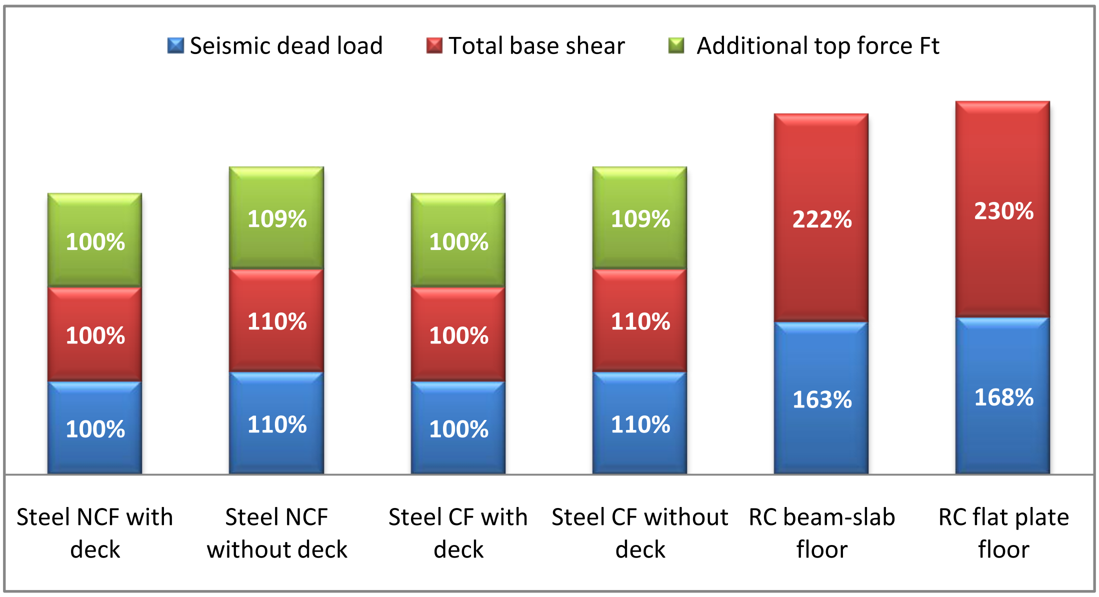

Figure 4.15 Comparison of seismic load

From the analysis and comparison shown in Table 4.4 and Figure 4.15 the major findings are as follows**:**

- Compared to steel structure, base shear due to seismic force is 102-122% more for RC beam-slab system and 109-130% more for RC flat plate system.
- Compared to steel structure, seismic dead load is 48-63% more for RC beam-slab system and 53-68% more for RC flat plate system.

4.6 Comparison of Gravity Loads

Significant variation is observed after calculation of gravity loads such as self weight of structure, dead load etc. Comparison of gravity loads are shown in Table 4.5. Graphical presentation of gravity load comparison is also shown in Figure 4.16.

Table 4.5 Comparison of gravity loads

Item	Steel NCF with deck	Steel NCF without deck	Steel CF with deck	Steel CF without deck	RC beam-slab floor	RC flat plate floor
Structural self weight	87185 kN	95694 kN	85726 kN	94235 kN	196450 kN	200320 kN
Total dead load	153714 kN	162223 kN	152255 kN	160764 kN	258562 kN	262917 kN
Total foundation axial load (un-factored)	270047 kN	278556 kN	268588 kN	277097 kN	377550 kN	381905 kN
Total foundation axial load (factored)	370590 kN	380802 kN	368842 kN	379050 kN	500654 kN	505880 kN
Typical footing axial load (un-factored)	3354 kN	3461 kN	3336 kN	3443 kN	4403 kN	4492 kN
Typical footing axial load (factored)	4350 kN	4479 kN	4328 kN	4457 kN	5733 kN	5893 kN

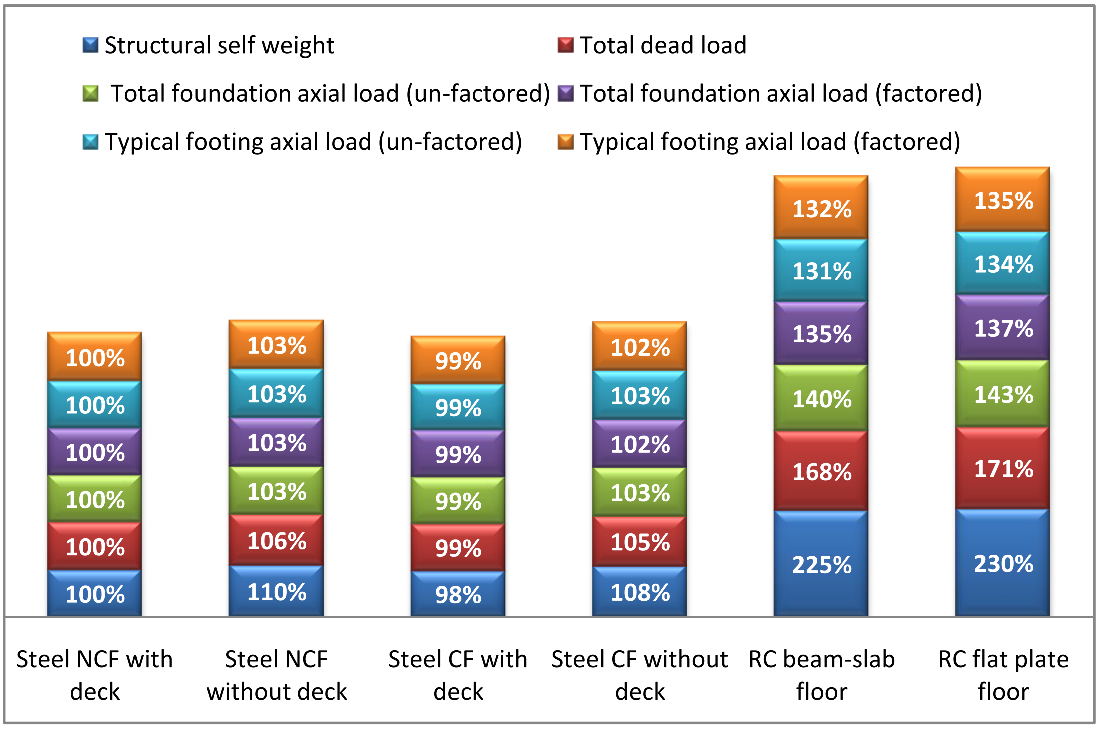

Figure 4.16 Comparison of gravity loads

From the analysis and comparison shown in Table 4.5 and Figure 4.16 the major findings are as follows**:**

- Structural self weight is 125% more for RC beam-slab system and 130% more for RC flat plate system compared to steel NCF system with deck.
- Dead load is 68% more for RC beam-slab system and 71% more for RC flat plate system compared to steel NCF system with deck.
- Compared to steel NCF system with deck, total foundation axial load (un-factored) is 40% more for RC beam-slab system and 43% more for RC flat plate system.

4.7 Comparison of Lateral Story Drift

Comparison of lateral story drift and top deflection are shown in Table 4.6 for two types of steel (NCF and CF system) and two types of RC structures (beam-slab and flat plate floor system).

Table 4.6 Comparison of lateral story drifts

Type of structures	Story drift at 1st slab level, mm	Story drift at 2nd slab level, mm	Story drift at 3rd slab level, mm	Story drift at 4th slab level, mm	Story drift at 5th slab level, mm	Story drift at 6th slab level (top deflection), mm
Steel NCF	6 (H/929)	14 (H/646)	22 (H/571)	29 (H/541)	37 (H/519)	42 (H/536)
Steel CF	7 (H/797)	16 (H/566)	26 (H/479)	36 (H/447)	45 (H/428)	51 (H/440)
RC beam-slab	3 (H/2057)	5 (H/1820)	8 (H/1575)	11 (H/1471)	14 (H/140)	16 (H/1409)
RC flat plate	4 (H/1649)	7 (H/1304)	11 (H/1100)	16 (H/1012)	20 (H/972)	23 (H/973)

Note**:** H=Story height from base.

From the comparison shown in Table 4.6 about lateral story drift and top deflection, the major findings are as follows**:**

- The lateral story drift and top deflection of all the four types of structure are within allowable limit.
- Lateral stiffness of RC beam-slab system is the highest of all. Lateral stiffness of RC flat plate system is the 2nd highest and that of steel non-composite system is the 3rd highest. Lateral stiffness of steel composite system is the lowest of all.

4.8 Comparison of Vertical Deflection

Comparison of vertical deflection is shown in Table 4.7 for all the four types of structure selected for the research work.

Table 4.7 Comparison of vertical deflection

Type of structure	Typical floor max^m Dead load deflection (mm)	Typical floor max^m Live load deflection (mm)
Steel non-composite floor	Beam: 9 Girder: 6	Beam: 10 Girder: 6
Steel composite floor	Beam: 18 Girder: 10	Beam: 18 Girder: 10
RC beam-slab floor	Beam: 5	Beam: 3
RC flat plate floor	Flat plate:11	Flat plate: 6

From the comparison shown in Table 4.7 about vertical deflection, the major findings are as follows**:**

- Vertical deflections of all the four types of structure are within allowable limit.
- Vertical deflection due to dead load**:** Lowest for RC beam-slab system, 2nd lowest for steel non-composite system, 3rd lowest for RC flat plate system and highest for steel composite system.
- Vertical deflection due to live load**:** Lowest for RC beam-slab system, 2nd lowest for RC flat plate system, 3rd lowest for steel non-composite system and highest for steel composite system.
- Though, pre-composite deflection (18 mm) of steel composite secondary beam is within allowable limit (minimum of L/360 and 25 mm); visible deflection may be cause of panic and objection from client about structural safety and ponding effect. Slab thickness will increase at mid span for excess deflection. Considering significant economic advantage of composite section, this problem may be reduced properly by using any of the following treatment**:**

  (a) During fabrication of steel beam at shop, cambering may be introduced up to 80% of calculated deflection.

  (b) Secondary beam continuous connection may be designed as friction type (slip critical) connection in lieu of bearing type connection. It must be confirmed by making the connection "snug tight" condition using high tension bolts and proper calibrated wrench during erection.

(c) Only secondary beam may be shored at mid span with single prop before concreting. After seven days, this prop may be removed and then self-weight deflection will not be significant as composite action starts.

(d) Structural engineers should be more careful during planning of floor system to make the secondary beam spacing and length effective to minimize vertical deflection.

4.9 Comparison of Stress Ratio

Steel non-composite and composite members are designed following AISC LRFD method. Using these designed values, finally section properties are changed into STAAD.Pro model which shows that the stress ratios are smaller than one. So designed sections are correct. Comparison of stress ratio is shown in Table 4.8 for two types of steel structure assigned for the research work.

Table 4.8 Comparison of stress ratio

Steel non-composite floor system						Steel composite floor system					
Beam		Girder		Column		Beam		Girder		Column	
Designed stress ratio	STAAD.Pro stress ratio	Designed stress ratio	STAAD.Pro stress ratio	Designed stress ratio	STAAD.Pro stress ratio	Designed stress ratio	STAAD.Pro stress ratio	Designed stress ratio	STAAD.Pro stress ratio	Designed stress ratio	STAAD.Pro stress ratio
≤ 1	0.97	≤ 1	0.99	≤ 1	0.98	≤ 1	0.62	≤ 1	0.81	≤ 1	0.97

4.10 Concluding Remarks

From the analysis and comparison of all data obtained from the design program assigned for the research, some important findings are achieved. From these findings, some conclusions and recommendations are drawn as the outcome of the research program.

Chapter 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 Introduction

Comparative study has been performed using all data obtained from load calculation, structural modeling, analysis, design, estimating and costing of the selected building using two types of RC structure (beam-slab and flat plate floor) and two types of steel structure (composite and non-composite floor). From the comparative study of steel structure with composite floor and non-composite floor system, important findings are obtained. Again from comparative study on two types of RC and two types of steel structure; important findings about economy, structural performance and other related structural matters are obtained as research outcome. Based on these findings, final conclusions are drawn and presented in the following sections.

5.2 Conclusions

From the comparative study of the four types of structure with different floor system of same building; major findings and conclusions about construction cost, structural behavior and other structural matters are as follows.

5.2.1 Steel Structure

Steel structure with composite and non-composite floor system has significant variation about economy and structural performance. Major conclusions are as follows**:**

- When floor system of steel structure is designed as composite then about 18% structural steel weight savings is possible.
- Composite system brings significant economic benefit for steel structure i.e. only structural steel cost savings is about 18-19%, total structural cost savings is 9-11% and finally total building cost savings is 6-8%.
- Weight of secondary steel I-beam is about 42-46% of total structural steel weight. So designers should be careful about cost effective spacing and span of secondary beam during planning of floor system.

- If fire proof spray is used for steel structure then total building cost increases about 14-18%.
- Pre-composite vertical deflection of composite secondary steel I-beam is a serviceability problem which should be minimized by cambering, introducing friction type connection and shored construction.

5.2.2 Reinforced Concrete and Steel Structure

When same building is designed as reinforced concrete and steel structure then construction cost varies significantly. The following conclusions are drawn from comparative study**:**

- Compared to steel structure, “foundation cost up to plinth” increases about 22% for RC beam-slab system and about 24% for RC flat plate system. So initial investment for RC structure is more than steel structure.
- Compared to RC beam-slab system, finally “complete building cost” increases 1.6% for RC flat plate system, 1.8% for steel composite system and about 9% for steel non-composite system.
- Compared to RC systems, finally “complete building cost” for steel structure increases about 15-18% for composite system and about 25-28% for non-composite system when fire proof spray is added.
- Only “total structural cost” increases about 3% for flat plate, about 3-4% for steel CF system and about 13-15% for steel NCF system compared to RC beam-slab system.
- Compared to RC beam-slab system, only “total structural cost” increases about 27-28% for steel CF system and about 42-44% for steel NCF system when fire proof spray is added.
- For the selected building, only structural cost is about 64-67% for steel and RC structural systems. So structural designers should care about economic and safe structural design.

5.2.3 Structural Behavior and Performance

When same building is designed as reinforced concrete and steel structure then structural behavior and performance varies widely. The following conclusions are drawn from comparative study**:**

- Considering lateral drift and vertical deflections, stiffness of RC beam-slab system is the highest of all. Stiffness of RC flat plate system is the 2nd highest and that of steel non-composite system is the 3rd highest. Stiffness of steel composite system is the lowest of all but within allowable limit.

- Compared to steel structure, base shear due to seismic force is 102-122% more for RC beam-slab system and 109-130% more for RC flat plate system.
- Compared to steel non-composite floor system with deck**:**
  Structural self weight is 125% more for RC beam-slab system and 130% more for RC flat plate system.
  Dead load is 68% more for RC beam-slab system and 71% more for RC flat plate system.
  Foundation load is 40% more for RC beam-slab system and 43% more for RC flat plate system.

5.3 Final Remarks

From the stand point of serviceability, RC beam-slab structural system is the most suitable one. But other three structural systems are also within allowable limit for serviceability criteria.

Steel non-composite system is costlier but steel composite system is slightly (0.2-1.8%) costlier than RC structure. But if effect of construction time duration is considered then RC structure may become relatively costlier than steel composite system which is beyond the scope of this research.

Steel structure which is more ductile and attracts less seismic force (only 43-45% of RC structure) may be considered as better structural system in seismically active zones.

Finally, steel structure with composite floor system which optimizes economy, serviceability, construction time, fire proofing system and seismic performance may be considered as optimum structural system for multistory industrial buildings in Bangladesh.

5.4 Recommendations for future work

Effect of construction time period, maintenance cost and composite steel column may be included for comparative study of steel and RC structures. These were beyond the scope of this research.

Comparative study of steel and RC structure may be conducted for long free span with plate girder, hybrid girder, composite plate girder, composite hybrid girder, deep lattice girder etc. with three way beam system for single and multistory buildings.

Comparative study about economy and serviceability of open-web joist and secondary I-beam may be conducted.

The weight of secondary I-beam is about 42-47% of total structural steel. So economically effective spacing and span length may be determined by comparative study.

Feasibility of pre cast and pre-stressed RC slab supported on steel beam may be studied as composite and non-composite.

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Appendix-A

A.1 Theory of Design of Steel Structure

Different components of steel structure such as column, beam, plate girder, hybrid girder, composite beam, composite girder etc. may be designed as per AISC Specifications. Steel structural members may be designed as non-composite or composite using LRFD method as per AISC Specifications 1993, 2005 and 2010. Welding, connections, anchor bolts, base plates, nut-bolts, stud anchors etc. may also be designed as per AISC Specifications 1993, 2005 and 2010. The design steps and procedures of different steel and composite structural members, base plate with anchor bolts, end plate rigid connection, simple shear connection, continuous connections etc. are stated here in brief which are used in designing steel and composite structural components for the research work.

A.1.1 Design of Steel Structural Components

This topics deals with the design procedure of laterally supported beam, laterally unsupported beam, plate girder, hybrid girder, column with bi-axial moment following AISC LRFD 1993. These design procedures are directly used for design of steel structural members for the research and study.

A.1.1.1 Laterally Supported Beam Design

Given data:

Ultimate moment, M_u

Ultimate shear, V_u

Built-up section.

Yield strength of flange, F_{yf}

Yield strength of web, F_{yw}

Yield strength of welding, F_{EXX}

Lateral supports are adequately stiff and braced.

Residual stress, F_r

Step-1: Section selection: Select a trial section.

Step-2: Calculate section properties:

Calculate,

I_x =Moment of inertia of I-section about strong axis.

$S_x = I_x/c$= Elastic section modulus about strong axis.

Z_x =Plastic modulus about strong axis.

Step-3: Check flange and web local buckling limit state:

For flange:

Calculate,

$K_c = 4/\sqrt{(h/t_w)}$ to be $0.35 \leq k_c \leq 0.763$

$\lambda = b_f/2t_f$

$\lambda_p = 65/\sqrt{F_{yf}}$

$\lambda_r = 141/\sqrt{(F_y - 10)}$ for rolled I-shaped section.

$\lambda_r = 162/\sqrt{\{(F_{yf} - 16.5)/K_c\}}$ for welded I-shaped section.

For web:

Calculate,

$\lambda = h/t_w$

$\lambda_p = 640/\sqrt{F_y}$

$\lambda_r = 970/\sqrt{F_y}$

Step-4: Calculate M_p and M_r:

$M_p = Z_x F_y$

$M_r = (F_y - F_r) S_x$

Step-5: Calculate M_n based on flange local buckling:

When $\lambda_p < \lambda \leq \lambda_r$ then $M_n = M_p - (M_p - M_r)(\lambda - \lambda_p)/(\lambda_r - \lambda_p) \leq M_p$

When $\lambda \leq \lambda_p$ then $M_n = M_p$

So, M_n may be calculated.

Step-6: Calculate M_n based on web local buckling:

When $\lambda_p < \lambda \leq \lambda_r$ then $M_n = M_p - (M_p - M_r)(\lambda - \lambda_p)/(\lambda_r - \lambda_p) \leq M_p$

When $\lambda \leq \lambda_p$ then $M_n = M_p$

So, M_n may be calculated.

Step-7: Calculate $Ø_b M_n$ when compression flange laterally supported:

Take lowest of the two values of M_n from step5 and step 6.

Calculate $Ø_b M_n$ where $Ø_b = 0.9$

Now if, $Ø_b M_n \geq M_u$ then section is satisfactory for bending moment.

Step-8: Design for shear strength requirement without stiffener:

Ultimate shear strength, $Ø_vV_n = Ø_v C_v(0.6 F_{yw} A_w)$

Calculate,

h/t_w

$418/\sqrt{F_{yw}}$

$523/\sqrt{F_{yw}}$

(i) If $h/t_w \leq 418/\sqrt{F_{yw}}$, then $C_v= 1$

(ii) If $418/\sqrt{F_{yw}} < h/t_w \leq 523/\sqrt{F_{yw}}$, then $C_v=418/\{(h/t_w)\sqrt{F_{yw}}\}$

(iii) If $h/t_w > 523/\sqrt{F_{yw}}$, then $C_v=2{,}20{,}000/\{(h/t_w)^2F_{yw}\}$

So, calculate value of C_v regarding above conditions and then calculate $Ø_vV_n$.

Now if, $Ø_VV_n \geq V_u$ then section is satisfactory for shear without stiffener.

Step-9: Strength check for combined bending and shear:

When $M_u/(Ø_bM_n)+0.625\{V_u/(Ø_vV_n)\} \leq 1.375$ then section is satisfactory without stiffener.

Step-10: Welding design for flange to web connection:

Assume, process of welding is SMAW process.

Select minimum weld size "a_{min}"

Calculate $a_{max.eff.} =(0.707F_ut_1)/F_{EXX}$

Shear flow = $(V_uQ)/I_x$

Equating the strength of the fillet to shear flow, $Ø2a(0.707)(0.6F_{EXX})= (V_uQ)/I_x$; which gives required fillet weld size "a" to be $\leq a_{max.eff.}$

Now decide weld size, electrode specification and recommend to provide.

Step-11: Pre-composite deflection check:

Calculate pre-composite deflection from structural analysis by computer soft-ware.

Deflection due to concrete plus self weight to be ≤ L/360 or 1 inch (use unfactored load).

Step-12: Composite deflection check:

Calculate composite deflection from structural analysis by computer soft-ware.

Deflection due to unfactored live load to be ≤ L/360 or 1 inch (considering 50% unfactored live load).

A.1.1.2 Laterally Unsupported Beam Design

Given data:

Ultimate moment, M_u

Ultimate shear, V_u

Unsupported length of compression flange KL_b

Built-up section.

Yield strength of flange, F_{yf}

Yield strength of web, F_{yw}

Yield strength of welding, F_{EXX}

Lateral supports are adequately stiff and braced.

Residual stress, F_r

Modulus of elasticity, E

Moment gradient, C_b

Step-1: Section selection: Select a trial section.

Step-2: Calculate section properties:

Calculate,

A=Gross cross sectional area.

I_x =Moment of inertia of I-section about strong axis.

$S_x= I_x/C$= Elastic section modulus about strong axis.

I_y =Moment of inertia of I-section about weak axis.

$r_y=\sqrt{(I_y/A)}$= Radius of gyration about weak axis.

$J =\sum(bt^3)/3$ = Torsional constant.

$C_w= (I_y h^2)/4$ = Warping constant.

$G= E/\{2(1+\nu)\}$ =Modulus of rigidity.

$X_1= (\pi/S_x)\sqrt{\{(EGJA)/2\}}$

$X_2=(4C_w/I_y)\{S_x/(GJ)\}^2$

Z_x =Plastic modulus about strong axis.

Step-3: Check flange and web local buckling limit state:

For flange:

Calculate,

$K_c= 4/\sqrt{(h/t_w)}$ to be $0.35\leq k_c\leq 0.763$

$\lambda=b_f/2t_f$

$\lambda_p= 65/\sqrt{F_{yf}}$

λ_r=141/√(F_y-10) for rolled I-shaped section.

λ_r=162/√{(F_{yf} -16.5)/K_c} for welded I-shaped section.

For web:

Calculate,

$\lambda=h/t_w$

$\lambda_p= 640/\sqrt{F_y}$

$\lambda_r=970/\sqrt{F_y}$

Step-4: Calculate M_p and M_r:

$M_p=Z_xF_y$

$M_r=(F_y-F_r)S_x$

Step-5: Calculate M_n based on flange local buckling:

When $\lambda_p<\lambda\leq\lambda_r$ then $M_n=M_p-(M_p-M_r)(\lambda-\lambda_p)/(\lambda_r-\lambda_p)\leq M_p$

When $\lambda\leq\lambda_p$ then $M_n=M_p$

So, M_n may be calculated.

Step-6: Calculate M_n based on web local buckling:

When $\lambda_p<\lambda\leq\lambda_r$ then $M_n=M_p-(M_p-M_r)(\lambda-\lambda_p)/(\lambda_r-\lambda_p)\leq M_p$

When $\lambda\leq\lambda_p$ then $M_n=M_p$

So, M_n may be calculated.

Step-7: Calculate M_n from lateral torsional buckling limit state:

Calculate,

$L_P=300\ r_y/\sqrt{F_{yf}}$

$L_r=\{r_yX_1/(F_y-F_r)\}\sqrt{[1+\sqrt{\{1+X_2(F_y-F_r)^2\}}]}$

L_b= unsupported length of beam.

Moment gradient $C_b=(12.5M_{max})/(2.5M_{max}+3M_A+4M_B+3M_C)$

When $L_b\leq L_p$ then $M_n=M_p$

When $L_p<L_b\leq L_r$ then $M_n= C_b\{M_p-(M_p-M_r)(L_b-L_p)/(L_r-L_p)\}\leq M_p$

When $L_b>L_r$ then $M_n=M_{cr,}$ where $M_{cr}\leq C_bM_r\leq M_p$

$M_{cr}=\{(C_bS_xX_1\sqrt{2})/(L_b/r_y)\}\sqrt{[(1+X_1^2X_2)/\{2(L_b/r_y)^2\}]}$

So, M_n may be calculated from above conditions.

Step-8: Calculate $Ø_bM_n$ when compression flange laterally unsupported:

Take lowest of the three values of M_n from step5, 6 and 7.

Calculate $Ø_bM_n$ where $Ø_b$=0.9

Now if, $Ø_bM_n\geq M_u$ then section is satisfactory for bending moment.

Step-9: Design for shear strength requirement without stiffener:

Ultimate shear strength, $Ø_vV_n= Ø_v C_v(0.6 F_{yw} A_w)$

Calculate,

h/t_w

$418/\sqrt{F_{yw}}$

$523/\sqrt{F_{yw}}$

(i) If $h/t_w \leq 418/\sqrt{F_{yw}}$, then $C_v = 1$

(ii) If $418/\sqrt{F_{yw}} < h/t_w \leq 523/\sqrt{F_{yw}}$, then $C_v = 418/\{(h/t_w)\sqrt{F_{yw}}\}$

(iii) If $h/t_w > 523/\sqrt{F_{yw}}$, then $C_v = 2{,}20{,}000/\{(h/t_w)^2 F_{yw}\}$

So, calculate value of C_v regarding above conditions and then calculate $Ø_v V_n$.

Now if, $Ø_V V_n \geq V_u$ then section is satisfactory for shear without stiffener.

Step-10: Strength check for combined bending and shear:

When $M_u/(Ø_b M_n) + 0.625\{V_u/(Ø_v V_n)\} \leq 1.375$ then section is satisfactory without stiffener.

Step-11: Welding design for flange to web connection:

Assume, process of welding is SMAW process.

Select minimum weld size "a_{min}"

Calculate $a_{max.eff.} = (0.707 F_u t_1)/F_{EXX}$

Shear flow = $(V_u Q)/I_x$

Equating the strength of the fillet to shear flow, $Ø2a(0.707)(0.6F_{EXX}) = (V_u Q)/I_x$; which gives required fillet weld size "a" to be $\leq a_{max.eff.}$

Now decide weld size, electrode specification and recommend to provide.

Step-12: Pre-composite deflection check:

Calculate pre-composite deflection from structural analysis by computer soft-ware.

Deflection due to concrete plus self weight to be ≤ L/360 or 1 inch (use unfactored load).

Step-13: Composite deflection check:

Calculate composite deflection from structural analysis by computer soft-ware.

Deflection due to unfactored live load to be ≤ L/360 or 1 inch(considering 50% unfactored live load).

A.1.1.3 Plate Girder and Hybrid Girder Design

Given data:

Ultimate moment, M_u

Ultimate shear, V_u

Unsupported length of compression flange KL_b

Modulus of elasticity, E

Yield strength of flange, F_{yf}

Yield strength of web, F_{yw}

Yield strength of welding, F_{EXX}

Moment gradient, C_b

Lateral supports are adequately stiff and braced.

Step-1: Section selection: Select a trial section. Typically, flange width is 20% to30% of depth.

Step-2: Depth check:

Calculate $\beta_w = h/t_w$

Maximum $h/t_w = 14000/\sqrt{F_{yw}(F_{yw}+16.5)}$ when $a/h > 1.5$

Maximum $h/t_w = 2000/\sqrt{F_{yw}}$ when $a/h \leq 1.5$

Maximum $h/t_w = 970/\sqrt{F_{cr}}$ when $F_{cr} = F_{yf}$ (bend buckling limit state of web).

Step-3: Optimum depth calculation:

$h = \sqrt[3]{3(\text{required } M_n)\beta_w/(2R_{PG}F_{cr})}$ where $F_{cr} = 0.96F_{yf}$ assumed.

Step-4: Determination of moment strength of trial section:

F_{cr} from lateral torsional buckling limit state:

$r_T = \sqrt{I_{fc}/(A_{fc}+A_{wc}/3)}$

$\lambda = L_b/r_T$

$\lambda_p = 300/\sqrt{F_{yf}}$

$\lambda_r = 756/\sqrt{F_{yf}}$

When, $\lambda \leq \lambda_p$, then $F_{cr} = F_{yf}$

When, $\lambda_p < \lambda \leq \lambda_r$, then $F_{cr} = C_b F_{yf}\{1-0.5(\lambda-\lambda_p)/(\lambda_r-\lambda_p)\} \leq F_{yf}$

Moment gradient $C_b = (12.5M_{max})/(2.5M_{max}+3M_A+4M_B+3M_C)$

When $\lambda > \lambda_r$, then $F_{cr} = 286000C_b/(L_b/r_T)^2$

So calculate value of F_{cr}.

F_{cr} from flange local buckling limit state:

$K_c = 4/\sqrt{(h/t_w)}$ to be $0.35 \leq k_c \leq 0.763$

$\lambda = b_f/2t_f$

$\lambda_p = 65/\sqrt{F_{yf}}$

$\lambda_r = 230/\sqrt{(F_{yf}/K_c)}$

When, $\lambda \leq \lambda_p$, then $F_{cr} = F_{yf}$

When, $\lambda_p < \lambda \leq \lambda_r$, then $F_{cr} = F_{yf}\{1-0.5(\lambda-\lambda_p)/(\lambda_r-\lambda_p)\} \leq F_{yf}$

When $\lambda > \lambda_r$, then $F_{cr} = 26200K_c/(b_f/2t_f)^2$

So calculate value of F_{cr}.

Strength reduction from web bend buckling (when $h/t_w > 970/\sqrt{F_{yw}}$**):**

Final value of F_{cr} is the lowest value calculated above from LTB and FLB limit state.

Calculate $\beta_w = h/t_w$

Maximum $h/t_w = 970/\sqrt{F_{cr}}$ when $F_{cr} = F_{yf}$ (bend buckling limit state of web)

$a_r = A_w/A_f$

$R_{PG} = 1 - \{a_r/(1200+300a_r)\}(h/t_w - 970/\sqrt{F_{cr}}) \leq 1$

Final moment strength:

Calculate I_x then S_x

Calculate $Ø_bM_n = Ø_bF_{cr}S_xR_{PG}$

Now if, $Ø_bM_n \geq M_u$ then section is satisfactory for bending moment.

Step-5: Moment strength for hybrid girder:

$m = F_{yw}/F_{yf}$

$R_e = \{12+a_r(3m-m^3)\}/(12+2a_r) \leq 1$

$Ø_bM_n = Ø_bF_{cr}S_xR_{PG}R_e$

Now if, $Ø_bM_n \geq M_u$ then section is satisfactory for bending moment.

Step-6: Design for shear strength requirement:

Check whether intermediate transverse stiffener is required or not:

Ultimate shear strength, $Ø_vV_n = Ø_v C_v(0.6 F_{yw} A_w)$

Calculate,

h/t_w

$418/\sqrt{F_{yw}}$

$523/\sqrt{F_{yw}}$

(i) If $h/t_w \leq 418/\sqrt{F_{yw}}$, then $C_v = 1$

(ii) If $418/\sqrt{F_{yw}} < h/t_w \leq 523/\sqrt{F_{yw}}$, then $C_v = 418/\{(h/t_w)\sqrt{F_{yw}}\}$

(iii) If $h/t_w > 523/\sqrt{F_{yw}}$, then $C_v = 2,20,000/\{(h/t_w)^2F_{yw}\}$

So, calculate value of C_v regarding above conditions and then calculate $Ø_vV_n$.

Now if, $Ø_vV_n \geq V_u$ then section is satisfactory for shear and transverse stiffener is not required.

If, $h/t_w \leq 260$ then transverse stiffener is not required.

Strength check for combined bending and shear:

When $M_u/(Ø_bM_n)+0.625\{V_u/(Ø_vV_n)\} \leq 1.375$ then section is satisfactory, stiffener is not required.

Step-7: Design of transverse stiffener spacing:

A. Without considering tension field action at end panel:

Without moment:

$V_n = C_v(0.6\ F_{yw}\ A_w)$

Calculate required C_v from above equation.

When $C_v \leq 0.8$ then $C_v = 44000K_v/\{(h/t_w)^2 F_{yw}\}$

When $C_v > 0.8$ then $C_v = \{187/(h/t_w)\}\sqrt{(K_v/F_{yw})}$

Calculate required K_v from above calculations.

$K_v = 5 + 5/(a/h)^2$

So calculate value of "a" from above equation.

Check $a/h \leq \{260/(h/t_w)\}^2 \leq 3$; then stiffener spacing "a" is satisfactory.

With moment:

Check whether $M_u/(Ø_b M_n) \leq 0.75$ then combined strength check is not required.

If combined strength check is required then,

$M_u/(Ø_b M_n) + 0.625\{V_u/(Ø_v V_n)\} \leq 1.375$ must be satisfied.

So, required $Ø_v V_n$ to be calculated from above equation.

Select a trial value of stiffener spacing "a".

So, $K_v = 5 + 5/(a/h)^2$

Assume $C_v \leq 0.8$ then $C_v = 44000K_v/\{(h/t_w)^2 F_{yw}\}$

Assume $C_v > 0.8$ then $C_v = \{187/(h/t_w)\}\sqrt{(K_v/F_{yw})}$

So, from above conditions take calculated value of $C_v \leq 1$

So, ultimate shear strength $Ø_v V_n = Ø_v\ C_v(0.6\ F_{yw}\ A_w)$

If calculated $Ø_v V_n \geq$ required $Ø_v V_n$ then stiffener spacing is satisfactory.

Check, if $a/h \leq \{260/(h/t_w)\}^2 \leq 3$; then stiffener spacing "a" satisfies.

B. Considering tension field action from 2nd. panel:

Without moment:

Calculate shear V_{u2} at panel 2 (use this design step for 3rd, 4th,5th etc. internal panels)

Select a trial value of stiffener spacing "a"

So, $K_v = 5 + 5/(a/h)^2$

Assume $C_v \leq 0.8$ then $C_v = 44000K_v/\{(h/t_w)^2 F_{yw}\}$

Assume $C_v > 0.8$ then $C_v = \{187/(h/t_w)\}\sqrt{(K_v/F_{yw})}$

So, take calculated value of $C_v \leq 1$

So, ultimate shear strength $Ø_v V_n = Ø_v(0.6F_{yw}\ A_w)[C_v + (1-C_v)/\{1.15\sqrt{(1+(a/h)^2)}\}]$

If calculated $Ø_v V_n \geq V_u$ then stiffener spacing is satisfactory.

Check, if $a/h \leq \{260/(h/t_w)\}^2 \leq 3$; then stiffener spacing "a" satisfies.

With moment:

Calculate shear V_{u2} at panel 2 (use this design step for 3rd, 4th, 5th etc. internal panels also.)

Calculate ultimate moment M_{u2} at panel 2 (use this design step for 3rd, 4th, 5th etc. internal panels also.)

Check, if $M_{u2}/(Ø_b M_n) \leq 0.75$ then combined strength check is not required.

If combined strength check is required then,

$M_{u2}/(Ø_b M_n)+0.625\{V_{u2}/(Ø_v V_n)\} \leq 1.375$ must be satisfied.

So, required $Ø_v V_n$ to be calculated from above equation.

Select a trial value of stiffener spacing "a".

So, $K_v= 5 +5/(a/h)^2$

Assume $C_v \leq 0.8$ then $C_v=44000K_v/\{(h/t_w)^2 F_{yw}\}$

Assume $C_v > 0.8$ then $C_v=\{187/(h/t_w)\}\sqrt{(K_v/F_{yw})}$

So, from above conditions take calculated value of $C_v \leq 1$

So, ultimate shear strength $Ø_v V_n=Ø_v(0.6F_{yw} A_w)[C_v+(1-C_v)/\{1.15\sqrt{(1+(a/h)^2)}\}]$

If calculated $Ø_v V_n \geq$ required $Ø_v V_n$ then stiffener spacing is satisfactory.

Check, if $a/h \leq \{260/(h/t_w)\}^2 \leq 3$; then stiffener spacing "a" satisfies.

Step-8: Design of transverse stiffener size:

D=2.4 for single plate stiffener and 1 for pair of plate stiffener.

C_v=(take value from step 6 which is calculated without stiffener)

V_u=(take from shear force diagram)

$Ø_v V_n$=(take from step 7 which is calculated and applicable)

a=(take from step 7 which is provided and applicable)

Strength requirement

Strength requirement, $A_{st}=(F_{yw}/F_{yst})\{0.15DA_w(1-C_v)(V_u/Ø_v V_n)-18t_w^2\}$

Stiffness requirement:

$J=2.5/(a/h)^2-2 \geq 0.5$

Required $I_{st} \geq Jat_w^3$

Local buckling:

b_{st}/t_{st} to be $\leq \lambda_r=95/\sqrt{F_{yst}}$ then satisfactory.

Step-9: Connection of intermediate stiffener with web and flange :

Welding connection design to be performed.

Provide $4t_w$-$6t_w$ gap between transverse stiffener and compression flange.

Step-10: Welding design for flange to web connection:

Assume, process of welding is SMAW process.

Select minimum weld size "a_{min}"

Calculate $a_{max.eff.}=(0.707F_u t_1)/F_{EXX}$

Shear flow = $(V_u Q)/I_x$

Equating the strength of the fillet to shear flow, $Ø2a(0.707)(0.6F_{EXX})= (V_u Q)/I_x$; which gives required fillet weld size "a" to be $\leq a_{max.eff.}$

Now decide weld size, electrode specification and recommend to provide.

Step-11: Pre-composite deflection check:

Calculate pre-composite deflection from structural analysis by computer soft-ware.

Deflection due to concrete plus self weight to be ≤ L/360 or 1 inch (use un-factored load).

Step-12: Composite deflection check:

Calculate composite deflection from structural analysis by computer soft-ware.

Deflection due to un-factored live load to be ≤ L/360 or 1 inch(considering 50% un-factored live load).

A.1.1.4 Beam-Column Member Design for Braced Frame

Given data:

Ultimate axial load, $P_u=P_{ux}=P_{uy}$

No translation ultimate moment, $M_{ntx}=M_{2x}$

No translation ultimate moment, $M_{nty}=M_{2y}$

Unsupported length of column, L_x

Unsupported length of column, L_y

Yield strength, F_y

Moment gradient, C_{bx}

Moment gradient, C_{by}

Assumed value of K_x (for braced frame)

Modulus of elasticity, E

Assumed value of K_y (for braced frame)

Residual stress, F_r

Step-1: Calculate K value: Assume K=1 for braced frame as side sway prevented.

Step-2: Section selection: Select a trial section.

Step-3: Calculate section properties:

Calculate,

A_g=Gross cross sectional area.

I_x =Moment of inertia of I-section about strong axis.

$S_x= I_x/C$= Elastic section modulus about strong axis.

$S_y= I_y/C$= Elastic section modulus about weak axis.

I_y= Moment of inertia of I-section about weak axis.

$r_x=\sqrt{(I_x/A_g)}$= Radius of gyration about strong axis.

$r_y=\sqrt{(I_y/A_g)}$= Radius of gyration about weak axis.

$J =\sum(bt^3)/3$= Torsional constant.

$C_w= (I_y h^2)/4$= Warping constant.

$G= E/\{2(1+\nu)\}$=Modulus of rigidity.

$X_1= (\pi/S_x)\sqrt{\{(EGJA)/2\}}$

$X_2=(4C_w/I_y)\{S_x/(GJ)\}^2$

Z_x=Plastic modulus about strong axis.

Z_y=Plastic modulus about weak axis.

Step-4: Calculation for column action:

Calculation of slenderness parameter:

Calculate K_xL_x/r_x and K_yL_y/r_y

Take largest of the above two values for calculation.

Now, slenderness parameter $\lambda_c=(KL/r)\sqrt{\{F_y/(\pi^2E)\}}$

When $\lambda_c \leq 1.5$ then short column and when $\lambda_c > 1.5$ then long column.

Calculation of reduction factor for flange local buckling:

Calculate,

$\lambda_r = 95/\sqrt{F_y}$

$\lambda =b_f/2t_f$

When $\lambda>\lambda_r$ then reduction factor Q_s is applicable and smaller than 1.

$K_c= 4/ \sqrt{(h/t_w)}$ to be $0.35\leq k_c\leq0.763$

When $109/\sqrt{(F_y/K_c)} < \lambda < 200/\sqrt{(F_y/K_c)}$ then,

$Q_s=1.415-0.0038 \lambda \sqrt{(F_y/K_c)}$

Calculation of reduction factor for web local buckling:

For first trial, take value of $F_{cr}\leq F_y$

$\lambda = h/t_w$

$\lambda_r = 253/\sqrt{F_y} = 253/\sqrt{F_{cr}}$

Only if $\lambda > \lambda_r$ then reduction factor Q_a is applicable and smaller than 1.

For first trial assume $Q_a = 1$

So, first trial value of $Q = Q_s Q_a$

Now calculate trial $\lambda_c \sqrt{Q}$

When $\lambda_c \sqrt{Q} \leq 1.5$ then $Ø_c F_{cr} = Ø_c \{0.658^{(Q\lambda c2)}\} Q f_y$ (short column equation)

For $\lambda_c \sqrt{Q} > 1.5$ then $Ø_c F_{cr} = Ø_c (0.877/\lambda_c^2) F_y$ (long column equation)

So, calculate $Ø_c F_{cr} = f$ from above condition.

$b_E = (326/\sqrt{f})[1 - 57.2/\{(h/t_w)\sqrt{f}\}] t_w$

So, $Q_a = A_{eff}/A_{gross}$

So, $Q = Q_s Q_a$

Now 2nd trial with the calculated value of F_{cr} and Q_a may be performed for more accuracy.

Determination of nominal strength as column:

Use calculated value of F_{cr} and Q_a

Now calculate, $Ø_c P_n = Ø_c F_{cr} A_g$

Step-5: Calculation for beam action:

Check flange compactness:

Section to be compact. Calculate**,**

$\lambda = b_f/2t_f$

$\lambda_p = 65/\sqrt{F_y}$

If $\lambda \leq \lambda_p$ then the section is compact.

Check web compactness:

Calculate $P_u/Ø_b P_y$

When $P_u/Ø_b P_y \leq 0.125$ then $\lambda_p = (640/\sqrt{F_y})(1 - 2.75 P_u/Ø_b P_y)$

When $P_u/Ø_b P_y > 0.125$ then $\lambda_p = (191/\sqrt{F_y})(2.33 - P_u/Ø_b P_y) \geq 253/\sqrt{F_y}$

Calculate λ_p from above conditions.

$\lambda = h/t_w$

If $\lambda \leq \lambda_p$ then the section is compact.

Calculate M_{nx} and M_{ny}:

Calculate,

$M_{px} = Z_x F_y$

$M_{rx} = (F_y - F_r) S_x$

L_{Px}=300 $r_y /\sqrt{F_{yf}}$

$L_{rx}=\{r_yX_1/(F_y-F_r)\}\sqrt{[1+\sqrt{\{1+X_2(F_y-F_r)^2\}}]}$

L_{bx}=unsupported length as beam.

When $L_{bx}\leq L_{px}$ then $M_{nx}=M_{px}$

When $L_{px}<L_{bx}\leq L_{rx}$ then $M_{nx}= C_{bx}\{M_{px}-(M_{px}-M_{rx})(L_{bx}-L_{px})/(L_{rx}-L_{px})\}\leq M_{px}$

When $L_{bx}>L_{rx}$ then $M_{nx}=M_{crx},$ where $M_{crx}\leq C_{bx}M_{rx}\leq M_{px}$

$M_{crx}=\{(C_{bx}S_xX_1\sqrt{2})/(L_{bx}/r_y)\}\sqrt{[(1+X_1^2X_2)/\{2(L_{bx}/r_y)^2\}]}$

So M_{nx} may be calculated from above conditions.

Again, $M_{py}=F_yZ_y$

$M_{ny}= M_{py}$ when flange is compact.

Calculate magnified moment M_{ux} and M_{uy} for braced frame:

Compute C_{mx} & C_{my} without transverse loading:

$C_{mx}=0.6-0.4M_{1X}/M_{2X}$

$C_{mY}=0.6-0.4M_{1Y}/M_{2Y}$

For single curvature, M_{1x}/M_{2x} and M_{1y}/M_{2y} is negative and for double curvature, positive.

Now,

$P_{e1x}=(\pi^2EA_g)/(K_xL_x/r_x)^2$

$B_{1x}= C_{mx}/(1-P_{ux}/p_{e1x})$

B_{1x} to be taken always ≥ 1

$P_{e1y}=(\pi^2EA_g)/(K_yL_y/r_y)^2$

$B_{1y}= C_{my}/(1-P_{uy}/p_{e1y})\ \geq 1$

Now calculate,

$M_{ux}= B_{1X}M_{ntx}$

$M_{uy}= B_{1y}M_{nty}$

Step-6: Unity check:

$P_u/Ø_cP_n$

$P_u/Ø_cP_n \geq 0.2$ then use $P_u/Ø_cP_n+ 8/9(M_{ux}/Ø_bM_{nx}+ M_{uy}/Ø_bM_{ny}) \leq 1$

$P_u/Ø_cP_n < 0.2$ then use $P_u/2Ø_cP_n+(M_{ux}/Ø_bM_{nx}+ M_{uy}/Ø_bM_{ny}) \leq 1$

Step-7: Story drift check:

Check using computer analysis.

A.1.2 Design of Composite Beam/Girder as per AISC LRFD 2010

Given data:

Built-up I-section to be used with un-shored construction.

Lateral supports are adequately stiff and braced.

Unsupported length of compression flange $=KL_b$

Yield strength of flange$=F_{yf}$

Yield strength of web$=F_{yw}$

Residual stress$=F_r$

Modulus of elasticity $=E_s$

Moment gradient$=C_b$

Tensile strength of plate$=F_u$

No additional weight for concrete ponding to be considered.

Pre-composite construction live load 25 psf.

Ultimate pre-composite positive moment $=+M_{\mathbf{u}}$

Ultimate pre-composite negative moment $=-M_{\mathbf{u}}$

Ultimate composite positive moment$=+M_{\mathbf{u}}$ **(com)**

Ultimate composite negative moment$= -M_{\mathbf{u}}$ **(com)**

Ultimate shear$=V_u$

Length of beam=L

Spacing of beam at left$=s_1$

Spacing of beam at right$=s_2$

For exterior beam, center to slab edge distance.

For ASTM A108 stud anchor, F_u

Slab reinforcement in positive zone$=A_{\mathbf{s}}'$

Slab reinforcement in negative zone $=A_{\mathbf{sr}}$

Yield strength of reinforcement $=F_{yr}$

Reduction factor for void of slab up to rib level.

Step-1: Applied loads: AISC design guide 3 recommends an additional 10% of the nominal slab weight to be applied for concrete ponding. For pre-composite construction, live load 25 psf. to be applied for concrete transport and placement by hose as per ASCE 2002.

Step-2: Check composite deck and anchor requirements:

Deck perpendicular/parallel to beam.

ASTM A108 stud anchor's tensile strength, F_u=65 ksi.

Use steel headed stud anchors ¾ inch or less in diameter.

Concrete strength to be, 3ksi ≤ f_c' ≤10 ksi.

Steel headed stud anchors, after installation, shall extend not less than 1.5 inch above the top of the steel deck.

Minimum stud anchor length to be equal to (rib height + 1.5")

Minimum length of stud anchors = $4d_{sa}$ where, d_{sa}= stud anchor diameter.

There shall be at least ½ inch of specified concrete cover above the headed stud anchor.

Burn off length of stud anchor may be used 3/8".

Steel headed stud anchor diameter to be smaller than or equal to 2.5 times flange thickness of beam.

Rib height of steel decking must not be greater than 3 inch.

Rib width of steel decking must be greater than or equal to 2 inch.

Steel deck shall be anchored to all supporting members at a spacing not to exceed 18 in.

Step-3: Design for pre-composite condition:

It is assumed that deck perpendicular to beam provide adequate bracing for compression flange during construction. So lateral torsional buckling is prevented.

Assume a compact section.

Check compactness (AISC 2010):

For flange: $\lambda=b_f/2t_f$, $\lambda_p= 0.38\sqrt{(E/F_y)}$

Check if $\lambda\leq\lambda_p$ then the section is compact for flange.

For web: $\lambda=h/t_w$, $\lambda_p= 3.76\sqrt{(E/F_y)}$

Check if $\lambda\leq\lambda_p$ then the section is compact for web.

So, pre-composite positive moment, $M_u = \O_b M_n=\O_b F_y Z_x$

Calculate Z_x . Now select a section so that Z_x exceeds this value.

When deck parallel to beam/girder then secondary beam position may be considered as braced location for pre-composite condition. Then lateral tortional buckling limit state must be investigated.

Step-4: Pre-composite deflection:

Calculate pre-composite deflection from structural analysis by computer soft-ware.

Deflection due to concrete plus self weight to be ≤ L/360 or 1 inch (use un-factored load).

If deflection does not satisfy then increase member size or induce camber into the member.

Use 80% of calculated deflection value as chamber.

Step-5: Design for composite condition:

Determine effective width:

For interior beam:

$b=L/4$

$b=(S_1+S_2)/2$

Selected 'b'= smaller of the above two value.

For exterior beam:

$b=L/8$ +slab edge distance.

$b=S_1/2$ + slab edge distance.

Selected 'b'= smaller of the above two value.

Step-6: Flexural strength calculation for composite action (plastic stress distribution):

A. When PNA within slab:

Concrete crushing, $C_{max}=0.85f_c'A_c$, here take effective area of concrete section whether deck perpendicular/parallel to beam.

Steel yielding, $T_{max}=A_sF_y$

$a= A_sF_y/(0.85f_c'b)$ to be $\leq t_{s,eff.}$

Effective slab thickness, $\mathbf{t_{s,eff.}}$ $=t_s-h_r$=total slab thickness-rib height of deck (for parallel deck, effective slab thickness=total slab thickness-rib height×void ratio may be used)

Now calculate moment,

$M_n = T(d/2 + t_s -a/2)$

$\text{Ø}_bM_n =0.85M_n$

Now if, $\text{Ø}_bM_n \geq$ composite positive moment, $+M_u$(com) then section is satisfactory for bending moment.

B. When PNA within steel section:

Here, $C=\sum Q_n$

(i) When PNA is inside flange:

So, $x=(A_sF_y-C)/(2b_fF_y)$; if x is smaller than or equal to flange thickness then PNA is inside flange.

If x is greater than flange thickness then PNA is inside web.

(ii)When PNA is inside web:

Calculate, $C_f=t_fb_fF_y$

So, $x=(A_sF_y-C-2C_f)/(2t_wF_y)$

Calculate centroidal distance y_1 of portion of steel beam in tension measured from bottom of steel section.

Now, $\sum Q_n=C_c=0.85f_c'ba$

So, $a = \sum Q_n / 0.85 f_c' b$

Calculation of M_n (method-1):

When PNA within flange:

$M_{n1} = \sum Q_n (d - y_1 + t_s - a/2)$

$M_{n2} = C_f (d - y_1 - x/2)$

$M_n = M_{n1} + M_{n2}$

$Ø_b M_n = 0.85 M_n$

So, if calculated value $Ø_b M_n \geq$ composite positive moment, $+M_u$(com) then section is satisfactory.

When PNA within web:

$M_{n1} = \sum Q_n (d - y_1 + t_s - a/2)$

$M_{n2} = C_f (d - y_1 - t_f/2)$

$M_{n3} = C_w (d - y_1 - t_f - x/2)$

$M_n = M_{n1} + M_{n2} + M_{n3}$

$Ø_b M_n = 0.85 M_n$

So, if calculated value $Ø_b M_n \geq$ composite positive moment, $+M_u$(com) then section is satisfactory.

Calculation of M_n (method-2):

When PNA within flange:

$M_{n1} = \sum Q_n (t_s - a/2 + x/2)$

$M_{n2} = P_y (d/2 - x/2)$

$M_n = M_{n1} + M_{n2}$

$Ø_b M_n = 0.85 M_n$

So, if calculated value $Ø_b M_n \geq$ composite positive moment, $+M_u$(com) then section is satisfactory.

When PNA within web:

$M_{n1} = \sum Q_n (t_s - a/2 + t_f + x/2)$

$M_{n2} = 2 C_f (t_f/2 + x/2)$

$M_{n3} = P_y (d/2 - t_f - x/2)$

$M_n = M_{n1} + M_{n2} + M_{n3}$

$Ø_b M_n = 0.85 M_n$

So, if calculated value $Ø_b M_n \geq$ composite positive moment, $+M_u$(com) then section is satisfactory.

Step-7: Flexural strength for composite action at support negative moment (plastic stress distribution):

$T_{sr}=A_{sr}F_{yr}$

$C_{max}=A_sF_y$

When C_{max} exceeds T_{max} then PNA is inside the steel section.

When PNA within flange:

$x= T_s/(F_yb_f)$ where $T_s=(C_{max}-T_{sr})/2$

If x is smaller than or equal to flange thickness then PNA is inside flange.

If x is greater than flange thickness then PNA is inside web.

When PNA within web:

$T_f=t_fb_fF_y$

So, $x=(A_sF_y-T_{sr}-2T_f)/(2t_wF_y)$

Locate the centroid y_1 of portion of steel beam in compression measured from bottom of steel section.

Calculation of M_n:

When PNA within flange:

$M_{n1}=T_{sr}(d-y_1+t_s-t_s/2)$

$M_{n2}=T_f(d-y_1-x/2)$

$M_n=M_{n1}+M_{n2}$

$Ø_bM_n =0.85M_n$

So, if calculated value $Ø_bM_n \geq$ composite negative moment, $-M_u$(com) then section is satisfactory.

When PNA within web:

$M_{n1}=T_{sr}(d-y_1+t_s-t_s/2)$

$M_{n2}=T_f(d-y_1-t_f/2)$

$M_{n3}=T_w(d-y_1-t_f -x/2)$

$M_n=M_{n1}+M_{n2}+M_{n3}$

$Ø_bM_n =0.85M_n$

So, if calculated value $Ø_bM_n \geq$ composite negative moment, $-M_u$(com) then section is satisfactory.

Similarly PNA within slab can also be analyzed.

Step-8: Design of stud anchor:

Stud anchor capacity, $Q_n=0.5A_{sa}\sqrt{(f_c' E_c)} \leq R_gR_pA_{sa}F_u$

Calculate cross sectional area of stud anchor, A_{sa} and $E_c=W_c^{1.5}\sqrt{f_c'}$

Number and spacing of stud anchor (for positive moment zone):

$C_{max}=0.85f_c{}'A_c$

$T_{max}=A_sF_y$

Max. spacing of stud connector=8$t_{s,eff.}$

Minm. spacing of stud connector=6d_{sa}

Minm. transverse spacing between anchor pairs=4d_{sa}

Minm. Dist. of anchor to slab free edge along shear force=8 inch

Max. spacing of deck attachment=18 inch

Deck flute spacing=2W_r

$\sum Q_n$=(reduction factor){min^m(C_{max} , T_{max})}

Reduction factor= as required

Number of stud anchor=$\sum Q_n/Q_n$ (between zero & maximum bending moment).

Number and spacing of stud anchor (for negative moment zone):

$V_{nh}= A_{sr}F_{yr}$

Number of stud anchor=V_{nh}/Q_n (between zero & maximum bending moment).

Step-9: Composite deflection check:

Calculate composite deflection from structural analysis by computer soft-ware.

Deflection due to un-factored live load to be ≤ L/360 or 1 inch(considering 50% un-factored live load).

Step-10: Available shear strength:

Beam should be assumed for available shear strength as a bare steel beam.

Step-11: Serviceability:

Vibration might need to be considered. See AISC design guide 11 for additional information.

Special Cases:

Design for pre-composite condition (non compact section): Use laterally supported/ unsupported beam design procedure.

Design for pre-composite condition (plate girder): Use plate girder design procedure.

Design for pre-composite condition (hybrid girder): Use hybrid girder design procedure.

Use elastic stress distribution method for non-compact beam, plate girder and hybrid girder for composite design.

Flexural strength calculation (elastic stress distribution):

Elastic section properties of composite section**:**

Take effective width of slab according to design step 5 as above.

$n=E_s/E_c$

Transformed width of slab (top portion) to be calculated.

Transformed width of slab (rib height portion) to be calculated for deck parallel to beam/girder.

Computation of I_X :

Calculate,

$I_x= \sum I_0 +\sum Ay^2$

$y_1=\sum Ay/\sum A$ (C.G. of composite section above centroid of steel section)

$I_{tr} = I_x - Ay_1^2$

y_t

y_{top}(steel beam)

y_b

S_{conc}

S_{tr}

Computation of stresses for positive moment (ENA with in steel beam only):

Factored pre-composite positive moment= $+M_{up}$

Factored pre-composite negative moment $=-M_{up}$

Factored additional positive moment at composite action$=+M_{uc}$

Factored additional negative moment at composite action$=-M_{uc}$

$f_{bot}= M_{up}/(Ø_b S_x)+ M_{uc}/(Ø_b S_{tr})$

$f_{top}= M_{up}/(Ø_b S_x)+ (M_{uc} y_{top}) /(Ø_b I_{tr})$

$f_{conc}= M_{uc}/(Ø_b n S_{conc})$ (concrete stress)

Computation of stresses may be computed for negative moment also by elastic stress distribution.

A.1.3 Design of Connections for Steel Structure by AISC LRFD method

This topics deals with the design procedure of base plate, extended end plate moment connection, simple shear connection and continuous connection. This design steps are directly followed to design connections of steel and composite structure for the research work.

A.1.3.1 Design of Base Plate and Anchor Bolt as per AISC LRFD 2005

Given data:

Ultimate axial compressive load=P_u

Ultimate moment= M_{ux}

Ultimate moment=M_{uy} (uni-axial moment is considered)

Ultimate base shear=V_u

Pedestal larger than base plate= a

Yield strength of base plate=F_y

Concrete strength=f_c'

Column flange width=b_f

Column total depth=d

Extension of base plate beyond column size=x_1 (minimum 3 inch)

Extension of base plate beyond column size=x_2 (minimum may be 3 inch)

Tensile strength of anchor rod= F_u

Yield strength of anchor rod

Number of anchor rod

Dia. Of anchor rod

Length of anchor rod

Step-1: Determination of trial base plate size:

As per OSHA requirement, base plate size **(N×B)** will be large enough for installation of 4 anchor rods.

Base plate size,

$N \geq d+2x_1$

$B \geq b_f + 2x_2$

Pedestal size,

L= N+2a

W=B+2a

Step-2: Determination of eccentricity e and $e_{critical}$:

$e = M_{ux}/P_u$

$f_{Pmax}=Ø_c(0.85f_c')\sqrt{(A_2/A_1)}$ where $\sqrt{(A_2/A_1)} \leq 2$

$q_{max}=f_{pmax}B$

$e_{critical}=N/2 - P_u/(2q_{max})$ must be greater than or equal to e.

Step-3: Determination of bearing length and bearing pressure check:

Bearing length, Y=N-**2e**

Bearing pressure, $q= P_u/Y$ must be smaller than or equal to q_{max}.

Step-4: Determination of minimum plate thickness:

For strong axis:

m=(N-0.95d)/2

$f_p=P_u/(BY)$

Y must be greater than or equal to **m**

$t_{p,req.}= 1.5m\sqrt{(f_p/F_y)}$

For week axis:

$n=(B-0.8b_f)/2$

$t_{p,req.}= 1.5n\sqrt{(f_p/F_y)}$

So select t_p as the larger of the two value.

Step-5: Determination of anchor rod size:

If no anchor rod force exists then to be decided based on **OSHA** requirement and practical considerations.

Use 4-3/4 inch dia. rods, ASTM F1554, grade 36, rod length 12 inch.

Step-6: Anchor rod shear strength check :

Calculate cross sectional area of anchor rod=A_b

Strength per anchor rod, $ØR_n=Ø(0.4F_u)A_b$ when threads included.

Shear strength for all anchor rods= nos. of anchor rod×$ØR_n$

When shear strength is greater than or equal to V_u then design is satisfactory.

A.1.3.2 Design of 4E Extended End Plate Moment Connection (AISC LRFD 2002)

Given data:

Cyclic/seismic detailing considered /not considered.

Unstiffened/stiffened end plate

Number of bolt at tension zone=4

Yield strength of end plate, F_{yp}

Tensile strength of end plate, F_{up}

Yield strength of column, F_{yc}

Tensile strength of column, F_{uc}

Yield strength of beam, F_{yb}

Tensile strength of beam, F_{ub}

Strength of bolt, F_u^b

Strength of bolt, F_t

Ultimate shear, V_u

Ultimate moment at support, M_u

R_y (R_y=1.1 for F_{yb}=50 ksi and R_y=1.5 for F_{yb}=36 ksi)

Yield strength of stiffener, F_{ys}

Modulus of elasticity E

Connection configuration:

Plate width, b_p (b_f + 1 inch)

Gage distance, g

Internal pitch P_{fi}

External pitch P_{fo}

Vertical edge dist. of outer hole, d_e

Dia. of bolt, d_b

Stiffener width, h_{st} (if reqd.)

Stiffener length, L_{st} (if reqd.)

Stiffener thickness, t_s (if reqd.)

Dia. of hole (d_b+1/16 inch)

Straight part of stiffener=(1" minimum)

Col. flange stiffener, t_s (if req.)

Beam section:

Flange width, b_{fb}

Flange thickness, t_{fb}

Beam depth, d_{beam}

Web thickness, t_{wb}

Web width, h_b

Column section:

Flange width, b_{fc}

Flange thickness, t_{fc}

Beam depth, d_c

Web thickness, t_{wc}

Web width, h_c

Specification of material:

For ASTM A992 steel, F_y=50 ksi and F_u= 65 ksi

For ASTM A572 grade 50 steel, F_y=50 ksi F_u= 65 ksi

For ASTM A490 bolts, F_u=150 ksi, F_t=113 ksi

For ASTM A325 bolts, F_u=120ksi, F_t=90 ksi

Beam side design:

Step-1: Determination of connection design moment:

For seismic provisions:

Calculate Z_{xb} for beam.

Connection design moment, $M_{pc}= 1.1R_yF_{yb}Z_{xb}$ as per AISC seismic provisions 2002, where R_y=1.1 for F_{yb}=50 ksi and R_y=1.5 for F_{yb}=36 ksi)

Location of plastic hinge, L_p =(minimum of $d_b/2$ and $3b_{fb}$) for unstiffened connection.

Moment at face of column, $M_{uc}=M_{pc}+V_uL_P$ (unstiffened connection design moment)

$L_p=L_{st} + t_p$ (for stiffened connection)

Moment at face of column, $M_{uc}=M_{pc}+V_uL_P$ (unstiffened connection design moment)

For low seismic provision:

$\O_bM_P=\O_bZ_{xb} F_{yb}$

M_{uc}=ultimate moment at support to be $\leq \O_bM_P$

Step-2: Calculate connection configurations:

Using selected connection geometric configuration we get,

$h_0= d_{beam} +P_{fo} -t_{fb}/2$

$h_1= d_{beam} -P_{\mathbf{fi}} -1.5t_{fb}$

Step-3: Determine required bolt diameter:

$d_{b,required}=\sqrt{[2M_{uc}/\{\pi \O F_t(h_0+h_1)\}]}$

Step-4: Calculate no prying moment:

Bolt tensile strength, $P_t=F_tA_b$

Prying moment, $ØM_{np}=Ø2P_t(h_0+h_1)$

Step-5: Determine required end plate thickness:

For un-stiffened end plate:

End plate yield line parameter,

$S=1/2\sqrt{(b_pg)}$ if $P_{fi} > S$ then use $P_{fi} = S$

$Y_p=(b_p/2)\{h_1(1/P_{fi} + 1/s) + h_0(1/P_{fo}) - 1/2\} + 2/g\{h_1(P_{fi}+S)\}$

$\mathbf{t_{p,required}} = \sqrt{\{(1.11ØM_{np})/(Ø_bF_{yp}Y_p)\}}$

For stiffened end plate:

$S=1/2\sqrt{(b_pg)}$ if $P_{fi} > S$ then use $P_{fi} = S$

d_e=vertical edge distance for outside bolt holes.

When $d_e < S$ then

$Y_P= (b_p/2)[h_1(1/P_{fi} + 1/s) + h_0\{1/P_{fo} + 1/(2s)\}] + 2/g\{h_1(P_{fi}+S)+h_0(d_e+P_{fo})\}$

When $d_e > S$ then

$Y_P= (b_p/2)\{h_1(1/P_{fi} + 1/s) + h_0(1/P_{fo} + 1/s)\} + 2/g\{h_1(P_{fi}+S)+h_0(s+P_{fo})\}$

$\mathbf{t_{p,required}} = \sqrt{\{(1.11ØM_{np})/(Ø_bF_{yp}Y_p)\}}$

Step-6: Calculate factored beam flange force:

$F_{fu}=M_{uc}/(d_{beam} - t_{fb})$

Step-7: Check shear rupture of extended portion of end plate:

$A_n=\{b_p-2(d_b+1/8)\}t_p$

$ØR_n=0.75(0.6F_{up})A_n$

$F_{fu}/2$ to be smaller than equal to $ØR_n$

Step-8: Check shear yielding of extended portion of end plate:

$ØR_n=0.9(0.6F_{yp})b_pt_p$

$F_{fu}/2$ to be smaller than equal to $ØR_n$

Step-9: Stiffener design (if required):

$t_{s,required} = t_{wb}(F_{yb}/F_{ys})$

$\lambda_r= 0.56\sqrt{(E/F_{ys})}$

Local buckling check: h_{st}/t_{st} to be smaller than equal to λ_r

Step-10: Check compression bolts shear rupture strength:

Bolt shear rupture strength, $ØR_n=Øn_bF_vA_b=Øn_b(0.4F_u^b)A_b$

V_u to be smaller than or equal to $ØR_n$ and then satisfactory.

Step-11: Check compression bolts bearing/tear out strength:

For end plate:

Calculate bearing strength per bolt=$2.4d_bt_pF_{up}$

Tearing out outer bolt**:** calculate L_c and then tear out strength, $R_{n,inner}$ **=1.2** $\mathbf{L_Ct_pF_{up}}$

Capacity per bolt=smaller of the bearing strength and tearing strength.

By inspection, bearing controls for the inner bolts.

Total strength for 4 bolts={2(capacity per inner bolt) +2(capacity per outer bolt)}, to be $\geq V_u$

For column flange:

Calculate bearing strength per bolt=$2.4d_bt_{fc}F_{uc}$

Total strength for 4 bolts=$4ØR_n$ to be $\geq V_u$

Step-12: Design welds:

Beam flanges to end plate weld connection , beam web to end plate weld connection and weld capacity for applied shear to be designed and checked.

Column side design:

Step-13: Check column flange for flexural yielding:

Calculate, $S=1/2\sqrt{(b_{fc}g)}$

$C=P_{fo} + t_{fb} + P_{fi}$

Calculate, $Y_c= (b_{fc}/2)\{h_1(1/s) +h_0(1/s)\}+ 2/g\{h_1(S+3C/4)+h_0(s+C/4)+C^2/2\}+g/2$

Required unstiffened column flange thickness:

$t_{fc,required} =\sqrt{\{(1.11ØM_{np}/(Ø_bF_{yc}Y_c)\}}$ to be $\geq$ column flange thickness; otherwise add flange stiffeners.

Stiffener:

Assume stiffener plate thickness, t_s

Calculate, $P_{so}=P_{si}=(C-t_s)/2$

For stiffened column flange, calculate $Y_c= (b_{fc}/2)\{h_1(1/s+1/P_{si}) +h_0(1/s+1/P_{so})\}+ 2/g\{h_1(S+ P_{si})+h_0(s+ P_{so})\}$

So reduced column flange thickness due to col. flange stiffener,

$t_{fc,required} =\sqrt{\{(1.11ØM_{np}/(Ø_bF_{yc}Y_c)\}}$ to be $\geq$ column flange thickness; otherwise increase column flange thickness.

Step-14: Calculate strength of un-stiffened column flange to determine stiffener design force:

Calculate $ØM_{cf} = Ø_bF_{yc}Y_ct_{fc}^2$

So, $ØR_n=ØM_{cf}/(d_b-t_{fb})$ if $\geq F_{uf}$ then stiffener is not required.

Step-15: Calculate local web yielding strength:

$ØR_n=ØC_t\,(6k_c+N+2t_p)F_{yc}t_{wc}$ and if $ØR_n \geq F_{uf}$ then stiffener is not required.

Step-16: Calculate web buckling strength:

$ØR_n=\{Ø24t_{wc}^3\sqrt{(Ef_{yc})}\}/h$ and if $ØR_n \geq F_{uf}$ then stiffener is not required.

Step-17: Web crippling strength:

$ØR_n=Ø0.80t_{wc}^2\,[1+3(N/d_c)(t_{wc}/t_{fc})^{1.5}]\sqrt{(E\,F_{yc}t_{fc}/t_{wc})}$ and if $ØR_n \geq F_{uf}$ then stiffener is not required.

Step-18: Determine stiffener design force:

$F_{cu}=F_{fu}$-min $ØR_n$ from above four steps.

Step-19: Stiffener design and panel zone checks: Separate design methods required.

A.1.3.3 Design of Simple Shear Connections (AISC LRFD 1993)

Given data:

Ultimate shear at support, V_u

Ultimate moment at support, M_u=near to zero.

Yield strength of beam, F_{yb}

Tensile strength of beam, F_{ub}

Strength of bolt,F_u^b

Strength of bolt, F_t

Yield strength of clip plate, F_{yp}

Tensile strength of clip plate, F_{up}

Connection configuration

Bearing type connection.

Threads of bolts included in shear plane.

Two or more bolt in a line of force.

Number of bolt, m

Dia. of bolt, d_b

Bolt spacing, s (minimum $3d_b$)

Value of L_c(minimum $1.5d_b$)

Clip plate thickness, t_p

Max. clip plate depth, d_p

Min. clip plate width, b_p

Dia. of bolt hole (d_b+1/16 inch)

Bolt center to web/clip plate end distance

Minimum clip plate depth, $d_{p,min}$

Number of row of bolt=1(usually)

Minimum eccentricity for clip moment, e

Beam section:

Flange width, b_{fb}

Flange thickness,t_{fb}

Beam depth, d_{beam}

Web thickness, t_{wb}

Column section:

Flange width, b_{fc}

Flange thickness,t_{fc}

Beam depth, d_c

Web thickness, t_{wc}

Specification of material:

For ASTM A992 steel, F_y=50 ksi and F_u= 65 ksi

For ASTM A572 grade 50 steel, F_y=50 ksi F_u= 65 ksi

For ASTM A490 bolts, F_u=150 ksi, F_t=113 ksi

For ASTM A325 bolts, F_u=120ksi, F_t=90 ksi

Design for shear:

Step-1:Connection design with beam web:

Number of bolt regarding beam web:

Check bolt spacing ,S to be $\geq 3d_b$

External bolt center to plate edge distance, L_c to be $\geq 1.5d_b$

Assume two or more bolt in a line of force.

Assume threads of bolts included in shear plane.

Now calculate, $ØR_n$ (bearing)=$Ø(2.4F_{ub})d_b t_{wb}$

$ØR_n$ (single shear)=$Ø(0.4F_u^b)A_b$

So design strength per bolt is the minimum of the two values obtained from bearing and single shear.

No of bolt required, m =V_u /design strength per bolt.

Check block shear on beam web:

Check, $F_u A_{nt} \geq 0.6F_u A_{nv}$ or not.

Calculate, A_{gv}, A_{nv}, A_{gt}, A_{nt} and then

F_uA_{nt} and $0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_uA_{nv}+F_yA_{gt})$ when $F_uA_{nt} < 0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_yA_{gv}+F_uA_{nt})$ when $F_uA_{nt} \geq 0.6F_uA_{nv}$

Now, calculate applicable $ØT_n$ and compare with ultimate shear V_u

Step-2: Clip plate design:

$mØR_n$ (bearing)=$mØ(2.4F_{up})d_bt_{clip}$ (plate bearing) must be $\leq V_u$

$Ø(0.6F_{up})A_{nv}$= (plate shear rupture) must be $\leq V_u$

$Ø(0.6F_y)A_{gv}$= (plate gross shear yielding) must be $\leq V_u$

Check block shear on clip plate:

Check, $F_uA_{nt} \geq 0.6F_uA_{nv}$ or not.

Calculate A_{gv}, A_{nv}, A_{gt}, A_{nt} and then

F_uA_{nt} and $0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_uA_{nv}+F_yA_{gt})$ when $F_uA_{nt} < 0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_yA_{gv}+F_uA_{nt})$ when $F_uA_{nt} \geq 0.6F_uA_{nv}$

Now, calculate applicable $ØT_n$ and compare with ultimate shear V_u

Flexural stress at clip plate:

Calculate $M_{\mathbf{u,clip}}$

$ØM_{\mathbf{n,clip}}=ØF_yS_x$ to be greater than equal to $M_{\mathbf{u,clip}}$

Clip local buckling limit state:

Calculate b/t

$95/\sqrt{F_y}$ must be greater than b/t

Step-3: Connection design of clip with girder web:

Welding design for clip with girder web and flange connection:

Assume ,process of welding is SMAW process.

Select minimum weld size "a_{min}"

Calculate $a_{max.eff.}=(0.707F_ut_1)/F_{EXX}$

Shear flow = V_u/A_{clip}

Equating the strength of the fillet to shear flow, $Ø2a(0.707)(0.6F_{EXX})=V_u/A_{clip}$;

which gives required fillet weld size "a" to be $\leq a_{max.eff.}$

Now decide weld size, electrode specification and recommend to provide.

Clip may be welded with flange of girder.

Combined stress check at welding to be performed.

A.1.3.4 Continuous Beam to Beam and Beam to Column Web Connection Design (AISC LRFD 1993)

Given data:

Ultimate shear at support, V_u

Ultimate moment at support, M_u

Yield strength of beam, F_{yb}

Tensile strength of beam, F_{ub}

Strength of clip bolt, F_u^b

Strength of clip bolt, F_t

Yield st. of clip plate, F_{yp}

Tensile strength of clip plate, F_{up}

Yield strength of cover plate, F_{yc}

Tensile strength of cover plate, F_{uc}

Strength of cover pl. bolt, F_u^b

Strength of cover plate bolt,F_t

Connection configuration for shear

Bearing type connection.

Threads of bolts included in shear plane.

Two or more bolt in a line of force.

Number of bolt, m

Dia. of bolt, d_b

Bolt spacing, s (minimum3d_b)

Value of L_c (minimum 1.5d_b)

Clip plate thickness, t_p

Max. clip plate depth, d_p

Min. clip plate width, b_p

Dia. of bolt hole (d_b+1/16 inch)

Bolt center to web/clip plate end dist.

Minimum clip plate depth, $d_{p,min}$

Number of row of bolt=1 (usually)

Minm. eccentricity for clip moment, e

Connection configuration for moment

Bearing type connection.

Threads of bolts included in shear plane.

Two or more bolt in a line of force.

Number of bolt, m

Dia. of bolt, d_b

Bolt spacing, s (minimum$3d_b$)

Value of L_c(minimum $1.5d_b$)

Cover plate thickness, t_p(equal to t_{fb})

Min. cover plate length, L_p

Min. cover plate width, b_p

Dia. of bolt hole (d_b+1/16 inch)

Bolt center to beam web clear distance to be practicable.

Number of row of bolt = 2 (usually).

Beam section:

Flange width, b_{fb}

Fl. thickness, t_{fb}

Beam depth, d_{beam}

Web thickness, t_{wb}

Column section:

Flange width, b_{fc}

Flange thichness,t_{fc}

Beam depth, d_c

Web thickness, t_{wc}

Specification of material:

For ASTM A992 steel, F_y=50 ksi and F_u= 65 ksi

For ASTM A572 grade 50 steel, F_y=50 ksi F_u= 65 ksi

For ASTM A490 bolts, F_u=150 ksi, F_t= 113 ksi

For ASTM A325 bolts, F_u=120ksi, F_t=90 ksi

Design of clip for shear:

Step-1: Connection design with beam web:

Number of bolt regarding beam web:

Check bolt spacing ,S to be $\geq 3d_b$

External bolt center to plate edge distance, L_c to be $\geq 1.5d_b$

Assume two or more bolt in a line of force.

Assume threads of bolts included in shear plane.

Now calculate, $ØR_n$ (bearing)=$Ø(2.4F_{ub})d_bt_{wb}$

$ØR_n$ (single shear)=$Ø(0.4F_u^b)A_b$

So design strength per bolt is the minimum of the two values obtained from bearing and single shear.

No of bolt required, m =V_u /design strength per bolt

Check block shear on beam web:

Check, $F_uA_{nt} \geq 0.6F_uA_{nv}$ or not.

Calculate, A_{gv} , A_{nv} , A_{gt} , A_{nt} and then

F_uA_{nt} and $0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_uA_{nv}+F_yA_{gt})$ when $F_uA_{nt} < 0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_yA_{gv}+F_uA_{nt})$ when $F_uA_{nt} \geq 0.6F_uA_{nv}$

Now, calculate applicable $ØT_n$ and compare with ultimate shear V_u

Step-2: Clip plate design:

$mØR_n$ (bearing)=$mØ(2.4F_{up})d_bt_{clip}$ (plate bearing) must be $\leq V_u$

$Ø(0.6F_{up})A_{nv}$= (plate shear rupture) must be $\leq V_u$

$Ø(0.6F_y)A_{gv}$= (plate gross shear yielding) must be $\leq V_u$

Check block shear on clip plate:

Check, $F_uA_{nt} \geq 0.6F_uA_{nv}$ or not.

Calculate A_{gv} , A_{nv} , A_{gt} , A_{nt} and then

F_uA_{nt} and $0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_uA_{nv}+F_yA_{gt})$ when $F_uA_{nt} < 0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_yA_{gv}+F_uA_{nt})$ when $F_uA_{nt} \geq 0.6F_uA_{nv}$

Now, calculate applicable $ØT_n$ and compare with ultimate shear V_u

Flexural stress at clip:

Calculate $M_{\mathbf{u,clip}}$

$ØM_{\mathbf{n,clip}}=ØF_yS_x$ to be greater than equal to $M_{\mathbf{u,clip}}$

Clip local buckling limit state:

Calculate b/t

$95/\sqrt{F_y}$ must be ≥ b/t

Step-3: Connection design of clip with girder web:

Welding design for clip with girder web and flange connection:

Assume, process of welding is SMAW process.

Select minm. weld size "a_{min}"

Calculate $a_{max.eff.}=(0.707F_ut_1)/F_{EXX}$

Shear flow = V_u/A_{clip}

Equating the strength of the fillet to shear flow, $Ø2a(0.707)(0.6F_{EXX})=V_u/A_{clip}$;

which gives required fillet weld size "a" to be $\leq a_{max.eff.}$

Now decide weld size,electrode specification and recommend to provide.

Clip may be welded to flange of girder.

Combined stress check at welding to be performed.

Design of cover plate for moment:

Step-1: Design bolt value:

$F_{uf}=M_u/(d_{beam}-t_{fb})$

$mØR_n$ (bearing)$=mØ(2.4F_{uc})d_bt_p$ (plate bearing) must be greater than equal to F_{uf}

$mØR_n$ (single shear)$=mØ(0.4F_u^b)A_b$ (bolts shearing) must be greater than equal toF_{uf}

Step-2: Block shear check for cover and joint plate:

Check, $F_uA_{nt} \geq 0.6F_uA_{nv}$ or not.

Calculate A_{gv}, A_{nv}, A_{gt}, A_{nt} and then

F_uA_{nt} and $0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_uA_{nv}+F_yA_{gt})$ when $F_uA_{nt} < 0.6F_uA_{nv}$

$ØT_n=0.75(0.6F_yA_{gv}+F_uA_{nt})$ when $F_uA_{nt} \geq 0.6F_uA_{nv}$

Now, calculate $ØT_n$ and compare with ultimate shear equal to F_{uf}

$ØT_n$ to be greater than or equal to F_{uf}

Step-3: Welding of joint plate with beam flange:

Use CJP groove weld with E70electrode.

Step-4: Joint plate and cover plate tension strength check:

Yield strength, $Ø_tT_n=Ø_tF_yA_g$ to be greater than or equal to F_{uf}

Fracture strength, $Ø_tT_n=Ø_tF_uA_e$ to be greater than or equaltoF$_{uf}$

Appendix-B

B.1 Design Loads

Both gravity loads and lateral loads are considered to design the selected building for four types of structural system.

B.1.1 Gravity loads

Live load and dead load are gravity loads considered for the design of the building for the intended design.

B.1.1.1 Design Live Loads

Live load considered to perform design work is given in Table B.1. Live loads are considered as per BNBC 1993. Live load reduction factors also considered which is given in Table B.2.

Table B.1 Design live loads

Occupancy or use	Live load (kN/m^2)
Production area	6
Stair case and lobby	5
Toilet block	2
Office block	3
Light ware house block	6
Roof top	2
Movable partition load	1.2
Pre-composite construction live load	1.2

Table B.2 Live load reduction factor used as per BNBC 1993

Structural member	Tributary area (square meter)	Reduction factor
Internal footing and	348	0.75

ground floor column		
External footing and ground floor column	174	0.81
Corner footing and ground floor column	115	0.92
Internal first floor column	290	0.79
External first floor column	145	0.84
Corner first floor column	115	0.92

B.1.1.2 Dead Load Calculations

Dead load was calculated for steel NCF and CF system with and without steel deck. Dead load was also calculated for RC slab-beam and flat plate floor system. The calculated loads have been assigned in STAAD.Pro models.

Dead Load of Steel NCF System (with Steel Deck):

Rib width of steel decking = 150 mm.
Rib height of steel decking=50 mm.
Total slab thickness=100 mm.
Average slab thickness=75 mm.
Self weight of average 75 mm thick slab =1.82 kN/m^2.
Floor finish =1 kN/m^2.
Roof top 75 mm lime concrete weight=1.43 kN/m^2.
Lift machine room floor slab (125 mm slab with steel decking) weight=2.40 kN/m^2.
Toilet block average wall load=3.73 kN/m^2 (calculated).
Lobby, stair and toilet block 100 mm slab without decking=2.40 kN/m^2.
125 mm thick (3.35 meter height) fixed wall load= 8 kN/m.
250 mm thick (3.35 meter height) fixed wall load=16 kN/m.
125 mm thick (3.35 meter height) roof top fixed parapet wall load=2.55 kN/m.
Lift load per column of lift cores=44.50 kN per column (assumed).
Water tank load per column of stair block columns=320 kN per column (calculated).

Dead load also includes self weight of steel beams, columns, girders, bracings, RC grade beams, RC columns etc. which is directly assigned in the structural analysis model. Tentative sections based on preliminary design are used for beams, columns, grade beams etc. and finally revised.

Dead Load of Steel NCF System (without Steel Deck):

Average slab thickness=100 mm.
Same load as considered with deck above except the floor slab self weight will increase 0.60 kN/m^2.

Dead Load of Steel CF System (with Steel Deck):

Same load as considered for steel NCF system with deck above except the self weight of steel frame will decrease slightly which is neglected.

Dead Load of Steel CF System (without Steel Deck):

Same load as considered for steel NCF system without deck above except the self weight of steel frame will decrease slightly which is neglected.

Dead Load of RC Beam-Slab System:

Self weight of 175 mm thick slab =4.19 kN/m^2.
Floor finish = 1 kN/m^2.
Roof top 75 mm lime concrete weight=1.43 kN/m^2.
Lift machine room 150 mm thick floor slab weight=3.60 kN/m^2.
Toilet block average wall load=3.73 kN/m^2 (calculated).
125 mm thick (3.35 meter height) fixed wall load= 8 kN/m.
250 mm thick (3.35 meter height) fixed wall load=16 kN/m.
125 mm thick (1 meter height) roof top fixed parapet wall load=2.55 kN/m.
Lift load per column of lift cores=44.50 kN per column (assumed).
Water tank load per column of stair block columns=320 kN per column (calculated).

Dead load also includes self weight of beams, columns, shear walls, grade beams etc. which is directly assigned in the structural analysis model. Tentative sections based on preliminary design are used for beams, columns, grade beams etc. and finally revised.

Dead Load of RC Flat Plate System:

Self weight of 225 mm thick slab =5.38 kN/m^2.
Floor finish =1 kN/m^2.
Roof top 75 mm lime concrete weight=1.43 kN/m^2.
Lift machine room 150 mm thick floor slab weight=3.60 kN/m^2.
Toilet block average wall load=3.73 kN/m^2 (calculated).
125 mm thick (3.35 meter height) fixed wall load= 8 kN/m.
250 mm thick (3.35 meter height) fixed wall load=16 kN/m.
125 mm thick (1 meter height) roof top fixed parapet wall load=2.55 kN/m.
Lift load per column of lift cores=44.50 kN per column (assumed).
Water tank load per column of stair block columns=320 kN per column (calculated).
Dead load also includes self weight of edge beams, columns, shear walls, grade beams etc. which is directly assigned in the structural analysis model. Tentative sections based on preliminary design are used for edge beams, columns, grade beams etc. and finally revised.

B.1.2 Design Lateral loads

Design lateral loads are wind load and earthquake load. These loads are calculated as per BNBC 1993.

B.1.2.1 Calculation of Wind Load

Wind load is calculated following BNBC 1993. The assumed location of the project is Gazipur. Basic wind speed is 215 kilometer per hour. Exposure category is assumed “A”.

Now sustained wind pressure, $q_z=C_cC_1C_zV_b^2$
Where,
q_z =sustained wind pressure at height z, kN/m2.
C_1= structure importance coefficient (1 for special occupancy structures).
C_c=velocity to pressure conversion coefficient=47.2×10^{-6}.

C_z=combined height and exposure coefficient.

V_b= basic wind speed in km/hour.

Design wind pressure, $P_z=C_GC_pq_z$

Where,

P_z=design wind pressure at height z, kN/m^2

C_G=gust coefficient

C_p=pressure coefficient for structure or components

q_z=sustained wind pressure

Pressure coefficient C_p:

Length of building =92 meter.

Width of building =56.45 meter.

Height of building = 20 meter.

For wind perpendicular to length, h/B= 20/92=0.21 and L/B=56.45/92=0.61

For wind perpendicular to width h/B= 20/56.45=0.354 and L/B=92/56.45=1.63

Now from table 6.2.15 of BNBC 1993 we get by interpolation,

C_p=1.52 when wind perpendicular to building length.

C_p= 1.24 when wind perpendicular to building width.

Design wind pressure:

Now design wind pressures are calculated at different height for wind perpendicular to building length and wind perpendicular to building width as shown in Table B.3 and Table B.4.

Table B.3 Design wind pressure (wind perpendicular to building length)

Height above ground level, z (meter)	C_z	Sustained wind pressure, $q_z=C_cC_1C_zV_b^2$ (kN/m^2)	C_G	Design wind pressure, $P_z=C_GC_pq_z$ (kN/m^2)
0-4.5	0.368	0.802	1.654	2.022
6	0.415	0.905	1.592	2.194

9	0.497	1.084	1.511	2.495
12	0.565	1.232	1.457	2.735
15	0.624	1.361	1.418	2.939
18	0.677	1.477	1.388	3.122
21	0.725	1.581	1.363	3.284
24	0.769	1.677	1.342	3.429
27	0.810	1.767	1.324	3.563

Table B.4 Design wind pressure (wind perpendicular to building width)

Height above ground level, z (meter)	C_z	Sustained wind pressure, $q_z=C_cC_1C_zV_b^2$ (kN/m^2)	C_G	Design wind pressure, $P_z=C_GC_pq_z$ (kN/m^2)
0-4.5	0.368	0.802	1.654	1.645
6	0.415	0.905	1.592	1.785
9	0.497	1.084	1.511	2.030
12	0.565	1.232	1.457	2.226
15	0.624	1.361	1.418	2.392
18	0.677	1.477	1.388	2.541
21	0.725	1.581	1.363	2.673
24	0.769	1.677	1.342	2.791
27	0.810	1.767	1.324	2.900

B.1.2.2 Seismic Load Calculation

Seismic load is calculated following BNBC 1993 for the design of selected four types of structure. Following are the calculations.

Seismic zone=II

Design base shear, $V = \frac{ZIC}{R}W$

Where,

Z =Seismic zone coefficient

=0.15 for zone II

I =Structural importance coefficient

=1 for special occupancy structure

R =Response modification coefficient for structural system

=10 for steel eccentric braced frame

=8 for intermediate moment resisting concrete frame

W = Total seismic dead load

C = Numerical coefficient given by the relation= $\frac{1.25S}{T^{2/3}}$

The value of C need not exceed 2.75.

S =Site coefficient for soil characteristics

=1.2 (considered)

T =Fundamental period of vibration in seconds, of the structure for the direction under consideration.

$=C_t\ (h_n)^{3/4}$

Where,

C_t =0.083 for steel moment resisting frame

=0.073 for reinforced concrete moment resisting frames

h_n=Height in meter above the base to level n

= 20 meter

Now after calculation we get,

T =0.7885 second for steel structure

=0.6935 second for RC structure

C = 1.753 for steel structure

=1.914 for RC structure

Value of C is greater than 2.75 and C/R ratio greater than 0.075.

So after calculation we get,

design base shear for steel structure,

V=0.0263W……………………………………………………………………………….(B.1)

and design base shear for concrete structure,

V=0.03588W……………………………………...(B.2)

A. Seismic Load for Steel NCF System with Steel Deck

Seismic dead load, W is the total dead load of the building (including permanent partitions) plus 25% of floor live load of wire house plus 1.2 kN/m^2 for movable partition load and total weight of permanent equipments.

Typical floor dead load:

4 nos. toilet block fixed partition wall load	=1255 kN.
Warehouse block partition wall load	=370 kN.
Office block partition wall load	=427 kN.
Stair, lift core and exterior fixed wall load	=4680 kN.
Slab and steel frame weight (3.40 kN/m^2)	=16106 kN.
25% storage live load	=422 kN.
Weight of permanent equipments (0.48 kN/m^2)	=1668 kN.
Movable partition (1.2 kN/m^2)	=4172 kN.
So, total typical floor dead load	=**29100** kN.

Roof slab level dead load:

Parapet wall dead load= 605 kN.

Slab level dead load =19509 kN.

Total dead load =**20114** kN.

Dead load at over head water tank level:

Over head water tank and water load	=4857 kN.
Lift load	=712 kN.
Roof top stair room load	=2348 kN.
Lift machine room load	=823 kN.
Total load	=**8740** kN.

Summary of seismic dead load:

Now a summary of calculated seismic dead load is prepared and given in Table B.5 which is helpful in calculating base shear.

Table B.5 Seismic dead load of steel NCF system with steel deck

Height from ground level	Slab level	Seismic dead load
h_1=3.35 meter	1st slab level	W_1=29100 kN
h_2=6.70 meter	2nd slab level	W_2=29100 kN
h_3=10.05 meter	3rd slab level	W_3=29100 kN
h_4=13.40 meter	4th slab level	W_4=29100 kN
h_5=16.75 meter	5th slab level	W_5=29100 kN
h_6=20.10 meter	6th slab level	W_6=20114 kN
h_7=23.45 meter	water tank level	W_7=8740 kN
So, total seismic dead load, W=**174354** kN		

Total base shear:

Now putting value of W in equation B.1 we get, total base shear,

V= 0.0263W=0.0263×174354= 4585 kN.

Vertical distribution of total base shear:

The concentrated force F_t acting at the top of the building,

F_t=0.07TV < 0.25V when T > 0.7 second.

F_t=0 when T< 0.7 second.

Here T=0.7885 second >0.7 second.

So, F_t= 0.07×0.7885×4585=253 kN.

The remaining portion of the base shear ($V-F_t$=4332kN) shall be distributed over the height of the building according to the relation, $F_x = \frac{(V-F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i}$

After calculation we get,

Story force at first slab level, F_1= 205 kN.

Story force at second slab level, F_2= 409 kN.

Story force at third slab level, F_3=614 kN.

Story force at forth slab level, F_4= 818 kN.

Story force at fifth slab level, F_5=1023 kN.

Story force at sixth slab level, F_6+F_t=845+253=1098 kN.

Story force at water tank level, F_7=450 kN.

Horizontal distribution of story shear:

Story forces to be distributed to the various elements of the vertical lateral force resisting system in proportion to their rigidities considering the rigidity of the floor or roof diaphragm. Here story shear is equally distributed to all beam-column connections at that story level for simplicity (total 50 nos. connections at typical floor level and 36 nos. connections at water tank level) and given in Table B.6 after calculation.

Table B.6 Seismic nodal load for steel NCF with steel deck

Location of horizontal point load	Load per column node (kN)
At 1st slab level	1.75
At 2nd slab level	3.50
At 3rd slab level	5.25
At 4th slab level	7.00
At 5th slab level	8.76
At 6th slab level	9.43
At water tank level	12.45

B. Seismic Load for Steel NCF system without Steel Deck

Same dead load as considered with deck above except the floor slab self weight will increase 0.6 kN/m^2. So seismic dead load will increase.

Summary of seismic dead load:

A summary of seismic dead load is prepared and given in Table B.7 which is helpful in calculating base shear.

Table B.7 Seismic dead load of steel NCF without steel deck

Height from ground level	Floor level	Seismic dead load
h_1=3.35 meter	1st slab level	W_1=31937 kN
h_2=6.70 meter	2nd slab level	W_2=31937 kN
h_3=10.05 meter	3rd slab level	W_3=31937 kN
h_4=13.40 meter	4th slab level	W_4=31937 kN
h_5=16.75 meter	5th slab level	W_5=31937 kN
h_6=20.10 meter	6th slab level	W_6=22952 kN
h_7=23.45 meter	water tank level	W_7=8740 kN
So, total seismic dead load, W=**191377** kN		

Total base shear:

Now putting value of W in equation B.1 we get, total base shear,

V= 0.0263W=0.0263×191377 = 5033 kN.

Vertical distribution of total base shear:

Here T=0.7885 second >0.7 second.

So, F_t=0.07TV= 0.07×0.7885×5033=278 kN.

After calculation we get,

Story force at first slab level, F_1= 222 kN.

Story force at second slab level, F_2= 445 kN.

Story force at third slab level, F_3=672 kN.

Story force at forth slab level, F_4=894 kN.

Story force at fifth slab level, F_5=116 kN.

Story force at sixth slab level, F_6+F_t=961+278=1239 kN.

Story force at water tank level, F_7=449 kN.

Horizontal distribution of story shear:

Horizontal seismic nodal load is calculated and given in Table B.8 which is assigned to STAAD.Pro model for analysis.

Table B.8 Seismic nodal load for steel NCF without steel deck

Location of horizontal point load	Load per column node (kN)
At 1st slab level	1.87
At 2nd slab level	3.82
At 3rd slab level	5.70
At 4th slab level	7.60
At 5th slab level	9.51
At 6th slab level	10.58
At water tank level	12.45

C. Seismic Load for Steel CF System with Steel Deck

Same dead load as considered for steel NCF with deck above except the self weight of steel frame decreases slightly which is neglected. So seismic load is considered same with negligible error.

D. Seismic Load for Steel CF System without Steel Deck

Same dead load as considered for steel NCF without decking above except the self weight of steel frame decreases slightly which is neglected. So seismic load is considered same with negligible error.

E. Seismic Load for RC Beam-Slab System

Compared to steel NCF system with decking, self weight of slab increases 2.40 kN/m^2. Due to self weight of RC beam and column, average dead load also increases 1.43 kN/m^2. So seismic dead load increases.

Summary of seismic dead load:

A summary of seismic dead load is prepared and given in Table B.9 which is helpful is calculating base shear for RC beam-slab structure.

Table B.9 Seismic dead load of RC beam-slab structure

Height from ground level	Floor level	Seismic dead load
h_1=3.35 meter	1st slab level	W_1=47247 kN
h_2=6.70 meter	2nd slab level	W_2=47247 kN
h_3=10.05 meter	3rd slab level	W_3=47247 kN
h_4=13.40 meter	4th slab level	W_4=47247 kN
h_5=16.75 meter	5th slab level	W_5=47247 kN
h_6=20.10 meter	6th slab level	W_6=38262 kN
h_7=23.45 meter	water tank level	W_7=8950 kN
So, total seismic dead load, W=**283447** kN		

Total base shear:

Now putting value of W in equation B.2 we get, total base shear,

V=0.03588W =0.03588×283447=10170 kN.

Vertical distribution of total base shear:

Here T=0.6935 second <0.7 second.

So, F_t=0

After calculation we get,

Story force at first slab level, F_1= 480 kN.

Story force at second slab level, F_2=956 kN.

Story force at third slab level, F_3=1436 kN.

Story force at forth slab level, F_4=1921 kN.

Story force at fifth slab level, F_5=2393 kN.

Story force at sixth slab level, F_6=2326 kN.

Story force at water tank level, F_7=667 kN.

Horizontal distribution of story shear:

Horizontal seismic nodal load is calculated and given in Table B.10 which are assigned to STAAD.Pro model for analysis.

Table B.10 Seismic nodal load for RC slab-beam system

Location of horizontal point load	Load per column node (kN)
At 1st slab level	4.00
At 2nd slab level	8.18
At 3rd slab level	12.63
At 4th slab level	16.36
At 5th slab level	20.46
At 6th slab level	19.88
At water tank level	18.50

F. Seismic Load for RC Flat Plate System

Compared to RC beam-slab system, 0.33 kN/m^2 average floor dead load increases. So seismic dead load increases a small amount.

Summary of seismic dead load:

A summary of seismic dead load is prepared and given in Table B.11 which is helpful in calculating base shear for RC flat plate structure.

Table B.11 Seismic dead load of RC flat plate structure

Height from ground level	Floor level	Seismic dead load
h_1=3.35 meter	1stslab level	W_1=48835 kN
h_2=6.70 meter	2nd slab level	W_2=48835 kN
h_3=10.05 meter	3rd slab level	W_3=48835 kN
h_4=13.40 meter	4th slab level	W_4=48835 kN
h_5=16.75 meter	5th slab level	W_5=48835 kN

h_6=20.10 meter	6^{th} slab level	W_6=39850 kN
h_7=23.45 meter	water tank level	W_7=8950 kN
So, total seismic dead load, W=**292975** kN		

Total base shear:

Now putting value of W in equation B.2 we get, total base shear,

V=0.03588W =0.03588×292975=10512 kN.

Vertical distribution of total base shear:

Here T=0.6935second <0.7 second.

So, F_t=0

After calculation we get,

Story force at first slab level, F_1= 494 kN

Story force at second slab level, F_2= 992 kN

Story force at third slab level, F_3=1486 kN

Story force at forth slab level, F_4=1980 kN

Story force at fifth slab level, F_5=2473 kN

Story force at sixth slab level, F_6=2424 kN

Story force at water tank level, F_7=667 kN

Horizontal distribution of story shear:

Horizontal seismic nodal load is calculated and given in Table B.12 which is assigned to STAAD.Pro model for analysis.

Table B.12 Seismic nodal load for RC flat plate system

Location of horizontal point load	Load per column node (kN)
At 1st slab level	4.22
At 2nd slab level	8.45
At 3rd slab level	12.67
At 4th slab level	16.90
At 5th slab level	21.12
At 6th slab level	20.68
At water tank level	18.50

Appendix-C

C.1 Schedule of Item Rates

Schedule of item rates is prepared by analyzing rates with the help of standard procedure and present practice, using the present market rate of materials and labors.

C.1.1 Schedule of Rates for Structural Steel Works

Schedule of item rates for structural steel works is prepared by rate analysis, following PWD item rate analysis procedure and present practice of different structural steel fabrication companies. In this case, the present market rates of materials and labors are used.

Table C.1 Schedule of rates for structural steel

Item no.	Description of item	Unit	Unit rate (BDT)
1	**Built-up I-sections** fabricated at shop from ASTM A572 grade 50 steel plates by SMAW or SAW welding as per AWS D.1.1 Structural Welding Code, brushing, grinding, surface painting with gray oxide, erecting at site, two coats enamel paint, transportation etc. all complete in all respect with necessary joint plates, stiffeners etc.	kg	125
2	Fitting of ASTM A325 or A490 high strength **bolts** with nuts and washers at joints tightening with proper calibrated wrench during erecting.	kg	148
3	Fitting and fixing galvanized ASTM F1554 grade 55 **anchor rods** with necessary site welding and accessories etc. all complete in all respect.	kg	138
4	Fitting and fixing ASTM A653M SS grade 550, Z 180 galvanized cold formed **steel deck** of 0.7 mm thickness and anchored to supporting members not more than 450 mm etc. all complete.	kg	122

5	Fitting and fixing ASTM A108 **stud shear connectors** of dia. 16 mm and 20 mm by welding as per AWS D.1.1 Structural Welding Code with steel beam flange.	kg	130
6	**Decking screw** to fasten steel deck.	kg	706
7	Supplying and spraying **fire proof spray** (25 mm thickness) at surface of structural steel.	sqm	1730

C.1.2 Schedule of Rates for Civil Works

Schedule of item rates for civil works is prepared by rate analysis, following PWD item rate analysis procedure. In this case, the present market rates of materials and labors are used.

Table C.2 Schedule of rates for RC structural works

Item no.	Description of item	Unit	Unit rate (BDT)
1	**Reinforced cement concrete (1:2:3.5)** works using wooden shutter, having minimum compressive strength f'c = 22 Mpa with standard quality cement, best quality Sylhet sand or coarse sand of equivalent F.M. 2.2 and 20 mm down well graded stone chips.	cum	9600
2	**Reinforced cement concrete (1:1.5:3)** works using wooden shutter, having minimum compressive strength f'c = 25 Mpa with standard quality cement, best quality Sylhet sand or coarse sand of equivalent F.M. 2.2 and 20 mm down well graded stone chips.	cum	9778
3	**Wooden shutter** (made of mango wood) making, leveling and fitting.	sqm	425
4	**Reinforcement works** using deformed bar with minimum fy = 400 MPa and tensile strength at least 460 MPa including cost of fabrication, wires etc. all complete.	kg	78

Table C.3 Schedule of rates for other civil works

Item no.	Description of item	Unit	Unit rate (BDT)
1	**Earthwork** in excavation in foundation trenches up to 1.5 m depth and maximum 10 m lead: in medium stiff clayey soil.	cum	441
2	**Extra rate for excavation** of each additional 0.5 meter depth exceeding 1.5 meter.	cum	10
3	**Sand filling** in foundation trenches and plinth with sand having F.M. 0.5 to 0.8 in 150mm layers including leveling, watering and compaction to achieve minimum dry density of 90% with optimum moisture content (Modified proctor test) by ramming each layer up to finished level as per design supplied by the design office only etc. all complete and accepted by the Engineer.	cum	654
4	**Earth filling** in foundation trenches and plinth in 150 mm layer with earth available within 90 m of the building site to achieve minimum dry density of 90% with optimum moisture content (Modified proctor test) including carrying watering, leveling, dressing and compacting to a specified percentage each layer up to finished level etc. all complete and accepted by the Engineer.	cum	96
5	**One layer of brick flat soling** in foundation or in floor with first class or picked jhama bricks including preparation of bed and filling the interstices with local sand, leveling etc. complete and accepted by the Engineer.	sqm	360
6	Supplying and laying of **single layer polythene** sheet weighing one kilogram per 6.5 square meter in floor or any where below cement concrete complete in all respect and accepted by the Engineer.	sqm	27
7	**50 mm concrete (1:3:6)** work under foundation with cement, stone chips and sand (50% Sylhet sand and 50% local coarse sand).	cum	7096
8	150 mm or 6" thick **damp proof course (1:1.5:3)** with Sylhet sand (F.M. 2.2) stone chips and water-proofing admixture/agent	cum	9778

	approved by the Engineer.		
9	**125 mm brick work** with first class bricks in cement sand (F.M. 1.2) mortar (1:5) and making bond with connected walls including necessary scaffolding, raking out joints, cleaning and soaking the bricks for at least 24 hours before use and washing of sand curing at least for 7 days in all floors including cost of water, electricity and other charges etc. all complete and accepted by the Engineer.	sqm	786
10	**Brick works** with first class bricks in cement sand (F.M. of sand 1.2) mortar (1:5) in foundation and plinth, filling the joints/interstices fully with mortar, racking out the joints, cleaning and soaking the bricks at least for 24 hours before use and curing at least for 7 days etc. all complete including cost of water, electricity and other charges and accepted by the Engineer.	cum	6000
11	**Brick works** with first class bricks in cement sand (F.M. of sand 1.2) mortar (1:5) in lift core wall, filling the joints/interstices fully with mortar, racking out the joints, cleaning and soaking the bricks at least for 24 hours before use and curing at least for 7 days etc. all complete including cost of water, electricity and other charges and accepted by the Engineer.	Cum	6185
12	Add for each additional floor up to 5th floor for brick work.	cum	79
13	Supplying, fitting and fixing glazed homogeneous **floor tiles** (local made) with cement sand (F.M. 1.2) mortar (1:4) base and raking out the joints with white cement including cutting and laying the tiles in proper way and finishing with care etc. all complete and accepted by the Engineer.	sqm	1453
14	Supplying, fitting and fixing glazed **wall tiles** 300mm × 300 mm size (local made) with 20 mm thick cement sand (F.M. 1.2) mortar (1:3) base and raking out the joints with white	sqm	1226

	cement including cutting, laying and charge of machine and finishing with care etc. including water, electricity and other charges complete all respect accepted by the Engineer.		
15	Supplying, fitting and fixing homogeneous **bathroom floor tiles** 300mm × 300 mm size (local made) with cement sand (F.M. 1.2) mortar (1:4) base and raking out the joints with white cement including cutting and laying the tiles in proper way and finishing with care etc. all complete and accepted by the Engineer.	sqm	1366
16	**Net cement** finishing works.	sqm	280
17	**Plaster works (**1:5) with best quality local sand and standard cement.	sqm	205
18	**Plastic painting** works two coats.	sqm	183
19	**Enamel painting** works with two coats.	sqm	172
20	**Window grill** works.	sqm	1652
21	**Window** work with 75 mm aluminum sections and 5 mm glass.	sqm	3228
22	**Plastic door** size 750mm×2100 mm.	nos.	5000
23	**Wooden door** work size 900mm×2100 mm.	nos.	25000
24	**Wooden door** work size 1800 mm×2100 mm.	nos.	45000

Table C.4 Schedule of rates for sanitary works

Item no.	Description of item	Unit	Unit rate (BDT)	Item no.	Description of item	Unit	Unit rate (BDT)
1	Basin	nos	2500	6	Towel rail	nos	450
2	Long pan	nos	950	7	Rose shower	nos	850
3	Low down	nos	2500	8	Bib cock	nos	950
4	Mirror	nos	550	9	Angle stop cock	nos	550
5	Soap case	nos	200	10	Pillar cock	nos	200

Appendix-D

Figure D.1 Steel column layout plan

Figure D.2(a) Typical floor steel girder layout plan

Figure D.2(b) Steel girder layout plan over roof level

Figure D.3(a) Typical floor steel secondary beam layout plan

Figure D.3(b) Steel secondary beam layout plan over roof level

Figure D.4 Steel bracing system

Figure D.5 Steel base plate layout plan