MULTILEVEL CAR PARK- OPTIMUM SOLUTION WHILE CONSIDERING COMPOSITE OPTION

By

Kakadia Savan B.
(07MCL006)

DEPARTMENT OF CIVIL ENGINEERING
Ahmedabad 382481
May 2009

MULTILEVEL CAR PARK- OPTIMUM SOLUTION WHILE CONSIDERING COMPOSITE OPTION

Major Project

Submitted in partial fulfillment of the requirements
For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

Kakadia Savan B.
(07MCL006)

Guide
Mr. Jayant Lakhlani

DEPARTMENT OF CIVIL ENGINEERING
Ahmedabad 382481
May 2009

CERTIFICATE

This is to certify that the Major Project entitled "Multilevel Car Parks – Optimum Solution While Adopting composite Option" submitted by Mr. Kakadia Savan B. (07MCL006), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

Mr. Jayant Lakhlani
Guide,
Director,
Lakhlani Associates,
Rajkot.

Dr. P. H. Shah
Professor and Head,
Department of Civil Engineering,
Institute of Technology,
Nirma University,
Ahmedabad.

Dr. K. Kotecha
Director,
Institute of Technology,
Nirma University,
Ahmedabad.

____
Examiner

____
Examiner

____
Date of Examination

ACKNOWLEDGEMENT

At the outset I would like to express gratitude to **Mr. Jayant Lakhlani**, Director, Lakhlani Associates, Rajkot who constantly encouraged me to think beyond the boundaries. They allowed exploring all the possibilities to execute the work and constantly inspire to keep the work in right direction. The discussions carried out with him throughout the entire course played a vital role in clearing my concepts regarding the topic and that helped me a lot during this study.

It gives me great pleasure in expressing thanks to **Prof. G.N. Patel**, Department of Civil Engineering and **Dr. P. H. Shah**, Head, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad who constantly supported and encouraged me during the course work. I am thankful to **Prof. C. H. Shah**, structural consultant for his valuable suggestions to improve quality of my work. I would be grateful to **Dr. P. V. Patel**, Professor, Department of Civil Engineering, Institute of Technology for his kind support in all respect during my study. I am also thankful to **Dr. K. Kotecha**, Director, Institute of Technology for his kind support.

I am very grateful to my co-guide **Mr. Chirag N. Patel**, Lecturer, Department of Civil Engineering, Institute of Technology, Nirma University, for his support and constant reviewing my work. I am also thankful to all the faculty members of Department of Civil Engineering, Nirma University, Ahmedabad who helped during my study.

I am really thankful to my friends and my family to supporting me during the course work. I also vote my thanks to them who helped me directly and indirectly.

Kakadia Savan B.

(07MCL006)

ABSTRACT

The Multilevel car park is a unique type of building. In India, the metropolitan cities have started to build this type of structure to solve a parking problem in congested traffic area. In nearer future, the multilevel car parks become a need of the day.

Present study is carried out with an objective to understand the various forms, geometry and the structural aspect of the multilevel car parks in India. Accordance with various structural system, type of decking system also has been studied.

From literature survey, it is observed that, for the construction of high rise multilevel car park system, the steel-concrete composite option is cost effective solution over the RCC. For deciding the geometry of car park system, the split level type car parks have been considered. It also provides effective and economical solution over the other functional system.

Design component of steel-concrete composite construction like: deck slab, and girder has been carried out using the excel worksheet. Design of deck slab and beams has been done using BS-5950 (Part-4) and Eurocode-4. Analysis has the analysis and design of columns been done using *STAAD.Pro 2006*.

The comparison of moment frame and braced frame has been done. The comparison of Deflections, weights and moments are compared for the above two system.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABBREVIATION NOTATION AND NOMENCLATURE

A_p	Effective area of cross-section
B	Width of supporting steel beam
b_o	Width of profiled at the neutral axis
d_s	Depth of the sheeting
E_s	Modulus of elasticity of steel
e	Distance of centroid above base
e_p	Distance of plastic neutral axis above base
f_{yp}	Yield strength of steel
f_{ck}	Characteristic strength of Concrete
f_{yp}	Yield strength of steel
I_p	Moment of Inertia
M_{pa}	Plastic moment of resistance
T_s	Thickness of the slab
t_p	Design thickness, allowing for zinc
y_p	Partial Safety Factor
y_{vs}	Partial Safety Factor
V_{pa}	Resistance to vertical shear
ρ	Allow a small contribution due to shearing

1. INTRODUCTION

1.1 GENERAL

Nowadays vehicular traffic in the metropolitan cities has been expanding at a very fast rate. It is now poised for greater growth as the country's economy enters take off stage. Many new companies have started manufacturing cars in India to cater market of Indian society. Today's scenario is more and more people can afford to buy cars. This upsurge in vehicles has created a big problem of parking particularly in congested commercial and office localities therefore concept of multilevel car parks has become a need of the day. The multi-storey car park is a exceptional style of building, one in which all elements of the structure are normally exposed to the environment. One must remember that these car parks must be completed quickly and without causing much hindrance to the busy traffic.

As early as 1918, pioneering Chicago began to innovate new architectural designs to keep cars. Holabird and Roche designed a vertically stacked 5-story structure with a spiraling ramp for the Hotel "La Salle".

1.2 TYPES AND ARRANGEMENTS OF PARKING STRUCTURE

1.2.1 Operational Types

- *Automated park facilities*: This form of car parks requires half the volume of a conventional car park to store the same number of cars. This is because these steel-framed car parks do not require access ramps or roadways within the car storage area. The driver parks the cars on a robot trolley within an entrance module. From this point the trolley takes the car to an empty parking space.

- *Self-park*: in the self-park facility, driver drives the car from street to the stalls without any obstruction. For exit and entrance the different stair or lift arrangement provided for the driver.

1.2.2 Material Type

Construction of multilevel car park with different material like:

- Reinforced concrete construction
- Steel-concrete composite construction
- Steel construction

1.2.3 Functional Types

- *Twin-spiral type*: The ramps, situated in opposite corners, are angled to facilitate the movement from the floors to the ramps. Entrances and exits have been provided on separate levels to take advantage of the different elevations.
- *Split-level type or staggered floor type*: The ramp systems feature separated one-way operation, and access is on only one street. Ninety-degree parking is utilized throughout the four floors Fig.1.1.

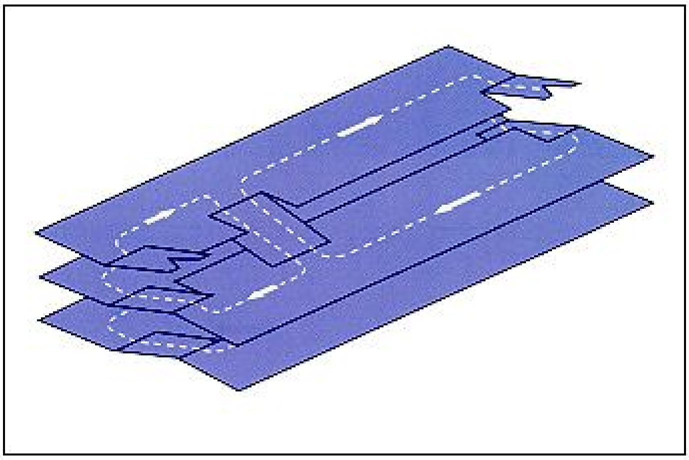


Fig.1.1 Split-level type

- *Straight ramp type*: Straight ramp is provided. A portion of the aisles is used in the floor-to-floor circulation. The widths of the ramps should not be less than 3.65m for a single ramp and 7.0m for a double ramp.
- *Spiral ramp type*: The preparation of a design for an irregular shape site presents many problems, especially when self-parking is to be provided. At that time this type of system can be used.

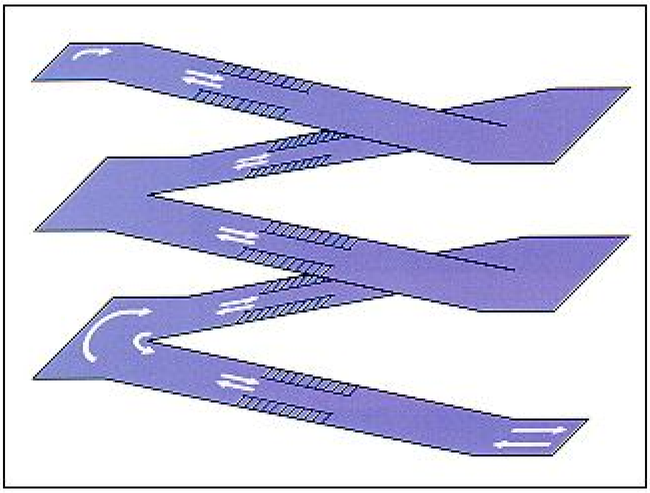


Fig.1.2 Sloping floor

- *Sloping floor or continuous-ramp type*: The aisles serve two purposes: access to the parking stalls, and floor-to-floor circulation Fig.1.2.

1.2.4 Attributes of good car park design

- Easy entry and egress to car park and stalls
- Uncomplicated traffic flow
- Unimpeded movement
- Light and airy
- Low maintenance
- Safe and secure

1.2.5 Design Aspects

- **Flow Patterns**
  - ✓ *One way flow system:* They ensure easier entry and exit to stalls and allow significant flow capacities to be achieved with the self enforcing flow pattern Fig.1.3.
  - ✓ *Two way flow system:* More familiar to the user and if properly designed can achieve a higher flow rate than one way systems. They require marginally more space and are therefore less structurally efficient than one way system. Two-way system is best used with 90^o parking as their use with angled parking can cause confusion while driving Fig.1.4.

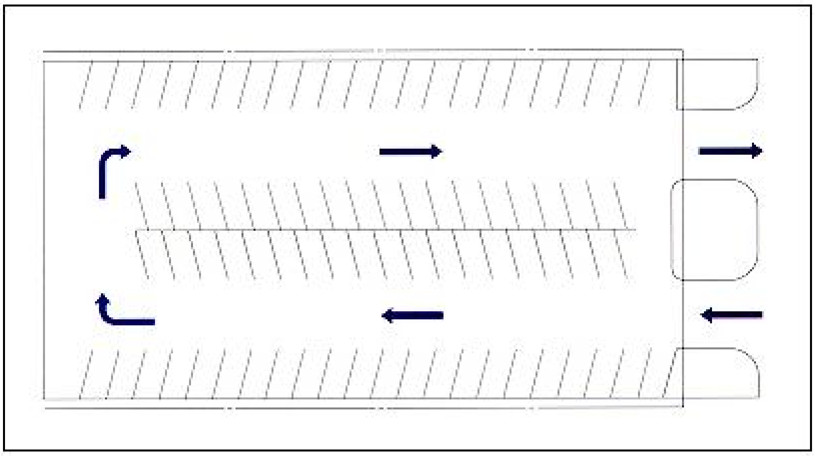


Fig.1.3 One way flow, angled parking

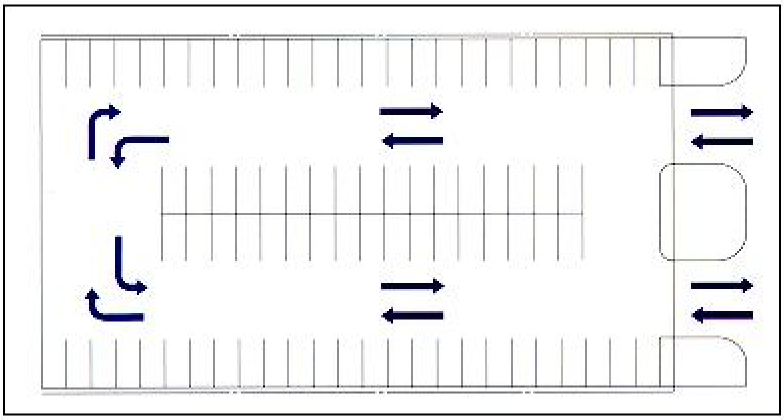


Fig.1.4 Two way flow, 90° parking

Table 1.1 Effect of varying parking angle on parking bin requirements

Parking angle	**Stall width (m)**	**Stall width parallel to Aisle (m)**	**aisle width (m)**	**Bin width (m)**
45°	2.3 2.4 2.5	3.25 3.39 3.54	3.60	13.85 13.71 13.57
60°	2.3 2.4 2.5	2.66 2.77 2.89	4.20	14.93 14.83 14.73
75°	2.3 2.4 2.5	2.38 2.49 2.60	4.98	15.34 15.39 15.45
90°	2.3 2.4 2.5	2.30 2.40 2.50	6.00	15.5

1.3 STRUCTURAL ARRAGEMENTS

- **Column Layout**

The structural design of car park will usually determine its quality as a user friendly structure. It required ease of entry and egress to and from stalls so that users can gain rapid entry and exit without the risk of damage to vehicle or injury to person. The driver should be guided through the park without encountering severe obstructions such as columns in the drive path and badly parked cars caused by insufficient design or layout. Desirable attributes indicated above will only be completely fulfilled if there are no internal columns.

✓ *Ideal column Position:*

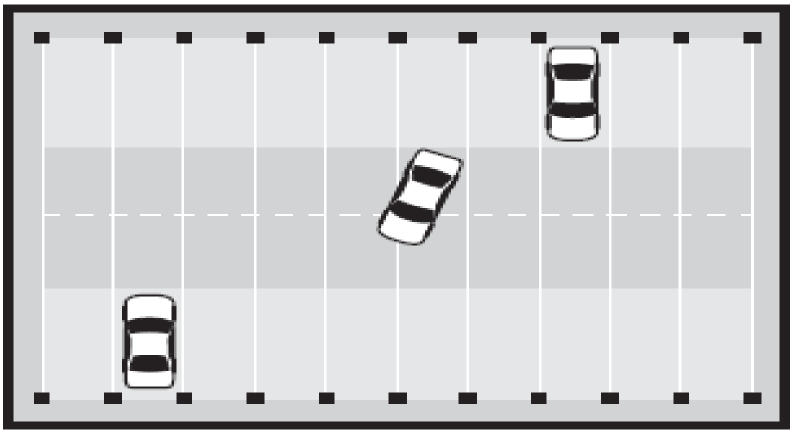


Fig.1.5 No internal columns to impede traffic flow

✓ *Alternative column Position:*

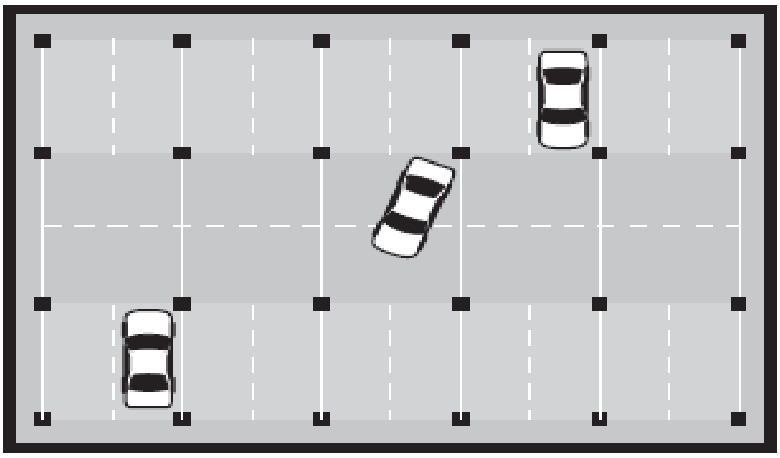


Fig.1.6 Alternative positions of columns impede traffic flow

Table 1. 2 Effect of internal columns on overall width

No. of cars between columns	Column cross centre option (2.4m stall width)	Reduced column cross centre for clear span
2	1800 x 2 + 600 x 3 + 300 = 5.7 m	4.8 m
3	1800 x 3 + 600 x 4 + 300 = 8.1 m	7.2 m
4	1800 x 4 + 600 x 5 + 300 = 10.7 m	9.6 m

1.4 OBJECTIVE OF STUDY

- The main objective of present work is optimum solution using composite construction. Comparison of RCC and steel-composite construction studied in INSDAG publication, as per that the steel-composite construction gives optimum solution over the RCC construction above the three storey parking and also it gives large column free area.
- So here, attempt has been made to achieve optimum solution for multi level car park, while adopting composite construction and making comparison of different types of structural system.

1.5 SCOPE OF WORK

To achieve above objectives the scope of work decided as:

(1) Analysis, design and estimation of

- Split-level type or Staggered floor system

(2) Analysis, design, estimation & Comparison as per structural system (For above functional type)

- Concentrically braced frames
- Moment frames (Fully Restrained)

(3) Geometry configuration

- No. of floors : 5

- Earthquake zone : III
- Wind speed : 44 m/s
- Safe bearing capacity(soil) : 300 kN/m^2
- Importance factor : 1
- Response reduction factor : 3
- Height of each storey : 3.1 m
- Slab thickness : 130 mm
- Width of ramp : 5 m
- Parking space : 2.5m X 5m (per car)
- Codes : IS, Euro Code, BS

1.6 ORGANIZATION OF MAJOR PROJECT

The report of thesis work entitled “Multilevel Car Parks- Optimum Solution While Adopting Composite Option” has been divided into five chapters as follows:-

Chapter-1 includes importance of multilevel car parks with its general aspects structural arrangement as well as functional arrangement. It also includes objectives of study and scope of work.

Chapter-2 includes the literature review. It gives the overview of publications available for the topic. Also it includes overview of various papers and reports which helping in understand the different aspects of topic in depth.

Chapter-3 includes the advantages and the composite action which occur in steel-concrete composite structure. Introduction of different component like deck slab, girder, column and shear connector has been discussed.

In Chapter-4, the basics of lateral resisting system have been discussed. The different types of lateral resisting system are given and also the advantages and disadvantages of system has been discussed.

Chapter-5 deals with the design of parking structure with moment resisting frame system. There are many advantages of composite floor using profile deck system have been presented. General design criteria for analysis have given in this chapter.

Chapter-6 deals with the design of parking structure with braced frame system. It also includes design results of this system.

Chapter-7 includes the comparative study of two system like moment resisting frame system and braced frame system.

Summary and conclusions of major project has discussed in Chapter-8. This chapter also includes future scope of work.

2. LITERATURE SURVEY

2.1 GENERAL

In India, the multilevel car park is a new approach in the building construction. In the past, very little research work has been done on this topic in India. So, it's need to study the literature available and research work has been done on this topic all over the world. In this chapter the review of various papers has been presented related to the analysis and design of the multilevel car parking and also the papers related to the steel-concrete composite structure. This literature survey helped to decide the scope of work, deciding the geometry and the way of doing the work in right direction.

2.2 LITERATURE REVIEW

In the beginning material related to the topic is searched out, collected and compiled through various sources e.g. local libraries, internet, journals, and INSDAG publication etc. many website has been surfed for getting information of work done in related area.

Pydi Lakshmana Rao et al. [1] discussed the requirement of multilevel car parking for the high density area of car. Also they discussed the analysis, design & cost comparisons of 3, 5, and 7 levels car parking with the RCC & steel-concrete composite option. After above study they concluded that the composite construction is economical over the RCC in case of 5 and 7 levels of parking. Composite option can provide large span so that it provides no columns interrupt movement and large column free area. Also in composite option of columns can be reduced by 50% allowing easy maneuvering of vehicles.

PCI [2] this precast prestressed concrete manual's is to show the uniqueness of precast prestressed concrete parking structures and to emphasize areas of special considerations required in the design of these structures. Chapter 1 and 2 explain some of the key considerations an owner/developer must entertain when creating a parking structure. The remaining six chapters are describing the basic knowledge of structural and prestressed concrete design. As well as it also gives the Parking structure durability, functionality, cost considerations, structural design consideration, and connection details etc. They offer a basic explanation of precast's advantages, design options, and techniques to improve functionality.

Steel Tips [3] presents information and tips on the design and construction of steel parking structures including information related to seismic behavior and design of such parking structures. Also it explained the various considerations for steel parking system like: painting guide, fire code requirement, and slab design consideration. Also the different type of frame system and seismic design has been discussed.

This is the third edition of the **Steel framed Car Parks [4]** brochure, prepared by Corus Construction. In this brochure, the various aspects of parking structure are given like: outline, circulation design, structural form, fire resistance, durability, aesthetic design, commercial viability, latest development. Also the some case study of various parking structure is given.

Open deck parking structure by **Emile Troup and John Cross [5]** content information like: advantages of steel framed parking structure; general planning parameters, the guideline for the inspection and its maintenance, the structural and non-structural system design parameters. Also the various aspects of costs, seismic resistance, aesthetics, durability, early occupancy and the efficiency has considered.

Cast in place concrete parking structure [6] this brochure contains case studies of 20 parking structures located throughout the United States. In most of these cases, a post-tensioned structural system was selected after careful economic studies and comparisons indicated initial or life cycle cost savings.

Guideline for the **Design of off-street car parking [7]** explain various aspects of parking structure. The different sizes required for the different ramp system, design guideline, various term define which uses in off-street parking, classification of off-street parking, different types of angled parking, various marking and signage guide to easy access, it also includes the sizes of cars and its wheel where it fits in stall.

INSDAG (*Institute for Steel Development & Growth*) [9] Publish "Hand Book on Composite construction (Multilevel car parking), Literature explain term composite construction, Fundamentals of Composite action, Various Construction Methods, Fundamentals of shear connection, Beam and Column behaviour, Flooring system. Also deals with practical problems and provide solutions. Also result carried out from comparisons of RCC and composite construction.

INSDAG (*Institute for Steel Development & Growth*) [10] Publish "[B+G+20] Storeyed Residential Building with Steel-Concrete Composite Option" in this the study has covered design principle and guidelines, analysis and design of building and cost estimation. Also this study will definitely encourage the builders to go in for steel intensive building for residential usage because using steel-concrete composite technology.

Kober and Dima [11] carried out study on behavior of eccentrically braced frames with short links. The paper is intended to illustrate some features of different bracing systems used for eccentrically braced frames located in seismic areas.

Composite floor system- A cost effective study by **Hedaoo and Athare [16]** present the study of comparison is between RCC and composite floor construction for a G+5 commercial Building. From study, it is found that direct construction cost required for composite floor is higher than RCC floor. But overall the net cost required for composite floor is only 0.45% more than RCC floor considering time related savings. Also the study says that the time required for composite floor construction is half than RCC floor.

2.3 SUMMARY

In this chapter, review of papers, publication has been carried out. In the literature review, the functional as well as structural requirement of multilevel car parking has been studied. This study are useful for moving forward for the fixing the plan geometry, load consideration, framing structure for seismic design consideration, for the functionality and the structural design consideration of the multilevel car parking.

3. COMPOSITE CONSTRUCTION

3.1 GENERAL

The most important and most frequently encountered combination of construction materials is that of steel and concrete with applications in multi-storey commercial buildings and factories, as well as in parking structure. These essentially different materials are completely compatible and complementary to each other; they have almost the same thermal expansion; they have an ideal combination of strengths with the concrete efficient in compression and the steel in tension; concrete also gives corrosion protection and thermal insulation to the steel at elevated temperatures and additionally can restrain slender steel sections from local or lateral-torsional buckling.

In multi-storey buildings, structural steelwork is typically used together with concrete; for example, steel beams with concrete floor slabs. It is a fact, however, that engineers are increasingly designing composite and mixed building systems of structural steel and reinforced concrete to produce more efficient structures when compared to designs using either material alone.

3.2 ADVANTAGES OVER OTHER TYPE OF STRUCTURE

- High ductility of steel material leads to better seismic resistance of the composite section. Steel component can be deformed in a ductile manner without premature failure and can withstand numerous loading cycles before fracture.

- Encased steel beam sections have improved fire resistance and corrosion.

- Steel component has the ability to absorb the energy released due to seismic forces.

- Ability to cover large column free area. This leads to more usable space. Area occupied by the composite column is less than the area occupied by the RCC column.

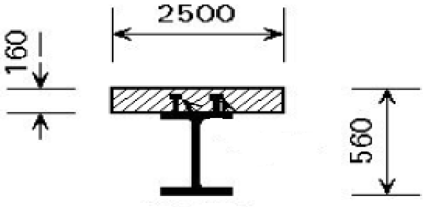
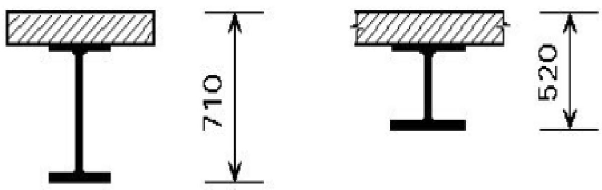
 Composite beam		 Bare steel beam	
Load resistance	100%	100%	100%
Steel weight	100%	160%	215%
Overall height	100%	130%	95%
Stiffness	70%	70%	45%

Fig.3.1 Importance of composite construction

- As the depth of beam reduces, the construction depth reduces, resulting in enhanced headroom.
- Faster construction by utilizing rolled and/or pre-fabricated components. Also, speedy construction facilitates quicker return on the invested capital.
- Quality of steel is assured since it is produced under controlled environment in the factory. Larger use of steel in composite construction compared to that in RCC ensures better quality control for the major part of the structure.
- Steel is more durable, highly recyclable and hence environment friendly.
- Keeping span/loading unaltered, smaller structural steel sections are required compared to non-composite construction. Therefore reduction in overall weight of the composite structure compared to the RCC construction results less foundation costs.
- Cost of formwork is lower compared to RCC construction.
- Cost of handling and transportation is minimised because major part of the structure is fabricated in the workshop.
- The steel and steel concrete composite construction is more resistant against terrorist activities as compared to RCC construction.

- Composite sections have higher stiffness and hence experience less deflection than the non-composite steel sections.
- Reduction in overall weight of structure compared to RCC construction is possible and thereby reducing the foundation cost.

3.3 COMPOSITE ACTION

Composite beams consist of steel sections acting compositely with reinforced concrete slab to connect them together; mechanical shear connectors are provided to transfer the horizontal (longitudinal) shear between the steel beam and concrete slab without consideration of any bond between two materials. The shear connectors are also resist the uplift force acting at the steel concrete interface.

The connection is considered to be complete if the composite beam resisted the bending resistance, not the horizontal shear resistance. Complete or incomplete interaction between the concrete slab and steel section results in a more stiff or less stiff composite beam. Incomplete interaction arises when flexible connectors are used.

Composite beams are often designed under the assumption that the un-propped steel beam supports its own weight and the weight of wet concrete plus construction loads. It may be decided for reasons of economy to provide only sufficient connectors to develop enough composite action to support the loads applied afterwards. This approach results less number of connectors are required for maximum bending resistance of the composite beam.

3.3.1 Degree of Interaction

When no slip occurs between the concrete slab and the supporting steel beam, it is termed as full interaction. It is considered that the complete shear connection has been achieved. In practice some slip is always occur and the term full interaction is used where it is considered that the effects of slip may be neglected in the design.

Partial interaction implies that slip occurs at the interface between the concrete flange and steel beam because in partial shear connection the number of connectors provided is less than required to achieve complete shear connection and it is not permissible as per Indian Standards.

3.3.2 Concept of Development of Shear and Uplift Force

An analysis of simply supported beam made from two member of equal size (rectangular section) placed one above the other shown in Fig.3.2

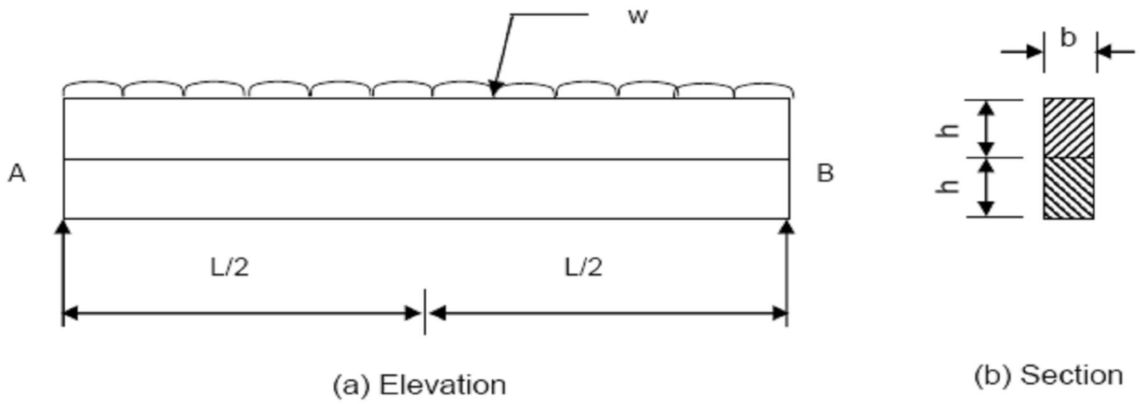


Fig.3.2 Equal rectangular section placed one above the other

Considered two cases one is no shear connection and second is full shear connection. Comparing the stresses and deflection of the two cases as shown in Fig.3.1, we find that for full shear connection though the maximum shear is unchanged, the maximum bending stress is half and the deflection is one-quarter of the no shear connection.

For no shear connection, Max. deflection $= \dfrac{5\,wL^4}{64\,Ebh^3}$

For full shear connection, Max. deflection $= \dfrac{5\,wL^4}{256\,Ebh^3}$

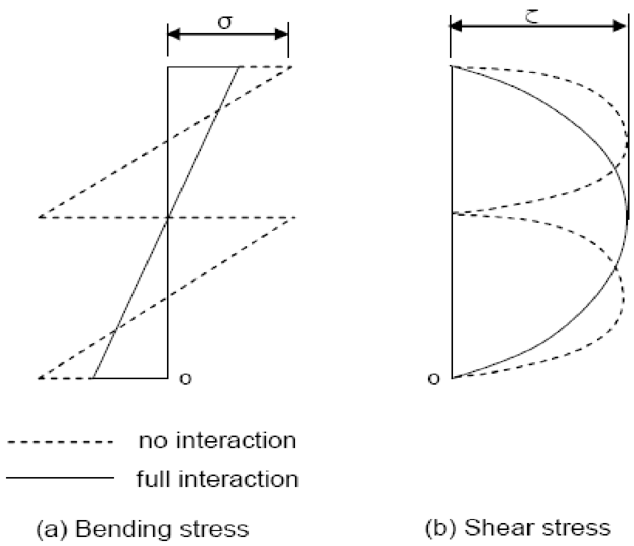


Fig.3.3 Effect of shear connection on bending and shear stresses

- ***Uplift***

Tension across the interface can also occur in beams of partially completed flanges. Fig. 3.4 shown below provides a simple example. Lets us assume that AB is supported on CD and carries distributed loading “w” per unit length. By elastic theory it can be shown that if the flexural rigidity of AB exceeds about one-tenth of that of CD, then the whole of the load on AB is transferred to CD at points A and B only, with separation of the beam between these point.

Generally, uplift forces are so much less compared to shear forces so, it is not necessary to calculate for design purposes.

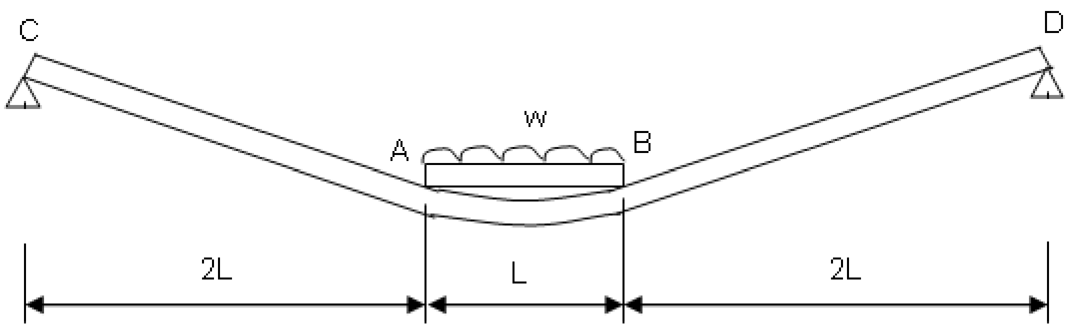


Fig.3.4 Uplift force at mid span of beam

3.4 COMPOSITE FLOOR ELEMENT

Steel–concrete composite floors consist of rolled or built-up structural steel beams and cast in-situ concrete floors connected using shear connectors in such a manner that they would act monolithically. Composite deck slabs are particularly competitive where the concrete floor has to be completed quickly and where medium level of fire protection to steel work is sufficient.

The main structural and other benefits of using composite floor with profiled steel decking are:

- Savings in steel weight are typically 30% to 50% over non-composite construction.
- Greater stiffness of composite beam results in shallower depth for the same span. Hence lower storey heights are adequate resulting in saving cladding costs, also reduction in wind loading and saving in foundation costs.

The steel decking performs a number of roles, such as:

- It supports loads during construction and acts as a working platform and develops adequate composite action with concrete to resist the imposed loading.
- It stabilizes the compression flanges of the beams against lateral buckling, until concrete hardens.
- It reduces the volume of concrete in tension zone & distributes shrinkage strains, thus preventing serious cracking of concrete.

The required composite action can be achieved by various means. Eurocode-4 [22] permits the following methods of achieving shear load:

a) Mechanical interlock provided by deformations in the profile (indentations or embossments).

b) Frictional interlock for profiles shaped in a re-entrant form.

c) End anchorage provided by welded studs or shot fired shear connectors.

d) End anchorages by deformation of the ribs at the end of the sheeting in combination with (b).

The use of profiled steel sheeting undoubtedly speeds up construction. It is also often used with lightweight concrete to reduce the dead load due to floor construction. The composite slabs are supported by steel beams, which normally act compositely with the concrete slab.

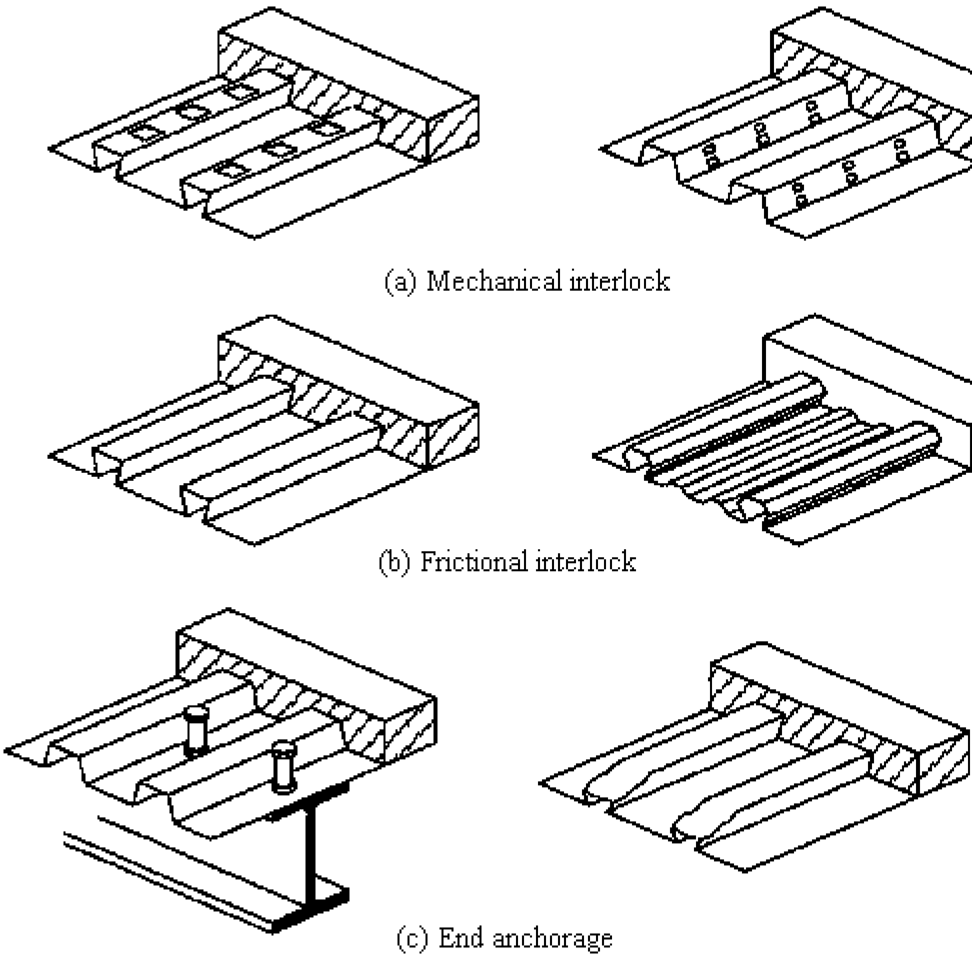


Fig. 3.5 Typical forms of interlock in composite floor

3.5 DESIGN STEPS OF PROFILED DECKING

The following are the steps for design of profiled decking sheets:

(i) List the decking sheet data (Preferably from manufacturer's data)

(ii) List the loading

(iii) Design the profiled sheeting as shuttering

- Calculate the effective length of the span

- Compute factored moments and vertical shear
- Check adequacy for moment
- Check adequacy for vertical shear
- Check deflections

(iv) Design the composite slab: Generally the cross sectional area of the profiled decking that is needed for the construction stage provides more than sufficient reinforcement for the composite slab. So, the design of short span continuous slabs can be done as series of simply supported slabs and top longitudinal reinforcement is provided for cracking. However, long-span slabs are designed as continuous over supports.

- Calculate the effective length of the span
- Compute factored moments and vertical shear
- Check adequacy for moment and vertical shear
- Check adequacy for longitudinal shear

Check for serviceability, i.e. cracking above supports and deflections

3.6 SHEAR CONNECTORS

Shear connectors are essential for steel-concrete composite construction to improve the load carrying capacity as well as overall rigidity. Though steel to concrete bond may help shear transfer between the two to certain extent, yet it is neglected as per the codes because of its uncertainty.

There are three main types of shear connectors, viz. rigid shear connectors, Flexible shear connectors and Anchorage type shear connectors.

3.6.1 Rigid Shear Connector

Rigid shear connector consist of the short length rod, angle or channel welded on the top of the steel beam flange and provide resistance to horizontal shear. Rigid connectors derive their resistance from bearing pressure of the concrete, distributed evenly over the surface because of the stiffness of the connector.

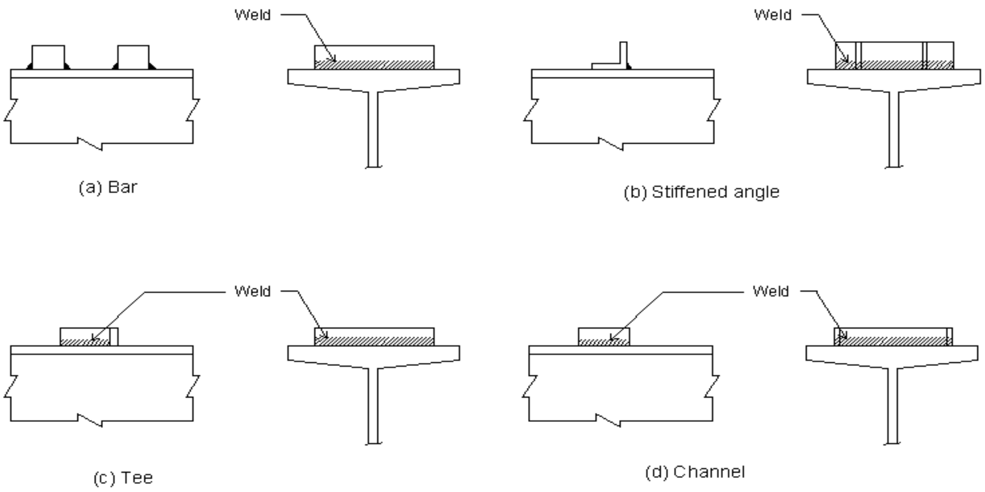


Fig. 3.6 Rigid shear connectors

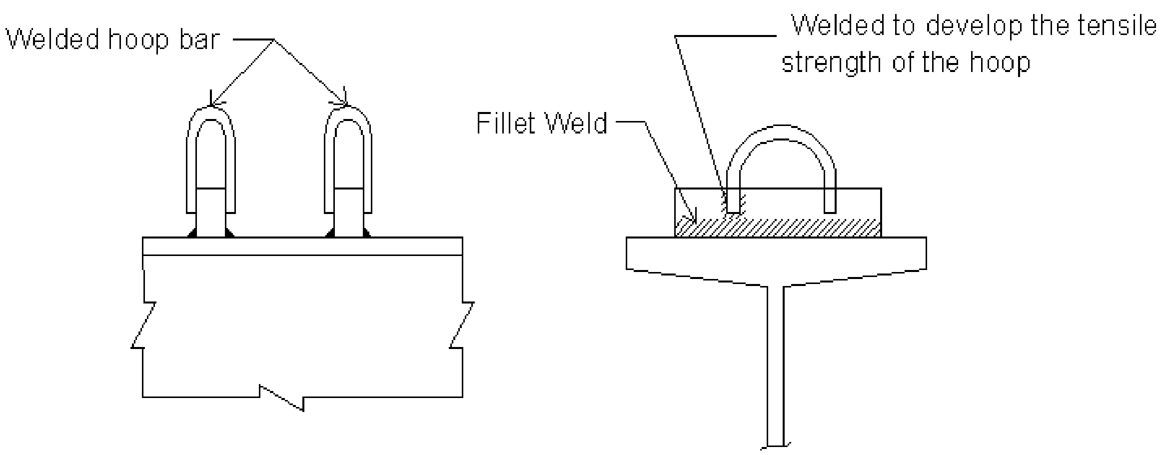


Fig. 3.7 Rigid shear connectors with hoop

3.6.2 Flexible Shear Connectors

Flexible shear connectors consist of the stud; angle, channels or tees welded on the top of the flange of the steel beams and derive the resistance to horizontal shear through bending of the connectors. And normally failure occurs when the yield stress in the connector is exceeded resulting in the slip between the steel beam and concrete slab.

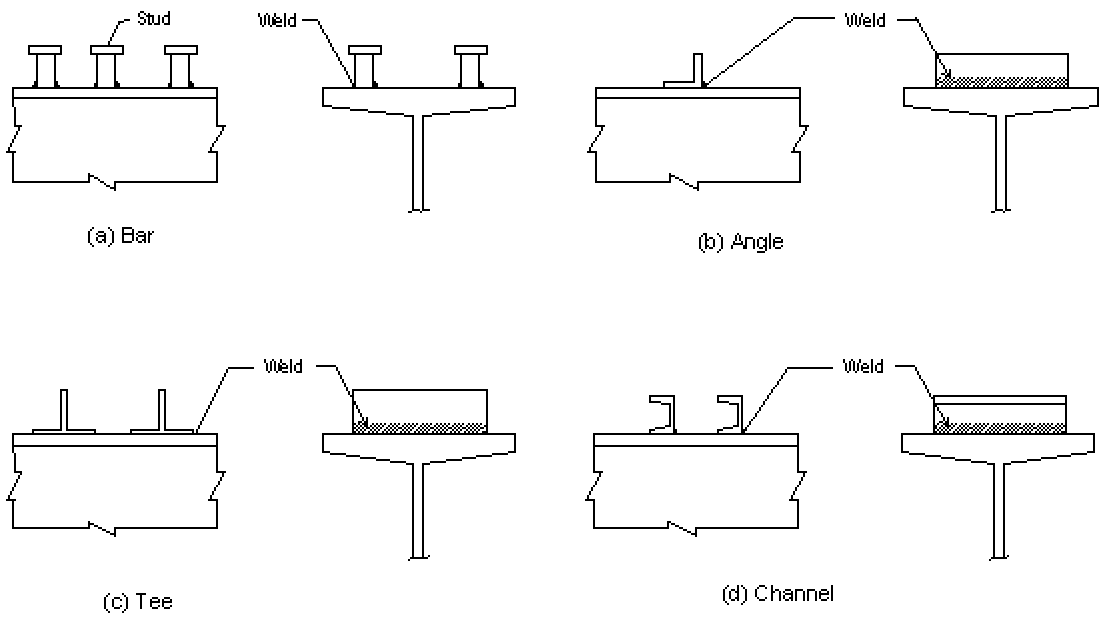


Fig. 3.8 Flexible shear connectors

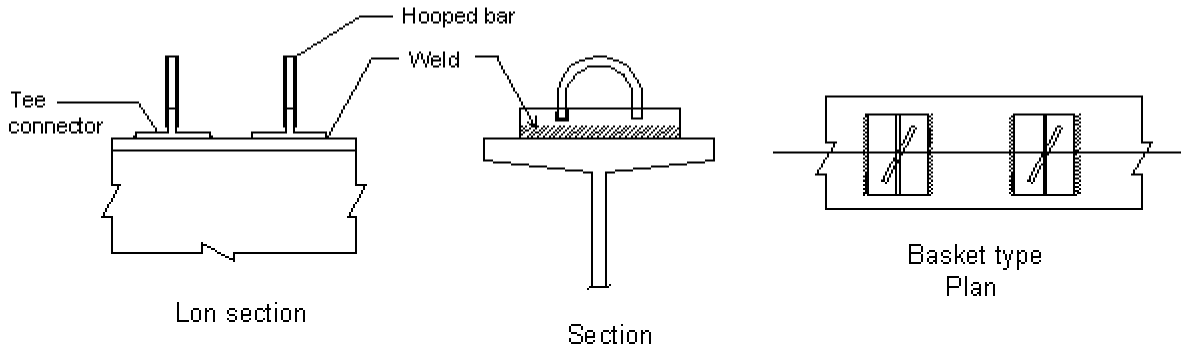


Fig. 3.9 Device with hoped bars welded to the flexible shear connectors

3.6.3 Anchorage Type Shear Connectors

Anchorage type of shear connector is used to resist longitudinal shear and to prevent separation of the beam from the concrete slab at the interface through bond.

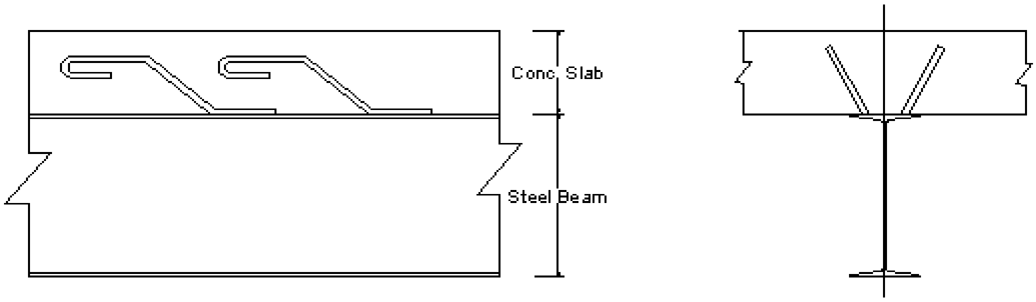


Fig. 3.10 Typical anchorage connectors

3.6.4 Deformation of Shear Connectors

The shear strength of connectors is established by the push-out test. If full shear connection is provided at the interface, failure will depend on the bending strength at the maximum moment zone of a simply supported beam or in case of a continuous beam at support location.

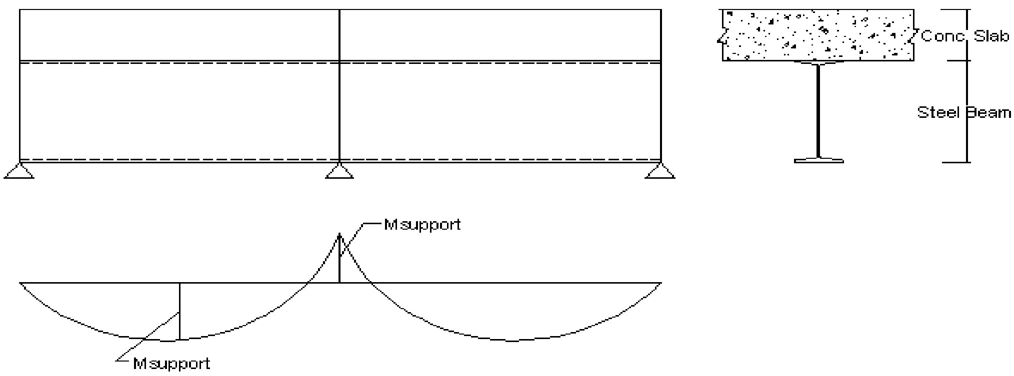


Fig. 3.11 Critical moment zone

The behaviour and resistance of headed studs and other connectors are examined by means of "shear" or "push out" tests. These tests yield load-slip curves such as is shown in Fig.3.12 for headed studs. The behaviour is characterised by great stiffness at low loading (under service conditions) and large deformations at high loadings up to failure. Such ductile behaviour makes shear force redistribution at the steel-concrete interface possible and allows for partial shear connection. In addition, headed studs may be spaced uniformly along the beam length between critical cross-sections.

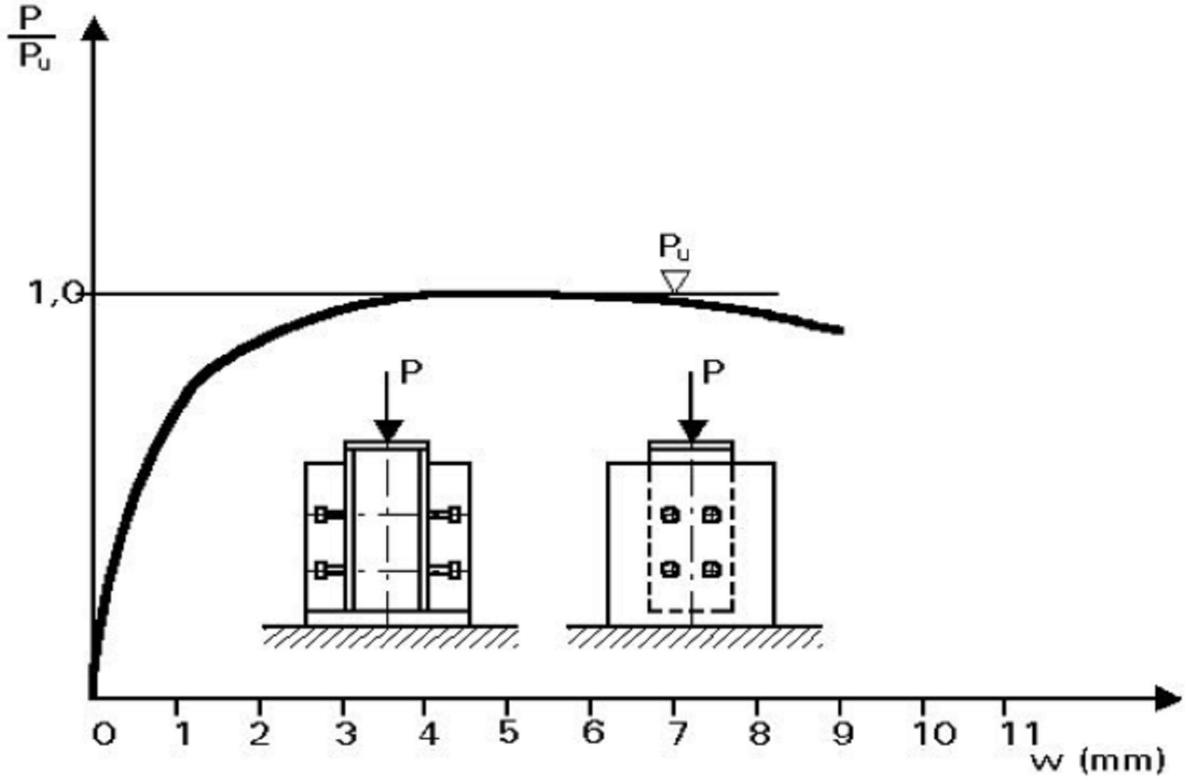


Fig. 3.12 Shear tests for headed studs

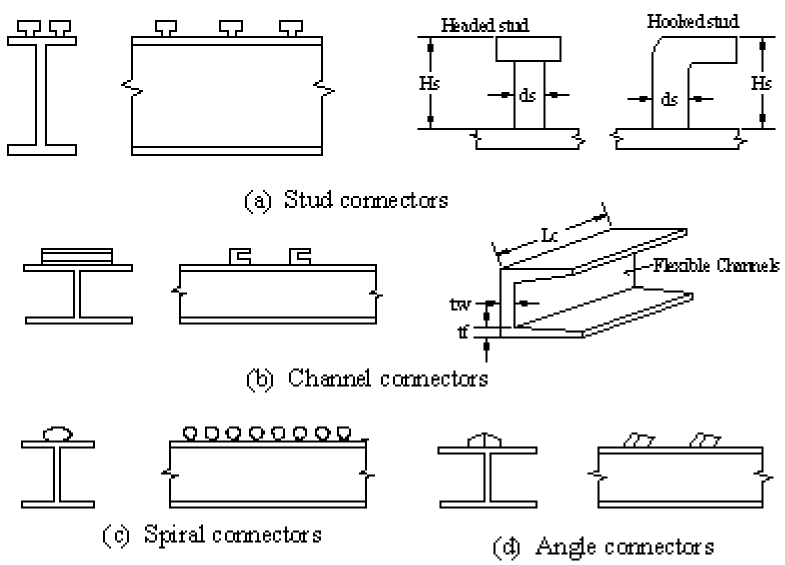


Fig. 3.13 Typical shear connectors

3.7 COMPOSITE BEAM

Composite beams, subject mainly to bending, consist of a steel section acting compositely with one (or two) flanges of reinforced concrete. The two materials are interconnected by means of mechanical shear connectors. It is current practice to achieve this connection by means of headed studs, semi-automatically welded to the steel flange.

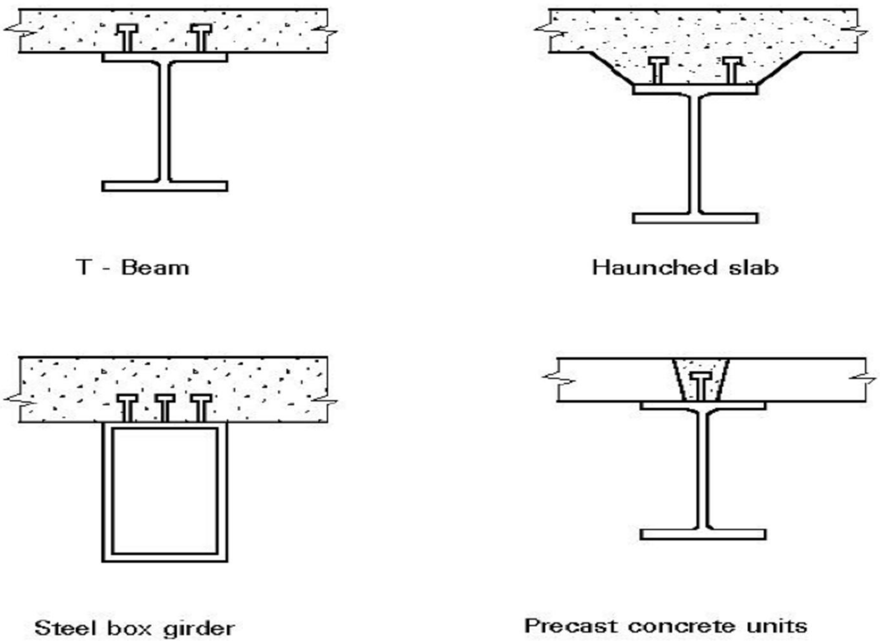


Fig. 3.14 Typical beam cross section

Fig.3.14 shows several composite beam cross-sections in which the wet concrete has been cast in situ on timber shuttering. For single span beams, sagging bending moments, due to applied vertical loads, cause tensile forces in the steel section and compression in the concrete deck thereby making optimum use of each material. Therefore, composite beams, even with small steel sections, have high stiffness and can carry heavy loads on long spans.

If slip is free to occur at the interface between the steel section and the concrete slab, each component will act independently, as shown in Fig.3.14. If slip at the interface is eliminated, or at least reduced, the slab and the steel member will act together as a composite unit. The resulting increase in resistance will depend on the extent to which slip is prevented. It should be noted that Fig.4 refers to the use of headed stud shear connectors. The degree of interaction depends mainly on the degree of shear connection used.

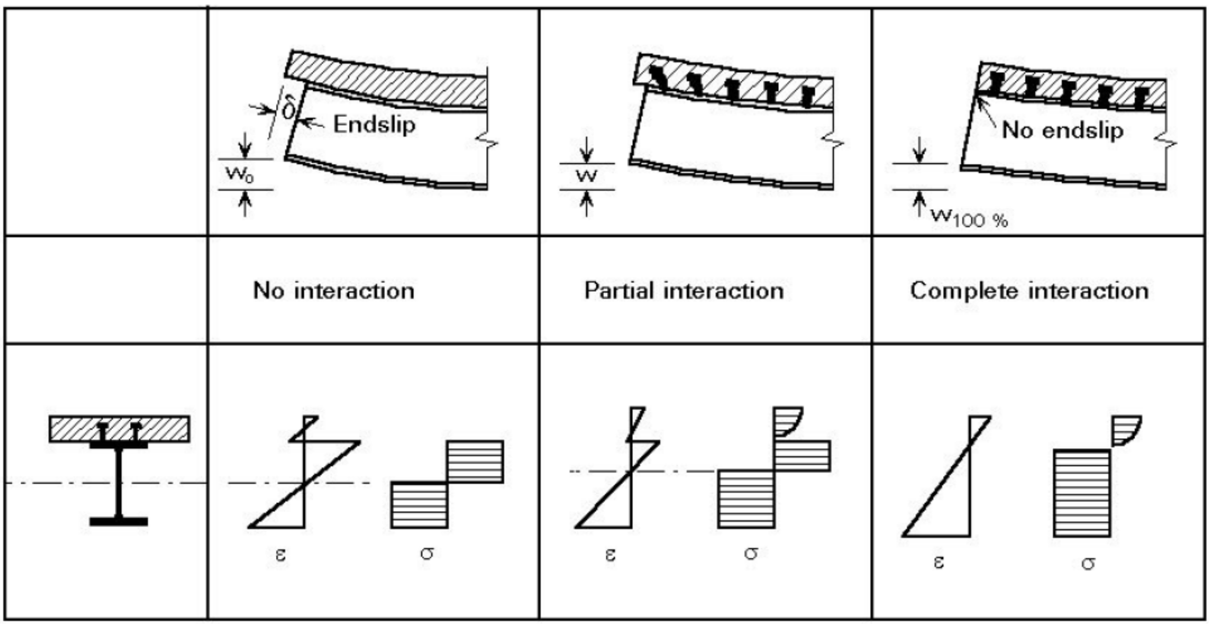


Fig. 3.15 Composite steel beam-concrete slab interaction

The following definitions are used to make clear the differences between resistance and stiffness properties:

- With regard to resistance, distinction is made between complete and partial shear connection. The connection is considered to be complete if the resistance of the composite beam is decided by the bending resistance, not the horizontal shear resistance.
- Complete or incomplete interaction between the concrete slab and the steel section results in a more or less stiff composite beam. Such incomplete interaction arises when flexible connectors such as headed studs are used and slip (relative displacement) occurs at the steel-concrete interface.

The use of composite action has certain advantages. In particular, a composite beam has greater stiffness and usually a higher load resistance than its non-composite counterpart, see Fig.3.1. Consequently, a smaller steel section is usually required. The result is a saving of material and depth of construction. In turn, the latter leads to lower storey heights in buildings and lower embankments for bridges.

3.8 COMPOSITE COLUMN

Three different types of composite columns are principally in use, see Fig.3.16

- Concrete encased steel columns Fig.3.16(a)
- Concrete filled steel tubes and Fig.3.2(c) & (d)
- Rolled section columns partly encased in concrete Fig.3.2(b).

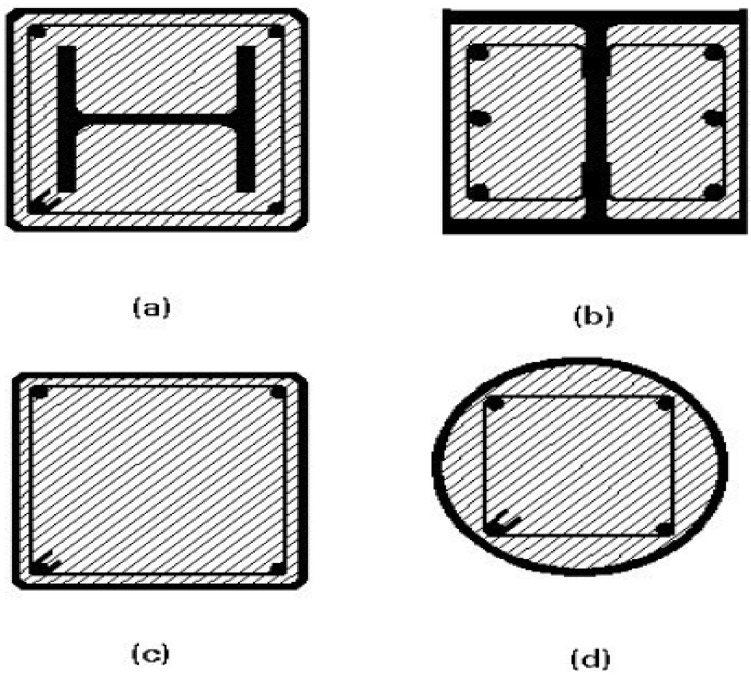


Fig. 3.16 Typical Cross section of composite column

In calculating the strength of such columns, full composite interaction without any slip at the steel-concrete-interface is assumed. Strictly speaking all geometrical and physical non-linearity of the different materials should be observed.

Concrete encased columns have the advantage that they meet fire resistance requirements without any other protection. In addition, they can be easily strengthened by reinforcing bars in the concrete cover. They do not, however, present an accessible structural steel surface for later fastenings and attractive surface treatment. In the case of prefabricated encased columns, the structural steel sections are fabricated in a workshop and include all welds, connection plates and other necessary attachments. These steel columns (the longest have been up to 30 m long) can then be transported to another workshop, where concreting takes place. After the concrete encasement has cured the completed columns can be brought to the construction site.

Concrete filled steel tubes are also in use. The tubes are generally filled with high strength concrete, with minimum cube strength 45 to 55 N/mm^2. These strengths, however, are far below those which have been developed recently in North America.

If the bearing forces from the floor beams are transferred by means of vertical connection plates, these plates run through the tube and are welded on both sides. This welding ensures both parts, the steel tube as well as the concrete core, are loaded directly without excessive slip at the steel-concrete interface. In order to meet the required fire resistance rating, the concrete core must be longitudinally reinforced. It is impossible, however, to take advantage of the full column resistance in many cases.

3.8.1 Partially Encased Steel Column

Partially encased steel sections, for both beams and columns, are an interesting development of the last 10 years. The most important feature of such a partially encased section is its inherent high fire resistance. The fire resistance is due to the fact that the concrete part prevents the inner steel parts - structural steel as well as reinforcing bars from heating up too fast. Fig.3.18 shows two partly encased composite beams compared with conventional fire protection by means of boards.

The concrete parts are cast in a workshop or on site before erection. This procedure enables rapid construction with prefabricated composite members. The concrete between the flanges should be reinforced by longitudinal bars and stirrups, and should be attached to the web by stud connectors, welded bars, or bars through holes.

In addition to the enhanced fire resistance, crippling and local buckling of the steel web is prevented and the resistance of the steel beam against lateral-torsional buckling is significantly increased. These beams also have greater stiffness under bending and vertical shear which results in a reduction of final deflection. They look very massive, as can be seen from Fig.17, and are characterised by their free bottom flange, to which ducts, other services and plant can be clamped or fastened.

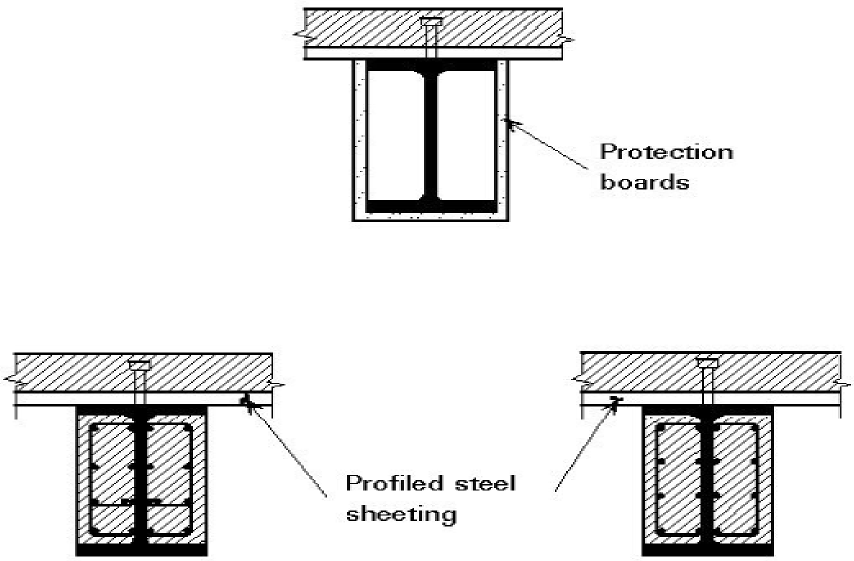


Fig.3.17 Fire resistance composite beam

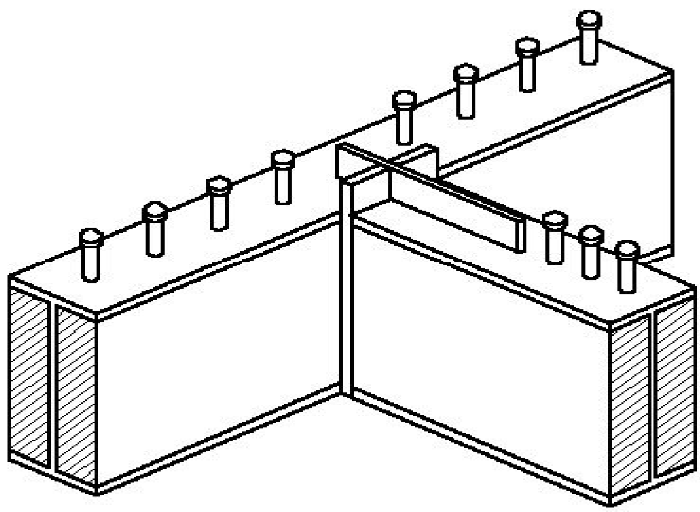


Fig.3.18 Partially encased beam with pinned connection

3.9 SUMMARY

This chapter includes various component of composite structure like floor, beam, column and shear connector. The basics of composite structure components and advantages are also discussed. the slab and beam interaction using shear connector is shown.

4. LATERAL RESISTING SYSTEMS

4.1 GENERAL

Stability is one of the primary concerns in designing a structure for any load condition. When load is applied, internal forces will be generated causing the structure to deform. In a stable structure, this deformation is usually small and after load is removed the structure will restore its original shape. In an unstable structure deformations are massive and the structure does not produce internal forces that tend to restore the original shape of the structure. Stability failures are often sudden and catastrophic.

Considering Fig.4.1(a), The simple (pin connected) column-beam frame appears to be stable. However, under any lateral load such as wind load, this frame will undergo deformation and collapse due to ability of joints to rotate and lack of members to resist lateral loads Fig.4.2(b).

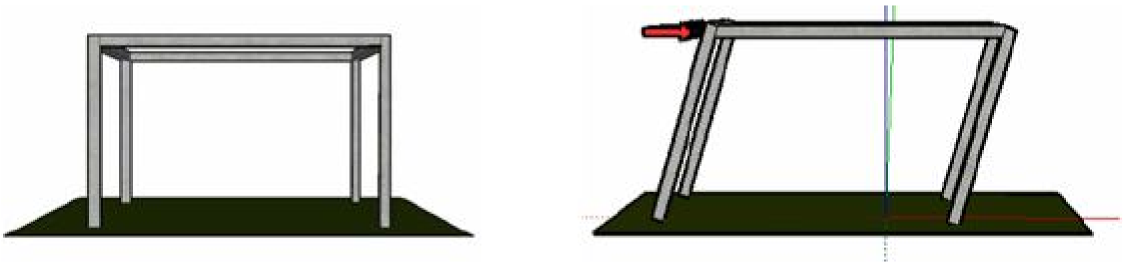


Fig.4.1 (a) Column beam frame (b) Instability of the frame under lateral loads

4.2 TYPES OF LATERAL RESISTING SYSTEM

Mainly there are three types of lateral resisting system. Each system has a specific advantages and disadvantages over the other system. As per different requirement, the different system has been used.

- Braced Frame Fig.4.22(c)
- A stiff diaphragm plane called as Shear wall Frame Fig.4.2(b)
- Moment resisting Frame Fig.4.2(a)

When a diagonal member (bracing) or a block wall (shear wall) is inserted or when rigid frame connections are used, the structures becomes stiff and capable of transferring the load, thus preventing the collapse Fig.4.3.

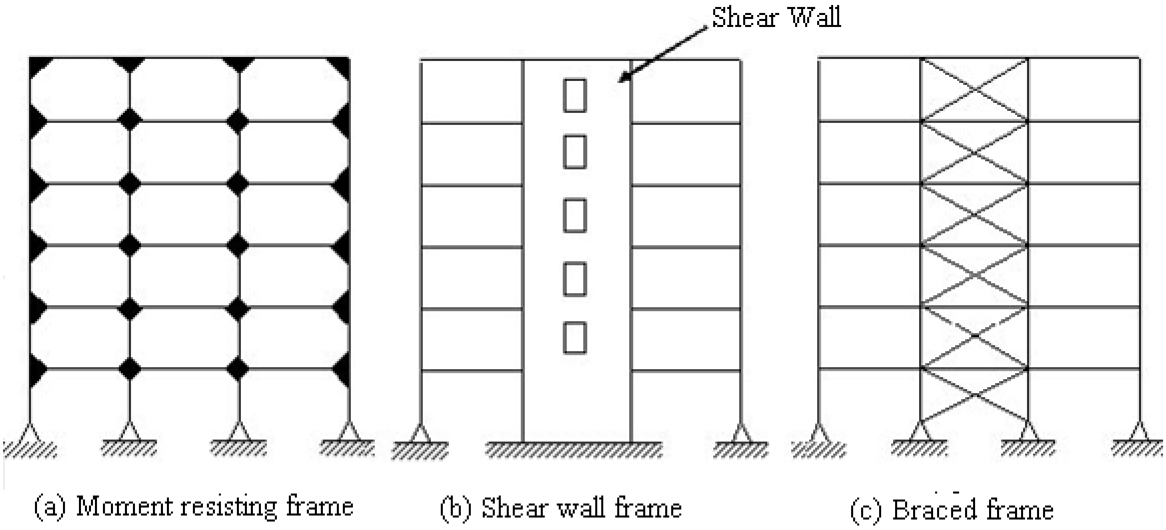


Fig.4.2 Lateral resisting frame

The term "Shear Plane" is used to describe any of these methods that provide the stiffness and load transmittal capabilities

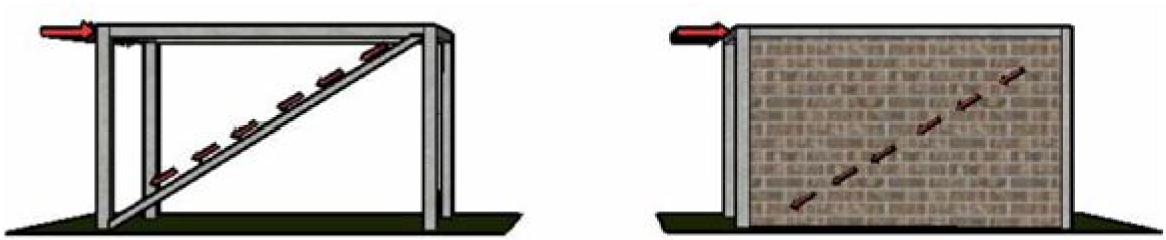
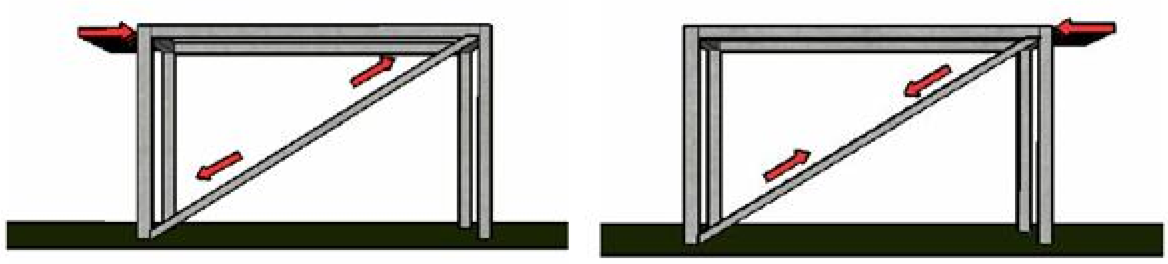


Fig. 4.3 System offering lateral resistance (a) Bracing (b) Shear wall

4.2.1 Different Bracing System

Braced system is one of the most widely used lateral resisting systems. When bracing is added, the structure cannot undergo the deformation under the lateral load; if the bracing is adequately sized. Following are the different types of bracing

- *Single Diagonal Bracing:* As the name suggests, only a single diagonal member is used. This member is designed to resist both, tension as well as compression caused by lateral forces acting in both directions on the frame Fig.4.4(a) & (b).



(a) (b)

Fig.4.4 Bracing in (a) Tension (b) Compression

- *X Bracing/Cross Bracing:* This is the most economical and efficient forms of bracing. When the cross bracing is used, lateral force from one direction induces tension in one member while the other brace is in tension when the force is reversed shown in Fig.4.5.

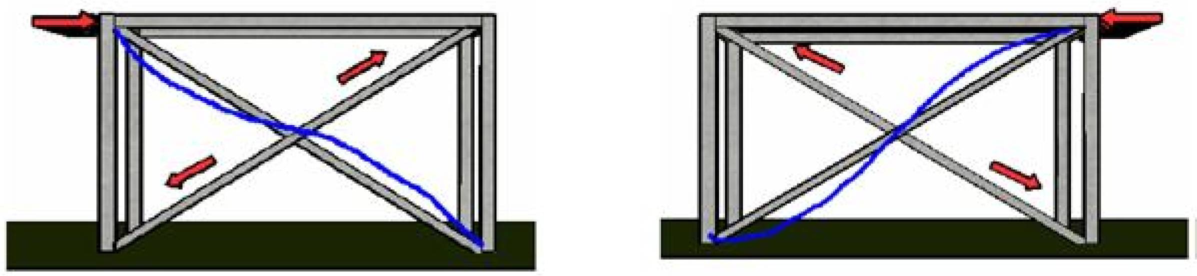


Fig.4.5 One Brace in tension, other goes in compression

Therefore, if two diagonals are used, in the form of cross-bracing, they only need to resist tension When one brace is in tension, compression is induced in the other. However, the slender compressive brace immediately sheds

This compression by buckling out of plane to avoid the force. Thus, the other brace then has to take 100% of the lateral force in tension. Therefore, the compressive brace may be ignored. Steel cables can be used for cross-bracing, as they can be stretched, but not squashed.

- *Inverted V bracing:* Though the X-bracing is the most efficient and economical type of bracing, many times it is not used. X-bracings run across the entire wall area and it becomes impossible to accommodate required openings such as for windows and doors. In such instances, inverted V-bracing is used that allows for the opening, such as a window or a door as shown in Fig. 4.6.

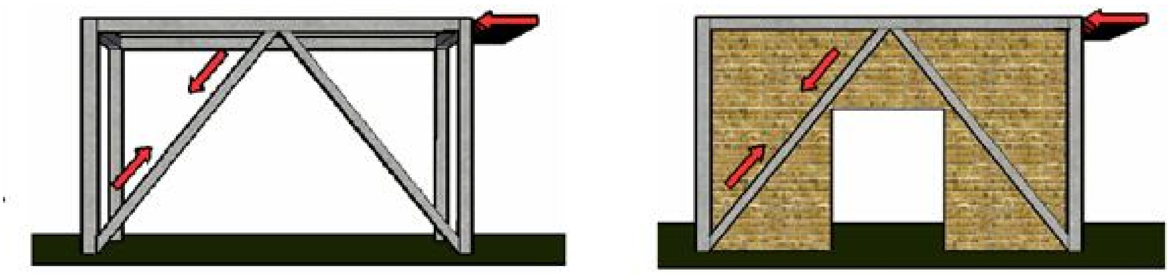


Fig.4.6 Inverted "V" Bracing

- *Corner/Knee Bracing:* This is another type of bracing that is used as the same purpose as V bracings, to allow for an opening as shown in Fig. 4.7.

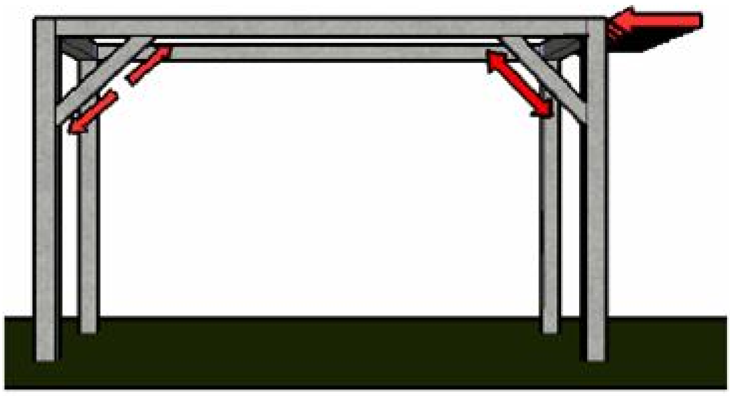
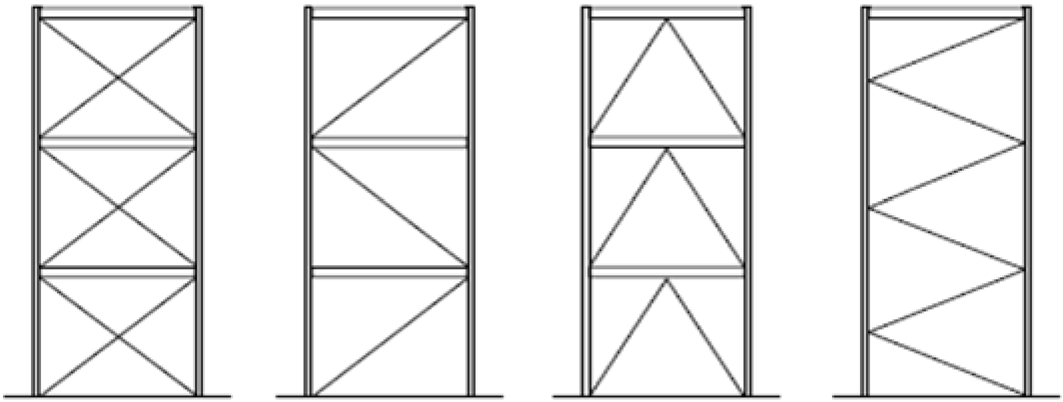


Fig.4.7 Knee bracing

- *K-Bracing:* K-bracing is used where full height openings are required. However K-bracing is not as efficient as X-bracing shown in Fig.4.5.

4.2.2 Vertical Bracing System

Vertical bracing to columns provides lateral stability to a structure and resistance to wind and earthquake loading. The bracing is thus subject to horizontal loading acting in either the left-to-right or right-to-left direction. The most commonly used configurations are illustrated in Fig.4.8 (a) to (c). It can be used in multi-storey buildings, with the floor beams being located at each panel height of the system. They could also be used, along with the configurations shown in (d), for tall columns in single-storey buildings. In this case the beams indicated in details (a) to (c) would be replaced by horizontal struts.



(a) (b) (c) (d)

Fig.4.8 Vertical bracing system

In type Fig.4.8(a) the diagonals could be designed to act either in tension only or in combined tension compression; in the latter case the horizontal members would carry no load. The tension-only system is very efficient since the diagonals can be designed to minimum size and with a large slenderness ratio. It is especially applicable to bracing systems with large panel sizes, i.e. in height or width or both.

In detail Fig.4.8(b) the diagonals act in tension and compression and thus need to be stiffer; the horizontal beams do not carry any bracing load. Note that at ground level the full horizontal load is resisted by a single column foundation, which is a less favorable situation than when it is shared between two column bases. It is nevertheless an efficient system, provided the lengths of the diagonals are not excessive, since a minimum number of members and connections are involved.

The inverted-V or chevron bracing Fig.4.8(c) is a tension compression system with shorter diagonal members and each horizontal member acting half in tension and half in compression. It is thus an efficient system, but if applied to a multi-storey building the bracings act as props at mid-length of each beam which would result in a lighter beam section, but a much heavier bracing section.

4.2.3 Shear Wall

Shear walls are rigid planer surface elements that inherently resist any change in the shape. A concrete block wall, or reinforced concrete wall can be used as a shear wall Fig.4.9. Depending upon the magnitude of the forces either a full or partial wall may be used.

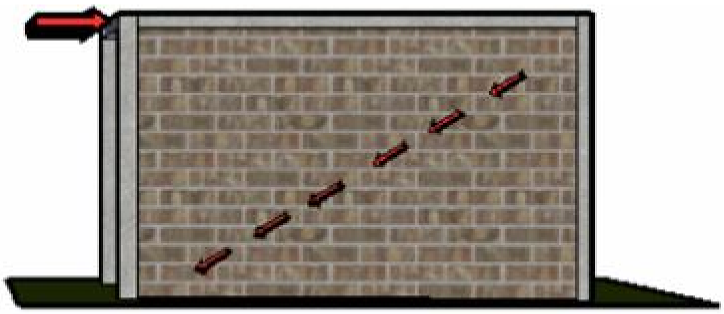


Fig.4.9 Shear wall

4.2.4 Moment Connection / Rigid Joint

A moment frame is a structure that utilizes moment resisting connections between columns and girders throughout its perimeter to resist the lateral loads applied. These frame structures are characteristic of early skyscrapers where three dimensional structural analyses were still in its infancy. The repetitive pattern with its small cross sectional changes from floor to floor allows simple construction. Moment frame also allows unobstructed bays that allows for flexibility in locations of opening. this feature is much desired by architects seeking flexibility in their design and also helps to introduce as much natural light into the space as possible.

Moment resisting frames Fig.4.2(c) rely on the ability of the frame itself to act as a partially (semi-) or fully rigid jointed frame while resisting the lateral loads. Due to their flexibility, moment resisting frames experience a large horizontal deflection called drift, especially in tall buildings but can be used for medium rise buildings having up to ten stories. The rigid connection type's beam to column connections can be used in such frames.

Under lateral forces, a structure collapses due to large angular deformations in the members. One method to prevent such angular deformation is making connections between the members rigid. Rigid joints prevent any angular deformation that could take place under lateral loading.

Moment connections offer the frame action that can be used to resist lateral loads, however these are less efficient than shear walls or braces. Moment or rigid connections induce high bending moments in structure resulting into large member sizes. Thus, rigid connections are not generally used for larger buildings.

4.3 Summary

- Several configurations are available for vertical bracing to columns, viz. X-bracing acting in tension only or in tension compression, single-strut tension compression bracing, chevron bracing, zig-zag tension compression bracing, knee bracing, etc.
- Study says that the tension-only X-braced configuration is the cheapest and most effective and is employed where other considerations do not prevent its use.
- Knee-bracing is less stiff and more expensive, but is necessary when clearance is required between the columns.
- Vertical bracings connecting to a beam or a column should have their axes meeting at the column-beam flange intersections rather than on the column-beam centerline point. More compact gussets are achieved in this way.

5. DESIGN OF PARKING STRUCTURE WITH MRF SYSTEM

5.1 GENERAL

A moment frame structure consists of columns and girders jointed by moment-resistant connections. The lateral stiffness of a moment frame bent depends on the bending stiffness of the columns, girders and connections in the plane of the bent. The principal advantage of moment frame is its open rectangular arrangement, which allows freedom of planning and easy fitting of doors and windows.

Gravity loading also is resisted by the moment 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.

There are many functional layouts used for multilevel car parks, each having specific advantage. Here, the split level type car park is used considering more economical option for composite construction. The moment frame building consists of composite floor considering profile decking and composite beam of steel section while steel column has been used for fast erection consideration.

There are two options considered for structural framing system for present study: (1) Moment Resisting Frame (2) Braced Frame.

5.2 GENERAL CONSIDERATION FOR MULTILEVEL CAR PARKS

Five storied split level type car park considered for achieving optimum solution using composite option. This system is one of the most popular in U.K. for car parks.

The design parameters considered for design of the two options are as follows:

1.	Built-up area	80m x 32m
2.	No. of floors	5 (Including Ground/basement)
3.	Width of ramp	5m (2 stall widths)
4.	Inclination of ramp	14% (For split level type)
5.	Height between floors	3.1 m (All floors)
6.	Clearance height	2.33m

7. Safe soil bearing capacity of soil 300 kN/m^2
8. Importance factor 1
9. Earthquake zone III
10. Response reduction factor 3
11. Slab thickness 130 mm
12. Wind speed 44 m/s
13. Code References
    - BS: 5950 (Part 3.1 & Part 4)
    - Euro Code 3 (Part 1.1)
    - Euro Code 4 (Part 1.1)
    - IS: 800-1984
    - IS: 1893-2002 (Part 1)
    - IS: 875-1987 (Part I to III)
    - IS: 11384-1985
    - IS: 456-2000

5.2.1 Design Methodology

a) Floor is prepared using profile steel deck, with topping of floor finish.

b) All beams, columns and bracings are fabricated from steel.

c) Profile steel deck and beams has been designed by the limit state method using partial safety factor for loads and material strength specified in BS: 5950 (Part 4) and EC4 respectively.

d) Shear Connector: Stud shear connector has been used and capacity is taken from IS: 11384-1985

e) Composite beam design is made as per BS: 5950 (Part 3.1)

f) Cambering is required in all 16m span beams to take care of deflection serviceability.

g) Facilities provided

- Attendant's room and toilet at ground floor
- Two nos. lifts
- Two stair case
- Parking for about 558 no. of cars

5.2.2 General Requirement of Split Level Type Car Parks

The ramp systems feature separated one-way operation, and access is on only one street. Ninety-degree parking is utilized throughout the floor. The cashier's booth is at the entrance and the stairs and elevators are strategically located to take advantage of the split-level and to afford minimum walking distances. Split level ramp shown in Fig.5.1.

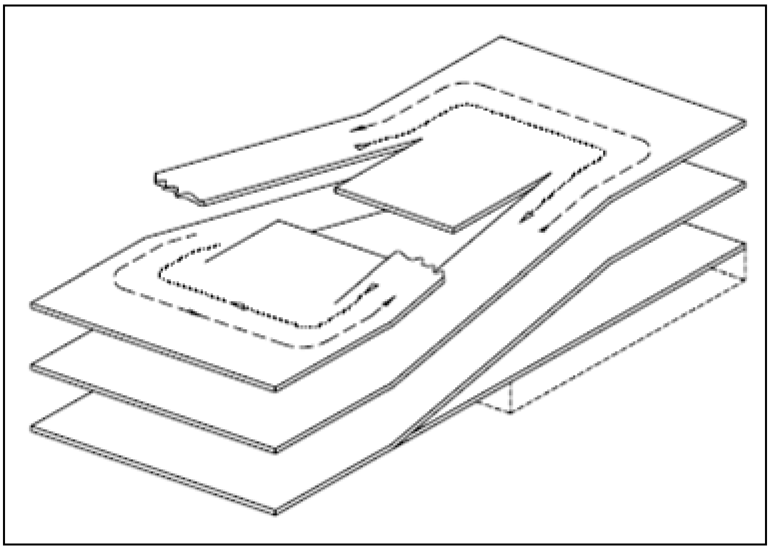


Fig. 5.1 Split level or staggered ramps

- *Ramp, transition and interlock*

  Split-level floor construction requires the length of ramp travel to be about one-half the usual inter-floor distance shown in Table 5.1. This was one of the most common designs for years. Split-level floors can overlap as much as five to six feet at the split, which increases space efficiency and makes a narrow site workable.

  Typically, two ramps are utilized supporting either one- or two-way traffic. Parking and walking does not take place on the ramps. Ramps can be sloped as greatly as 16 percent, but any slopes over 14 percent will require transitions seen in Fig.5.2. Each ramp climbs half the height of the tier.

For utilizing maximum space on the floor, the interlocked system can be used. In the interlock system the car parked at angle and it interlocked the stall depth seen in Fig. 5.3, Table 5.2.

Table 5.1 Ramp length for straight ramp

Floor to Floor Height						Half Height Split Level	
Slope	8 ft	9 ft	10 ft	11 ft	12 ft	4 ft	5 ft
5%	160 ft	180 ft	200 ft	220 ft	240 ft	80 ft	100 ft
6%	114	128	153	157	172	57	77
7%	67	75	83	92	100	34	42
8%	57	64	72	79	86	29	36

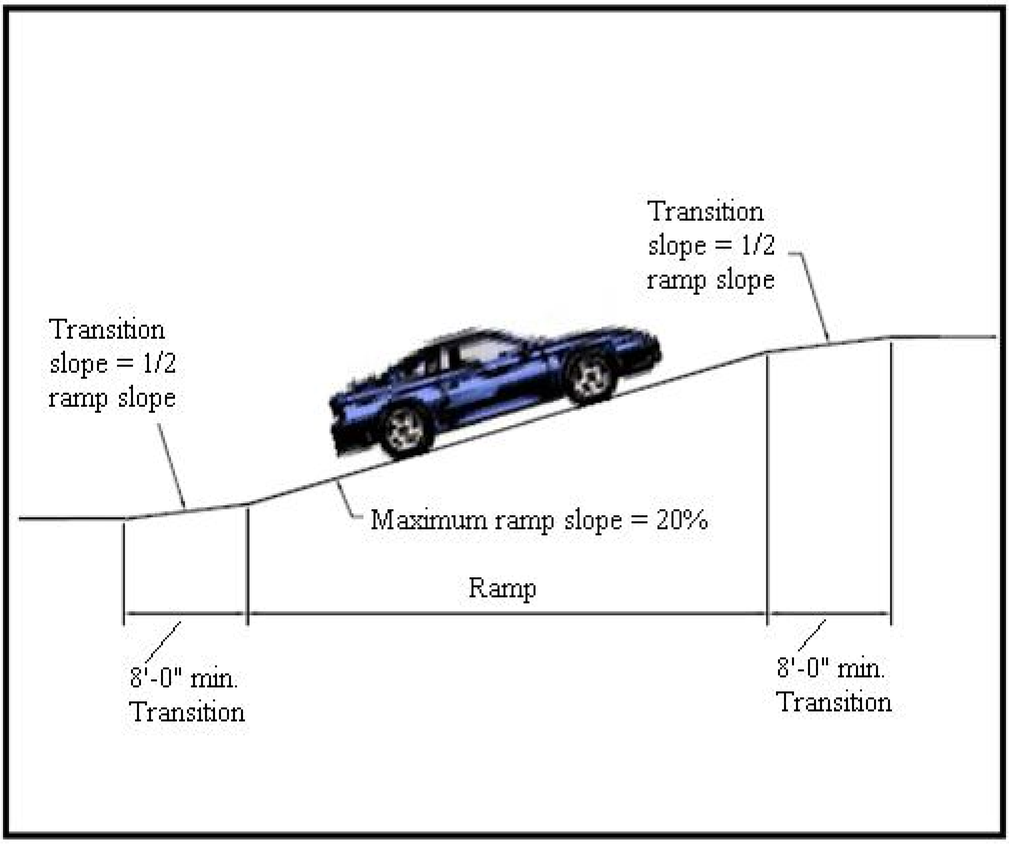


Fig. 5.2 Ramp slope and transition

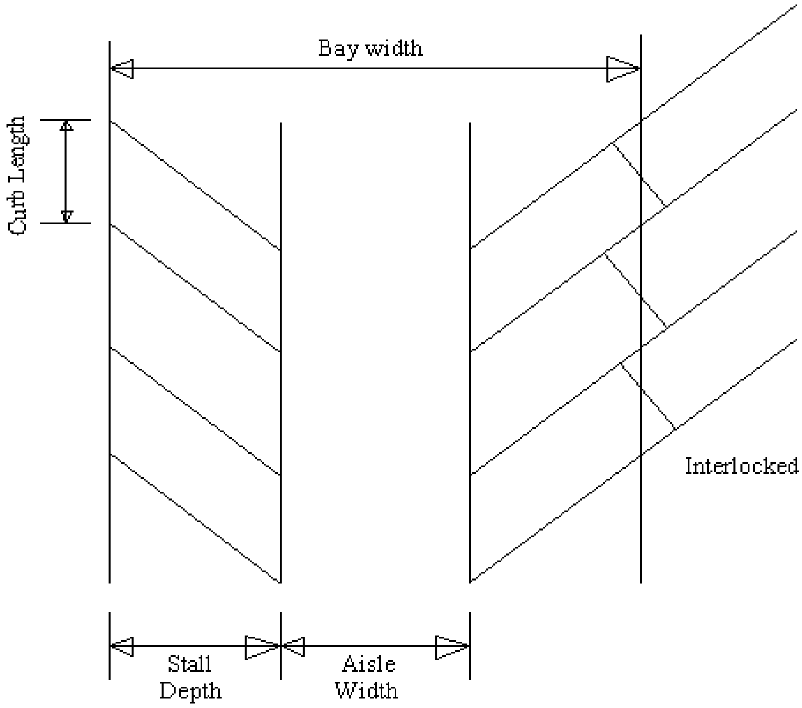


Fig. 5.3 Interlocked system parking

Table 5.2 Parking angle and different sizes

	Park Angle							
	Inter locked				Non-Locked			
	45°	60°	70°	90°	45°	60°	70°	90°
Curb Length (ft)	12.33	10.10	9.33	8.75	12.33	10.10	9.33	8.75
Depth of Stall(ft)	13.60	16.60	17.50	17.75	17.50	18.75	19.00	17.75
Aisle Width(ft)	13.67	15.50	17.50	25.00	13.67	15.5	17.50	25.00
Bay Width(ft)	40.87	48.70	52.50	60.50	48.67	53.00	55.5	60.50

- *Stairs and elevator*

  Every parking structure is required to have a minimum of two means of egress (stairs) which are separated from each other. These stairwells should be located based upon the requirements of local safety codes. One of the stairwells is normally located adjacent to the elevator locations. If the parking structure supports a high peak flow, extra wide stairs may be necessary. The minimum clear width for the stairway is 36 in. Stairwells on the perimeter of the structure are often left open or glass enclosed to create a sense of security within the structure.

The number and locations of stairs is determined by the maximum travel distance or the distance the patron must travel along a normal path from any point in the structure to the closest stair. In open structures the controlling distance is 300 ft.

The number of elevators required to service the parking structure varies by the usage of the structure. Elevators should be located along the natural direction of travel for a patron exiting the structure. Some parking designers take into account elevator locations in specifying the traffic flow through the facility. In doing so, all incoming traffic is routed past the elevators as a means of orienting the patron to the structure layout.

5.3 PLAN GEOMETRY

From the literature survey, the plan of multilevel car parking is prepared and selects the split-level ramp system. In this system the four single ramps are provided. Two ramps are place at extreme end of rectangular long side and two are placed at the intermediate of previous two. Also the two staircase and two elevators are placed at exterior side of the end ramps. The cashier's room and toilet unit are provided at left and right side of entrance.

The following figures indicate the general arrangement, plan, elevation and cross section details including marking of beams, columns and bracing of 5 level split level car park considered for design.

Fig. 5.4 Ground Floor Plan

Fig. 5.5 Typical Floor Plan

Fig. 5.6 Top Floor Plan

Fig. 5.7 Elevation on Row A & C

Fig. 5.8 Elevation on Row B

Fig. 5.9 Cross Section on Line 5a

Fig. 5.10 Cross Section on Line 5b

Fig. 5.11 Roof Plan & Plan 2-2

Fig. 5.12 Cross Section on Line 1 & 9

Fig. 5.13 Elevation on Row A''

Fig. 5.14 Cross Section on Line 1' & 9'

Fig. 5.15 Typical Cross Section

Fig. 5.16 Typical Intermediate Cross Section

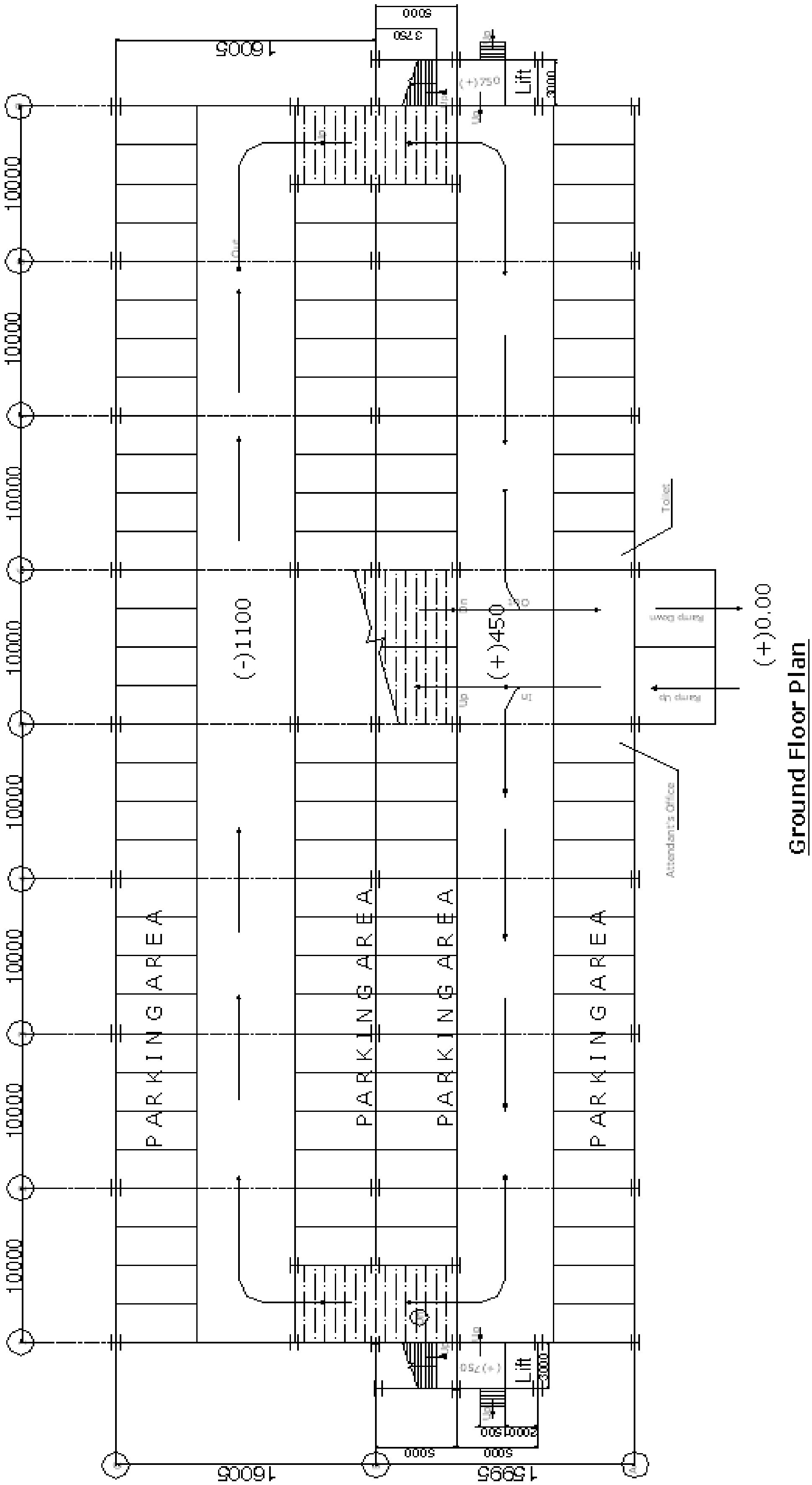


Fig. 5.4 Ground Floor Plan

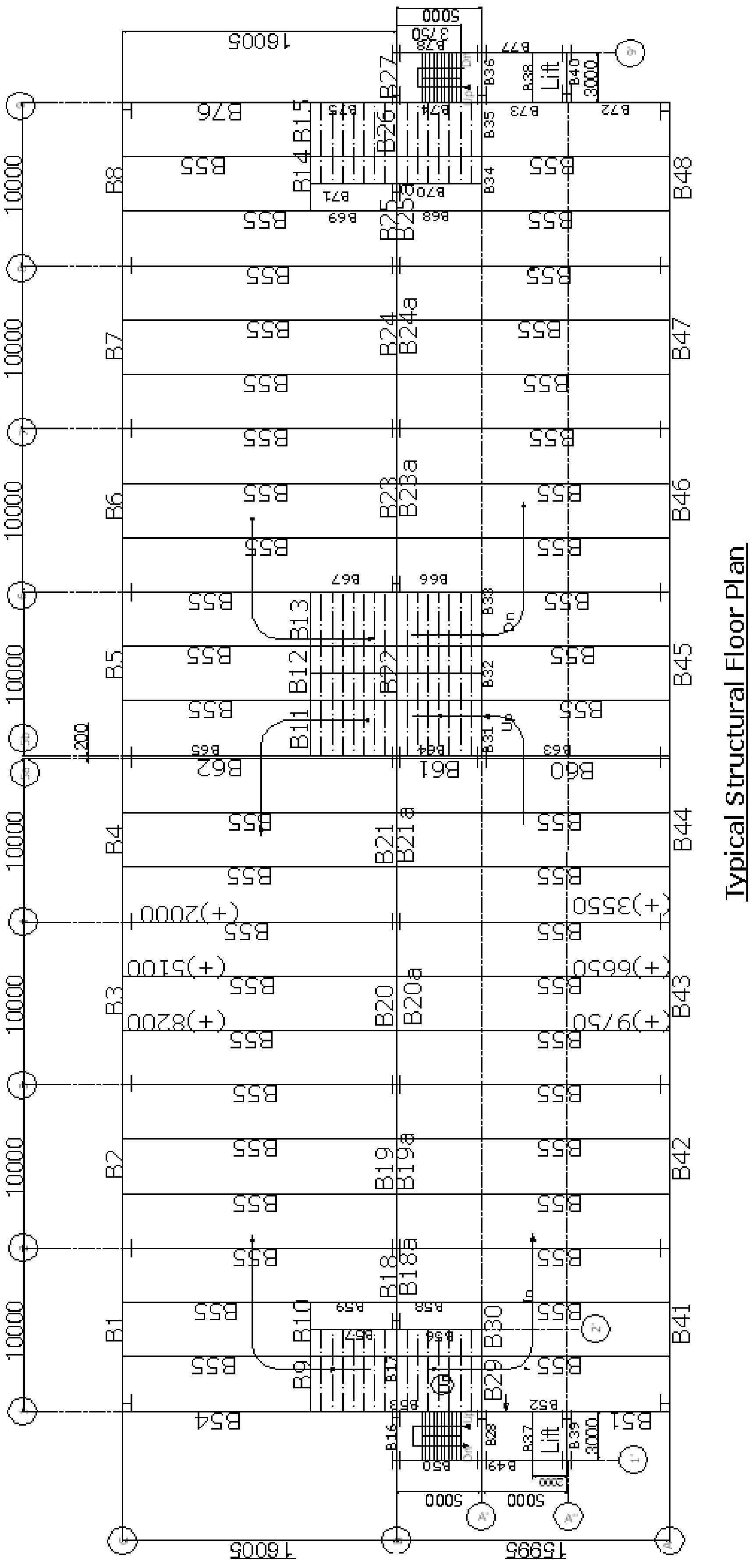


Fig. 5.5 Typical Floor Plan

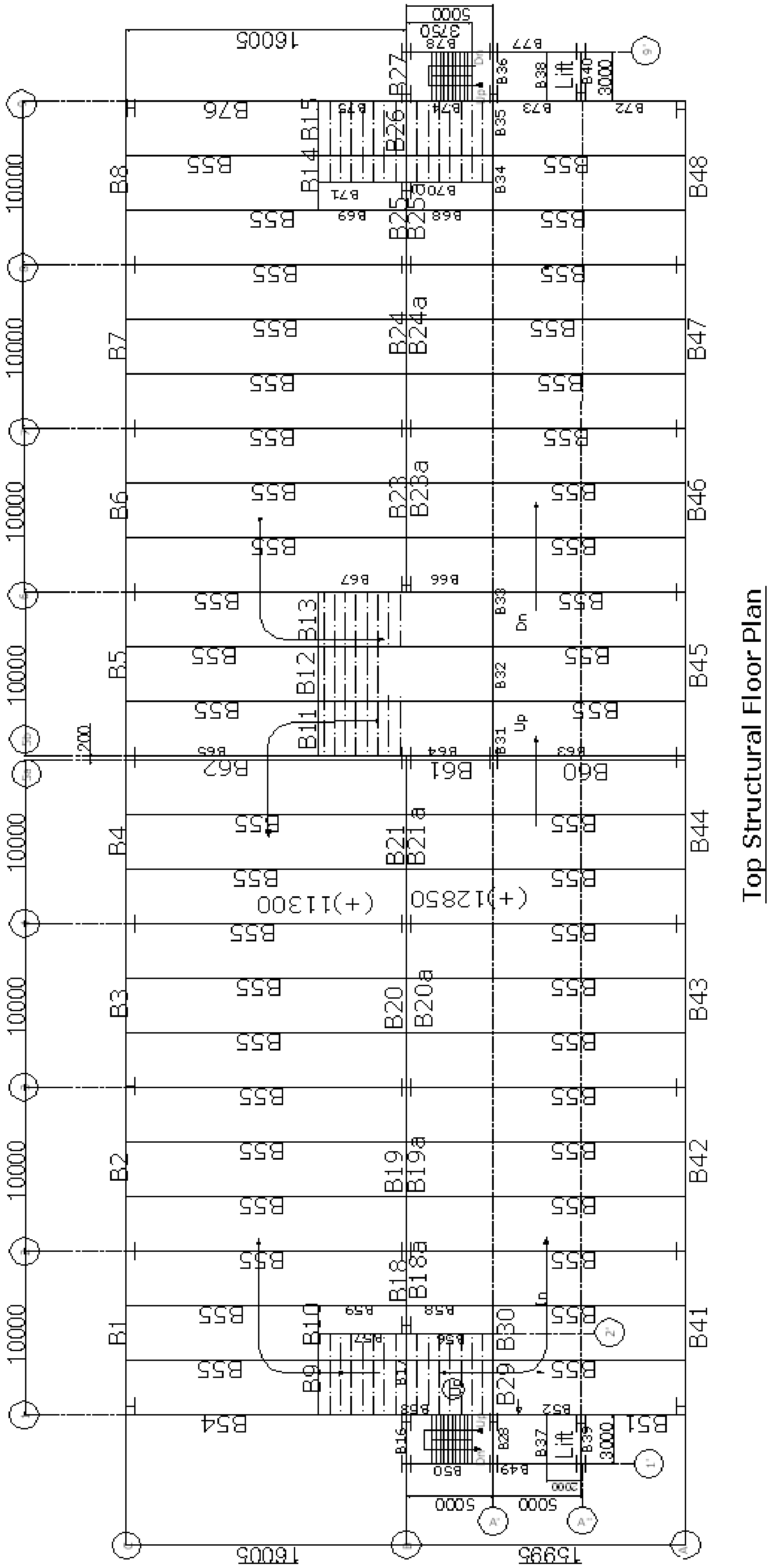


Fig. 5.6 Top Floor Plan

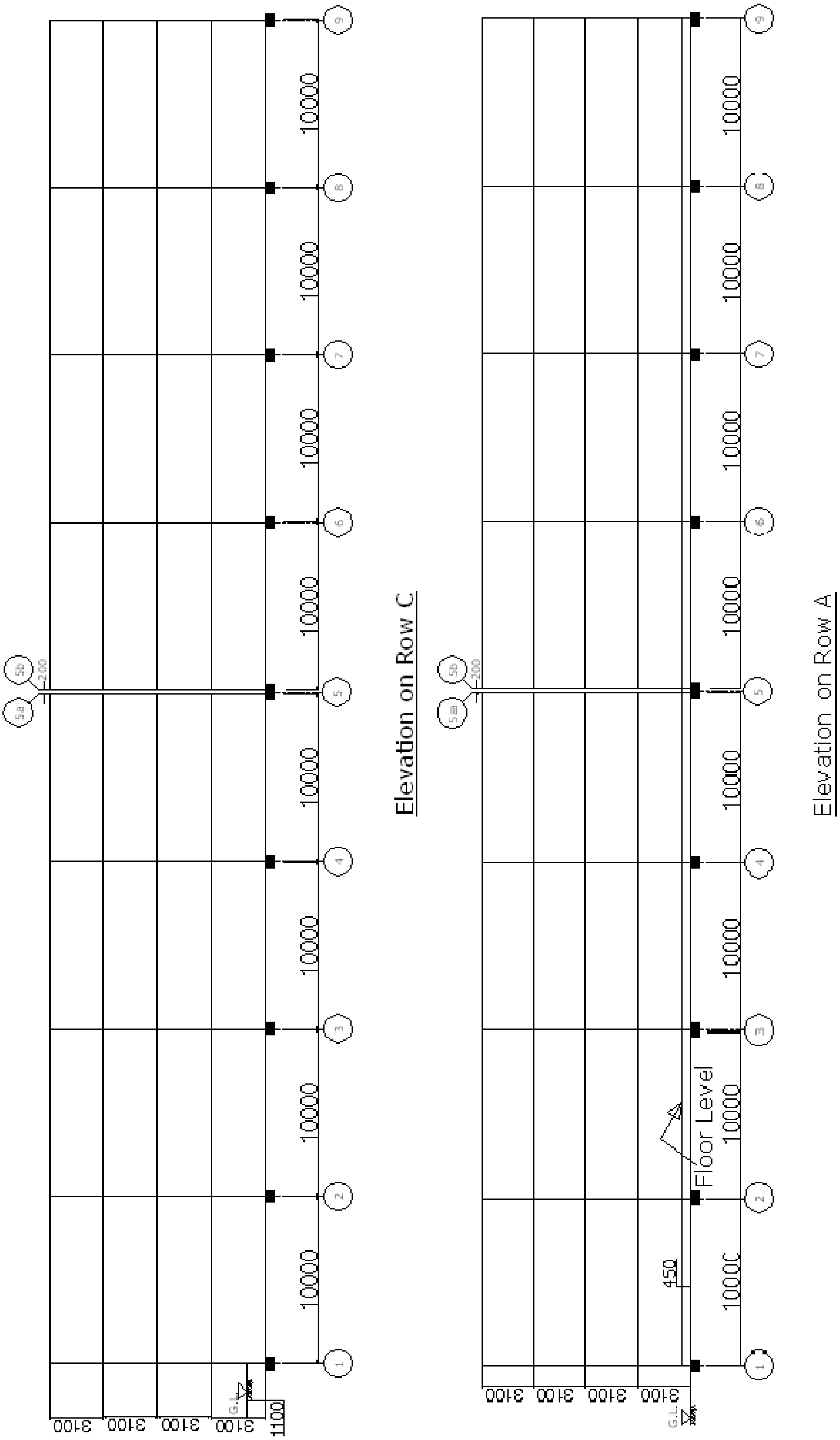


Fig. 5.7 Elevation on Row A & C

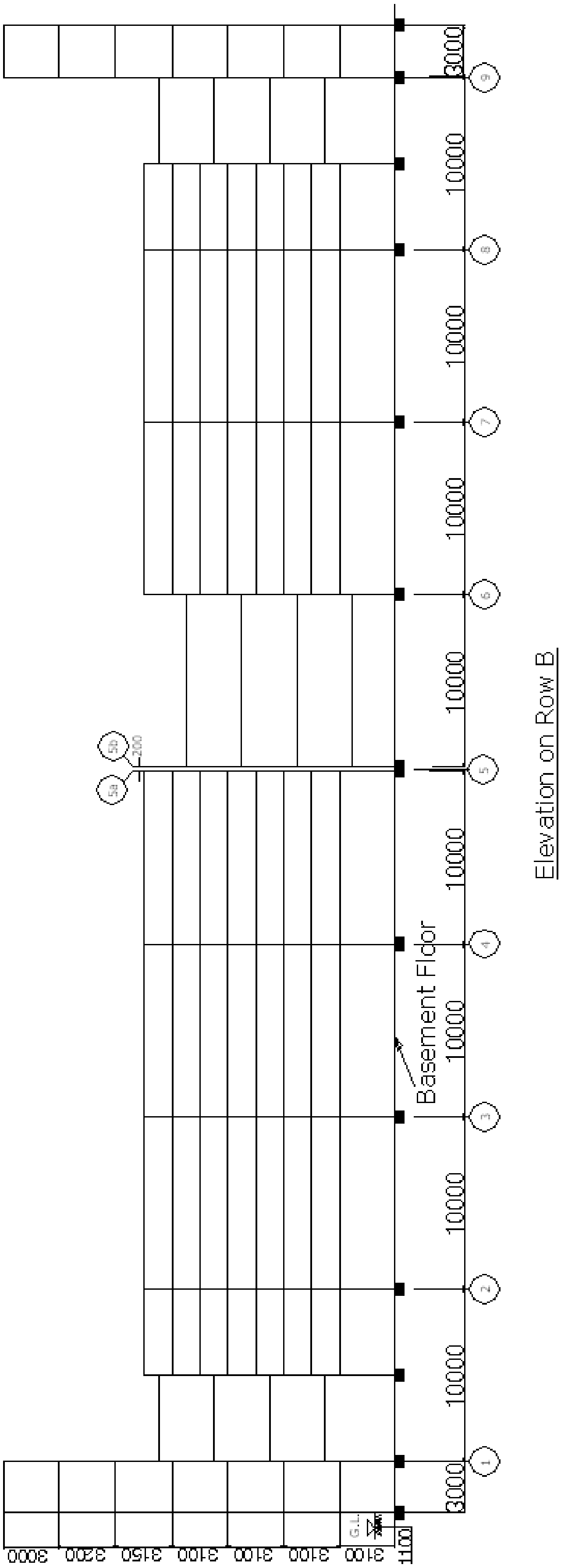
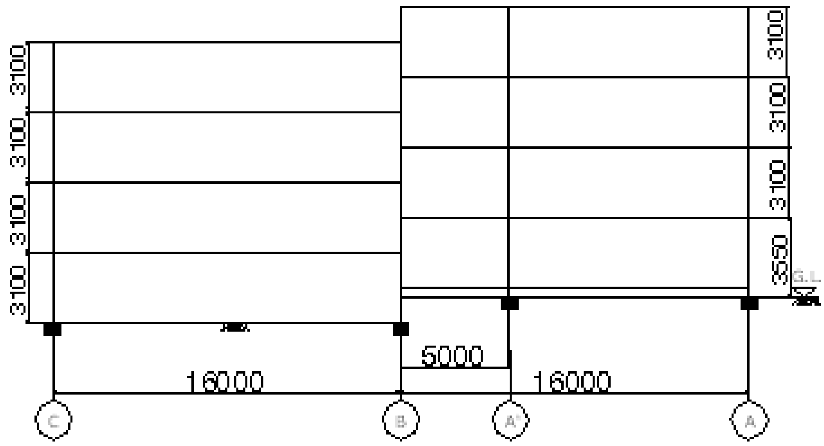
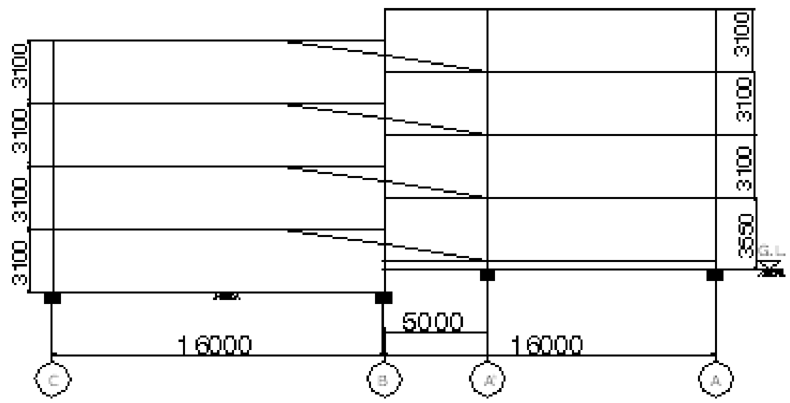


Fig. 5.8 Elevation on Row B



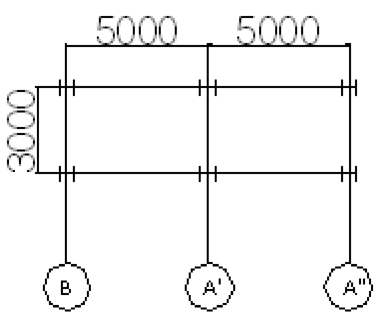
Cross Section on Line 5a

Fig. 5.9 Cross Section on Line 5a

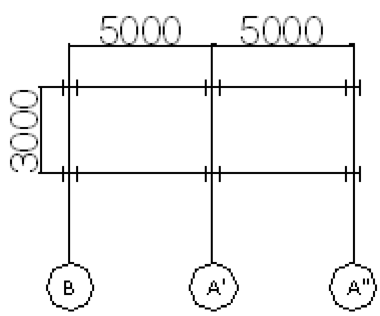


Cross Section on Line 5b

Fig. 5.10 Cross Section on Line 5b



Roof Plan



Plan 2-2

Fig. 5.11 Roof Plan & Plan 2-2

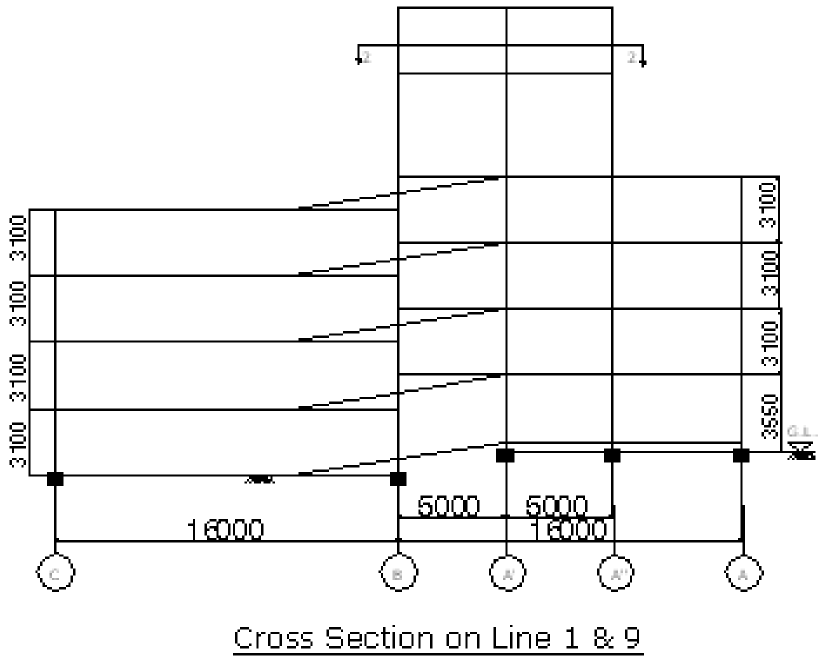
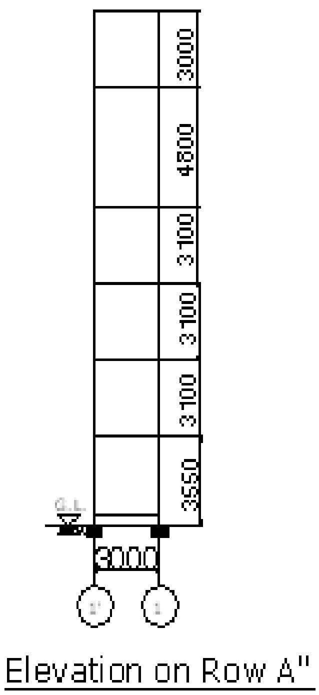


Fig. 5.12 Cross Section on Line 1 & 9



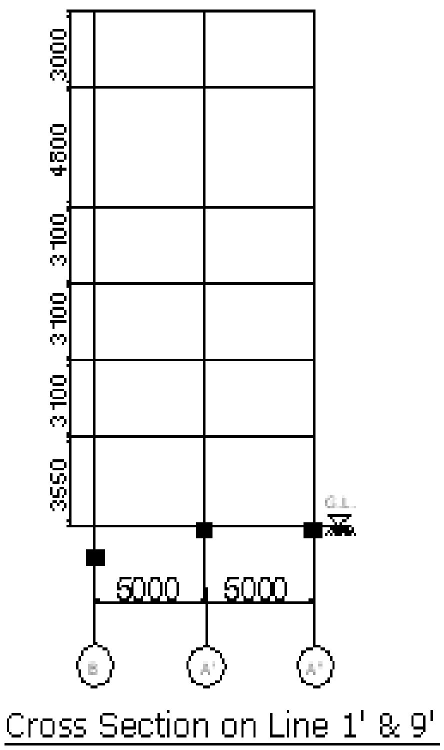


Fig. 5.13 Elevation on Row A''

Fig. 5.14 Cross Section on Line 1' & 9'

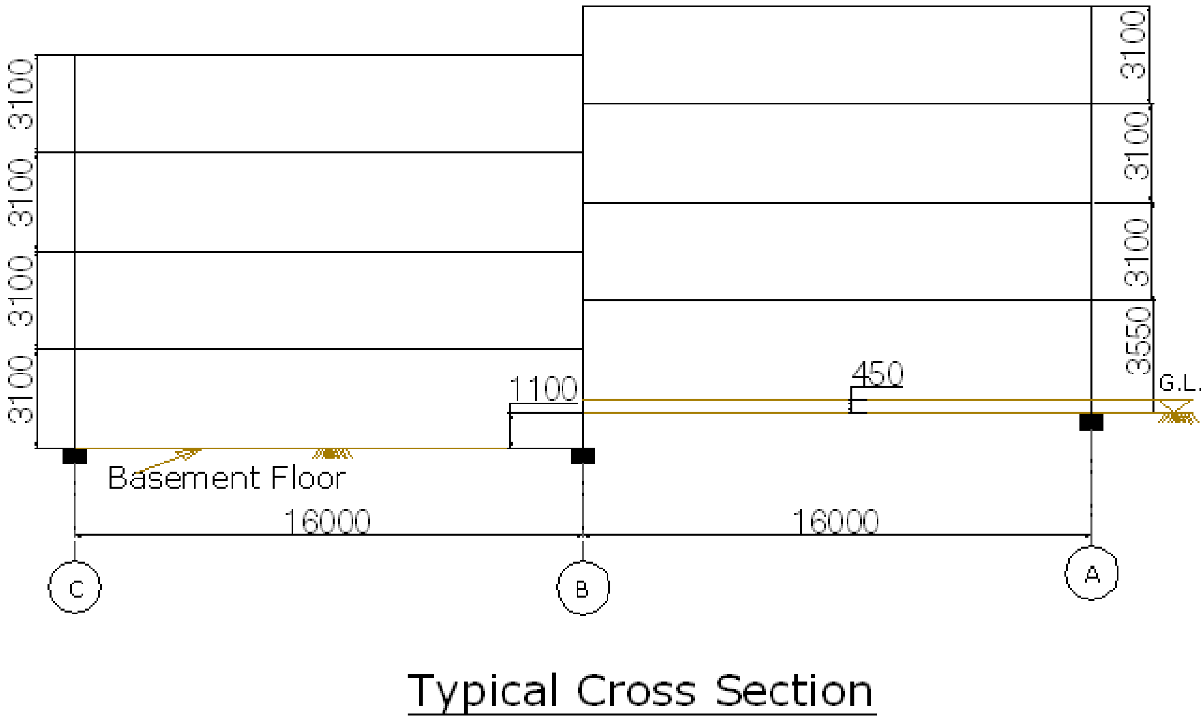


Fig. 5.15 Typical Cross Section

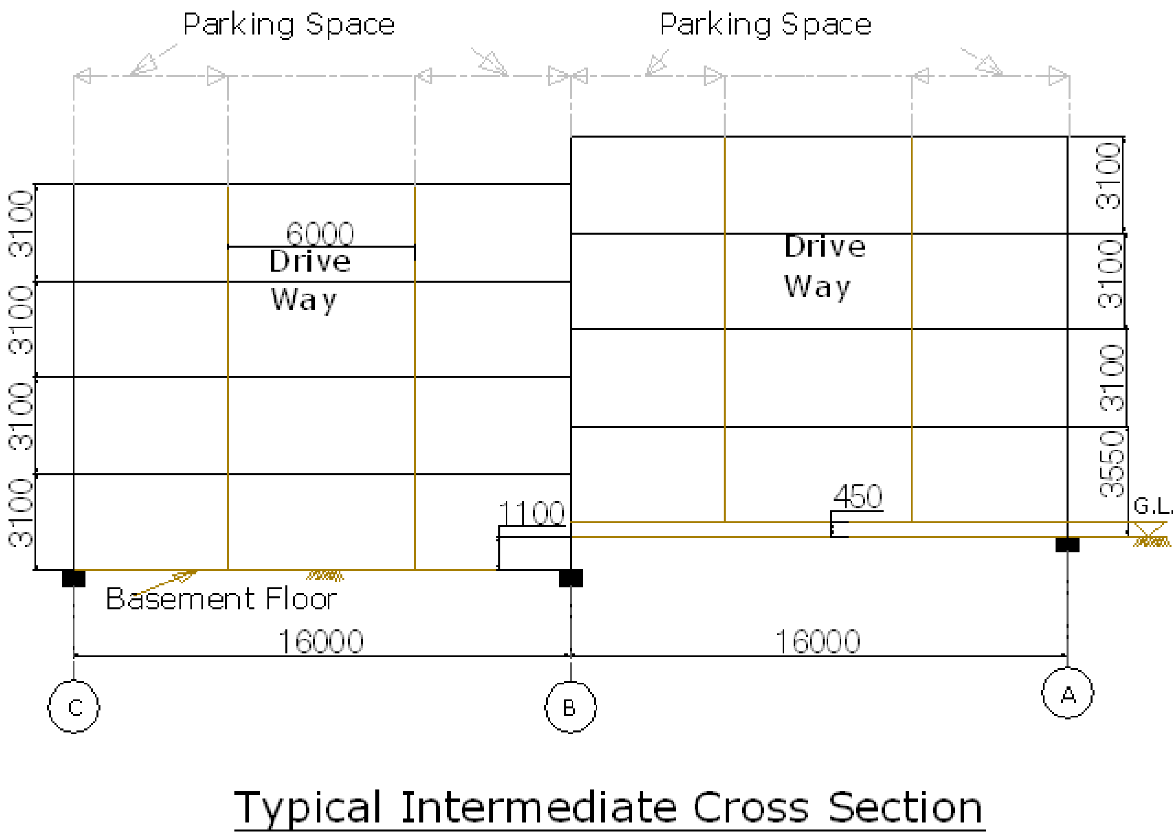


Fig. 5.16 Typical Intermediate Cross Section

5.4 COMPOSITE FLOOR

Composite floors using profiled sheet decking consists of steel beams, steel deck, shear connectors, steel mesh and cast-in-situ concrete in such a manner that they would act monolithically. Composite floor comprises profiled steel decking as the permanent formwork to the underside of concrete floor spanning between support beams. The decking acts compositely with the concrete under service loading. It also supports the loads applied to it before the concrete has gained adequate strength. A steel mesh is placed in the concrete floor to avoid effect of cracks and shrinkage.

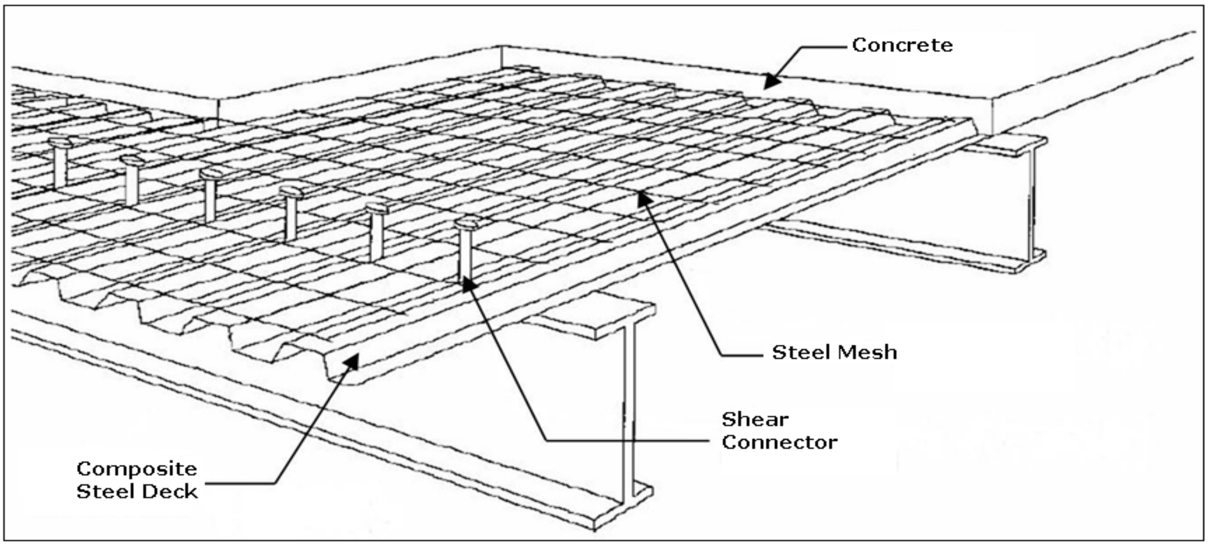


Fig. 5.17 Typical Composite Floor System Arrangement

Fig. 5.18 Installed Deck Floor and Shear Connector at the Site

Here, Fig.5.17 shows the general arrangement of various component of composite deck system. Fig.5.18 shows installed deck floor and shear connector at the actual site. Also in Fig.5.19 the reinforcement provided at the top of the profiled sheeting. These reinforcements control the shrinkage and temperature effects on the slab surface.

Fig.5.19 Shrinkage and temperature reinforcement at top of the profiled sheeting

Based on different study has carried out in India shows that the composite deck slabs has various advantages over RCC slab. Based on this study, the composite deck slab has been considered for multilevel car parks over RCC slab. Detailed has been given in chapter 7.

Design of composite deck has given in appendix A. there is no Indian code available for designing a composite deck slab using profile steel decking, so design has been carried out which suggested in British Standard. Design has been done using the excel work sheet.

5.5 COMPOSITE BEAM

Basics of composite beam have been discussed in chapter three. Here, the beam is designed using Eurocode-4 considering composite deck floor. The detailed design calculation is given in appendix B.

5.6 COLUMN

For Multilevel car parks, the steel column is used because it constructed at highly traffic area. Due to this traffic consideration the fast construction requires for this type of structure. Design of steel column is carried out using STAAD.Pro software.

5.7 ANALYSIS OF MULTILEVEL CAR PARK STRUCTURE

Analysis has been carried out using *STAAD.Pro 2006* version software. Plan geometry of multilevel car park structure which is taken for analysis is given below in Fig. 5.2. Sectional and line diagram of multilevel car park is viewing in Fig. 5.20 and Fig. 5.21 respectively.

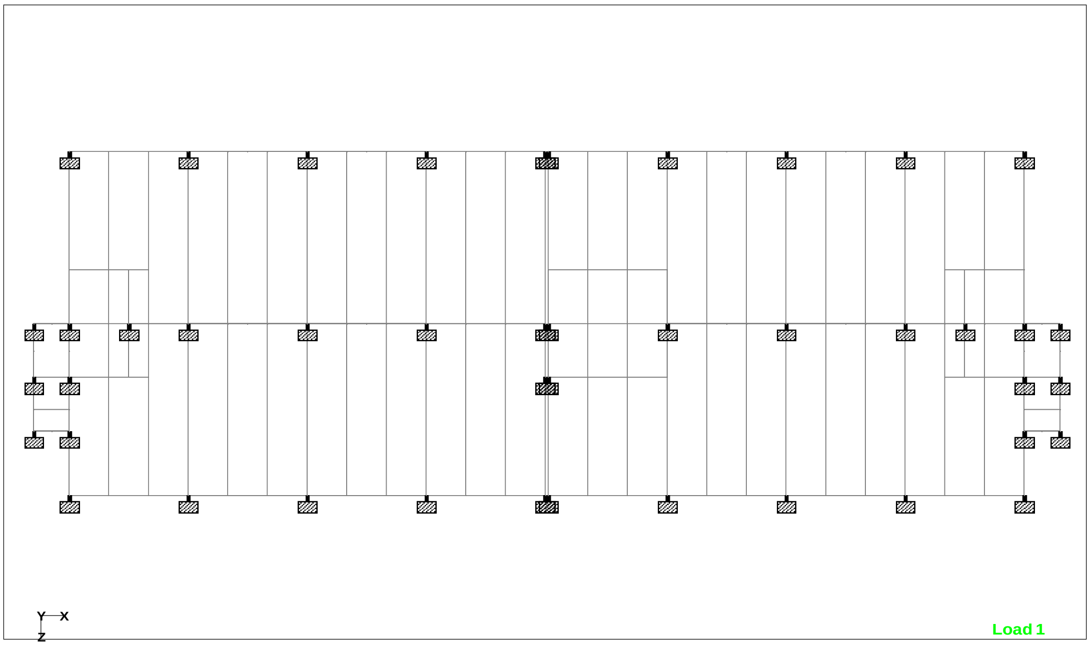


Fig. 5.20 Beam-Column Layout for the STAAD.*Pro* Modeling

5.7.1 Load Consideration

- Earthquake Load Consideration

  As per IS: 1893-2002[17], there are two methods are recommended for the evaluation of the earthquake forces as seismic coefficient method and response reduction method.

  Here, the seismic coefficient method has been used. The procedure of this method has given in code.

- Dead Load

  As per the design worksheet of composite floor, the dead load has been considered. (IS 875: Part-2 and CRIL documents).

- Live Load (From Vehicular wheel load)

  Uniformly Distributed Load 2.5kN/m^2

  Concentrated Load at any point 9.0kN

- Lift Load 10 kN/m^2
- Staircase load is taken as UDL on adjacent beam
- Wall Load 20 kN/m^3

  (Wall provided around the lift block)

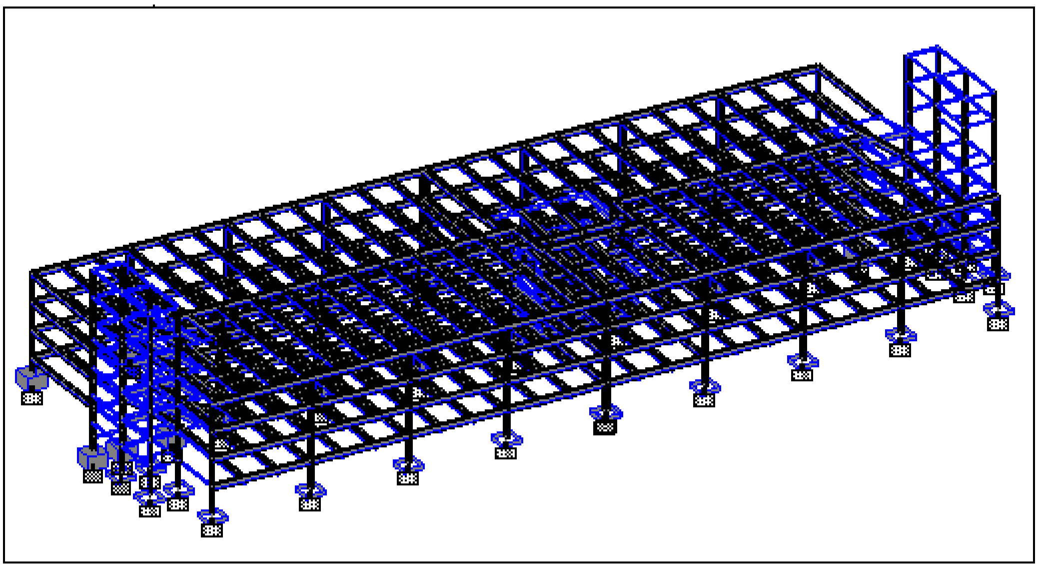


Fig. 5.21 Sectional 3D view of Multilevel Car Park

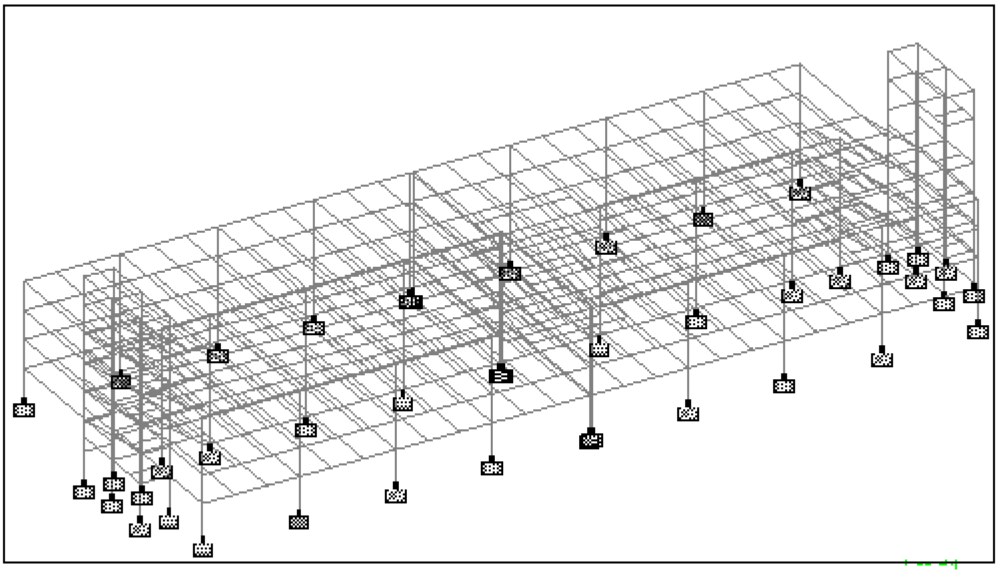


Fig. 5.22 3D line View of Multilevel Car Park

5.8 DESIGN RESULTS

The design of column and periphery beam has been carried out using S*TAAD.Pro 2006* version software. Design results of every floor and column are given below.

Table 5.3 First Floor Beam

Table 5.4 Second Floor Beam

Table 5.5 Third Floor Beam

Table 5.6 Top Floor Beam

Table 5.7 Column Section

5.9 SUMMARY

This chapter includes general arrangement of split level type multilevel car park. The design of floor system is carried out using the profile steel decking and also the economical as well as time dependant aspects has been discussed. The composite beam is designed considering profile steel decking above it. Also the design of column has been done using STAAD.*pro* software. The design results have given of moment resisting frame system in tabular form of each and every floor.

Table 5.3 First Floor Beam (Moment Frame)

Beam Mark	Beam section (ISMB)	Plate		No.of Plate	Weight of Plate	Length of Beam	Weight Of Beam	Total Weight
		Width	Thickness					
B1	600	300	12	2	56.52	10	122.6	1791.2
B2	600	300	12	2	56.52	10	122.6	1791.2
B3	600	300	12	2	56.52	10	122.6	1791.2
B4	600	300	12	2	56.52	10	122.6	1791.2
B5	600	300	16	2	75.36	10	122.6	1979.6
B6	600	300	8	2	37.68	10	122.6	1602.8
B7	600	300	8	2	37.68	10	122.6	1602.8
B8	600	300	8	2	37.68	10	122.6	1602.8
B9	200				0	5	25.4	127
B10	600				0	5	122.6	613
B11	250				0	3.33	37.3	124.209
B12	250				0	3.33	37.3	124.209
B13	250				0	3.33	37.3	124.209
B14	200				0	5	25.4	127
B15	200				0	5	25.4	127
B16	200				0	3	25.4	76.2
B17	600				0	5	122.6	613
B18	600				0	5	122.6	613
B18a	600				0	5	122.6	613
B19	600	300	12	2	56.52	10	122.6	1791.2
B19a	600	300	12	2	56.52	10	122.6	1791.2
B20	600	300	8	2	37.68	10	122.6	1602.8
B20a	600	300	12	2	56.52	10	122.6	1791.2
B21	600				0	10	122.6	1226
B21a	600	300	10	2	47.1	10	122.6	1697
B22	600	400	20	2	125.6	10	122.6	2482
B23	600	300	8	2	37.68	10	122.6	1602.8
B23a	600	300	12	2	56.52	10	122.6	1791.2
B24	600				0	10	122.6	1226
B24a	600	300	12	2	56.52	10	122.6	1791.2
B25	600				0	5	122.6	613
B25a	600				0	5	122.6	613
B26	600				0	5	122.6	613
B27	200				0	3	25.4	76.2
B28	200				0	3	25.4	76.2
B29	200				0	5	25.4	127
B30	550				0	5	103.7	518.5
B31	600				0	3.33	122.6	408.258
B32	200				0	3.33	25.4	84.582
B33	200				0	3.33	25.4	84.582

B34	400				0	5	61.6	308
B35	200				0	5	25.4	127
B36	200				0	3	25.4	76.2
B37	150				0	3	14.9	44.7
B38	150				0	3	14.9	44.7
B39	400				0	3	61.6	184.8
B40	400				0	3	61.6	184.8
B41	600				0	10	122.6	1226
B42	600				0	10	122.6	1226
B43	600				0	10	122.6	1226
B44	600				0	10	122.6	1226
B45	600	300	6	2	28.26	10	122.6	1508.6
B46	600				0	10	122.6	1226
B47	600				0	10	122.6	1226
B48	600				0	10	122.6	1226
B49	400				0	5	61.6	308
B50	150				0	5	14.9	74.5
B51	200				0	6	25.4	152.4
B52	250				0	5	37.3	186.5
B53	250				0	5	37.3	186.5
B54	600				0	16	122.6	1961.6
B56	550				0	5	103.7	518.5
B57	600	300	10	2	47.1	5	122.6	848.5
B58	600				0	5	122.6	613
B59	600				0	5	122.6	613
B60	350				0	11	52.4	576.4
B61	250				0	5	37.3	186.5
B62	500				0	16	86.9	1390.4
B63	600	300	6	2	28.26	11	122.6	1659.46
B64	600				0	5	122.6	613
B65	600	300	6	2	28.26	16	122.6	2413.76
B66	600	300	6	2	28.26	5	122.6	754.3
B67	600				0	5	122.6	613
B68	600				0	5	122.6	613
B69	600				0	5	122.6	613
B70	250				0	5	37.3	186.5
B71	600				0	5	122.6	613
B72	250				0	6	37.3	223.8
B73	300				0	5	44.2	221
B74	250				0	5	37.3	186.5
B75	600				0	5	122.6	613
B76	600	300	8	2	37.68	11	122.6	1763.08
B77	500				0	5	86.9	434.5
B78	300				0	5	44.2	221

Total weight = 69660 kg

Table 5.4 Second Floor Beam (Moment Frame)

Beam Mark	Beam section (ISMB)	Plate		No.of Plate	Weight of Beam	Length of Beam	Weight of Beam	Total Weight
		Width	Thickness					
B1	600				0	10	122.6	1226
B2	600				0	10	122.6	1226
B3	600				0	10	122.6	1226
B4	600				0	10	122.6	1226
B5	600	300	8	2	37.68	10	122.6	1602.8
B6	600	300	6	2	28.26	10	122.6	1508.6
B7	600	300	6	2	28.26	10	122.6	1508.6
B8	600	300	6	2	28.26	10	122.6	1508.6
B9	200				0	5	25.4	127
B10	600				0	5	122.6	613
B11	200				0	3.33	25.4	84.582
B12	200				0	3.33	25.4	84.582
B13	200				0	3.33	25.4	84.582
B14	600				0	5	122.6	613
B15	200				0	5	25.4	127
B16	200				0	3	25.4	76.2
B17	600				0	5	122.6	613
B18	600				0	5	122.6	613
B18a	600				0	5	122.6	613
B19	600	300	10	2	47.1	10	122.6	1697
B19a	600	300	12	2	56.52	10	122.6	1791.2
B20	600	300	8	2	37.68	10	122.6	1602.8
B20a	600	300	12	2	56.52	10	122.6	1791.2
B21	600	300	8	2	37.68	10	122.6	1602.8
B21a	600	300	12	2	56.52	10	122.6	1791.2
B22	600	400	32	2	200.96	10	122.6	3235.6
B23	600	300	12	2	56.52	10	122.6	1791.2
B23a	600	300	12	2	56.52	10	122.6	1791.2
B24	600	300	12	2	56.52	10	122.6	1791.2
B24a	600	300	12	2	56.52	10	122.6	1791.2
B25	600				0	5	122.6	613
B25a	600				0	5	122.6	613
B26	600				0	5	122.6	613
B27	200				0	3	25.4	76.2
B28	250				0	3	37.3	111.9
B29	250				0	5	37.3	186.5
B30	550				0	5	103.7	518.5
B31	600				0	3.33	122.6	408.258
B32	200				0	3.33	25.4	84.582
B33	200				0	3.33	25.4	84.582

B34	400				0	5	61.6	308
B35	250				0	5	37.3	186.5
B36	250				0	3	37.3	111.9
B37	150				0	3		0
B38	150				0	3		0
B39	400				0	3	61.6	184.8
B40	400				0	3	61.6	184.8
B41	600	300	10	2	47.1	10	122.6	1697
B42	600	300	10	2	47.1	10	122.6	1697
B43	600	300	12	2	56.52	10	122.6	1791.2
B44	600	300	12	2	56.52	10	122.6	1791.2
B45	600	300	12	2	56.52	10	122.6	1791.2
B46	600	300	12	2	56.52	10	122.6	1791.2
B47	600	300	12	2	56.52	10	122.6	1791.2
B48	600	300	12	2	56.52	10	122.6	1791.2
B49	500				0	5	86.9	434.5
B50	150				0	5	14.9	74.5
B51	350				0	6	52.4	314.4
B52	300				0	5	44.2	221
B53	250				0	5	37.3	186.5
B54	600	300	8	2	37.68	16	122.6	2564.48
B56	550				0	5	103.7	518.5
B57	600	300	8	2	37.68	5	122.6	801.4
B58	600				0	5	122.6	613
B59	600				0	5	122.6	613
B60	350				0	11	52.4	576.4
B61	200				0	5	25.4	127
B62	600				0	16	122.6	1961.6
B63	600	300	6	2	28.26	11	122.6	1659.46
B64	600				0	5	122.6	613
B65	600	300	6	2	28.26	16	122.6	2413.76
B66	600				0	5	122.6	613
B67	600				0	5	122.6	613
B68	600				0	5	122.6	613
B69	600				0	5	122.6	613
B70	400				0	5	61.6	308
B71	200				0	5	25.4	127
B72	300				0	6	44.2	265.2
B73	400				0	5	61.6	308
B74	250				0	5	37.3	186.5
B75	600				0	5	122.6	613
B76	600	300	6	2	28.26	11	122.6	1659.46
B77	500				0	5	86.9	434.5
B78	300				0	5	44.2	221

Total weight = 74052 kg

Table 5.5 Third Floor Beam (Moment Frame)

Beam Mark	Beam section (ISMB)	Plate		No.of Plate	Weight of Beam	Length Of Beam	Weight Of Beam	Total Weight
		Width	Thickness					
B1	600				0	10	122.6	1226
B2	600	300	8	2	37.68	10	122.6	1602.8
B3	600	300	8	2	37.68	10	122.6	1602.8
B4	600				0	10	122.6	1226
B5	600	300	6	2	28.26	10	122.6	1508.6
B6	600	300	6	2	28.26	10	122.6	1508.6
B7	600	300	6	2	28.26	10	122.6	1508.6
B8	600	300	6	2	28.26	10	122.6	1508.6
B9	200				0	5	25.4	127
B10	600				0	5	122.6	613
B11	200				0	3.33	25.4	84.582
B12	200				0	3.33	25.4	84.582
B13	200				0	3.33	25.4	84.582
B14	600				0	5	122.6	613
B15	175				0	5	19.3	96.5
B16	200				0	3	25.4	76.2
B17	600				0	5	122.6	613
B18	600				0	5	122.6	613
B18a	600				0	5	122.6	613
B19	600	300	12	2	56.52	10	122.6	1791.2
B19a	600	300	12	2	56.52	10	122.6	1791.2
B20	600	300	10	2	47.1	10	122.6	1697
B20a	600	300	8	2	37.68	10	122.6	1602.8
B21	600	300	8	2	37.68	10	122.6	1602.8
B21a	600	300	12	2	56.52	10	122.6	1791.2
B22	600	400	32	2	200.96	10	122.6	3235.6
B23	600	300	12	2	56.52	10	122.6	1791.2
B23a	600	300	12	2	56.52	10	122.6	1791.2
B24	600	300	12	2	56.52	10	122.6	1791.2
B24a	600	300	12	2	56.52	10	122.6	1791.2
B25	600				0	5	122.6	613
B25a	600				0	5	122.6	613
B26	600				0	5	122.6	613
B27	200				0	3	25.4	76.2
B28	250				0	3	37.3	111.9
B29	200				0	5	25.4	127
B30	550				0	5	103.7	518.5
B31	600				0	3.33	122.6	408.258
B32	200				0	3.33	25.4	84.582
B33	200				0	3.33	25.4	84.582

B34	500				0	5	86.9	434.5
B35	500				0	5	86.9	434.5
B36	250				0	3	37.3	111.9
B37	150				0	3	14.9	44.7
B38	150				0	3	14.9	44.7
B39	450				0	3	72.4	217.2
B40	500				0	3	86.9	260.7
B41	600				0	10	122.6	1226
B42	600				0	10	122.6	1226
B43	600				0	10	122.6	1226
B44	600				0	10	122.6	1226
B45	600	300	8	2	37.68	10	122.6	1602.8
B46	600	300	8	2	37.68	10	122.6	1602.8
B47	600	300	6	2	28.26	10	122.6	1508.6
B48	600	300	6	2	28.26	10	122.6	1508.6
B49	450				0	5	72.4	362
B50	150				0	5	14.9	74.5
B51	400				0	6	61.6	369.6
B52	300				0	5	44.2	221
B53	250				0	5	37.3	186.5
B54	450				0	16	72.4	1158.4
B56	550				0	5	103.7	518.5
B57	600	300	8	2	37.68	5	122.6	801.4
B58	600				0	5	122.6	613
B59	600				0	5	122.6	613
B60	550				0	11	103.7	1140.7
B61	400				0	5	61.6	308
B62	500				0	16	86.9	1390.4
B63	600	300	6	2	28.26	11	122.6	1659.46
B64	600				0	5	122.6	613
B65	600	300	6	2	28.26	16	122.6	2413.76
B66	600				0	5	122.6	613
B67	600				0	5	122.6	613
B68	600				0	5	122.6	613
B69	600				0	5	122.6	613
B70	500				0	5	86.9	434.5
B71	600				0	5	122.6	613
B72	250				0	6	37.3	223.8
B73	500				0	5	86.9	434.5
B74	250				0	5	37.3	186.5
B75	500				0	5	86.9	434.5
B76	500				0	11	86.9	955.9
B77	500				0	5	86.9	434.5
B78	300				0	5	44.2	221

Total weight = 70745.5 kg

Table 5.6 Top Floor Beam (Moment Frame)

Beam Mark	Beam section (ISMB)	Plate		No.of Plate	Weight of Beam	Length Of Beam	Weight Of Beam	Total Weight
		Width	Thickness					
B1	600	300	12	2	56.52	10	122.6	1791.2
B2	600	300	12	2	56.52	10	122.6	1791.2
B3	600	300	12	2	56.52	10	122.6	1791.2
B4	600	300	12	2	56.52	10	122.6	1791.2
B5	600	300	12	2	56.52	10	122.6	1791.2
B6	600	300	8	2	37.68	10	122.6	1602.8
B7	600	300	8	2	37.68	10	122.6	1602.8
B8	600	300	10	2	47.1	10	122.6	1697
B9	250				0	5	37.3	186.5
B10	600				0	5	122.6	613
B11	175				0	3.33	19.3	64.269
B12	175				0	3.33	19.3	64.269
B13	175				0	3.33	19.3	64.269
B14	600				0	5	122.6	613
B15	200				0	5	25.4	127
B16	200				0	3	25.4	76.2
B17	600				0	5	122.6	613
B18	600				0	5	122.6	613
B18a	600				0	5	122.6	613
B19	600	300	12	2	56.52	10	122.6	1791.2
B19a	600	300	12	2	56.52	10	122.6	1791.2
B20	600	300	12	2	56.52	10	122.6	1791.2
B20a	600	300	12	2	56.52	10	122.6	1791.2
B21	600	300	10	2	47.1	10	122.6	1697
B21a	600	300	12	2	56.52	10	122.6	1791.2
B22	600	400	32	2	200.96	10	122.6	3235.6
B23	600	300	12	2	56.52	10	122.6	1791.2
B23a	600	300	16	2	75.36	10	122.6	1979.6
B24	600	300	12	2	56.52	10	122.6	1791.2
B24a	600	300	12	2	56.52	10	122.6	1791.2
B25	600				0	5	122.6	613
B25a	600				0	5	122.6	613
B26	600				0	5	122.6	613
B27	200				0	3	25.4	76.2
B28	350				0	3	52.4	157.2
B29	300				0	5	44.2	221
B30	600				0	5	122.6	613
B31	600				0	3.33	122.6	408.258
B32	600				0	3.33	122.6	408.258
B33	600				0	3.33	122.6	408.258

B34	300				0	5	44.2	221
B35	500				0	5	86.9	434.5
B36	500				0	3	86.9	260.7
B37	150				0	3	14.9	44.7
B38	150				0	3	14.9	44.7
B39	400				0	3	61.6	184.8
B40	500				0	3	86.9	260.7
B41	600	300	12	2	56.52	10	122.6	1791.2
B42	600	300	12	2	56.52	10	122.6	1791.2
B43	600	300	12	2	56.52	10	122.6	1791.2
B44	600	300	12	2	56.52	10	122.6	1791.2
B45	600	300	12	2	56.52	10	122.6	1791.2
B46	600	300	16	2	75.36	10	122.6	1979.6
B47	600	300	12	2	56.52	10	122.6	1791.2
B48	600	300	12	2	56.52	10	122.6	1791.2
B49	500				0	5	86.9	434.5
B50	150				0	5	14.9	74.5
B51	400				0	6	61.6	369.6
B52	400				0	5	61.6	308
B53	150				0	5	14.9	74.5
B54	600				0	16	122.6	1961.6
B56	600				0	5	122.6	613
B57	600	300	8	2	37.68	5	122.6	801.4
B58	600				0	5	122.6	613
B59	600				0	5	122.6	613
B60	550				0	11	103.7	1140.7
B61	550				0	5	103.7	518.5
B62	600	300	10	2	47.1	16	122.6	2715.2
B63	600	300	6	2	28.26	11	122.6	1659.46
B64	600				0	5	122.6	613
B65	600	300	6	2	28.26	16	122.6	2413.76
B66	600	300	25	2	117.75	5	122.6	1201.75
B67	600				0	5	122.6	613
B68	600				0	5	122.6	613
B69	600				0	5	122.6	613
B70	250				0	5	37.3	186.5
B71	600				0	5	122.6	613
B72	300				0	6	44.2	265.2
B73	500				0	5	86.9	434.5
B74	250				0	5	37.3	186.5
B75	600				0	5	122.6	613
B76	600	300	6	2	28.26	11	122.6	1659.46
B77	500				0	5	86.9	434.5
B78	300				0	5	44.2	221

Total weight = 81426.3 kg

Table 5.7 Column Section (Moment Frame)

column Mark	Column Section(ISMB)	Plate Width	Plate Thickness	No.of Plate	Weight of Plate	Column Of Length	Weight Of Column	Total Weight
A1	300	300	25	2	117.75	6.2	44.2	1004.09
	300	400	25	2	157	6.15	44.2	1237.38
A2	350	400	25	2	157	6.2	52.4	1298.28
	350	400	32	2	200.96	6.15	52.4	1558.164
A3	350	400	25	2	157	6.2	52.4	1298.28
	350	400	32	2	200.96	6.15	52.4	1558.164
A4	350	400	25	2	157	6.2	52.4	1298.28
	350	400	32	2	200.96	6.15	52.4	1558.164
A5a	300	400	25	2	157	6.2	44.2	1247.44
	300	400	32	2	200.96	6.15	44.2	1507.734
A5b	300	300	12	2	56.52	12.35	44.2	1243.892
A6	350	400	32	2	200.96	6.2	52.4	1570.832
	350	400	40	2	251.2	6.15	52.4	1867.14
A7	350	400	32	2	200.96	6.2	52.4	1570.832
	350	400	40	2	251.2	6.15	52.4	1867.14
A8	350	400	32	2	200.96	6.2	52.4	1570.832
	350	400	40	2	251.2	6.15	52.4	1867.14
A9	300	300	32	2	150.72	6.2	44.2	1208.504
	300	350	32	2	175.84	6.15	44.2	1353.246
A"1'	300				0	3	44.2	132.6
	300	250	20	2	78.5	11	44.2	1349.7
	300	300	20	2	94.2	6.15	44.2	851.16
A"1	300				0	3	44.2	132.6
	300	250	20	2	78.5	11	44.2	1349.7
	300	300	20	2	94.2	6.15	44.2	851.16
A"9	300	350	16	2	87.92	14	44.2	1849.68
	300	300	32	2	150.72	6.15	44.2	1198.758
A"9'	300	350	16	2	87.92	14	44.2	1849.68
	300	300	32	2	150.72	6.15	44.2	1198.758
A'1'	250	200	12	2	37.68	3	37.3	224.94
	250	300	20	2	94.2	4.8	37.3	631.2
	250	200	12	2	37.68	12.35	37.3	926.003
A'1	250	200	12	2	37.68	3	37.3	224.94
	250	300	20	2	94.2	4.8	37.3	631.2
	250	200	12	2	37.68	12.35	37.3	926.003
A'5a	250				0	6.2	37.3	231.26
	250	200	8	2	25.12	6.15	37.3	383.883
A'5b	300	400	40	2	251.2	12.35	44.2	3648.19
A'9	300	350	16	2	87.92	20.15	44.2	2662.218
A'9'	300	350	16	2	87.92	20.15	44.2	2662.218

B1'	250	180	8	2	22.608	3	37.3	179.724
	250	200	12	2	37.68	3.2	37.3	239.936
	250	180	8	2	22.608	13.95	37.3	835.7166
B1	300	300	20	2	94.2	20.15	44.2	2788.76
B2'	450	500	40	2	314	12.35	72.4	4772.04
B2	400	500	32	2	251.2	6.2	61.6	1939.36
	400	500	40	2	314	6.15	61.6	2309.94
B3	400	500	32	2	251.2	6.2	61.6	1939.36
	400	500	40	2	314	6.15	61.6	2309.94
B4	600	400	20	2	125.6	3.1	122.6	769.42
	600	400	40	2	251.2	9.25	122.6	3457.65
B5a	350	400	32	2	200.96	6.2	52.4	1570.832
	350	400	40	2	251.2	6.15	52.4	1867.14
B5b	300	200	10	2	31.4	5.4	44.2	408.24
	300	250	25	2	98.125	6.95	44.2	989.1588
B6	400	520	40	2	326.56	12.35	61.6	4793.776
B7	400	500	32	2	251.2	6.2	61.6	1939.36
	400	520	40	2	326.56	6.15	61.6	2387.184
B8	400	500	32	2	251.2	6.2	61.6	1939.36
	400	520	40	2	326.56	6.15	61.6	2387.184
B8'	350	400	25	2	157	7.75	52.4	1622.85
	350	480	40	2	301.44	4.6	52.4	1627.664
B9	350	300	12	2	56.52	20.15	52.4	2194.738
B9'	250	200	12	2	37.68	20.15	37.3	1510.847
C1	300	400	25	2	157	10.8	44.2	2172.96
C2	350	400	20	2	125.6	6.2	52.4	1103.6
	350	400	25	2	157	4.6	52.4	963.24
C3	350	400	20	2	125.6	6.2	52.4	1103.6
	350	400	25	2	157	4.6	52.4	963.24
C4	350	400	20	2	125.6	6.2	52.4	1103.6
	350	400	25	2	157	4.6	52.4	963.24
C5a	300	400	32	2	200.96	10.8	44.2	2647.728
C5b	300	350	32	2	175.84	10.8	44.2	2376.432
C6	350	400	20	2	125.6	6.2	52.4	1103.6
	350	400	25	2	157	4.6	52.4	963.24
C7	350	400	20	2	125.6	6.2	52.4	1103.6
	350	400	25	2	157	4.6	52.4	963.24
C8	350	400	20	2	125.6	6.2	52.4	1103.6
	350	400	25	2	157	4.6	52.4	963.24
C9	300	350	32	2	175.84	10.8	44.2	2376.432

Total weight = 120356 kg

6. DESIGN OF PARKING STRUCTURE WITH BF SYSTEM

6.1 GENERAL

The braced frame is a common system employed to resist the significant lateral loads that exceptionally tall structures are subjected to. The advantages of braced frames from a structural engineering standpoint are enormous. Braced frames carry the lateral forces in an axial manner than through the bending of elements which is highly inefficient. The largest drawback for braced frames is that the scheme is obstructive and significantly reduces openings within bays. But in the case of multilevel car parks, the requirement of opening is not much. However, it should be noted that while the bracing element covers a significant portion in criteria bays, the separation of the lateral system from vertical system leads to much large column spacing which allows more flexibility in programming of the interior space within those bays.

Here, also the same general consideration has been taken which are describes in chapter 5. Like design methodology, split level type system, profile steel decking, beam and column. In this chapter, the braced frame analysis and design will be studied.

6.2 PLAN GEOMETRY

The plan geometry of bracing frame is same as the moment frame plan geometry. In this frame, the “Tata Structura” hollow rectangular section is used for bracing member. The following figures indicate the general arrangement, plan, elevation and cross section details including marking of beams, columns and bracing of 5 levels split level car park considered for design.

Fig. 5.4 Ground Floor Plan	Fig. 5.5 Typical Floor Plan
Fig. 5.6 Top Floor Plan	Fig. 6.1 Elevation on Row A & C
Fig. 6.2 Elevation on Row B	Fig. 6.3 Cross Section on Line 5a
Fig. 6.4 Cross Section on Line 5b	Fig. 6.5 Roof Plan & Plan 2-2
Fig. 6.6 Cross Section on Line 1 & 9	Fig. 6.7 Elevation on Row A’’
Fig. 6.8 Cross Section on Line 1’ & 9’	Fig. 6.9 Typical Cross Section

Fig. 6.10 Typical Intermediate Cross Section

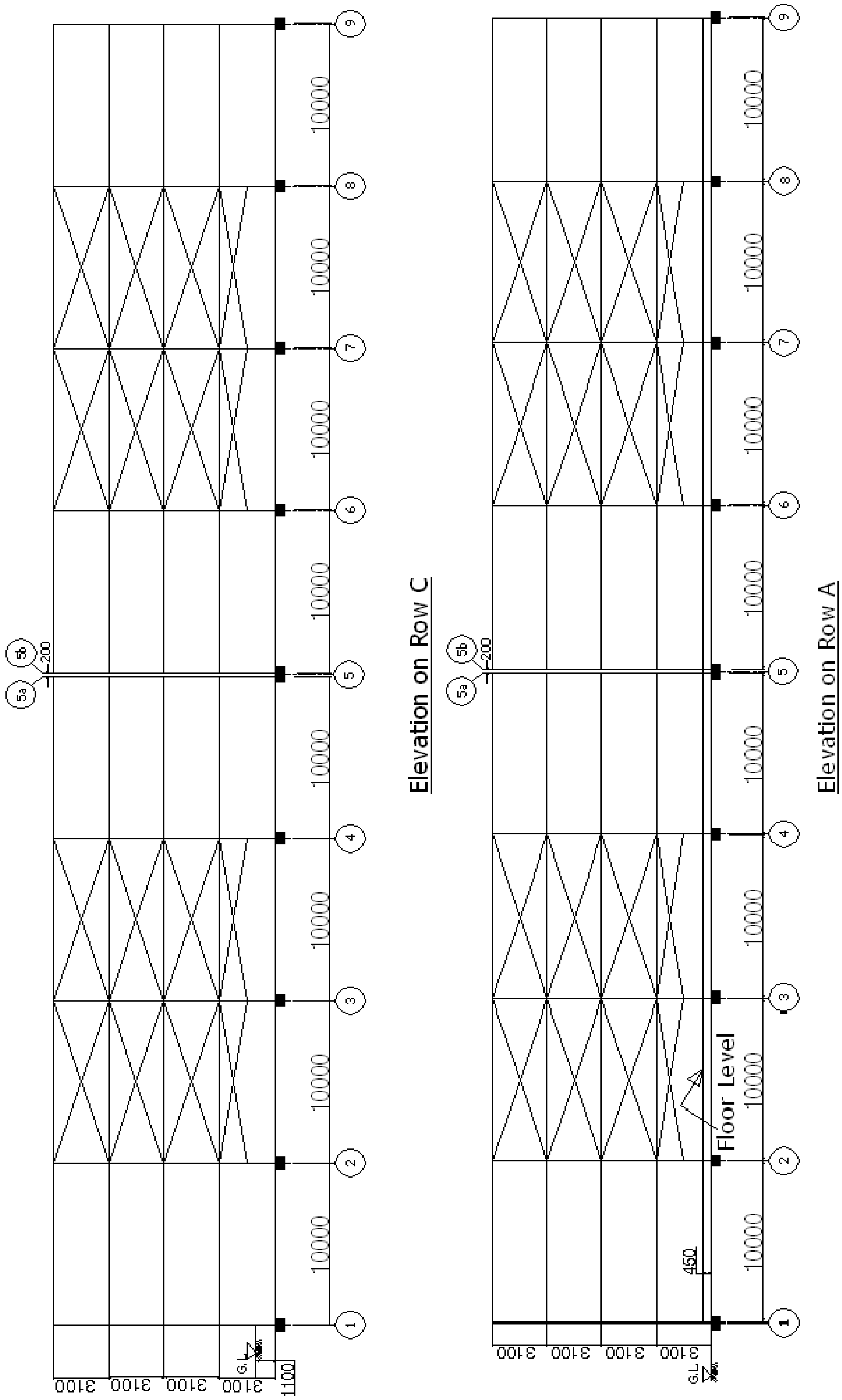


Fig. 6.1 Elevation on Row A & C

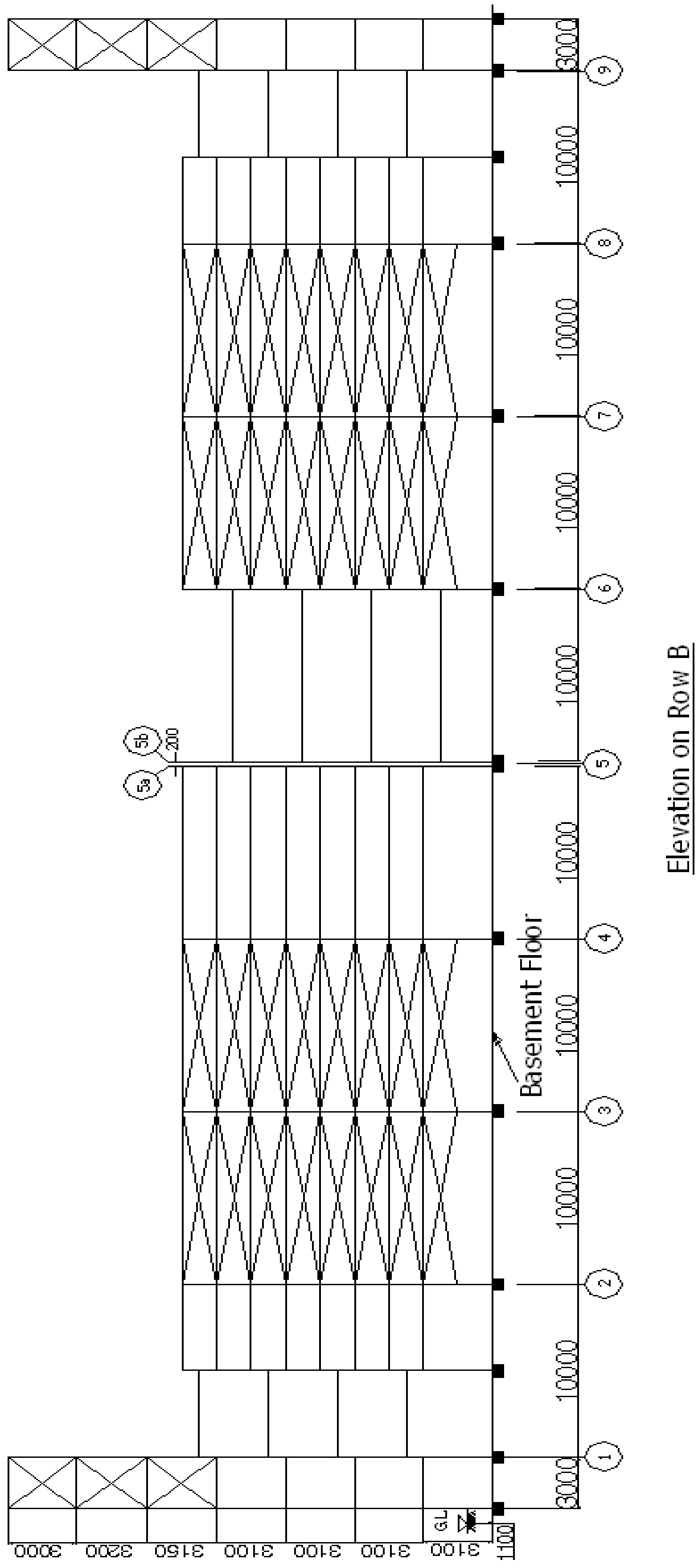


Fig. 6.2 Elevation on Row B

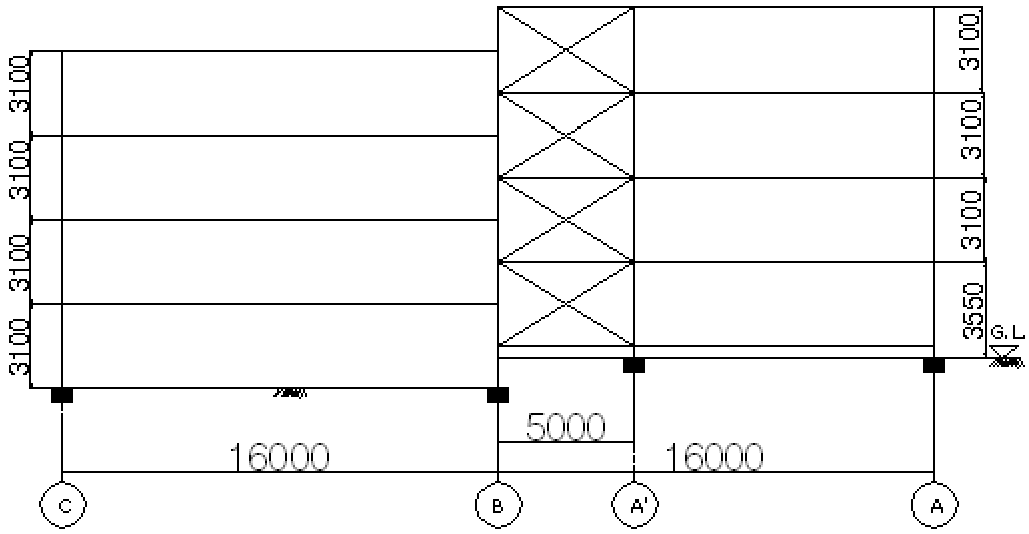


Fig. 6.3 Cross Section on Line 5a

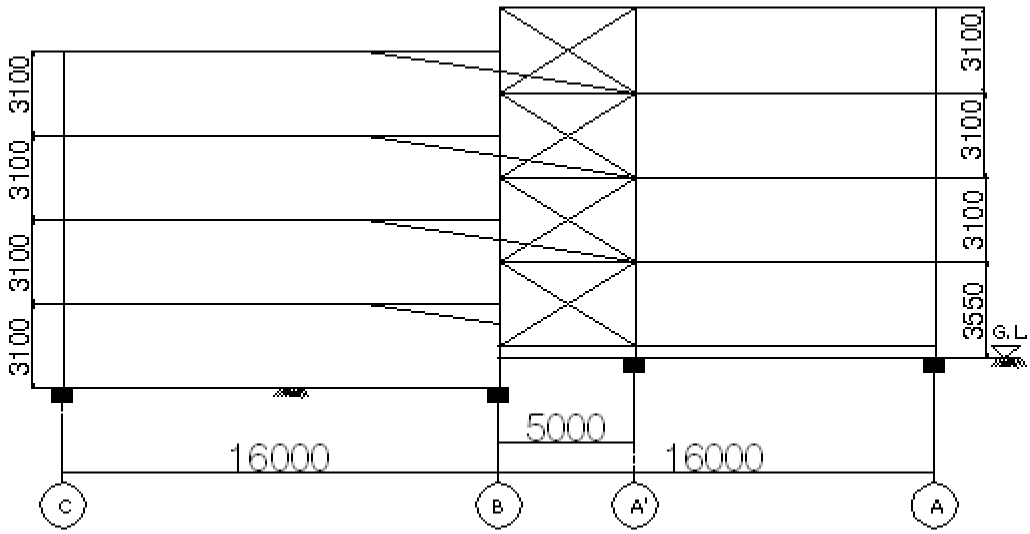


Fig. 6.4 Cross Section on Line 5b

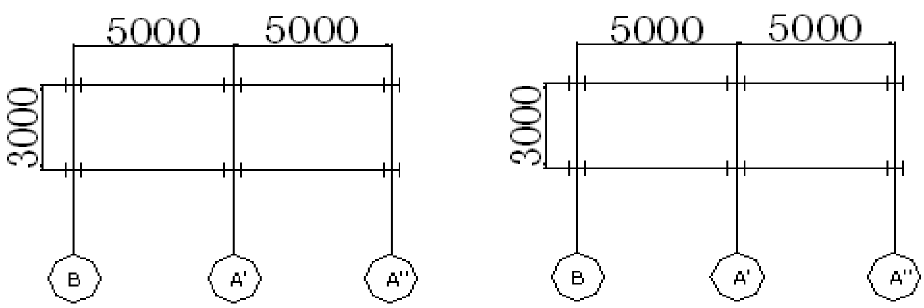


Fig. 6.5 Roof Plan & Plan 2-2

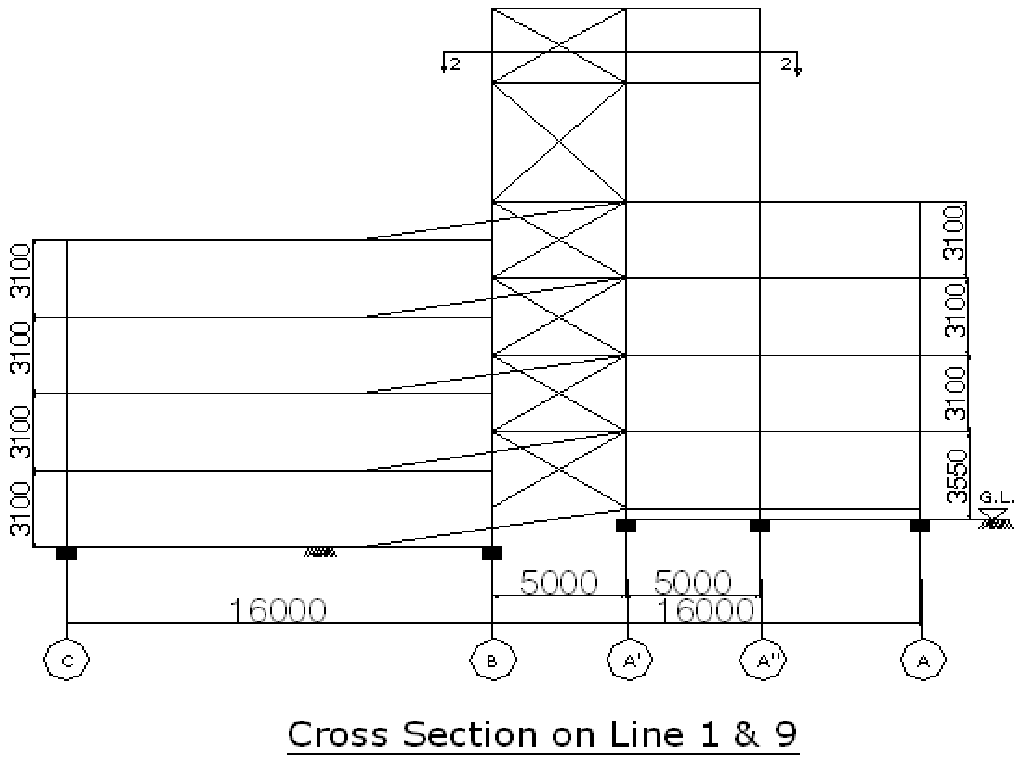
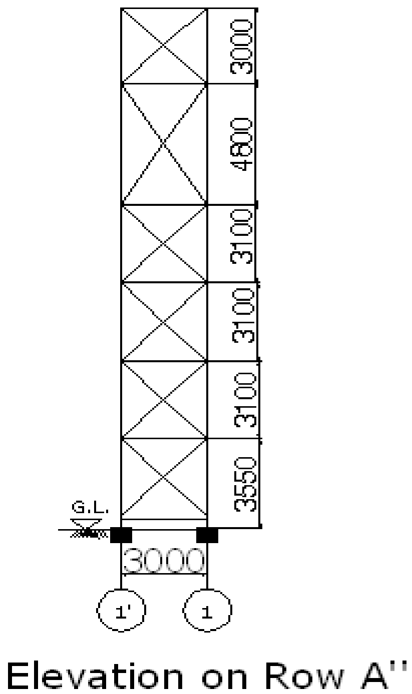


Fig. 6.6 Cross Section on Line 1 & 9



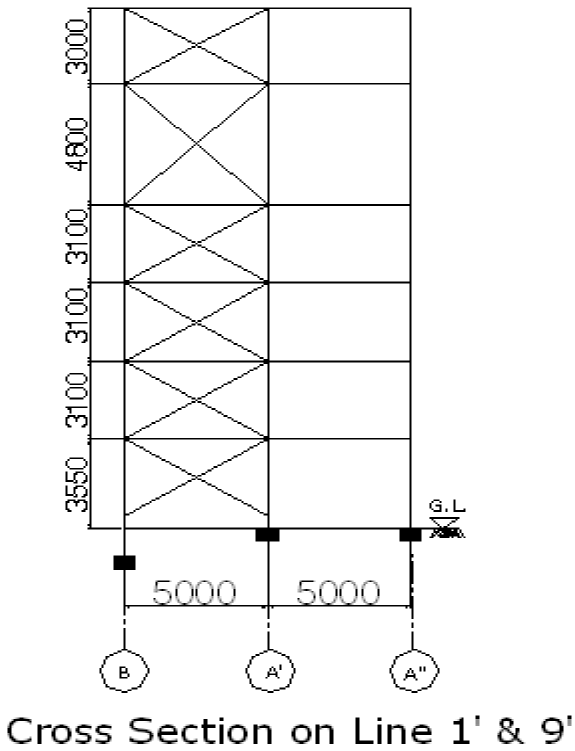


Fig. 6.7 Elevation on Row A’’

Fig. 6.8 Cross Section on Line 1’ & 9’

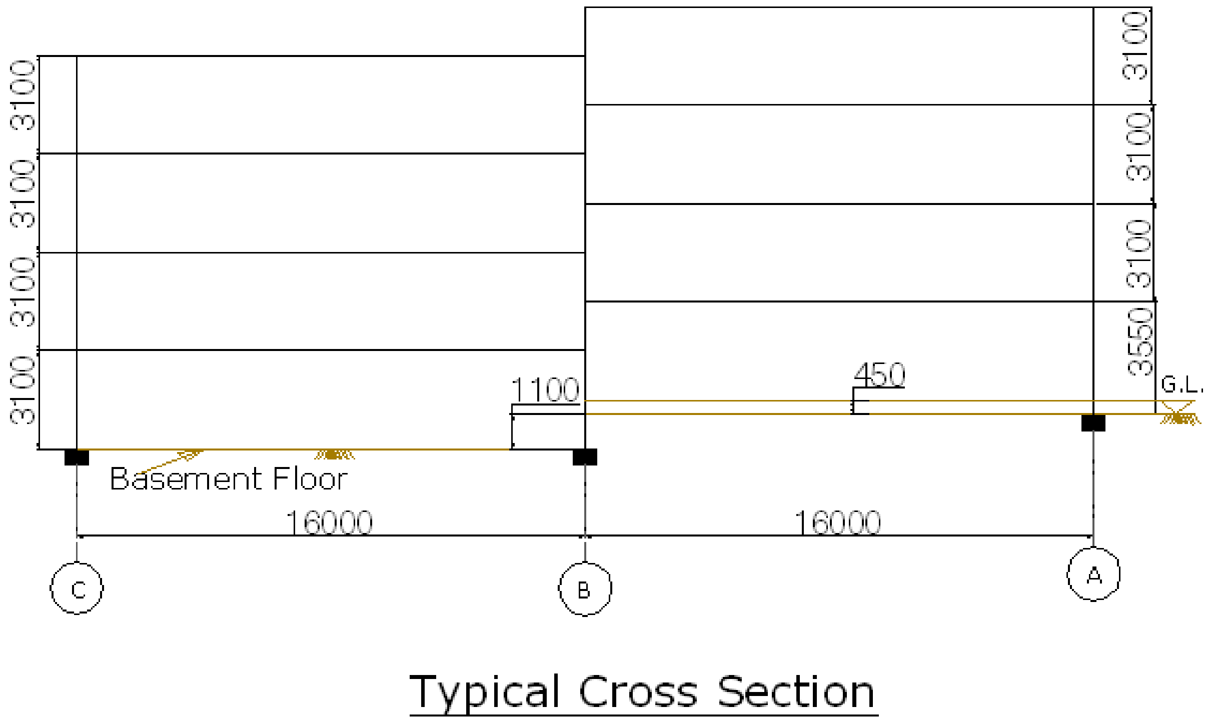


Fig. 6.9 Typical Cross Section

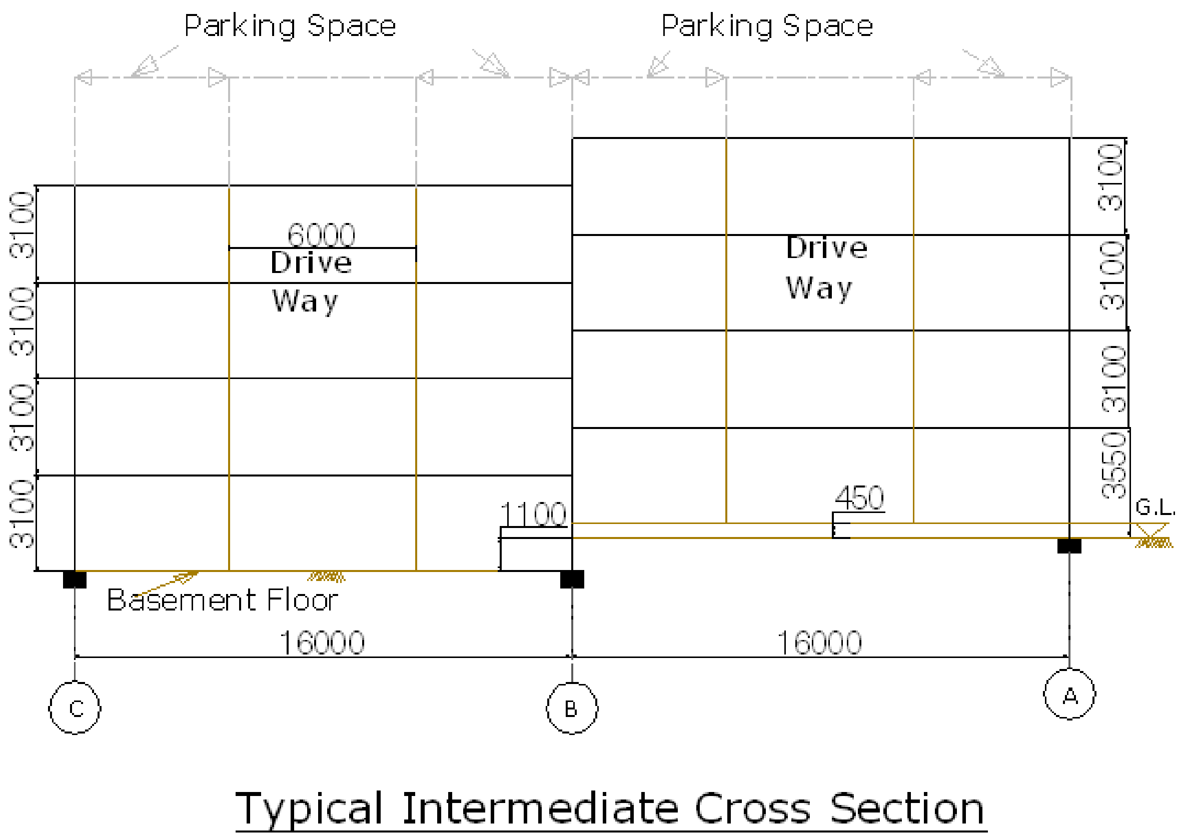


Fig. 6.10 Typical Intermediate Cross Section

6.3 ANALYSIS OF MULTILEVEL CAR PARK STRUCTURE

Analysis has been carried out using STAAD.*Pro 2006* version software. Plan geometry of multilevel car park structure which is taken for analysis is given below (fig. 6.11). Sectional and line diagram of multilevel car park is viewing in fig. 6.12 and fig. 6.13 respectively.

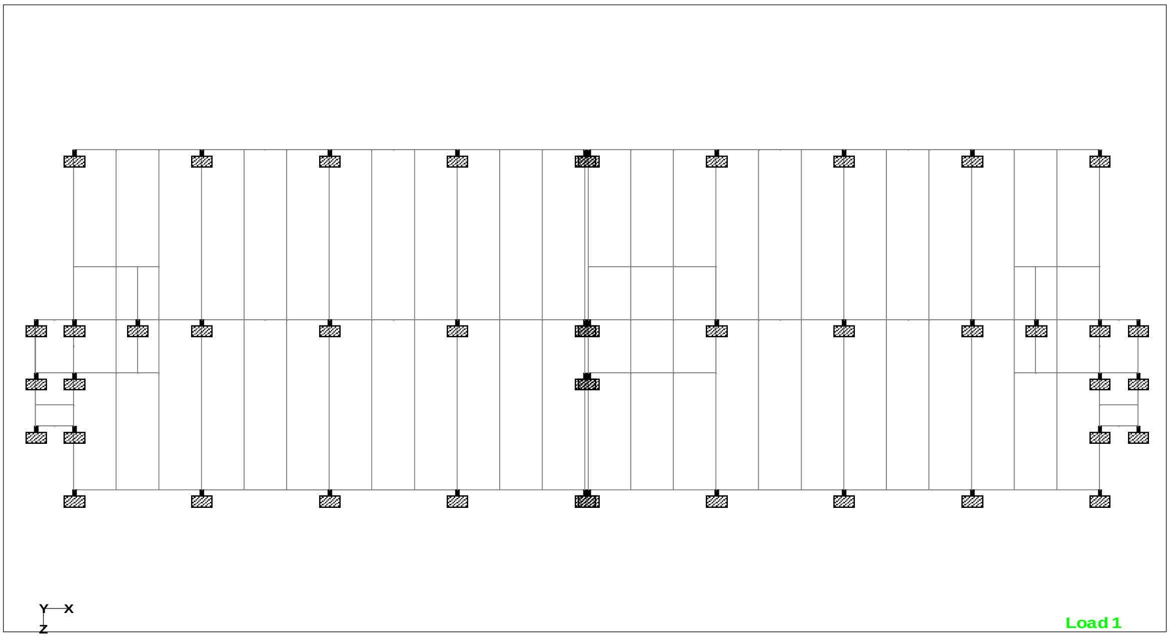


Fig. 6.11 Beam-Column Layout for the STAAD.Pro Modeling

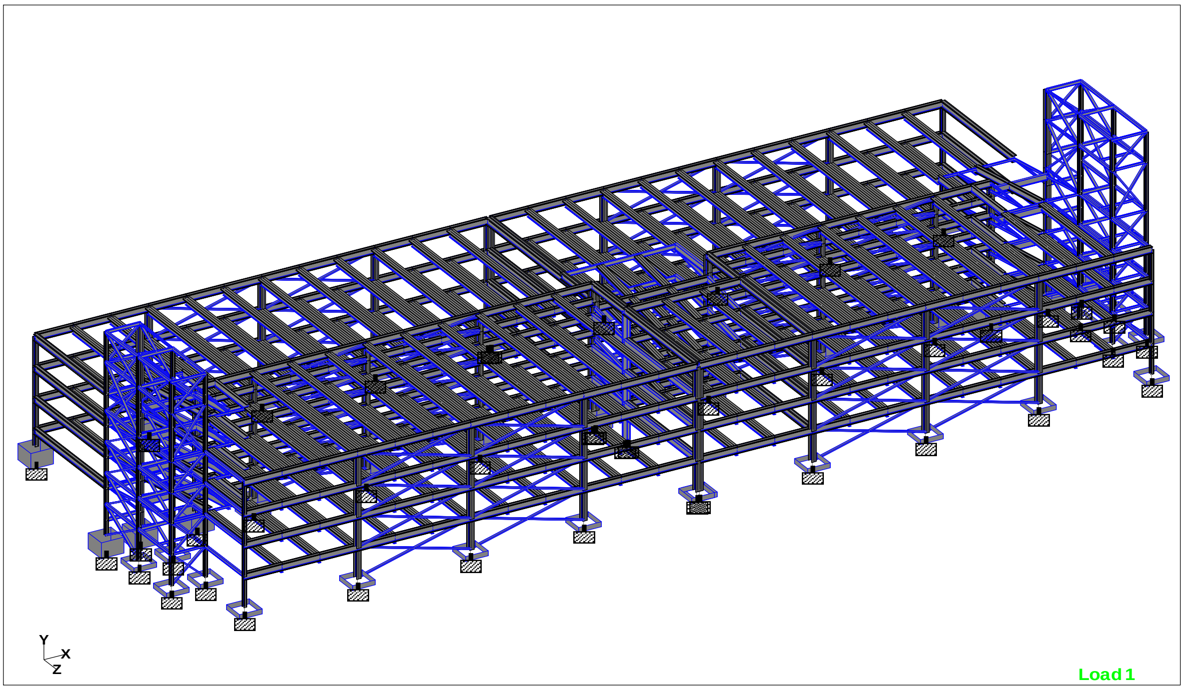


Fig. 6.12 Sectional 3D view of Multilevel Car Park

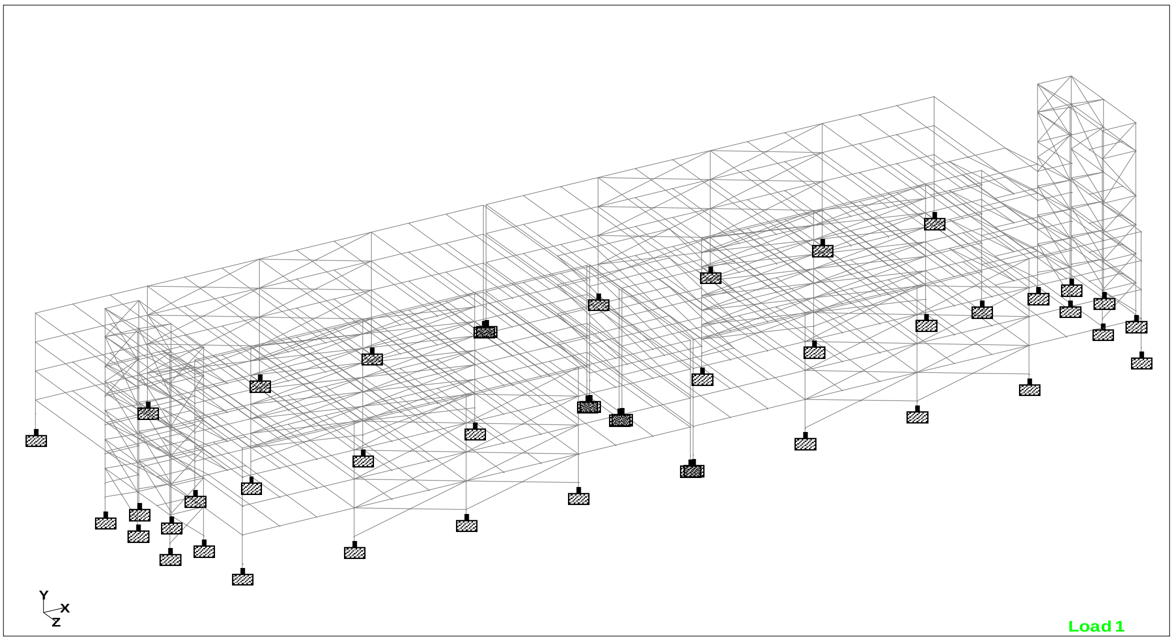


Fig. 6.13 3D line View of Multilevel Car Park

6.4 DESIGN RESULTS

The design of column and periphery beam has been carried out using S*TAAD.Pro 2006* version software. Design results of every floor and column are given below.

Table 6.1 First Floor Beam

Table 6.2 Second Floor Beam

Table 6.3 Third Floor Beam

Table 6.4 Top Floor Beam

Table 6.5 Column Section

6.5 SUMMARY

This chapter includes general arrangement of split level type multilevel car park. Design of floor system, beams and columns are carried out as per describe in chapter 5. The design results have given of Braced frame system in tabular form of each and every floor.

Table 6.1 First Floor Beam (Braced Frame)

Beam Mark	Beam section (ISMB)	Plate		No.of Plate	Weight of Plate	Beam Length	Weight Beam	Total Weight
		Width	Thickness					
B1	600	300	6	2	28.26	10	122.6	1508.6
B2	600	300	6	2	28.26	10	122.6	1508.6
B3	600	300	6	2	28.26	10	122.6	1508.6
B4	600	300	8	2	37.68	10	122.6	1602.8
B5	600	300	6	2	28.26	10	122.6	1508.6
B6	600	300	8	2	37.68	10	122.6	1602.8
B7	600	300	8	2	37.68	10	122.6	1602.8
B8	600	300	8	2	37.68	10	122.6	1602.8
B9	200				0	5	25.4	127
B10	600				0	5	122.6	613
B11	250				0	3.33	37.3	124.209
B12	250				0	3.33	37.3	124.209
B13	250				0	3.33	37.3	124.209
B14	600				0	5	122.6	613
B15	200				0	5	25.4	127
B16	200				0	3	25.4	76.2
B17	600				0	5	122.6	613
B18	600				0	5	122.6	613
B18a	600				0	5	122.6	613
B19	600	300	6	2	28.26	10	122.6	1508.6
B19a	600	300	6	2	28.26	10	122.6	1508.6
B20	600				0	10	122.6	1226
B20a	600	300	6	2	28.26	10	122.6	1508.6
B21	600				0	10	122.6	1226
B21a	600	300	6	2	28.26	10	122.6	1508.6
B22	600	400	20	2	125.6	10	122.6	2482
B23	600	300	6	2	28.26	10	122.6	1508.6
B23a	600				0	10	122.6	1226
B24	600				0	10	122.6	1226
B24a	600	300	6	2	28.26	10	122.6	1508.6
B25	600				0	5	122.6	613
B25a	600				0	5	122.6	613
B26	600				0	5	122.6	613
B27	200				0	3	25.4	76.2
B28	200				0	3	25.4	76.2
B29	200				0	5	25.4	127
B30	450				0	5	72.4	362
B31	500				0	3.33	86.9	289.377
B32	200				0	3.33	25.4	84.582
B33	200				0	3.33	25.4	84.582

Continued…..

B34	600				0	5	122.6	613
B35	200				0	5	25.4	127
B36	200				0	3	25.4	76.2
B37	125				0	3	13	39
B38	150				0	3	14.9	44.7
B39	125				0	3	13	39
B40	150				0	3	14.9	44.7
B41	600				0	10	122.6	1226
B42	600				0	10	122.6	1226
B43	600				0	10	122.6	1226
B44	600				0	10	122.6	1226
B45	600	300	6	2	28.26	10	122.6	1508.6
B46	600				0	10	122.6	1226
B47	600				0	10	122.6	1226
B48	600				0	10	122.6	1226
B49	200				0	5	25.4	127
B50	100				0	5	11.5	57.5
B51	200				0	6	25.4	152.4
B52	225				0	5	31.2	156
B53	100				0	5	11.5	57.5
B54	500				0	16	86.9	1390.4
B56	400				0	5	72.4	362
B57	600	300	6	2	28.26	5	122.6	754.3
B58	600				0	5	122.6	613
B59	600				0	5	122.6	613
B60	350				0	11	52.4	576.4
B61	175				0	5	19.3	96.5
B62	600	300	6	2	28.26	16	122.6	2413.76
B63	600	300	6	2	28.26	11	122.6	1659.46
B64	600				0	5	122.6	613
B65	600	300	6	2	28.26	16	122.6	2413.76
B66	600	300	6	2	28.26	5	122.6	754.3
B67	600	300	6	2	28.26	5	122.6	754.3
B68	600				0	5	122.6	613
B69	600				0	5	122.6	613
B70	250				0	5	37.3	186.5
B71	600				0	5	122.6	613
B72	250				0	6	37.3	223.8
B73	250				0	5	37.3	186.5
B74	200				0	5	25.4	127
B75	600				0	5	122.6	613
B76	600	300	6	2	28.26	11	122.6	1659.46
B77	200				0	5	25.4	127
B78	150				0	5	14.9	74.5

Total= 65265.51 kg

Table 6.2 Second Floor Beam (Braced Frame)

Beam Mark	Beam section (ISMB)	Plate		No.of Plate	Weight of Plate	Beam Length	Weight Beam	Total Weight
		Width	Thickness					
B1	600	300	6	2	28.26	10	122.6	1508.6
B2	600				0	10	122.6	1226
B3	600				0	10	122.6	1226
B4	600	300	6	2	28.26	10	122.6	1508.6
B5	600	300	8	2	37.68	10	122.6	1602.8
B6	600	300	6	2	28.26	10	122.6	1508.6
B7	600				0	10	122.6	1226
B8	600	300	6	2	28.26	10	122.6	1508.6
B9	200				0	5	25.4	127
B10	600				0	5	122.6	613
B11	175				0	3.33	19.3	64.269
B12	200				0	3.33	25.4	84.582
B13	175				0	3.33	19.3	64.269
B14	600	300	6	2	28.26	5	122.6	754.3
B15	200				0	5	25.4	127
B16	200				0	3	25.4	76.2
B17	600				0	5	122.6	613
B18	600				0	5	122.6	613
B18a	600				0	5	122.6	613
B19	600	300	6	2	28.26	10	122.6	1508.6
B19a	600	300	6	2	28.26	10	122.6	1508.6
B20	600	300	6	2	28.26	10	122.6	1508.6
B20a	600	300	8	2	37.68	10	122.6	1602.8
B21	600	300	6	2	28.26	10	122.6	1508.6
B21a	600	300	6	2	28.26	10	122.6	1508.6
B22	600	400	32	2	200.96	10	122.6	3235.6
B23	600				0	10	122.6	1226
B23a	600	300	8	2	37.68	10	122.6	1602.8
B24	600	300	6	2	28.26	10	122.6	1508.6
B24a	600	300	8	2	37.68	10	122.6	1602.8
B25	600				0	5	122.6	613
B25a	600				0	5	122.6	613
B26	600				0	5	122.6	613
B27	200				0	3	25.4	76.2
B28	200				0	3	25.4	76.2
B29	200				0	5	25.4	127
B30	450				0	5	72.4	362
B31	500				0	3.33	86.9	289.377
B32	200				0	3.33	25.4	84.582
B33	200				0	3.33	25.4	84.582

Continued…..

B34	300				0	5	44.2	221
B35	250				0	5	37.3	186.5
B36	200				0	3	25.4	76.2
B37	125				0	3	13	39
B38	150				0	3	14.9	44.7
B39	125				0	3	13	39
B40	150				0	3	14.9	44.7
B41	600	300	6	2	28.26	10	122.6	1508.6
B42	600				0	10	122.6	1226
B43	600				0	10	122.6	1226
B44	600	300	6	2	28.26	10	122.6	1508.6
B45	600	300	8	2	37.68	10	122.6	1602.8
B46	600	300	6	2	28.26	10	122.6	1508.6
B47	600	300	6	2	28.26	10	122.6	1508.6
B48	600	300	6	2	28.26	10	122.6	1508.6
B49	250				0	5	37.3	186.5
B50	100				0	5	11.5	57.5
B51	250				0	6	37.3	223.8
B52	300				0	5	44.2	221
B53	100				0	5	11.5	57.5
B54	600				0	16	122.6	1961.6
B56	400				0	5	72.4	362
B57	600	300	6	2	28.26	5	122.6	754.3
B58	600				0	5	122.6	613
B59	600				0	5	122.6	613
B60	350				0	11	52.4	576.4
B61	175				0	5	19.3	96.5
B62	600	300	6	2	28.26	16	122.6	2413.76
B63	600	300	6	2	28.26	11	122.6	1659.46
B64	600				0	5	122.6	613
B65	600				0	16	122.6	1961.6
B66	600				0	5	122.6	613
B67	600				0	5	122.6	613
B68	600				0	5	122.6	613
B69	600				0	5	122.6	613
B70	300				0	5	44.2	221
B71	600				0	5	122.6	613
B72	250				0	6	37.3	223.8
B73	300				0	5	44.2	221
B74	200				0	5	25.4	127
B75	450				0	5	72.4	362
B76	450				0	11	72.4	796.4
B77	200				0	5	25.4	127
B78	150				0	5	14.9	74.5
						Total=	66162.88	kg

Table 6.3 Third Floor Beam (Braced Frame)

Beam Mark	Beam section (ISMB)	Plate		No.of Plate	Weight of Plate	Beam Length	Weight Beam	Total Weight
		Width	Thickness					
B1	600	300	6	2	28.26	10	122.6	1508.6
B2	600				0	10	122.6	1226
B3	600				0	10	122.6	1226
B4	600				0	10	122.6	1226
B5	600	300	12	2	56.52	10	122.6	1791.2
B6	600	300	6	2	28.26	10	122.6	1508.6
B7	600	300	6	2	28.26	10	122.6	1508.6
B8	600	300	6	2	28.26	10	122.6	1508.6
B9	200				0	5	25.4	127
B10	600				0	5	122.6	613
B11	175				0	3.33	19.3	64.269
B12	200				0	3.33	25.4	84.582
B13	175				0	3.33	19.3	64.269
B14	600				0	5	122.6	613
B15	200				0	5	25.4	127
B16	200				0	3	25.4	76.2
B17	600				0	5	122.6	613
B18	600				0	5	122.6	613
B18a	600				0	5	122.6	613
B19	600	300	6	2	28.26	10	122.6	1508.6
B19a	600	300	6	2	28.26	10	122.6	1508.6
B20	600	300	6	2	28.26	10	122.6	1508.6
B20a	600				0	10	122.6	1226
B21	600	300	6	2	28.26	10	122.6	1508.6
B21a	600	300	6	2	28.26	10	122.6	1508.6
B22	600	400	32	2	200.96	10	122.6	3235.6
B23	600	300	8	2	37.68	10	122.6	1602.8
B23a	600	300	6	2	28.26	10	122.6	1508.6
B24	600				0	10	122.6	1226
B24a	600	300	6	2	28.26	10	122.6	1508.6
B25	600				0	5	122.6	613
B25a	600				0	5	122.6	613
B26	600				0	5	122.6	613
B27	175				0	3	19.3	57.9
B28	200				0	3	25.4	76.2
B29	200				0	5	25.4	127
B30	450				0	5	72.4	362
B31	500				0	3.33	86.9	289.377
B32	200				0	3.33	25.4	84.582
B33	200				0	3.33	25.4	84.582

Continued…..

B34	250				0	5	37.3	186.5
B35	300				0	5	44.2	221
B36	250				0	3	37.3	111.9
B37	125				0	3	13	39
B38	150				0	3	14.9	44.7
B39	125				0	3	13	39
B40	150				0	3	14.9	44.7
B41	600	300	6	2	28.26	10	122.6	1508.6
B42	600				0	10	122.6	1226
B43	600				0	10	122.6	1226
B44	600	300	6	2	28.26	10	122.6	1508.6
B45	600	300	8	2	37.68	10	122.6	1602.8
B46	600	300	6	2	28.26	10	122.6	1508.6
B47	600	300	6	2	28.26	10	122.6	1508.6
B48	600	300	6	2	28.26	10	122.6	1508.6
B49	225				0	5	31.2	156
B50	100				0	5	11.5	57.5
B51	300				0	6	44.2	265.2
B52	300				0	5	44.2	221
B53	100				0	5	11.5	57.5
B54	450				0	16		0
B56	450				0	5	72.4	362
B57	600	300	6	2	28.26	5	122.6	754.3
B58	600				0	5	122.6	613
B59	600				0	5	122.6	613
B60	450				0	11	72.4	796.4
B61	300				0	5	44.2	221
B62	600	300	6	2	28.26	16	122.6	2413.76
B63	600	300	6	2	28.26	11	122.6	1659.46
B64	600				0	5	122.6	613
B65	600	300	6	2	28.26	16	122.6	2413.76
B66	600				0	5	122.6	613
B67	600				0	5	122.6	613
B68	600				0	5	122.6	613
B69	600				0	5	122.6	613
B70	250				0	5	37.3	186.5
B71	600				0	5	122.6	613
B72	250				0	6	37.3	223.8
B73	300				0	5	44.2	221
B74	250				0	5	37.3	186.5
B75	600	300	6	2	28.26	5	122.6	754.3
B76	600	300	6	2	28.26	11	122.6	1659.46
B77	200				0	5	25.4	127
B78	150				0	5	14.9	74.5
						Total=	65882.7	kg

Table 6.4 Top Floor Beam (Braced Frame)

Beam Mark	Beam section (ISMB)	Plate		No.of Plate	Weight of Plate	Beam Length	Weight Beam	Total Weight
		Width	Thickness					
B1	600	300	10	2	47.1	10	122.6	1697
B2	600	300	8	2	37.68	10	122.6	1602.8
B3	600	300	8	2	37.68	10	122.6	1602.8
B4	600	300	10	2	47.1	10	122.6	1697
B5	600	300	16	2	75.36	10	122.6	1979.6
B6	600	300	8	2	37.68	10	122.6	1602.8
B7	600	300	8	2	37.68	10	122.6	1602.8
B8	600	300	8	2	37.68	10	122.6	1602.8
B9	225				0	5	31.2	156
B10	600				0	5	122.6	613
B11	175				0	3.33	19.3	64.269
B12	200				0	3.33	25.4	84.582
B13	175				0	3.33	19.3	64.269
B14	600				0	5	122.6	613
B15	200				0	5	25.4	127
B16	175				0	3	19.3	57.9
B17	600				0	5	122.6	613
B18	600				0	5	122.6	613
B18a	600				0	5	122.6	613
B19	600	300	6	2	28.26	10	122.6	1508.6
B19a	600	300	8	2	37.68	10	122.6	1602.8
B20	600	300	6	2	28.26	10	122.6	1508.6
B20a	600	300	8	2	37.68	10	122.6	1602.8
B21	600	300	6	2	28.26	10	122.6	1508.6
B21a	600	300	12	2	56.52	10	122.6	1791.2
B22	600	300	12	2	56.52	10	122.6	1791.2
B23	600	300	16	2	75.36	10	122.6	1979.6
B23a	600	300	12	2	56.52	10	122.6	1791.2
B24	600	300	6	2	28.26	10	122.6	1508.6
B24a	600	300	12	2	56.52	10	122.6	1791.2
B25	600				0	5	122.6	613
B25a	600				0	5	122.6	613
B26	600				0	5	122.6	613
B27	175				0	3	19.3	57.9
B28	300				0	3	44.2	132.6
B29	300				0	5	44.2	221
B30	600				0	5	122.6	613
B31	600				0	3.33	122.6	408.258
B32	600				0	3.33	122.6	408.258
B33	600				0	3.33	122.6	408.258

Continued…..

B34	300				0	5	44.2	221
B35	400				0	5	61.6	308
B36	400				0	3	61.6	184.8
B37	125				0	3	13	39
B38	150				0	3	14.9	44.7
B39	125				0	3	13	39
B40	150				0	3	14.9	44.7
B41	600	300	8	2	37.68	10	122.6	1602.8
B42	600	300	8	2	37.68	10	122.6	1602.8
B43	600	300	8	2	37.68	10	122.6	1602.8
B44	600	300	12	2	56.52	10	122.6	1791.2
B45	600	300	16	2	75.36	10	122.6	1979.6
B46	600	300	16	2	75.36	10	122.6	1979.6
B47	600	300	6	2	28.26	10	122.6	1508.6
B48	600	300	12	2	56.52	10	122.6	1791.2
B49	250				0	5	37.3	186.5
B50	100				0	5	11.5	57.5
B51	250				0	6	37.3	223.8
B52	300				0	5	44.2	221
B53	100				0	5	11.5	57.5
B54	600				0	16	122.6	1961.6
B56	600				0	5	122.6	613
B57	600	300	6	2	28.26	5	122.6	754.3
B58	600				0	5	122.6	613
B59	600				0	5	122.6	613
B60	600				0	11	122.6	1348.6
B61	500				0	5	86.9	434.5
B62	600	300	6	2	28.26	16	122.6	2413.76
B63	600	300	8	2	37.68	11	122.6	1763.08
B64	600				0	5	122.6	613
B65	600	300	6	2	28.26	16	122.6	2413.76
B66	600				0	5	122.6	613
B67	600				0	5	122.6	613
B68	600				0	5	122.6	613
B69	600				0	5	122.6	613
B70	300				0	5	44.2	221
B71	600				0	5	122.6	613
B72	300				0	6	44.2	265.2
B73	400				0	5	61.6	308
B74	150				0	5	14.9	74.5
B75	600				0	5	122.6	613
B76	600	300	6	2	28.26	11	122.6	1659.46
B77	250				0	5	37.3	186.5
B78	200				0	5	25.4	127

Total= 75026.7 kg

Table 6.5 Column (Braced Frame)

column Mark	Column Section(ISMB)	Plate Width	Plate Thickness	No.of Plate	Weight of Plate	Column Length	Weight Column	Total Weight
A1	300	350	12	2	65.94	6.2	44.2	682.868
	300	350	20	2	109.9	6.15	44.2	947.715
A2	300	400	20	2	125.6	12.35	44.2	2097.03
A3	300	350	12	2	65.94	6.2	44.2	682.868
	300	350	20	2	109.9	6.15	44.2	947.715
A4	300	350	20	2	109.9	6.2	44.2	955.42
	300	350	25	2	137.375	6.15	44.2	1116.686
A5a	300	400	25	2	157	12.35	44.2	2484.82
A5b	300	400	16	2	100.48	12.35	44.2	1786.798
A6	350	400	20	2	125.6	12.35	52.4	2198.3
A7	350	350	16	2	87.92	6.2	52.4	869.984
	350	400	20	2	125.6	6.15	52.4	1094.7
A8	350	350	16	2	87.92	6.2	52.4	869.984
	350	400	20	2	125.6	6.15	52.4	1094.7
A9	300	350	16	2	87.92	6.2	44.2	819.144
	300	400	16	2	100.48	6.15	44.2	889.782
A''1'	250				0	3	37.3	111.9
	250	200	6	2	18.84	11	37.3	617.54
	250	200	10	2	31.4	6.15	37.3	422.505
A''1	250				0	3	37.3	111.9
	250	200	6	2	18.84	11	37.3	617.54
	250	200	10	2	31.4	6.15	37.3	422.505
A''9	250	200	12	2	37.68	20.15	37.3	1510.847
A''9'	250	200	12	2	37.68	20.15	37.3	1510.847
A'1'	250	200	10	2	31.4	3	37.3	206.1
	250	250	20	2	78.5	4.8	37.3	555.84
	250	200	10	2	31.4	12.35	37.3	848.445
A'1	250	200	10	2	31.4	3	37.3	206.1
	250	250	20	2	78.5	4.8	37.3	555.84
	250	200	10	2	31.4	12.35	37.3	848.445
A'5a	300	200	8	2	25.12	6.2	44.2	429.784
	300	200	8	2	25.12	6.75	44.2	467.91
A'5b	300	350	32	2	175.84	12.35	44.2	2717.494
A'9	300	200	10	2	31.4	3	44.2	226.8
	300	250	20	2	78.5	4.8	44.2	588.96
	300	200	10	2	31.4	12.35	44.2	933.66
A'9'	300	200	10	2	31.4	3	44.2	226.8
	300	250	20	2	78.5	4.8	44.2	588.96
	300	200	10	2	31.4	12.35	44.2	933.66

Continued…..

B1'	250				0	9.35	37.3	348.755
	250	200	6	2	18.84	6.2	37.3	348.068
	250	200	8	2	25.12	4.6	37.3	287.132
B1	250				0	8.57	37.3	319.661
	250	250	20	2	78.5	3.88	37.3	449.304
	250	300	20	2	94.2	7.7	37.3	1012.55
B2'	400	450	40	2	282.6	12.35	61.6	4250.87
B2	400	450	32	2	226.08	7.75	61.6	2229.52
	400	450	40	2	282.6	4.6	61.6	1583.32
B3	400	400	16	2	100.48	4.65	61.6	753.672
	400	450	32	2	226.08	7.7	61.6	2215.136
B4	400	400	25	2	157	6.2	61.6	1355.32
	400	450	32	2	226.08	6.15	61.6	1769.232
B5a	350	400	32	2	200.96	12.35	52.4	3128.996
B5b	300	200	8	2	25.12	5.4	44.2	374.328
	300	250	20	2	78.5	6.95	44.2	852.765
B6	400	500	40	2	314	12.35	61.6	4638.66
B7	400	500	20	2	157	6.2	61.6	1355.32
	400	500	32	2	251.2	6.15	61.6	1923.72
B8	400	500	25	2	196.25	6.2	61.6	1598.67
	400	500	32	2	251.2	6.15	61.6	1923.72
B8'	350	400	20	2	125.6	6.2	52.4	1103.6
	350	400	32	2	200.96	6.15	52.4	1558.164
B9	350	250	8	2	31.4	9.35	52.4	783.53
	350	300	12	2	56.52	10.8	52.4	1176.336
B9'	250				0	9.35	37.3	348.755
	250	200	6	2	18.84	10.8	37.3	606.312
C1	300	400	20	2	125.6	10.8	44.2	1833.84
C2	350	300	16	2	75.36	6.2	52.4	792.112
	350	400	20	2	125.6	4.6	52.4	818.8
C3	350	300	16	2	75.36	6.2	52.4	792.112
	350	400	20	2	125.6	4.6	52.4	818.8
C4	350	300	16	2	75.36	6.2	52.4	792.112
	350	400	20	2	125.6	4.6	52.4	818.8
C5a	300	400	32	2	200.96	10.8	44.2	2647.728
C5b	300	300	32	2	150.72	6.2	44.2	1208.504
	300	350	32	2	175.84	4.6	44.2	1012.184
C6	350	300	16	2	75.36	6.2	52.4	792.112
	350	400	25	2	157	4.6	52.4	963.24
C7	350	300	16	2	75.36	6.2	52.4	792.112
	350	400	20	2	125.6	4.6	52.4	818.8
C8	350	300	16	2	75.36	6.2	52.4	792.112
	350	400	20	2	125.6	4.6	52.4	818.8
C9	300	350	32	2	175.84	10.8	44.2	2376.432
						Total=	92382.9	kg

Table 6.6 Bracing Member

Line	Section (RHS)	Lentgth	Weight of Section	No.of Member	Total Weight
A(left)	220x140x4	10.45	21.78	16	3641.616
A(right)	240x120x4	10.45	21.78	16	3641.616
B(left)	200x100x4	10.11	18.01	32	5826.595
	96x48x3.2	4.35	6.71	4	116.754
	122x61x3.6	4.39	9.67	2	84.9026
B(right)	200x100x4	10.11	18.01	32	5826.595
	96x48x3.2	4.35	6.71	4	116.754
	96x48x3.2	4.39	9.67	2	84.9026
C(left)	220x140x4	10.47	21.78	12	2736.439
	200x100x4	10.11	18.01	4	728.3244
C(right)	240x120x4	10.47	21.78	12	2736.439
	200x100x4	10.11	18.01	4	728.3244
1(left)	200x100x4	5.86	18.01	6	633.2316
	200x100x4	7.07	18.01	2	254.6614
	122x61x3.6	5.88	9.67	4	227.4384
9(right)	200x100x4	6.93	18.01	6	748.8558
	200x100x4	5.88	18.01	2	211.7976
	122x61x3.6	5.83	9.67	4	225.5044
1'(left)	122x61x3.6	5.86	9.67	10	566.662
	200x100x4	6.93	18.01	2	249.6186
9'(right)	122x61x3.6	5.86	9.67	10	566.662
	200x100x4	6.93	18.01	2	249.6186
A"(left)	200x100x4	5.66	18.01	2	203.8732
	122x61x3.6	4.31	9.67	8	333.4216
	96x48x3.2	4.24	6.71	2	56.9008
A"(right)	200x100x4	5.66	18.01	2	203.8732
	122x61x3.6	4.31	9.67	8	333.4216
	96x48x3.2	4.24	6.71	2	56.9008
5a	122x61x3.6	5.88	9.67	2	113.7192
	200x100x4	5.88	18.01	6	635.3928
5b	122x61x3.6	5.88	9.67	2	113.7192
	200x100x4	5.88	18.01	6	635.3928
			Total =	32889.93	kg

7. COMPARISON OF THE SYSTEMS

7.1 GENERAL

In multi-storey buildings, structural steelwork is typically used together with concrete; for example, steel beams with concrete floor slabs. It is a fact, however, that engineers are increasingly designing composite and mixed building systems of structural steel and reinforced concrete to produce more efficient structures when compared to designs using either material alone. Study says that above three storeys, the composite construction is economical for multilevel car parks. Literature survey shows that in most multilevel car parks the steel framing is used. Purpose behind the use of steel framing is saving the time and achieving the fast construction. So here, the comparison has been carried out considering the two framing system. One is moment resisting frame system and other is braced frame system. For the braced frame, it is find that the x-bracing is economical over the other types of bracing system. And it is better solution for resisting a lateral loads which coming from the either direction.

7.2 COMPOSITE FLOOR COMPARISON

Based on below study, the composite floor using profile sheeting has been considered as the better solution for the floor construction of multilevel car parks. If we considered only the cost than it is more than the simple solid RCC slab but in consideration of time saving it is highly economical.

A G+5 commercial building considered for study is a rectangular (48 x 13.5 m) in plan with nominal height of 16.2m (2.7m floor to floor) and gross floor area of 3900 m^2 (650 m^2 area at each floor) constructed at pune. For considering same total floor area, height and loading condition, this building is designed and constructed by two different methods.

From Study of table (5.3 to 5.5), it is found that direct construction cost required for composite floor is 27.64% higher than RCC floor. But overall the net cost required for composite floor is only 0.45% more than RCC floor considering time related saving. The time saving of 55.29% is achieved due to use of composite floor construction rather than RCC floor.

Table 7.1 Floor to Floor Time Comparison between RCC & Composite Floor Construction

Slab Level	RCC Floor Construction				Composite Floor Construction					
	Erection of Slab & Beam Formwork (Days)	Lifting & Laying of Steel Reinforcement	Concreting (Days)	Total Duration (Days)	Lifting of Steel Decks (Days)	Placing &Installation of Steel Deck (days)	Cleaning of Steel Decks (Days)	Lifting & Placing of Reinforcement (Days)	Concreting (Days)	Total Duration (Days)
1	8	6	2	15	1	3	1	1	1	7
2	9	7	2	17	2	3	1	1	1	8
3	10	8	2	19	2	3	1	2	1	9
4	11	9	2	21	3	3	1	2	1	10
5	13	10	2	24	3	3	1	2	1	10
6	15	11	2	27	4	3	1	2	1	11
Total duration for RCC floor construction =123 days					Total duration for Composite floor construction =55 days					

Table 7.2 Floor to Floor Time Savings

Slab Level	Duration for Composite Floor (Days)	Duration For RCC Floor (Days)	Time Saving (Days)	Time Savings (%)
1	7	15	8	53.33
2	8	17	9	52.94
3	9	19	10	52.63
4	10	21	11	52.38
5	10	24	14	58.33
6	11	27	16	59.26
Total	55	123	68	55.29

Table 7.3 Net Costs for RCC & Composite Floor Construction

Cost	RCC Floor Construction	Composite Floor Construction
Direct Cost	6846150	8737210
11% interest on direct cost (For RCC: 123 days & For Composite: 55 days)	253780	144825
10% interest on 1st installment of selling amount (for 68 days)	1077686.3	
10% interest on 2nd installment of selling amount (for 38 days)	451677.3	
10% interest on 3rd installment of selling amount (for 18 days)	213952.4	
Total net Cost (Rs)	8843246	8882035
Total net Cost (Rs/m^2)	2267.5	2277.5
Extra Cost required for Composite over RCC floor considering net cost (Rs/m^2)	10	
Extra Cost required for Composite over RCC floor considering net cost (%)	0.45(39000)	

7.3 DEFLECTION COMPARISON

Deflections of two systems have been compared at two different heights. Table 5.5 shows that the deflections are reduced in braced frame and it is more in case of moment frame. Deflection of two systems as follows in Table 7.4.

Table 7.4 Net Costs for RCC & Composite Floor Construction

SL	Description	Moment Frame		Braced Frame	
		X	Z	X	Z
1	Top of Lift Room	60	42	36	24
2	Top Floor	52	18	21	21

7.4 WEIGHT COMPARISON

Weight comparison of two systems has been carried out. Detailed weight comparison has given in Table 7.5. The braced frame system is more economical than the moment frame system in case of weight comparison. The weight comparison of two framing system has given below:

Table 7.5 Quantity comparison

SL	Description	MRF Weight in kg	BF Weight in kg
1	First Floor Beam	69660	65265.5
2	Second Floor Beam	74052	66162.88
3	Third Floor Beam	70745.5	65882.7
4	Top Floor Beam	81426.3	75026.65
5	Column	120356	92382.91
6	Ramp Beam	7093.4	11143.84
7	Bracing	0	32889.93
8	Lift Room Beam	4797.8	3401.9
	Total Weight	428131	412156
	Weight Difference	15975	
	% Difference	3.73%	

7.5 FOUNDATION MOMENT COMPARISON

When the bracing is provided in the structure, very less difference is observed in case of dead and live load as shown in Table 7.8 & 7.9. But the moment is reduced in case of earthquake loading at the foundation (Table 7.2 & 7.3). For comparison purpose the forces has been taken at bottom of column of grid line B which values are shown in Table 7.6 to Table 7.9

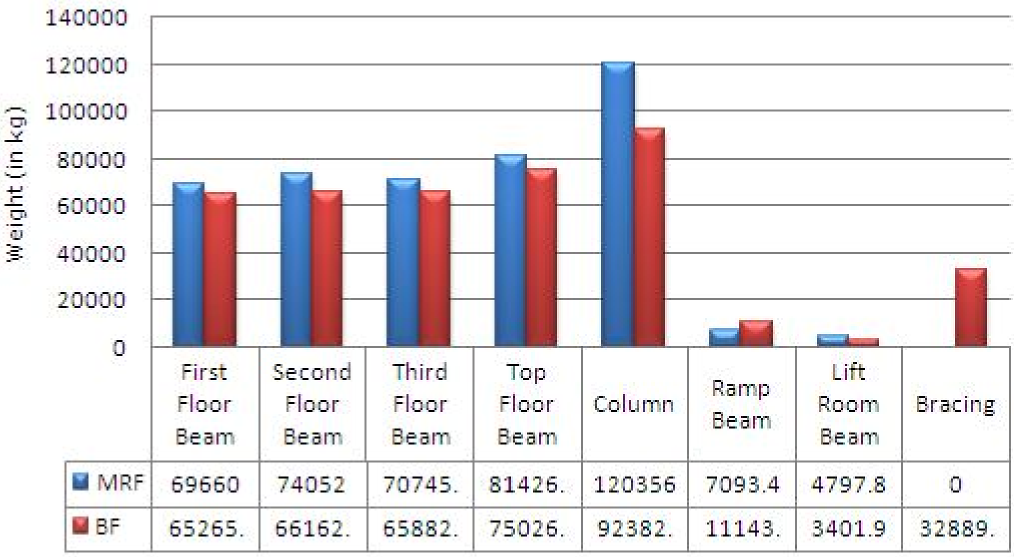


Fig. 7.1 Weight comparison

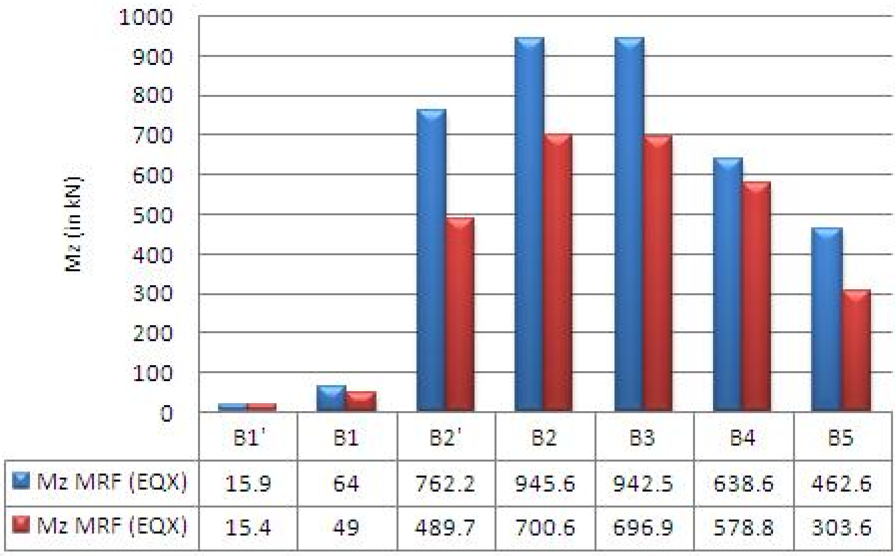


Fig. 7.2 Foundation moment comparison

Table 7.6 Foundation Forces due to EQX

EQX MF						EQX BF					
Fn No.	Fx kN	Fy kN	Fz kN	Mx kNm	Mz kNm	Fn No.	Fx kN	Fy kN	Fz kN	Mx kNm	Mz kNm
B1'	-6.4	-99.2	-2.3	-10.8	15.9	B1'	-6.0	-145.0	-5.7	-10.0	15.4
B1	-18.4	108.9	3.3	8.3	64.0	B1	-14.3	142.5	4.1	6.2	49.0
B2'	-259.7	-636.2	-6.3	-23.2	762.2	B2'	-175.6	-401.1	1.1	2.5	489.7
B2	-348.6	358.5	-4.5	-17.2	945.6	B2	-290.8	97.4	-0.7	-3.0	700.6
B3	-343.9	44.8	0.2	0.8	942.5	B3	-311.9	19.1	-0.6	-2.3	696.9
B4	-242.1	9.4	7.8	33.6	638.6	B4	-241.3	155.6	-1.0	-3.7	578.8
B5	-160.3	254.0	7.1	34.9	462.6	B5	-109.0	158.5	4.9	11.6	303.6

Table 7.7 Foundation Forces due to EQZ

EQZ MF						EQZ BF					
Fn No.	Fx kN	Fy kN	Fz kN	Mx kNm	Mz kNm	Fn No.	Fx kN	Fy kN	Fz kN	Mx kNm	Mz kNm
B1'	-0.1	-21.5	-13.2	-60.9	0.1	B1'	0.0	-425.5	-74.2	-135.9	-0.1
B1	0.0	-239.8	-134.0	-373.1	0.0	B1	0.1	-644.6	-149.6	-305.0	0.0
B2'	-3.9	-1.1	-246.9	-937.8	7.7	B2'	-4.6	10.1	-204.9	-741.0	10.0
B2	1.8	-2.4	-189.4	-735.7	-3.0	B2	-0.3	2.2	-195.9	-745.8	1.6
B3	-0.7	-2.8	-210.2	-818.6	1.8	B3	-2.6	8.9	-215.7	-781.6	5.7
B4	-0.4	-5.1	-303.8	-1373.3	0.9	B4	-2.4	-1.3	-199.4	-722.3	5.6
B5	-0.2	7.2	-98.0	-486.0	0.6	B5	-1.6	-451.4	-159.3	-367.1	4.1

Table 7.8 Foundation Forces due to DL

DL MF						DL BF					
Fn No.	Fx kN	Fy kN	Fz kN	Mx kNm	Mz kNm	Fn No.	Fx kN	Fy kN	Fz kN	Mx kNm	Mz kNm
B1'	5	343.1	0.3	1.6	-10.4	B1'	5.9	359.8	8.9	15.1	-12.3
B1	-8.0	1006.0	-35.7	-55.7	16.5	B1	-6.9	1040.1	-2.0	-16.3	14.0
B2'	24.2	2288.8	-0.1	149.7	-54.2	B2'	21.7	2233.3	-18.9	87.8	-49.9
B2	88.0	2239.2	-25.5	28.9	-182.8	B2	92.6	2287.8	-28.2	19.0	-189.5
B3	0.8	2994.4	-26.4	25.4	-4.3	B3	2.8	3005.2	-30.7	6.6	-6.0
B4	-1.2	3027.5	-23.4	40.9	3.7	B4	-8.5	3001.7	-32.5	1.6	17.1
B5	-111.6	1365.9	-35.3	-28.8	233.4	B5	-109	1380.0	-24.3	-11.6	228.0

Table 7.9 Foundation Forces due to LL

LL MF						LL BF					
Fn No.	Fx kN	Fy kN	Fz kN	Mx kNm	Mz kNm	Fn No.	Fx kN	Fy kN	Fz kN	Mx kNm	Mz kNm
B1'	2.0	65.1	0.1	0.5	-4.2	B1'	2.4	69.3	2.6	4.3	-5.1
B1	-3.1	408.5	-19.8	-31.9	6.3	B1	-2.5	431.8	-5.0	-15.5	4.9
B2'	13.6	1194.6	-0.4	85.4	-30.5	B2'	12.2	1168.7	-10.4	52.9	-28.1
B2	51.4	1192.0	-15.9	16.6	-106.6	B2	52.6	1220.8	-17.4	11.1	-108.5
B3	0.7	1634.3	-16.3	15.2	-2.9	B3	2.0	1641.8	-18.9	3.8	-4.2
B4	0.0	1656.6	-14.4	24.9	0.9	B4	-3.1	1648.3	-20.0	0.7	7.2
B5	-65.8	675.1	-20.6	-16.2	137.6	B5	-64.1	681.9	-12.8	-5.6	134.5

7.6 MOMENT COMPARISON

In braced frame system, moments are reduced in beam and column than the moment frame system as seen in Fig.7.3 to 7.6. Due to reduction in moment at the joint, the connection cost reduces in braced frame than the moment. This connection cost is a major consideration in the economical point of view in steel and composite structure.

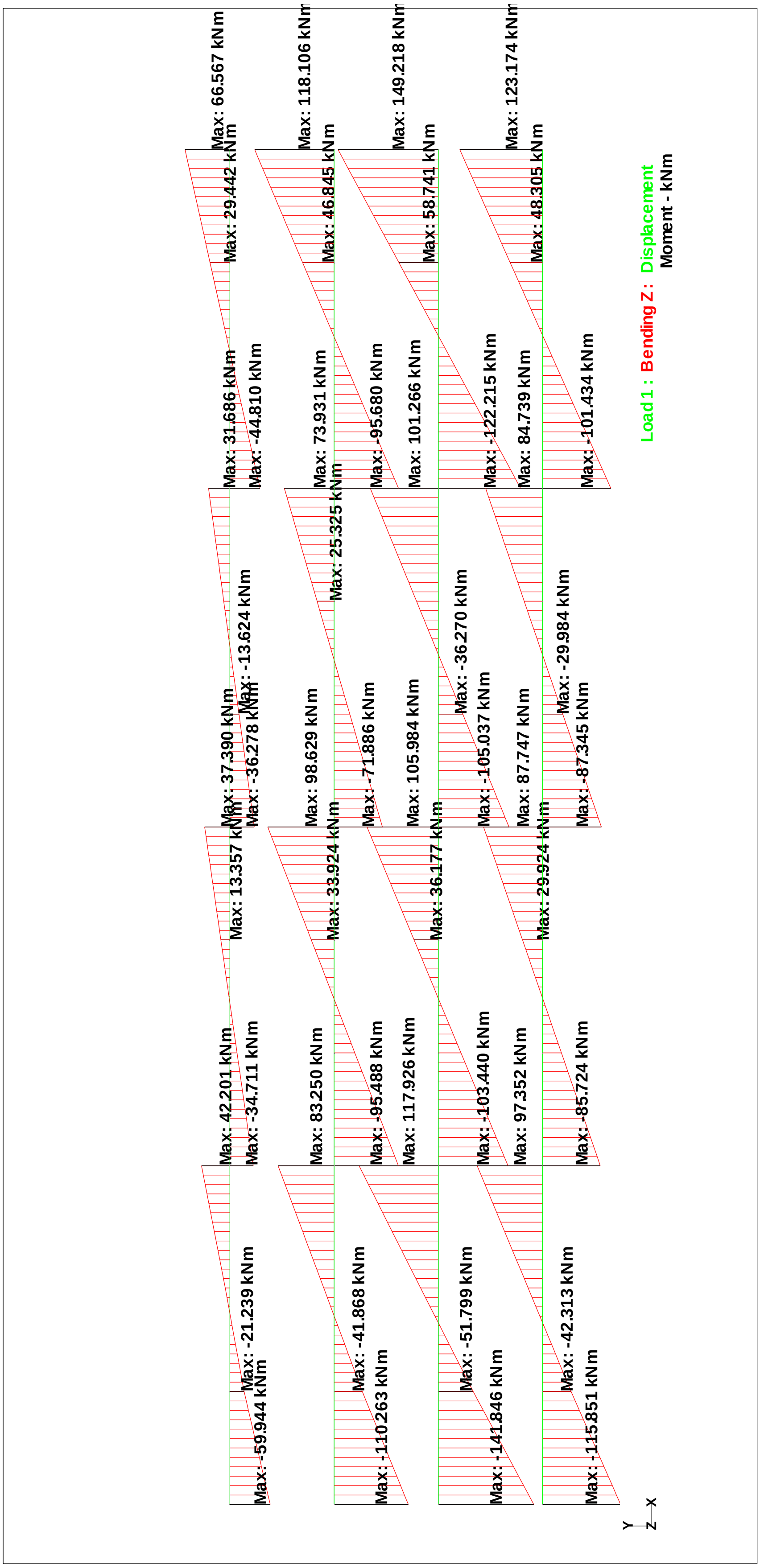


Fig. 7.3 Beam Moment Due to EQX in Moment Frame

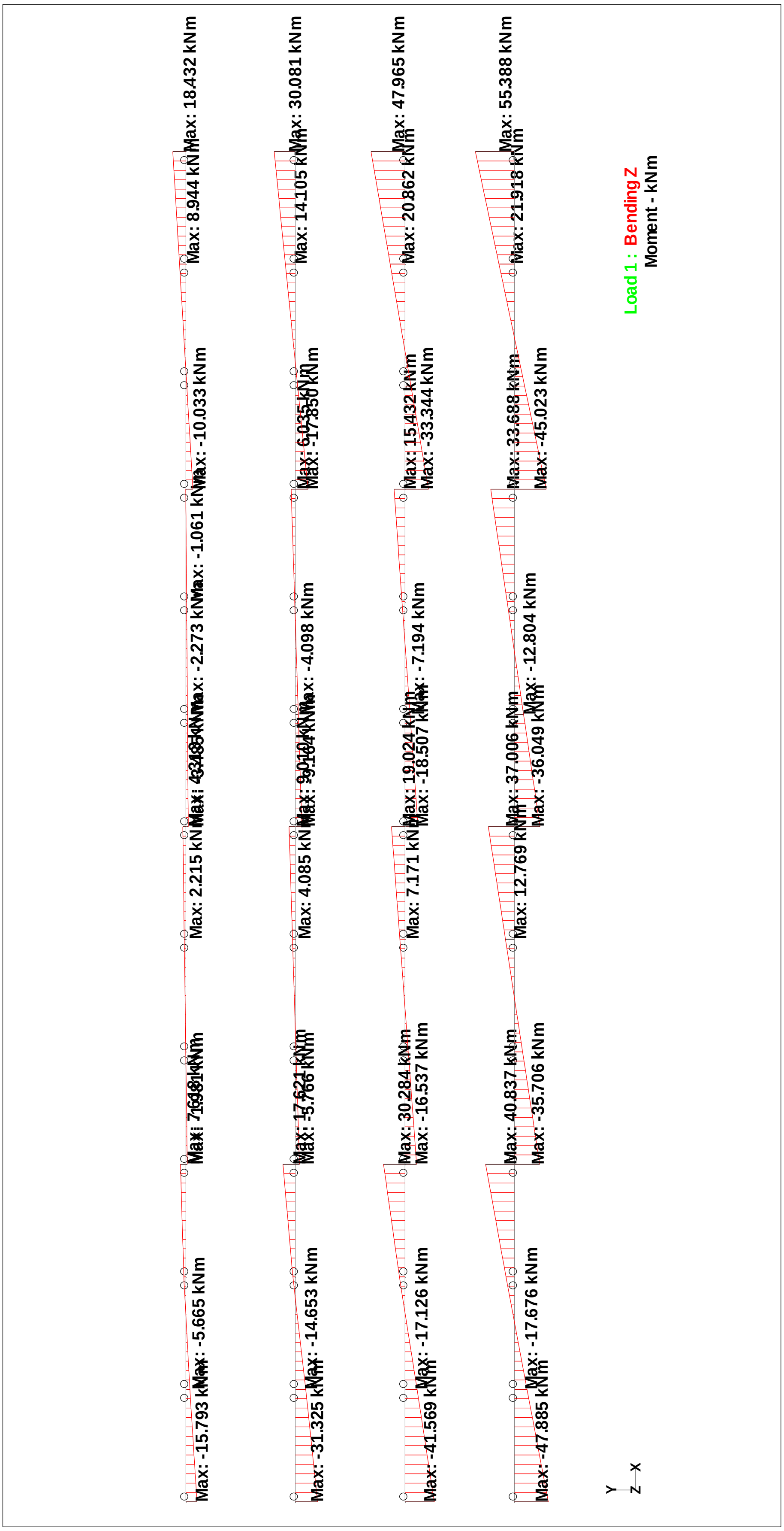


Fig.7.4 Beam Moment Due to EQX in Braced Frame

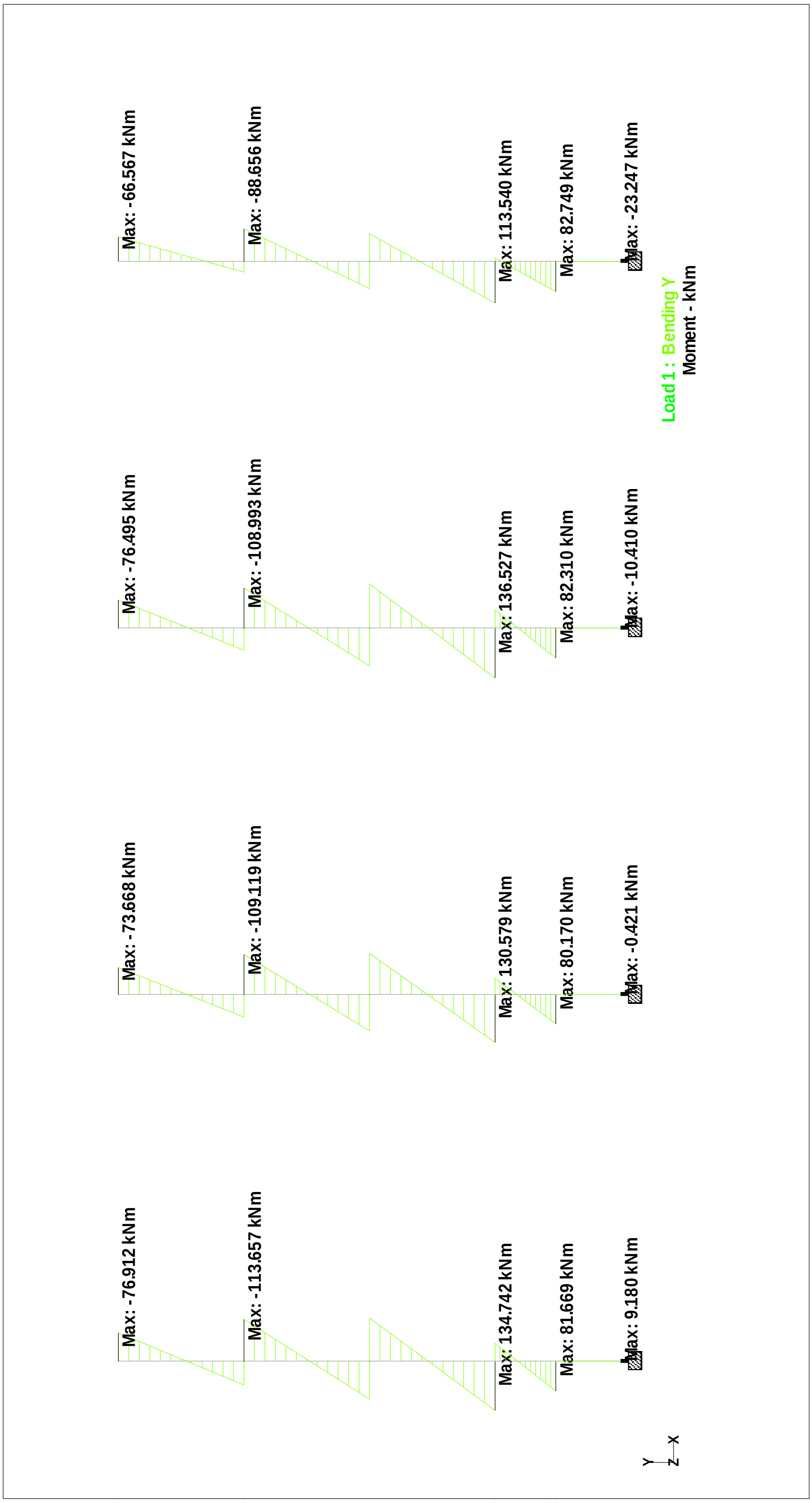


Fig.7.5 Column Moment Due to EQX in Moment Frame

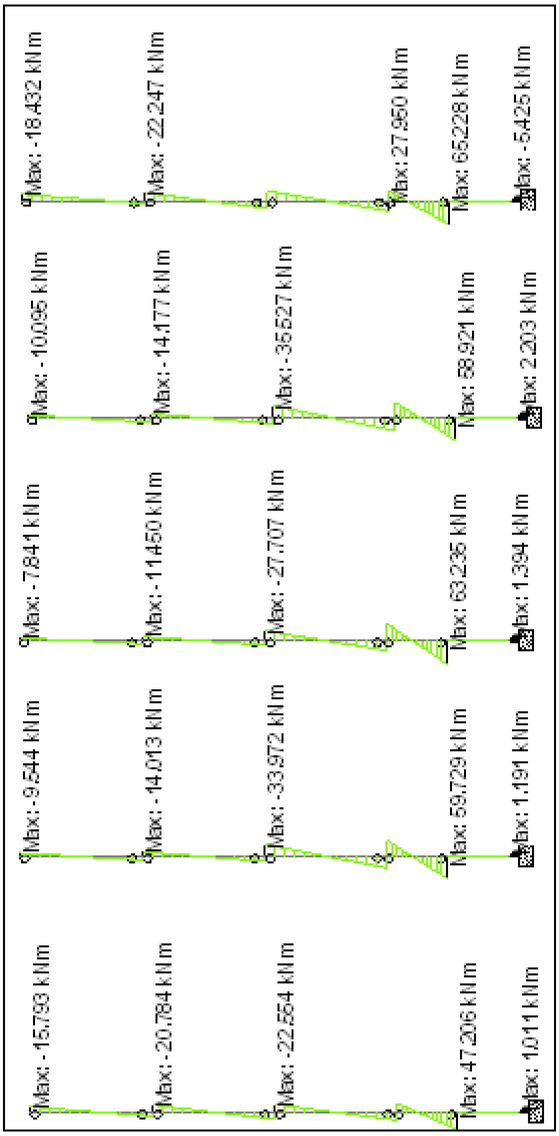


Fig.7.6 Column Moment Due to EQX in Braced Frame

7.7 SUMMARY

This chapter includes various comparisons like: composite floor, weight, deflection, foundation moment, moment in beam and column. These comparisons have been done considering two different frames. Results and figures of comparison also presents in this chapter. Based on previous study has done on economization of composite floor and its results.

8. SUMMARY AND CONCLUSION

8.1 SUMMARY

Multilevel car parks have become necessity in many metropolitan cities, due to insufficiency of land. As increase in traffic, it is no longer possible to park the cars by the side of the road. Hence multilevel car park is the right solution to meet the increased demand of parking facility.

Also it has been proved that the composite option is economical compared to RCC construction for five and above level car parks from the literature review.

The car parks made of composite construction are ideal solution since they are economical and offer large column free area apart from lot of advantages due to its lightweight. It has got more earthquake resistant properties and large column free areas for free movement of traffic.

The report contains the various aspects of multilevel car parks. It gives general requirement of multilevel car parks with its structural and functional arrangement. Also it includes the various component of the steel-concrete composite construction.

As a functional point of view, the split level type ramp system is used because this system is more efficient for user to drive and park the car in parking stall. The survey said that the most multilevel car parks in U.K. are of split level type.

The floor system considered of profile steel decking. No formwork is required in this system. It gives faster construction. The design of decking floor system has been carried out using BS 5950 (Part-4) [19]. The limit state method has used for design.

The seismic analysis has been carried out using IS 1893 (Part-2) [17]. The steel framing is used in the study while the dead load and live load are considered from IS 875 (Part-2) [25]. The comparison of two structural systems has been done like one system taken as moment resisting frame and other is braced frame. Analysis of above two systems has been done using STAAD*.Pro 2006*. Design has been carried out using working stress method. The results of deflection, bending moment, weight difference have been studied.

8.2 CONCLUSIONS

The study shows that the multilevel car park construction is cost effective while considering composite construction. The result of analysis shows that the saving in weight while the braced frame is used rather than the moment resisting frame. Also, the profile decking floor slab is time effective solution. Hence, the composite construction can be adopted for saving in time and in cost also. There are some other conclusion is:

- The 3.7% weight reduces while considering braced frame over the moment resisting frame.
- The reduction in the deflection is also observed.
- The load carrying behavior (Through element bending) of moment frame results in significant column and girder end moment.
- This larger moment leads to design the larger section
- Due to higher design end moment the connection cost will be affected
- The bending moment is reduced in column and beam when bracings are provided.
- Due to reduction in bending moment, the smaller section can be provided. Also it reduces connection cost.
- The bending moment gets reduced in foundation also. So it gives lighter foundation while bracing is used.
- The construction of profile deck floor system is saving a half the time over the solid concrete slab.
- Also the profile deck floor system is lighter than the solid concrete slab system and this reduction in weight will affect the foundation cost.

8.3 FUTURE SCOPE OF WORK

Based on the present study of multilevel car parks work can be extended further in:

- The same type of study considering different functional system of multilevel car parks.
- One can do the comparative study between automatic parking with different self park system.
- Also study can be done considering precast prestressed concrete system.

REFERENCES

1. Pydi Lakshman Rao, “Comparative study of multilevel car parks with RCC and composite option”.
2. Prestressed precast concrete institute, “parking structures recommended practice for design and construction”, Chicago.
3. Roy Becker, & Michael Ishler, “Seismic design practice for eccentrically braced frames”, Steel Tips, December, 1996.
4. R.P. Johnson “Composite structures of steel and concrete”, Volume 1, Blackwell Scientific Publications, UK, 1994.
5. Corus construction & Industrial, “Steel framed car parks”, Corus, North Lincolnshire.
6. Emile Troup, & John Cross, “Innovative Solutions in steel: Open Deck Parking Structures, 2003.
7. Concrete reinforcing steel institute, “Cast in place concrete parking structure”.
8. Kingdom of Bahrain, Urban planning affairs, “Guideline for the design of off-street car parking facilities.
9. IS 11384:1985, “Code of practice for composite construction in structural steel and concrete” Bureau of Indian Standards, New Delhi, 1985.
10. Institute for Steel Development & Growth, “Hand Book on Composite construction (Multilevel car parking)”, and INSDAG publication no ins/pub/019, January, 2002.
11. Institute for Steel Development & Growth, “[B+G+20] Storied Residential Building with Steel-Concrete Composite Option”, and INSDAG publication no ins/pub/047, May, 2003.
12. Kober, & Dima, “The behaviour of eccentrically braced frames with short links”, Steel department, Bucharest.
13. Hedaoo and Athare, “Composite Floor System-A Cost Effective Study”.
14. Smith, Bryan S., Coull, Alex, *Tall Building Structures: Analysis and Design*, Wiley, New York, 1991.
15. IS 3935:1966, “Code of practice for composite construction” Bureau of Indian Standards, New Delhi, 1998.
16. Euro code 4, “Structural steelwork euro code”, composite structure guideline given.

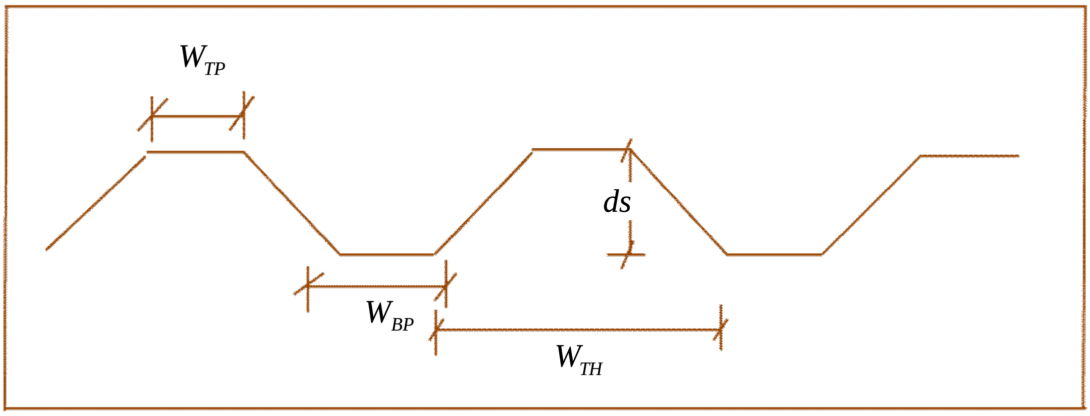
17. IS 1893 (Part-1):2002, “Criteria for earthquake resistant design of structures” Bureau of Indian Standards, New Delhi, 2002.
18. IS 800:1984,”Code of practice for general construction in steel” Bureau of Indian Standards, New Delhi, 1995.
19. IS 456:2000,”Plain and reinforced concrete- Code of practice” Bureau of Indian Standards, New Delhi, 1995.
20. BS 5950 (Part-4):1994,”Structural use of steel work in building, code of practice for design of composite slabs with profiled steel sheeting” British Standard.
21. BS 5950 (Part-3.1):1990,”Structural use of steel work in building, code of practice for design of simple and continuous composite beams” British Standard.
22. Eurocode-4:2004,”Design of composite steel and concrete structures-general rules and rules for building” European Standard.
23. IS 456:2000,”Plain and reinforced concrete- Code of practice” Bureau of Indian Standards, New Delhi, 1995.
24. IS 875 (Part-1):1987,”Code of practice for design load (other than earthquake) for building and structure- Dead Load” Bureau of Indian Standards, New Delhi, 2002.
25. IS 875 (Part-2):1987,”Code of practice for design load(other than earthquake) for building and structure – Imposed Loads” Bureau of Indian Standards, New Delhi, 1989.

Appendix – A

Composite Slab Design (Using Profiled Sheeting)

1) Data:

Slab Span (L)	=	3.33	m
Partial Safety Factor (y_p)	=	1.15	
Longitudinal Shear Partial Safety Factor (y_{vs})	=	1.25	
Characteristic strength of Concrete (f_{ck})	=	20	N/mm^2
Width of supporting steel beam (B)	=	190	mm
Thickness of the slab (T_s) or (h_t)	=	130	mm
Factor of Safety (For L.L. & Construction Load)	=	1.5	
Factor of Safety (For D.L.)	=	1.35	
Density of Concrete	=	25	kN/m^3
Thickness of Finishes (h_f)	=	40	mm



Profile Steel Decking Section

2) Decking Sheet Data:

(Data input below is from manufacturer's information)

Depth of the sheeting (d_s)	=	55	mm
Trough Spacing (S_T)	=	267	mm
Width of Trough Heal (W_{TH})	=	160	mm
Width at Top of Profile (W_{TP})	=	105	mm
Width at Bottom of Profile (W_{BP})	=	107	mm
Yield strength of steel (f_{yp})	=	250	N/mm^2

Design thickness (t_p)		=	0.8 mm
Effective area of cross-section (A_p)		=	1163.19 mm^2
Moment of Inertia (I_p)		=	443840 mm^4
Plastic moment of resistance (M_{pa})		=	3.527 kNm
Resistance to vertical shear (V_{pa})		=	34.776 kN
For resistance to longitudinal shear,	m	=	88 N/mm^2
	k	=	0.032 N/mm^2
Modulus of elasticity of steel (E_a)		=	200000 N/mm^2
Weight of Profile Sheeting		=	0.15 kN/m^2
Distance of centroid above base (e)		=	27.5 mm
Distance of plastic neutral axis above base (e_p)		=	27.5 mm
Weight of Concrete		=	2.56 kN/m^2
Floor Finish Load		=	1.00 kN/m^2

3) Load Data:

		Loads (kN/m^2)	Factored Loads (kN/m^2)
Imposed Load	=	2.5	3.75
Dead Load of Slab	=	3.71	5.01
Construction Load	=	1.5	2.25

4) Profiled Steel Sheeting as Shuttering: (EN 1993-1-3)

4.1) Effective length of the span:

$$l_e = \frac{L - B + d_s}{2}$$

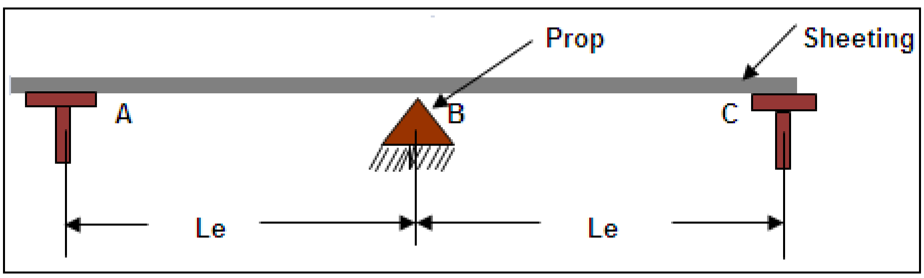
L= Actual span of the composite floor

B= Width of top flange of the supporting steel beams

d_s= The depth of the sheeting

Taking width of top flanges of the supporting steel beams are at least 150 mm.

le = 1597.5 mm



4.2) Factored moments and vertical shear :

(If slab is propped at center during construction stage)

Assume end Support are Unrestrained

Sagging Moment $= \left(\frac{Wu,_{DL}}{12} + \frac{Wu,_{LL}}{10} \right) \times (le)^2$

$= \left(\frac{5.01}{12} + \frac{2.25}{10} \right) \times 2.55$

= 1.64 kNm

Hogging Moment $= \left(\frac{Wu,_{DL}}{10} + \frac{Wu,_{LL}}{9} \right) \times (le)^2$

$= \left(\frac{5.01}{10} + \frac{2.25}{9} \right) \times 2.55$

= 1.92 kNm

Vertical Shear Force at beam section

$= \left(\frac{Wu,_{DL}}{2} + \frac{Wu,_{LL}}{2} \right) \times (le)$

$= \left(\frac{5.01}{2} + \frac{2.25}{2} \right) \times 1.59$

= 5.8 kN

At prop section = $[0.6 \times Wu,_{DL} + 0.6 \times Wu,_{LL}] \times le$

= 6.96

4.3) Check for moment:

Design moment = M_{pa}/ y_p = 3.07 kNm > 1.92 kNm

Hence, the profiled deck is SAFE in flexure at construction stage.

4.4) Check for vertical shear:

Design shear = V_{pa} / y_p = 30.2 kN > 6.96 kN

Design Shear is more than actual shear, Hence O. K.

4.5) Check for deflection:

Design load at construction stage = 3.7125 + 1.5

= 5.2125 kN/m^2

(Assume that the Prop does not deflect)

The maximum deflection in span AB, if BC is unloaded,

$$\delta max = \frac{w\, l_e^4}{185\, E_a\, I_p} = 2.1 \text{ mm}$$

$$\frac{Span}{180} = \frac{1597.5}{180} = 8.88 \text{ mm} > 2.07 \text{ mm}$$

Hence, the ponding effect may be ignored in the design of profiled deck.

5) Composite Slab Design:

5.1) Effective length of the span: (Propping is removed)

le = Clear distance between the supports + effective depth of the slab

le = 3.24 m

5.2) Factored moments and vertical shear:

The design ultimate loading = $(Wu,_{DL} + Wu,_{LL})$ = 8.76 kN/m^2

$$M_s = \frac{W*l_e^2}{8} = 11.5 \text{ kNm}$$

For vertical shear effective span consider as 3.33 m

$$V_s = \frac{W*l_e}{2} = 14.59 \text{ kN}$$

5.3) Check for moment:

Neutral Axis Lies above Steel Decking: Governing Case.

Compressive force in concrete Ncf is equal to steel yield force Npa,

$$N_{cf} = N_{pa} = \frac{A_p * f_{yp}}{y_p}$$

A_p = Effective area per meter width

y_p = Partial safety factor

f_{yp} = Yield strength of steel

= 253 kN

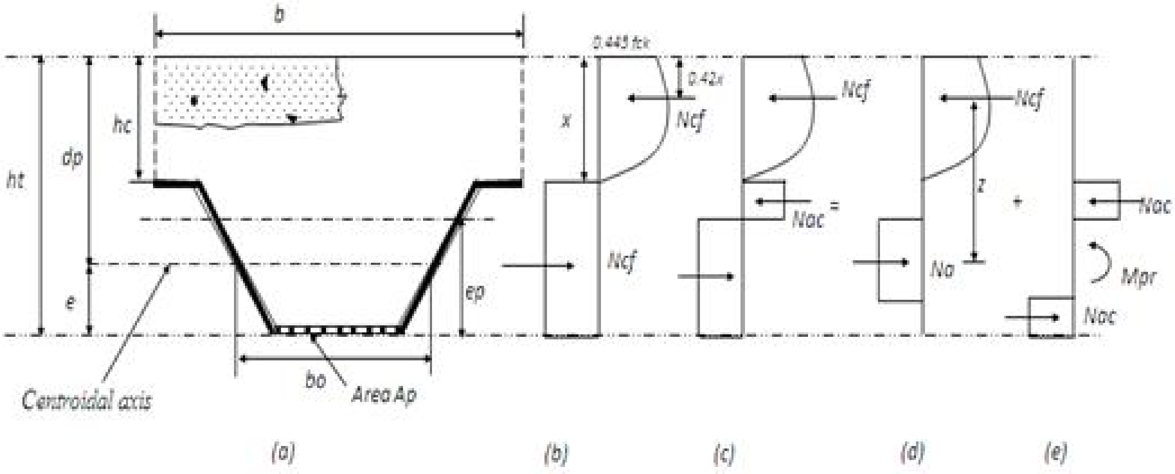


Fig. A.1 Resistance of composite slab to Sagging Bending

The neutral axis depth x is

$$x = \frac{Ncf}{b*0.36*fck} = 35.1 \text{ mm} < 75 \text{ mm}$$

(x <= hc ,neutral axis lies above steel decking)

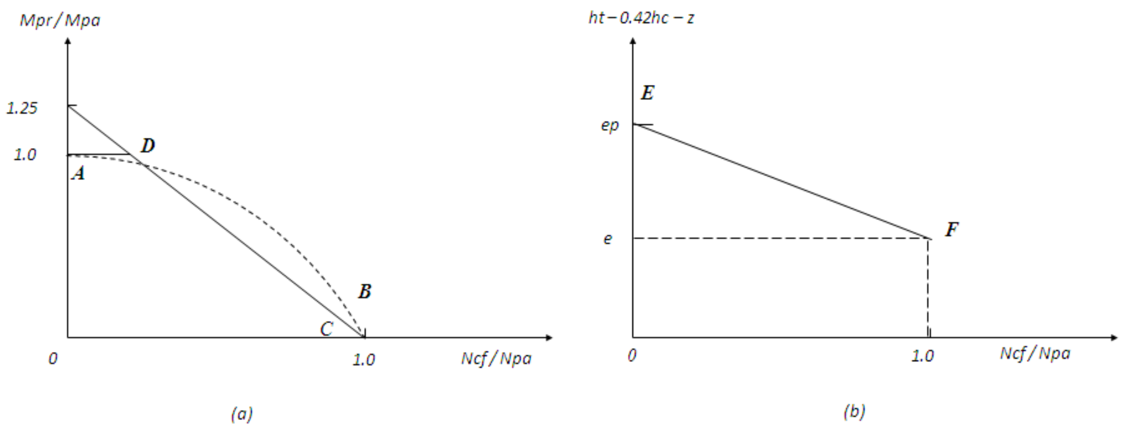


Fig. A.2 Resistance moment of profile

Design resistance to sagging bending moment,

MpRd = Ncf * (dp -0.42*x) = 22.19 kNm > 11.52 kNm

Hence, bending resistance is sufficient

Neutral Axis within Sheeting and Full Shear Connection:

Case not Governing.

N_{cf} = $bh_c \times 0.36f_{ck}$

= 540 kN < N_{pa}

Tensile Force = Na + Nac

Na = Ncf,

$$Mpr = 1.25Mpa\left[1 - \frac{Ncf}{Npa}\right] \le Mpa$$

M_{pr} = -64.05

M_{pRd} = $(N_{cf})z + M_{pr}$

For Lever Arm (z),The Two Cases:

(i) $N_{cf} = N_{pa}$ or $N_{cf}/N_{pa} = 1$ so, $N_{ac} = 1$

Hence $M_{pr} = 0$

$M_{pRd} = N_{pa} * (d_p - 0.42*x) = 0.25$ kNm

and $z = d_p - 0.42h_c$

$= h_t - e - 0.42h_c$

= 71 mm

(ii) $N_{cf} = 0$; $N_a = 0$

So, $M_{pr} = M_{pa}$

Than z is

$$z = ht - 0.42 * hc - ep\frac{(ep - e)Ncf}{Npa}$$

= 102.5 mm

$M_{pRd} = (N_{cf})z + M_{pr}$ = -8.70 < 11.52 kNm

5.4) Check for vertical shear:

$V_{vRd} = (b_o/b) * d_p * \tau_{rd} * K_v * (1.2 + 40\rho)$ per unit width

= 36.70 kN > 14.59 kN

d_p = depth to the centroidal axis = 102.5 mm

τ_{rd} = basic shear strength of concrete

= 0.3 N/mm_2

K_v = allow higher shear strength for shallow members

= $(1.6 - d_p) \ge 1$ with dp in m = 1.498

ρ = allow a small contribution due to shearing

= $(A_p/ b_0 d_p)$ = 0.009 < .02

A_p = effective area of shearing within width b_0 = 134.79 mm^2

b= width of slab is one typical wavelength of profiled sheeting = S_T

b_o= width of profiled at the neutral axis = 133.5 mm

Hence, Composite slab is SAFE in shear.

5.4) Check for punching shear:

Cp = 2*π*hc +2(2*dp + ap -2*hc) + 2*bp + 8*hf

= 2101.2 mm

ap = Length of Concentrated Load = 300 mm

bp = Width of Concentrated Load = 300 mm

hf = Thickness of Finishes = 40 mm

V_{vRd} = (Cp * hc * τrd * Kv * (1.2 + 40ρ) per unit width

= 112.9 kN > 14.59 kN

5.4) Concentrated Point and Live Load :

(BS: 5950-4-1994)

bm = bp + 2*(hc + hf)

= 530.0 mm

am= ap +2*(hc + hf)

= 530.0 mm

$$be = bm + kLp\left[1 - \frac{Lp}{L}\right] \leq L$$

b_e = 2.2 m

k = 2 2, For Bending and Longitudinal Shear

1.33, For Interier Span of Composite Slab

1, For Vertical Shear

Lp = 1.5 m Center Distance From the Nearer Support of a Slab of span

For Simply Supported Slab

Point Load (Q_d) = 9 kN

The Sagging Moment Per Unit Width of the Slab

$$m_{Sd} = Qd * Lp * \frac{\left(1 - \frac{Lp}{L}\right)}{be}$$

m_{Sd} = 3.4 kNm

The Maximum Sagging Bending Moments

$$M_{Sd} = Qd * \frac{(be - bm)}{8}$$

M_{Sd} = 2.8 kNm < 11.52 kNm

Hence,Not Governing.

5.5) Check for longitudinal shear:

m-k shear bond test method gives the vertical shear resistance as

$$V_{L,Rd} = \frac{b * dp * ((m\ Ap / b * (l/4)) + (k))}{y_{vs}}$$

= 12.98 kN < 36.7 kN

This is less than design vertical shear, so safe against longitudinal shear.

5.6) Check for serviceability:

Cracking of concrete above supporting beam:

The minimum amount of reinforcement as per EC 4 is 0.4 % of the area of concrete above the sheeting if propping is used

As = 300 mm^2 @ over support

Deflection:

Properties of Cracked Section:

Neutral axis Depth below Upper Surface on Slab

$$x_e = de\left[\sqrt{(\alpha_e P)^2 + 2\alpha_e P} - \alpha_e P\right]$$

d_e = 102.5 mm

α_e = Effective Modular Ratio= $\alpha s + pl*(\alpha l - \alpha s) = 18.0$

α_s = 6 Modular Ratio for Short-term Loading (Cl.4.1, BS5950-3.1)

α_l = 18 Modular Ratio for Long-term Loading

α_s = 6,For Normal Weight Concrete

= 10,For Light Weight Concrete

α_l = 18,For Normal Weight Concrete

= 25,For Light Weight Concrete

$p_l = \dfrac{\text{Long Term Load}}{\text{Total Load}}$ = 1 (Assume)

$P = \dfrac{Ap}{Ac}$ Here, Ac = bs*d = 102500 mm^2

P = 0.01135

x_e = 47.8 mm

Moment of Inertia of Cracked Section

Ic = 7165.2 Width mm4 /mm

w = 2.5 kN/m2

Deflection Under Service Load

$\delta_c = \dfrac{5*w*l^4}{384*EI} = 2.8 < 5.4$

Hence O.K.

$\dfrac{\text{span}}{\text{depth}} = \dfrac{3242.5}{102.5} = 31.6 \text{ mm} < 32 \text{ mm}$

Hence, O.K.

(If One End Continuous)

Span/Depth =	25	For Simply Supported Slab
	32	For One End Continuous
	35	For Internal Span

Appendix – B

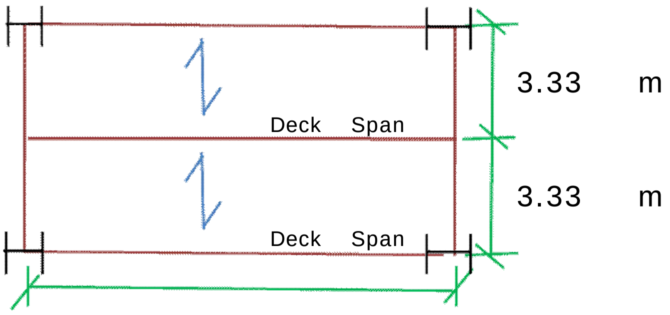
Design of Composite Beam with EC-4

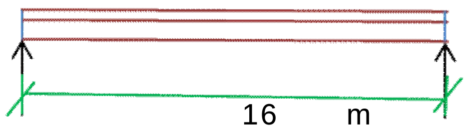
(Unpropped Construction)

1.) Design Data:

1.1) Floor Dimensions:

Span (L)	=	16	m
Beam Spacing (b)	=	3.33	m
Slab Depth (ht)	=	130	mm
Depth above Profile (hc)	=	75	mm
Deck Profile Height (hp)	=	55	mm





1.2) Shear Connector:

Overall Height of Studs	=	100 mm	(IS:11384)
Diameter of Studs	=	20 mm	(IS:11384)
Welded Length	=	95 mm	(IS:11384)
Partial Factor for Design Shear Resistance of a Headed Stud (γv)	=	1.25	(EC,Cl.6.6.3.1), (IS800,Table 5)
Ultimate Tensile Test (fy or fu)	=	250 N/mm^2	(IS:11384)

Characrteristic Strength of			
Concrete (fck)	=	25	N/mm^2
Secant modulus of Elasticity			
of Concrete (Ecm)	=	27.4	kN/mm^2

h/d	=	4.75	>	4
α	=	1		

1.3) Deck: (From Manufacturer's Data)

Thickness of the slab (Ds)	=	130	mm
Depth of the sheeting (Dp)	=	55	mm
Trough Spacing	=	267	mm
Average Trough Width	=	133.5	mm
Design Strength of Profile			
Sheets (fyp or Pyp)	=	250	N/mm^2
Thickness of Profile Sheets (ts or tp)	=	0.8	mm
Steel Deck Weight (allow)	=	0.15	kN/m^2
Area of sheet (Ap)	=	1163.2	mm^2
Partial Safety Factor (γap)	=	1.05	

1.4) Materials:

Structural Steel Grade	=	250	
Concrete Grade	=	30	
Yield strength of steel (Py)	=	250	N/mm^2
Char. Strength of concrete (fcu)	=	30	N/mm^2
Char. Strength of concrete (fcy)	=	25	N/mm^2
Modulus of elasticity of steel (E)	=	210000	N/mm^2
Density of Concrete	=	23550	N/m^3
Density of Concrete	=	23.55	kN/m^3
Load Factor for DL (γG)	=	1.35	
Load Factor for LL (γQ)	=	1.5	(IS800)
Partial safety factors for Steel(γa)	=	1.15	
Partial safety factors for con. (γc)	=	1.5	
Design Strength (fd)	=	217.3913043	N/mm^2

2.) Loading:

Concrete Slab

Weight = 2.41 kN/m^2

2.1) Construction Stage:

Concrete Slab	=	2.41	kN/m^2
Steel Deck (allow)	=	0.15	kN/m^2
Reinforcement (allow)	=	0.04	kN/m^2
Steel Beam (allow)	=	0.15	kN/m^2
Total	=	2.75	kN/m^2
Construction Load	=	0.5	kN/m^2

2.2) Composite Stage:

Concrete Slab	=	2.41	kN/m^2
Steel Deck (allow)	=	0.15	kN/m^2
Reinforcement (allow)	=	0.04	kN/m^2
Steel Beam (allow)	=	0.15	kN/m^2
Total	=	2.75	kN/m^2

Ceiling and Services	=	0.5	kN/m^2
Imposed Load	=	2.5	kN/m^2
Partition Load	=	0.5	kN/m^2
Imposed Load	=	3	kN/m^2

3.) Initial Selection of Beam section: ISMB 600

Depth of web (h)	=	600	mm
Thickness of web (tw)	=	12	mm
Depth of Flange (b)	=	210	mm
Thickness of Flange (tf)	=	20.8	mm
Depth of Clear Web (d)	=	509.7	mm
Moment of Inertia (Ip)	=	9.18E+08	mm^4
Section Modulus (Wpl)	=	3510630	mm^3
Area of Steel Beam (A)	=	15621	mm^2
Radius of Gyration	=	242.4	mm

Nominal Value of Yield Strength (fy) = 250 N/mm2 (when tf < 40mm)

$$\varepsilon = \sqrt{\frac{250}{f_y}} = 0.97 \quad \text{(IS800,Table 2)}$$

4.) Section Classification:

$$\left.\begin{array}{llll} b/2tf = & 5.05 < & 9.4\varepsilon = & 9.11 \\ d/tw = & 42.48 < & 84\varepsilon = & 81.44 \end{array}\right\} \text{Class 1 and Plastic (IS800,Table 2)}$$

The Cross-section is in Class 1 and plastic hinge can be developed.

Since the Beam is Simply Supported and Uniformly Loaded, only the Mid-span hinge will Form and Thus Plastic Analysis can be used for a Class 2 Cross Section as Well.

5.) Construction Stage Design:
Ultimate Limit State Loading:

Slab + Beam	=	3.72	kN/m2
Construction	=	0.75	kN/m2
Total	=	4.47	kN/m2

Total Design Load (F) = 238.0 kN

Design Moment (M_{sd}) = 476.1 kNm

Assume the beam in the Construction Stage is Laterally Restrained by the decking since The decking spans Perpendicular to the beam and is directly attached to it.

Moment Resistance of Steel Beam = $M_{apl.Rd}$ where:

$M_{apl.Rd} = W_{pl} \times f_d$ = 763.2 > 476.1

Hence O.K. (EC3, part1, Cl.5.4.5.2)

Beam is satisfactory for positive moment resistance in the construction stage.

6.) Composite Stage Design:

Ultimate Limit State Loading:

Slab + Beam		=	3.72	kN/m2
Ceiling and Services		=	0.68	kN/m2
	Total	=	4.39	kN/m2
Imposed Load		=	4.5	kN/m2
	Total	=	8.89	kN/m2

Total Design Load (F) = 473.80 kN

Design Moment (Msd) = 947.6 kNm

7.) Effective Width of Compression Flange, beff

beff = 2*lo / 8   lo = Span for Simply Supported Beam

= 4 m > 3.33 m

Take beff= 3.33 m

8.1) Compressive Resistance of Slab, Rc

$$Rc = \frac{0.85\,fck}{\gamma c} * beff * hc$$

0.85*fck / γ ≈ 0.45*fcu

Rc = 3371.625 kN

8.2) Tensile Resistance of Steel Section, Rs

Rs = fd * Aa

= 3395.9 kN

8.3) Moment Resistance with Full Shear Connection

(i) Neutral Axis in the Concrete Flange, Rs<Rc

Neutral axis Not in Concrete flange.

therefore the Moment Resistance of The Composite Beam is:

Mpl.Rd = $Rs\left[\frac{h}{2}+hc+hp-\frac{Rs}{Rc}*\frac{hc}{2}\right]$

= 1332.0 kNm > 947.6 kNm

Hence O.K.

(ii) Neutral Axis in Steel Beam , Rs>Rc

Neutral Axis in Steel Web

$$M_{pl.Rd}=\frac{Rs*D}{2}+\frac{Rs*(Ds+Dp)}{2}-\frac{(Rs-Rc)^2T}{4*Rsf}$$

Mpl.Rd = 1332.8 kNm > 947.6 kNm

Hence O.K.

8.4) Shear Connector Resistance:

The Design Shear Resistance of a Shear Connector is:

Prd = 0.29*α*d2*((fck*Ecm)/γv)0.5 (EC,Cl.6.6.3.1)

or which ever is smaller

Prd = 0.8*fu*(Πd2 /4)/γv (EC,Cl.6.6.3.1)

Prd = 76.8 kNm

Prd = 50.3 kNm

take Prd= 50.3 kNm

8.5) Influence of Deck Shape: 1 (EC,Cl.6.6.4)

1. Deck Crosses the Beam (Transverse)

One Stud per Trough, Nr = 1

$$Kt=\frac{0.7}{\sqrt{Nr}}*\left(\frac{bo}{hp}\right)*\left[\left(\frac{h}{hp}\right)-1\right]\leq 1.0$$

Kt = Reduction Factor

= 1.23 > 1

Reduction not Required

Prd = 50.3 kN

Two Stud per Trough, Nr = 2

Kt = Reduction Factor ≤ 0.8

= 0.87 > 0.8

take Kt= 0.80

Prd = 40.2 kN

2. Deck Parellal to the Beam

Stud per Trough, Nr = 1 or 2

hsc = Overall height of Stud < (75+hp) = 130

Kl = Reduction Factor

= 1.19 > 1

Reduction not Required

Prd = 50.3 kN

8.5) Shear Connector:

Therefore, 16 troughs are available for the positioning of the Shear Stud Connectors.

8.6) Longitudinal Shear Force Transfer, Rq:

Rq(1 Stud) = Shear Connector * Prd = 804.2 kN

Rq(2 Stud) = 2 * Shear Connector *Prd = 1286.8 kN

Degree of Shear Connection, N/Nf (One Stud per Trough):

Minimum Degree of Shear Connection = 0.4 < 1

N / Nf = Rq / Rs = 0.24 (1 Stud/Trough) < 0.4

8.7) Moment Resistance with Partial Shear Connection:

Moment Resistance of the Composite Beam is Obtained Using the Linear Interaction Method as Follows:

MRd = Mapl.Rd + (N/Nf) * (Mpl.Rd-Mapl.Rd) (EC,Cl.6.2.1.3(5))

Mapl.Rd = 763.2 kNm

Mpl.Rd = 1332.9 kNm

Using 1 Stud per Trough

MRd = 898.1 < 947.6 kNm

Revised Section

Using 2 Stud per Trough

Minimum Degree of Shear Connection = 0.25+0.03XL

= 0.73

(When area of bottom and top flange are same)

Rq / Rs = 0.379

MRd = 978.7 kNm > 947.60 kNm

Hence O.K.

Studs Per Trough	Moment Resistance Based on Linear Interaction (kNm)	Design Factored Moment (kNm)
1	898.1	947.6
2	978.7	

9) Vertical Shear:

Slab + Beam = 99.04 kN

Ceiling and Services = 17.98 kN

117.02 kN

Imposed Load = 119.88 kN

Total Shear Force, VSd = 236.90 kN

Shear Resistance, Vpl.Rd ii

i) Vpl.Rd = $Av * fd / (3)^{0.5}$ (For Builtup I Section)

(EN 1993-1-1,6.2.6)

Av = h*tw (Cl.6.2.2.4)

Vpl.Rd= 903.7 kN

ii) Vpl.Rd= 0.6*Av*fd (For Rolled I,H,C Section)

Vpl.Rd= 939.1 kN

Vpl.Rd / 2 = 469.56 kN > 236.90 kN

Hence O.K. (For All Cases)

With a uniformly distributed load, shear force does not influence the moment resistance of the section in this example.

10.) Serviceability Limit States:

Elastic Stresses:

No stress checks are required for normal conditions and consequently. No limits are given in EC4

10.1) Deflections:

Non-Composite Stage Deflection,δ

Slab + Beam = 2.75 kN/m2

Design Load, F = 146.72646 kN

$$\delta = \frac{5 * F * L^3}{384 * Ea * Iay}$$

δ = 40.59 mm

Composite Stage Deflection,δ

Imposed Load	=	3 kN/m2
Design Load, F	=	159.84 kN

Second moment of area of the composite section based on elastic properties (Uncracked Inertia), Ic, is obtained as follows:

$$Ic = \frac{Aa(h + 2 * hp + hc)^2}{4(1 + nr)} + \frac{beff * hc^3}{12 * n} + Iay$$

r = Aa / (beff * hc) = 0.052

n = Modular ratio

= 10 (For Normal Weight Concrete)

This modular ratio is used for floor loadings with modest permanent loads.

Ic = 2.5E+09 mm4

Deflection with Full Shear Connection

$$\delta c = \frac{5 * F * L^3}{384 * Ea * Ic} = 16.16 \text{ mm}$$

As partial shear connection exists, take effect of slip into Account as Follows:

$$\frac{\delta}{\delta c} = 1 + 0.3\left(1 - \frac{N}{Nf}\right)\left(\frac{\delta a}{\delta c} - 1\right)$$

δa = 44.2 mm

δ = 22.6 mm < 45.71 mm

Hence O.K.

So, Deflection due to Imposed Load is Satisfactory.

Total Deflection:

Construction Stage	=	40.6	mm
Imposed Load	=	22.6	mm
Ceiling and Services	=	3.8	mm
Total	=	66.9	mm

Normally in British Practice, the Limit on the Maximum Total Deflection for a Composite Beam is = (L/200) = 80 mm > 66.9 mm

Hence O.K.

Also, pre-cambering would not be considered for a construction stage Deflection of 40.6 mm over span of 16 m

11.) Transverse Reinforcement:

Use mesh reinforcement in Slab = A142

Check Resistance of Concrete Flange to Splitting.

Shear Resistance per Shear Surface, vRd

$$v_{Rd} = 2.5 * Acv * \eta * \tau_{Rd} + \frac{Ae * fsk}{\gamma s} \leq 0.2 * Acv * \frac{\eta * fck}{\gamma c}$$

(Neglecting Contribution of Decking)

Ae = 142 mm2/m

Acv = 105000 mm2/m

fsk = 25 N/mm2

fck = 460 N/mm2

η = 1 (For normal weight concrete)

γc = 1.5 (EN 1992-1-1)

γs = 1.15 (EN 1992-1-1)

τRd = 0.25*fctko.5/γc = 0.3 (EN1992-1-1, Table 3.1)

fctko.5 = Mean Tensile Strength of Concrete

vRd = 135.55 kNm < 350 kNm

Hence O.K.

Shear Force Per Unit Length, v

Using 1 Shear Connector per Trough,

v = 83.78 kN/m < 135.55

Hence O.K.

So, Mesh is Satisfactory.

For illustration Purposes, in a Situation where vRd < v, a Component a rising From the Tensile Strength of the Deck vpd is added to vRd determine above. The Value of vpd is Obtained as Follows:

For Profiled Sheets Continuous Across the Top Flange of Steel Beam,

$$vpd = Ap * fyp / \gamma c$$

and for Profiled Sheets Discontinuous Across the Top Flange of The Steel Beam,

$$vpd = (Nr/s)*(4*d*ts*pys)$$

Nr = number of studs in a group

= 1 or 2

$$pys = fyp / \gamma ap$$

For Continuos Profiled Decking

vpd = 276.95 kN/m

For Discontinuous Profiled Decking

vpd = 50.79 kN/m

It may be shown that the design is adequate if the decking is discontinuous and the studs are in pairs.

12.) Vibration Simplified Approach:

Loading:

Slab + Beam	=	2.75	kN/m2
Ceiling and Services	=	0.5	kN/m2
10% Imposed Load	=	0.3	kN/m2
Total	=	3.55	kN/m2

Total Weight of Floor (F) = 189.4 kN

Increase the Inertia Ic, by 10% to Allow for the Increased Dynamic Stiffness of the Composite Beam, Icl

$Icl = 2763574301 \quad mm^4$

Instantaneous Deflection Caused by re-application of the Self Weight of the floor and the beam to the composite beam,

$\delta a = 17.4 \quad mm$

Natural Frequency $= 18 / \delta a_{0.5}$

$= 4.3 \quad Hz > 4 \quad Hz$

Hence O.K.

The Composite Beam Satisfactory Agaist Vibration.

Conclusion

The Design is Strongly Influenced by the Requirements for Limitation of Total Deflections, Rather than Moment Resistance or Other Serviceability Criteria.