DESIGN OF POST-TENSIONED FLOORS

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DESIGN OF POST-TENSIONED FLOORS

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

Sandeep N. Deshmukh
(05MCL012)

Guide

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May 2007

CERTIFICATE

This is to certify that the Major Project entitled **"Design of Post-Tensioned Floors"** submitted by **Mr. Sandeep N. Deshmukh (05MCL012)** 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.

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ABSTRACT

Concrete is strong in compression and weak in tension. Plain (unreinforced) concrete is placed when the structure must resist mostly compression forces and when tensile stresses are low. Reinforced concrete is used for structures that must resist significant tensile forces. Reinforcing materials (such as steel rebar or welded-wire fabric) that perform well in tension are embedded in the concrete. Prestressed concrete is concrete that is pre-compressed by stressing the reinforcement before loads are applied. This greatly increases its ability to resist tensile forces without excessive cracking. Concrete can be prestressed in a factory by tensioning the steel reinforcement first and then placing concrete around it-"pre-tensioned" reinforcement. Or concrete can be cast in place and the steel reinforcement tensioned after the concrete has reached a required strength-"post-tensioned" reinforcement. Structural engineers calculate the limits for tensioning.

There are two types of Post-Tensioning systems: bonded and unbonded. Unbonded systems use strands surrounded with special corrosion-inhibiting grease and encased in waterproof plastic sheaths. This assembly is positioned, and then the concrete is placed, similar to standard reinforced concrete. With a bonded system, before the concrete is cast, empty steel or plastic ducts are positioned in the formed area and attached to the anchorages at either end. After the concrete is placed and gains strength, strands are threaded through the ducts, tensioned, and the ducts are filled with a special grout designed to prevent corrosion. Unbonded systems are nearly always used for building and slab construction, while bonded systems are mostly for bridge construction.

As a method of reinforcing, post-tensioning is growing in popularity because it saves money, has many construction advantages, and contractors and designers no longer regard it as a mysterious method of reinforcement. Also, owners of structures are beginning to understand the process and its benefits. Tendon corrosion problems, an earlier issue, have been overcome by the development of corrosion-resistant tendons and new materials that electrically isolate tendons from the concrete eliminating the corrosion reaction.

Load balancing method and the Equivalent frame method are the two methods which are generally used for the design of the post-tensioned slabs. As the popularity of the post-tensioning method is widely increasing day by day in the Indian construction the emphasis is given on the design of the post-tensioned flat slab in the present report. Along with this the parametric study of different parameters, such as thickness of slab, grade of concrete, shear, normal reinforcement, prestressing force, losses due to stressing and deflection etc. with the variation of span (7 to 12m at an interval of 0.5m) of flat slab is carried out. To understand the design procedure a case study of the office building is taken and it is designed by considering the different floor system such as post-tensioned flat slab, reinforced concrete flat slab, post-tensioned slab with reinforced concrete beams and reinforced concrete slab and beams. For the analysis of the post-tensioned and reinforced concrete building the ADAPT and STAAD PRO software are used respectively. The estimation and costing of the case study is carried out for the economic point of view as it is of great importance in any structure.

INDEX

LIST OF FIGURES

LIST OF TABLES

ABBREVIATION NOTATION AND NOMENCLATURE

A	Cross section Area (mm^2)
A_{cf}	Gross cross section area of the design strip (mm^2)
A_p	Cross-section area of ordinary reinforcement (mm^2)
E_c	Modulus of elasticity of concrete (N/mm^2)
E_p	Modulus of elasticity of steel (N/mm^2)
I	Moment of inertia (mm^4)
J	Polar moment of inertia (mm^4)
L	Length of the span (m)
M	Bending moment (KNm)
M_r	Moment of resistance (KNm)
P	Post tension force
R	Ultimate strength of the cross section
S	Moment and shear force due to applied loads
V	Vertical shear force
V_u	Ultimate shear force
a_{cs}	Deflection due to shrinkage
a_u	Limit deflection
b	Width
c_1	Width of column
c_2	Depth of column
d_s	Effective depth of ordinary reinforcement
d_p	Effective depth of post-tension reinforcement
f_{cd}	Design value of compressive strength of concrete
f_c	Compressive strength of concrete
f_{sy}	Yield strength of reinforcing steel
g	Dead load
h_p	Sag of tension parabola
k	Wobble coefficient per meter length of steel
m_u	Plastic moment (in span)
n	Lateral membrane force per unit width

p_p	Content of prestressing steel
p_s	Content of ordinary reinforcement
q	Live load
x_c	Depth of concrete compression zone
y_p	Internal lever arm (post tensioning steel)
z_s	Tension force due to ordinary reinforcement
z_p	Tension force due to post tension reinforcement
β	Ratio, coefficient
ε_{cs}	Final shrinkage factor of concrete
σ_{po}	Stress in post tension steel
$\Delta\sigma_p$	Increase of force in prestressing steel
Δl	Tendon elongation
Y_f	Load factor (partial safety factor)
Y_m	Cross-sectional factor (partial safety factor)
τ	Shear stress
τ'_c	Permissible shear stress
δ	Deflection coefficient of friction
μ	Coefficient of friction
ϕ	Diameter of bar (mm)
$\varnothing$	Creep coefficient

1. INTRODUCTION

1.1 GENERAL

For the successful construction of new buildings the more importance is given to the successful planning. Successful planning includes the good communication and close cooperation between all parties involved in the project, in particular the owner, the architect, the engineer and the contractor. One of the key aspects of successful planning is the constructability of the building. This is of paramount importance for the success of the project since constructability most markedly affects the time to completion of a turn-key project and thus the final cost to the owner. Because the major part of the total cost of large developments is financing cost rather than actual construction cost, the completion time is often a more important consideration than material consumption. If we see the total cost of the building, only the structural cost is about 30 to 50% of the total construction cost and on the other hand more than half of the structural cost is labour cost, related mainly to formwork.

If we take a building, the floor system is one of the major parts which contribute more in the structural cost of a building. The floor framing system affects the cost in two ways, first it has a direct influence on the rest of the structure in that its weight determines the size of columns, walls and foundations, and its structural depth determines the total building height and thereby the quantity of cladding and vertical trunk lines. In seismic areas the floor weight also determines the member sizes of the lateral load resisting system. While the floor framing accounts for just over 50% of the total, any reduction of floor weight would cause a corresponding weight reduction also for the peripheral frames and the service core and would thus affect almost the entire structural weight. The second way the floor framing system affects the cost of the building which relates to the total construction time. The time required constructing one floor and the time required for the commencement of fit-out work such as electrical and mechanical services, suspended ceilings and decorating, are major factors influencing the time to completion of the building. These considerations demonstrate that the optimization of the floor framing with regard to weight, structural depth and constructability goes a long way towards successful planning.

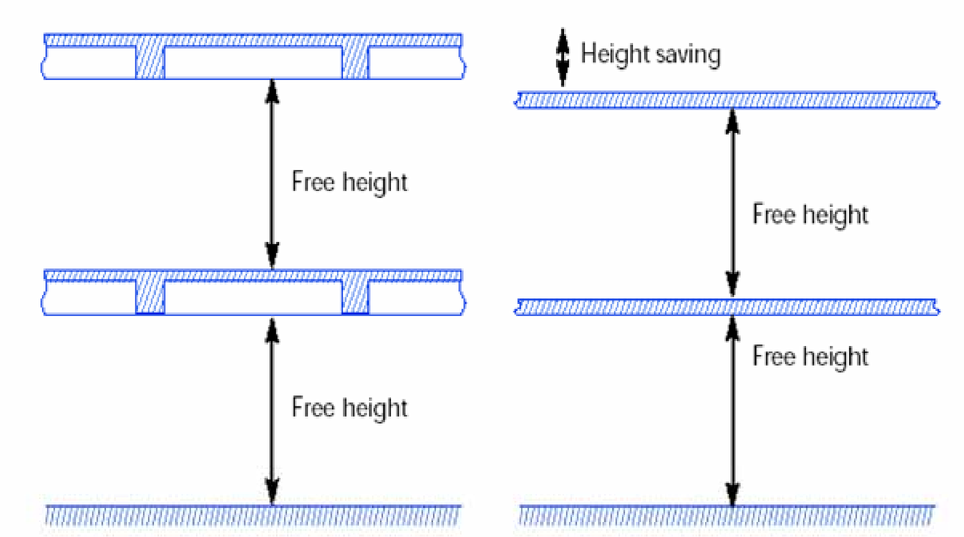
As the floor system plays an important role in the overall cost of a building, a post-tensioned floor system is invented which reduces the time for the construction and finally the cost of the structure. In some countries, including the U.S., Australia, South Africa, Thailand and India, a great number of large buildings have been successfully constructed using post-tensioned floors. The reason for this lies in its decisive technical and economical advantages. The most important advantages offered by post-tensioning systems are as follows -

- By comparison with reinforced concrete, a considerable saving in concrete and steel since, due to the working of the entire concrete cross-section more slender designs are possible.
- Smaller deflections compared to with steel and reinforced concrete structures.
- Good crack behavior and therefore permanent protection of the steel against corrosion.
- Almost unchanged serviceability even after considerable overload, since temporary cracks close again after the overload has disappeared.
- High fatigue strength, since the amplitude of the stress changes in the prestressing steel under alternating loads are quite small.
- If a significant part of the load is resisted by post-tensioning the non-prestressed reinforcement can be simplified and standardized to a large degree. Furthermore, material handling is reduced since the total tonnage of steel (non-prestressed + prestressed) and concrete is less than for a Reinforced Concrete floor.
- Assembling of precast elements by post-tensioning avoids complicated reinforcing bar connections with insitu closure pours, or welded steel connectors, and thus can significantly reduce erection time.
- Usually the permanent floor load is largely balanced by draped post-tensioning tendons so that only the weight of the wet concrete of the floor above induces flexural stresses. These are often of the same order as the design live load stresses. Hence back-propping of one floor below is usually sufficient.

- Post-tensioning usually balances most of the permanent loads thus significantly reducing deflections and tensile stresses.
- The P/A stress provided by post-tensioning may prevent tensile stresses causing the floor to crack.

For the above reasons post-tensioned construction has also come to be used in many situations in buildings. In addition to the above mentioned general features of post-tensioned construction systems, the following advantages of post-tensioned slabs over reinforced concrete slabs are listed as follows

- More economical structures resulting from the use of prestressing steels with a very high tensile strength instead of normal reinforcing steels.
- Larger spans and greater slenderness, which results in reduced dead load, which also has a beneficial effect upon the columns and foundations and reduces the overall height of buildings or enables additional floors to be incorporated in buildings of a given height.



Traditional R.C.C. design PT slab design

Fig. 1.1 Height comparison of R.C.C. design and PT slab design

- Under permanent load, very good behavior in respect of deflections and cracking.
- Higher punching shear strength obtainable by appropriate layout of tendons.
- Post-tensioning allows earlier striking of formwork which results in considerable reduction in construction time.

1.2 HISTORICAL BACKGROUND

Although some post-tensioned slab structures had been constructed in Europe quite early on, the real development took place in the USA and Australia. The first post-tensioned slabs were erected in the USA in 1955 using unbonded post-tensioning systems. In the succeeding years numerous post-tensioned slabs were designed and constructed. In early stage the construction of post-tensioned slab has been done by using the lift slab method. Post-tensioning enabled the lifting weight to be reduced and the deflection and cracking performance to be improved. Attempts were made to improve knowledge in depth by theoretical studies and experiments on post-tensioned plates. Joint efforts by researchers, design engineers and prestressing firms resulted in corresponding standards and recommendations and assisted in promoting the widespread use of this form of construction in the USA and Australia. To date, in the USA alone, more than 50 million m^2 of slabs have been constructed by using post tensioned systems.

In Europe renewed interest in this form of construction was again exhibited in the early seventies. Some constructions were completed at that time in Great Britain, the Netherlands and Switzerland. Intensive research work, especially in Switzerland, the Netherlands and Denmark and more recently also in the Federal Republic of Germany have expanded the knowledge available on the behavior of such structures These studies form the basis for standards, now in existence or in preparation in some countries. From purely empirical beginnings, a technically reliable and economical form of construction has arisen over the years as a result of the efforts of many participants. Thus the method is now also fully recognized in Europe and has already found considerable spreading various countries (in the Netherlands, in Great Britain, Switzerland and in India for example).

1.3 POST-TENSIONING METHODS

There are two methods of the post-tensioning systems as follows

a) Bonded post-tensioning
b) Unbonded post-tensioning

1.3.1 Bonded post-tensioning

As is well-known, in this method of post-tensioning the prestressing steel is placed in ducts, and after stressing is bonded to the surrounding concrete by grouting with cement suspension. Round corrugated ducts are normally used. For the relatively thin floor slabs of buildings, the reduction in the possible eccentricity of the prestressing steel with this arrangement is, however, too large, in particular at cross-over points, and for this reason flat ducts have become common. They normally contain tendons comprising four strand of nominal diameter 12.7 mm (0.5"), which have proved to be logical for constructional reasons.

1.3.2 Unbonded post-tensioning

In the early stages of development of post-tensioned concrete in Europe, post-tensioning without bond was also used to some extent. After a period without any substantial applications, some important structures have again been built with unbonded post-tensioning in recent years. In the first applications in building work in the USA, the prestressing steel was greased and wrapped in wrapping paper, to facilitate its longitudinal movement during stressing during the last few years. The strand is first given a continuous film of permanent corrosion preventing grease in a continuous operation, either at the manufacturer's works or at the prestressing firm. A plastics tube of polyethylene or polypropylene of at least 1 mm wall thickness is then extruded over this. The plastics tube forms the primary and the grease the secondary corrosion protection. Strands sheathed in this manner are known as monostrands. This method of producing the sheathing has become popular and common for the unbonded post-tensioning systems. The nominal diameter of the strands used is 12.7 mm (0.5") and 15 mm (0.6"); the later have come to be used more often in recent years.

As there are two methods of the post-tensioning the question arises that which one should be used? Which one is more effective than the other? This question was and still is frequently the subject of serious discussions. One can not say directly that this method is superior to the other. Each method has its own benefits and some disadvantages, sothat some important arguments in favor and against the bonded and unbonded post-tensioning systems are put forward which are as follows-

Arguments in favor of post-tensioning without bonding

- Maximum possible tendon eccentricities, since tendon diameters are minimal, of special importance in thin slabs.
- Prestressing steel protected against corrosion works.
- Simple and rapid placing of tendons.
- Very low losses of prestressing force due to friction.
- Grouting operation is eliminated.
- In general more economical.

Arguments for post-tensioning with bonding:

- Larger ultimate moment.
- Local failure of a tendon (due to fire, explosion, earthquakes etc.) has only limited effects.

Whereas in the USA post-tensioning without bonding is used almost exclusively, bonding is deliberately employed in Australia. Among the arguments for bonded post-tensioning, the better performance of the slabs in the failure condition is frequently emphasized. It has, however, been demonstrated that equally good structures can be achieved in unbonded post-tensioning by suitable design and detailing. It is always possible that local circumstances or limiting engineering conditions (such as standards) may become the decisive factor in the choice of the post-tensioning method.

1.4 OBJECTIVES IN THE DESIGN OF BUILDING

Now a day a number of buildings are constructed for the different purposes. Each building has its own requirements and objectives according to the purpose for which it is to be constructed. While constructing these building the most important factor is the economy and for economical point of view we have to save the construction cost of the building. If the objectives of the building are met then we can save the more cost of construction. For this purpose the buildings are classified in many different ways. They can be distinguished by their use or occupancy, by the construction materials, by their owners (public / private), or by their height (low-rise / high-rise).

In high rise building the construction progress is vertical. A typical high rise building is as shown is fig. 1.2. In this system there is a repetition of floors in the vertical direction and the floors are identical. In order to reduce the construction time the prime design objective is to achieve the fast floor cycle that is to minimize the time required to complete a floor. To minimize the size of vertical members and foundations the floor weight must be kept as low as possible and to maximize rentable space and flexibility of occupancy the floor framing is usually required to have relatively long spans, which is in conflict with the objective to minimize structural floor height and weight.

Fig. 1.2- Typical high rise building

In low rise building the construction progress is horizontal with some simultaneous vertical progression. A typical low rise building is as shown in fig. 1.3. The objectives of the construction in low rise building are same as for the high rise building. The low rise buildings are generally used for the industrial, retail and parking purpose. This often implies some specific requirements such as strict cracking and deflection limitations or avoidance of expansion joints.

Fig. 1.3- Typical low rise building

Post-tensioning helps to meet each objective in the design of the building. The reasons for this are different in each case. The most prominent ones are that post-tensioning allows the floor framing to be more slender, solving the problem of the conflicting needs for long spans and small structural depth, and that it replaces a significant amount of reinforcement, thus reducing steel quantities and allowing standardization and simplification of the reinforcement. Further, the reasons why post-tensioning helps to improve the design is that the concrete quantities are reduced and the formwork can be stripped earlier than for non-prestressed floors. Also, the often required strict limitation of deflections and crack widths can be effectively achieved by post-tensioning. Since the draped prestressing tendons typically balance a significant part of the permanent floor loading, deflections and cracking are substantially reduced compared to a reinforced floor. In addition, the in-plane compression forces from the prestressed tendons neutralize tensile stresses in the concrete to a degree, delaying the formation of cracks.

The table 1.1 summarizes the objective of the building and the benefits offered by these objectives to the project and suggestions as how each design objective can meet. These suggestions include the use of simple and efficient formwork, post-tensioning, pre-fabrication of reinforcing assemblages, complete or partial prefabrication of entire concrete elements, and the choice of a suitable floor framing system, simple details, high degree of standardization, and the use of high early strength concrete.

Table 1.1 Objectives of concrete floor design

Sr. No.	Objective	Benefits for the project	Suggestion
1	Smallest possible floor to floor height	Saving on vertical structural members, cladding, mechanical risers, lift stairs, air conditioning	Post-tensioning (greater span/depth ratio)
2	Largest possible column free space, i.e. long span	Flexibility of occupancy, maximum rentable space	Post-tensioning (greater span/depth ratio)
3	Lowest possible weight of floor	Saving on vertical structural members and foundation and, in seismic areas, on lateral load resisting system	Light-weight concrete, ribbed or waffle slab, slab with voids, post-tensioning (floors require less concrete)
4	High repeatability from floor to floor	Improvement of constructability and thus saving of time	Simple, standardized details for reinforcement and formwork, Post-tensioning (a significant part of the load is resisted by it sothat the non-prestressed reinforcement can be simplified and standardized to a large degree)
5	Quickest possible floor cycle	Saving of time, avoidance of clashes between different trades, reduction of required number of formwork sets	High early strength concrete, simple reinforcing and formwork in large pre-assembled units, simple details with high repeatability, pre-fabrication of critical path elements (columns, beams, slab soffits, or wall) Post-tensioning (allows earlier stripping of formwork), post-tensioning (avoids complicated reinforcing bar connections with insitu closure pours, or welded steel connectors while precsting element)
6	No back-propping wherever possible	Direct saving of time, indirect saving of time by allowing building fit-out to start earlier (early access)	Use of self-supporting formwork that only needs to be supported near vertical elements, high early strength concrete, post-tensioning (Usually the permanent floor load is largely balanced by draped tendons so that only the weight of the wet concrete of the floor above induces flexural stresses.

7	For some warehouse or industrial buildings: floor free of expansion joint	Improved riding surface, for fork lifts, easy-to-clean surfaces	Lay-out of columns and walls to avoid restraint of shrinkage and temperature shortening temporary or permanent separation of floor from restraining vertical elements, careful concrete mix design, careful curing of concrete, well distributed non- prestressed steel Post-tensioning (as it balances most of the permanent loads thus significantly reducing deflections and tensile stresses)
8	For parking (typically): lowest-possible floor-to-floor height	More rentable space for given total building Height, shorter ramps	Post-tensioning (the P/A stress may prevent tensile stresses causing the floor to crack)

1.5 APPLICATIONS OF POST-TENSIONING IN BUILDINGS

The multitudes of different floor systems a designer can choose from are reviewed with respect to the selection criteria and the compatibility with post-tensioning. Floor systems can be classified in different ways, for instance insitu versus precast floors, single span versus multi span floors, slab on beams versus flat slab, one-way versus two-way systems. In all these systems the post-tensioning can be used. As the present study is concerned with the post-tensioning in slabs (mainly two way flat slabs) and the beams, these are discussed in details in the succeeding chapters. Excluding these two components of a multi-storey building the post-tensioning also can be adopted economically for the following components of the building-

- Foundation slabs.
- Cantilevered structures, such as overhanging buildings.
- Facade elements of large area; here light post-tensioning is a simple method of preventing cracks.
- Main beams in the form of girders, lattice girders or north-light roofs.

Typical applications for post-tensioned slabs may be found in the frames or skeletons for office buildings, multi-storey car parking, school building, warehouses etc. and also in multistory flats where, for reasons of internal space, frame construction has been selected.

What are the types of slab system in which post-tensioning is used?

- For spans of 7 to 12 m, and live loads up to 5 kN/m^2, flat slabs or slabs with shallow main beams running in one direction without column head drops or flares are usually selected.
- For larger spans and live loads, flat slabs with column head drops or flares, slabs with main beams in both directions or waffle slabs are used.

As the post-tensioning systems are used in multi-storey buildings, the question asked by many contractors and owner is that, are there any disadvantages of these systems? In short the answer is no. One argument frequently used against post-tensioning by owners and contractors is the lack of flexibility to accommodate floor penetrations, either planned or as part of future changes to meet specific tenant requirements. For planned or future small penetrations the contrary is the case. Because of the reduced reinforcement content the rebars are usually spaced further apart, leaving more flexibility for small penetrations. There is no doubt that drilling or coring small holes into or through post-tensioned floors requires a certain amount of discipline in order to avoid cutting any post-tensioning strands. However, it is very simple to locate the tendons in a floor to make sure that small penetrations or fixing holes miss them. In most cases the construction drawings will give a good estimate where it is safe to drill. If the drawings show tendons close to the proposed penetration then the exact tendon location can readily be determined on site with the aid of a metal detector.

1.6 SCOPE OF THE PRESENT STUDY

The post-tensioning method is now a days increasing widely, due to its application. By using the post-tensioning method one can design the most economic and the safe design. But while using this method more precautions has to be made for the shear and the deflection criteria for the slabs. The design of the post-tension flat slab can be done by using load balancing and equivalent frame method. Among of both the equivalent frame method is widely used. In the load balancing method the 65 to 80% of the dead load is carried by the tendon itself. So that there is an upward deflection due to tendon profile resulting the reduction in overall deflection.

In the present study the design of the post-tensioned flat slab is done by using both methods, load balancing method and equivalent frame method. As the shear and deflection check is the most important for the post-tensioned slabs the detail design for the shear and deflections (short term deflection and long term deflections due to creep and shrinkage) is carried out. The parametric study of the post-tensioned flat slab by varying the span by 0.5m interval is done and results of the different parameters such as thickness of slab, grade of concrete, loss due to stress, normal reinforcement, reinforcement for the shear, number of tendons, stressing force per tendon and deflection etc. are presented in the graphical form. Continuing to this a design of post-tensioned beam is also done. For the study of post-tensioned slab and beams a case study of a multistory office building (G+4) is taken and it is designed by four cases, the post-tensioned flat slab, post-tensioned beams and the R.C.C. slab, only R.C.C. flat slab and the R.C.C. slab and beams. After the design of these four cases the comparative study with respect to the economy is carried out.

2. LITERATURE SURVEY

2.1 INTRODUCTION

Post-tensioning is a method of reinforcing (strengthening) concrete or other materials with high-strength steel strands or bars, typically referred as tendons. Post-tensioning applications include office and apartment buildings, parking structures, slabs-on-ground, bridges, sports stadiums, rock and soil anchors, and water-tanks. In many cases, post-tensioning allows construction that would otherwise be impossible due to either site constraints or architectural requirements.

The post-tensioning systems require specialized knowledge for the design of structure and expertises to fabricate assemble and install but the concept of post-tensioning is easy to explain. Imagine a series of wooden blocks with holes drilled through them, into which a rubber band is threaded. If one holds the ends of the rubber band, the blocks will sag. Post-tensioning can be demonstrated by placing wing nuts on either end of the rubber band and winding the rubber band so that the blocks are pushed tightly together. If one holds the wing nuts after winding, the blocks will remain straight. The tightened rubber band is comparable to a post-tensioning tendon that has been stretched by hydraulic jacks and is held in place by wedge-type anchoring devices. Some authors already discussed about the post-tensioning, load balancing concept for the post-tensioning buildings, shear and deflection criteria for the post-tensioning building. Some experimental work considering the different conditions for the post-tensioning has also carried out to study the concept of the post-tensioning.

2.2 LITERATURE SURVEY

The concept of load balancing was introduced by **T. Y. Lin** [1] for prestressed concrete structures, as a third approach after the elastic stress and the ultimate strength method of design and analysis. It was first applied to simple beams and cantilevers and then to continuous beams and rigid frames. Principles of load balancing for flat slabs, grid systems, and certain forms of shell and folded plates, the amount of load to be balanced by prestressing, accuracy and limitations of the load balancing are introduced. This load-balancing method represents the simplest approach to prestressed design and analysis. Its advantage over the elastic stress and ultimate strength methods is not significant for statically determinate structures. When dealing with statically indeterminate systems including flat slabs and certain thin shells, this load-balancing method offers tremendous advantage both in calculating and visualizing the parameters for the design. According to this load-balancing method, prestressing balances a certain portion of the gravity loads so that flexural members, such as slabs, beams, and girders, will not be subjected to bending stresses under a given load condition. Thus a structure carrying transverse loads is subjected only to axial stresses.

The load balancing method can be applied for the one, two and three dimensional load balancing. Two dimensional load balancing differs from linear load balancing for beams and columns in that the transverse component of the tendons in one direction either adds to or subtracts from that component in the other direction. Thus, the prestress design in the two directions is closely related one to the other. However, the basic principle of load balancing still holds, and the main aim of design is to balance a given loading so that the entire structure (weather a slab or grid) will posses uniform stress distribution in each direction and will have no deflection nor camber under this loading. Any deviation from this balance loading will then be analyzed as loads acting on an elastic slab, without further considering the transverse component of prestress.

Load-balancing method transforms a prestressed structure into a nonprestressed one subjected only to the unbalanced portion of the loading. Hence it reduces the design and analysis of prestressed structures to a point simpler than that of conventional structures. Its application to statically indeterminate beams and frames not only saves the time but also presents a realistic approach which helps the engineer to visualize the effect of prestressing. Indeed, the horizon of prestressing will be greatly extended when this method is applied to grids, slabs, thin shells and folded plates. Furthermore, the method is easily adaptable to a combination of prestressed and reinforced concrete where varying amount of the live load may be carried by nonprestressed reinforcement. Attention is again called to the occasional necessity of checking for proper behavior under full live load as well as the capacity to resist ultimate load.

Elastic behavior and ultimate strength of a continuous concrete slab prestressed in two directions were investigated by **A. C. Scordelis, T. Y. Lin and R. Itaya** [2]. The slab, consisting of four panels, supported at nine points and simulated a flat slab. Prestressing was accomplished by means of unbonded post-tensioned cables. Experimental values for moments, deflections, and reactions were compared with theoretical values. The elastic plate theory and approximate theories were used for calculating the values of moment, deflection and the reactions in present design method. The purpose of this investigation was to determine the behavior, through and above the elastic range, of a continuous concrete slab prestressed in two directions. For this study a 15ft x 15ft slab 3in. thick and supported at nine points was prestressed. The slab was post-tensioned with 12 cables in each direction, spaced at 15in. on centers. Each cable consist of a single ¼ in. high strength steel wires greased and placed in a plastic tube to provide for post-tensioning. The concrete for the slab was proportioned for a minimum strength of 35 N/mm^2 at 28 days. Cable prestress was applied by a 30-ton capacity hydraulic jack. For slab uniform load had been provided on each of the four panels independently. For that air bags and plywood sheathing supported with steel framing was used.

Uniform load is applied to the slab. The test on the slab consisted of subjecting it to an increasing uniform live load on all four panels until failure. The first tensile crack, as indicated by strain readings, seems to have occurred over the center support. Cracks were then observed at the edges of the slab. The first cracks on the bottom of the slab were began at the edges of the slab and extended inward about 2 ft. On further increasing load, the crack opened to 1/8 in. wide and extended across the width of slab. After flexural cracking, ultimate failure occurred at a further increase in the load, with the center support punching through the slab. The failure occurred directly around the edges of the 9 x 9 in. center support at a shear angle of about 45^0. On the basis of the studies carried out, the following conclusions are made

- The elastic plate theory may be used satisfactorily to predict the behavior of a prestressed concrete slab loaded within the elastic range.

- The cracking load has little practical significance since initial cracking is localized at points of high moments. The slab can sustain large increase in load before widespread cracking takes place.
- Moments due to only equal prestress in all cables can be calculated with sufficient accuracy for design purposes by the beam method.
- A quantitative study of the results indicates that for elastic behavior under uniform load the total negative moment calculated by the beam method should be distributed approximately 75% to column strips and 25% to the middle strips, while the total positive moment calculated by beam method should be distributed approximately 60% to the column strips and 40% to the middle strips.
- Deflections under uniform load obtained by the beam method are within 15% of those obtained experimentally of by the elastic plate theory.
- For the design live loads acting on one panel only, in combination with dead load and uniform prestress, small and relatively insignificant tensile stresses are produced in the slab.

The tentative recommendations are presented by **ACI-ASCE Committee 423** [3] as a guideline to the design of prestressed concrete flat plates in post-tensioned buildings with bonded or unbonded tendons. The guideline for the tendon distribution, spacing of the tendons and the nonprestressed reinforcement are described here. Some of the guidelines are as follows.

The ultimate strength of a flat plate is controlled primarily by the total amount of tendons in each direction. However, tests indicate that tendons passing through columns or directly around the column edges contribute more to load carrying capacity than tendons remote from the columns. For this reason, it is recommended that some tendons should be placed through the columns or at least around their edges. In lift slab construction, some tendons should be placed over the lifting collars. For panels with length-width ratios not exceeding 1.33 the following approximate distribution may be used

- Simple spans, 55 to 60 percent of the tendons in the column strip with the remainder in the middle strip.

- Continuous spans, 65 to 75 percent of the tendons in the column strip with a remainder in the middle strip.

The tendon spacing recommendations have been arbitrarily determined based on satisfactory field performance. As a general guide, the recommended maximum spacing of tendons in the column strip is about four times the slab thickness, and the recommended maximum spacing in the middle strips is about six times the slab thickness. However, for very short spans, tendon spacing up to eight times the slab thickness may be appropriate.

The minimum amount of nonprestressed bonded reinforcement placed in the top of post-tensioned flat plates in each direction in column areas should be 0.15 percent of the cross-sectional area of the column strip. Within the limitations, of details and minimum bar spacing specifications, this reinforcement should be concentrated directly over and immediately adjacent to the column. The spacing of the bars should not exceed 300mm (12 in.) on centers, and not less than four bars should be used in each direction. For normal span ratios, these bars should have a total length equal to 1/3 of the span (use average span length when adjacent spans are unequal in length). For the openings in post-tensioned flat plates tendons should be continuous and splayed horizontally to get around smaller openings. If tendons are terminated at edges of larger openings, such as at stairwells, an analysis should be made to insure sufficient strength and proper behavior. Edges around openings may be reinforced in a manner similar to conventionally reinforced slabs, or, in the case of larger openings, supplementary post-tensioning tendons may be used to strengthen the edges.

Some nonprestressed reinforcement should be added at end anchorages to avoid possible splitting of the concrete. Two bars are commonly used continuously around the perimeter of the slab behind the anchorages for this purpose. For highly prestresses slabs, or where anchorages are concentrated in a narrow slab width, the need for reinforcement to resist horizontal splitting of the slab should be investigated. Additional special reinforcement, required for the performance of the anchorage, should be indicated by the tendon supplier.

The comparative study of prestressed concrete beams with and without bond was carried out by **Alan H. Mattock, Jun Yamazaki and Basil T. Kattula** [4]. For that they made seven simple beams of 28 ft. span and three beams continuous over two spans of 28 ft. each. The primary variable was presence or absence of bond, the amount of unprestressed reinforcement. For the unprestressed reinforcement they used seven wire strands. The unbonded post-tensioned beams with minimum recommended unprestressed reinforcement had serviceability characteristics, strength and ductility equal to or better than bonded post-tensioned beams. Seven wire strands can be used effectively as additional unprestressed bonded reinforcement in unbonded post-tensioned beams. Redistribution of design support ultimate moments by an amount equal to the positive secondary prestress moment should be allowed in design without a special limitation on amount of reinforcement. The minimum amount of bonded unprestressed reinforcement that should be provided in an unbonded post-tensioned beam is 0.4 percent of the area of that section between the flexural tension face and the neutral axis of the gross section. This will ensure satisfactory serviceability characteristics in beams with unbonded tendons.

A technical note on bonded reinforcement required to supplement post-tensioning tendons in building is given by **Bijan O. Aalami** [5]. As with conventionally reinforced structures, post-tensioned members must be designed for both serviceability and strength. The serviceability design of post-tensioned floor systems includes a check for cracking and deflection under service loads. Cracking is controlled by using a hypothetically calculated tensile stress. In the bonded post-tensioning systems, supplemental bonded reinforcement is not required if the post-tensioning meets the stress requirements of the code under service loading and the post-tensioning by itself is adequate strength requirements. Hence it is possible to construct a slab reinforced with grouted tendons. Strength requirements during construction and construction sequence should be reviewed carefully when this method of construction is used. On the other hand the floor systems constructed with unbonded tendons, the minimum bonded reinforcement is required. The objective of this minimum reinforcing is crack control under service loads and adequate ductility when member is overloaded. A minimum area of bonded reinforcement must be provided at the supports, regardless of the service load stress condition. The area of this reinforcement is a function of the geometry of the slab and the support layout. The minimum reinforcement is given by

$$A_s = 0.00075 * A_{cf} \qquad \ldots (2.1)$$

Where,

A_s = area of reinforcement, and

A_{cf} = larger gross cross sectional area of the design strips of the two orthogonal directions inter section at the support under consideration.

The area A_s is calculated separately for each of the two orthogonal directions at a support. The larger of two values is selected for the support. Hence the area of minimum rebar will be same in both directions.

In the paper published by **James Loper** [6] on the BWI Airport, they have given the concepts and options of post-tensioning. Two options for post-tensioning parking garages are unbonded and bonded systems. The characteristic feature of an unbonded tendon is that it does not form a bond along its length in the concrete. Unbonded tendons are comprised of single strands (monostrands) covered with a grease coating and enclosed in high-density plastic extruded sheathing. Primarily, the end anchors transfer the force in the stressed tendon to the concrete. A bonded tendon is usually comprised of multiple post-tensioning strands and, by design, forms a continuous bond along its length with the surrounding concrete slab, beam, or girder. A cementitious grout that surrounds the strands bonds them to the concrete. The grout acts with the duct that is encased in the concrete member to complete the bond path between the post-tensioning strands and the concrete member. Flat, corrugated, polypropylene (PP) ducts that accommodates two to five strands are used in thinner members such as slabs, whereas larger, round ducts (PP or galvanized metal) are used in beams and girders. Bonded systems are becoming more popular with long-term owners such as airports, hospitals, government agencies, and universities. This interest is based on the life-cycle economies associated with bonded systems compared to unbonded systems. Bonded systems offer a significant design advantage that leads to life-cycle savings.

The key design feature is that the hardened grout locks the movement of the post-tensioning strands to that of the surrounding concrete. Hence, the force in a bonded strand is a function of the deformation of the surrounding concrete. This is the well-known concept of strain compatibility and internal equilibrium used in reinforced concrete design. Another design advantage of bonded post-tensioning is the inherent capacity to provide resistance to progressive collapse. This may be especially important in the event of localized blast loading. Like conventional steel reinforcement, a bonded post-tensioning tendon is capable of developing its force at a relatively near distance along its length. In the event that an anchorage fails or a strand is severed, the loss of tendon force would be localized. The remainder of the tendon would retain its force at the development length away from the failure point and would remain functional. This functionality may be used in the design phase when planning for alternative load paths. Bonded systems also offer several practical benefits compared to unbonded systems. The most important benefit is reduction of steel reinforcing bars, particularly at the top of slabs. This is especially important because most parking garage maintenance costs are due to repairs associated with spalled concrete and corroded reinforcement. Another benefit is complete encapsulation i.e. strands are fully protected by cementitious grout, the duct, and the surrounding concrete. Bonded systems also offer more flexibility regarding structural modifications such as openings for stairwells, utility access, and future expansion.

In a typical slab carrying gravity loads, shear and unbalanced moment will be present at the edge –column connections. The transfer of unbalanced bending moment causes the distribution of shear stress in the slab around the column to become nonuniform and reduces the shear strength of the connection. In tests of post-tensioned concrete slab-edge column connections by **Douglas A. Foutch** and **William L. Gamble** [7], they carried out experimental investigation to study the strength and behavior of prestressed slab-edge column connections with unbonded tendons under the static loading. For this they constructed four two-third scale models of slab-edge columns among which two specimens had many closely spaced tendons while the other two had only a few widely spaced tendons. Each slab consist of 1524 mm square prestressed concrete slab 100 mm thick and 300 mm square column located adjacent to and centered along the edge of slab. The geometry of each specimen is symmetrical. 10 mm deformed bars were used as bonded reinforcement in the vicinity of column for crack control as recommended by the code. After casting of specimens they were post-tensioned after seven days. The tendons were prestressed individually using 270 KN hydraulic jacks.

By applying the loads (line load), the specimens were tested and different readings were taken with the help of dial gauges. The moment, shear and edge deflection for each specimen is calculated from the experiment data. The moment capacity obtained by the experiment is larger than the calculated moment capacity for each specimen. The some conclusions made on the basis of the experiment are as follows

- It is apparent that considerable rotation occurs at the face of the column and this made the largest contribution to the edge displacement.
- The section contains both post-tensioned unbonded tendons and bonded 10mm reinforcing bars; these both made the significant contribution in the moment capacity.
- It has been recognized that the shear strength of a slab-column connection cannot be considered independently of the flexural behavior.
- All the specimens resisted the larger forces than predicted on the basis of limiting shear stress.
- The moment deflection behavior was linear to the first cracking point. Thereafter, the stiffness of the slab decreased gradually to the point of yielding of the bonded steel. And after yielding of steel occurred, the stiffness of the slab decreased more rapidly.
- As the ultimate moment was approached, crushing began to develop at the intersection of the bottom surface of the slab and the column face.
- The first crack was developed at the face of the column and then it increases towards the other end of the slab.
- The average stress increase across the width of slab ranged from 65 to 81% if the maximum stress increase.

- The failures of the two different specimens are different i.e. for the first two specimens (with closely spaced tendons) fail in flexure and other two (with widely spaced tendons) fail in shear.

The effects of combined gravity and lateral loading on the reinforced concrete flat plate were presented in the experimental study of slab-column connections by **Austin D. Pan** and **Jack P. Moehle**[8]. For the present study four specimens of interior slab-column connection of a building were considered. By doing the experimental setup the gravity load and the lateral load (biaxial and uniaxial) were applied to the specimens. While testing different measuring instruments were installed to obtain the data on lateral forces, vertical reactions, slab displacements, rotation, twist, column deformation and secondary displacements. After obtaining the results of the test a through study was made on these results and finally some following conclusions were find out.

- Biaxial lateral loading reduces the lateral stiffness, strength and available drift capacity of reinforced slab-column connection.
- The magnitude of gravity load shear carried by the slab is a primary variable affecting the lateral behavior of the reinforced concrete flat plates.
- Significant increase in the strength, stiffness and displacement capacities were observed in the test specimens when gravity shear stress on the slab critical section was reduced.
- The magnitude of gravity load and lateral inter-story drift should be controlled t insure that the integrity of slab-column connections is maintained under seismic loading.
- Actual lateral inter-story drift under an extreme earthquake loading should not exceed 1.5% of inter-story height.
- Continuous bottom slab reinforcement should be placed directly over the columns of flat plates to prevent progressive collapse in the event of connection shear failure. Bottom bars effectively suspend the slab after punching failure and enable the slab-column connection to sustained gravity loads under continued cycles of lateral loading.

3. ULTIMATE AND SERVICEABILITY LIMIT STATE

3.1 ULTIMATE LIMIT STATE

3.1.1 Flexure

Bonded and unbonded post-tensioned slabs can be designed according to the known methods of the theories of elasticity and plasticity in an analogous manner to ordinarily reinforce slabs. We can distinct these methods according to their use such as

A. Calculation of moments and shear forces according to the theory of elasticity; the sections are designed for ultimate load.

B. Calculation and design according to the theory of plasticity.

Method A

In this method, still frequently chosen today, moments and shear forces resulting from applied loads are calculated according to the elastic theory for thin plates by the method of equivalent frames, by the beam method or by numerical methods (finite differences, finite elements etc.). The prestress should not be considered as an applied load. It should intentionally be taken into account only in the determination of the ultimate strength. No moments and shear forces due to prestress and therefore also no secondary moments should be calculated.

The moments and shear forces due to applied loads multiplied by the load factor must be smaller at every section than the ultimate strength divided by the cross-section factor. The ultimate limit state condition to be met may therefore be expressed as follows:

$$S \cdot \gamma_f \leq R / \gamma_m \qquad \text{. . . (3.1)}$$

Where,

S = Moments and shear forces due to applied loads

R = Ultimate strength of the cross section

γ_f = Load factor (partial safety factor)

γ_m = Cross sectional factor (partial safety factor)

This apparently simple and frequently encountered procedure is not without its problems. Care should be taken to ensure that both flexure and torsion are allowed for at all sections (and not only the section of maximum loading). If carefully applied this method, which is similar to the static method of the theory of plasticity, gives an ultimate load which lies on the safe side. Sometimes the forces resulting from the curvature of prestressing tendons (transverse components) are also treated as applied loads. This transverse component of the forces should not be considered for the ultimate load calculation, since in slabs we determine the secondary moments and therefore a correct ultimate load calculation is difficult. The consideration of transverse components does however illustrate very well the effect of prestressing in service state. It is therefore highly suitable in the form of the load balancing method for calculating the deflections.

Method B

In practice, the theory of plasticity is being increasingly used for calculation and design. The following explanations show how its application to flat slabs leads to the ultimate load calculation which will be easily understood. The condition to be fulfilled at failure here is

$$(g+q)_u / (g+q) \geq \gamma \qquad \text{. . . (3.2)}$$

Where,

$\gamma = \gamma_f \cdot \gamma_m$

g= self weight of slab

q= distributed variable load

γ_f = Load factor (partial safety factor)

γ_m = Cross sectional factor (partial safety factor)

$(g+q)_u$ = ultimate design load

$(g+q)$ = service load

The ultimate design loading $(g+q)_u$ divided by the service loading $(g+q)$ must correspond to a value at least equal to the safety factor γ. The simplest way of determining the ultimate design loading $(g+q)_u$ is by the kinematic method, which provides an upper boundary for the ultimate load. The ultimate load calculation can then be carried out for a strip of unit width. The top layer ordinary reinforcement should be concentrated over the columns. The load corresponding to the individual mechanisms can be obtained by the principle of virtual work. This principle states that, for a virtual displacement, the sum of the work W_e, performed by the applied forces and of the dissipation work W_i, performed by the internal forces must be equal to zero.

$$W_e + W_i = 0 \qquad \text{. . . (3.3)}$$

Where,

W_e = virtual work of applied forces

W_i = virtual work of internal forces

In special cases with irregular plan shape simple equilibrium considerations (static method) very often prove to be a suitable procedure. This leads in the simplest case to the carrying of the load by means of beams (beam method). The moment distribution according to the theory of elasticity may also be calculated with the help of computer programme and internal stress states may be superimposed upon these moments. The design has then to be done according to Method A.

3.1.2 Ultimate strength of a cross-section

For given dimensions and concrete qualities, the ultimate strength of a cross-section is dependent upon the following variables:

- Ordinary reinforcement
- Prestressing steel, bonded or unbonded
- Membrane effect

The membrane effect is usually neglected when determining the ultimate strength. In many cases this simplification constitutes a considerable safety reserve. The ultimate strength due to ordinary reinforcement and bonded post-tensioning can be calculated on the assumption, which in slabs is almost always valid, that the steel yields. This is usually true also for cross-sections over intermediate columns, where the tendons are highly concentrated. In bonded post- tensioning, the prestressing force in cracks is transferred to the concrete by bond stresses on either side of the crack. Around the column mainly radial cracks open and a tangentially acting concrete compressive zone are formed. Thus the so called effective width is considerably increased. In unbonded post-tensioning, the prestressing force is transferred to the concrete by the end anchorages and, by approximation, is therefore uniformly distributed over the entire width at the columns. For unbonded post-tensioning steel, the question of the steel stress that acts in the ultimate limit state arises. If this steel stress is known, the ultimate strength of a cross-section (plastic moment) can be determined from fig.3.1 as follows

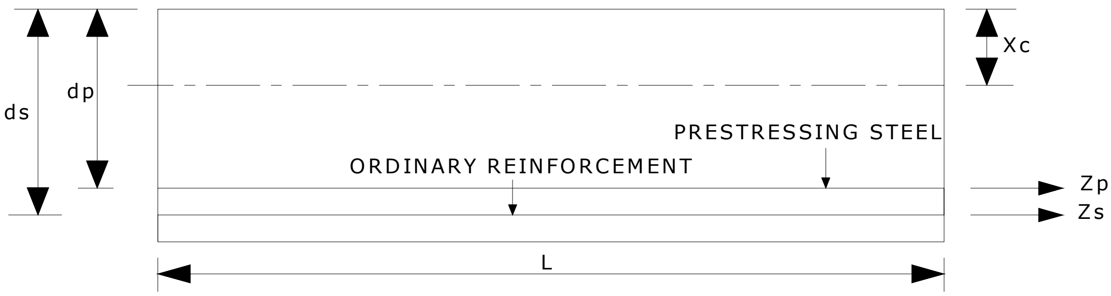


Fig. 3.1 Ultimate strength of a cross section

$$M_u = Z_s.(d_s - x_c)/2 + Z_p.(d_p - x_c)/2 \qquad . . . (3.4)$$

Where,

Z_s = Tension force due to ordinary reinforcement.

$Z_s = A_s.f_{sy}$

Z_p = Tension force due to post-tensioned reinforcement.

$Z_p = A_p.(\sigma_p + \Delta\sigma_p)$

A_p = cross sectional area of post-tensioned steel.

A_s = cross sectional area of ordinary reinforcement.

f_{sy} = yield strength of reinforcing steel.

σ_p = stress in post-tensioning steel.

$\Delta\sigma_p$= increase of stress in prestressing steel.

x_c = Depth of concrete compression zone.

$x_c = (Z_s+Z_p)/b.f_{cd}$

f_{cd} = design value of compressive strength of concrete.

b = width of the section.

d_p = effective depth of post-tensioned reinforcement.

d_s = effective depth of ordinary reinforcement.

3.1.3 Stress increase in unbonded post-tensioned steel

The stress increase in the unbonded post-tensioned steel has either been neglected or introduced as a constant value or as a function of the reinforcement content and the concrete compressive strength. A differentiated investigation shows that this increase in stress is dependent both upon the geometry and the deformation of the entire system. There is a substantial difference depending upon whether a slab is laterally restrained or not. In a slab system, the internal spans may be regarded as slabs with lateral restraint, while the edge spans in the direction perpendicular to the free edge or the cantilever, and also the corner spans are regarded as slabs without lateral restraint. The stress increase in the unbonded post-tensioned steel at a nominal failure state is estimated and is incorporated into the calculation together with the effective stress present (after losses due to friction, shrinkage, creep and relaxation). The nominal failure state is established from a limit deflection a_u. With this deflection, the extensions of the prestressed tendons in a span can be determined from geometrical considerations. Where no lateral restraint is present (edge spans in the direction perpendicular to the free edge or the cantilever, and corner spans) the relationship between tendon extension and the span L is given by

$$\Delta L/L = 4.\ (a_u/L).\ (y_p/L)$$

$$\Delta L/L = 3.\ (a_u/L).\ (d_p/L) \qquad ...\ (3.5)$$

Where,

$y_p = 0.75.d_p$

For a rigid lateral restraint (internal spans) the relationship for the tendon extension can be calculated approximately as

$$\Delta L/L = 2.\ (a_u^2/L) + 4.\ (a_u/L).(h_p/L) \qquad ...\ (3.6)$$

The stress increase is obtained from the actual stress-strain diagram for the steel and from the elongation of the tendon ΔL uniformly distributed over the free length L’ of the tendon between the anchorages. In the elastic range and with a modulus of elasticity E_p for the prestressing steel, the increase in steel stress is found to be

$$\Delta\sigma_p = (\Delta L/L).(L/L').E_p$$

$$\Delta\sigma_p = (\Delta L/L').E_p \qquad ...\ (3.7)$$

Where,

ΔL = tendon elongation.

L = length of span.

a_u = limit deflection.

y_p = internal lever arm (post-tensioning steel).

h_p = sag of tendon parabola.

L' = free length of tendon between two anchorage.

$\Delta\sigma_p$= increase of stress in prestressing steel.

d_p = effective depth of post-tensioned reinforcement.

E_p = modulus of elasticity of prestressing steel.

The steel stress, plus the stress increase $\Delta\sigma_p$ must, of course, not exceed the yield strength of the steel. In the ultimate load calculation, care must be taken to ensure that the stress increase is established from the determining mechanism.

3.1.4 Punching shear

Punching shear has a position of special importance in the design of flat slabs. Slabs, which are practically always under-reinforced against flexure, exhibit pronounced ductile bending failure. In beams, due to the usually present shear reinforcement, a ductile failure is usually assured in shear also. Since slabs, by contrast, are provided with punching shear reinforcement only in very exceptional cases, because such reinforcement is avoided if at all possible for practical reasons, punching shear is associated with a brittle failure of the concrete.

In the last twenty years, numerous design formulae have been developed, which were obtained from empirical investigations and, in a few practical cases, by model repetition. The calculation methods and specifications in most common use today limit the nominal shear stress in a critical section around the column in relation to a design value.

3.1.4.1. Influence of post tensioning

Post-tensioning can substantially alleviate the punching shear problem in flat slabs if the tendon layout is correct. A portion of the load is transferred by the transverse components resulting from prestressing directly to the column. The tendons located inside the critical shear periphery can still carry loads in the form of a cable system even after the concrete compressive zone has failed and can thus prevent the collapse of the slab. The zone in which the prestress has a load relieving effect is here intentionally assumed to be smaller than the punching cone.

After the shear cracks have appeared, the tendons located outside the critical shear periphery rupture the concrete vertically unless heavy ordinary reinforcement is present, and they can therefore no longer provide a load bearing function.

If for constructional reasons it is not possible to arrange the tendons over the column within the critical shear periphery or column strip, then the transfer of the transverse components resulting from tendons passing near the column should be investigated with the help of a space frame model. The distance between the outermost tendons to be taken into account for direct load transfer and the edge of the column should not exceed d_s on either side of the column.

3.1.4.2. Carrying out the calculation

The shear stress in case of the flat slab is calculated by the following procedure. Critical section for the shear is at d/2 from the periphery of column/capital/drop panel; perpendicular to the plane of slab (IS 456:2000 Clause 31.6.6). Generally in flat slab the shear check is given at the interior column, exterior column and at the edge column. The procedure to calculate the shear at each section of the slab is as explained as follows.

(a) Shear stress in the interior column

In this case centroid of the critical section coincides with the symmetrical axis of the cross section resisting shear i.e. AB, DC and AD, BC as shown in fig 3.2.

Resultant shear stress is given by

$$\tau_{AB} = V/A + [(1 - \alpha). M.X_{AB} / J] \quad \text{. . . (3.8)}$$

$$\tau_{CD} = V/A - [(1 - \alpha. M.X_{CD} / J] \quad \text{. . . (3.9)}$$

Where,

A = area of concrete resisting shear along the critical section

$= b_o.d = [2(c_1 + d) + 2(c_2 + d)]. d$

$= 2.d.(c_1 + c_2 + 2d)$

$X_{AB} = X_{CD} = (c_1 + d) / 2$ - (due to symmetry)

J = properties analogous to polar moment of inertia of the critical section about the axis about which moment acts.

= polar moment of inertia of the shear surface over sides AD and BC plus moment of inertia of two surfaces over AB and CD about X-X axis.

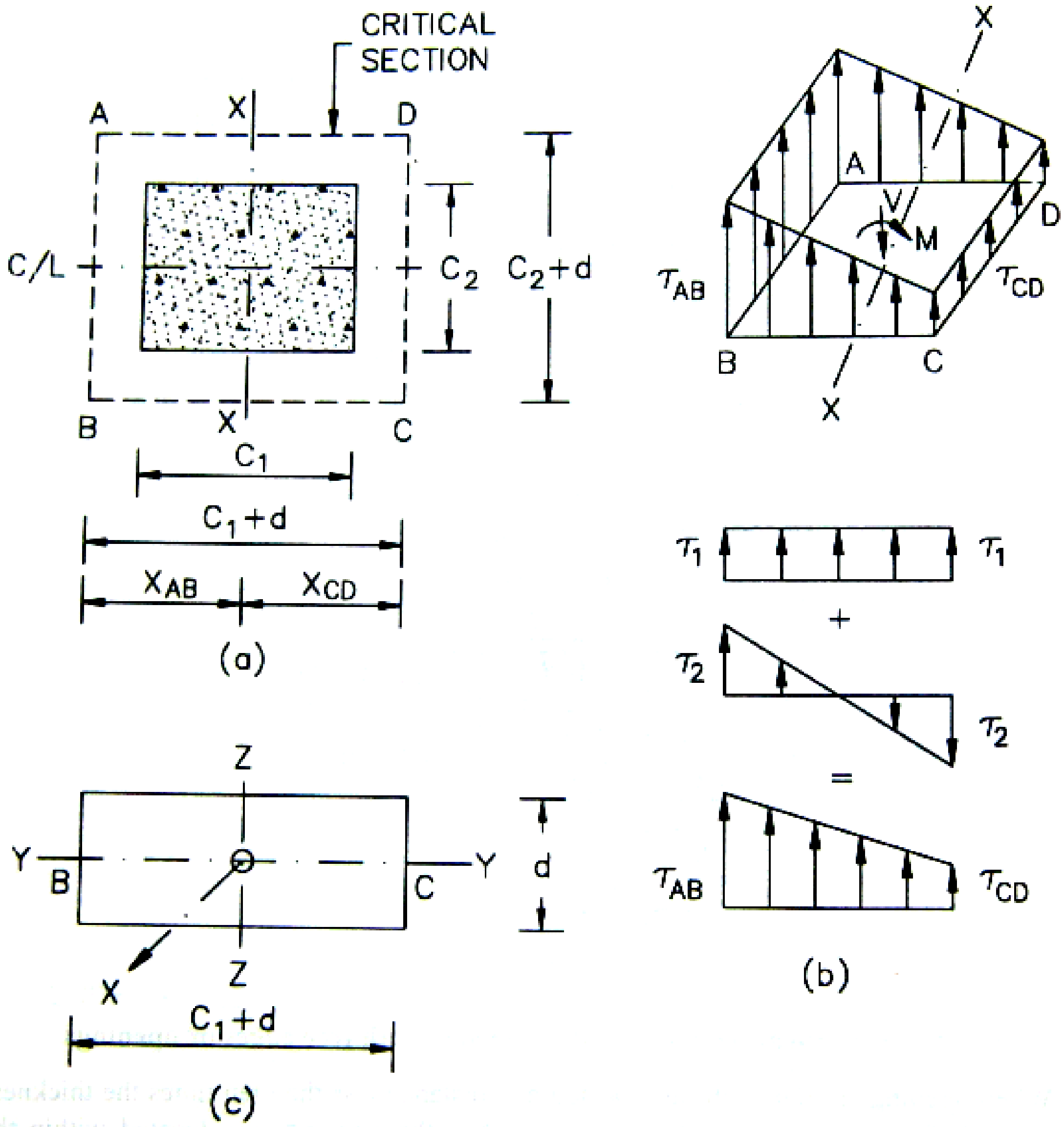


Fig. 3.2 Shear stress distribution around interior panel

J_{BC} = J_{AD} = $I_{yy} + I_{zz}$

$$= (c_1 + d).d^3 / 12 + (c_2 + d)3.d / 12 \qquad \text{. . . (3.10)}$$

J_{AB} = J_{CD} = area x distance^2

$$= (c_2 + d).d.[(c_1 + d)/2]^2 \qquad \text{. . . (3.11)}$$

$J = J_{AB} + J_{CD} + J_{BC} + J_{AD}$

(b) Shear stress in the edge column

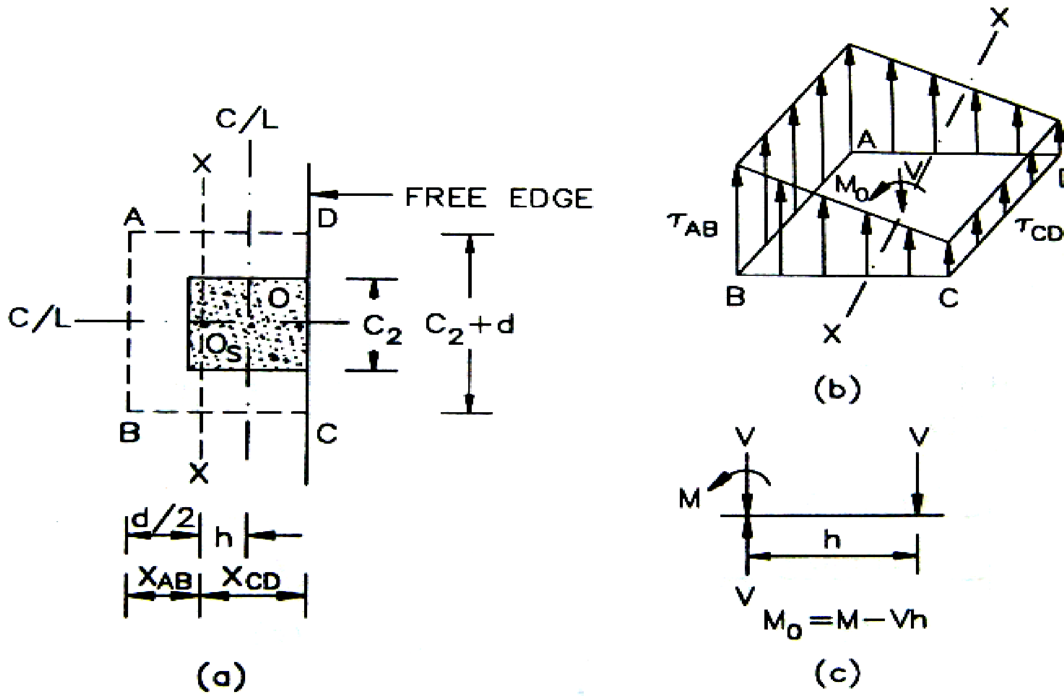


Fig. 3.3 Shear stress distribution around an edge column

Taking moment of shear areas about the face AB (fig. 3.3)

$$X_{AB} = [2(c_1 + d/2).d.(c_1 + d/2)/2] / A$$

$$X_{AB} = (c_1 + d/2)2.d / A \qquad \text{. . . (3.12)}$$

$$X_{CD} = (c_1 + d/2) - X_{AB} \qquad \text{. . . (3.13)}$$

The shear stress is given by

$$\tau_{AB} = V/A + [(1 - \alpha).(M - Vh).X_{AB} / J] \qquad \text{. . . (3.14)}$$

$$\tau_{CD} = V/A - [(1 - \alpha).(M - Vh).X_{CD} / J] \qquad \text{. . . (3.15)}$$

Where,

$$h = 0.5(c_1 + d/2) - X_{AB}$$

$$A = 2(c_1 + d/2).d + (c_2 + d).d$$

$$A = (2c_1 + c_2 + 2d).d$$

$$J_{BC} = J_{DA} = (c_1 + d/2).d^3/12 + (c_1 + d/2)^3.d/12 + (c_1 + d/2).d.[(c_1 + d/2)/2 - X_{AB}]^2 \qquad \text{. . . (3.16)}$$

$$J_{AB} = (c_2 + d).d.[(c_1 + d/2).d/A]^2 \qquad \text{. . . (3.17)}$$

$$J = J_{AB} + J_{BC} + J_{AD}$$

(c) Shear stress at the corner column

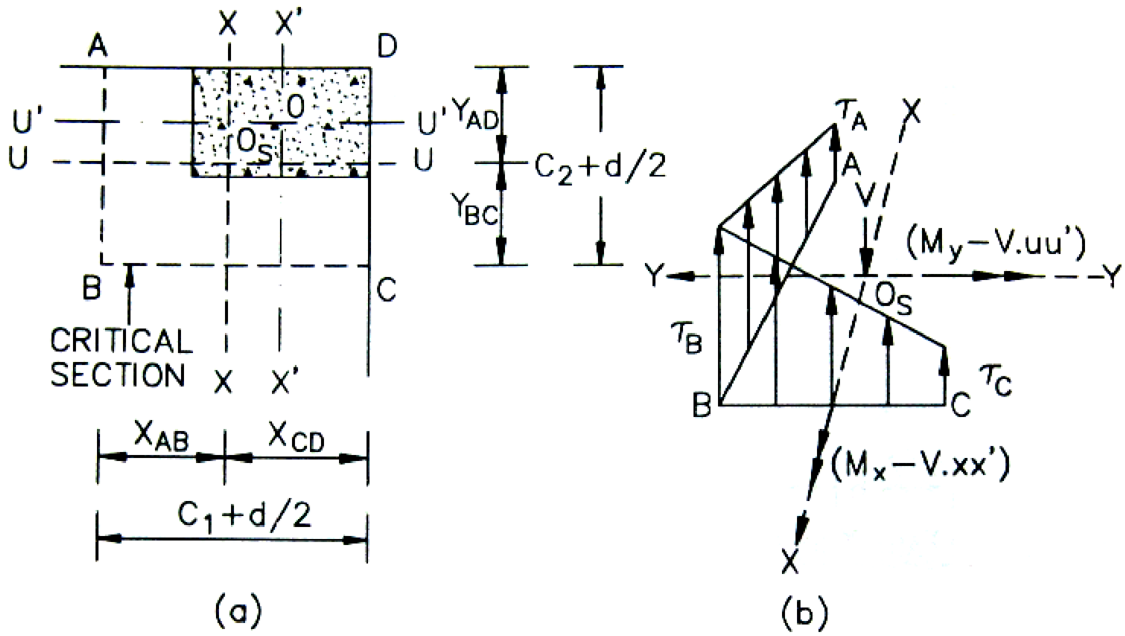


Fig. 3.4 Shear stress distribution around a corner column

Shear stress are given by (fig. 3.4)

$$\tau_A = V/A + [(1 - \alpha_x).(M_x - V.xx').\ X_{AB} / Jx] - [(1 - \alpha_y).\ (M_y - V.uu').\ Y_{AD}/ J_y] \quad ...\ (3.18)$$

$$\tau_B = V/A + [(1 - \alpha_x).(M_x - V.xx').\ X_{AB} / Jx] - [(1 - \alpha_y).\ (M_y - V{*}uu').\ Y_{BC}/ J_y] \quad ...\ (3.19)$$

$$\tau_C = V/A - [(1 - \alpha_x).(M_x - V.xx').\ X_{CD} / Jx] + [(1 - \alpha_y).\ (M_y - V{*}uu').\ Y_{BC}/ J_y] \quad ...\ (3.20)$$

Where,

$$A = (c_1 + d/2).d + (c_2 + d/2).d$$

$$A = (c_1 + c_2 + d).d$$

$$J_X = (c_1 + d/2).d^3 / 12 + (c_1 + d/2)^3.d / 12 + (c_1 + d/2).d.[(c_1 + d/2)/2 - X_{AB}]^2 + (c_2 + d/2).d.\ X_{AB}^2 \quad ...\ (3.21)$$

$$J_y = (c_2 + d/2).d^3 / 12 + (c_2 + d/2)^3.d / 12 + (c_2 + d/2).d.[(c_2 + d/2)/2 - Y_{BC}]^2 + (c_1 + d/2).d.\ Y_{BC}^2 \quad ...\ (3.22)$$

$$X_{AB} = (c_1 + d/2).d.(c_1 + d/2) / 2A$$

$$= (c_1 + d/2)^2.d\ / 2A \quad ...\ (3.23)$$

$X_{CD} = (c_1 + d/2) - X_{AB}$ . . . (3.24)

$y_{BC} = (c_2 + d/2).d.(c_2 + d/2) / 2A$

$= (c_2 + d/2)^2.d / 2A$ . . . (3.25)

Permissible shear stresses are as

The shear strength of concrete in slab is given by

$\tau_C' = Ks * \tau_C$ . . . (3.26)

Where,

$Ks = 0.5 + \beta \leq 1$

β = ratio of short side to long side of the column or column head

$\tau_C = 0.25 \sqrt{f_c}$ (Ref. IS 456 : 2000 Clause 36.6.3.1).

If, $\tau_C' < \tau_v < 1.5\tau_C'$, Shear reinforcement is provided in the slab.

If, $\tau_v > 1.5\tau_C'$, Flat slab is redesigned. (Ref. IS 456:2000 Clause 36.6.3.2).

While designing the shear reinforcement, the shear stress carried by the concrete is assumed to be 0.5 τ_C and reinforcement carries remaining shear.

3.2 SERVICEABILITY LIMIT STATE

3.2.1 Crack limitation

In slabs with ordinary reinforcement or bonded post-tensioning, the development of cracks is dependent essentially upon the bond characteristics between steel and concrete. The tensile force at a crack is almost completely concentrated in the steel. This force is gradually transferred from the steel to the concrete by bond stresses. As soon as the concrete tensile strength or the tensile resistance of the concrete tensile zone is exceeded at another section, a new crack forms. The influence of unbonded post-tensioning upon the crack behavior cannot be investigated by means of bond laws. Only very small frictional forces develop between the unbonded stressing steel and the concrete. Thus the tensile force acting in the steel is transferred to the concrete almost exclusively as a compressive force at the anchorages.

Theoretical and experimental investigations have shown that normal forces arising from post-tensioning or lateral membrane forces influence the crack behavior in a similar manner to ordinary reinforcement. In the ordinary reinforcement content p* required for crack distribution is given as a function of the normal force arising from prestressing and from the lateral membrane force n.

$$p^* = p_p - n/(d_p.\sigma_{po}) \quad ...(3.27)$$

Where,

p^* = reinforcement content

p_p = content of prestressing steel

σ_{po}= stress in post-tensioned steel

n = lateral membrane force per unit width (if n is a compressive force, it is to be provided with a negative sign)

Various methods are set out in different specifications for the assessment and control of crack behavior

- Limitation of the stresses in the ordinary reinforcement calculated in the cracked state.
- Limitation of the concrete tensile stresses calculated for the homogeneous cross section.
- Determination of the minimum quantity of reinforcement that will ensure crack distribution.
- Checking for cracks by theoretically or empirically obtained crack formulae.

3.2.2 Ordinary reinforcement

For determining the ordinary reinforcement required, a distinction must be made between edge spans, internal spans and column zones.

Edge spans

Required ordinary reinforcement

$p_s \geq 0.15 - 0.50 . p_p$

Lower limit: $p_s \geq 0.05\%$

Where,

p_p = content of prestressing steel

p_s = content of ordinary reinforcement

For the edge span generally the reinforcement in both the direction is provided in slab reinforcement.

Internal spans

For internal spans, adequate crack distribution is in general assured by the post-tensioning and the lateral membrane compressive forces that develop with even quite small deflections. In general, therefore, it is not necessary to check for minimum reinforcement. The quantity of normal reinforcement required for the ultimate limit state must still be provided. It may necessary for design reasons, such as point loads, dynamic loads (spalling of concrete) etc. to provide limited ordinary reinforcement.

Column zone

In the column zone of flat slabs, considerable additional ordinary reinforcement must always be provided. The proposal of DIN 4227 may be taken as a guideline, according to which in the zone $b_{cd} = b_c + 3.d_s$ at least 0.3% reinforcement must be provided and, within the rest of the column strip (b_g=0.4.L) at least 0.15% must be provided. The length of this reinforcement including anchor length should be 0.4.L. Care should be taken to ensure that the bar diameters are not too large.

3.2.3 Deflections

Post-tensioning has a favorable influence upon the deflections of slabs under service loads. Since, however, post-tensioning also makes possible thinner slabs; a portion of this advantage is lost. The load-balancing method is very suitable for calculating deflections. Fig. 3.5 and 3.6 illustrate the procedure diagrammatically. Under permanent loads, which may with advantage be largely compensated by the transverse components from post-tensioning, the deflections can be determined on the assumption of uncracked concrete. Under live loads, however, the stiffness is reduced by the formation of cracks. In slabs with bonded post-tensioning, the maximum loss of stiffness can be estimated from the normal reinforced concrete theory. In slabs with unbonded post-tensioning, the reduction in stiffness, which is very large in a simple beam reinforced by unbonded post-tensioning, is kept within limits in edge spans by the ordinary reinforcement necessary for crack distribution, and in internal spans by the effect of the lateral restraint.

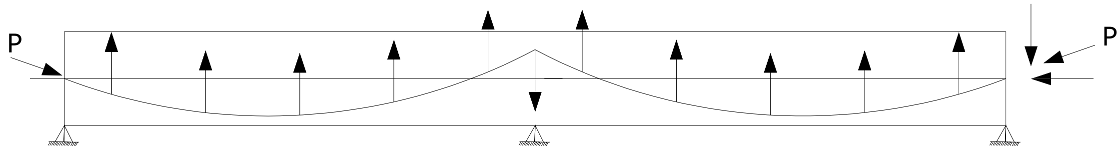


Fig. 3.5 Transverse components and panel forces resulting from post-tensioning

In the existing specifications, the deflections are frequently limited by specifying an upper limit to the slenderness ratio. In structures that are sensitive to deflection, the deflections to be expected can be estimated as follows

$$a = a_{d\text{-}u} + a_{g+qr\text{-}d} + a_{q\text{-}qr} \quad \text{. . . (3.28)}$$

Where,

a = deflection

$a_{d\text{-}u}$ = deflection due to permanent load minus transverse component from prestressing

$a_{g+qr\text{-}d}$ = deflection due to cracking load minus permanent load

$a_{q\text{-}qr}$ = deflection due to live load minus portion of live load in cracking load

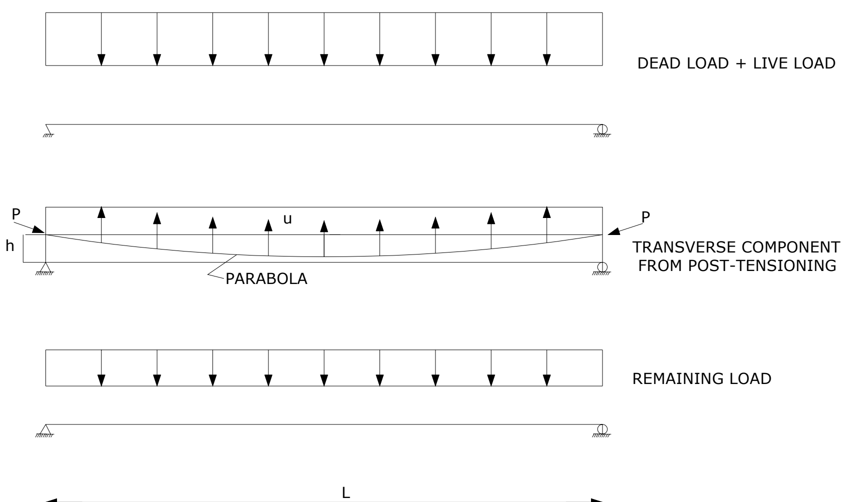


Fig. 3.6 Principle of the load-balancing method

The deflection $a_{d\text{-}u}$, should be calculated for the homogeneous system making an allowance for creep. Up to the cracking load g+qr' which for reasons of prudence should be calculated ignoring the tensile strength of the concrete, the deflection $a_{g+qr\text{-}d}$ should be established for the homogeneous system under short-term loading. Under the remaining live loading, the deflection a_{q-qr} should be determined by using the stiffness of the cracked cross section. For this purpose, the reinforcement content from ordinary reinforcement and prestressing can be assumed as approximately equivalent, i.e. $p=p_s+p_p$ is used.

In many cases, a sufficiently accurate estimate of deflections can be obtained if they are determined under the remaining load (g+q-u) for the homogeneous system and the creep is allowed for by reduction of the elastic modulus of the concrete to

$$E_c' = E_c/(1+\Phi) \qquad \text{---------------} \ (3.29)$$

Where,

Φ = creep coefficient which depends upon the age of concrete.

3.2.4 Post-tensioning force in the tendon

3.2.4.1 Losses due to friction

In the post-tensioned members, the tendons are housed in ducts performed in concrete. The ducts are either straight of follow a curved profile depending upon the design requirements. Consequently, on tensioning the curved tendons, the loss of stress occurs in the post-tensioned members due to friction between the tendons and the surrounding concrete ducts. The magnitude of this loss is of the following types

- Loss of stress due to curvature effect, which depends upon the tendon form or alignment which generally follows a curved profile along the length of the beam.
- Loss of stress due to the wobble effect, which depends upon the local deviations in the alignment of the cable.

The calculation of the frictional losses is carried out by means of the well known formula

$$P_x = P_o.e^{-(\mu a+kx)} \qquad . . . (3.30)$$

Where,

P_x = magnitude of the prestressing force.

P_o = prestressing force at the jacking end.

μ = coefficient of friction between cable and duct.

α = the cumulative angle in radians through which the tangent to the cable profile has turned between any two points under consideration.

K = friction coefficient for wave effect.

The frictional losses can be reduced by several methods, such as

- Over tensioning the tendons by amount equal to the maximum frictional loss.
- Jacking the tendons from both ends of the beam or slab, generally when the tendons are long or when the angles of bending are large.
- By reducing the coefficient of friction. This can be done by using various lubricants, particularly greases, oils, oil and graphite mixtures and paraffin.

3.2.4.2 Long-term losses

The long-term losses in slabs amount to about 10 to 12% of the initial stress in the prestressing steel. Long term losses consist of loss due to creep, shrinkage, relaxation and elastic deformation. These are discussed in the following sections.

Creep losses

Since the slabs are normally post-tensioned for dead load, there is a constant compressive stress distribution over the cross-section. The compressive stress generally is between 1.0 and 2.5 N/mm^2 and thus produces only small losses due to creep. A simplified estimation of the loss of stress can be obtained with the final value for the creep deformation is as follows

$$\Delta\sigma_{pc} = \varepsilon_{cc}.E_p$$

$$\Delta\sigma_{pc} = \Phi_n\,.\,(\sigma_c/E_c).\,E_p \qquad ...\ (3.31)$$

Where,

$\Delta\sigma_{pc}$ = loss of stress in prestressing steel due to creep

ε_{cc} = creep strain of concrete

Φ_n = final creep coefficient

σ_c = concrete stress

E_p = modulus of elasticity of prestressing steel

E_c = reduced modulus of elasticity of concrete.

Although the final creep coefficient φ_n due to early post-tensioning is high, creep losses exceeding 2 to 4% of the initial stress in the prestressing steel do not in general occur.

Shrinkage losses

The shrinkage concrete in prestressed members results in a shortening of the tensioned wires and hence contribute to the loss of stress. The shrinkage of concrete is influenced by the type of cement, aggregates and the method of curing. Use of high strength concrete with low water cement ratios results in a reduction of shrinkage and consequent loss of prestress. The rate of shrinkage is high at the surface of the members. In post-tensioned members a portion of shrinkage will take place by the time of transfer of stress. The stress losses due to shrinkage are given by the final shrinkage factor as

Loss of stress $= \varepsilon_{cs}.E_s$

Where,

ε_{cs} =total residual shrinkage strain having the values of 300×10^{-6} for pretensioning and $[200 \times 10^{-6}/(\log_{10}(t+2))]$ for the post-tensioning

t =age of concrete at transfer in days

E_s = modulus of elasticity of steel

The shrinkage loss is approximately 5% of the initial stress in the prestressing steel.

Relaxation losses

The stress losses due to relaxation of the post-tensioning steel depend upon the type of steel and the initial stress. With the very low relaxation prestressing steels commonly used today, for an initial stress of $0.7f_{pu}$ and ambient temperature of 20°C, the final stress loss due to relaxation is approximately 3%. The Indian standard code recommends a value varying from 0 to 90 N/mm^2 for the stress in wires varying from $0.5f_{pu}$ to $0.8f_{pu}$. The losses of prestress due to relaxation of steel recommended in Indian code (IS 1343- 1980) are shown in table 3.1. Temporary overstressing by 5-10% for a period of 2 minutes is sometimes used to reduce this loss as in the case of drawn wires.

Losses due to elastic deformation of the concrete

The loss of prestress due to elastic deformation of concrete is depends on the modular ratio and the average stress in concrete at the level of steel.

Strain in concrete at the level of steel = f_c / E_c

Stress in steel corresponding to this strain = $(f_c / E_c).E_s$

Loss of stress in steel = $\alpha_e.f_c$

Table 3.1 Relaxation losses for prestressing steel

Initial stress	Relaxation loss N/mm^2
$0.5f_{pu}$	0
$0.6f_{pu}$	35
$0.7f_{pu}$	70
$0.8f_{pu}$	90

Where,

$\alpha_e = E_s / E_c$ = Modular ratio

f_c = prestress in concrete at the level of steel

E_s = modulus of elasticity of steel

E_c = modulus of elasticity of concrete

In slabs or girders the cables are curved with maximum eccentricity at the centre of span. In such cases the loss of stress due to elastic deformation of concrete is estimated by considering the average stress in concrete at the level of steel. In post-tensioned construction there is no loss due to elastic deformation if the cables are stress at a time but if the cables are stressed successively then there is loss due to elastic deformation. For the low centric compression due to prestressing that exists, the average stress loss is only approximately 0.5% and can therefore be neglected.

3.2.5 Vibrations

For dynamically loaded structures, special vibration investigations should be carried out. For a coarse assessment of the dynamic behavior, the inherent frequency of the slab can be calculated on the assumption of homogeneous action.

3.2.6 Fire resistance

In a fire, post-tensioned slabs, like ordinarily reinforced slabs, are at risk principally on account of two phenomena i.e. spalling of the concrete and rise of temperature in the steel. The fire resistance of post-tensioned slabs is virtually equivalent to that of ordinarily reinforced slabs, as demonstrated by corresponding tests. The strength of the prestressing steel does indeed decrease more rapidly than that of ordinary reinforcement as the temperature rises, but on the other hand in post-tensioned slabs better protection is provided for the steel as a consequence of the uncracked cross-section. The behavior of slabs with unbonded posttensioning is hardly any different from that of slabs with bonded post-tensioning, if the appropriate design specifications are followed. The failure of individual unbonded tendons can, however, jeopardize several spans. This circumstance can be allowed for by the provision of intermediate anchorages. From the static design aspect, continuous systems and spans of slabs with lateral constraints exhibit better fire resistance.

3.2.7 Corrosion protection

3.2.7.1 Bonded post-tensioning

The corrosion protection of grouted tendons is assured by the cement suspension injected after stressing. If the grouting operations are carefully carried out no problems arise in regard to protection. The anchorage block-outs are filled with low shrinkage mortar.

3.2.7.2 Unbonded post-tensioning

The corrosion protection of monostrands must satisfy the following conditions

- Freedom from cracking and no embrittlement or liquefaction in the temperature range -20° to +70 °C
- Chemical stability for the life of the structure
- No reaction with the surrounding materials
- Not corrosive or corrosion-promoting
- Watertight

A combination of protective grease coating and plastics sheathing will satisfy these requirements. Experiments have demonstrated that both polyethylene and polypropylene ducts satisfy all the above conditions. As grease, products on a mineral oil base are used; with such greases the specified requirements are also complied with. The corrosion protection in the anchorage zone can be satisfactorily provided by appropriate constructive detailing, in such a manner that the prestressing steel is continuously protected over its entire length. The anchorage block-out is filled with low shrinkage mortar.

4. OPENINGS IN SLAB

4.1 GENERAL

In an advanced concrete design we had a good introduction to the design of two-way slabs. Although all multistory buildings require multiple small slab penetrations for routing of plumbing, fire protection piping, and ductwork between floors and larger openings for stairwells and elevator shafts. For newly constructed slabs, the locations and sizes of the required openings are usually determined in the early stages of design and can be easily accommodated in the majority of instances. However, you may also be asked to modify an existing structure, where the analysis and strengthening (if required) are typically more involved than for similar openings in a new slab.

Here, some guidance on selecting locations and sizes for openings in two-way slabs for both new and existing structures are introduced. By carefully selecting their locations, small openings can often be accommodated without requiring strengthening. You will probably run across situations, however, where the opening requires strengthening or the location of these openings is dictated by concerns other than the strength of the structure. For these cases, we'll also introduce some methods for strengthening existing structures.

4.2 TYPES OF TWO-WAY SLABS

Although there are several different variations of two-way slabs, they can be generally described as one or a combination of three two-way systems: flat plates, flat slabs, and two-way beam-supported slabs. The selection of the most advantageous location for a floor opening depends on the type of two-way slab we are designing or evaluating.

The simplest type of two-way slab to construct is known as a flat plate. These slabs are supported directly by the columns and have a completely flat soffit. For live loads of about 2.5 KPa (50 psf.), column spacing typically ranges from 4.5 to 7.5 m (15 to 25 ft) with minimum slab thicknesses of 150 to 250 mm (6 to 10 in.). For longer spans, drop panels (thickened portions of the slab) are added at the columns. This system is referred to as a flat slab and has an economical span range of 7.5 to 9 m (25 to 30 ft) with minimum slab thicknesses of 200 to 250 mm (8.5 to 10 in.). Two-way beam-supported slabs have beams spanning between columns in both directions that act with the slab to support gravity loads.

4.3 OPENINGS IN NEW SLABS

For the purposes of design, two-way slab systems are divided into column and middle strips in two perpendicular directions. The column strip width on each side of the column centerline is equal to 1/4 of the length of the shorter span in the two perpendicular directions. The middle strip is bounded by two column strips. Section 13.4.1 of ACI 318-05 and IS 456:2000 clause 31.8 permits openings of any size in any new slab system provided you perform an analysis that demonstrates both strength and serviceability requirements are satisfied. As an alternative to detailed analysis for slabs with openings, ACI 318-05 and IS 456:2000 gives the following guidelines for opening size in different locations for flat plates and flat slabs. These guidelines are illustrated in Fig. 4.1 for slabs with $l_2 \geq l_1$

- Opening of any size may be placed within the middle half of the span in each direction, provided the total amount of reinforcement required for the panel without the opening is maintained.
- In the area common to intersecting column strips, not more than one-eighth of width of strip in either span shall be interrupted by the openings. The equivalent of reinforcement interrupted shall be added on all sides of the openings.
- In the area common to one column strip and one middle strip, not more than one-quarter of the reinforcement in either strip shall be interrupted by the openings. The equivalent of reinforcement interrupted shall be added on all sides of the openings.

In addition to flexural requirements, the reduction in slab shear strength must also be considered when the opening is located anywhere within a column strip of a flat slab or within 10 times the slab thickness from a concentrated load or reaction area. The effect of the slab opening is evaluated by reducing the perimeter of the

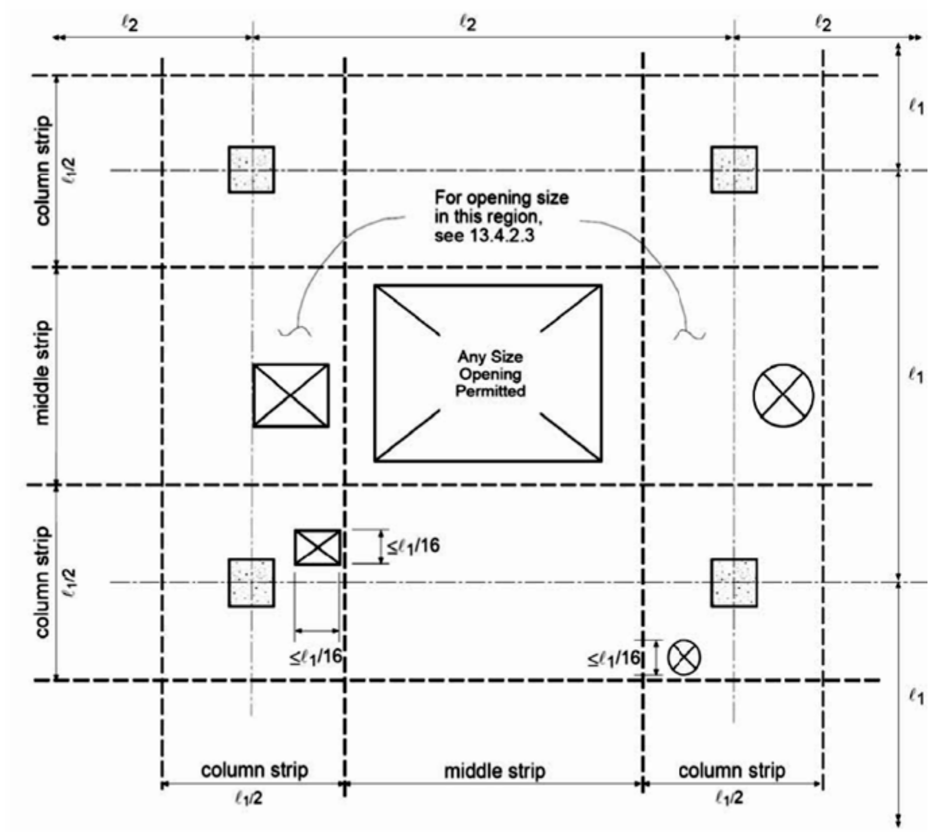


Fig. 4.1 Suggested opening sizes and locations in flat plates with $l_2 \geq l_1$

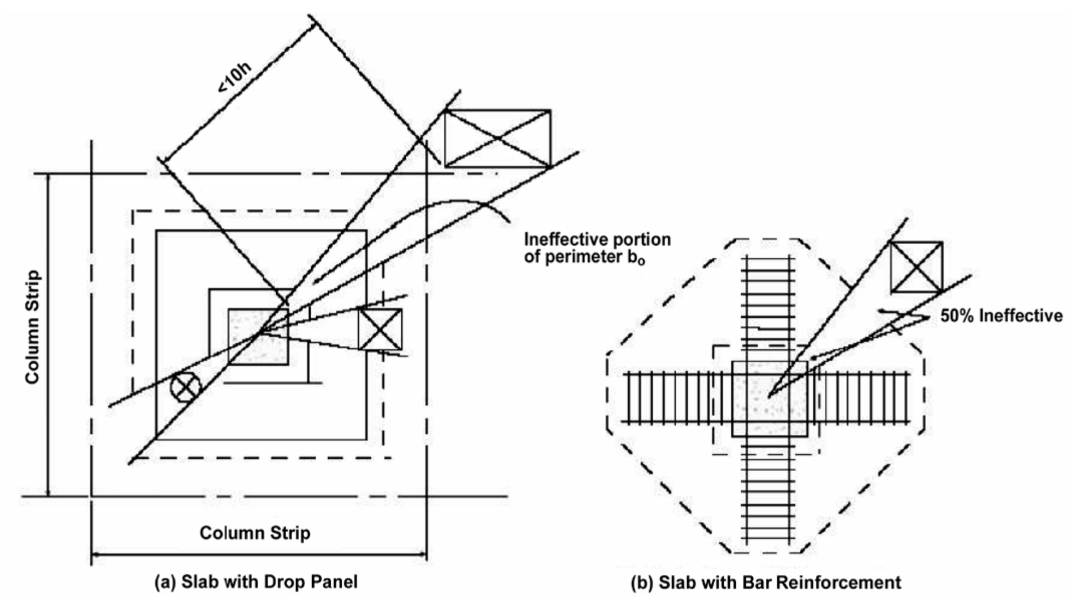


Fig.4. 2 Reduction to perimeter of critical section b_0 for a flat plate or flat slab with openings in column strips or within a distance of 10 times the thickness of the slab from a column: (a) no shearheads and (b) with shear heads

critical section b_o by a length equal to the projection of the opening enclosed by two lines extending from the centroid of the column and tangent to the opening, as shown in Fig. 4.2(a). For slabs with shear heads to assist in transferring slab shear to the column, the effect of the opening is reduced, and b_o is reduced by only half the length enclosed by the tangential lines, as shown in Fig. 4.2(b).

4.4 OPENINGS IN EXISTING SLABS

Small openings in existing slabs are usually core-drilled to the required diameter. Larger openings are cut with a circular saw or a concrete chain saw with plunge cutting capabilities. Because a circular saw makes a longer cut on the top of the slab than on the bottom, small cores drilled at the corners can be used to help avoid over-cutting the opening when a circular saw is used. Cutting openings in existing slabs should be approached with caution and avoided if possible. When cutting an opening in an existing slab, the effect on the structural integrity of the slab must be analyzed. It's advisable to analyze the slab for excess capacity and possible moment redistribution before making the final decision on the sizes and locations of the openings, but the following guidelines can assist in making preliminary decisions with the best chance to avoid having to reinforce the slab.

4.4.1 Openings in existing flat plates and flat slabs

Because the punching shear capacity of the slab around the columns typically governs the thickness of flat plates, any openings at the intersection of column strips (Area 3 in Fig. 4.3) should be avoided as much as possible. This is especially critical near corner and edge columns where the shear in the slab is typically highest. If openings must be made in Area 3, to install a drainage pipe for example, the size of the opening should be no larger than 300 mm (12 in.). Because they reduce the critical section for resisting punching shear, openings cut in this area should be evaluated carefully. One possible exception to this guideline is when column capitals, commonly seen in older structures, are present to reduce shear stresses in the slab.

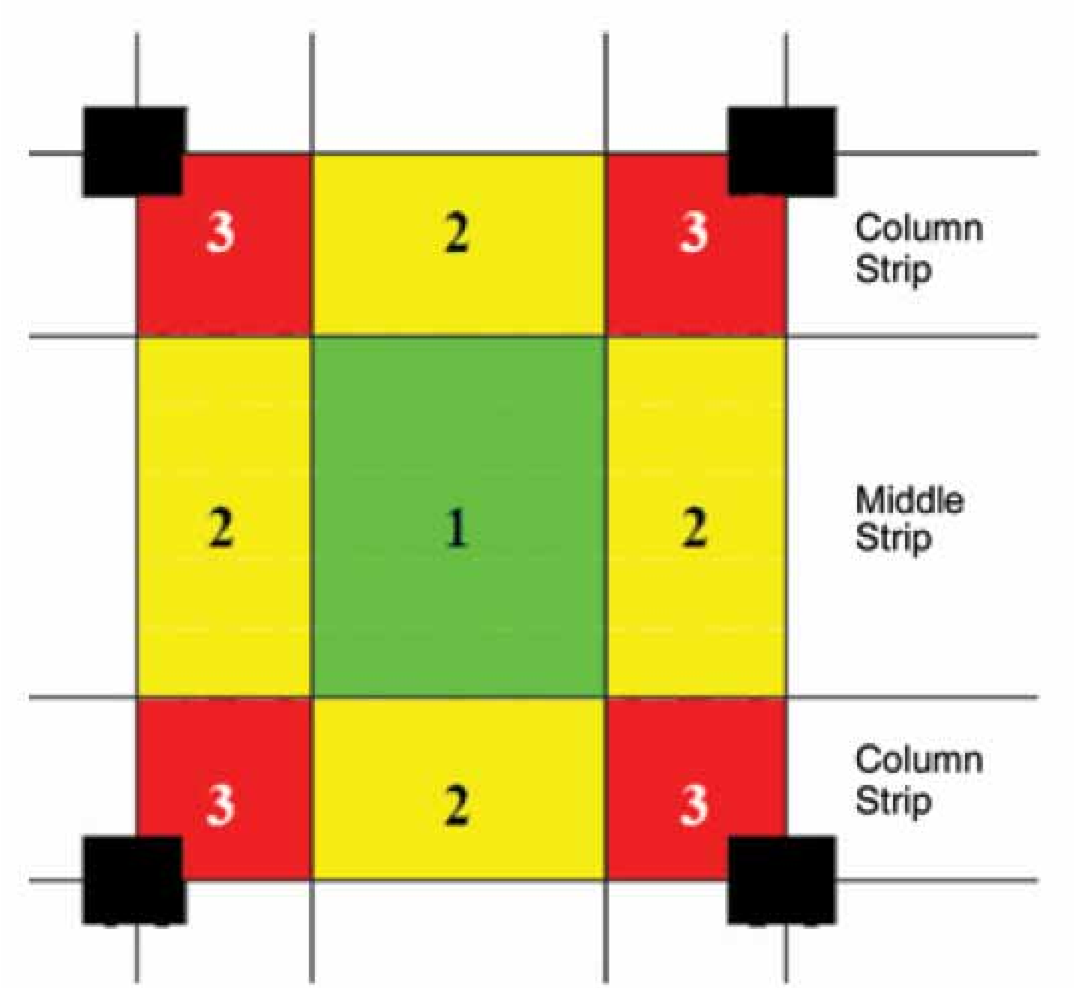


Fig. 4.3 Areas for slab openings

Openings in Area 2, located at the intersection of column and middle strips, are less critical than in Area 3, and small openings having a width less than 15% of the span length can often be made in this area. The most favorable location for openings from a structural point of view is often the intersection of two middle strips (Area 1). This is also often the least favorable location from an architectural point of view, however, because it is the most disruptive to the function of the space. The guidelines for openings in flat slabs generally follow the recommendations for flat plates, but the chances of accommodating larger openings in Area 3 are increased due to the lower shear stresses in the region of the drop panels.

4.4.2 Openings in existing two-way beam-supported slabs

For openings in two-way beam-supported slabs, the situation is reversed because much of the shear is transferred to the column through the beams. The total width of openings in Area 3 (intersection of two column strips) can often be up to 1/4 of the span, as long as the beams are left intact. Openings in Area 2 can be more problematic because they may intersect the portion of the slab used as a T-beam. Although Area 1 is the least desirable location, openings with maximum dimensions up to 1/8 of the span can often be located at the intersection of two middle strips.

When removing an entire panel of slab between beams, it's often an advantage to leave enough of an overhang to allow development of reinforcing bars from adjacent spans. In this case, the beams should be checked for torsion because the balancing moments from the portion of the slab that was removed will no longer be present.

4.5 STRENGTHENING METHODS

Openings in new structures can often be accommodated by the proper detailing of additional reinforcing steel in the slab or beams, beams spanning between columns or other beams, or thickening of portions of the slab around openings. When you determine that an existing structure can't accommodate new openings without strengthening, the situation becomes more complex; however, there are several common strengthening methods you can consider. The selection of the most appropriate method to use will depend on several factors, such as the amount of strengthening required, the location where strengthening is required, and architectural requirements.

One of the most common methods for increasing moment capacity is to add steel plates to the surface of a slab, using either through-bolts or post-installed anchors. The installation is fairly simple, but because plates and through-bolts would interfere with flooring surfaces, plates are normally installed on the bottom of the slab using post-installed anchors. Also, because overlapping of the plates is difficult, this method works best when strengthening is required in only one direction. A similar method is to use fiber-reinforced polymer or steel-reinforced polymer strips to strengthen the slab. The strips can be overlapped at the corners of the opening, making strengthening in two directions simpler, and does not interfere with the floor surface as much as anchored steel plates. Their installation, however, requires more highly skilled labor.

When there are existing concrete beams, steel beams can be installed that span between the concrete beams. Shrink or nonshrink grout should be installed between the top flange of the steel beam and the bottom of the slab to ensure uniform bearing. When shear strengthening is required around columns, a common solution is to install steel or concrete collars around the columns to increase the perimeter of the critical section for punching shear. It's important to remember that exposed reinforcing systems may require fire protection. Systems that incorporate epoxy adhesives must be carefully evaluated, as they can lose strength rapidly at elevated temperatures. For low levels of strengthening, the contribution of the exposed reinforcing system to the strength of the slab can be neglected, and the strength checked using factored loads for fire conditions that are lower than under normal temperatures. For higher levels of strengthening, special coatings may be required to achieve a specific fire rating.

5. DESIGN METHODOLOGY

5.1 GENERAL

The design of post-tensioned slab is done by two methods, load balancing method and the equivalent frame method. The load balancing method introduced by T. Y. Lin is most suitable for the indeterminate structures rather than the determinate structures. In this method the 65 to 80% of dead load is balanced by the tendons so that the flexural member will not be subjected to bending stress under a given load conditions. On the other hand the equivalent frame method is widely use for the design of post-tensioned slabs. Here load balancing method and equivalent frame method are discussed in the following section.

5.2 LOAD-BALANCING METHOD

The concept of load balancing is introduced for prestressed concrete structures, as a third approach after the elastic stress and the ultimate strength method of design and analysis. It is first applied to simple beams and cantilevers and then to continuous beams and rigid frames. This load-balancing method represents the simplest approach to prestressed design and analysis, its advantage over the elastic stress and ultimate strength methods is not significant for statically determinate structures. When dealing with statically indeterminate systems including flat slabs and certain thin shells, load-balancing method offers tremendous advantage both in calculating and visualizing.

According to load-balancing method, prestressing balances a certain portion of the gravity loads so that flexural members, such as slabs, beams, and girders, will not be subjected to bending stresses under a given load condition. Thus a structure carrying transverse loads is subjected only to axial stresses. For example in fig. 5.1, a uniformly distributed gravity load on a simple beam is balanced by the transverse component from the tendon so that the beam is subjected to a net axial force of $F\cos\theta$ and the stresses F in the concrete at any section are given by the simple expression

$$F = F\cos\theta / A_s \qquad \text{. . . (5.1)}$$

Where, A_s is the area of concrete at the section.

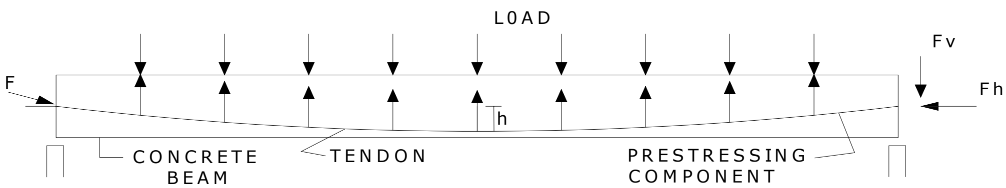


Fig. 5.1: Load balancing design of prestressed concrete.

5.2.1 Life History of Prestressed Concrete Member under Flexure -

To properly compare the load-balancing method with the elastic stress or the ultimate strength method, it will be desirable that one should know about the history of a prestressed concrete member under flexure. Fig. 5.2 is intended to describe the load-deflection relationship of a section of a member.

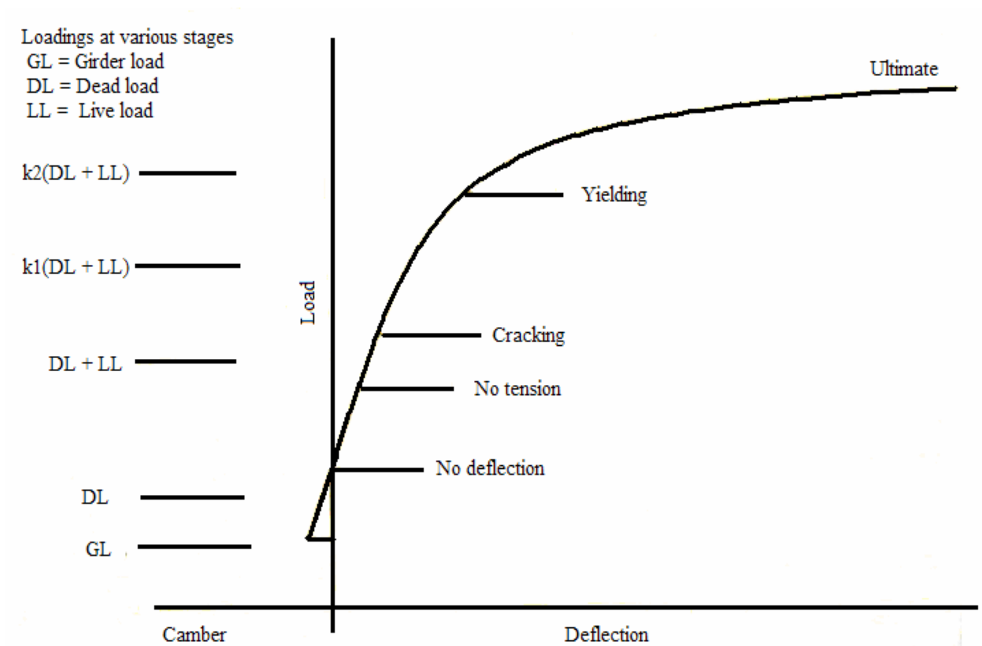


Fig.5.2 Life history of prestressed concrete member under flexure

From the load deflection curve it will be noted that under static loadings, there are several critical points, as follows

1. The point of no deflection which usually indicates a rectangular stress block across all sections of a beam.

2. The point of no tension indicates a triangular stress block with zero stress at either the top or the bottom fiber.
3. The point of cracking which generally occurs when the extreme fiber is stressed to the modulus of rupture.
4. The point of yielding at which the steel is stressed beyond its yield point so that complete recovery will not be obtained.
5. The ultimate load which represents the load carried by the member at failure.

On the left of fig. 5.2 are shown the various loading conditions to which the beam is subjected to are as follows

1. Girder load GL
2. Total dead load DL
3. The working load, made up of dead plus live load DL + LL
4. A safety factor k_1 applied to the working load to obtain the yield point load k_1(DL + LL)
5. Another safety factor K_2 applied to obtain the ultimate load k_2(DL +LL).

The conventional elastic design actually consists of matching the DL + LL with the point of 'no tension' (or some allowable tension) on the beam. Design by ultimate strength consists of matching the k_2(DL + LL) with the ultimate strength of beam. Design by load balancing consists of matching the DL + k_3LL (where k_3 is zero or some value much less than 1) with the point of no deflection. It is clear that depending on the relative values of the three stages of loading as compared to the relative values of the three stage of beam behavior, design based on the three approaches could yield the same proportions or widely varying ones. Table 5.1 shows the different loading applied and the different stages of beam behavior under that loading.

It is also noted that, regardless of which method is used in the design, it is common practice to check for the behavior of the beam at the other stages. For example, if elastic design is used, the ultimate strength and deflections of the beam are usually computed in addition. If the load-deflection of a beam or the moment-curvature relationship of a section is of a definite shape, it is then possible to determine all critical points whenever one point is known. Actually, on account of the difference in the shape of the section, the amount and location of prestressed and nonprestressed steel as well as different stress-strain relationship of both the concrete and steel, this load-deflection or moment-curvature relationship may posses divergent forms. Thus it is often necessary to determine more than one critical point to be sure that the beam will behave properly under various load conditions.

Table 5.1 Loadings compared to relative values of the beam behavior

Applied loadings	**Stage of beam behavior**
$(DL + k_3LL)$	No deflection
$(DL + LL)$	No tension
$K_2(DL + LL)$	Ultimate

Which method is best to follow will depend on the circumstances. Generally it is desirable to choose the one which will control the proportioning of the member. If it is not certain that the other requirements will be met automatically, analysis for these other critical stages will be made and modifications of the design may be effected. Since the balance load point is often indicative of the behavior during the greater portion of the life span of a prestressed structure, it could deserve more consideration than either the working load or the ultimate load.

A further advantage of the load-balancing method is the convenience in the computation of deflections. Since the loading under which there will no deflections as already known, the net deflection produced under any loading (up to the point of cracking) is simply computed by treating the loading differential acting on an elastic beam. If the effective prestress balances the sustained loading, the beam will remain level regardless of the modulus of elasticity or the flexural creep of concrete.

5.2.2 Simple Beams and Cantilevers -

While the load-balancing approach is not usually the best method for designing a simple beam, it can be well introduced with this simple case. Fig.5.3 illustrate how to balance a concentrated load by sharply bending the cgs, at midspan, creating an upward component

$$V = 2F\sin\theta$$

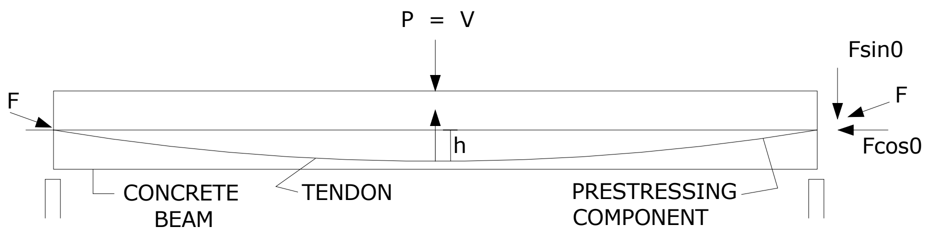


Fig. 5.3 Balancing of a concentrated load.

If this *V* exactly balances a concentrated load P applied also at midspan, the beam is not subjected to any transverse load (neglecting the weight of the beam). At the ends, the vertical component of prestress *Fsinθ* is transmitted directly into the supports, while the horizontal component *Fcosθ* creates a uniform compression along entire beam. Thus the stresses in the beam at any section (except for local stress concentrations) is simply given by

$$F = F\cos\theta / A_s = F / A_s$$

For small values of θ.

Any loading in addition to P will now cause bending in an elastic homogeneous beam (up to the point of cracking) and the additional stresses can be computed simply by

$$F = Mc / I \qquad \text{. . . (5.2)}$$

Where M is the moment produced by load in addition to P.

5.2.3 Two Dimensional Load Balancing

Two dimensional load balancing differs from linear load balancing for beams and columns. In two way load balancing the transverse component of the tendons in one direction either adds to or subtracts from that component in the other direction. Thus, the prestress design in the two directions is closely related one to the other. However, the basic principle of load balancing still holds, and the main aim of design is to balance a given loading so that the entire structure (weather a slab or grid) will posses uniform stress distribution in each direction and will have no deflection nor camber under this loading. Any deviation from this balance loading will then be analyzed as loads acting on an elastic slab, without further considering the transverse component of prestress.

As a simple example of two-dimensional load balancing let us consider a two-way slab simply supported (as shown in fig. 5.3). The cables in both directions exert an upward force on the slab, and if the sum of the upward components balances the downward load w, then we have a balanced design. Thus if F_1 and F_2 are the prestressing forces in the two directions per meter width of slab, we have

$$(8\ F_1h_1 / L_1^2) + (8\ F_2h_2 / L_2^2) = w \qquad \text{. . . (5.3)}$$

Many combinations of F_1 and F_2 will satisfy the above equation. While the most economical design is to carry the load only in the short direction (or to carry 0.5w in each direction in case of a square panel), practical consideration might suggest different distributions. For example, if both directions are properly prestressed, it is possible to obtain a crack free slab.

Under the action of F_1 and F_2 and the load w, the entire slab has uniform stress distribution in each direction equal to F_1/t and F_2/t respectively, where t is the thickness of slab. Any change in loading from the balanced amount of w can be analyzed by the elastic theory for slabs.

If uniform stress distribution and zero deflection are not essential for a structure, balance load design may not be the most economical approach. For example, a cable placed along the middle strips will evidently be more effective than one along the wall. If more cables are located along the middle strips than along the walls, a stronger design might be obtained than the balanced load design suggested. If this is done, the slab will not be level under the uniform load of its own weight. However, it will have no deflection under a varying load intensity which is everywhere equal and opposite to the upward component of the prestress.

Load-balancing method transforms a prestressed structure into a nonprestressed one subjected only to the unbalanced portion of the loading. Hence it reduces the design and analysis of prestressed structures to a point simpler than that of conventional structures. Its application to statically indeterminate beams and frames not only saves the time but also presents a realistic approach which helps the engineer to visualize the effect of prestressing. Indeed, the horizon of prestressing will be greatly extended when this method is applied to grids, slabs, thin shells and folded plates. Furthermore, the method is easily adaptable to a combination of prestressed and reinforced concrete where varying amount of the live load may be carried by nonprestressed reinforcement.

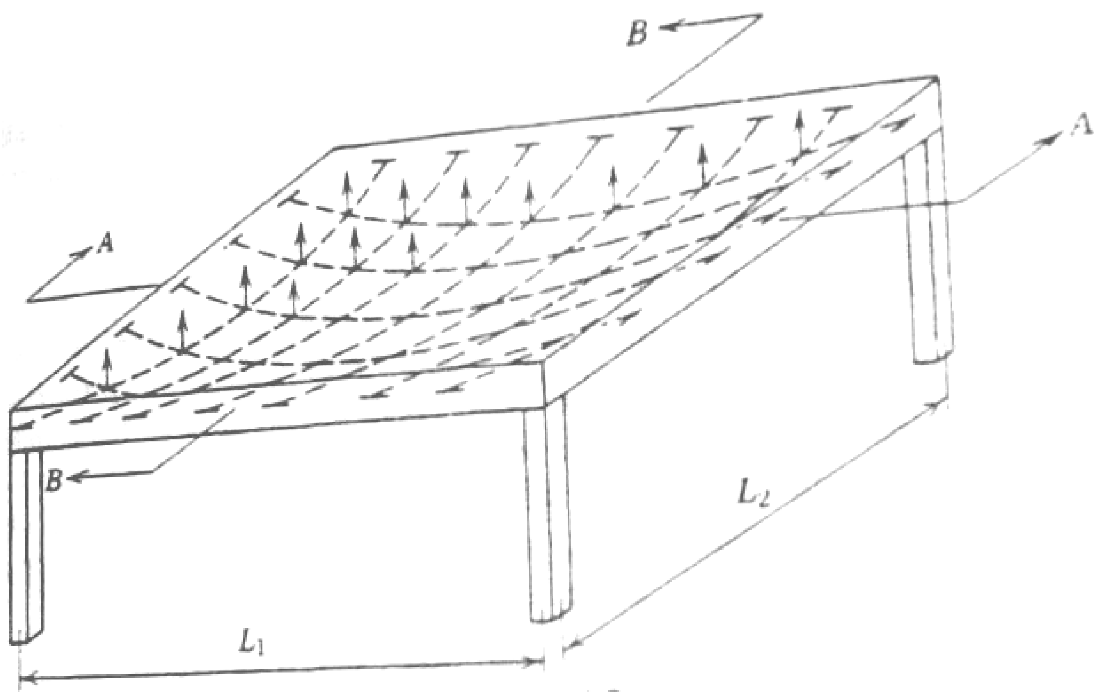


Fig. 5.4 Isometric view of slab and support.

5.3. SECONDARY MOMENTS

In continuous post-tensioned structures, secondary prestressing moments are developed due to restrains to the deformations resulting from the primary prestressing moments. To illustrate this let us consider a two span continuous beam as shown in fig.5.4.

A two-span continuous beam which has a post-tensioning force P acting at a constant eccentricity, e. the moment created, Pe, is considered the primary moment as in fig. 5.4b. If gravity loads are temporarily ignored, the moment due to post-tensioning will cause a theoretical upward deflection as shown in fig. 5.4a. If the beam be vertically restrained at centre support, the change in reactions resulting from the primary prestressing moment will be as shown in fig.5.4c. Because the upward deflection at the interior support is prevented, the secondary

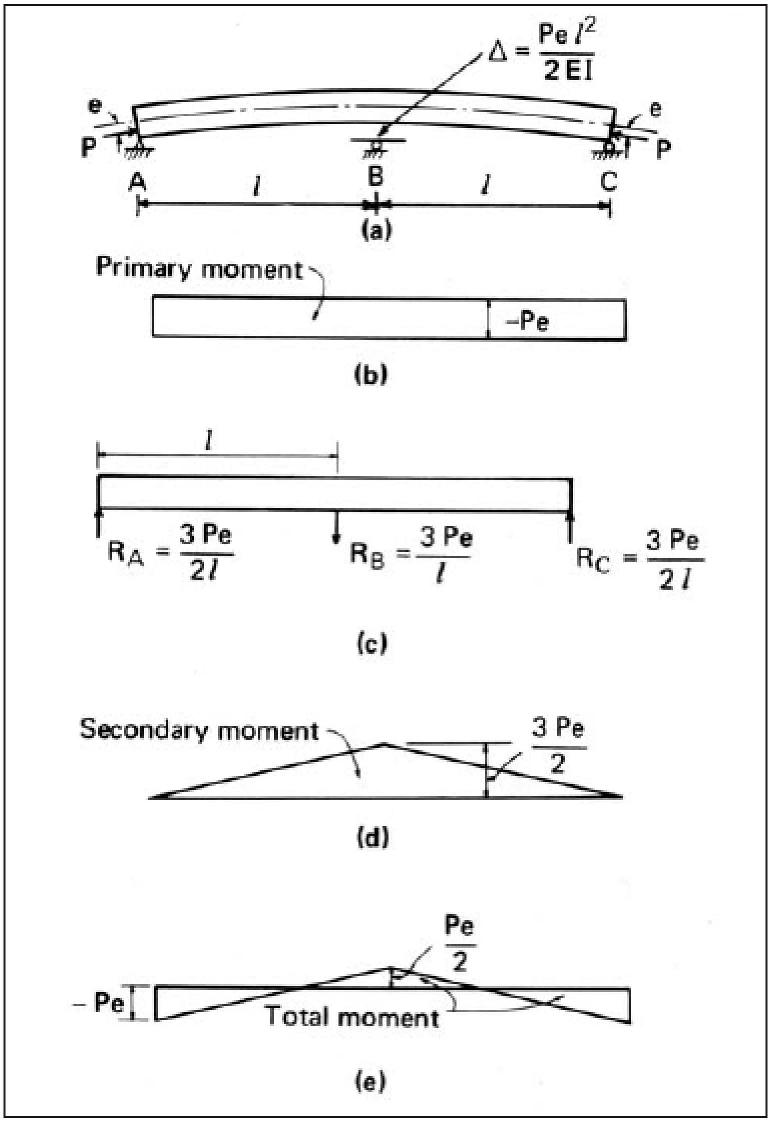


Fig. 5.5 Primary and secondary moments

prestressing moments (fig.5.4d) are induced. For continuous beams the secondary prestressing moment can be expressed as function of the reactions and they always vary linearly between supports. The moment at any section may then be expressed as the superposition of the primary and secondary prestressing moments, fig.5.4e. The term secondary prestressing moment is used because the moment is induced by the primary prestressing moment, and not because the secondary prestressing moment is not negligible or necessarily smaller than the primary moment.

In practical applications, the total post-tensioning moments are obtained directly. If necessary, the secondary prestressing moments can be found by subtracting the primary prestressing moment, Pe, from the total moment. The total moment effect of the post-tensioning tendons on a member may be derived from the loads exerted on the member by the tendons. Simplified design procedures based on load balancing or equivalent load concepts have been developed which eliminate the need to directly consider either primary or secondary prestressing moments due to post-tensioning.

5.4. EQUIVALENT FRAME METHOD OF ANALYSIS

The equivalent frame method of analysis is known as the beam method. This method of analysis utilizes the conventional elastic analysis assumption and models the slab or slab and columns, as a beam or as a frame, respectively. This is the most widely used and applied method of analysis for the post-tensioned flat plates.

The effect of vertical of lateral services and design loading on post-tensioned flat plates, bonded or unbonded, may be analyzed as for rigid frames in accordance with the provisions of the code (IS, ACI etc.). When the columns are relatively slender or not rigidly connected to the slab, their stiffness may be neglected and continuous beam analysis applied. The moment induced by prestressing may also be determined by a similar analysis of a rigid frame or continuous beam, using equivalent load or load balancing concept. However it should be kept in mind that the distribution of moments due to loads may differ considerably from the distribution of moments due to prestressing. Service loads produce very pronounced moments peaks at columns, whereas the moment curve produced by post-tensioning has a more gentle undulating variation of the same form as the tendon profile.

The effects of reversed tendon curvature at supports are generally neglected in applying the load balancing method to design of flat plates since the reverse curvature has only a minor influence on the elastic moments (in the order of 5 to 10 percent), and dose not affect the ultimate moment capacity. It is necessary to consider reverse tendon curvature to adequately evaluate the shear carried by the tendons inside the critical section.

5.5 DESIGN PROCEDURE FOR POST-TENSIONED SLABS

As stated earlier the design of post-tensioned slab can be done by two methods, the design procedure for both the methods are described here along with the process in which design progresses. The steps involved in the design of the post-tensioned slabs are as follows.

A) Design steps for the post-tensioned slab by equivalent frame method

1. Select the slab thickness (according to IS code).
2. Load calculation – calculate the dead load, live load, floor finish and the superimposed dead load. Also calculate the service load and ultimate load.
3. Select the strand diameter, area of strand, ultimate strength of strand and average effective stress in strand.
4. Select the system for the strands i.e. bonded or unbonded.
5. Choose the cable/tendon profile.
6. Calculate the losses due to stressing of tendons.
7. Find out the effective force per tendons.
8. Calculate the section properties i.e. area, section modulus and moment of inertia.
9. Calculate the load balanced by the tendons and the net load causing bending for the span.
10. Determine the section properties such as area, section modulus and the moment of inertia.
11. Calculate equivalent frame properties i.e. determine column stiffness, equivalent column stiffness, slab stiffness and distribution factor at each interior and exterior joint.

12. Determine the stresses at faces of the support and at the midspan.
13. Check for the ultimate flexural capacity.
14. Determine the flexural strength.
15. Calculate the shear, locate the critical section for shear and determine shear at critical section.
16. Check the shear at exterior, interior and edge column.
17. Determine the deflection of span considered and check it with allowable deflection.

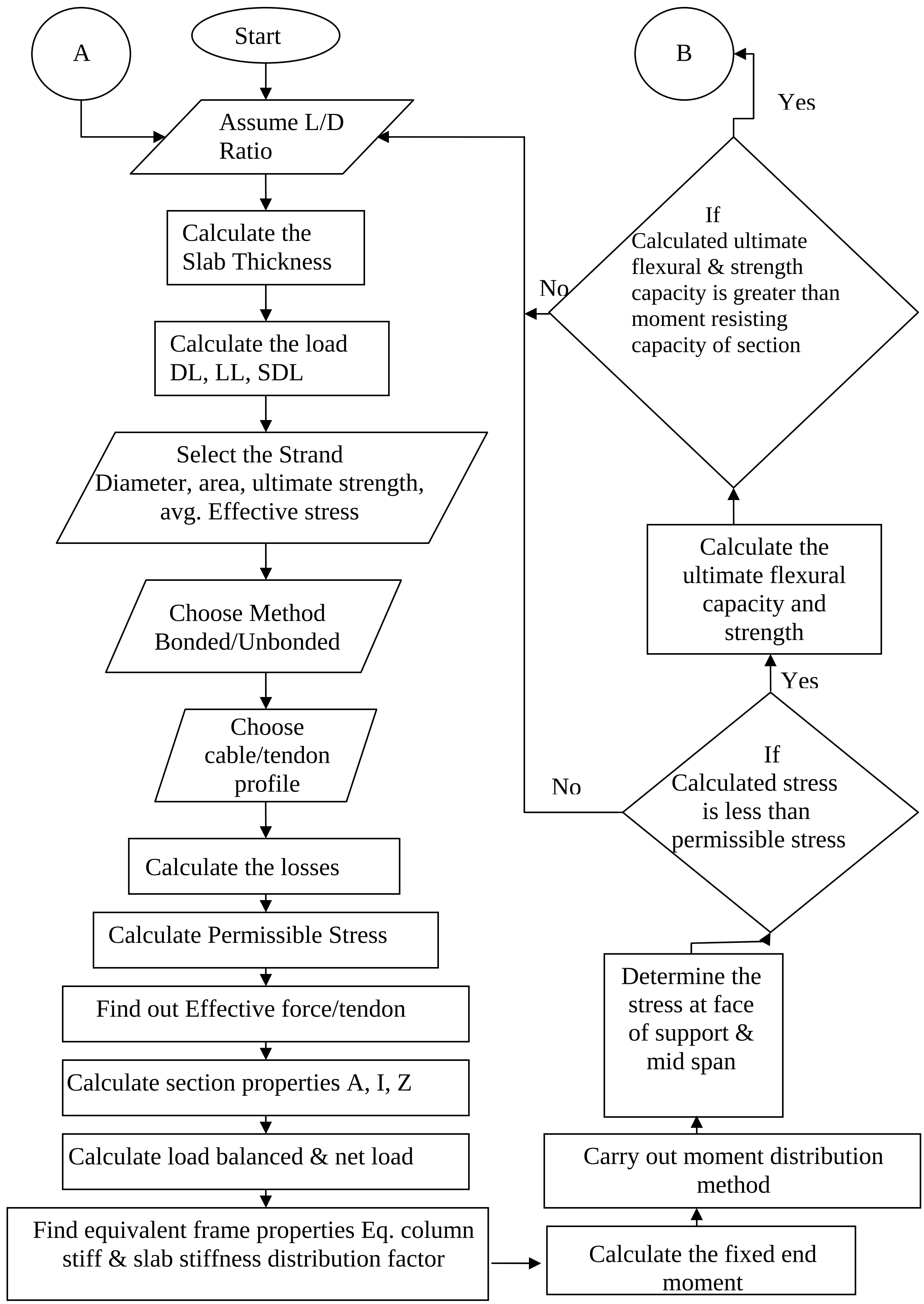
B) Design steps for the post-tensioned slab by load balancing method

1. Select the preliminary thickness of slab. (According to IS Code).
2. Determine the dead load, live load, floor finish and Superimposed dead load.
3. Select the strand diameter, area of strand, ultimate strength of strand and average effective stress in strand.
4. Select the system for the strands i.e. bonded or unbonded.
5. Choose the cable/tendon profile.
6. Calculate the losses due to stressing of tendons.
7. Determine the section properties such as area, section modulus and the moment of inertia.
8. Calculate allowable stresses at the time of jacking and at the service load.
9. Select target load to be balance by tendons (only DL).
10. Calculate the actual load balance for each span.
11. Determine the moment for each span for dead load, live load and balance load separately.
12. Check the slab stresses immediately after jacking (dead load + BL) and service load (dead load + Live load + BL). For the exterior and interior span. (Both at midspan and support).
13. Check for the ultimate moment.
14. Calculate minimum bonded reinforcement.
15. Check the shear at exterior, interior and edge column.

16. Determine the deflection of span considered and check it with allowable deflection.

C) Execution of post-tensioned building

When the building is designed as the post-tensioned building then we must go through the execution procedure. On the field the architectural drawing is given to the designer. The first work to be done is the study of detail plan and to decide the post-tensioned areas and non post-tensioned areas. Then select the thickness of slab, size and thickness of drop cap/panel. Make the grid line and find out the most critical grid lines and do the analysis for those grids. After the analysis the design of the post-tensioned flat slab is to be done. In the office there are two divisions one is design and second is drawing. Each one i.e. designer and drafter are having their separate responsibilities. The detail procedure in which the post-tensioned building design progresses in any office is as per the flow chart given in fig. 5.7.


A
Start
B
Yes
Assume L/D Ratio
If Calculated ultimate flexural & strength capacity is greater than moment resisting capacity of section
No
Calculate the Slab Thickness
Calculate the load DL, LL, SDL
Select the Strand Diameter, area, ultimate strength, avg. Effective stress
Calculate the ultimate flexural capacity and strength
Choose Method Bonded/Unbonded
Yes
Choose cable/tendon profile
No
If Calculated stress is less than permissible stress
Calculate the losses
Calculate Permissible Stress
Determine the stress at face of support & mid span
Find out Effective force/tendon
Calculate section properties A, I, Z
Calculate load balanced & net load
Carry out moment distribution method
Find equivalent frame properties Eq. column stiff & slab stiffness distribution factor
Calculate the fixed end moment

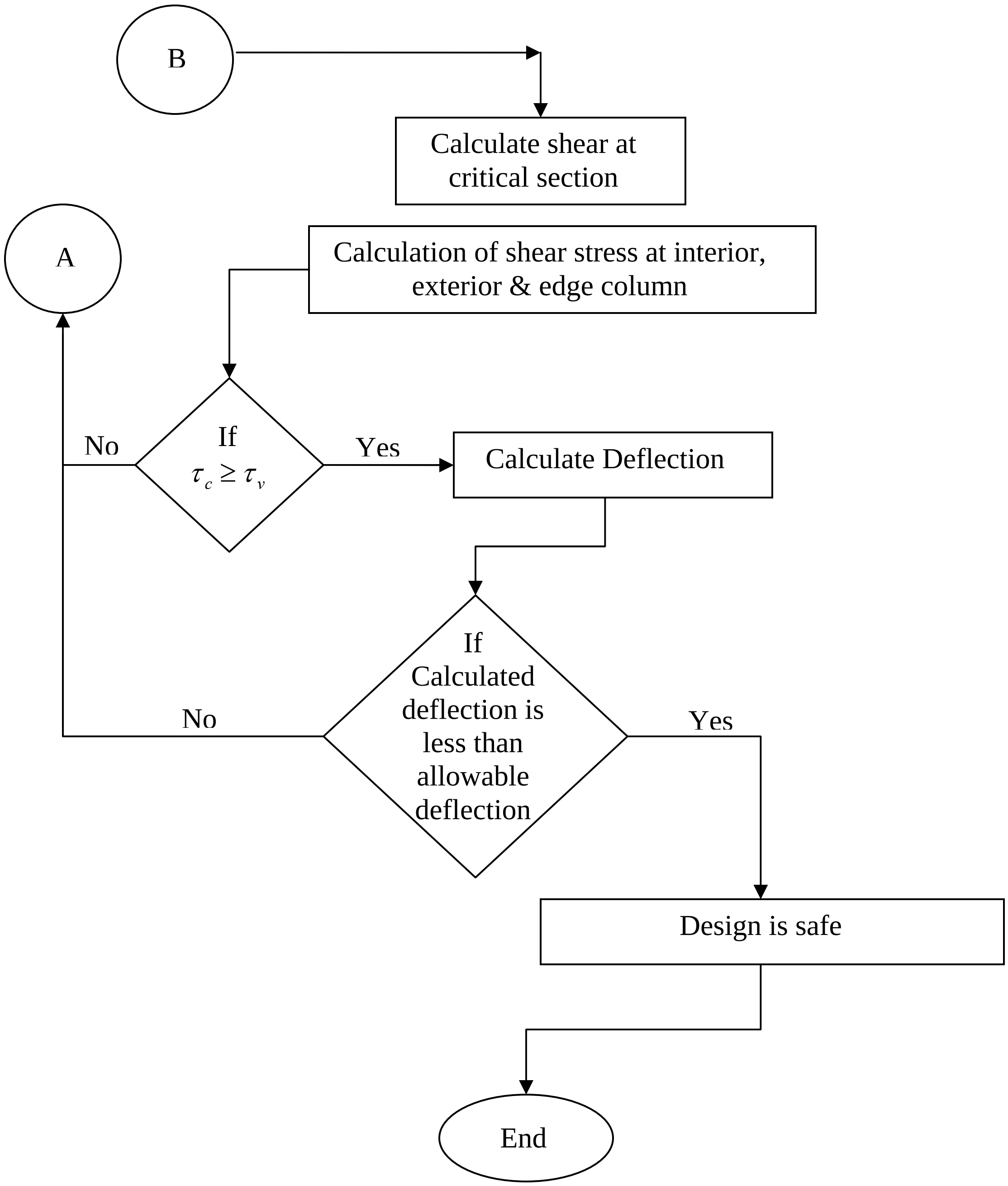
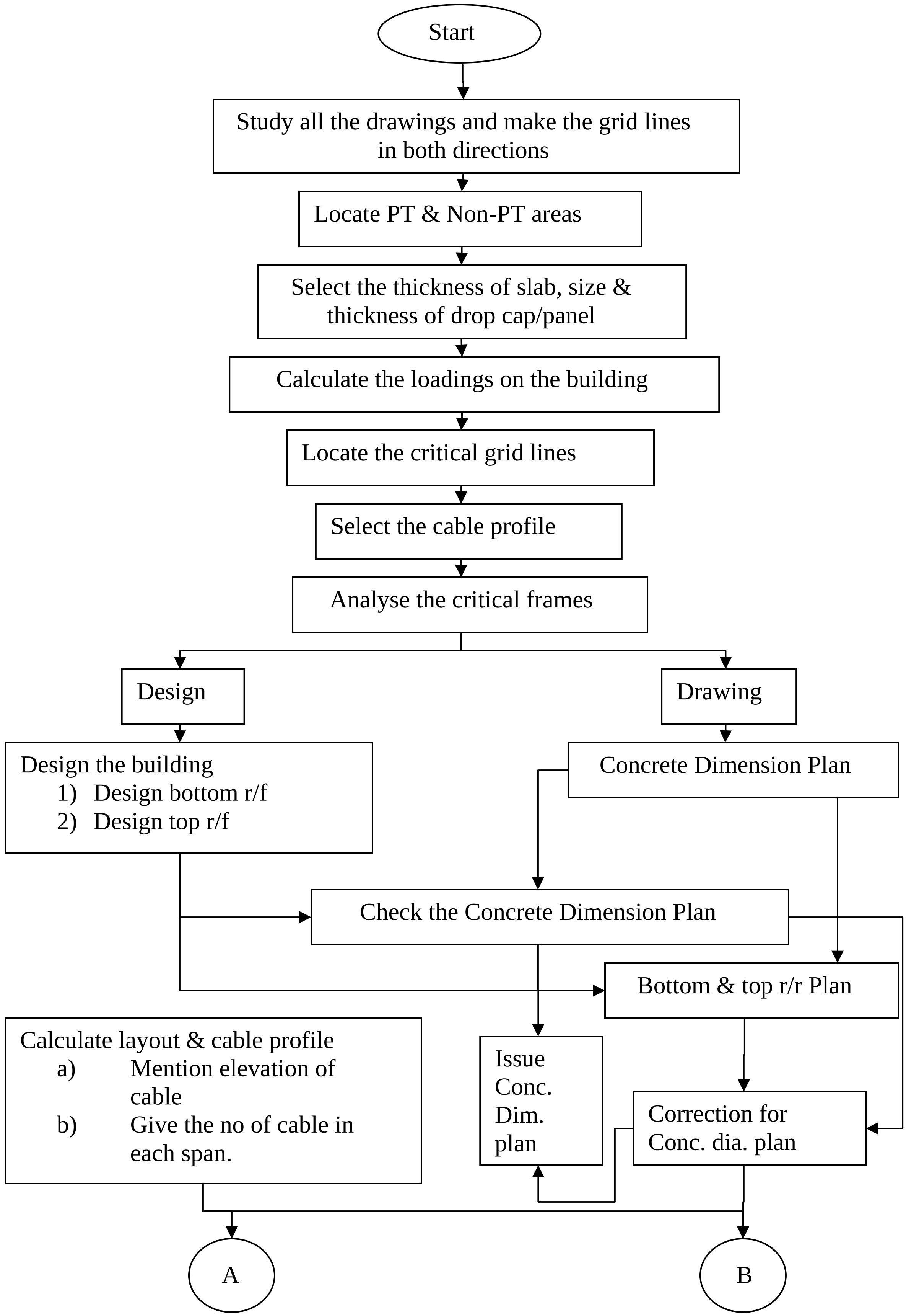


Fig. 5.6 Flow chart for the design of post-tensioned slab


Start
Study all the drawings and make the grid lines in both directions
Locate PT & Non-PT areas
Select the thickness of slab, size & thickness of drop cap/panel
Calculate the loadings on the building
Locate the critical grid lines
Select the cable profile
Analyse the critical frames
Design
Drawing
Design the building
1) Design bottom r/f
2) Design top r/f
Concrete Dimension Plan
Check the Concrete Dimension Plan
Bottom & top r/r Plan
Calculate layout & cable profile
a) Mention elevation of cable
b) Give the no of cable in each span.
Issue Conc. Dim. plan
Correction for Conc. dia. plan
A
B

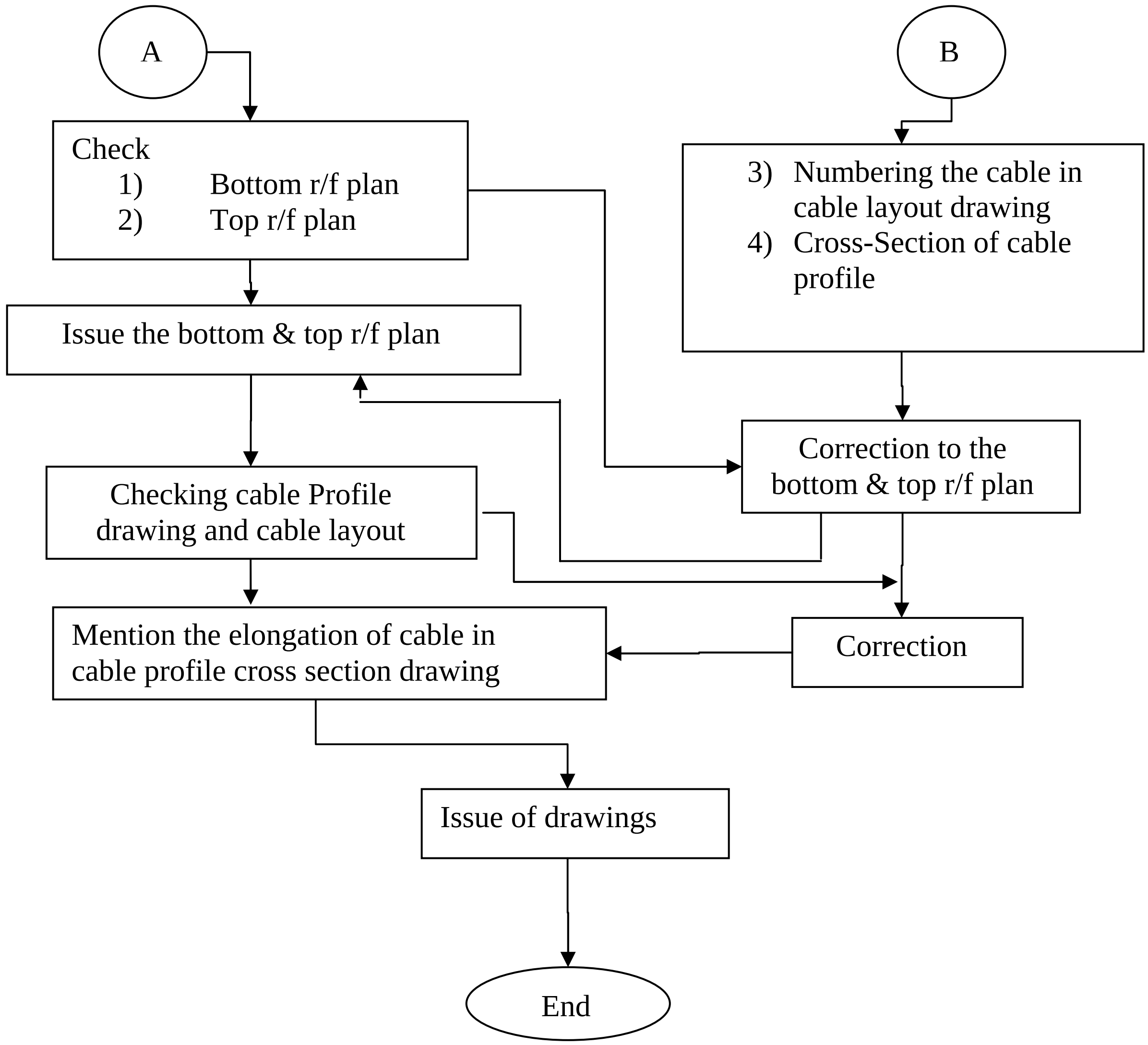


Fig. 5.7 Flow chart for the execution process for post-tensioned buildings

6.1 INTRODUCTION

When a designer is designing a post-tensioned flat slab, he has to consider different parameter that governs the design of the slab. This each parameter has its own influence on the design. By keeping this in mind the parametric study of the post-tensioned flat slab is done here by varying the span of the slab at an interval of 0.5m. The different parameters such as depth of slab, grade of concrete, size of column, loss due to stressing, number of tendons, post-tensioned force required for each tendon, shear force, deflection (long term and short term), elongation of the tendon, normal reinforcement and the reinforcement at support are considered while the study of slab. The study also includes the slab span with and without drop panel. The objective of this study is to observe the different parameters as mentioned above under the varying span by keeping the loading condition same for each case. The detail design of each case is done with the shear and deflection check. In the next section the analysis and design of the post-tensioned flat slab is done.

6.2 ANALYSIS AND DESIGN

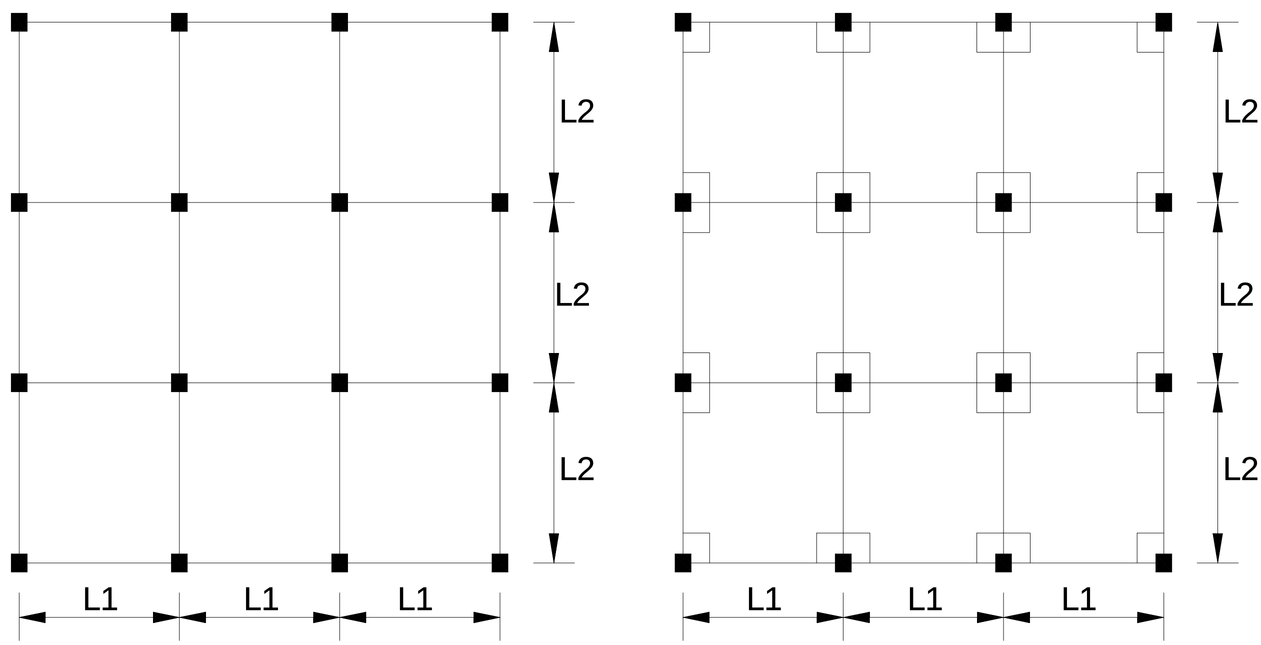


Fig 6.1 Plan of the flat slab without drop and with drop

The plan shown in fig.6.1 is considered for the study of the various parameters of the post-tensioned flat slab. Here square panels are considered. The length L_1 and L_2 are varies from 7m to 12m at an interval of 0.5m each. The live load for the slab is 2KN/m^2 and the superimposed dead load as 1KN/m^2. The dead load includes only the self weight of the slab. The loading on the slab remains same for all the spans of the flat slab. The analysis and design of the post-tensioned flat slab is done by two methods and by using ADAPT software. For the design of post-tensioned flat slab the load balancing method and equivalent frame method are used. The detail design of all the cases is done by both methods and the design procedure for both the methods is as per the appendix A. The output of design by the ADAPT software is also given in the appendix A. The analysis and design of all the cases is according the procedure explained in the appendix A and the results of the design of slab are given in the table 6.1 and table 6.2. Here first design is done by considering the span of slab without drop and then again the design is carried out for the same span by considering the span with drop panel. Table 6.1 and table 6.2 gives the design details of all the cases without drop panel and with drop panel respectively.

The bending moment diagram, shear force diagram, deflection of the span and the stress diagram for each span variation without drop and with drop are given here (fig.6.4 to fig.6.91). Fig.6.2 and 6.3 explains the layout of the tendon across the span without drop and with drop.

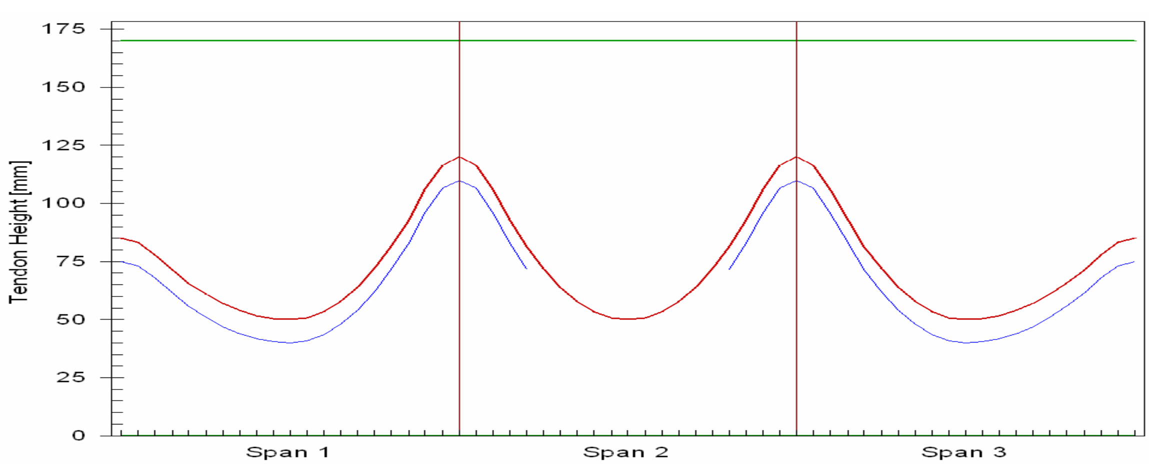


Fig. 6.2 Tendon profile for the slab without drop

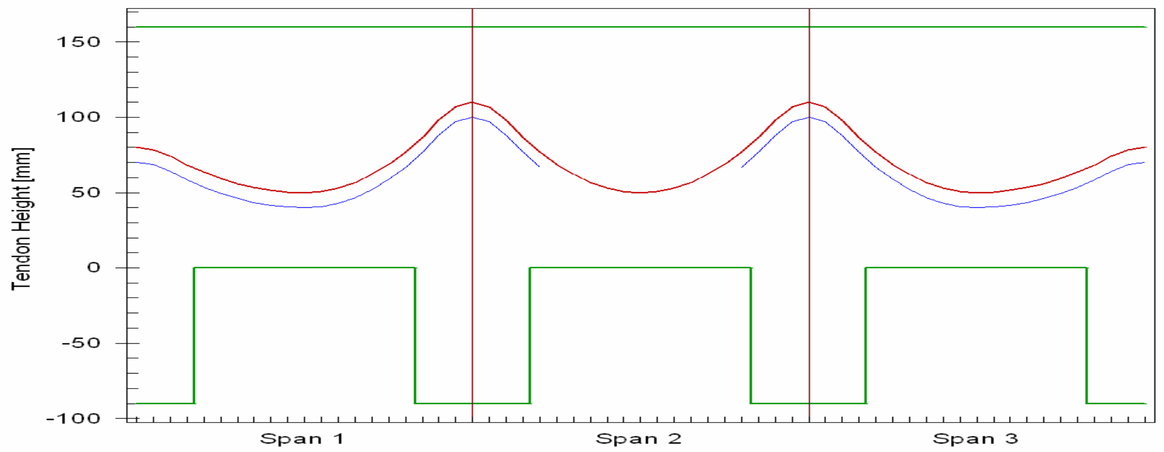


Fig. 6.3 Tendon profile for the slab with drop

Table 6.1 Design results for different parameters of various spans without drop panel

Parameters		Span (m)										
		7	7.5	8	8.5	9	9.5	10	10.5	11	11.5	12
Thickness (mm)		160	170	180	190	200	220	230	240	250	260	270
Grade of conc. (N/mm^2)		35	35	35	40	40	40	40	45	45	45	45
Size of column (mm)		450x450	450x450	450x450	500x500	500x500	500x500	600x600	600x600	600x600	600x600	650x650
Max. moment(KNm)	Support	188	245	312	369	474	581	698	818	1004	1253	1373
	Midspan	131	170	217	256	329	404	484	567	697	894	951
Max. SF at support (KN)	Exterior	224	264	264	350	412	473	559	539	708	773	885
	Interior	534	635	635	841	1004	1154	1346	1310	1721	1950	2155
No. of Tendons	Long	10	13	15	19	20	23	23	27	31	37	41
	Short	2	2	2	3	3	4	4	5	5	4	6
Force per tendon (KN)	Long	125	125.14	125.28	125.39	125.87	125.96	126.41	125.46	126.41	125.59	126.34
	Short	121	121.72	122.24	122.79	123.67	123.93	124.71	124.04	125.25	124.51	125.49
Loss of stress (N/mm^2)		70.49	72.53	75.32	77.15	73.72	74.64	70.83	81.53	72.75	81.61	74.23
Rebar at support	Exterior	6-16mmΦ	8-16mmΦ	8-16mmΦ	8-16mmΦ	8-20mmΦ	10-16mmΦ	10-16mmΦ	12-16mmΦ	12-16mmΦ	14-16mmΦ	14-16mmΦ
	Interior	14-16mmΦ	16-16mmΦ	18-16mmΦ	20-16mmΦ	16-20mmΦ	18-20mmΦ	20-20mmΦ	20-20mmΦ	16-25mmΦ	18-25mmΦ	20-25mmΦ
Normal reinforcement		8mmΦ @ 200 mm c/c	8mmΦ @ 190 mm c/c	8mmΦ @ 180 mm c/c	8mmΦ @ 175 mm c/c	8mmΦ @ 170mm c/c	8mmΦ @ 160mm c/c	8mmΦ @ 150 mm c/c	8mmΦ @ 140mm c/c	8mmΦ @ 130mm c/c	8mmΦ @ 120mm c/c	8mmΦ @ 100 mm c/c
Deflection (mm)		16.3	18.2	20.6	23.5	23.9	26.4	27.69	28.6	31.4	32.34	33.5
Elongation of tendon (mm)	Long	141	151	161	171	180	190	199	209	218	227	237
	Short	45	49	52	56	59	63	66	70	73	77	81

Table 6.2 Design results for different parameters of various spans with drop panel

Parameters		Span (m)										
		7	7.5	8	8.5	9	9.5	10	10.5	11	11.5	12
Thickness (mm)		160	170	180	190	200	220	230	240	250	260	270
Drop size (mm)		2400x2400	2500x2500	2700x2700	2850x2850	3000x3000	3200x3200	3360x3360	3500x3500	3700x3700	3900x3900	4000x4000
Drop thickness (mm)		230	240	260	270	280	290	300	310	320	330	350
Grade of conc. (N/mm^2)		35	35	35	40	40	40	40	45	45	45	45
Size of column (mm)		450x450	450x450	450x450	500x500	500x500	500x500	600x600	600x600	600x600	600x600	650x650
Max. moment(KNm)	Support	212	270	357	414	534	654	765	892	1092	1316	1500
	Midspan	107	142	183	219	282	350	430	508	627	786	853
Max. SF at support (KN)	Exterior	224	264	316	358	424	484	572	551	722	827	905
	Interior	537	635	769	861	1031	1185	1374	1334	1757	2003	2200
No. of Tendons	Long	8	9	10	11	12	15	16	20	24	27	31
	Short	2	2	3	4	4	3	4	5	5	5	6
Force per tendon (KN)	Long	125.40	125.74	125.88	126.28	126.63	126.78	126.98	126.78	126.88	126.2	126.86
	Short	121.34	122.32	122.94	123.69	124.44	124.75	125.28	125.37	125.73	125.12	126.01
Loss of stress (N/mm^2)		66.90	66.50	68.30	68.07	65.99	66.43	65.06	68.15	67.94	75.38	69
Rebar at support	Exterior	4-16mmΦ	5-16mmΦ	6-16mmΦ	6-16mmΦ	7-16mmΦ	8-16mmΦ	10-16mmΦ	10-16mmΦ	8-20mmΦ	8-20mmΦ	12-16mmΦ
	Interior	8-16mmΦ	10-16mmΦ	12-16mmΦ	12-20mmΦ	12-20mmΦ	16-20mmΦ	18-20mmΦ	18-20mmΦ	22-20mmΦ	16-25mmΦ	18-25mmΦ
Normal reinforcement		8mmΦ @ 240 mm c/c	8mmΦ @ 230 mm c/c	8mmΦ @ 210 mm c/c	8mmΦ @ 190 mm c/c	8mmΦ @ 180 mm c/c	8mmΦ @ 160mm c/c	8mmΦ @ 140 mm c/c	8mmΦ @ 130mm c/c	8mmΦ @ 120mm c/c	8mmΦ @ 110 mm c/c	8mmΦ @ 100 mm c/c
Deflection (mm)		11.4	14.2	16.8	20.9	21.3	25.3	26.2	26.8	29.5	31.29	33.15
Elongation of tendon (mm)	Long	141	151	161	171	180	190	199	209	218	227	237
	Short	45	49	52	56	59	63	66	70	73	77	81

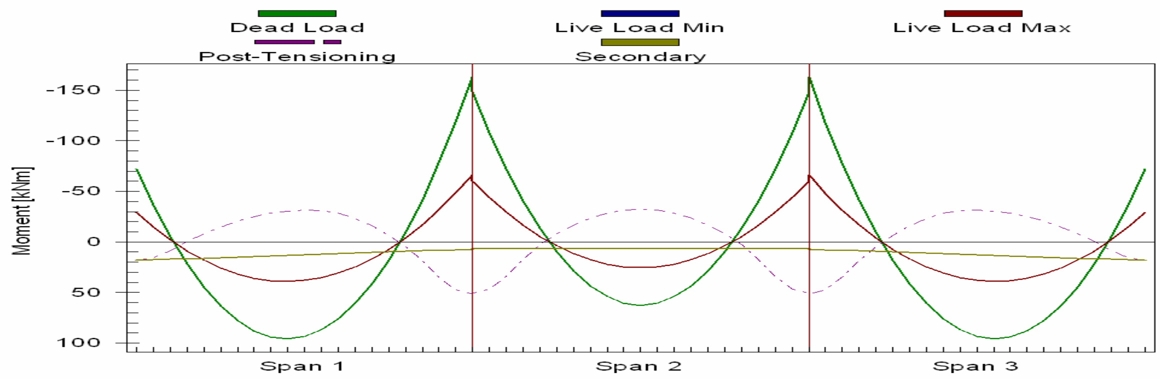


Fig. 6.4 Bending moment diagram for 7m span without drop

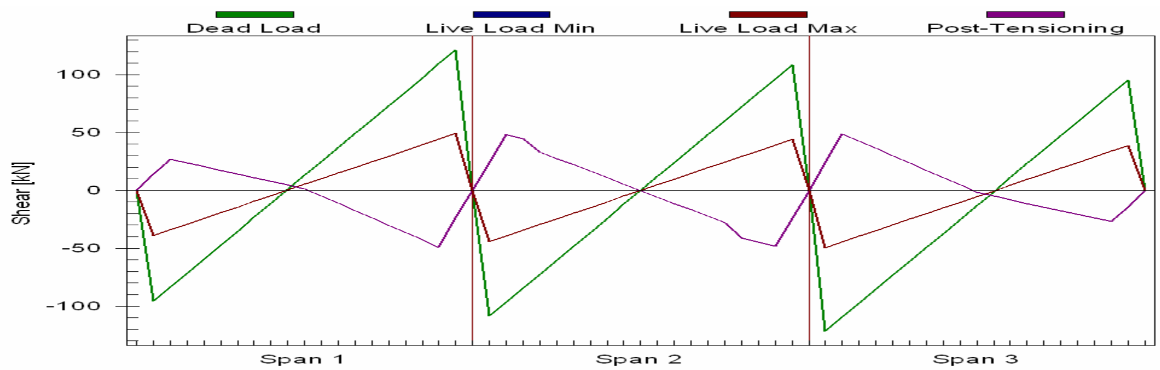


Fig. 6.5 Shear force diagram for 7m span without drop

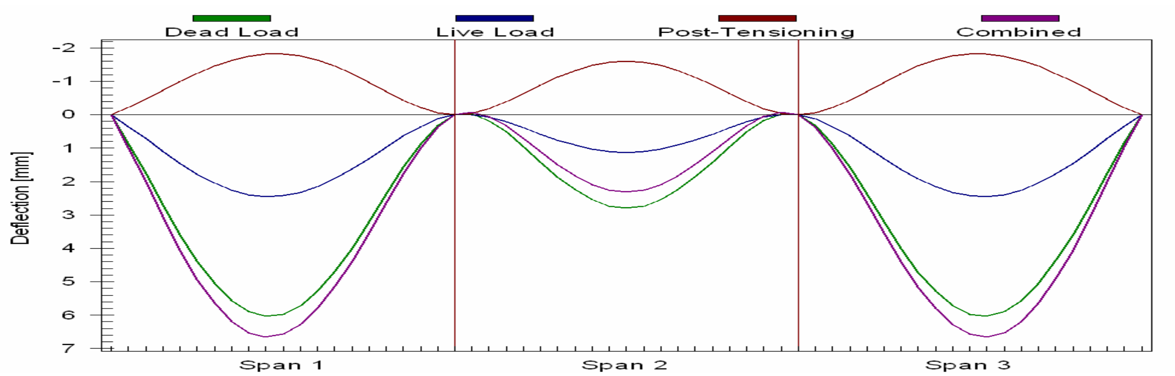


Fig. 6.6 Deflection for 7m span without drop

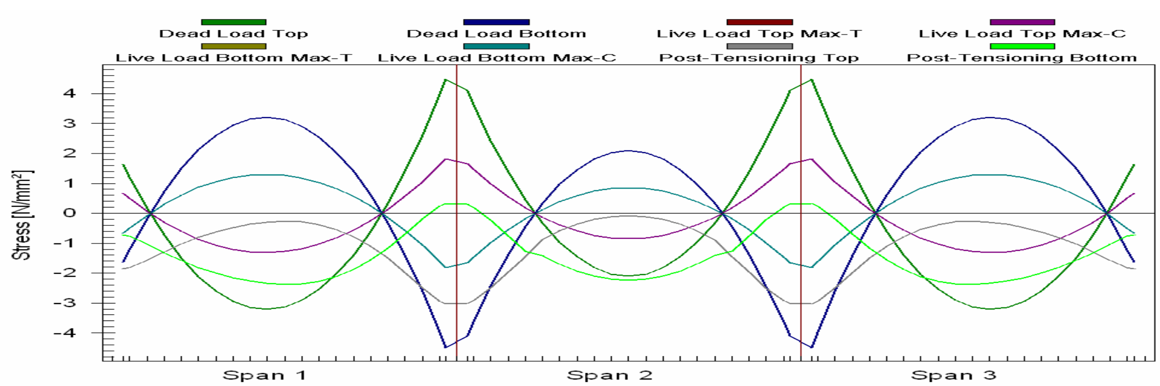


Fig. 6.7 Stress diagram for 7m span without drop

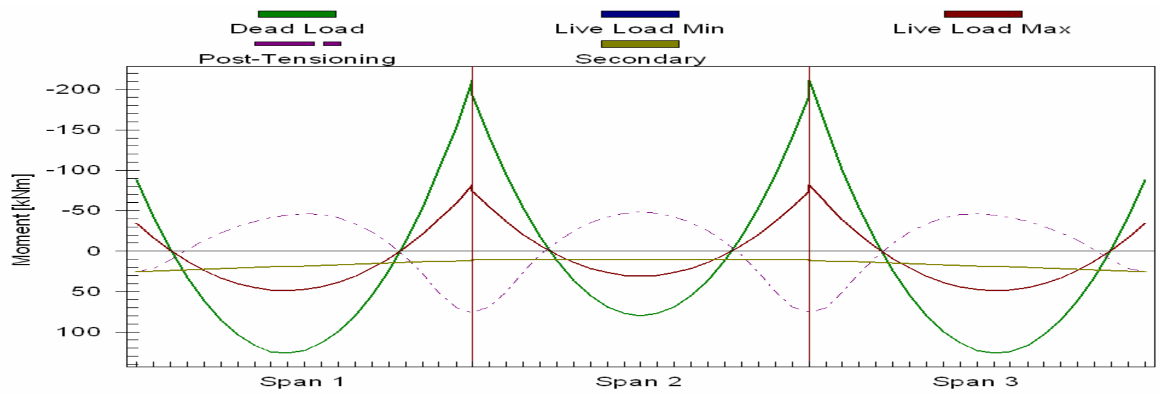


Fig. 6.8 Bending moment diagram for 7.5m span without drop

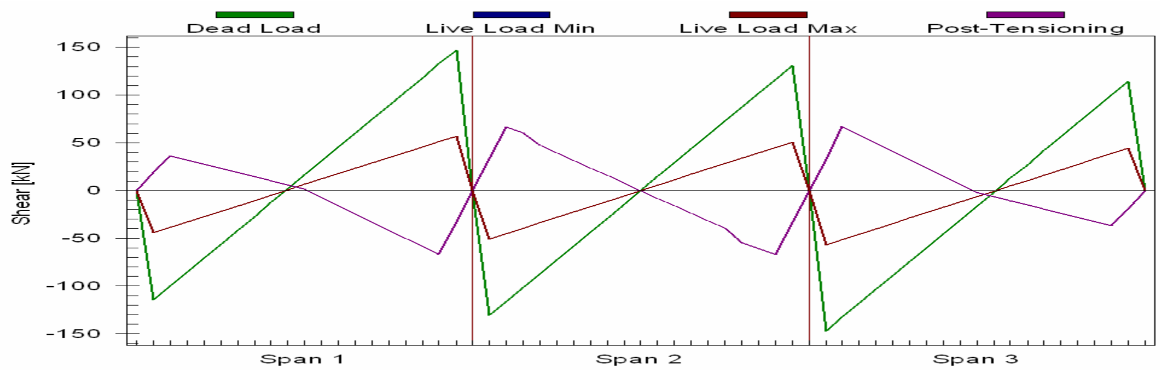


Fig. 6.9 Shear force diagram for 7.5m span without drop

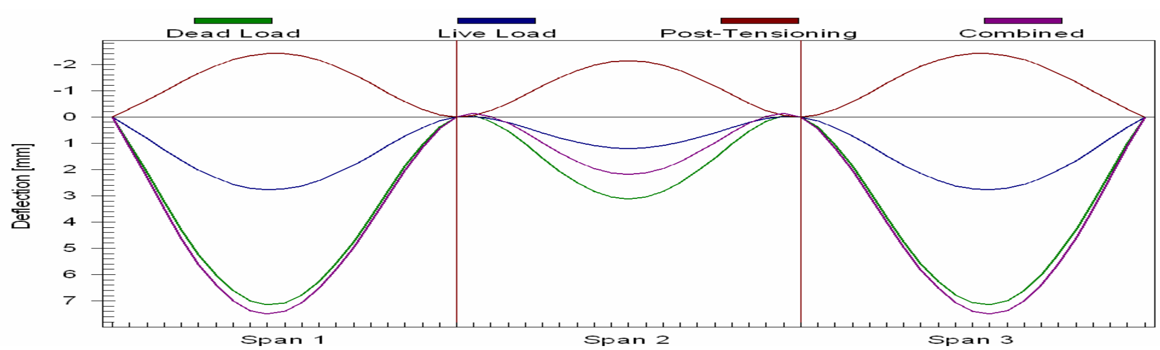


Fig. 6.10 Deflection for 7.5m span without drop

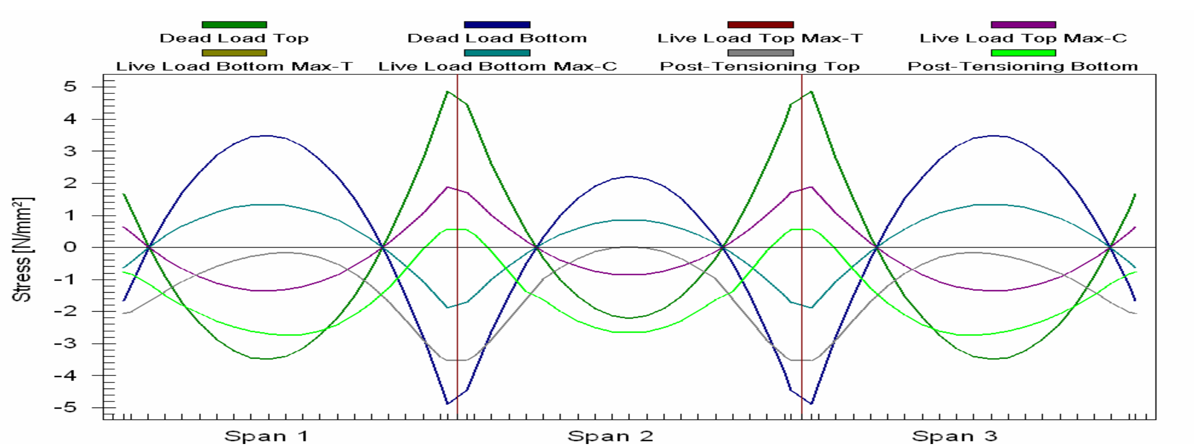


Fig. 6.11 Stress diagram for7.5m span without drop

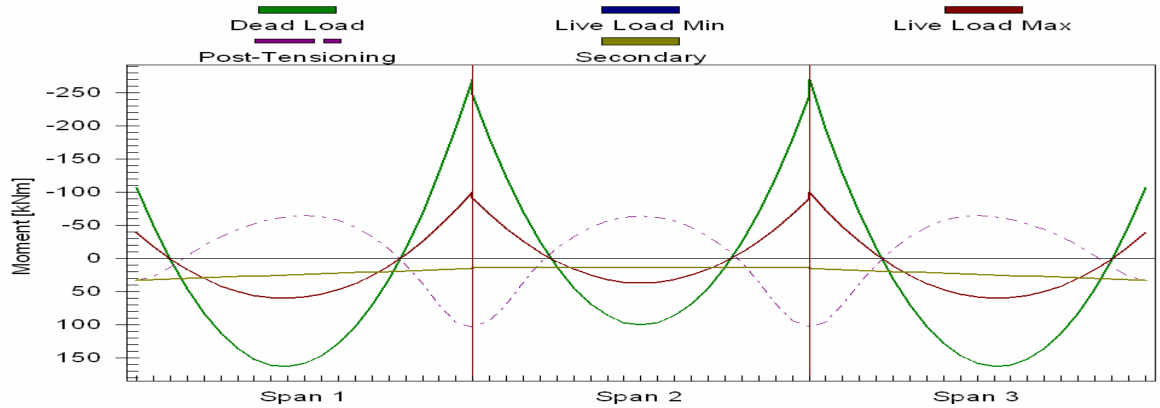


Fig. 6.12 Bending moment diagram for 8m span without drop

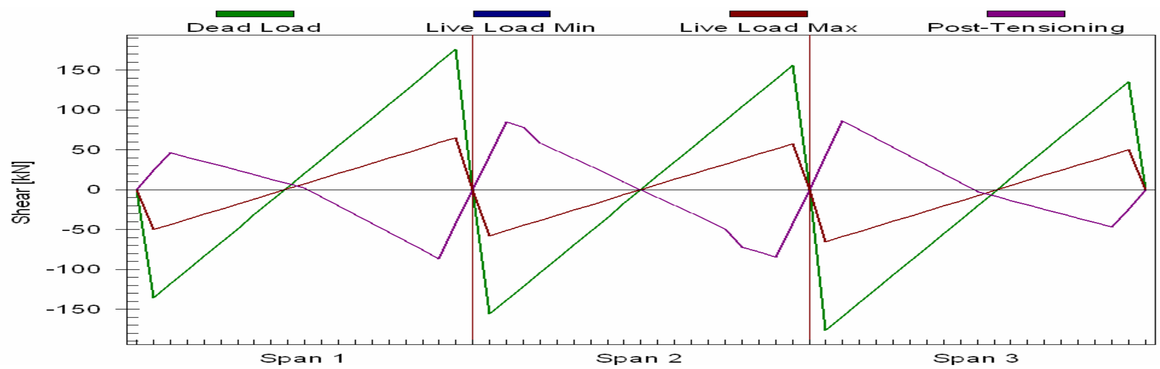


Fig. 6.13 Shear force diagram for 8m span without drop

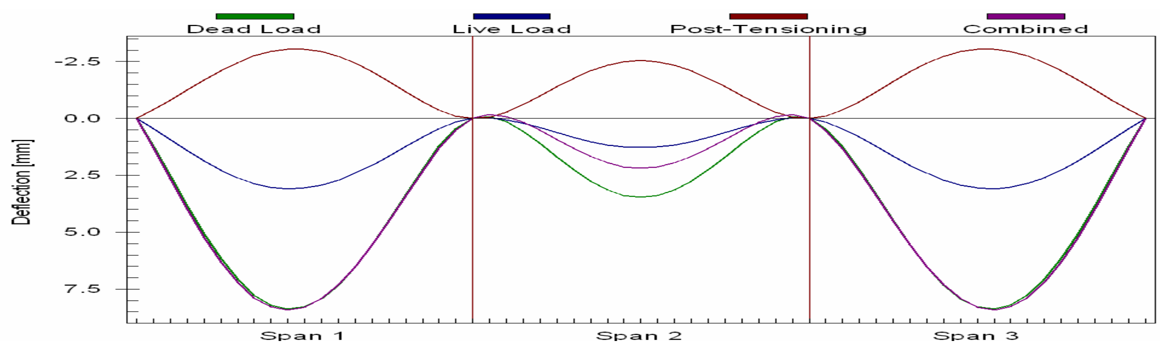


Fig. 6.14 Deflection for 8m span without drop

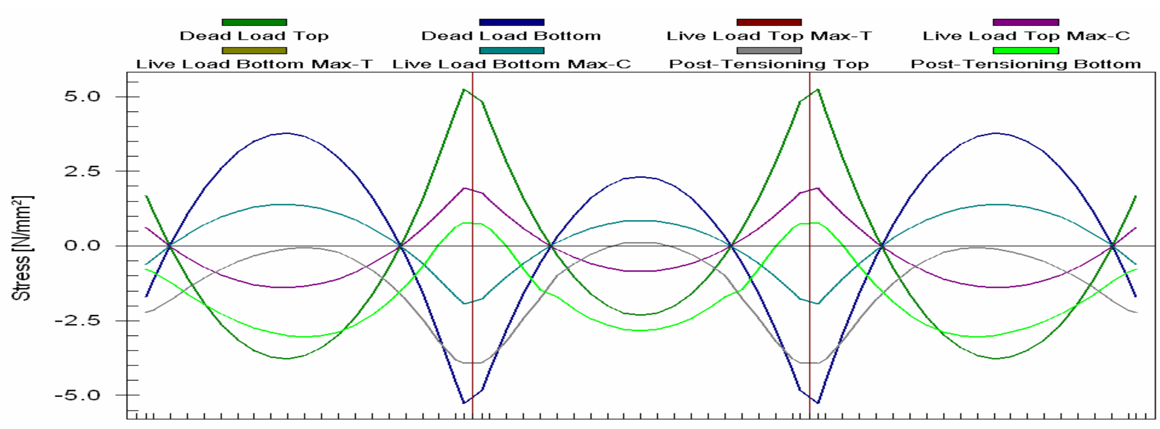


Fig. 6.15 Stress diagram for 8m span without drop

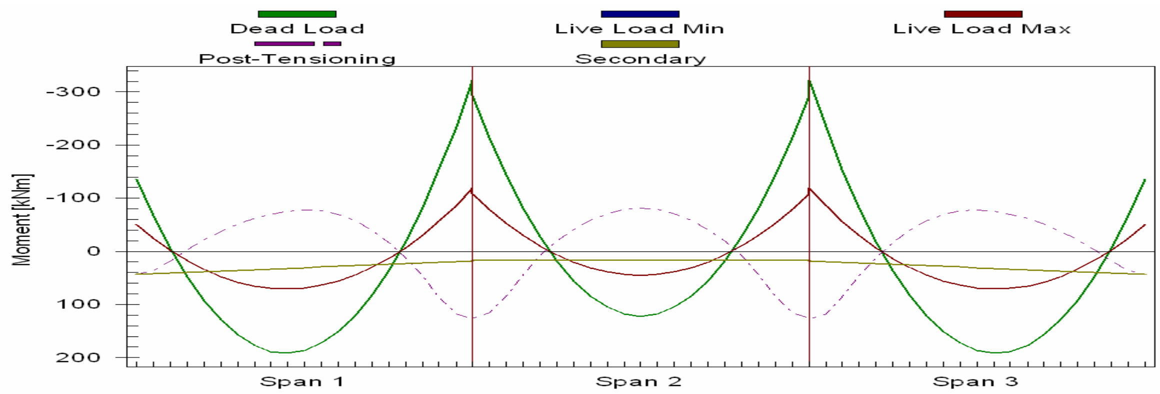


Fig. 6.16 Bending moment diagram for 8.5m span without drop

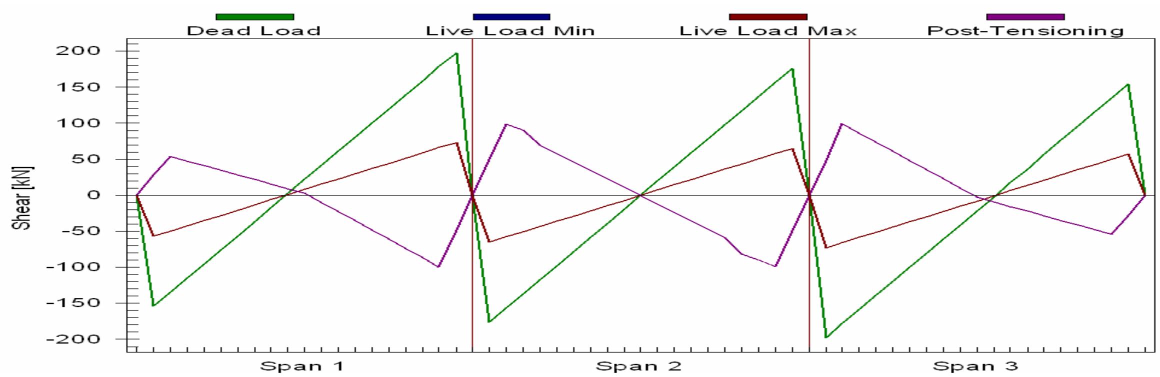


Fig. 6.17 Shear force diagram for 8.5m span without drop

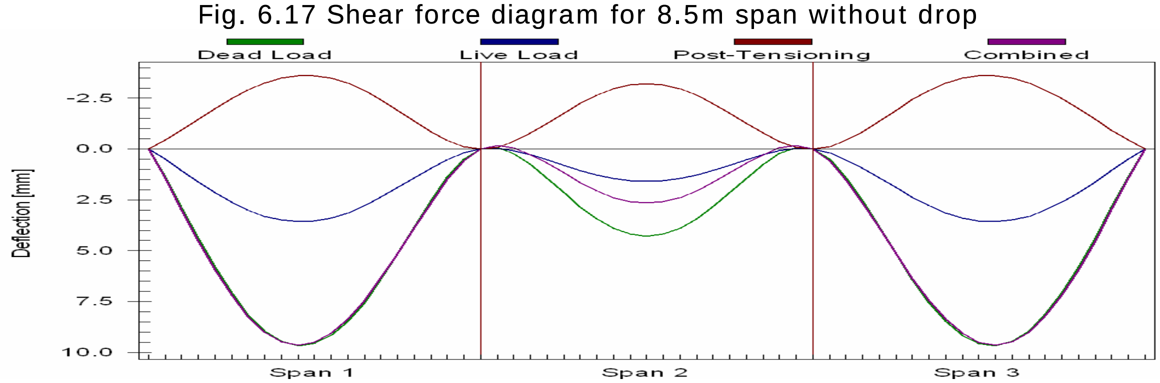


Fig. 6.18 Deflection for 8.5m span without drop

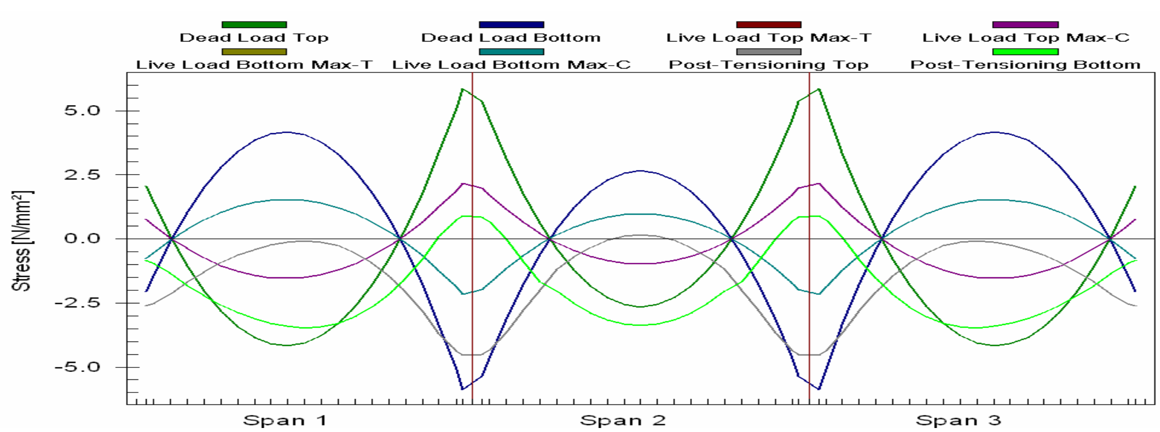


Fig. 6.19 Stress diagram for 8.5m span without drop

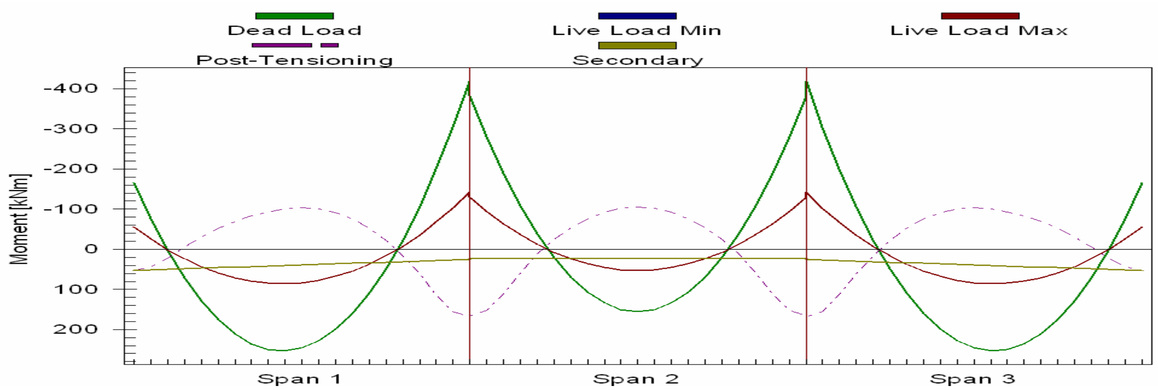


Fig. 6.20 Bending moment diagram for 9m span without drop

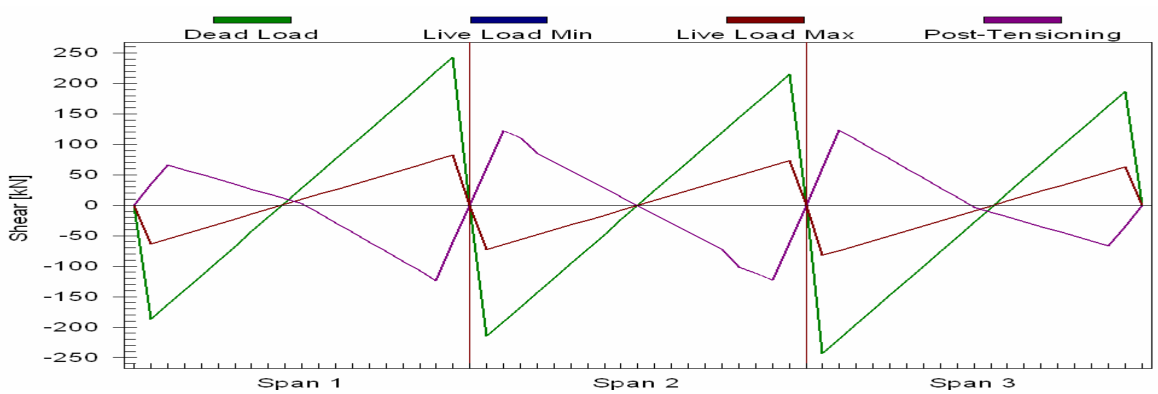


Fig. 6.21 Shear force diagram for 9m span without drop

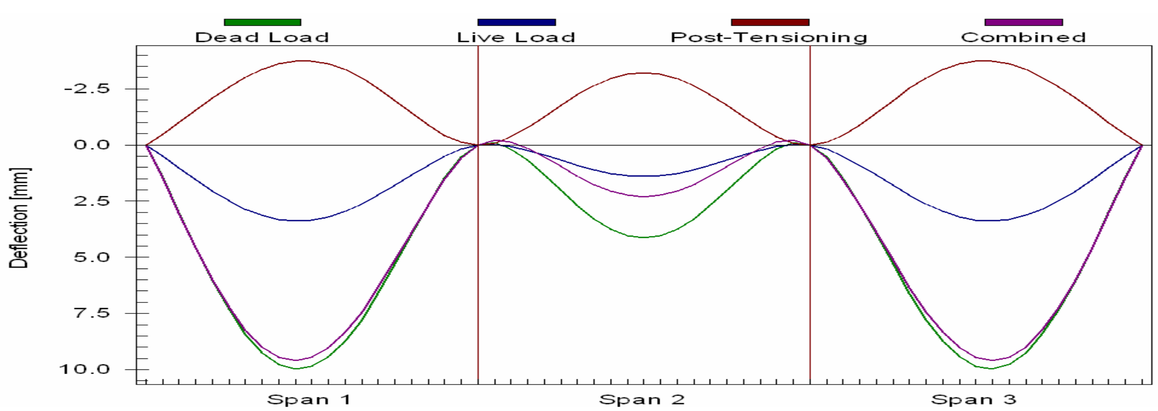


Fig. 6.22 Deflection for 9m span without drop

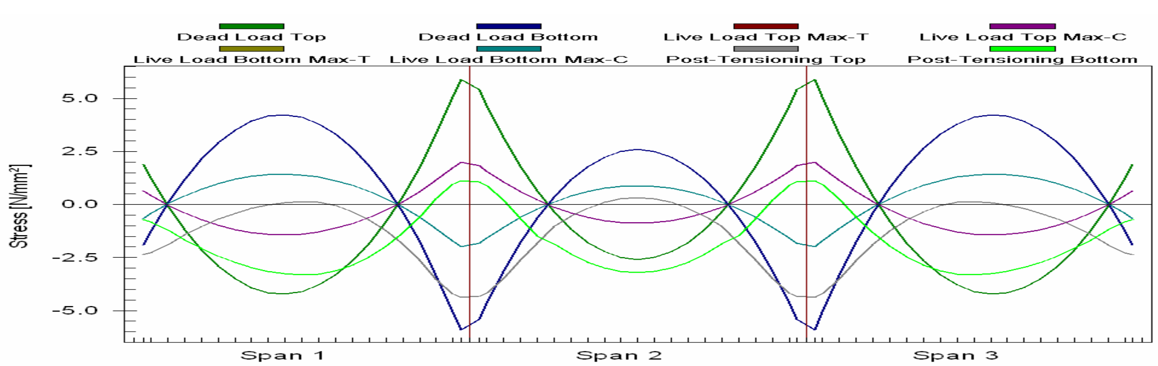


Fig. 6.23 Stress diagram for 9m span without drop

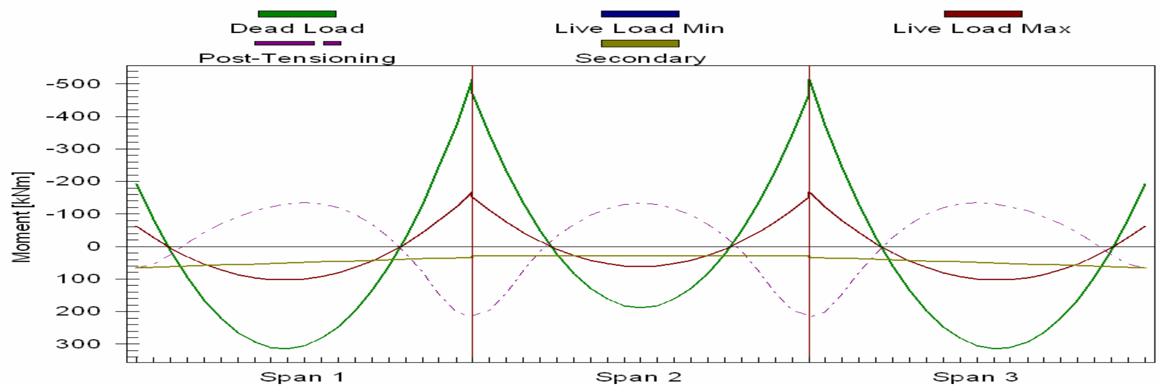


Fig. 6.24 Bending moment diagram for 9.5m span without drop

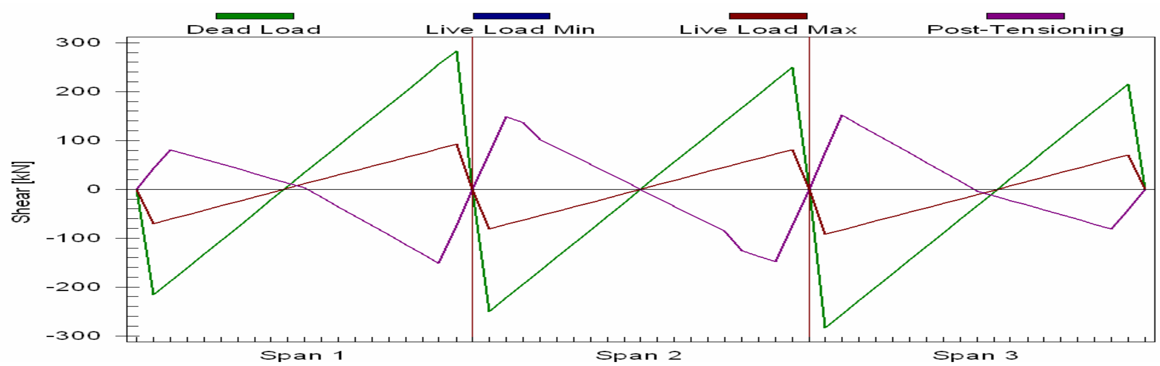


Fig. 6.25 Shear force diagram for 9.5m span without drop

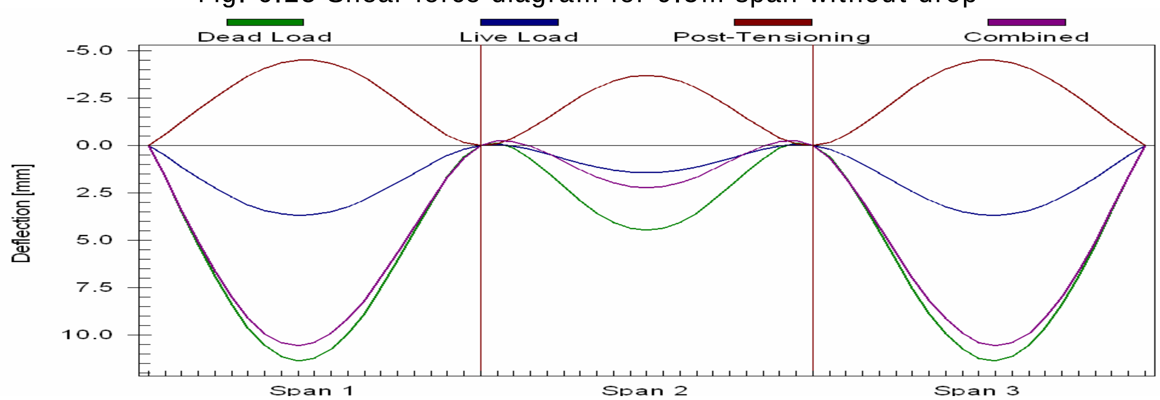


Fig. 6.26 Deflection for 9.5m span without drop

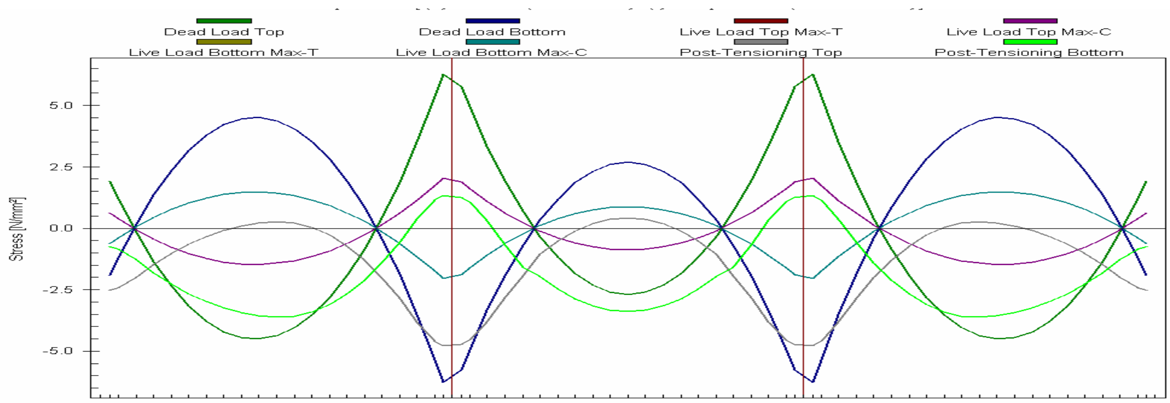


Fig. 6.27 Stress diagram for 9.5m span without drop

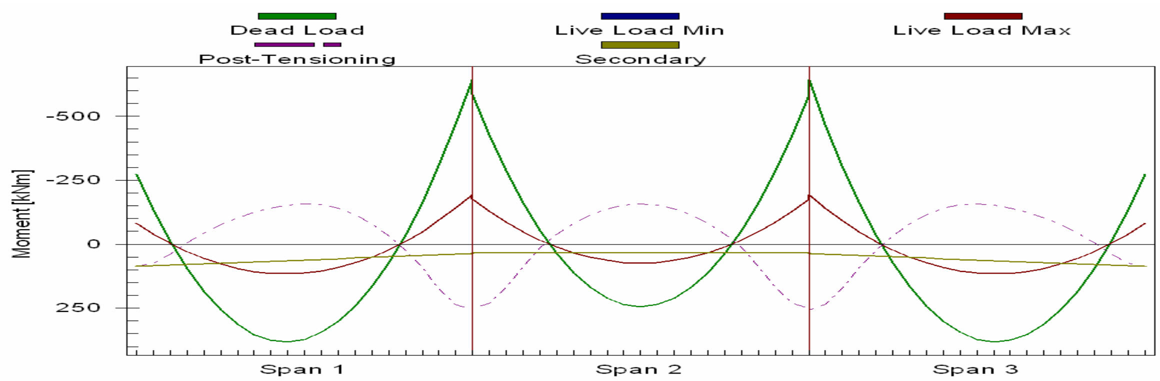


Fig. 6.28 Bending moment diagram for 10m span without drop

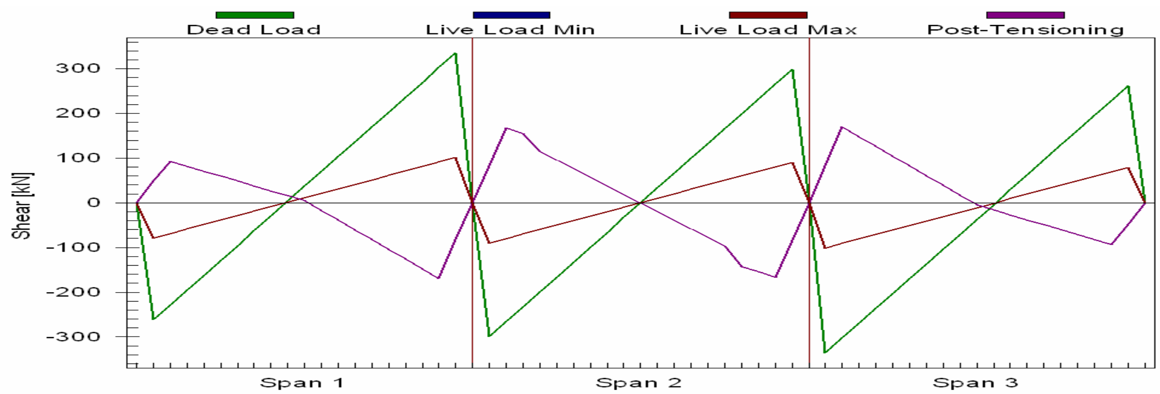


Fig. 6.29 Shear force diagram for 10m span without drop

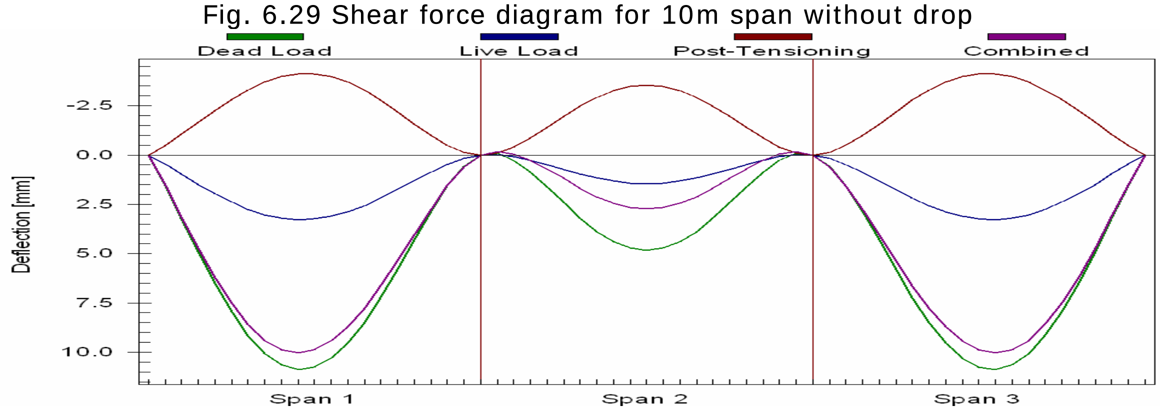


Fig. 6.30 Deflection for 10m span without drop

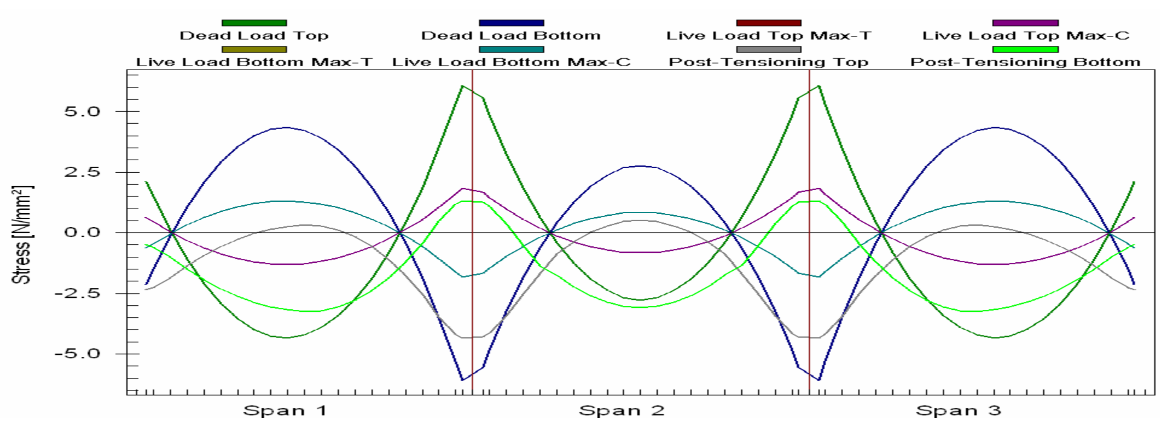


Fig. 6.31 Stress diagram for 10m span without drop

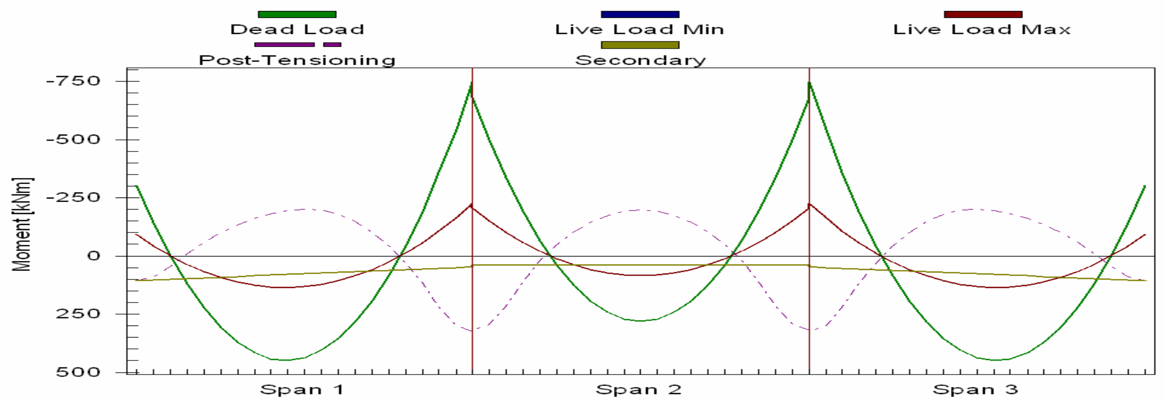


Fig. 6.32 Bending moment diagram for 10.5m span without drop

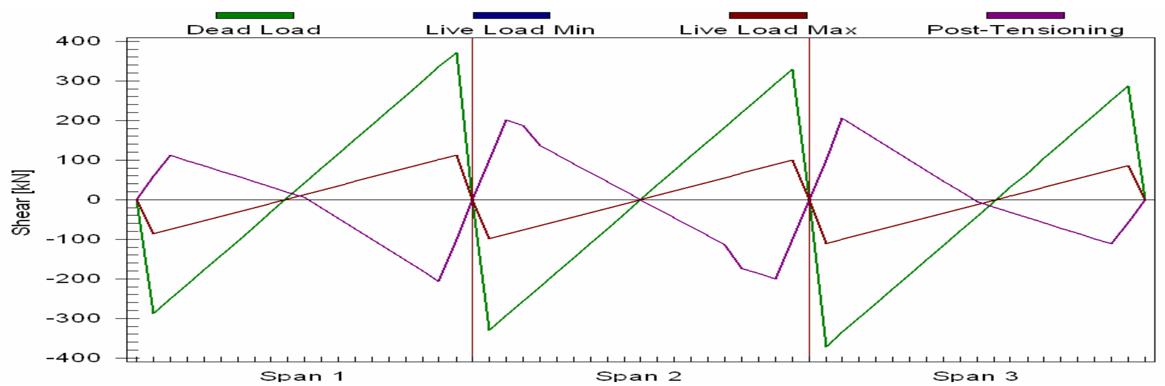


Fig. 6.33 Shear force diagram for 10.5m span without drop

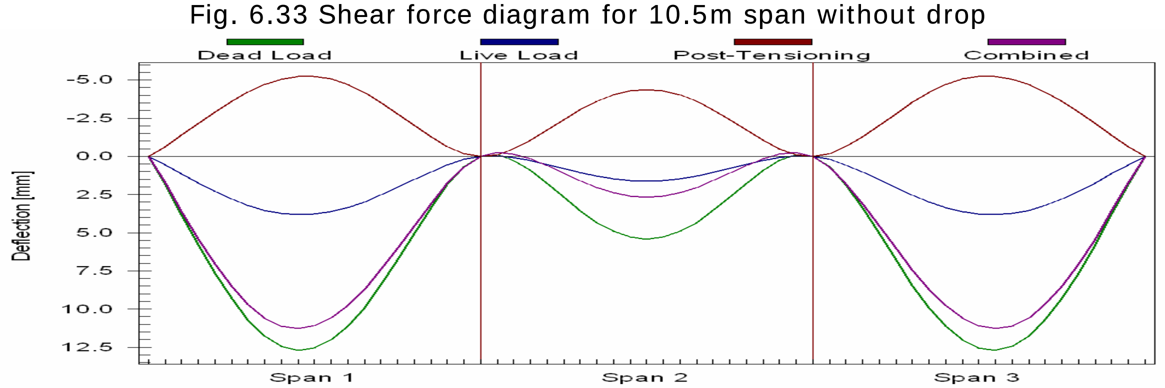


Fig. 6.34 Deflection for 10.5m span without drop

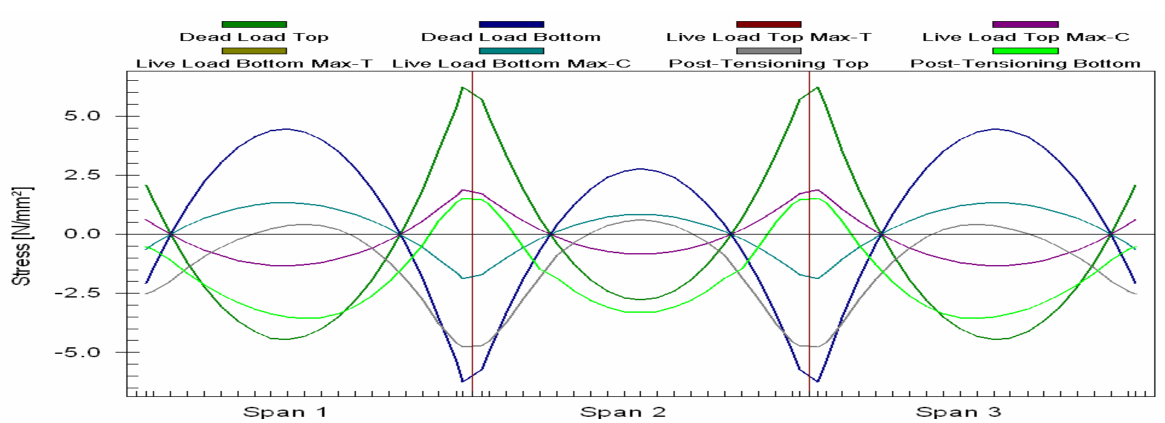


Fig. 6.35 Stress diagram for 10.5m span without drop

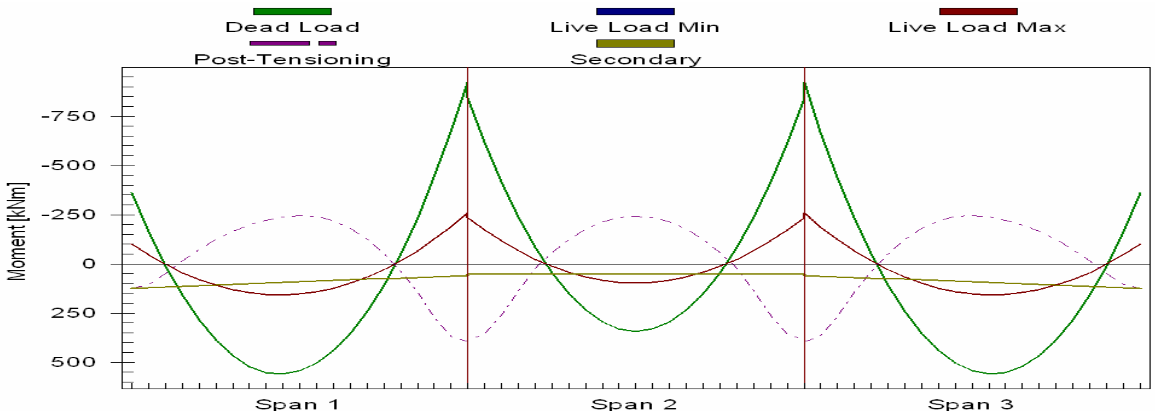


Fig. 6.36 Bending moment diagram for 11m span without drop

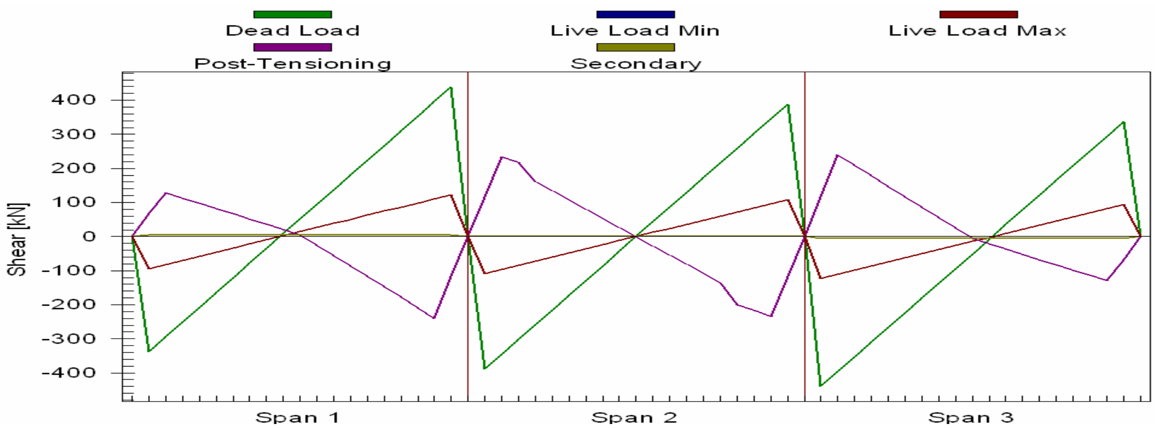


Fig. 6.37 Shear force diagram for 11m span without drop

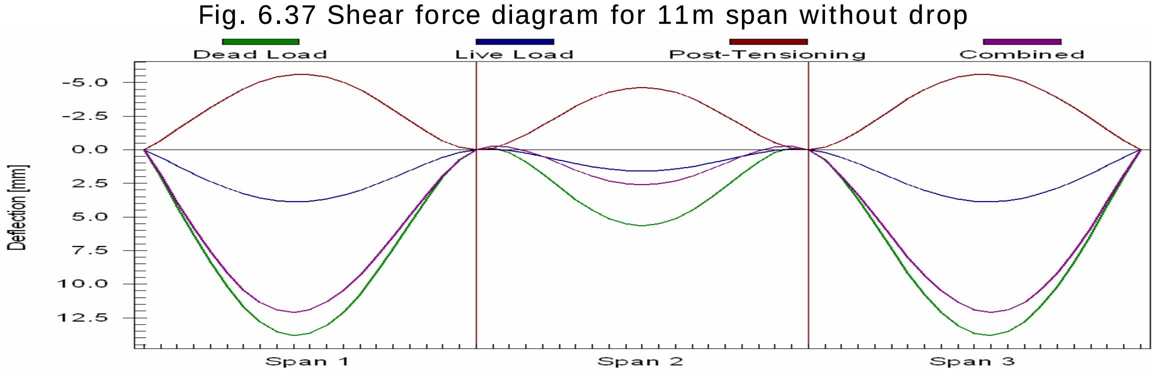


Fig. 6.38 Deflection for 11m span without drop

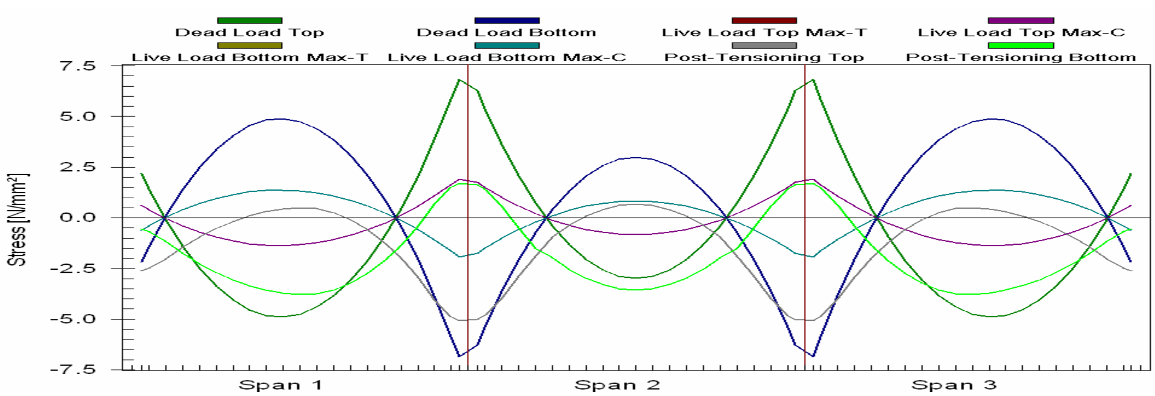


Fig. 6.39 Stress diagram for 11m span without drop

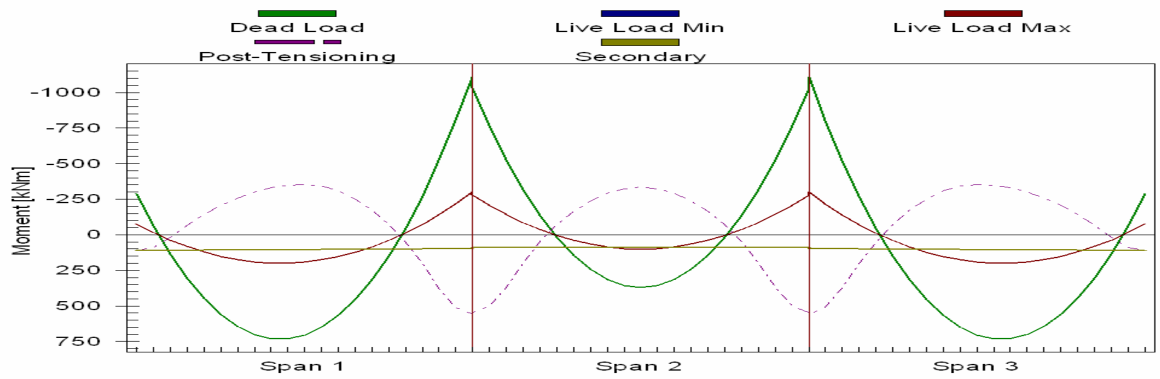


Fig. 6.40 Bending moment diagram for 11.5m span without drop

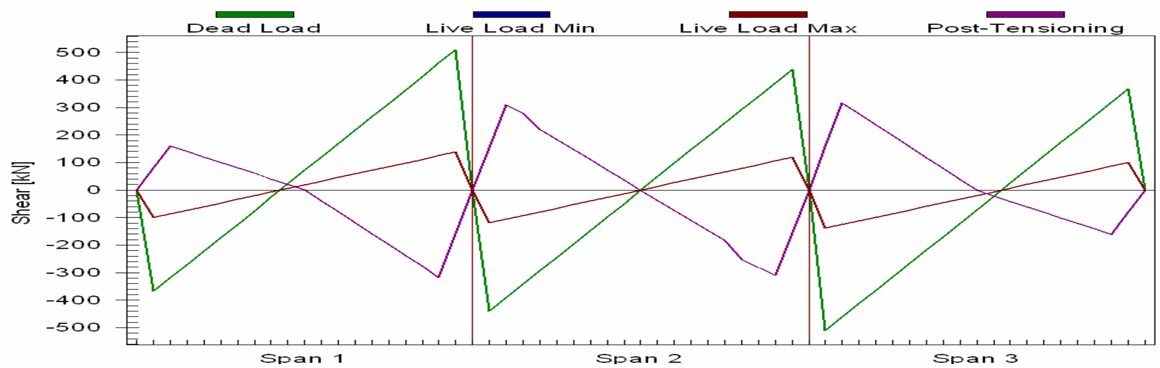


Fig. 6.41 Shear force diagram for 11.5m span without drop

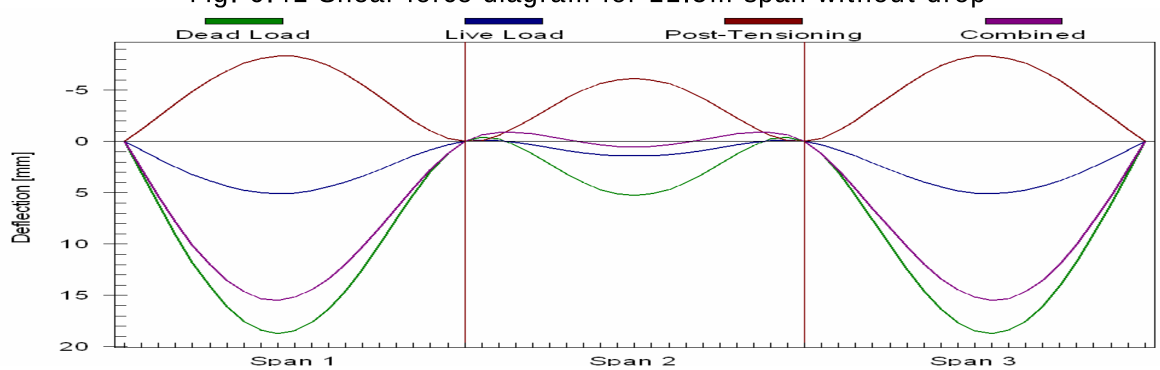


Fig. 6.42 Deflection for 11.5m span without drop

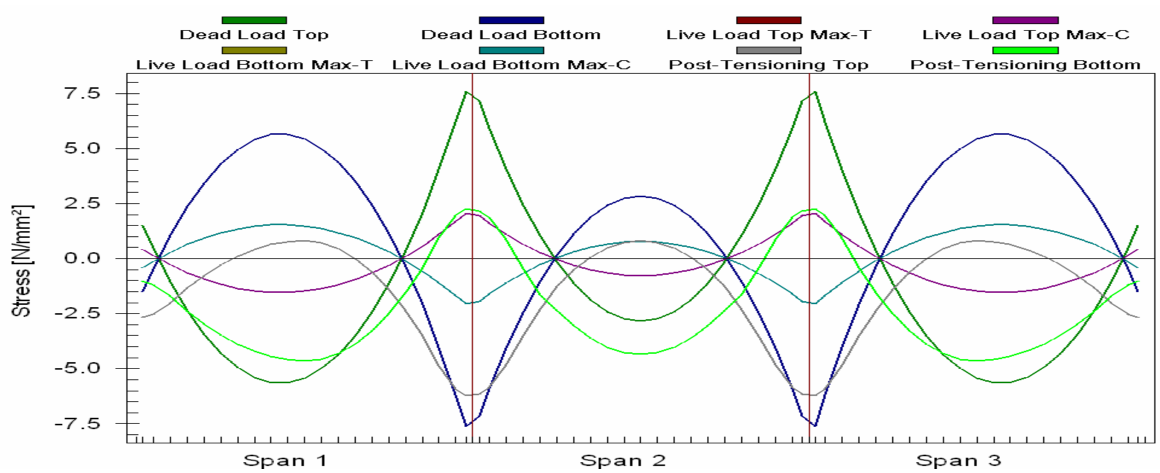


Fig. 6.43 Stress diagram for 11.5m span without drop

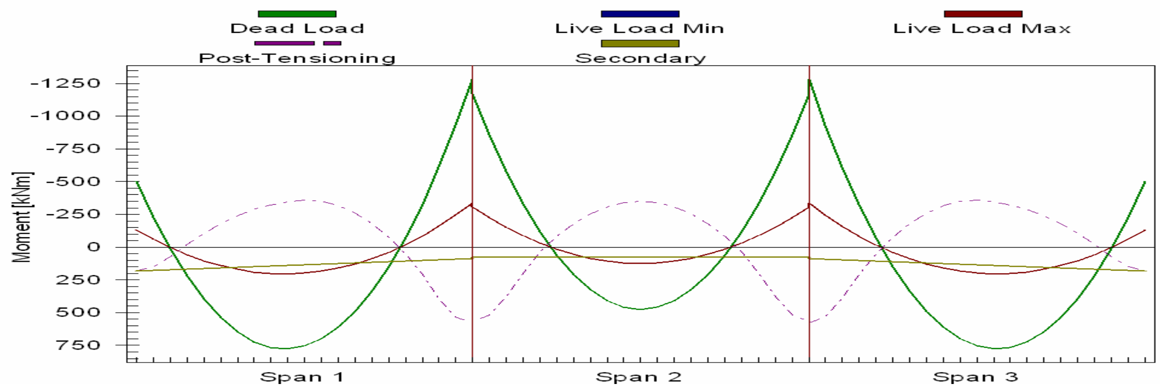


Fig. 6.44 Bending moment diagram for 12m span without drop

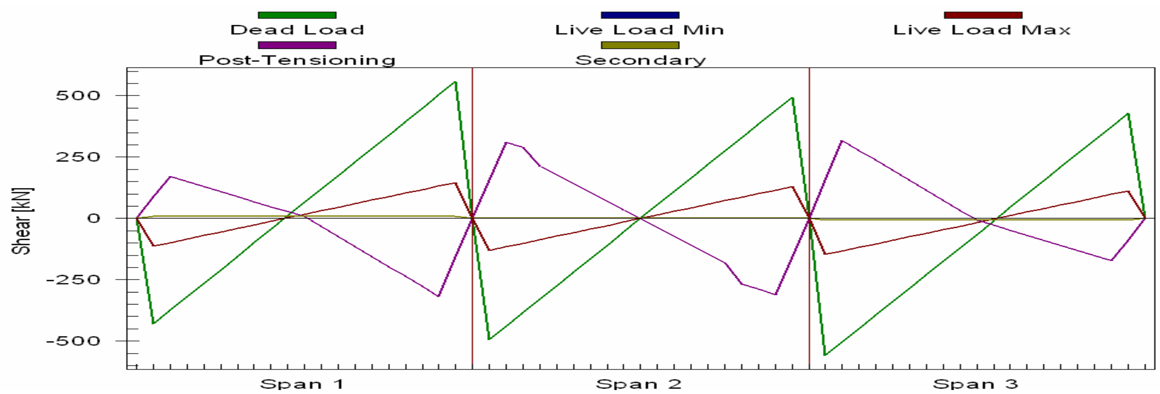


Fig. 6.45 Shear force diagram for 12m span without drop

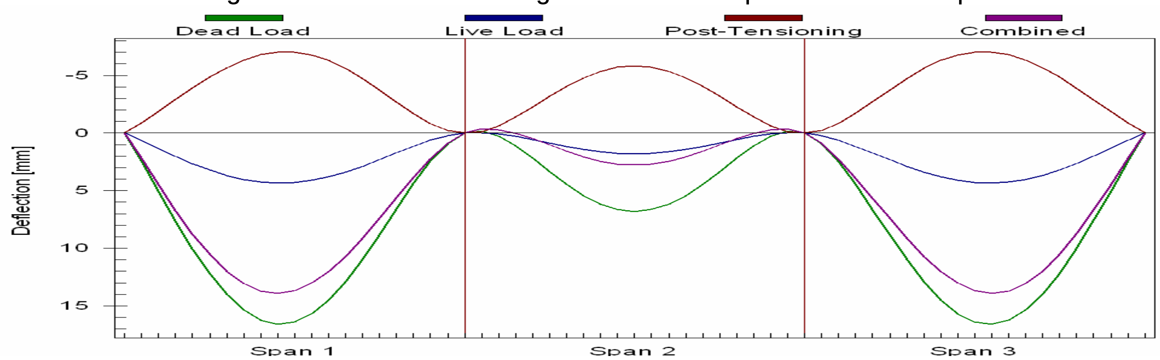


Fig. 6.46 Deflection for 12m span without drop

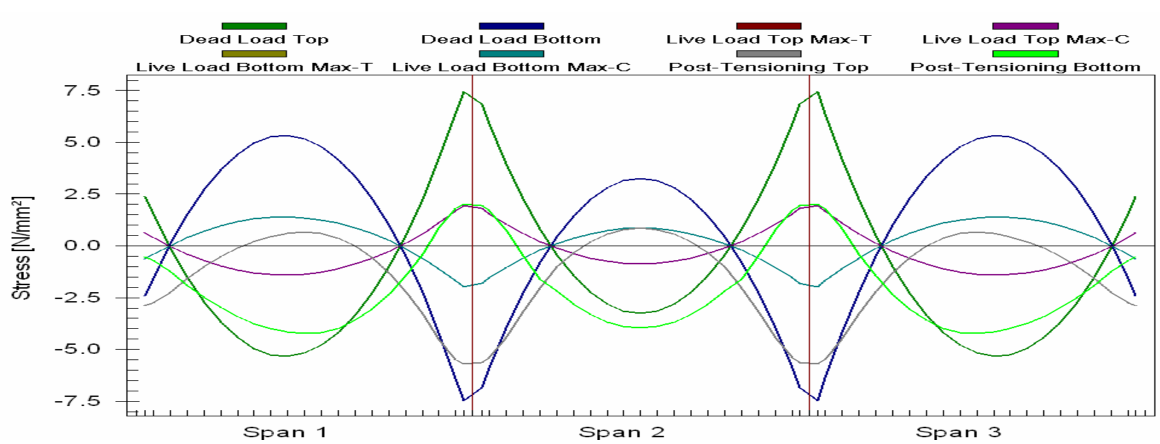


Fig. 6.47 Stress diagram for 12m span without drop

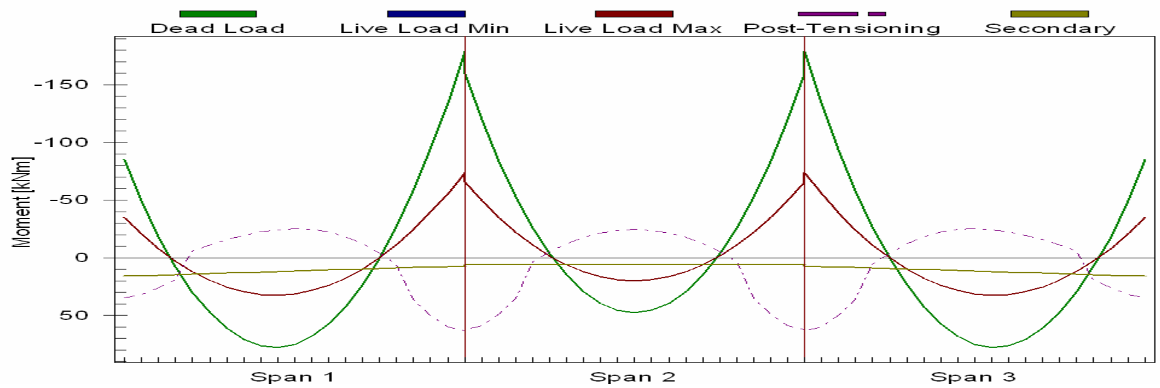


Fig. 6.48 Bending moment diagram for 7m span with drop

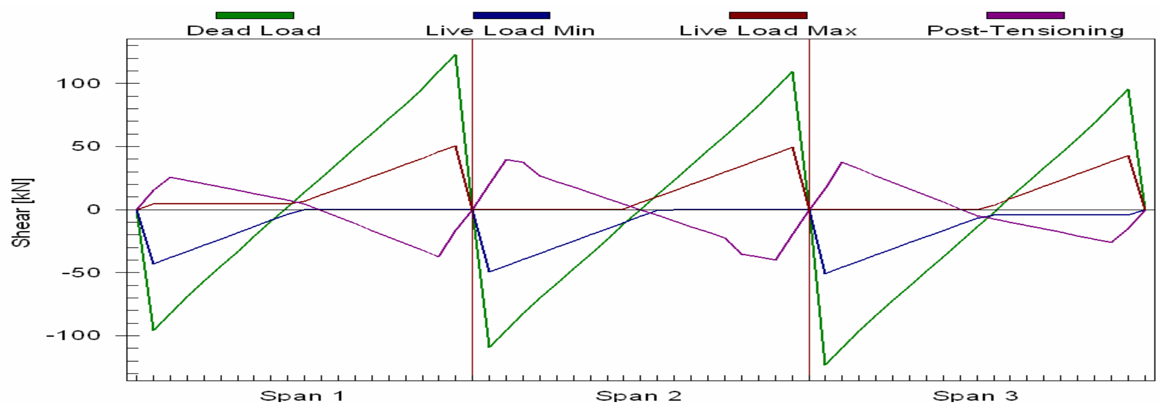


Fig. 6.49 Shear force diagram for 7m span with drop

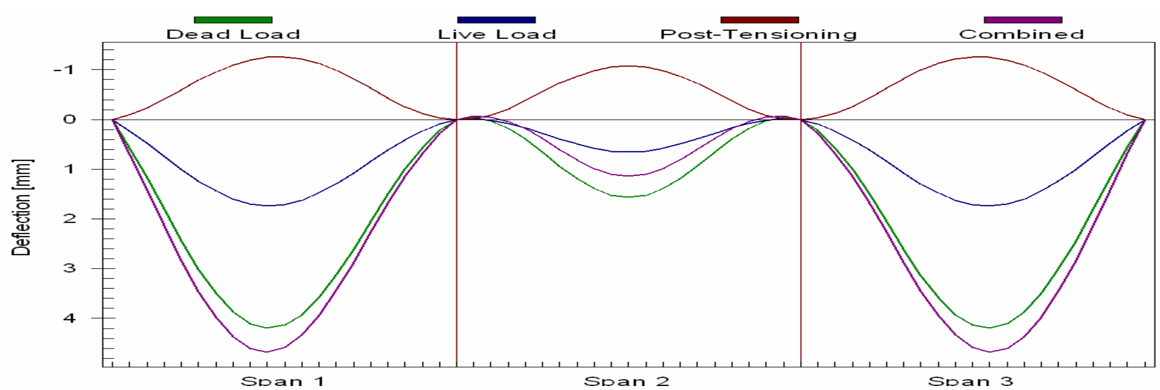


Fig. 6.50 Deflection for 7m span with drop

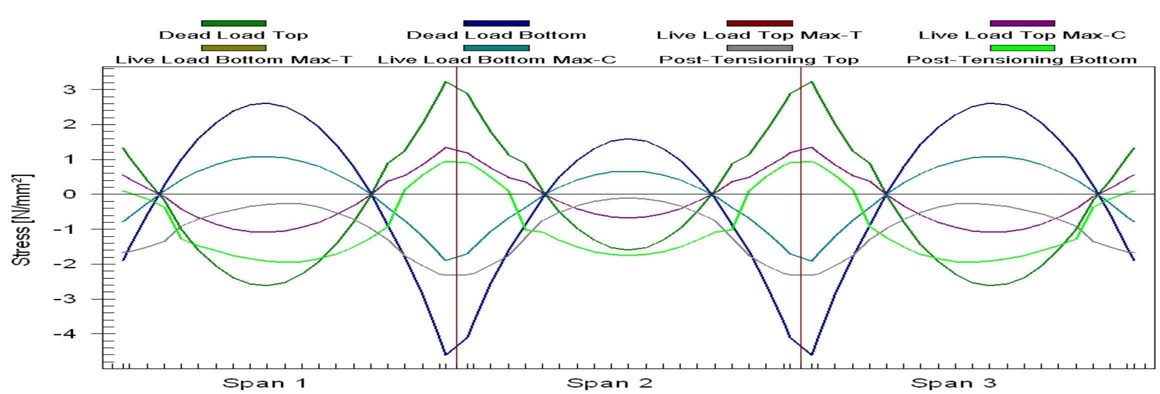


Fig. 6.51 Stress diagram for 7m span with drop

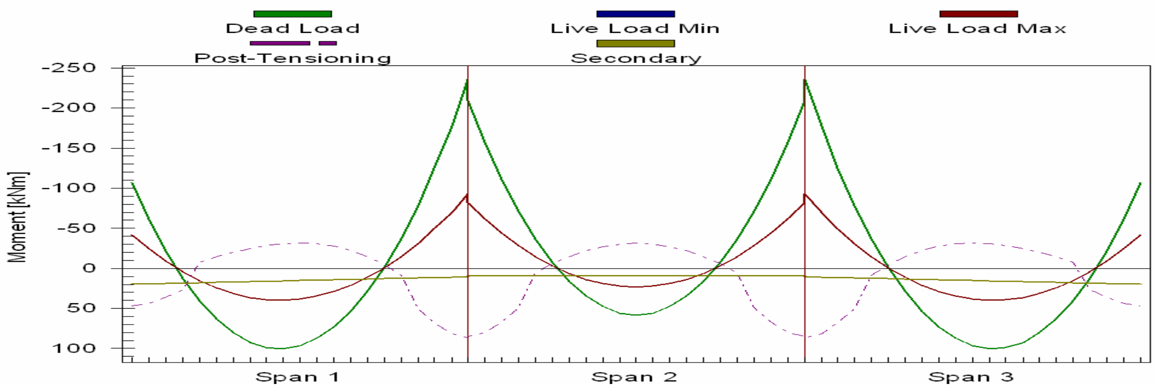


Fig. 6.52 Bending moment diagram for 7.5m span with drop

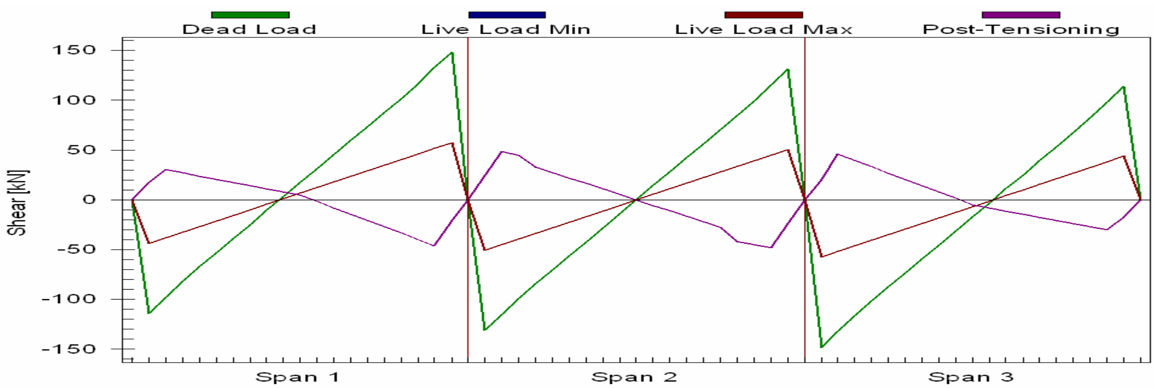


Fig. 6.53 Shear force diagram for 7.5m span with drop

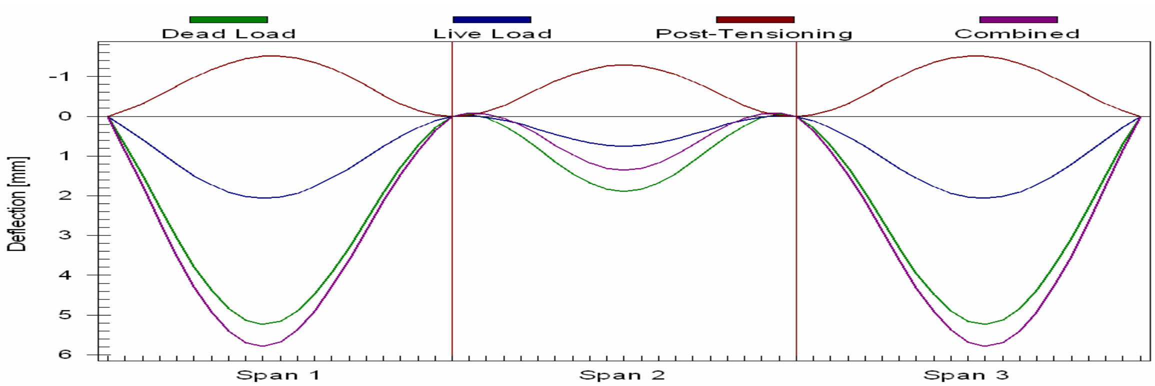


Fig. 6.54 Deflection for 7.5m span with drop

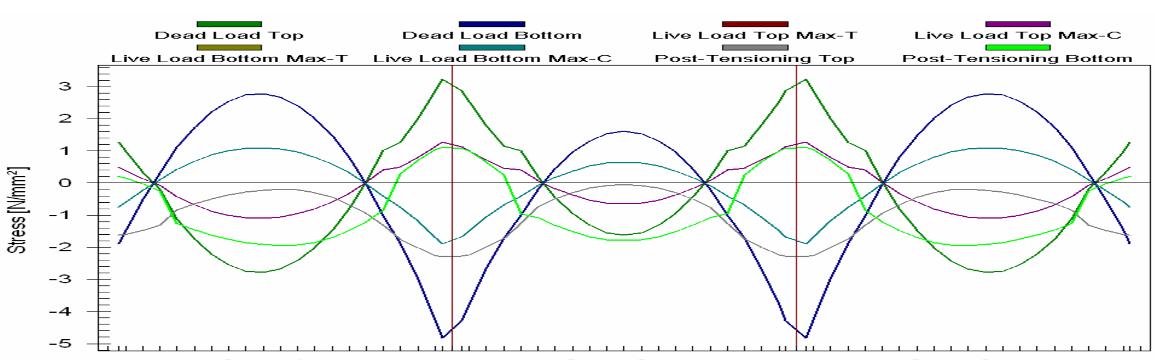


Fig. 6.55 Stress diagram for7.5m span with drop

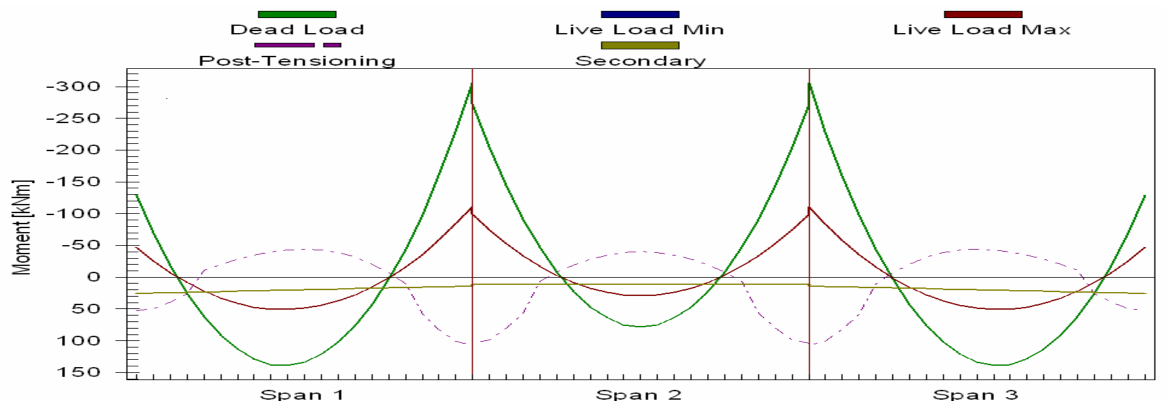


Fig. 6.56 Bending moment diagram for 8m span with drop

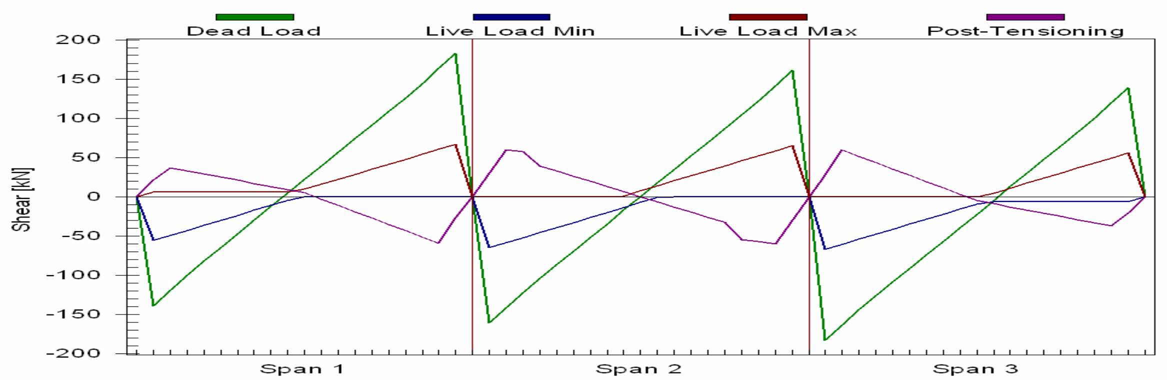


Fig. 6.57 Shear force diagram for 8m span with drop

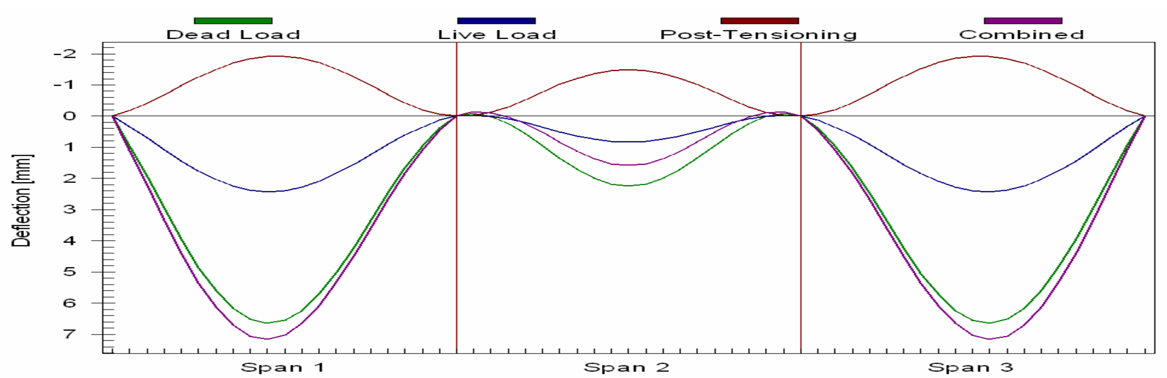


Fig. 6.58 Deflection for 8m span with drop

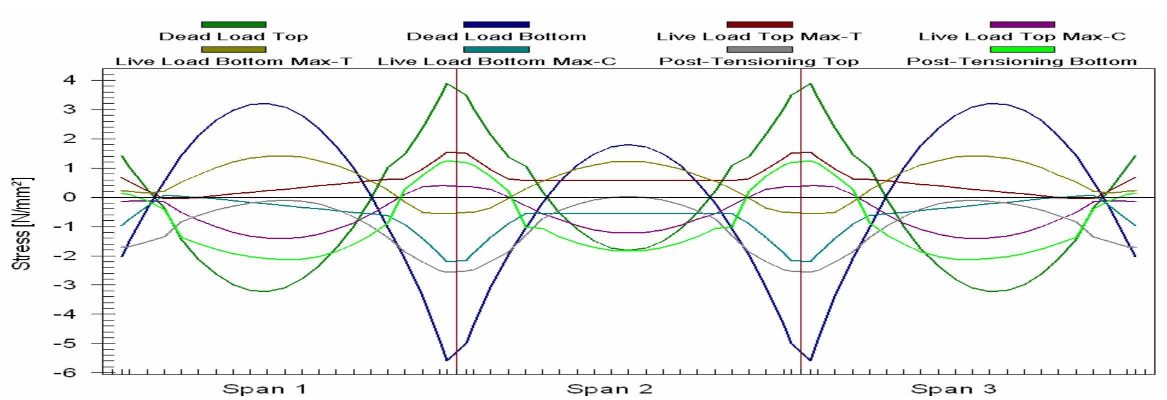


Fig. 6.59 Stress diagram for 8m span with drop

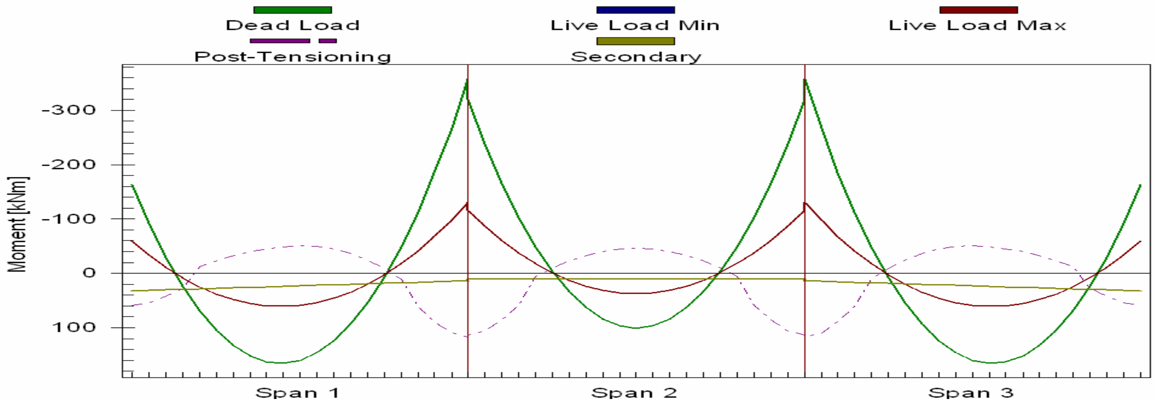


Fig. 6.60 Bending moment diagram for 8.5m span with drop

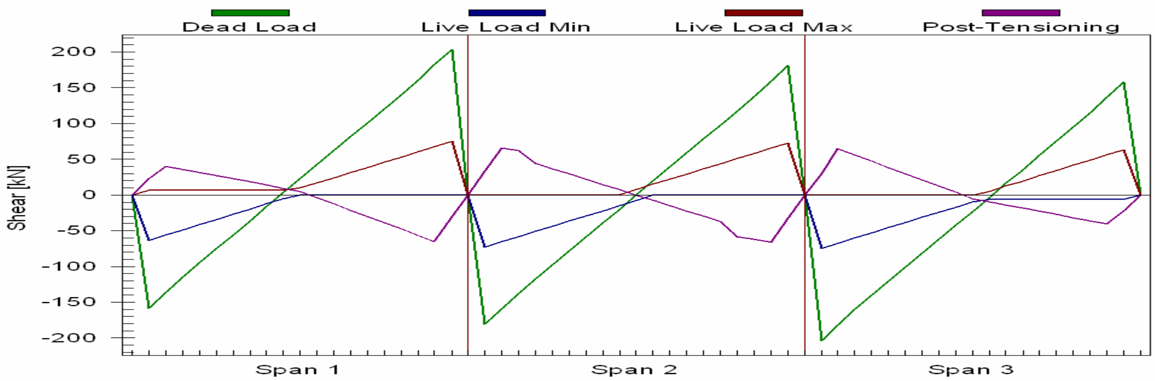


Fig. 6.61 Shear force diagram for 8.5m span with drop

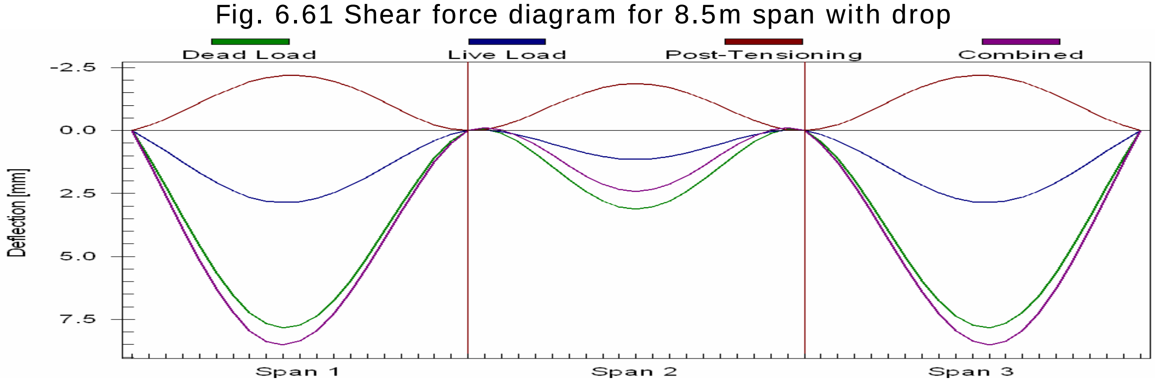


Fig. 6.62 Deflection for 8.5m span with drop

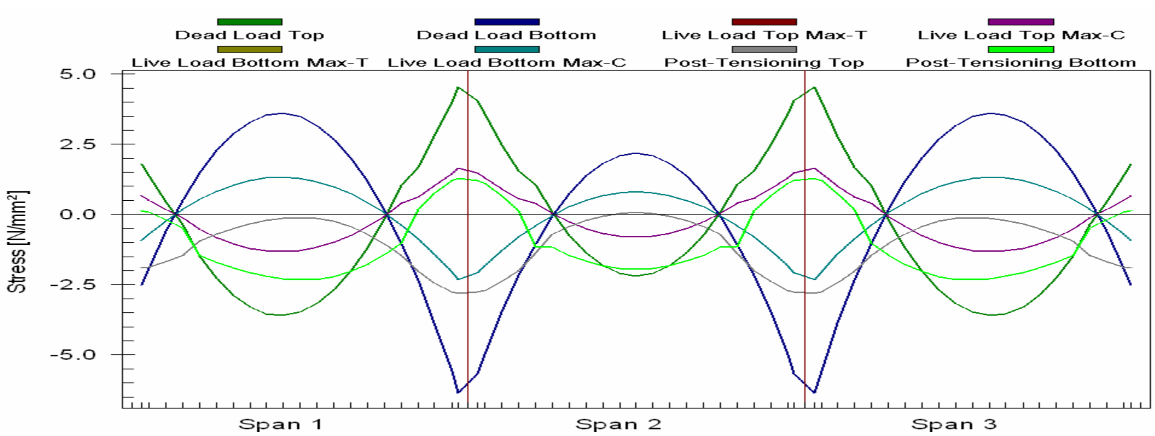


Fig. 6.63 Stress diagram for 8.5m span with drop

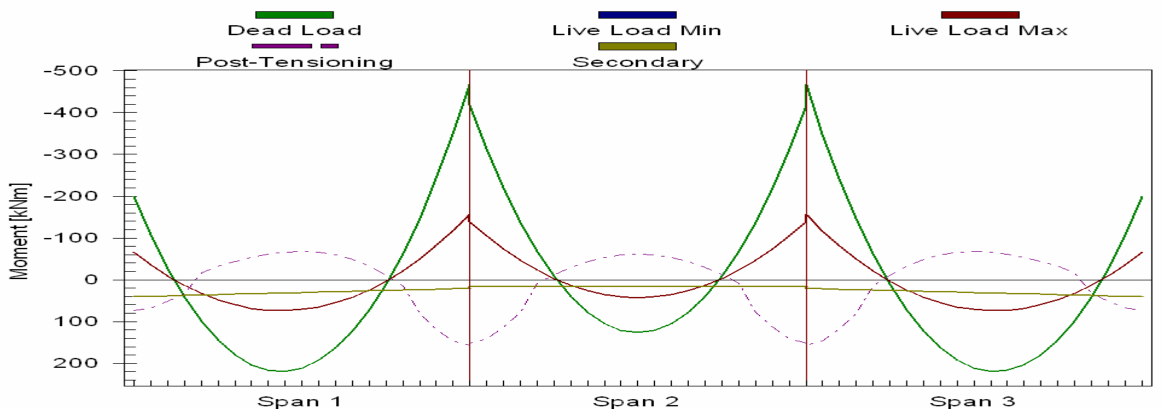


Fig. 6.64 Bending moment diagram for 9m span with drop

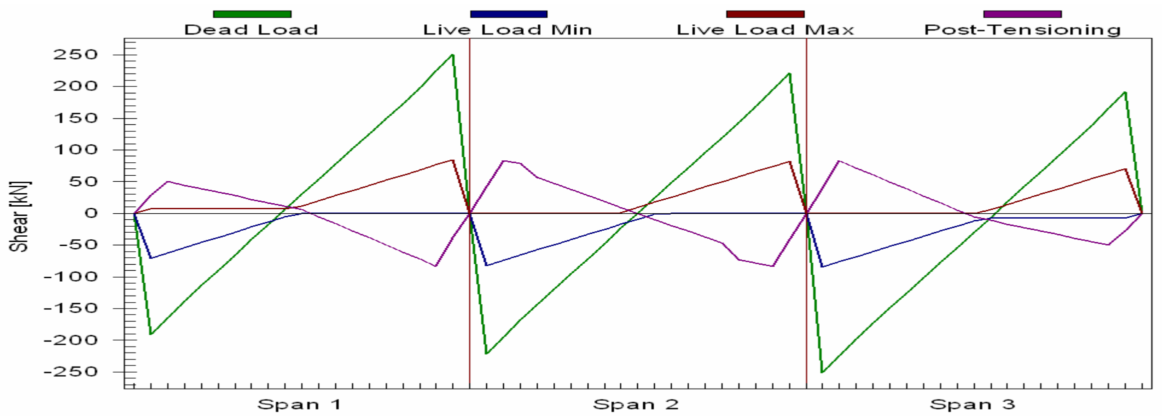


Fig. 6.65 Shear force diagram for 9m span with drop

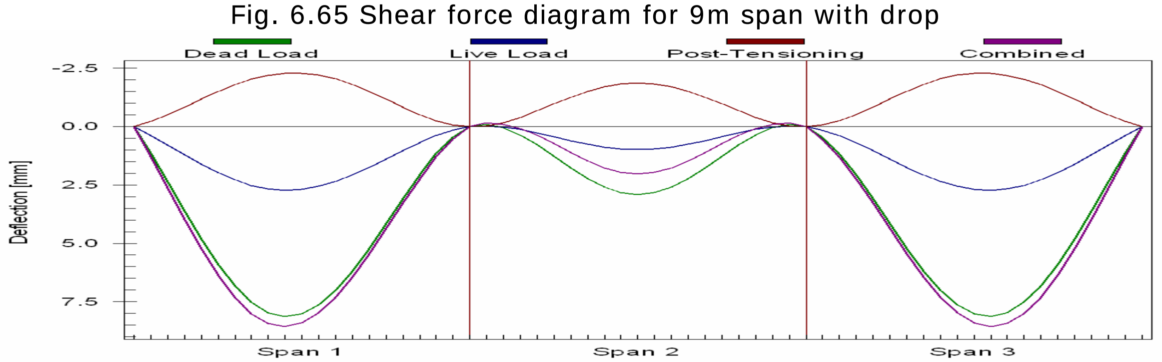


Fig. 6.66 Deflection for 9m span with drop

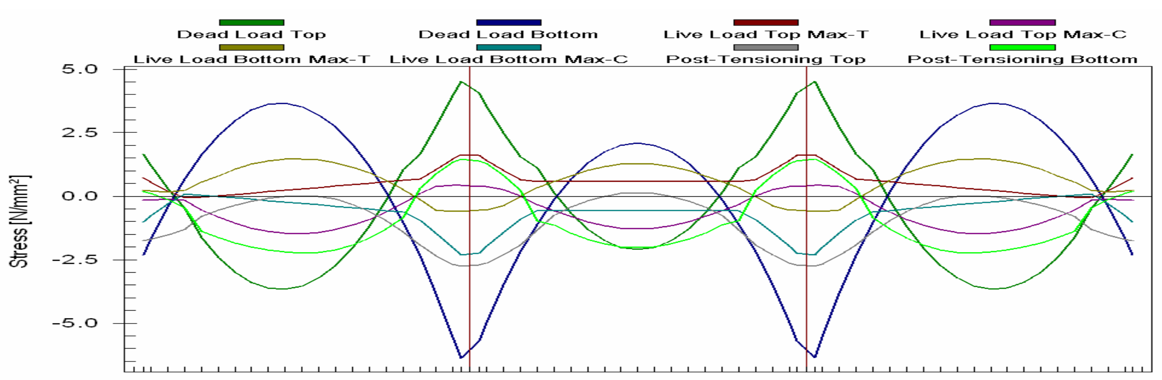


Fig. 6.67 Stress diagram for 9m span with drop

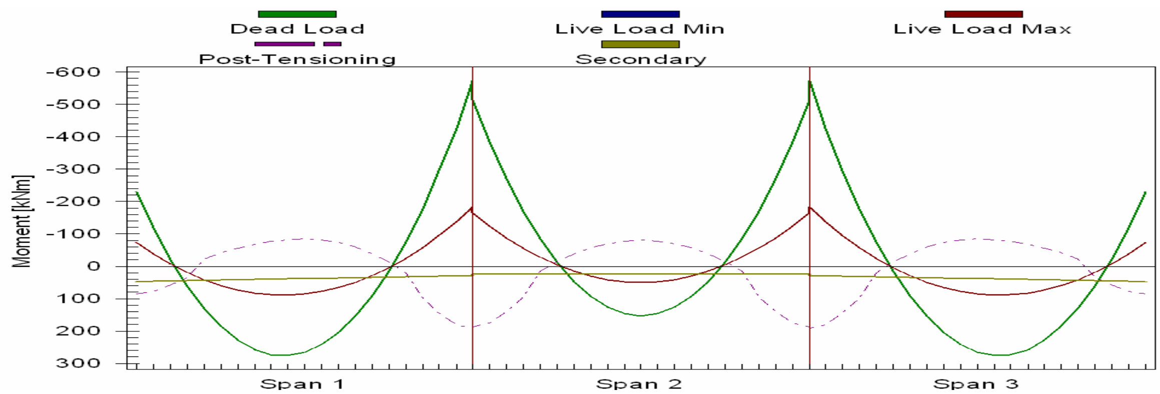


Fig. 6.68 Bending moment diagram for 9.5m span with drop

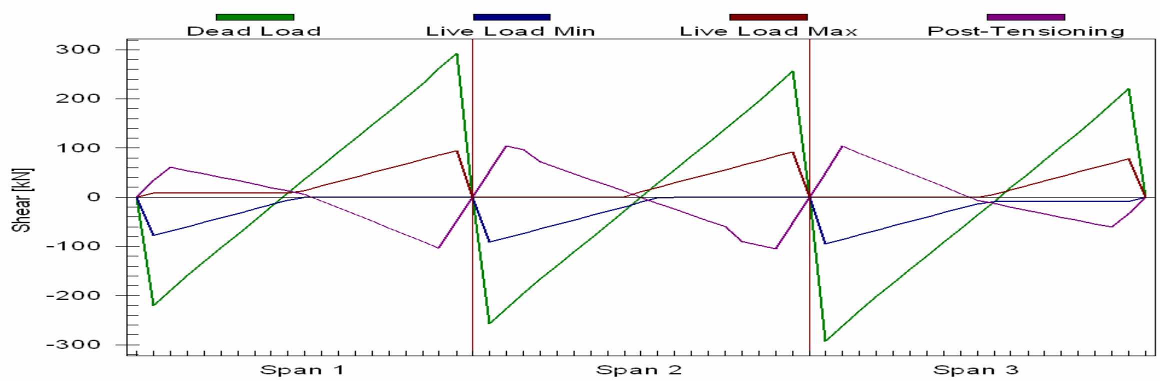


Fig. 6.69 Shear force diagram for 9.5m span with drop

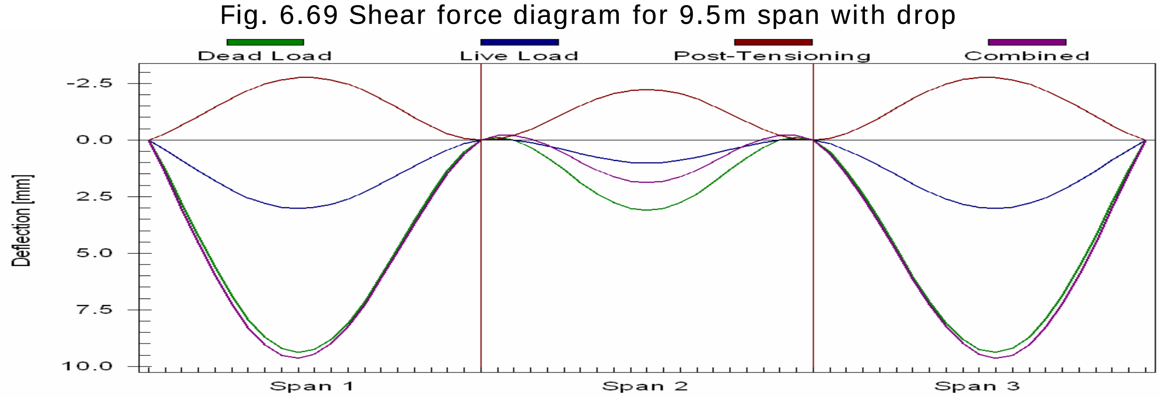


Fig. 6.70 Deflection for 9.5m span with drop

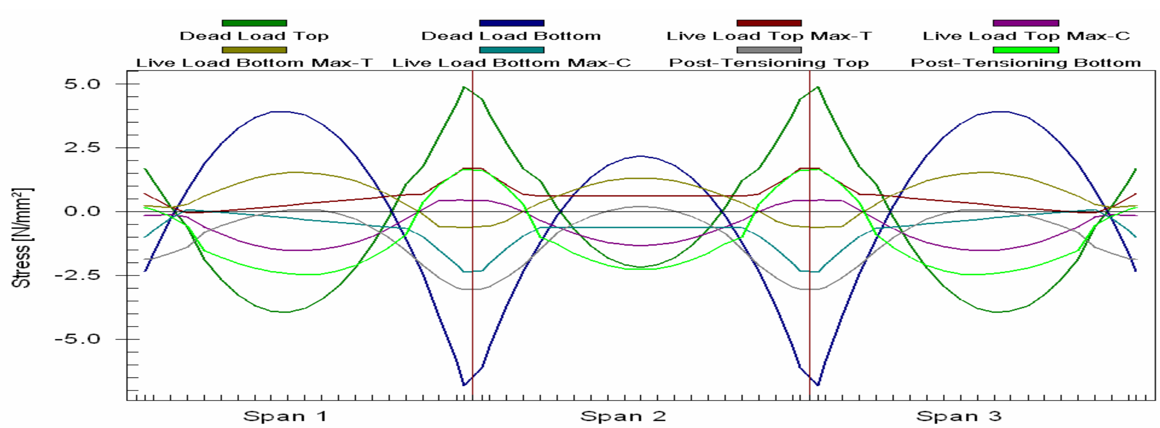


Fig. 6.71 Stress diagram for 9.5m span with drop

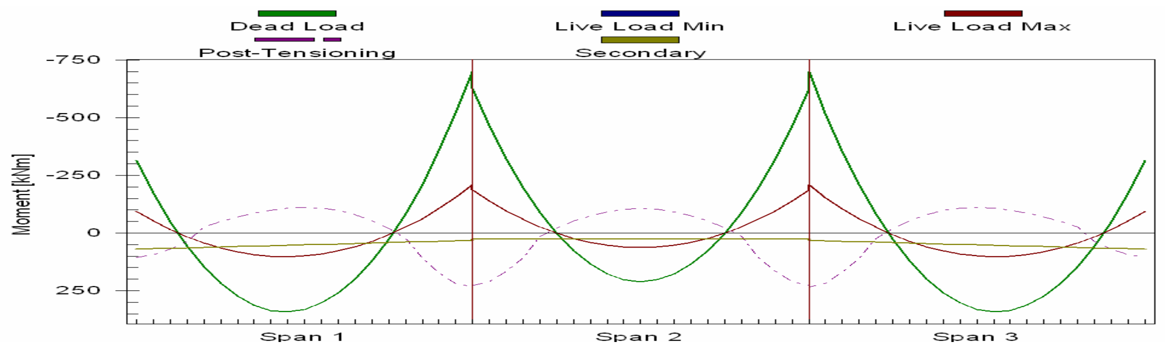


Fig. 6.72 Bending moment diagram for 10m span with drop

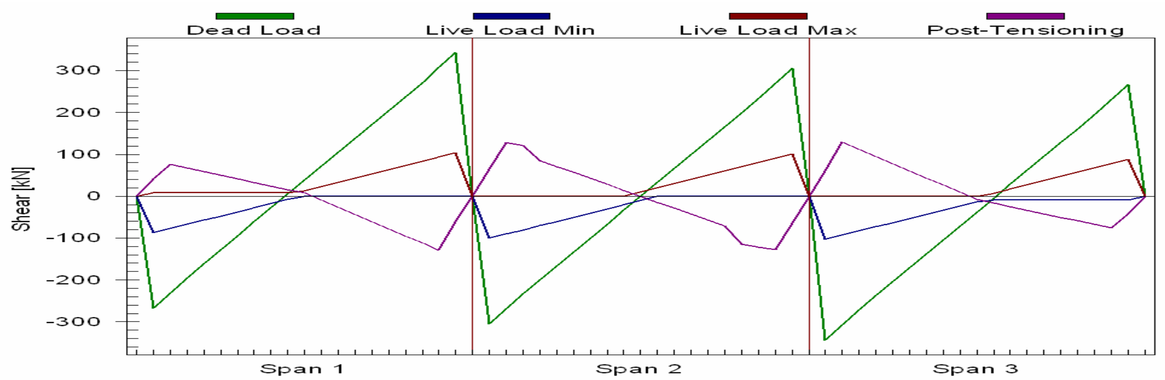


Fig. 6.73 Shear force diagram for 10m span with drop

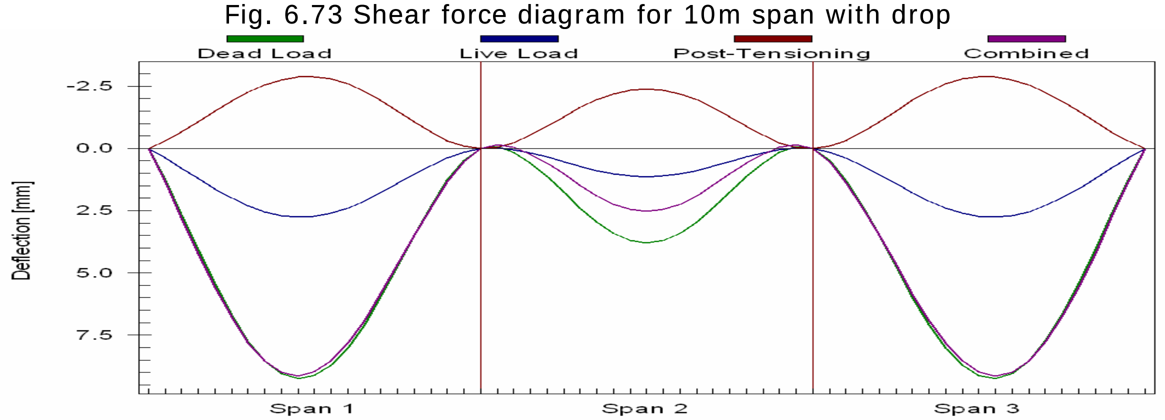


Fig. 6.74 Deflection for 10m span with drop

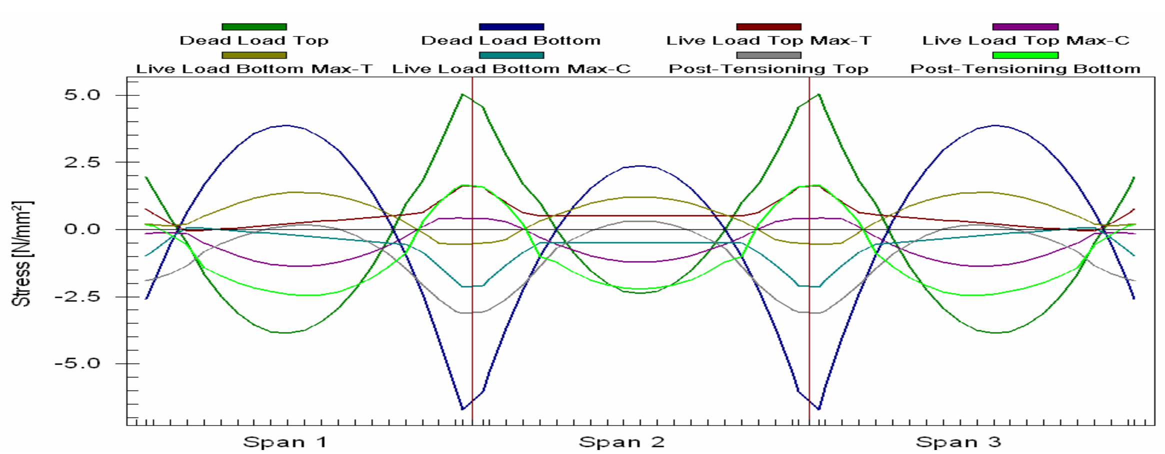


Fig. 6.75 Stress diagram for 10m span with drop

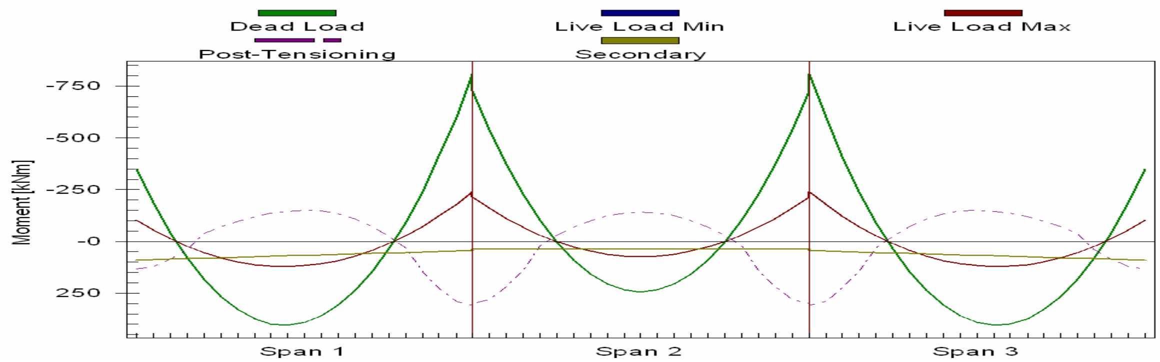


Fig. 6.76 Bending moment diagram for 10.5m span with drop

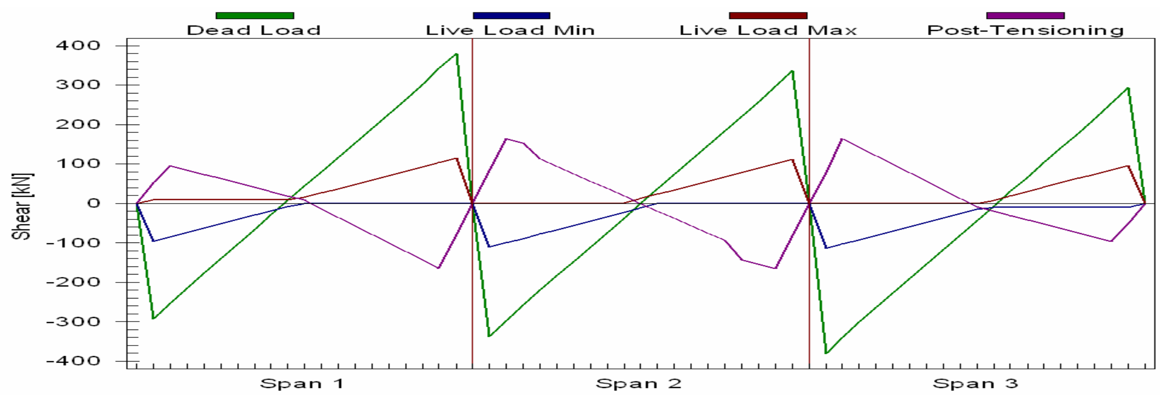


Fig. 6.77 Shear force diagram for 10.5m span with drop

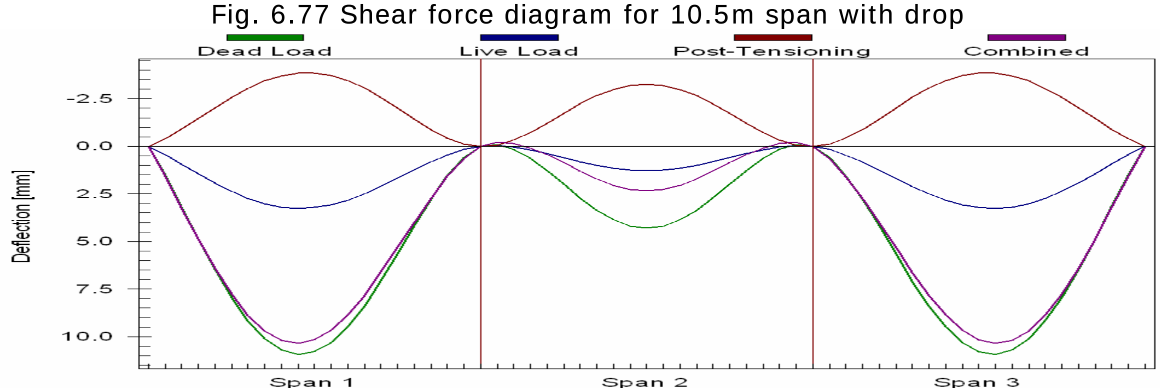


Fig. 6.78 Deflection for 10.5m span with drop

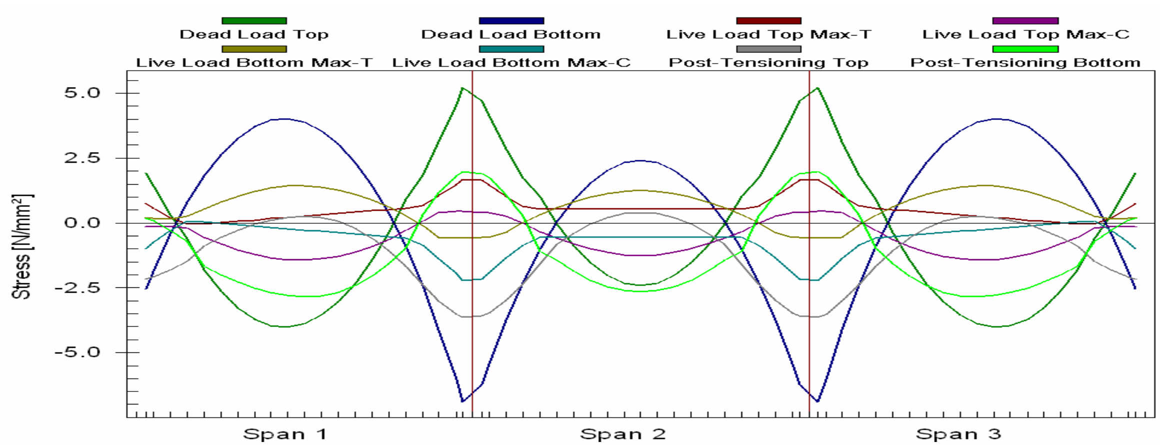


Fig. 6.79 Stress diagram for 10.5m span with drop

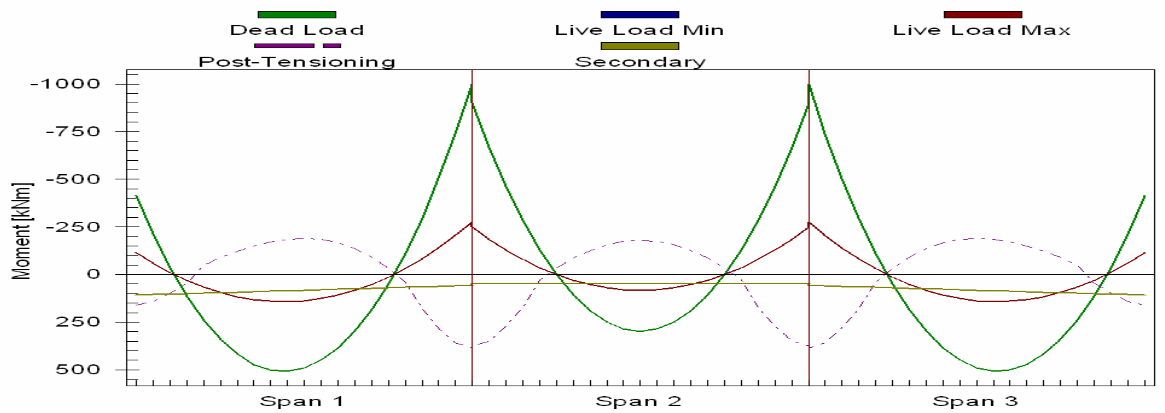


Fig. 6.80 Bending moment diagram for 11m span with drop

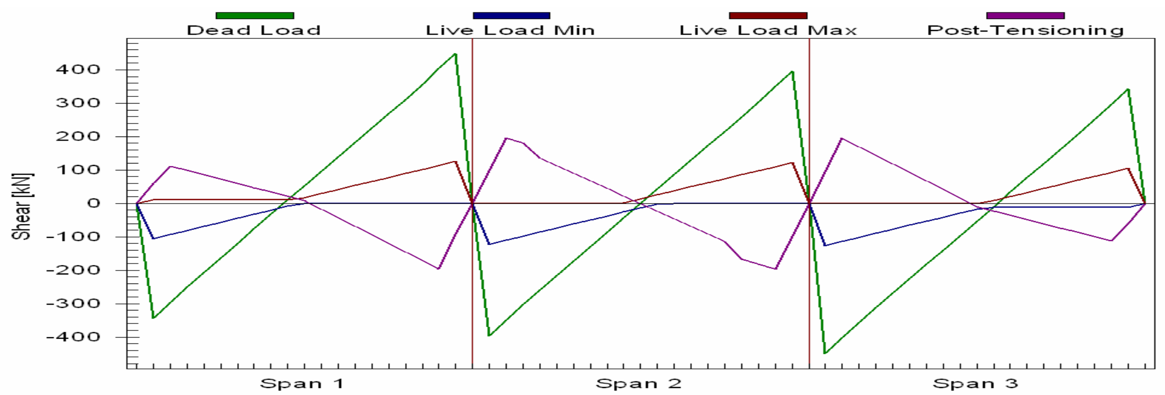


Fig. 6.81 Shear force diagram for 11m span with drop

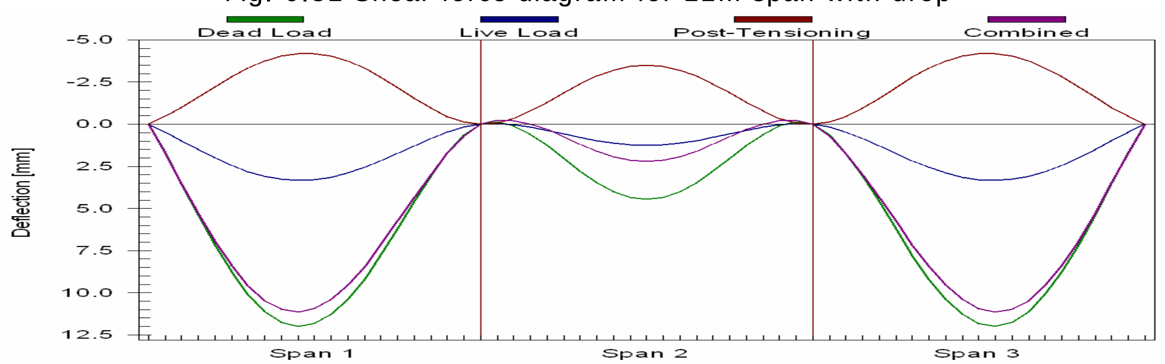


Fig. 6.82 Deflection for 11m span with drop

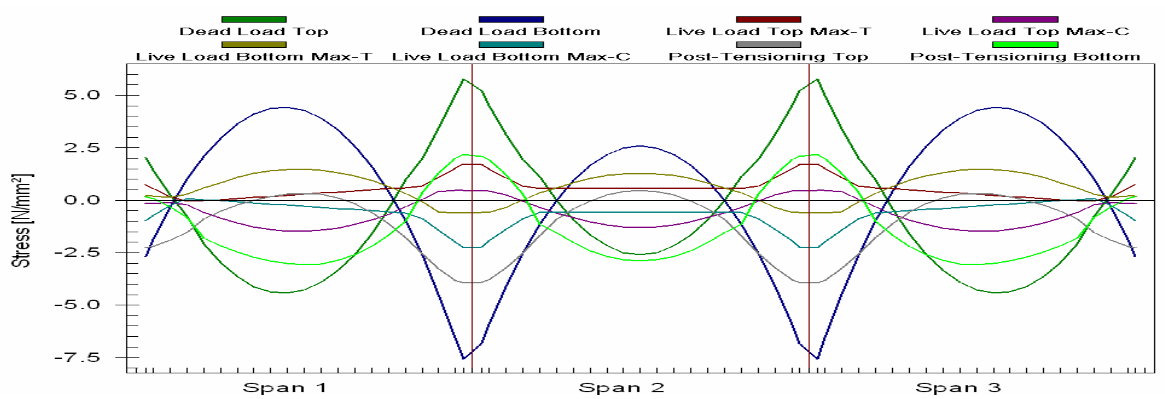


Fig. 6.83 Stress diagram for 11m span with drop

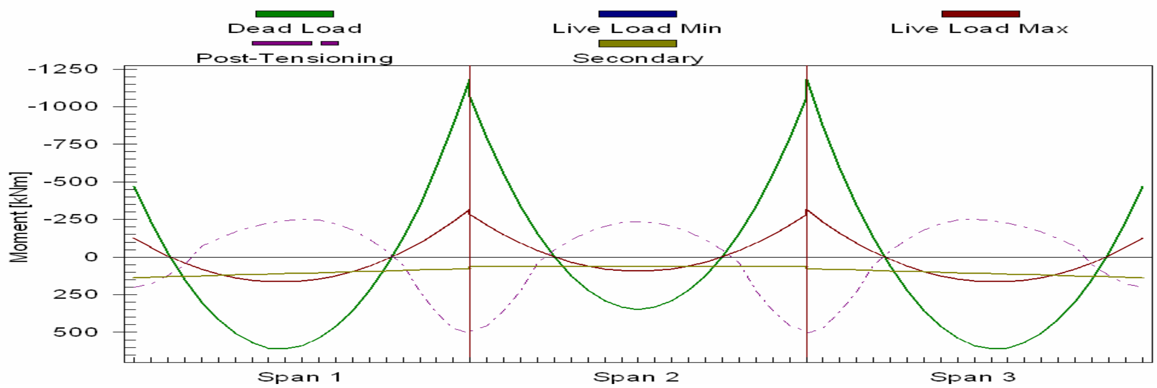


Fig. 6.84 Bending moment diagram for 11.5m span with drop

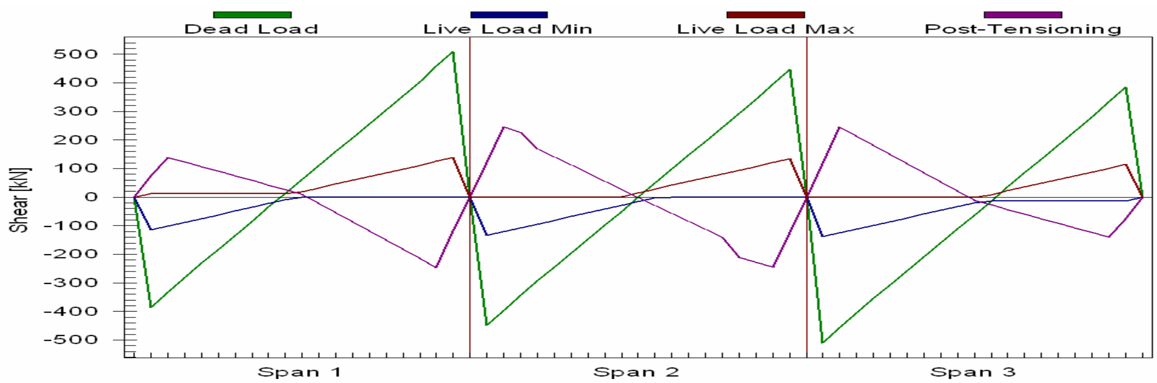


Fig. 6.85 Shear force diagram for 11.5m span with drop

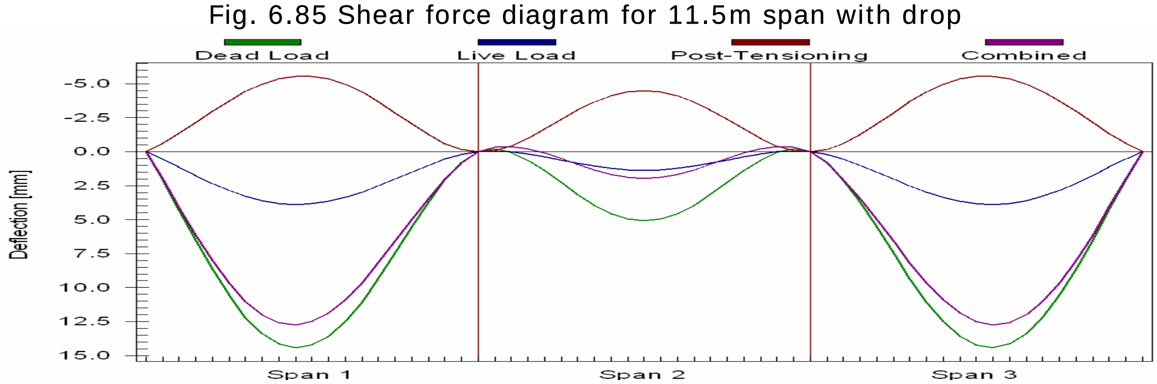


Fig. 6.86 Deflection for 11.5m span with drop

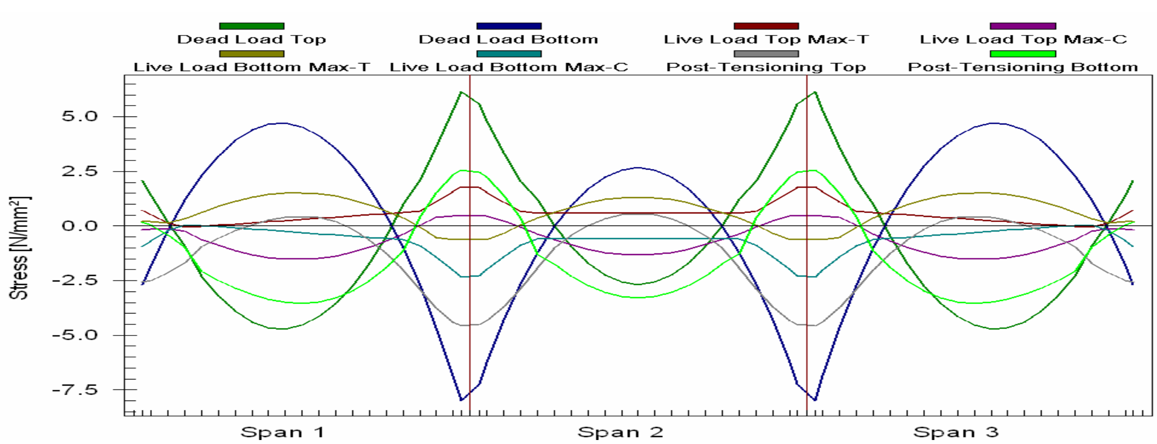


Fig. 6.87 Stress diagram for 11.5m span with drop

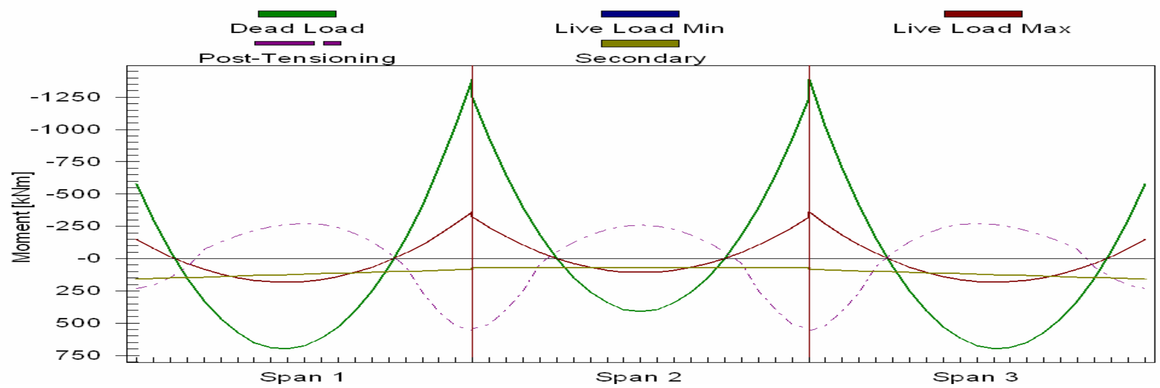


Fig. 6.88 Bending moment diagram for 12m span with drop

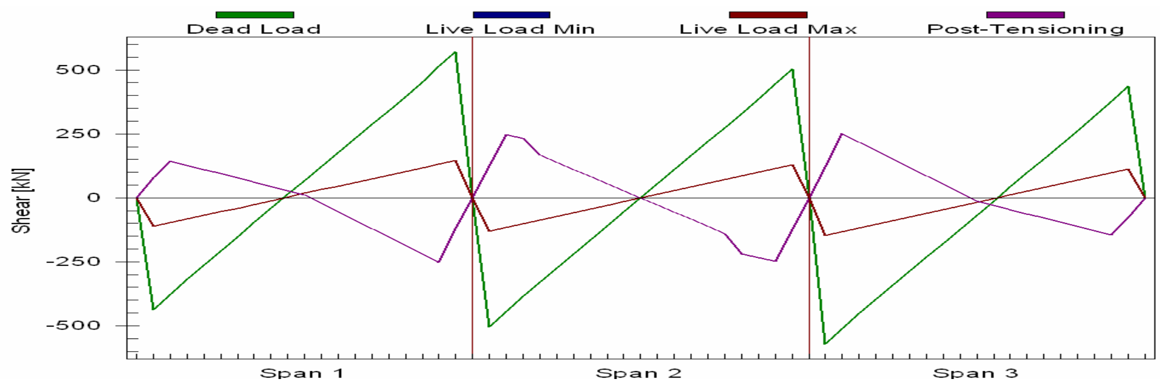


Fig. 6.89 Shear force diagram for 12m span with drop

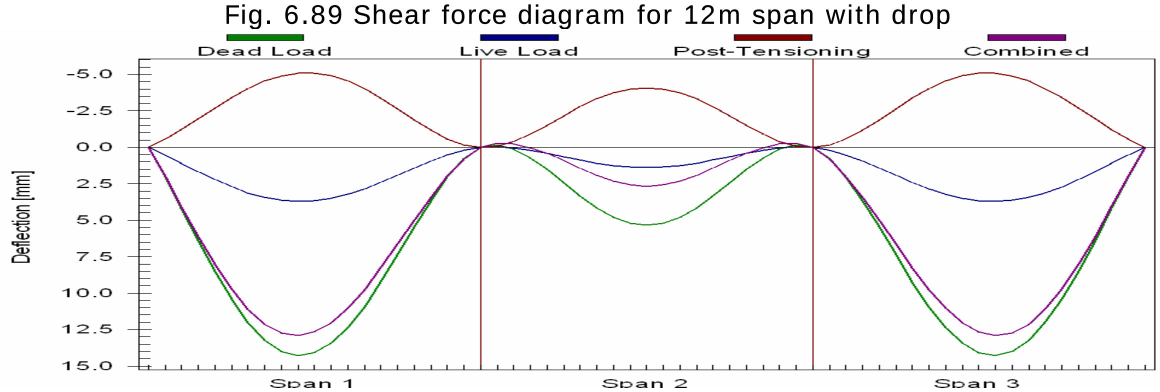


Fig. 6.90 Deflection for 12m span with drop

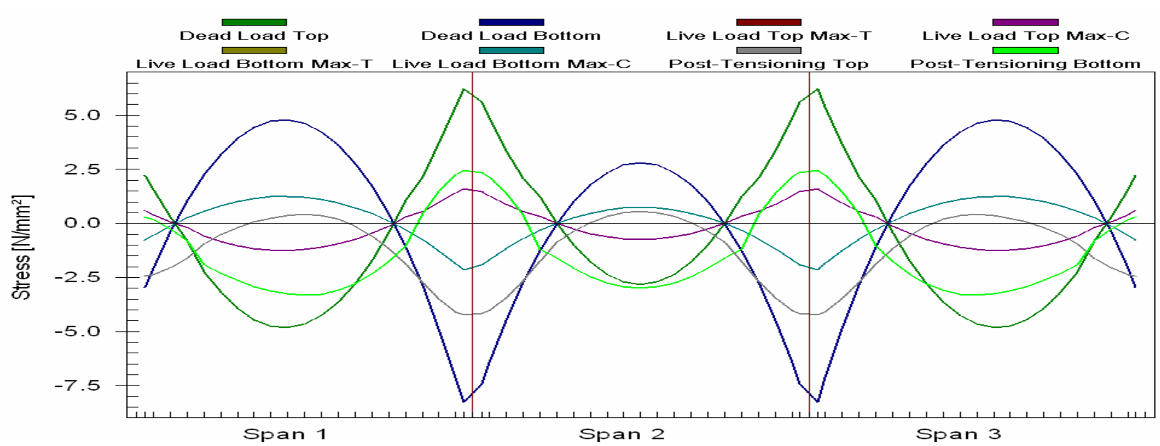


Fig. 6.91 Stress diagram for 12m span with drop

6.3 RESULTS AND DISCUSSION

The design of post-tensioned flat slab for the various spans is done and the design results are given in the table 6.1 and 6.2. Here the graphical representation of the results is given.

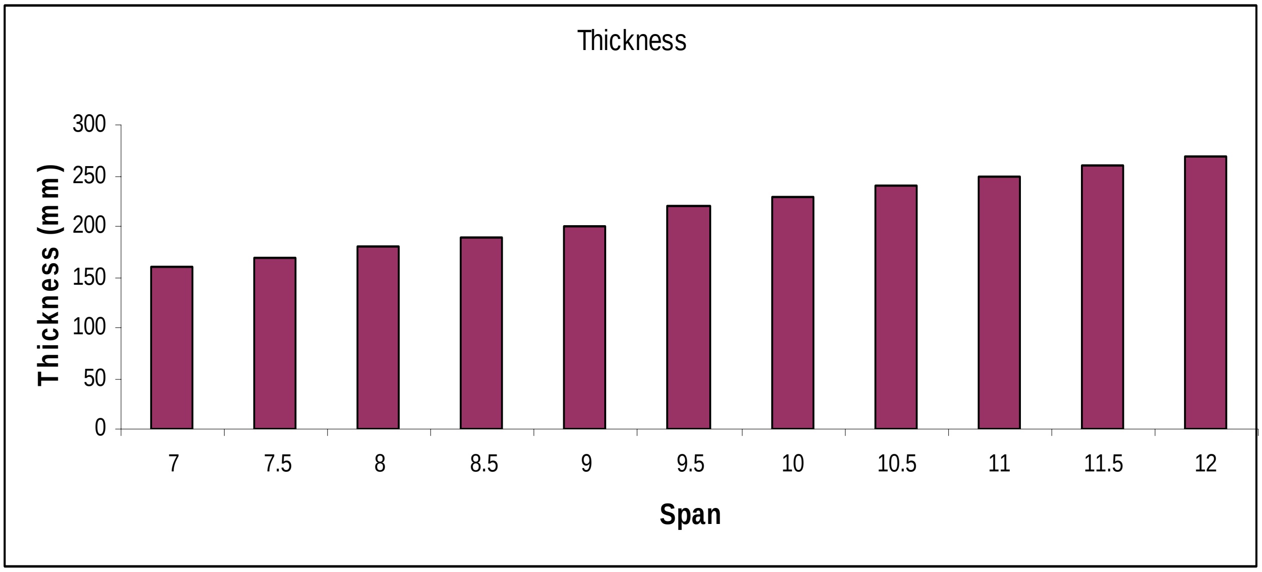


Fig 6.92 Variation of thickness for various spans

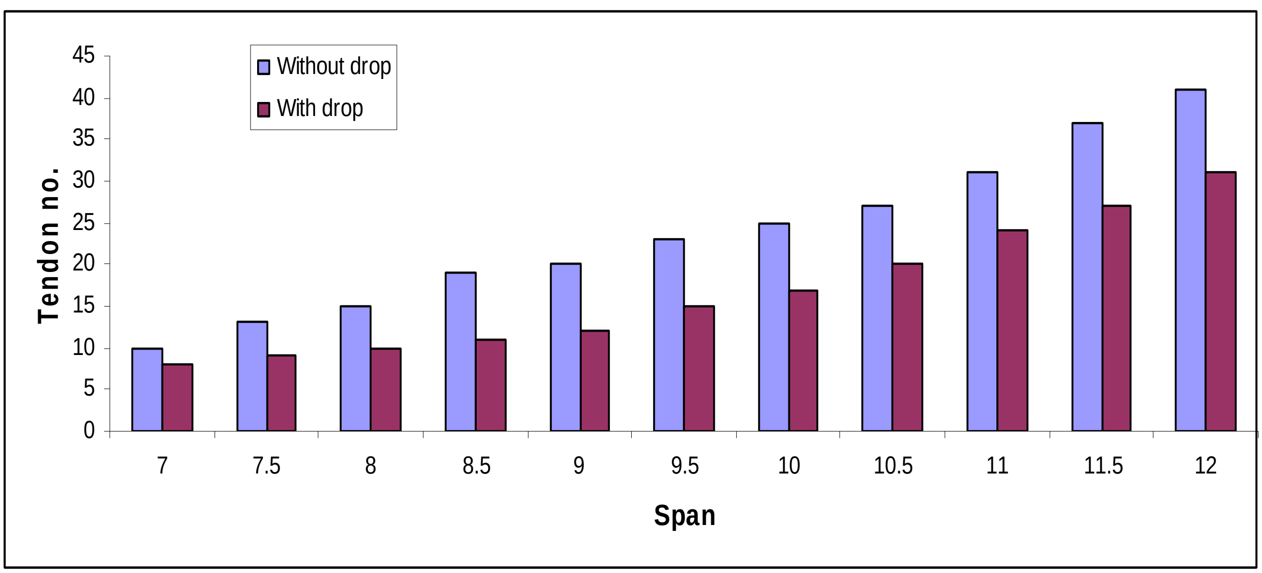


Fig 6.93 Variation of number of tendons for flat slab with and without drop

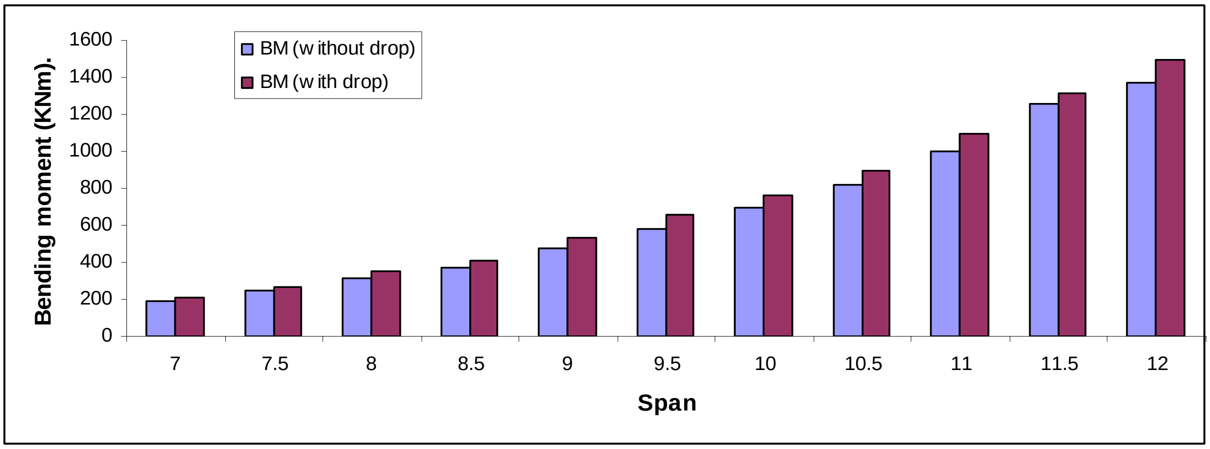


Fig 6.94 Variation of BM at support for flat slab with and without drop

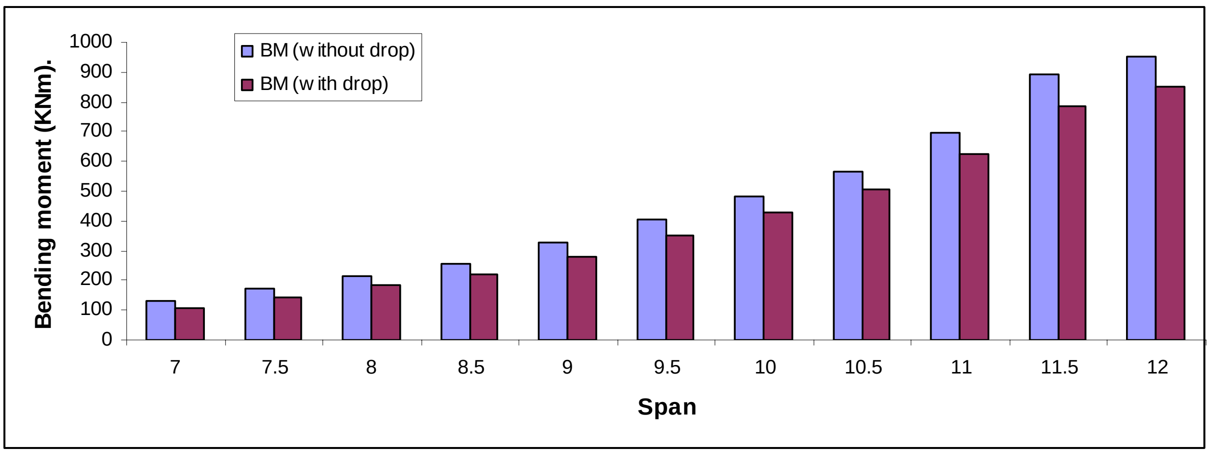


Fig 6.95 Variation of BM at midspan for flat slab with and without drop

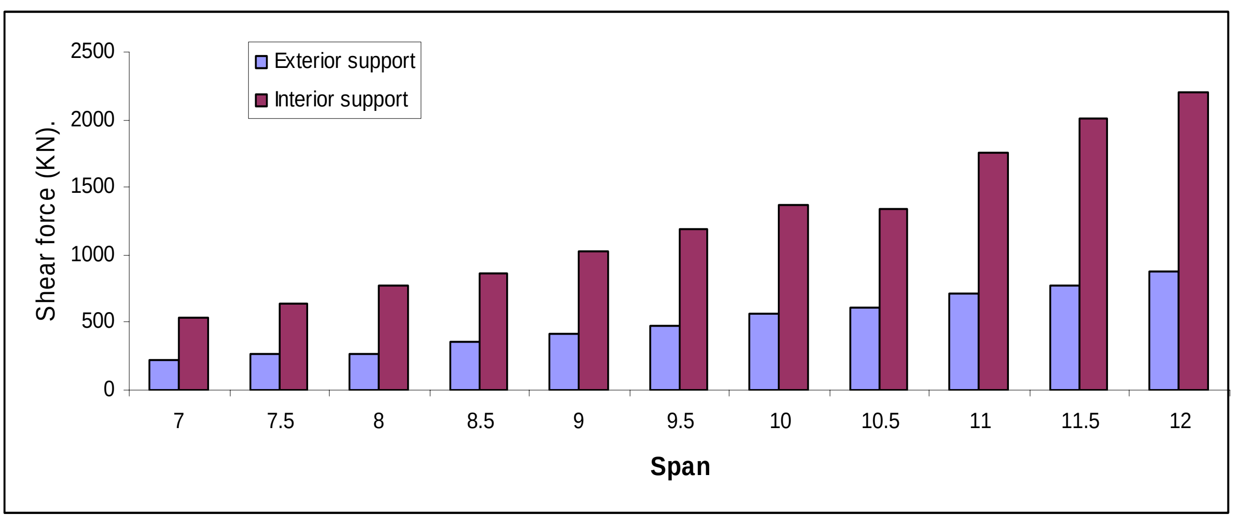


Fig 6.96 Variation of shear force at exterior and interior span

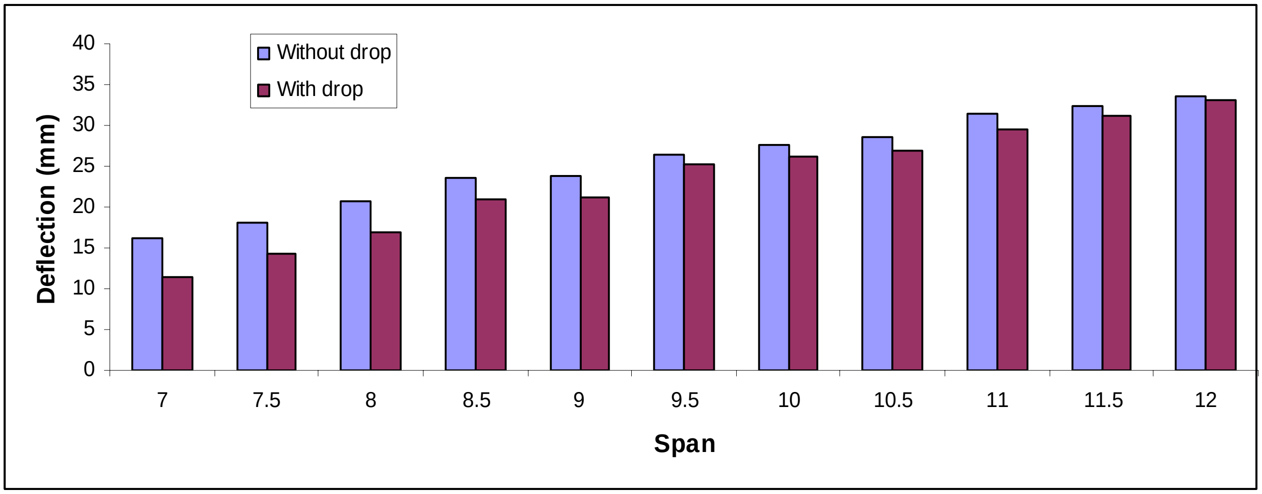


Fig 6.97 Variation of deflection for flat slab with and without drop

From the results of the design of post-tensioned flat slab for different spans the following observations are made.

1. Thickness of slab is directly proportional to the span, as span increases the depth of the slab increases (span/depth =constant).
2. The number of tendon increases with increase in span in both the cases. But the number of tendons is less in the slab with drop than the slab without drop for the same span.
3. The moment at the support is grater in case of flat slab with drop than the flat slab without drop, but the midspan moment for the flat slab with drop is less than the flat slab without drop.
4. As the moment at the mid span reduce in the flat slab with drop the number of prestressing cable reduces in the span, and moment at the support increases resulting in the increase in the normal reinforcement at the support. For the flat slab without drop panel the case is the totally inverse than that of the flat slab with drop panel.
5. The variation of the tendon in the flat slab with and without drop panel is 20 to 40% for the span range of 7m to 12m.
6. The allowable deflection limit is span/325 according to the IS 456: 2000, but the calculated deflection is span/350 to span/450 for the flat slab without drop panel and span/350 to span/610 for the flat slab with drop panel i.e. the

deflection is less in case of slab with drop compared to the slab without drop panel.

7. The reinforcement at the exterior and the interior support increases with increase in span. It is less in case of the flat slab with drop panel.

7. A CASE STUDY

7.1 INTRODUCTION

The parametric study of the post-tensioned flat with and without drop panel is carried out in the previous chapter. The design procedure is explained in the appendix A which we have to use while designing any post-tensioned building. For the application of design procedure a office building is consider as a case study. The plan of the office building (G+4) is as shown in fig. 7.1. This building is designed by considering four cases with different floor systems. The different floor systems used for these four cases are as follows

Case 1: Post-tensioned flat slab (fig. 7.2)

Case 2: Reinforced concrete flat slab (fig. 7.2)

Case 3: Post-tensioned slab with reinforced concrete beams (fig. 7.3)

Case 4: Reinforced concrete slab with reinforced concrete beams (fig.7.4).

This building is to be constructed at Baner, Pune. The live load for this building is 4KN/m^2 (as per IS 875 Part II) and the superimposed dead load is 2KN/m^2. The dead load is only the self weight of the building; there is no special loading on the building. The analysis and design of the building is done in the following sections.

For the above four cases the quantities of reinforcing steel, prestressing steel, concrete required for the slab, beam and column is calculated and are presented in the tabular form. Along with this a total cost of the building per square meter is found and the comparison of all the four cases with respect to cost is given here.

7.2 ANALYSIS

The analysis of all the four cases is done by using the software (ADAPT and STAAD PRO 2005) and the analysis results (bending moment at left support, right support and the mid span, shear forces at right and left support for the beams, the axial force, moment in X and Y direction for the column, moments in the grid consider for slab etc.) are tabulated in the following tables. The tables are formed separately for each case.

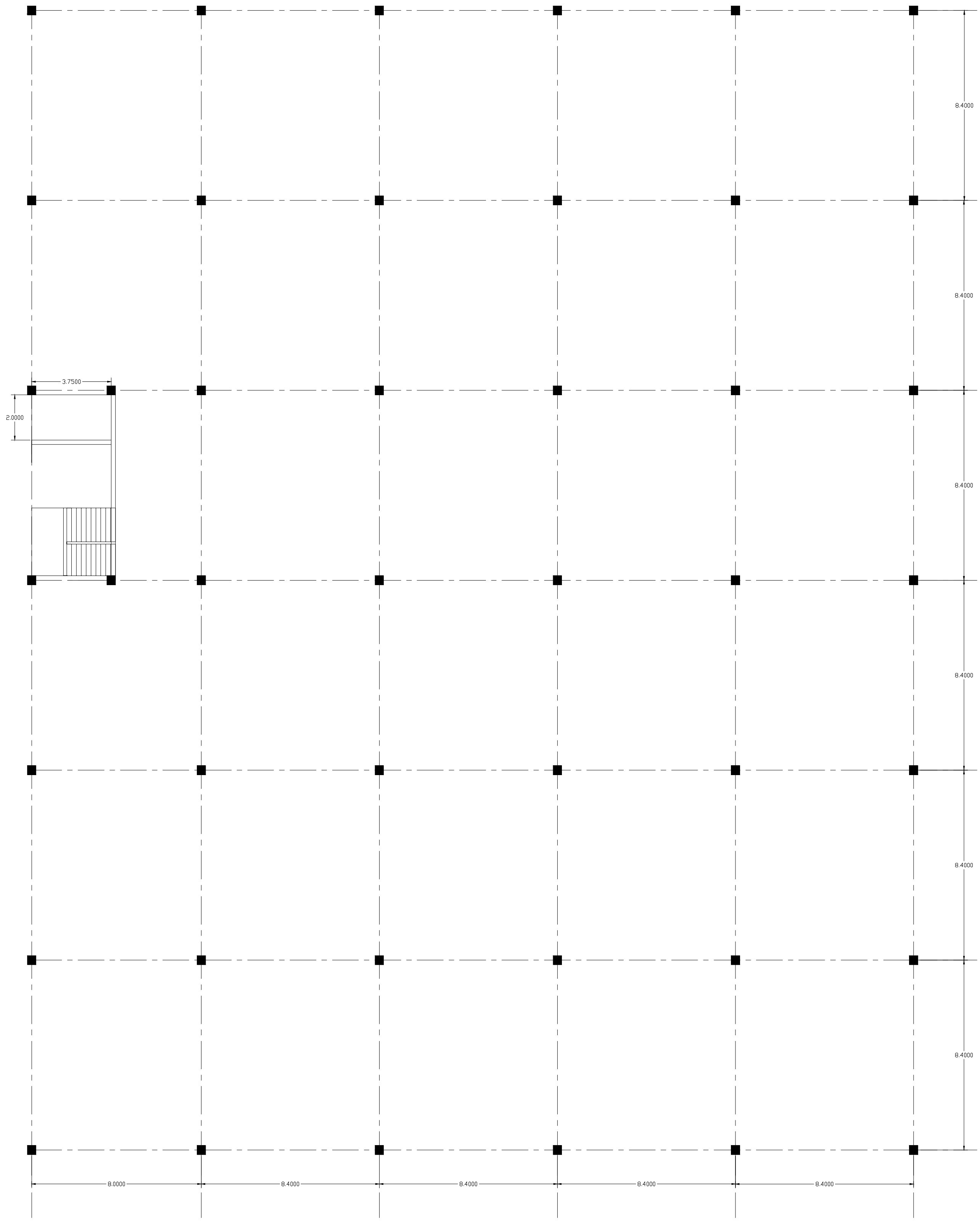


Fig. 7.1 Plan of the office building (G+4)

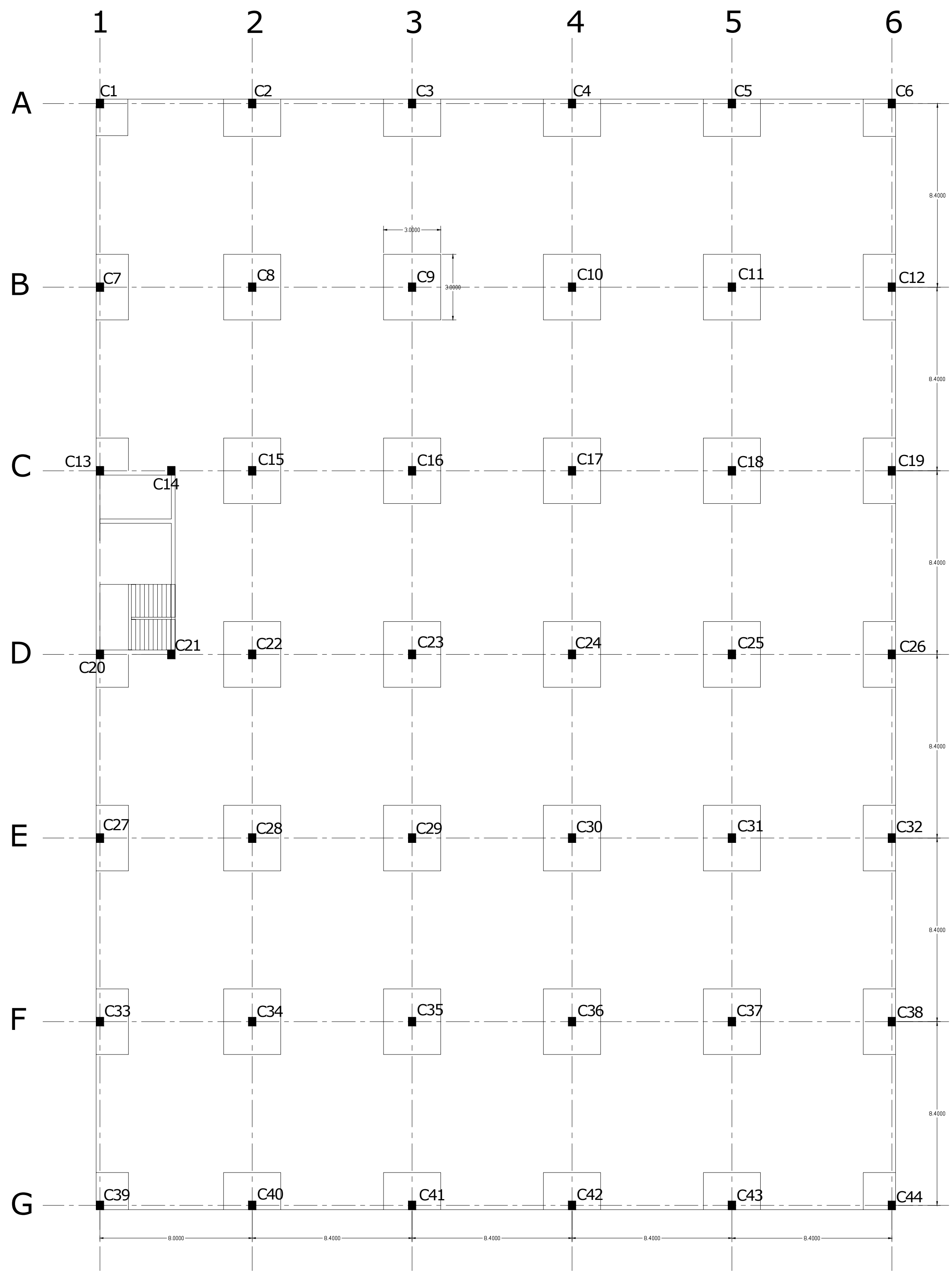


Fig.7.2 Floor system for the Post-tensioned flat slab and Reinforced concrete flat slab

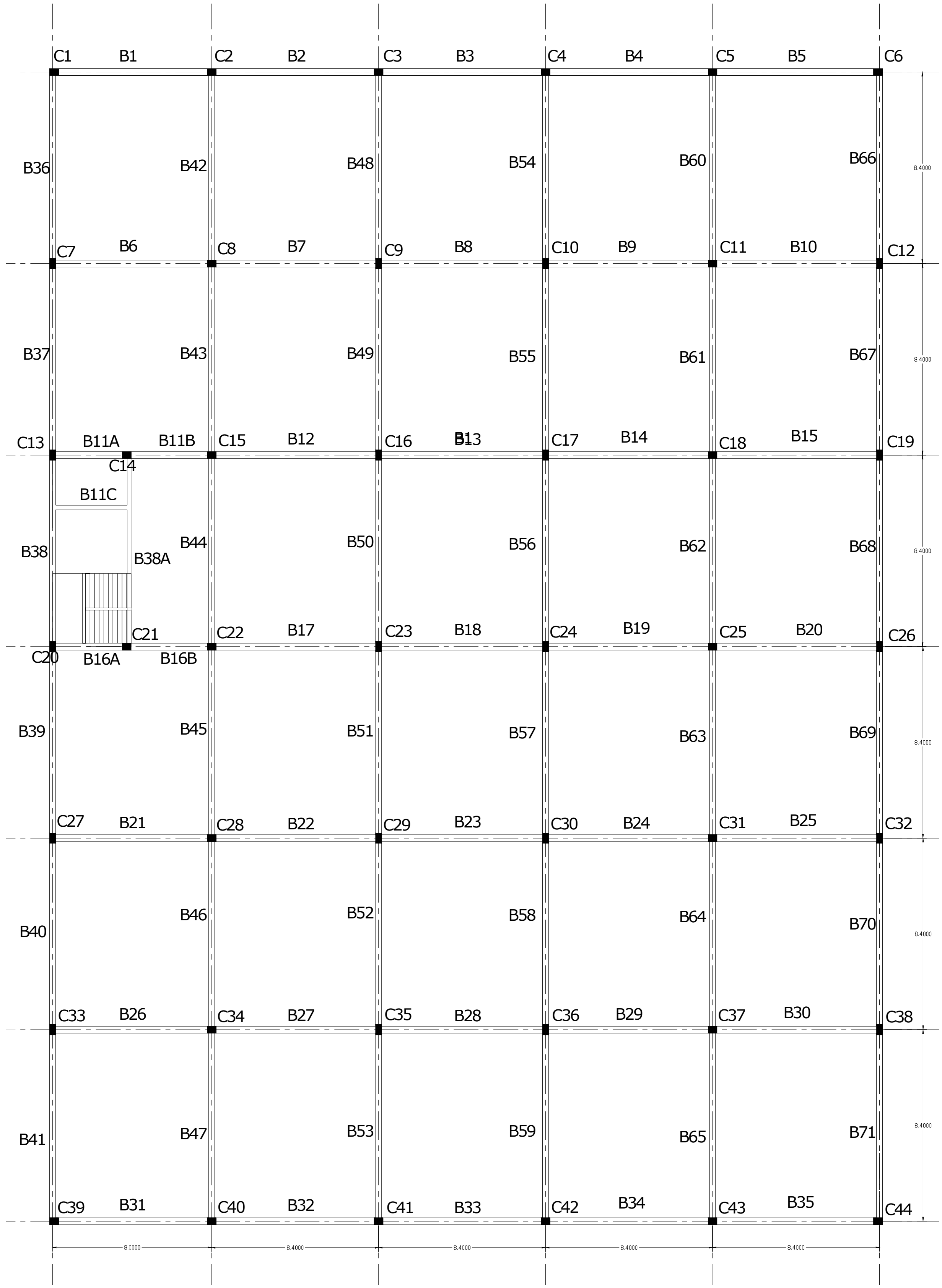


Fig 7.3 Floor system for post-tensioned slab with RCC beams

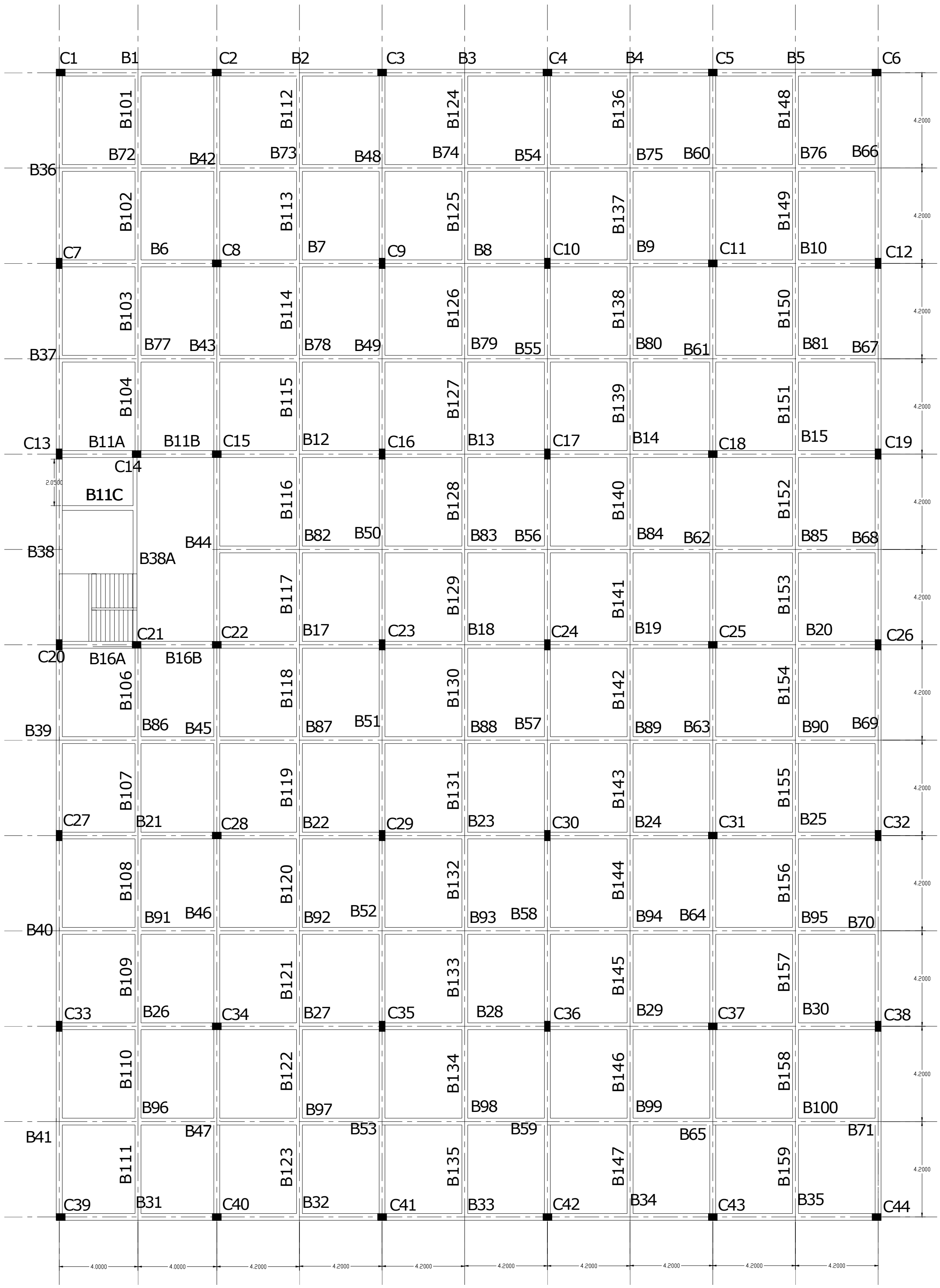


Fig. 7.4 Floor system for RCC slab With RCC beams

CASE 1 POST-TENSIONED FLAT SLAB

The moment and the shear forces in the each grid of the slab, moments and shear forces in beams at plinth level and at typical floor, axial forces and the moments in the column for the post-tensioned flat slab are tabulated in the table 7.1 to 7.8. The floor system for the post-tensioned flat slab is as shown in fig. 7.2.

CASE2 REINFOCED CONCRETE FLAT SLAB

The moment and the shear forces in the each grid of the slab, moments and shear forces in beams at plinth level and at typical floor, axial forces and the moments in the column for the reinforced concrete flat slab are same as that of post-tensioned flat slab which are tabulated in the table 7.1 to 7.8. The floor system used for the reinforced concrete flat slab is shown in fig. 7.2.

CASE 3 POST-TENSIONED SLAB WITH REINFORCED CONCRETE BEAMS

The moments and shear forces in beams at plinth level and the beams at each floor level, axial forces and moments in the column at each floor level are tabulated in the table 7.9 to 7.10. The floor system is as shown in fig. 7.3.

CASE 4 REINFORCED CONCRETE SLAB WITH REINFORCED CONCRETE BEAMS

The floor system used for the reinforced concrete slab and reinforced concrete beams is as shown in fig. 7.4. The moment and shear forces in the beam and the axial forces and moments in the column are tabulated in the table 7.11 to 7.12

7.3 DESIGN

The design of post-tensioned flat slab, reinforced concrete flat slab, reinforced concrete continuous beams, column subjected to axial force and biaxial moment, post-tensioned slab, one way slab, two way slab etc. is carried out from the analysis results. The design of each case is done in the following sections.

Table 7.1 Moments and Shear forces in PT flat slab at first floor

Span			Grid 1	Grid 2	Grid 3,4,5	Grid 6	Grid A,G	Grid B,E,F	Grid C,D
1	Moment (KNm)	Left	-293.92	-338.82	-383.71	-293.92	-299.01	-388.80	-467.47
		Mid	95.58	160.70	225.82	95.58	95.58	225.82	167.30
		Right	-307.77	-459.78	-611.78	-307.77	-160.09	-464.10	-271.68
	Shear (KN)	Left	-272.12	-383.44	-494.76	-272.12	-281.19	-503.83	-353.18
		Right	217.57	329.63	441.68	217.57	221.75	445.86	347.11
2	Moment (KNm)	Left	-303.77	-430.68	-557.58	-303.77	-156.09	-409.90	-409.90
		Mid	94.09	138.55	183.01	94.09	94.09	183.01	183.01
		Right	-304.21	-415.55	-526.88	-304.21	-309.58	-532.25	-532.25
	Shear (KN)	Left	-216.24	-316.20	-416.16	-216.24	-220.42	-420.34	-420.34
		Right	216.47	312.34	408.20	216.47	220.42	412.15	412.15
3	Moment (KNm)	Left	-465.75	-419.26	-534.28	-304.24	-309.61	-544.55	-544.55
		Mid	167.30	141.47	188.81	94.12	94.12	188.81	188.81
		Right	-420.42	-422.17	-539.64	-304.69	-309.82	-539.87	-539.87
	Shear (KN)	Left	-342.03	-314.09	-411.68	-216.50	-220.45	-416.91	-416.91
		Right	343.33	314.85	413.09	216.61	220.34	415.53	415.53
4	Moment (KNm)	Left	-94.70	-422.17	-539.64	-304.70	-309.81	-532.47	-532.47
		Mid	94.12	141.47	188.81	94.12	94.09	183.01	183.01
		Right	-304.99	-420.02	-535.04	-304.99	-309.64	-563.45	-563.45
	Shear (KN)	Left	-216.62	-314.85	-413.08	-216.62	-220.32	-412.05	-412.05
		Right	216.57	314.17	411.76	216.57	219.73	419.65	419.65
5	Moment (KNm)	Left	-304.98	-416.31	-527.64	-304.98	-313.65	-617.65	-617.65
		Mid	94.09	138.55	183.01	94.09	95.58	2225.82	2225.82
		Right	-305.32	-432.23	-559.13	-305.32	-299.50	-389.30	-389.30
	Shear (KN)	Left	-216.55	-312.42	-408.28	-216.55	-221.17	-445.19	-445.19
		Right	216.38	316.34	416.30	216.38	281.18	503.83	503.83
6	Moment (KNm)	Left	-309.33	-461.33	-613.33	-309.33	0.00	0.00	0.00
		Mid	95.58	1160.70	2225.82	95.58	0.00	0.00	0.00
		Right	-296.07	-340.97	-385.87	-296.07	0.00	0.00	0.00
	Shear (KN)	Left	-217.82	-329.83	-441.84	-217.82	0.00	0.00	0.00
		Right	272.11	383.44	494.76	272.11	0.00	0.00	0.00

Table 7.2 Moments and Shear forces in PT flat slab at second floor

Span			Grid 1	Grid 2	Grid 3,4,5	Grid 6	Grid A,G	Grid B,E,F	Grid C,D
1	Moment (KNm)	Left	-262.31	-307.21	-352.10	-262.31	-264.81	-354.60	-433.27
		Mid	95.58	160.70	225.82	95.58	95.58	225.82	167.30
		Right	-283.25	-435.26	-587.26	-283.25	-289.45	-593.46	-401.04
	Shear (KN)	Left	-269.88	-381.20	-492.52	-269.88	-274.75	-497.39	-346.74
		Right	218.33	330.39	442.44	218.33	223.48	447.59	348.84
2	Moment (KNm)	Left	-279.25	-406.16	-533.06	-279.25	-285.45	-539.26	-539.26
		Mid	94.09	138.55	183.01	94.09	94.09	183.01	183.01
		Right	-279.22	-390.56	-501.89	-279.22	-284.38	-507.05	-507.05
	Shear (KN)	Left	-217.00	-316.96	-416.92	-217.00	-222.15	-422.07	-422.07
		Right	216.94	312.81	408.67	216.94	221.20	412.93	412.93
3	Moment (KNm)	Left	-440.76	-394.27	-509.29	-279.25	-284.41	-519.35	-519.35
		Mid	167.30	141.47	188.81	94.12	94.12	188.81	188.81
		Right	-395.34	-397.09	-514.56	-279.61	-284.97	-515.02	-515.02
	Shear (KN)	Left	-342.50	-314.56	-412.15	-216.97	-221.23	-417.69	-417.69
		Right	343.54	315.06	413.30	216.82	221.36	416.55	416.55
4	Moment (KNm)	Left	-69.62	-397.09	-514.56	-279.62	-284.96	-507.62	-507.62
		Mid	94.12	141.47	188.81	94.12	94.09	183.01	183.01
		Right	-280.35	-395.38	-510.40	-280.35	-285.05	-538.86	-538.86
	Shear (KN)	Left	-216.83	-315.06	-413.29	-216.83	-221.34	-413.07	-413.07
		Right	216.96	314.56	412.15	216.96	220.85	420.77	420.77
5	Moment (KNm)	Left	-280.34	-391.67	-503.00	-280.34	-289.06	-593.06	-593.06
		Mid	94.09	138.55	183.01	94.09	95.58	2225.82	2225.82
		Right	-281.49	-408.40	-535.30	-281.49	-265.15	-354.95	-354.95
	Shear (KN)	Left	-216.94	-312.81	-408.67	-216.94	-222.29	-446.31	-446.31
		Right	217.39	317.35	417.31	217.39	274.74	497.39	497.39
6	Moment (KNm)	Left	-285.50	-437.50	-589.50	-285.50	0.00	0.00	0.00
		Mid	95.58	1160.70	2225.82	95.58	0.00	0.00	0.00
		Right	-262.30	-307.20	-352.10	-262.30	0.00	0.00	0.00
	Shear (KN)	Left	-218.83	-330.84	-442.85	-218.83	0.00	0.00	0.00
		Right	269.87	381.20	492.52	269.87	0.00	0.00	0.00

Table 7.3 Moments and Shear forces in PT flat slab at third floor

Span			Grid 1	Grid 2	Grid 3,4,5	Grid 6	Grid A,G	Grid B,E,F	Grid C,D
1	Moment (KNm)	Left	-244.09	-288.99	-333.88	-244.09	-248.85	-338.64	-417.31
		Mid	95.58	160.70	225.82	95.58	95.58	225.82	167.30
		Right	-266.76	-418.77	-570.77	-266.76	-273.03	-577.04	-384.62
	Shear (KN)	Left	-263.93	-375.25	-486.57	-263.93	-268.45	-491.09	-340.44
		Right	217.62	329.68	441.73	217.62	222.18	446.29	347.54
2	Moment (KNm)	Left	-262.76	-389.67	-516.57	-262.76	-269.03	-522.84	-522.84
		Mid	94.09	138.55	183.01	94.09	94.09	183.01	183.01
		Right	-262.40	-373.74	-485.07	-262.40	-268.87	-491.54	-491.54
	Shear (KN)	Left	-216.29	-316.25	-416.21	-216.29	-220.85	-420.77	-420.77
		Right	216.01	311.88	407.74	216.01	220.08	411.81	411.81
3	Moment (KNm)	Left	-423.94	-377.45	-492.47	-262.43	-268.90	-503.84	-503.84
		Mid	167.30	141.47	188.81	94.12	94.12	188.81	188.81
		Right	-377.90	-379.65	-497.12	-262.17	-268.65	-498.70	-498.70
	Shear (KN)	Left	-341.57	-313.63	-411.22	-216.04	-220.11	-416.57	-416.57
		Right	342.57	314.09	412.33	215.85	219.96	415.15	415.15
4	Moment (KNm)	Left	-52.18	-379.65	-497.12	-262.18	-268.64	-491.30	-491.30
		Mid	94.12	141.47	188.81	94.12	94.09	183.01	183.01
		Right	-261.77	-376.80	-491.82	-261.77	-268.88	-522.69	-522.69
	Shear (KN)	Left	-215.86	-314.09	-412.32	-215.86	-219.94	-411.67	-411.67
		Right	216.03	313.63	411.22	216.03	219.39	419.31	419.31
5	Moment (KNm)	Left	-261.76	-373.09	-484.42	-261.76	-272.89	-576.89	-576.89
		Mid	94.09	138.55	183.01	94.09	95.58	2225.82	2225.82
		Right	-261.44	-388.35	-515.25	-261.44	-248.84	-338.64	-338.64
	Shear (KN)	Left	-216.01	-311.88	-407.74	-216.01	-220.83	-444.85	-444.85
		Right	216.29	316.25	416.21	216.29	268.44	491.09	491.09
6	Moment (KNm)	Left	-265.45	-417.45	-569.45	-265.45	0.00	0.00	0.00
		Mid	95.58	1160.70	2225.82	95.58	0.00	0.00	0.00
		Right	-244.08	-288.98	-333.88	-244.08	0.00	0.00	0.00
	Shear (KN)	Left	-217.73	-329.74	-441.75	-217.73	0.00	0.00	0.00
		Right	263.92	375.25	486.57	263.92	0.00	0.00	0.00

Table 7.4 Moments and Shear forces in PT flat slab at fourth floor

Span			Grid 1	Grid 2	Grid 3,4,5	Grid 6	Grid A,G	Grid B,E,F	Grid C,D
1	Moment (KNm)	Left	-245.31	-290.21	-335.10	-245.31	-250.55	-340.34	-419.01
		Mid	95.58	160.70	225.82	95.58	95.58	225.82	167.30
		Right	-268.49	-420.50	-572.50	-268.49	-275.85	-579.86	-387.44
	Shear (KN)	Left	-258.10	-369.42	-480.74	-258.10	-260.26	-482.90	-332.25
		Right	214.93	326.99	439.04	214.93	218.63	442.74	343.99
2	Moment (KNm)	Left	-264.49	-391.40	-518.30	-264.49	-271.85	-525.66	-525.66
		Mid	94.09	138.55	183.01	94.09	94.09	183.01	183.01
		Right	-264.63	-375.97	-487.30	-264.63	-271.23	-493.90	-493.90
	Shear (KN)	Left	-213.60	-313.56	-413.52	-213.60	-217.30	-417.22	-417.22
		Right	213.15	309.02	404.88	213.15	216.46	408.19	408.19
3	Moment (KNm)	Left	-426.17	-379.68	-494.70	-264.66	-271.26	-506.20	-506.20
		Mid	167.30	141.47	188.81	94.12	94.12	188.81	188.81
		Right	-380.55	-382.30	-499.77	-264.82	-271.19	-501.24	-501.24
	Shear (KN)	Left	-338.71	-310.77	-408.36	-213.18	-216.49	-412.95	-412.95
		Right	339.43	310.95	409.19	212.71	216.11	411.30	411.30
4	Moment (KNm)	Left	-54.83	-382.30	-499.77	-264.83	-271.18	-493.84	-493.84
		Mid	94.12	141.47	188.81	94.12	94.09	183.01	183.01
		Right	-264.71	-379.74	-494.76	-264.71	-269.31	-523.12	-523.12
	Shear (KN)	Left	-212.72	-310.95	-409.18	-212.72	-216.09	-407.82	-407.82
		Right	213.17	310.77	408.36	213.17	215.94	415.86	415.86
5	Moment (KNm)	Left	-264.70	-376.03	-487.36	-264.70	-273.32	-577.32	-577.32
		Mid	94.09	138.55	183.01	94.09	95.58	2225.82	2225.82
		Right	-264.08	-390.99	-517.89	-264.08	-250.39	-340.19	-340.19
	Shear (KN)	Left	-213.15	-309.02	-404.88	-213.15	-217.38	-441.40	-441.40
		Right	213.60	313.56	413.52	213.60	260.25	482.90	482.90
6	Moment (KNm)	Left	-268.09	-420.09	-572.09	-268.09	0.00	0.00	0.00
		Mid	95.58	1160.70	2225.82	95.58	0.00	0.00	0.00
		Right	-245.76	-290.66	-335.56	-245.76	0.00	0.00	0.00
	Shear (KN)	Left	-215.04	-327.05	-439.06	-215.04	0.00	0.00	0.00
		Right	257.81	369.14	480.46	257.81	0.00	0.00	0.00

Table 7.5 Moments and Shear forces in PT flat slab at fifth floor

Span			Grid 1	Grid 2	Grid 3,4,5	Grid 6	Grid A,G	Grid B,E,F	Grid C,D
1	Moment (KNm)	Left	-226.11	-271.01	-315.90	-226.11	-239.02	-328.81	-407.48
		Mid	95.58	160.70	225.82	95.58	95.58	225.82	167.30
		Right	-237.19	-389.20	-541.20	-237.19	-261.02	-565.03	-372.61
	Shear (KN)	Left	-249.37	-360.69	-472.01	-249.37	-257.12	-479.76	-329.11
		Right	197.18	309.24	421.29	197.18	211.69	435.80	337.05
2	Moment (KNm)	Left	-233.19	-360.10	-487.00	-233.19	-257.02	-510.83	-510.83
		Mid	94.09	138.55	183.01	94.09	94.09	183.01	183.01
		Right	-233.03	-344.37	-455.70	-233.03	-255.09	-477.76	-477.76
	Shear (KN)	Left	-195.85	-295.81	-395.77	-195.85	-210.36	-410.28	-410.28
		Right	195.80	291.67	387.53	195.80	208.58	400.31	400.31
3	Moment (KNm)	Left	-394.57	-348.08	-463.10	-233.06	-255.12	-490.06	-490.06
		Mid	167.30	141.47	188.81	94.12	94.12	188.81	188.81
		Right	-348.78	-350.53	-468.00	-233.05	-255.18	-485.23	-485.23
	Shear (KN)	Left	-321.36	-293.42	-391.01	-195.83	-208.61	-405.07	-405.07
		Right	322.54	294.06	392.30	195.82	208.66	403.85	403.85
4	Moment (KNm)	Left	-23.06	-350.53	-468.00	-233.06	-255.17	-477.83	-477.83
		Mid	94.12	141.47	188.81	94.12	94.09	183.01	183.01
		Right	-233.05	-348.08	-463.10	-233.05	-255.50	-509.31	-509.31
	Shear (KN)	Left	-195.83	-294.06	-392.29	-195.83	-208.64	-400.37	-400.37
		Right	195.82	293.42	391.01	195.82	209.35	409.27	409.27
5	Moment (KNm)	Left	-233.04	-344.37	-455.70	-233.04	-259.51	-563.51	-563.51
		Mid	94.09	138.55	183.01	94.09	95.58	2225.82	2225.82
		Right	-233.19	-360.10	-487.00	-233.19	-238.40	-328.20	-328.20
	Shear (KN)	Left	-195.80	-291.67	-387.53	-195.80	-210.79	-434.81	-434.81
		Right	195.85	295.81	395.77	195.85	256.99	479.64	479.64
6	Moment (KNm)	Left	-237.20	-389.20	-541.20	-237.20	0.00	0.00	0.00
		Mid	95.58	1160.70	2225.82	95.58	0.00	0.00	0.00
		Right	-226.10	-271.00	-315.90	-226.10	0.00	0.00	0.00
	Shear (KN)	Left	-197.29	-309.30	-421.31	-197.29	0.00	0.00	0.00
		Right	249.36	360.69	472.01	249.36	0.00	0.00	0.00

Table 7.6 Moment and Axial forces in the column of PT flat slab building

Floor	Upto plinth level			Upto first floor			Upto second floor		
Column no.	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)
C1,C6, C39,C44	2129.14	132.22	115.2	1905.02	185.2	161.29	1568.8	197.89	172.85
C2,C3,C4, C5,C40,C41, C42,C43	3060.6	150.47	90.69	2652.52	255.51	111.54	2040.4	154.14	101.15
C7,C12,C33, C38,C27,C32	3197.7	37.625	106.6	2771.34	65.475	153.048	2131.8	59.885	80.198
C8,C9,C10, C11,C28,C34, C35,C36,C37	6338.4	40.336	44.87	5493.28	70.586	86.479	4225.6	65.396	76.789
C13,C20	3707.4	51.852	105.7	3213.08	72.662	132.715	2471.6	70.762	131.7
C14,C21	2634.75	104.64	71.18	2283.45	98.97	128.06	1756.5	154.03	89
C15,C22	5953.5	50.44	44.14	5159.7	63.34	81.17	3969	53.16	71.33
C16,C17,C18 C29,C30,C31	6346.125	52.131	60.59	5499.98	64.671	52.39	4230.8	52.251	63.22
C19,C26	3864.225	51.912	68.86	3349	65.272	82.728	2576.2	114.172	152.09
C23,C24,C25	6346.125	51.452	61.24	5499.98	74.172	51.485	4230.8	51.242	61.895

Table 7.6 continued

Floor	Upto third floor			Upto fourth floor			Upto fifth floor		
Column no.	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)
C1,C6, C39,C44	896.48	158.15	124	672.36	177.8	161.29	224.12	177.05	155.89
C2,C3,C4, C5,C40,C41, C42,C43	1224.24	230.94	140.5	816.16	195.31	111.54	408.08	210.67	165.75
C7,C12,C33, C38,C27,C32	1279.08	91.535	112.9	852.72	108.915	395.968	426.36	28.845	154.26
C8,C9,C10, C11,C28,C34, C35,C36,C37	2535.36	56.266	72.96	1690.24	57.576	77.409	845.12	56.716	40.869
C13,C20	1482.96	101.18	116.6	988.64	102.082	129.915	494.32	38.462	109.55
C14,C21	1053.9	79.3	106.4	702.6	91.36	58.03	351.3	47.07	33.23
C15,C22	2381.4	48.98	65.83	1587.6	44.13	71.87	793.8	24.05	41.65
C16,C17,C18 C29,C30,C31	2538.45	50.901	58.8	1692.3	46.221	51.26	846.15	41.531	24.13
C19,C26	1545.69	151.4	196.8	1030.46	176.752	188.118	515.23	38.482	140.53
C23,C24,C25	2538.45	51.132	59.07	1692.3	42.092	51.905	846.15	23.412	35.195

Table 7.7 Moment and shear forces in beams at plinth level of PT flat slab

Floor	Plinth level beams				
Beam no.	Shear force		Moment		
	Left	Right	Left	Mid	Right
B1,B31	140.85	-147.15	199.24	-124.17	224.43
B2,B32	154.78	-155.18	243.70	-136.50	245.35
B3,B33	155.03	154.93	245.22	-136.04	244.82
B4,B34	154.90	-155.06	244.77	-135.92	245.16
B5,B35	157.99	-151.97	252.27	-141.42	226.95
B6,B26	141.24	-146.76	225.24	-147.74	247.28
B7,B27	158.57	-158.95	276.91	-166.81	278.53
B8,B28	158.81	-158.71	278.60	-166.15	278.15
B9,B29	158.62	-158.90	277.83	-166.10	279.04
B10,B30	161.43	-156.09	284.10	-171.64	261.69
B11,B16	141.27	-146.73	225.37	-147.72	247.20
B12,B17,B22	158.57	-158.95	276.94	-166.87	278.50
B13,B18,B23	158.82	-158.70	278.62	-166.15	278.14
B14,B19,B24	158.62	-158.90	277.83	-166.10	279.04
B15,B20,B25	161.41	-156.11	284.04	-171.61	261.81
B21	141.27	-146.73	225.37	-147.72	247.20
B36,B66	151.85	-157.75	226.81	-141.10	251.61
B37,B67	154.96	-154.46	245.36	-135.59	244.04
B38,B68	154.83	-154.77	244.73	-135.69	244.47
B39,B69	154.77	-154.83	244.47	-135.69	244.73
B40,B70	154.64	-154.96	244.04	-135.59	245.35
B41,B71	157.75	-151.85	251.61	-141.10	226.81
B42,B48,B54,B60	156.00	-161.16	261.67	-171.25	283.31
B43,B49,B55,B61	158.83	-158.33	279.06	-165.77	276.96
B44,B50,B56,B62	158.65	-158.51	278.23	-165.85	277.65
B45,B51,B57,B63	158.51	-158.65	277.65	-165.85	278.23
B46,B52,B58,B64	158.33	-158.83	276.96	-165.77	279.06
B47,B53,B59,B65	161.16	-156.00	283.31	-171.30	261.67

Table 7.8 Moments and shear forces in beams at typical floor of PT flat slab

Beam no.	Shear force (KN)		Moment (KNm)
	Left	Right	
B1	29.54	-29.54	19.33
B2	57.42	-57.42	47.58
B3	79.92	-79.92	63.64
B4	102.13	-102.1	107.45
B5	146.67	-146.7	122.45

Table 7.9 Moment and shear forces in beams (PT slab + RCC beams)

Floor	Plinth level beams					Beams at first to fifth floor				
Beam no.	Shear force		Moment			Shear force		Moment		
	Left	Right	Left	Mid	Right	Left	Right	Left	Mid	Right
B1,B31	53.23	-48.77	86.99	-39.91	69.18	186.27	-194.01	269.45	-199.84	247.83
B2,B32	54.54	-55.08	85.39	-47.10	87.65	197.56	-190.89	270.25	-187.25	267.33
B3,B33	53.97	-55.65	82.90	-47.19	89.95	185.29	-185.29	265.47	-187.62	265.47
B4,B34	53.47	-56.15	80.82	-47.18	92.08	190.89	197.65	267.33	-187.25	270.25
B5,B35	51.45	-58.17	73.58	-45.92	101.83	194.01	186.33	247.83	-197.25	269.45
B6,B26	157.81	-158.42	282.04	-210.62	289.31	324.69	-337.85	447.11	-365.23	418.96
B7,B27	153.63	-159.62	281.84	-217.75	284.25	333.85	-347.61	448.56	-351.42	439.58
B8,B28	152.56	-149.98	276.85	-218.05	279.47	336.54	-336.84	437.24	-351.47	437.24
B9,B29	151.62	-152.22	272.90	-217.84	271.36	347.59	-333.85	439.58	-351.98	448.56
B10,B30	149.12	-148.29	262.89	-216.88	269.61	337.29	-325.85	418.96	-359.88	448.56
B11,B16	103.11	-109.26	159.80	-114.08	160.11	331.89	-325.41	397.87	-289.95	424.58
B12,B17,B22	151.40	-153.89	270.21	-193.71	274.92	340.07	-314.62	451.20	-297.59	441.02
B13,B18,B23	149.25	-150.64	258.99	-195.45	260.19	339.54	-340.12	435.10	-297.81	435.10
B14,B19,B24	149.59	150.36	260.72	-195.25	260.16	314.59	-331.58	441.02	-297.85	451.20
B15,B20,B25	147.31	152.59	252.42	-193.48	254.33	325.67	-336.33	424.58	-293.45	397.87
B21	160.26	161.62	272.62	-209.29	273.52	324.85	-324.82	449.61	351.30	452.14
B36,B66	55.42	-54.08	90.73	-45.44	85.14	178.29	-214.48	298.63	-164.52	331.52
B37,B67	86.36	-86.42	141.02	-102.66	135.67	201.27	-209.25	324.10	-203.15	344.51
B38,B68	53.55	-55.95	81.10	-47.24	91.18	189.67	-217.17	279.14	-203.45	344.51
B39,B69	87.54	-88.01	145.91	-102.72	144.51	217.17	-189.68	344.51	-203.92	279.14
B40,B70	87.27	-88.61	145.40	-102.08	144.79	209.25	-201.27	344.51	-186.61	324.10
B41,B71	85.67	-86.27	139.39	-101.38	140.23	214.48	-178.29	331.52	-186.21	298.63
B42,B48,B54,B60	85.21	-77.21	154.52	-69.74	120.94	220.36	-314.14	361.57	-168.25	452.95
B43,B49,B55,B61	164.31	-162.15	288.69	-221.56	285.99	301.59	-317.25	464.20	-384.29	462.30
B44,B50,B56,B62	116.68	-118.29	199.05	-133.60	200.06	298.88	-316.51	398.20	-284.36	471.34
B45,B51,B57,B63	164.56	-165.38	290.21	-221.10	281.57	316.51	-298.68	471.34	-284.61	398.20
B46,B52,B58,B64	163.43	-163.29	286.69	-219.85	288.65	317.25	-301.59	462.30	-387.91	464.20
B47,B53,B59,B65	161.08	-160.89	277.28	-219.41	267.47	314.14	-221.48	452.95	-387.91	361.57

Table 7.9 continued

Floor	Beams at second floor					Beams at third floor				
Beam no.	Shear force		Moment			Shear force		Moment		
	Left	Right	Left	Mid	Right	Left	Right	Left	Mid	Right
B1,B31	201.28	-206.72	296.53	-188.61	318.25	203.06	-204.94	303.65	-188.58	311.19
B2,B32	220.54	-221.74	357.04	-206.74	361.96	220.67	-221.59	357.57	-206.76	361.40
B3,B33	221.23	-221.03	361.34	-205.11	360.52	221.22	-221.04	361.38	-205.22	360.66
B4,B34	221.32	-220.94	361.61	-205.42	360.04	221.20	-221.06	361.20	-205.35	360.58
B5,B35	224.09	-218.17	362.13	-216.55	337.25	222.44	-219.82	355.19	-216.57	344.16
B6,B26	260.74	-267.24	415.64	-275.39	441.58	263.67	-264.33	427.31	-275.36	429.95
B7,B27	290.09	-292.03	503.08	-307.10	511.99	290.41	-291.71	505.23	-307.10	510.67
B8,B28	291.17	-290.95	510.63	-304.81	509.68	291.17	-290.95	510.87	-304.64	509.91
B9,B29	291.37	-290.75	511.23	-305.10	508.60	291.09	-291.03	510.23	-304.84	510.00
B10,B30	294.64	-287.44	509.73	-320.43	479.35	291.98	-290.13	498.37	-320.48	490.60
B11,B16	260.69	-267.31	415.32	-275.45	441.78	263.61	-264.39	427.00	-275.44	430.11
B12,B17,B22	290.08	-292.04	503.77	-307.10	511.99	290.42	-291.71	505.25	-307.02	510.64
B13,B18,B23	291.18	-290.94	510.66	-304.81	509.65	291.17	-290.95	510.84	-304.60	509.89
B14,B19,B24	291.39	-290.73	511.23	-305.10	508.54	291.09	-291.03	510.23	-304.83	510.02
B15,B20,B25	294.75	-287.37	509.96	-320.52	478.95	292.05	-290.07	498.54	-320.59	490.22
B21	260.69	-267.31	415.32	-275.45	441.78	263.61	-264.39	427.00	-275.44	430.11
B36,B66	217.87	-223.79	336.42	-215.92	361.50	219.28	-222.32	342.54	-215.95	355.31
B37,B67	220.71	-220.89	359.73	-204.80	360.44	220.74	-220.86	359.86	-204.77	360.37
B38,B68	220.80	-220.80	360.13	-204.27	360.14	220.80	-220.80	360.16	-204.73	360.15
B39,B69	220.80	-220.80	360.13	-204.27	360.14	220.80	-220.80	360.15	-204.73	360.16
B40,B70	220.89	-220.71	360.44	-204.79	359.73	220.86	-220.74	360.37	-204.77	359.86
B41,B71	223.79	-217.81	361.50	-215.92	336.43	222.32	-219.28	355.31	-215.95	342.54
B42,B48,B54,B60	287.15	-294.36	478.59	-319.78	509.06	289.58	-291.88	488.90	-319.87	498.58
B43,B49,B55,B61	290.53	-290.93	508.33	-304.42	510.10	290.72	-290.74	509.27	-304.29	509.37
B44,B50,B56,B62	290.72	-290.74	509.30	-304.24	509.42	290.77	-290.70	509.58	-304.17	509.29
B45,B51,B57,B63	290.74	-290.72	509.42	-304.24	509.30	290.70	-290.77	509.29	-304.17	509.58
B46,B52,B58,B64	290.93	-290.53	510.10	-304.42	508.33	290.74	-290.72	509.37	-304.29	509.27
B47,B53,B59,B65	294.36	-287.10	509.06	-319.78	478.59	291.88	-289.58	498.58	-319.87	488.90

Table 7.9 Continued

Floor	Beams at fourth floor					Beams at fifth floor				
Beam no.	Shear force		Moment			Shear force		Moment		
	Left	Right	Left	Mid	Right	Left	Right	Left	Mid	Right
B1,B31	201.64	-206.36	299.95	-186.60	318.86	140.55	-147.45	210.25	-135.95	237.85
B2,B32	219.43	-222.83	351.71	-207.40	365.99	155.71	-158.03	255.60	-152.31	265.33
B3,B33	221.15	-221.11	360.16	-206.18	359.96	156.88	-156.86	261.01	-151.79	260.96
B4,B34	222.63	-219.63	366.03	-206.53	353.41	157.77	-155.97	264.77	-151.78	257.23
B5,B35	223.74	-218.52	363.34	-213.86	341.43	160.23	-153.52	270.43	-156.44	242.25
B6,B26	261.63	-266.37	422.27	-272.25	441.24	210.36	-221.64	331.23	-222.20	376.37
B7,B27	288.51	-293.61	496.29	-307.98	517.69	236.30	-239.98	407.81	-251.26	423.25
B8,B28	291.08	-291.04	508.96	-306.09	508.81	238.15	-238.13	416.45	-250.37	416.39
B9,B29	293.33	-288.79	517.80	-306.70	498.74	239.51	-236.78	422.20	-250.33	410.73
B10,B30	293.87	-288.25	510.63	-316.15	487.00	243.61	-232.67	431.79	-257.99	385.81
B11,B16	261.58	-266.42	422.08	-272.26	441.40	210.32	-221.68	331.05	-222.23	376.50
B12,B17,B22	288.50	-293.62	496.20	-308.00	517.74	236.29	-239.99	407.76	-251.27	423.28
B13,B18,B23	291.08	-291.04	508.96	-306.10	508.78	238.15	-238.13	416.44	-250.38	416.38
B14,B19,B24	293.35	-288.77	517.88	-306.72	498.63	239.52	-236.76	422.24	-250.34	410.68
B15,B20,B25	293.92	-288.20	510.82	-316.17	486.78	243.65	-232.63	431.92	-258.03	385.61
B21	261.58	-266.42	422.08	-272.26	441.40	210.32	-221.68	331.05	-222.23	376.50
B36,B66	217.53	-224.07	337.87	-213.29	365.32	152.91	-160.29	240.24	-155.93	271.21
B37,B67	218.81	-222.79	350.64	-205.87	367.37	155.39	-157.81	255.33	-151.24	265.51
B38,B68	220.22	-221.38	356.90	-205.55	361.76	156.20	-157.00	258.75	-151.23	262.11
B39,B69	221.38	-220.22	361.76	-205.55	356.90	157.00	-156.20	262.11	-151.23	258.75
B40,B70	222.79	-218.81	367.37	-205.87	350.64	157.81	-155.39	265.51	-151.24	255.33
B41,B71	224.07	-217.53	365.32	-213.29	337.87	160.29	-152.91	271.21	-155.93	240.24
B42,B48,B54,B60	287.00	-294.46	482.36	-315.60	513.66	231.91	-243.83	383.18	-257.48	433.21
B43,B49,B55,B61	287.73	-293.73	494.96	-306.03	520.19	236.02	-239.72	408.14	-249.77	423.68
B44,B50,B56,B62	289.88	-291.58	504.59	-305.43	511.75	237.25	-238.49	413.31	-249.78	418.48
B45,B51,B57,B63	291.58	-289.88	511.75	-305.43	504.59	238.49	-237.25	418.48	-249.78	413.31
B46,B52,B58,B64	293.73	-287.73	520.19	-306.03	494.96	239.72	-236.02	423.68	-249.77	408.14
B47,B53,B59,B65	294.46	-287.00	513.66	-315.60	482.36	243.83	-242.35	433.21	-257.48	429.56

Table 7.10 Axial forces and moments in the column (PT slab + RCC beams)

Floor	Upto plinth level			Upto first floor			Upto second floor		
Column no.	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)
C1,C6, C39,C44	2261.15	105.83	92.58	1968.72	158.81	138.64	1554.1	171.5	150.2
C2,C3,C4, C5,C40,C41, C42,C43	3848.94	111.95	57.63	3391.14	216.99	78.48	2677.5	115.62	68.09
C7,C12,C33, C38,C27,C32	3800	33.63	103.2	3346.4	61.48	149.62	2646.5	55.89	76.77
C8,C9,C10, C11,C28,C34, C35,C36,C37	6282.24	32.32	37.99	5504.72	62.57	79.6	4354.4	57.38	69.91
C13,C20	3785.48	47.92	102.4	3334.73	68.73	129.34	2636.6	66.83	128.32
C14,C21	3450.6	102.93	69.47	3432.63	97.26	126.35	2624.6	152.32	87.29
C15,C22	6113.17	48.9	42.6	5491.43	61.8	79.63	4348.8	51.62	69.79
C16,C17,C18 C29,C30,C31	6255.07	48.95	57.86	5619.81	61.49	49.66	4454.9	49.07	60.49
C19,C26	3932.88	47.98	65.48	3467.03	61.34	79.35	2741.2	110.24	148.71
C23,C24,C25	6254.74	47.47	57.86	5619.6	70.19	48.11	4454.9	47.26	58.52

Table 7.10 Continued

Floor	Upto third floor			Upto fourth floor			Upto fifth floor		
Column no.	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)
C1,C6, C39,C44	1134.97	131.76	101.4	712.63	151.41	119.4	293.46	150.66	133.24
C2,C3,C4, C5,C40,C41, C42,C43	1968.87	192.42	107.4	1247.87	156.79	114.74	535.68	172.15	132.69
C7,C12,C33, C38,C27,C32	1937.27	87.54	109.5	1230.54	104.92	160.55	526.03	24.85	150.83
C8,C9,C10, C11,C28,C34, C35,C36,C37	3212.2	48.25	66.08	2074.86	49.56	70.53	937.79	48.7	33.99
C13,C20	1934.2	97.25	113.3	1228.93	98.15	126.54	524.33	34.53	106.17
C14,C21	1814.31	77.59	104.7	1027.22	89.65	56.32	306.18	45.36	31.52
C15,C22	3209.78	47.44	64.29	2073.47	42.59	70.33	934.95	22.51	40.11
C16,C17,C18 C29,C30,C31	3289.73	47.72	56.07	2124.68	43.04	48.53	955.66	38.35	21.4
C19,C26	2011.84	147.47	193.5	1279.11	172.82	184.74	547.18	34.55	137.15
C23,C24,C25	3289.14	47.15	55.69	2121.21	38.11	48.53	955.66	19.43	31.82

Table 7.11 Moments and shear forces in the beams (RCC slab +RCC beams)

Floor	Plinth level beams					Beams at first to fifth floor				
Beam no.	Shear force		Moment			Shear force		Moment		
	Left	Right	Left	Mid	Right	Left	Right	Left	Mid	Right
B1,B31	53.23	-48.77	86.99	-39.91	69.18	186.27	-194.01	269.45	-199.84	247.83
B2,B32	54.54	-55.08	85.39	-47.10	87.65	197.56	-190.89	270.25	-187.25	267.33
B3,B33	53.97	-55.65	82.90	-47.19	89.95	185.29	-185.29	265.47	-187.62	265.47
B4,B34	53.47	-56.15	80.82	-47.18	92.08	190.89	-197.65	267.33	-187.25	270.25
B5,B35	51.45	-58.17	73.58	-45.92	101.83	194.01	-186.33	247.83	-197.25	269.45
B6,B26	157.81	-158.42	282.04	-210.62	289.31	324.69	--337.85	447.11	-365.23	418.96
B7,B27	153.63	-159.62	281.84	-217.75	284.25	333.85	-347.61	448.56	-351.42	439.58
B8,B28	152.56	-149.98	276.85	-218.05	279.47	336.54	-336.84	437.24	-351.47	437.24
B9,B29	151.62	-152.22	272.90	-217.84	271.36	347.59	-333.85	439.58	-351.98	448.56
B10,B30	149.12	-148.29	262.89	-216.88	269.61	337.29	-325.85	418.96	-359.88	448.56
B11,B16	103.11	-109.26	159.80	-114.08	160.11	331.89	-325.41	397.87	-289.95	424.58
B12,B17,B22	151.40	-153.89	270.21	-193.71	274.92	340.07	-314.62	451.20	-297.59	441.02
B13,B18,B23	149.25	-150.64	258.99	-195.45	260.19	339.54	-340.12	435.10	-297.81	435.10
B14,B19,B24	149.59	150.36	260.72	-195.25	260.16	314.59	-331.58	441.02	-297.85	451.20
B15,B20,B25	147.31	152.59	252.42	-193.48	254.33	325.67	-336.33	424.58	-293.45	397.87
B21	160.26	161.62	272.62	-209.29	273.52	324.85	-324.82	449.61	351.30	452.14
B36,B66	55.42	-54.08	90.73	-45.44	85.14	178.29	-214.48	298.63	-164.52	331.52
B37,B67	86.36	-86.42	141.02	-102.66	135.67	201.27	-209.25	324.10	-203.15	344.51
B38,B68	53.55	-55.95	81.10	-47.24	91.18	189.67	-217.17	279.14	-203.45	344.51
B39,B69	87.54	-88.01	145.91	-102.72	144.51	217.17	-189.68	344.51	-203.92	279.14
B40,B70	87.27	-88.61	145.40	-102.08	144.79	209.25	-201.27	344.51	-186.61	324.10
B41,B71	85.67	-86.27	139.39	-101.38	140.23	214.48	-178.29	331.52	-186.21	298.63
B42,B48,B54,B60	85.21	-77.21	154.52	-69.74	120.94	220.36	-314.14	361.57	-168.25	452.95
B43,B49,B55,B61	164.31	-162.15	288.69	-221.56	285.99	301.59	-317.25	464.20	-384.29	462.30
B44,B50,B56,B62	116.68	-118.29	199.05	-133.60	200.06	298.88	-316.51	398.20	-284.36	471.34
B45,B51,B57,B63	164.56	-165.38	290.21	-221.10	281.57	316.51	-298.68	471.34	-284.61	398.20
B46,B52,B58,B64	163.43	-163.29	286.69	-219.85	288.65	317.25	-301.59	462.30	-387.91	464.20
B47,B53,B59,B65	161.08	-160.89	277.28	-219.41	267.47	314.14	-221.48	452.95	-387.91	361.57

Table 7.11 Continued

Floor	Plinth level beams					Beams at first to fifth floor				
Beam no.	Shear force		Moment			Shear force		Moment		
	Left	Right	Left	Mid	Right	Left	Right	Left	Mid	Right
B72,B96	69.54	-69.24	98.75	-69.99	97.56	178.25	-179.58	301.52	-201.48	287.56
B73,B97	71.25	-70.99	101.22	-72.59	99.89	185.27	-181.10	324.20	-199.84	324.25
B74,B98	79.68	-82.36	110.36	-88.84	101.36	187.25	-187.36	332.34	-199.71	332.34
B75,B99	73.65	-74.69	107.58	-79.71	97.94	181.10	-185.27	324.25	-200.52	324.20
B76,B100	54.69	-73.65	29.22	-78.39	105.09	179.58	-185.54	287.56	-200.38	301.52
B77,B86,B91	68.09	-68.00	106.10	-55.49	105.72	182.41	-163.18	321.06	-187.20	333.14
B78,B82,B87,B92	67.98	-68.10	105.20	-55.94	105.74	183.21	-179.58	320.14	-187.61	328.46
B79,B83,B88,B93	67.25	-68.83	104.78	-53.30	111.43	183.24	-183.64	315.01	-187.61	315.01
B80,B84,B89,B94	77.20	-58.88	112.83	-87.04	35.91	179.50	-182.36	328.46	-183.41	320.14
B81,B85,B90,B95	57.78	-78.30	32.21	-86.12	118.37	162.59	-183.55	323.21	-179.62	324.10
B101,B112,B124,B136,B148	28.59	-28.35	56.87	-14.88	59.54	170.16	-184.75	324.01	-139.43	265.31
B102,B113,B125,B137,B149	28.35	-29.36	59.54	-133.95	238.14	135.69	-189.99	283.67	-329.14	434.54
B103,B114,B126,B138,B150	89.26	22.51	207.44	-44.15	-44.46	145.92	-182.36	434.54	-204.31	283.67
B104,B115,B127,B139,B151	22.51	44.24	44.46	-50.37	18.40	175.95	-183.54	265.31	-289.61	324.01
B105	89.64	-91.10	54.22	-39.82	63.54	189.99	-183.74	355.16	-268.24	359.12
B106,B118,B130,B142,B154	59.64	-56.27	83.64	-66.57	83.21	171.36	-184.39	341.29	-264.31	336.34
B107,B119,B131,B143,B155	56.30	-10.45	29.67	-64.42	83.82	184.49	-172.00	299.99	-264.39	345.64
B108,B120,B132,B144,B156	10.45	-77.20	83.82	-20.54	117.43	183.64	-189.24	317.85	-210.38	304.35
B109,B121,B133,B145,B157	59.64	-58.62	89.64	-67.59	88.95	182.36	-176.29	304.35	-210.46	317.85
B110,B122,B134,B146,B158	67.21	-69.57	98.52	-68.96	98.57	189.48	-154.26	345.64	-211.36	299.99
B111,B123,B135,B147,B159	71.20	-72.22	97.25	-78.96	97.25	184.75	-145.97	336.34	-231.20	341.29
B116,B128,B140,B152	69.79	-70.15	91.39	-72.59	98.52	179.61	-179.82	345.27	-271.30	326.54
B117,B129,B141,B153	68.22	-67.81	89.36	-63.99	89.59	178.66	-179.68	326.54	-271.30	345.27

Table 7.12 Axial forces and moments in the column (RCC slab + RCC beams)

Floor	Upto plinth level			Upto first floor			Upto second floor		
Column no.	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)
C1,C6, C39,C44	2507.73	70.44	101.9	2217.5	88.19	107.03	1800.9	77.46	69.42
C2,C3,C4, C5,C40,C41, C42,C43	4271.13	200.13	98.19	3813.33	219.1	57.54	3099.7	149.14	76.89
C7,C12,C33, C38,C27,C32	4222.19	70.14	103.4	3768.59	75.9	162.14	3068.7	52.38	149.47
C8,C9,C10, C11,C28,C34, C35,C36,C37	6704.43	76.1	101.3	5926.91	74.32	119.83	4776.6	66.39	97.17
C13,C20	4207.67	72.59	99.55	3756.92	91.77	45.08	3058.8	96.66	52.44
C14,C21	3872.79	52.13	117.1	3854.82	70.47	94.34	3046.8	42.83	84.86
C15,C22	6535.36	73.37	94	5913.62	85.57	124.44	4771	89.65	116.13
C16,C17,C18 C29,C30,C31	6677.26	83.01	66.32	6042	43.4	52.34	4877.10	65.19	77.36
C19,C26	4355.07	80.82	131.5	3889.22	94.25	119.06	3163.4	62.22	107.97
C23,C24,C25	6676.93	83.66	32.4	6041.79	97.87	49.6	4877.1	68.91	37.4

Table 7.12 continue

Floor	Upto third floor			Upto fourth floor			Upto fifth floor		
Column no.	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)	Axial force (KN)	Mx (KNm)	My (KNm)
C1,C6, C39,C44	1381.75	141.87	110.4	957.86	133.74	128.89	538.69	113.92	119.69
C2,C3,C4, C5,C40,C41, C42,C43	2391.06	143.31	57.29	1670.06	140.98	55.63	957.87	101.44	45.81
C7,C12,C33, C38,C27,C32	2359.46	33.87	115.9	1652.73	36.98	129.02	948.22	47.66	97.27
C8,C9,C10, C11,C28,C34, C35,C36,C37	3634.39	43.95	60.45	2497.05	46.17	58.96	1360	42.34	31.29
C13,C20	2356.39	66.66	85.66	1651.12	63.3	81.82	946.52	103.09	69.36
C14,C21	2236.5	31.65	66.7	1449.41	34.03	60.47	657.44	26.82	49.56
C15,C22	3631.97	87.02	96.84	2495.66	97.01	115.22	1357.1	84.13	104.72
C16,C17,C18 C29,C30,C31	3711.92	46.93	31.32	2546.87	51.26	24.42	1377.9	38.14	37.62
C19,C26	2434.03	46.52	109.6	1701.3	47.55	133.51	969.37	35.54	86.91
C23,C24,C25	3711.33	49.57	60.3	2543.4	51.35	32.01	1377.9	38.43	38.41

CASE 1 POST-TENSIONED FLAT SLAB

The design of the post-tensioned flat slab, column subjected to axial force and biaxial moment from the analysis results is carried out and the detail design procedure is given in the appendix B. According to the design procedure in the appendix B the whole design is carried out and the design results are tabulated in the following tables (Table 7.13 to 7.17). The cable profiles, the layout of the cable along the span, the number of cable in the section are shown in the sheet 1. The reinforcement at the bottom and the top with the cross section are given in the sheet 2.

CASE 2 REINFORCED CONCRETE FLAT SLAB

The detail design of reinforced concrete column is given in the appendix B. The design details of slab reinforcement and column reinforcement are given in table 7.18 to 7.21. The schedule of reinforcement for the plinth level beams is same as for the post-tensioned plinth beams as in the table 7.17. The detail reinforcement in the flat slab is as per the sheet 3.

CASE 3 POST-TENSIONED SLAB WITH REINFORCED CONCRETE BEAMS

The design of post-tensioned slab and the reinforced concrete continuous beam is given in the appendix B, and the design details are for each beam, column and slab are as per table 7.22 to 7.25. The cable layout and the cable profile for the post-tensioned slab are given in sheet 4.

CASE 4 REINFORCED CONCRETE SLAB WITH REINFORCED CONCRETE BEAMS

The detail design of the reinforced concrete one way slab, two way slabs, the design continuous beams and the column design is given in the appendix B. the schedule of the beam, slab and column are given in the following tables (7.26 to 7.29). The reinforcement details are given in sheet 6.

Table 7.13 Drop dimensions

Drop notation	Size (m)	Depth (mm)
1	1.4 x 1.4	350
2	2.8 x 1.4	350
3	2.8 x 2.8	350

Table 7.14 Schedule of drop for PT flat slab

Floor		Drop1		Drop 2		Drop 3	
		A	B	A	B	A	B
1	Bar dia. (mm)	16	12	16	16	12	12
	C/C spacing (mm)	150	130	150	150	130	130
2	Bar dia. (mm)	12	12	12	12	12	12
	C/C spacing (mm)	100	100	100	100	130	130
3	Bar dia. (mm)	16	16	16	16	12	12
	C/C spacing (mm)	150	150	150	150	130	130
4	Bar dia. (mm)	16	16	16	16	12	12
	C/C spacing (mm)	150	150	150	150	130	130
5	Bar dia. (mm)	12	12	12	12	12	12
	C/C spacing (mm)	100	100	100	100	140	140

Table 7.15 Schedule of the slab reinforcement

Slab	Thickness (mm)	Main steel	Distribution steel
S1	200	10 mm Φ @ 200 mm c/c	8 mm Φ @ 200 mm c/c
S2	200	8 mm Φ @ 200 mm c/c	8 mm Φ @ 200 mm c/c

Table 7.16 Schedule of column for PT flat slab

SCHEDULE OF COLUMN UPTO PLINTH LEVEL			
Column No.	Size (mm)	Main reinforcement	Shear reinforcement
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C8, C15, C22, C28, C34	600 x 600	12-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C13, C20	600 x 600	10-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 600	12-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	10-20mm Φ + 10-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 600	8-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
SCHEDULE OF COLUMN UPTO FIRST FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 600	10-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	600 x 600	6-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 600	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	8-20mm Φ + 10-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 600	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO SECOND FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	500 x 500	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	500 x 500	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	500 x 500	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	500 x 500	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 500	6-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c

Table 7.16 continued

SCHEDULE OF COLUMNUPTO THIRD FLOOR			
Column No.	Size (mm)	Main reinforcement	Shear reinforcement
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	500 x 500	6-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	500 x 500	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	500 x 500	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	500 x 500	6-16mm Φ + 6-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	8-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 500	6-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO FOURTH FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	450 x 450	8-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	450 x 450	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	450 x 450	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
C7, C27, C33	450x 450	8-16mm Φ + 4-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 450	6-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	450 x 450	6-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO FIFTH FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	450 x 450	6-16mm Φ + 2-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	450 x 450	8-16mm Φ + 6-12mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	450 x 450	6-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
C7, C27, C33	450x 450	8-16mm Φ + 2-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 450	4-16mm Φ + 6-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	450 x 450	6-16mm Φ + 4-12mmΦ	8 mm Φ @ 200 mm c/c

Table 7.17 Scheduling of plinth beams (PT flat slab)

Beam no	Width	Depth	Top reinf.	Bottom reinf.	Shear reinf.	Extra top bars
1,2,3,4,5,31,32, 33, 34, 35.	450	450	2-16mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
6,10,26,30.	450	450	2-16mm Φ	3-16mm Φ	8mmΦ@150mm c/c	3-20mm Φ
7,8,9,27,28,29.	450	450	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
11.	450	450	2-12mm Φ	3-12mm Φ	8mmΦ@150mm c/c	3-12mm Φ
12,13,14,15.	450	600	2-12mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-12mm Φ
16.	600	450	2-12mm Φ	4-12mm Φ	8mmΦ@150mm c/c	2-12mm Φ
17,18,19,20.	600	600	2-16mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
21,22,23,24,25.	450	600	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	2-20mm Φ
36,38,41,66,71.	450	450	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	3-16mm Φ
37,40,67,70.	450	450	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
39,69.	450	450	2-12mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-16mm Φ
42,47,60,65.	450	450	3-12mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
43,46,61,64.	600	600	2-12mm Φ	3-12mm Φ	8mmΦ@150mm c/c	2-25mm Φ
44,45,62,63.	600	600	2-16mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
48,53,54,59.	600	600	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-25mm Φ
49,52,55,58.	600	600	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	3-20mm Φ
50,51,56,57.	600	600	3-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-25mm Φ
68	450	450	2-12mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ

Table 7.18 Drop dimensions for RCC flat slab

Drop notation	Size (m)	Depth (mm)
1	1.4 x 1.4	350
2	2.8 x 1.4	350
3	2.8 x 2.8	350

Table 7.19 Schedule of drop for RCC flat slab

Floor		Drop1		Drop 2		Drop 3	
		A	B	A	B	A	B
1	Bar dia. (mm)	16	16	16	16	16	16
	C/C spacing (mm)	150	130	150	150	130	130
2	Bar dia. (mm)	16	16	16	16	16	16
	C/C spacing (mm)	100	100	100	100	130	130
3	Bar dia. (mm)	16	16	16	16	16	16
	C/C spacing (mm)	150	150	150	150	130	130
4	Bar dia. (mm)	16	16	16	16	12	12
	C/C spacing (mm)	150	150	150	150	120	120
5	Bar dia. (mm)	16	16	12	16	12	12
	C/C spacing (mm)	120	120	120	120	150	150

Table 7.20 Schedule of the slab reinforcement

Slab	Thickness (mm)	Main steel	Distribution steel
S1	225	10 mm Φ @ 200 mm c/c	8 mm Φ @ 200 mm c/c
S2	225	As per the detailing in sheet 3	

Table 7.21 Schedule of column for reinforced concrete flat slab

SCHEDULE OF COLUMN UPTO PLINTH LEVEL			
Column No.	Size (mm)	Main reinforcement	Shear reinforcement
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C8, C15, C22, C28, C34	600 x 600	12-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C13, C20	600 x 600	10-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 600	12-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	10-20mm Φ + 10-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 600	8-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
SCHEDULE OF COLUMN UPTO FIRST FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 600	10-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	600 x 600	6-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 600	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	8-20mm Φ + 10-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 600	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO SECOND FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	500 x 500	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	500 x 500	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	500 x 500	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	500 x 500	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 500	6-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c

Table 7.21 continued

SCHEDULE OF COLUMNUPTO THIRD FLOOR			
Column No.	Size (mm)	Main reinforcement	Shear reinforcement
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	500 x 500	6-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	500 x 500	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	500 x 500	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	500 x 500	6-16mm Φ + 6-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	8-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 500	6-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO FOURTH FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	450 x 450	8-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	450 x 450	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	450 x 450	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
C7, C27, C33	450x 450	8-16mm Φ + 4-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 450	6-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	450 x 450	6-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO FIFTH FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	450 x 450	6-16mm Φ + 2-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	450 x 450	8-16mm Φ + 6-12mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	450 x 450	6-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
C7, C27, C33	450x 450	8-16mm Φ + 2-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 450	4-16mm Φ + 6-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	450 x 450	6-16mm Φ + 4-12mmΦ	8 mm Φ @ 200 mm c/c

Table 7.22 Scheduling for plinth beams (PT slab + RCC beams)

Beam no	Width	Depth	Top reinf.	Bottom reinf.	Shear reinf.	Extra top bars
1,2,3,4,5,31,32, 33, 34, 35.	450	450	2-16mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
6,10,26,30.	450	450	2-16mm Φ	3-16mm Φ	8mmΦ@150mm c/c	3-20mm Φ
7,8,9,27,28,29.	450	450	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
11.	450	450	2-12mm Φ	3-12mm Φ	8mmΦ@150mm c/c	3-12mm Φ
12,13,14,15.	450	600	2-12mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-12mm Φ
16.	600	450	2-12mm Φ	4-12mm Φ	8mmΦ@150mm c/c	2-12mm Φ
17,18,19,20.	600	600	2-16mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
21,22,23,24,25.	450	600	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	2-20mm Φ
36,38,41,66,71.	450	450	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	3-16mm Φ
37,40,67,70.	450	450	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
39,69.	450	450	2-12mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-16mm Φ
42,47,60,65.	450	450	3-12mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
43,46,61,64.	600	600	2-12mm Φ	3-12mm Φ	8mmΦ@150mm c/c	2-25mm Φ
44,45,62,63.	600	600	2-16mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
48,53,54,59.	600	600	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-25mm Φ
49,52,55,58.	600	600	2-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	3-20mm Φ
50,51,56,57.	600	600	3-12mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-25mm Φ
68	450	450	2-12mm Φ	3-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ

Table 7.23 Scheduling for beams at first to fifth floor (PT slab + RCC beams)

Beam no	Width	Depth	Top reinf.	Bottom reinf.	Shear reinf.	Extra top bars
1,2,3,4,5,31,32, 33, 34, 35.	450	600	3-12mm Φ	3-20mm Φ	8mmΦ@150mm c/c	3-20mm Φ
6,10,26,30.	450	600	3-16mm Φ	4-20mm Φ	8mmΦ@150mm c/c	3-20mm Φ
7,8,9,27,28,29.	450	600	4-16mm Φ	4-20mm Φ	8mmΦ@150mm c/c	2-25mm Φ
11.	450	600	3-16mm Φ	3-20mm Φ	8mmΦ@150mm c/c	4-20mm Φ
12,13,14,15.	450	600	3-16mm Φ	4-16mm Φ	8mmΦ@150mm c/c	3-25mm Φ
16.	600	600	2-16mm Φ	4-16mm Φ	8mmΦ@150mm c/c	2-25mm Φ
17,18,19,20.	600	600	3-16mm Φ	4-20mm Φ	8mmΦ@150mm c/c	2-20mm Φ
21,22,23,24,25.	450	600	3-16mm Φ	3-20mm Φ	8mmΦ@190mm c/c	3-25mm Φ
36,38,41,66,71.	450	600	3-16mm Φ	3-20mm Φ	8mmΦ@150mm c/c	3-20mm Φ
37,40,67,70.	450	600	4-12mm Φ	5-16mm Φ	8mmΦ@150mm c/c	3-20mm Φ
39,69.	450	600	4-12mm Φ	5-16mm Φ	8mmΦ@150mm c/c	2-20mm Φ
42,47,60,65.	450	600	3-16mm Φ	4-20mm Φ	8mmΦ@150mm c/c	4-25mm Φ
43,46,61,64.	600	600	3-16mm Φ	4-20mm Φ	8mmΦ@150mm c/c	3-25mm Φ
44,45,62,63.	600	600	2-16mm Φ	4-20mm Φ	8mmΦ@150mm c/c	3-25mm Φ
48,53,54,59.	600	600	3-16mm Φ	4-20mm Φ	8mmΦ@150mm c/c	2-25mm Φ
49,52,55,58.	600	600	2-16mm Φ	3-20mm Φ	8mmΦ@150mm c/c	3-20mm Φ
50,51,56,57.	600	600	3-16mm Φ	4-20mm Φ	8mmΦ@150mm c/c	3-25mm Φ
68	450	600	3-16mm Φ	5-16mm Φ	8mmΦ@150mm c/c	3-20mm Φ

Table 7.24 Schedule of column (PT slab + RCC beams)

SCHEDULE OF COLUMNUPTO PLINTH LEVEL			
Column No.	Size (mm)	Main reinforcement	Shear reinforcement
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 600	12-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C8, C15, C22, C28, C34	600 x 600	10-16mm Φ + 6-25mmΦ	8 mm Φ @ 150 mm c/c
C13, C20	600 x 600	810-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 450	10-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	10-20mm Φ + 10-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 600	8-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
SCHEDULE OF COLUMN UPTO FIRST FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 600	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	600 x 600	6-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 600	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	8-20mm Φ + 10-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 600	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO SECOND FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 600	8-16mm Φ + 10-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 600	10-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	600 x 450	10-16mm Φ + 6-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 450	10-16mm Φ + 4-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 600	6-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c

Table 7.24 Continued

SCHEDULE OF COLUMNUPTO THIRD FLOOR			
Column No.	Size (mm)	Main reinforcement	Shear reinforcement
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 450	10-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 450	10-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	600 x 450	10-16mm Φ + 4-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600x 450	10-16mm Φ + 4-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 450	8-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO FOURTH FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 450	8-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 450	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	450 x 450	10-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C7, C27, C33	450x 450	10-16mm Φ + 4-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 450	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 450	8-16mm Φ + 2-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO FIFTH FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	450 x 450	8-16mm Φ + 2-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	450 x 450	8-16mm Φ + 4-12mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	450 x 450	8-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
C7, C27, C33	450x 450	8-16mm Φ + 4-12mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	450 x 450	8-16mm Φ + 4-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	450 x 450	6-16mm Φ + 4-12mmΦ	8 mm Φ @ 200 mm c/c

Table 7.25 Scheduling of slab (PT slab+ RCC beams)

Slab	Thickness (mm)	Main steel	Distribution steel
S1	200	10 mm Φ @ 200 mm c/c	8 mm Φ @ 200 mm c/c
S2	200	8 mm Φ @ 200 mm c/c	8 mm Φ @ 200 mm c/c
S3	200	10 mm Φ @ 150 mm c/c	8 mm Φ @ 200 mm c/c

Table 7.26 Schedule of slabs for RCC slab + RCC beams

Slab	Thickness mm	Steel in direction		Remark
		Short direction	Long direction	
S1	140	8mm Φ @ 140mm c/c	8mm Φ @ 160mm c/c	Two way
S2	140	8mm Φ @ 140mm c/c	8mm Φ @ 140mm c/c	Two way
S3	140	10mm Φ @ 150mm c/c	8mm Φ @ 200mm c/c	One way
S4	140	8mm Φ @ 140mm c/c	8mm Φ @ 140mm c/c	Two way

Table 7.27 Schedule for beams at plinth level (RCC slab + RCC beams)

Beam no	Width	Depth	Top reinf.	Bottom reinf.	Shear reinf.	Extra top bars
1,2,3,4,5,31,32, 33,34, 35.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	-
6,10,26,30.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-16mm Φ
7,8,9,27,28,29.	300	450	3-16mm Φ	6-12mm Φ	8mmΦ@190mm c/c	2-16mm Φ
11.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-12mm Φ
12,13,14,15.	300	450	3-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	2-16mm Φ
16.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-16mm Φ
17,18,19,20.	300	450	3-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	2-16mm Φ
21,22,23,24,25.	300	450	3-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	2-20mm Φ
36,38,41,66,71.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-16mm Φ
37,40,67,70.	300	450	3-12mm Φ	4-12mm Φ	8mmΦ@190mm c/c	2-12mm Φ
39,69.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-16mm Φ
42,47,60,65.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
43,46,61,64.	300	450	3-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	3-16mm Φ
44,45,62,63.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-16mm Φ
48,53,54,59.	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-12mm Φ
49,52,55,58.	300	450	3-12mm Φ	3-20mm Φ	8mmΦ@190mm c/c	3-16mm Φ
50,51,56,57.	300	450	3-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	2-16mm Φ
68	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
72,76,96,100.	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
73,75,97,99.	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
74,98.	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	-
77,81,91,95	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
78,80,92,94.	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
79,93.	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	-
85,86,90.	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	-
82,87,84,89	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
83,88.	300	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
101 to 104, 106 to 159	300	450	2-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	At each junction 2-10mm Φ
105	300	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-12mm Φ

Table 7.28 Schedule of beams at first to fifth floor (RCC slab + RCC beams)

Beam no	Width	Depth	Top reinf.	Bottom reinf.	Shear reinf.	Extra top bars
1,2,3,4,5,31,32, 33, 34, 35.	600	450	3-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	2-20mm Φ
6,10,26,30.	450	450	3-16mm Φ	4-16mm Φ	8mmΦ@190mm c/c	3-20mm Φ
7,8,9,27,28,29.	450	450	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	2-20mm Φ
11.	450	450	2-10mm Φ	4-12mm Φ	8mmΦ@190mm c/c	2-12mm Φ
12,13,14,15.	450	600	2-12mm Φ	4-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
16.	450	450	2-12mm Φ	4-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
17,18,19,20.	450	600	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	2-12mm Φ
21,22,23,24,25.	450	600	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	2-20mm Φ
36,38,41,66,71.	600	450	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	3-20mm Φ
37,40,67,70.	600	450	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	2-20mm Φ
39,69.	600	450	2-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-16mm Φ
42,47,60,65.	600	450	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	3-20mm Φ
43,46,61,64.	450	600	2-12mm Φ	4-12mm Φ	8mmΦ@190mm c/c	2-20mm Φ
44,45,62,63.	450	600	2-16mm Φ	4-12mm Φ	8mmΦ@190mm c/c	2-20mm Φ
48,53,54,59.	450	600	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	2-20mm Φ
49,52,55,58.	450	600	2-12mm Φ	4-16mm Φ	8mmΦ@190mm c/c	3-20mm Φ
50,51,56,57.	600	600	2-12mm Φ	2-16mm Φ	8mmΦ@190mm c/c	2-25mm Φ
68	600	450	2-12mm Φ	4-12mm Φ	8mmΦ@190mm c/c	3-20mm Φ
72,76,96,100.	600	450	2-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	2-16mm Φ
73,75,97,99.	600	450	2-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	2-12mm Φ
74,98.	600	450	2-12mm Φ	3-16mm Φ	8mmΦ@190mm c/c	2-12mm Φ
77,81,91,95	600	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
78,80,92,94.	600	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
79,93.	600	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	-
85,86,90.	600	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	-
82,87,84,89	600	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
83,88.	600	450	3-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-10mm Φ
101 to 104, 106 to 159	450	450	2-10mm Φ	3-12mm Φ	8mmΦ@190mm c/c	At each junction 2-10mm Φ
105	450	450	3-12mm Φ	3-12mm Φ	8mmΦ@190mm c/c	2-12mm Φ

Table 7.29 Schedule of columns (RCC slab + RCC beams)

SCHEDULE OF COLUMNUPTO PLINTH LEVEL			
Column No.	Size (mm)	Main reinforcement	Shear reinforcement
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 450	8-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C8, C15, C22, C28, C34	600 x 450	10-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C13, C20	600 x 450	8-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 450	10-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	8-20mm Φ + 12-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 450	6-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
SCHEDULE OF COLUMN UPTO FIRST FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	600 x 450	6-16mm Φ + 8-25mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 450	8-16mm Φ + 8-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	600 x 450	6-16mm Φ + 10-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 450	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	6-20mm Φ + 12-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 450	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO SECOND FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	450 x 450	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 450	6-16mm Φ + 8-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	600 x 450	8-16mm Φ + 8-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	600 x 450	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	6-16mm Φ + 12-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	600 x 450	6-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c

Table 7.29 Continued

SCHEDULE OF COLUMNUPTO THIRD FLOOR			
Column No.	Size (mm)	Main reinforcement	Shear reinforcement
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	450 x 450	6-16mm Φ + 8-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	600 x 450	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	600 x 450	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 150 mm c/c
C7, C27, C33	450x 450	6-16mm Φ + 6-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 600	8-16mm Φ + 2-20mmΦ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	450 x 450	6-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO FOURTH FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	450 x 300	6-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	450 x 450	8-16mm Φ + 6-20mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	450 x 450	8-16mm Φ + 6-25mmΦ	8 mm Φ @ 200 mm c/c
C7, C27, C33	450x 450	8-16mm Φ + 4-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	600 x 450	6-16mm Φ + 8-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	450 x 300	6-16mm Φ + 2-20mmΦ	8 mm Φ @ 200 mm c/c
SCHEDULE OF COLUMN UPTO FIFTH FLOOR			
C1, C2, C3, C4, C5, C6, C39, C40, C41, C42, C43, C44, C14, C21	350 x 300	6-16mm Φ + 2-20mmΦ	8 mm Φ @ 200 mm c/c
C8, C15, C22, C28, C34	450 x 300	8-16mm Φ + 2-12mmΦ	8 mm Φ @ 200 mm c/c
C13, C20	450 x 300	6-16mm Φ + 4-20mmΦ	8 mm Φ @ 200 mm c/c
C7, C27, C33	450x 300	8-16mm Φ + 2-20mmΦ	8 mm Φ @ 150 mm c/c
C9, C10, C11, C16, C17, C18, C23, C24, C25, C29, C30, C31, C35, C36, C37	450 x 450	4-16mm Φ + 6-25mmΦ	8 mm Φ @ 150 mm c/c
C12, C19, C26, C32, C38	350 x 300	6-16mm Φ + 2-12mmΦ	8 mm Φ @ 200 mm c/c

7.4 ESTIMATING AND COSTING

From the analysis and design results of the office building the total estimation for the quantities for a typical floor is calculated. The quantities of concrete, reinforcing steel, prestressing steel and the formwork and their cost according to the current rate excluding the labour charges for all the four cases are given in the table 7.30 to 7.37. The rate per square meter for a typical floor (which includes slab, beam and column) of a building in each case is according to the values calculated form the detail estimation.

Table 7.30 Quantities for PT flat slab

Item	Concrete (m^3)	Reinforcing steel (Kg)	Prestressing steel (Kg)
Slab	415.1	12261.56	8400
Drop	41.84	11163	-
Beam	8.58	463	-
Column	42	7772	-
Total	507.52	31659.56	8400

Table 7.31 Quantities for RCC flat slab

Item	Concrete (m^3)	Reinforcing steel (Kg)	Prestressing steel (Kg)
Slab	471.74	65249.24	-
Drop	27.37	12213.54	-
Beam	8.58	463	-
Column	42	7625	-
Total	549.69	85550.78	-

Table 7.32 Quantities for PT slab with RCC beams

Item	Concrete (m^3)	Reinforcing steel (Kg)	Prestressing steel (Kg)
Slab	419.33	8328	6720
Drop	-	-	-
Beam	180	26630	-
Column	42	7313	-
Total	641.33	42271	6720

Table 7.33 Quantities for RCC slab with RCC beams

Item	Concrete (m^3)	Reinforcing steel (Kg)	Prestressing steel (Kg)
Slab	293.53	18430	-
Drop	-	-	-
Beam	290.78	58426.2	-
Column	42	9845	-
Total	626.31	86701.2	-

Table 7.34 Cost of a typical floor for PT flat slab

Item	Quantity	Unit	Rate	Amount
Concrete	507.52	m^3	4400	2233088
Reinforcing Steel	31659.56	Kg	50	1582978
Prestressing steel	8400	Kg	130	1092000
Form work	2100	m^2	400	840000
			Total	5748066

Rate per Sq.m = 2741.56 i.e. **Rs. 2800/-**

Table 7.35 Cost of a typical floor for RCC flat slab

Item	Quantity	Unit	Rate	Amount
Concrete	549.69	m^3	4400	2418636
Reinforcing Steel	85550.78	Kg	50	4277539
Prestressing steel	-	Kg	130	-
Form work	2100	m^2	400	840000
			Total	7536175

Rate per Sq.m = 3594.41 i.e. **Rs. 3600/-**

Table 7.36 Cost of a typical floor for PT slab with RCC beams

Item	Quantity	Unit	Rate	Amount
Concrete	641.33	m^3	4400	2821852
Reinforcing Steel	42271	Kg	50	2113550
Prestressing steel	6720	Kg	130	873600
Form work	2100	m^2	400	840000
			Total	6649002

Rate per Sq.m = 3171.26 i.e. **Rs. 3200/-**

Table 7.37 Cost of a typical floor for RCC slab with RCC beams

Item	Quantity	Unit	Rate	Amount
Concrete	626.31	m^3	4400	2755764
Reinforcing Steel	86701.2	Kg	50	4335060
Prestressing steel	-	Kg	130	-
Form work	2100	m^2	400	840000
			Total	7930824

Rate per Sq.m = 3782.63 i.e. **Rs. 3800/-**

7.5 RESULTS AND DISCUSSION

The analysis, design and the estimation of the office building for the four different floor systems is done and finally the rate per square meter for the construction of this building is found out. The fig 7.5 shows the variation of the rate per square meter for these four different cases. The observation made from the above work is as follows

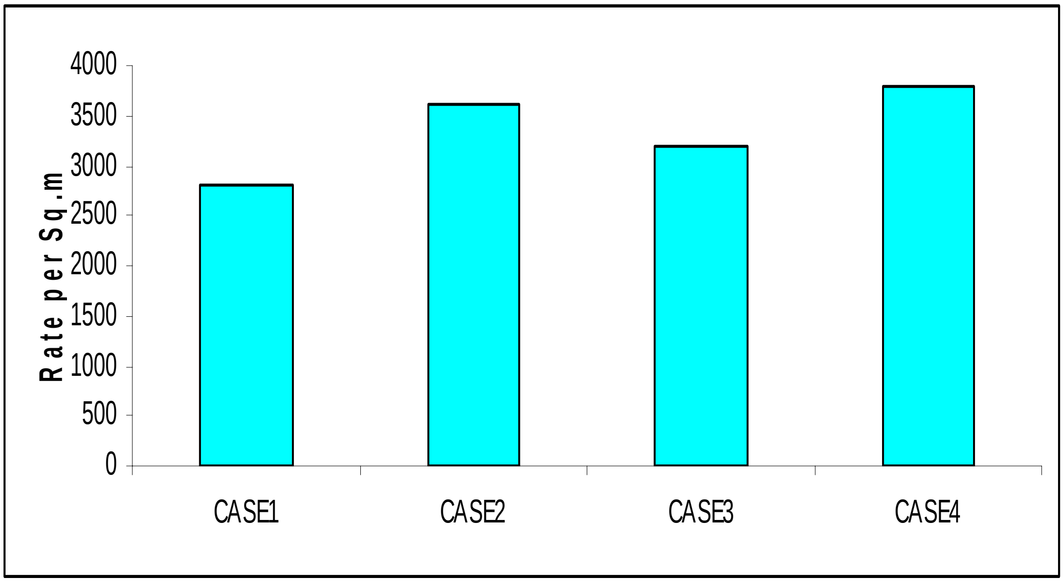
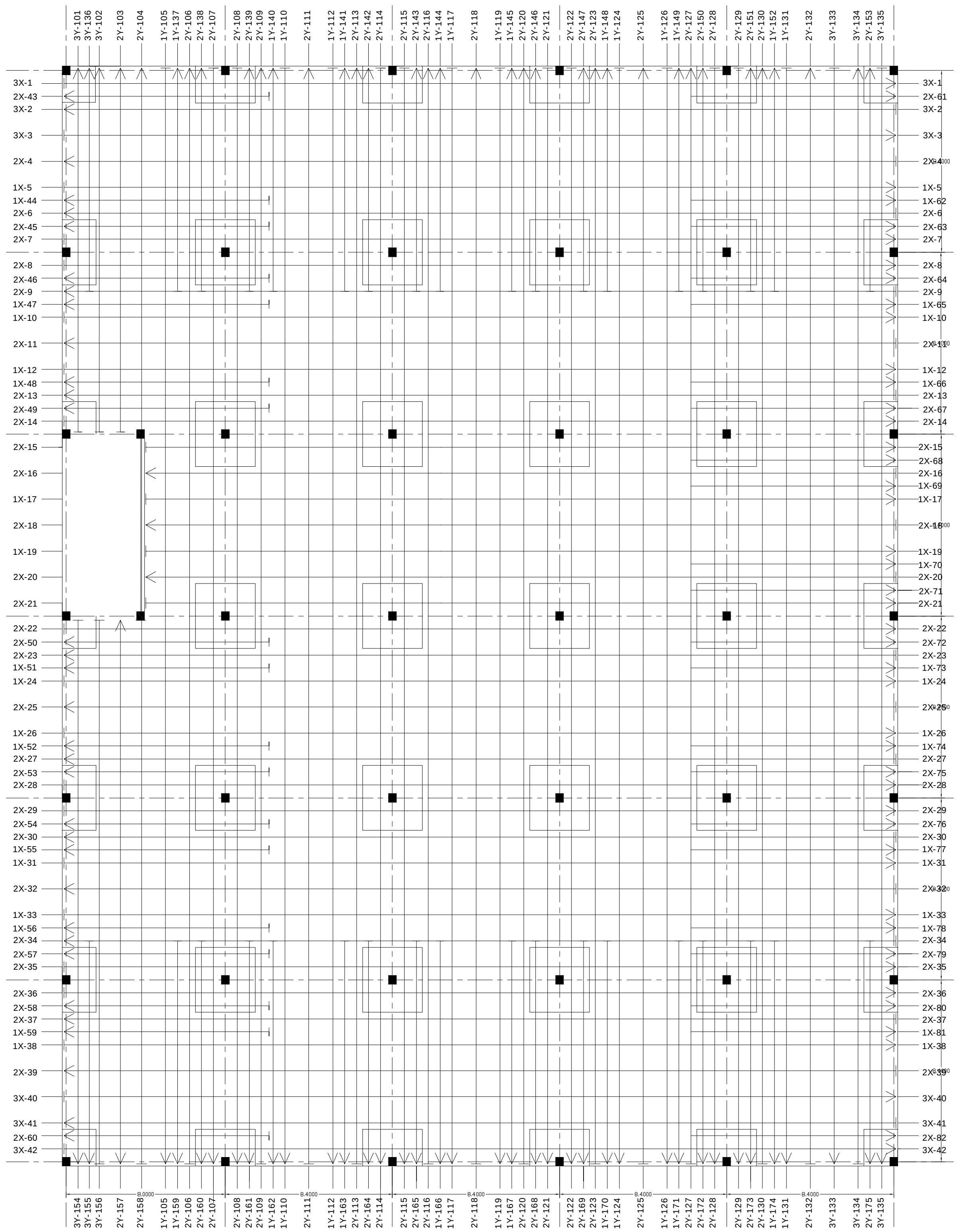


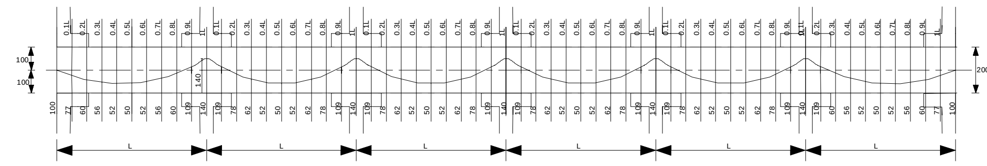
Fig. 7.5 Variation of rate for each floor system

1. From the economic point of view the post-tensioned flat slab is the most economical among all four floor systems and the reinforced concrete slab with reinforced concrete beam is the costlier one for this span.
2. If we consider the post-tensioned flat slab and reinforced concrete flat slab, the thickness of reinforced concrete flat slab is 12.5% greater and its cost is 27% greater than the post-tensioned flat slab.
3. From both post-tensioned floor system building the post-tensioned flat slab is more economical than the post-tensioned slab with reinforced concrete beams.
4. The quantity of prestressing steel is 4 Kg/m^2 for post-tensioned flat slab and 3.2 Kg/m^2 for post-tensioned slab with reinforced concrete beams i.e. the prestressing steel required for the post-tensioned flat slab is greater.
5. The reinforcing steel required for the post-tensioned flat slab and post-tensioned slab with reinforced concrete beam is 15 Kg/m^2 and 20.15 Kg/m^2 respectively.
6. The reinforcing steel is more in case of post-tensioned slab with reinforced concrete beams because the slab transfers the load on the beam and more loads is taken by the beams itself.

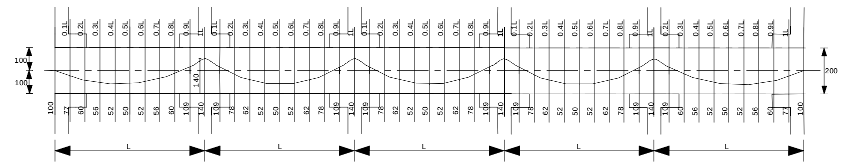
7. The reinforcing steel for the reinforced concrete flat slab is 41 Kg/m^2 while for the reinforced concrete slab and beam it is 40 Kg/m^2.
8. The amount of concrete required for a floor is more in case of post-tensioned slab with reinforced concrete beams while it is least for the post-tensioned flat slab floor system.
9. The floor to floor height available in case of post-tensioned flat and reinforced concrete flat slab is 2.65m while in case of post-tensioned slab with reinforced concrete beams and reinforced concrete slab and beams is 2.4m.
10. If we consider the period of construction for a floor it is less in case of post-tensioned flat slab than the other three cases as the post-tensioning allows the earlier removal of the formwork. In case of post-tensioned slab with reinforced concrete beams the formwork of slab can be removed earlier but the formwork for the reinforced concrete beams can not be removed earlier.
11. While estimating the cost of the each building the labour charges are not considered, as the time period reduce the labour charges will reduce in case of post-tensioned flat slab.
12. The wall load is considered on allover the floor (KN/m^2) for the post-tensioned building While analysis. So there is flexibility to the user to construct a wall wherever required in case of post-tensioning.



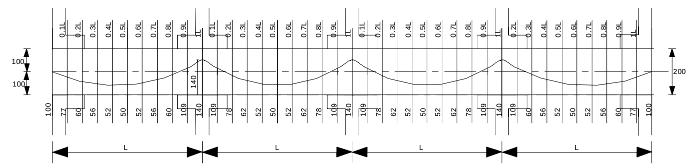
CABLE LAYOUT IN X & Y DIRECTIONS



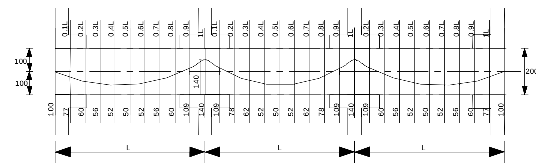
FOR TENDON Y-105 TO Y-135



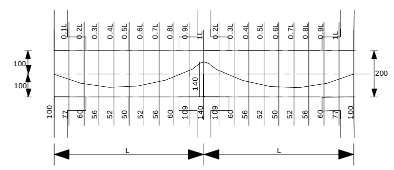
FOR TENDON X-1 TO X-14,
X-22 TO X-42



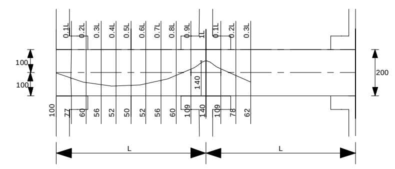
FOR TENDON X-15 TO X-21



FOR TENDON Y-154, Y-156
TO Y-158



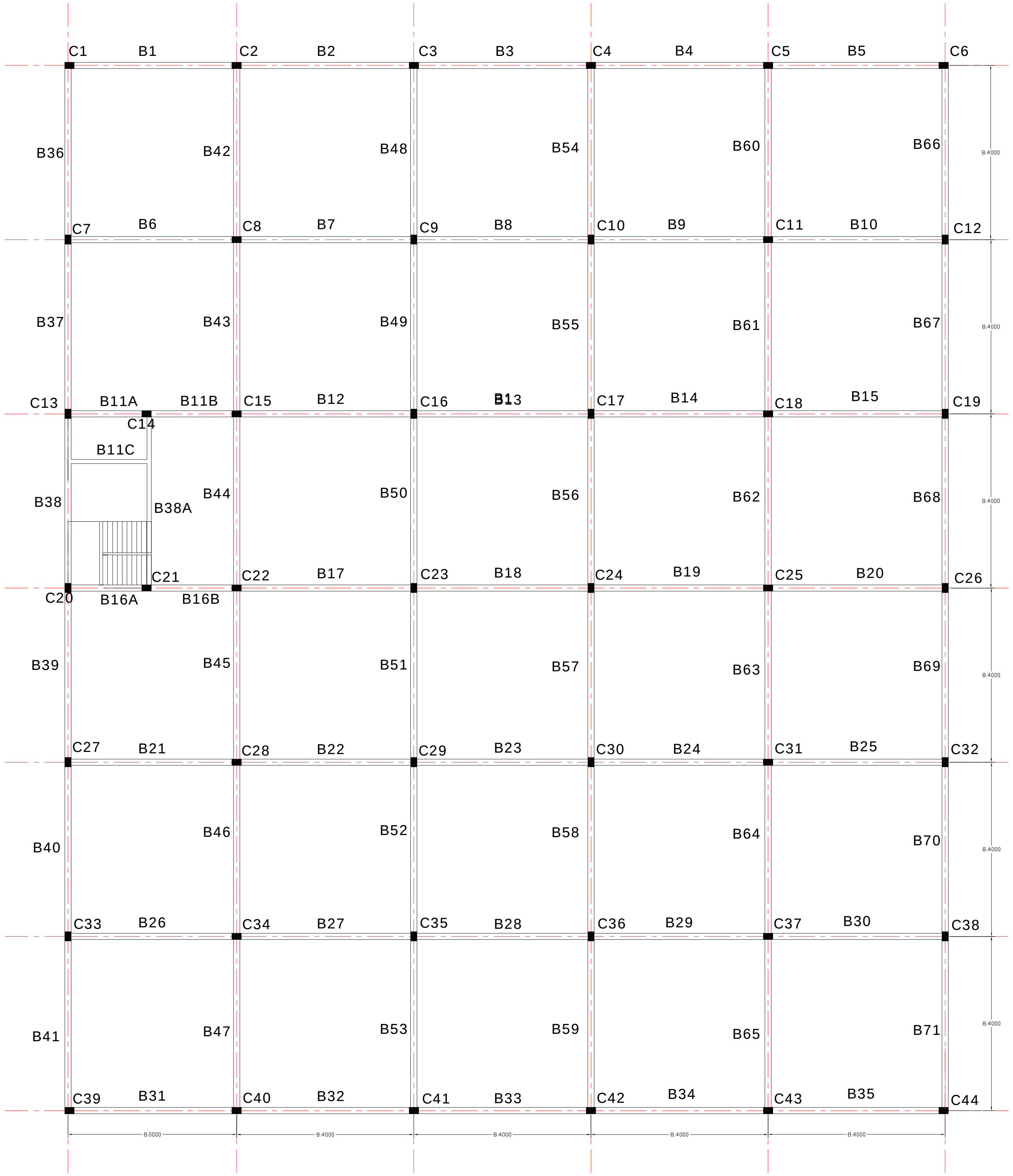
FOR TENDON Y-101
TO Y-104

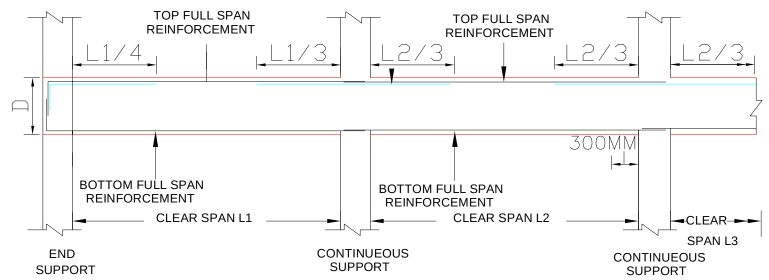


FOR TENDON Y-136 TO Y-153,Y-155,
Y-159 TO Y-175, X-43 TO X-60,
X-61 TO X-82

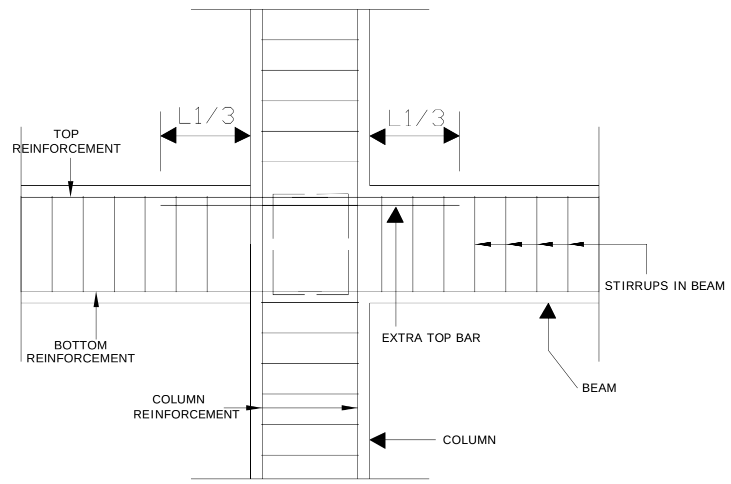
CABLE LAYOUT AND CALBE PROFILE FOR PT FLAT SLAB

CIVIL DEPARTMENT, INSTITUTE OF TECHNOLOGY, NIRMA UNIVERSITY
TITLE : CABLE LAYOUT AND CALBE PROFILE FOR PT FLAT SLAB
PROJECT : DESIGN OF POST-TENSIONED FLOORS
CLIENT : -
DWG.NO:SHEET 1 DRAWN BY: SANDEEP CHECK BY :
DATE : 12/04/2007 SCALE : 1:1 REVISION: R 0





TYPICAL DETAIL OF BEAM REINFORCEMENT

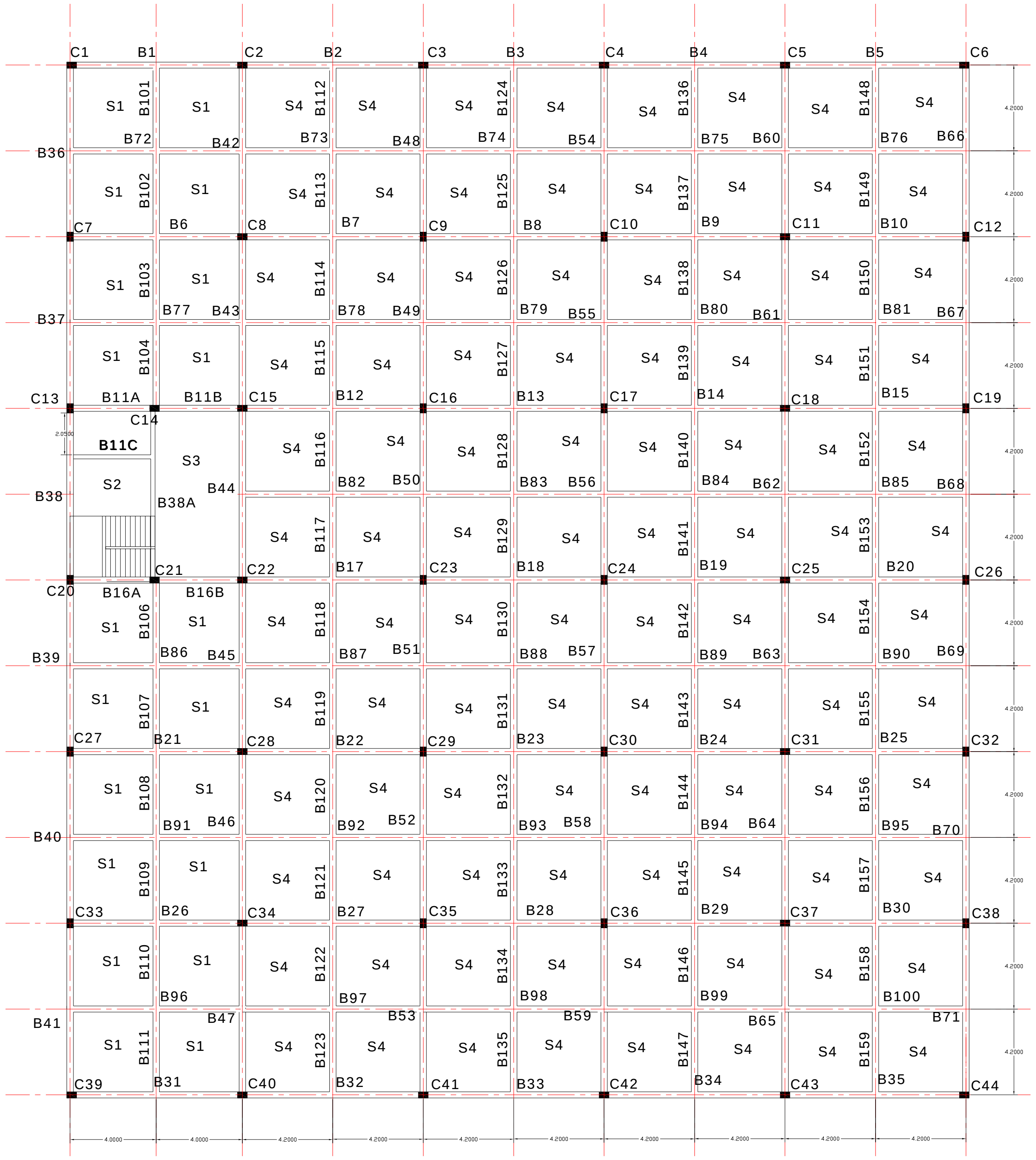


TYPICAL DETAIL OF BEAM COLUMN JUNCTION

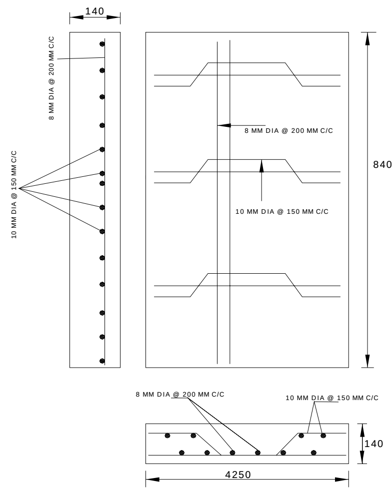
CIVIL DEPARTMENT, INSTITUTE OF TECHNOLOGY, NIRMA UNIVERSITY		
TITLE : PLAN, DETAILING OF BEAM AND COLUMN FOR PT SLAB WITH RCC BEAMS		
PROJECT : DESIGN OF POST-TENSIONED FLOORS		
CLIENT : -		
DWG.NO:SHEET 5	DRAWN BY: SANDEEP	CHECK BY :
DATE : 12/04/2007	SCALE : 1:1	REVISION: R0

PLAN AND DETAILING OF BEAM AND COLUMN

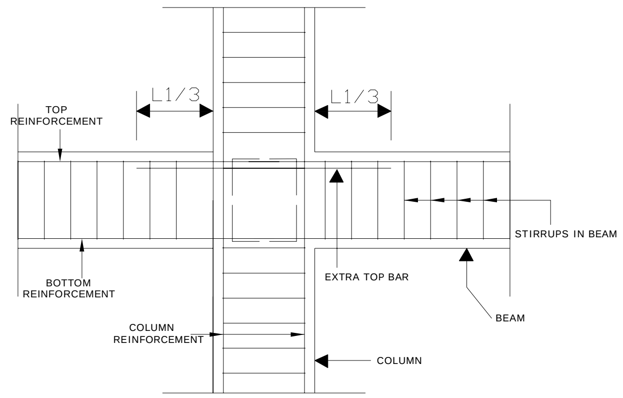
SHEET 5



PLAN



REINFORCEMENT DETAIL FOR ONE WAY SLAB



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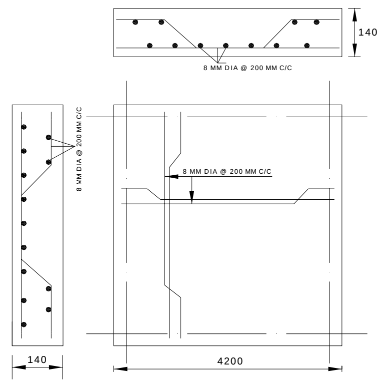
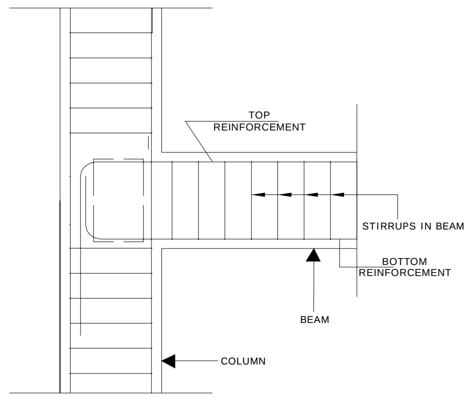
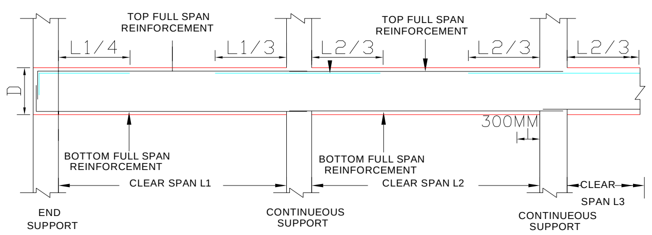


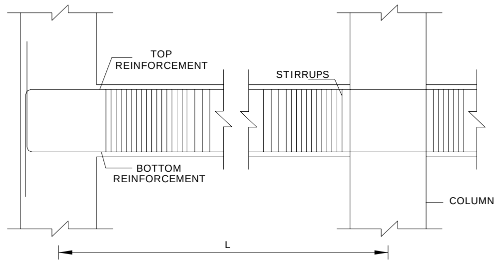
REINFORCEMENT DETAIL FOR TWO WAY SLAB



TYPICAL DETAIL OF BEAM EDGE COLUMN JUNCTION



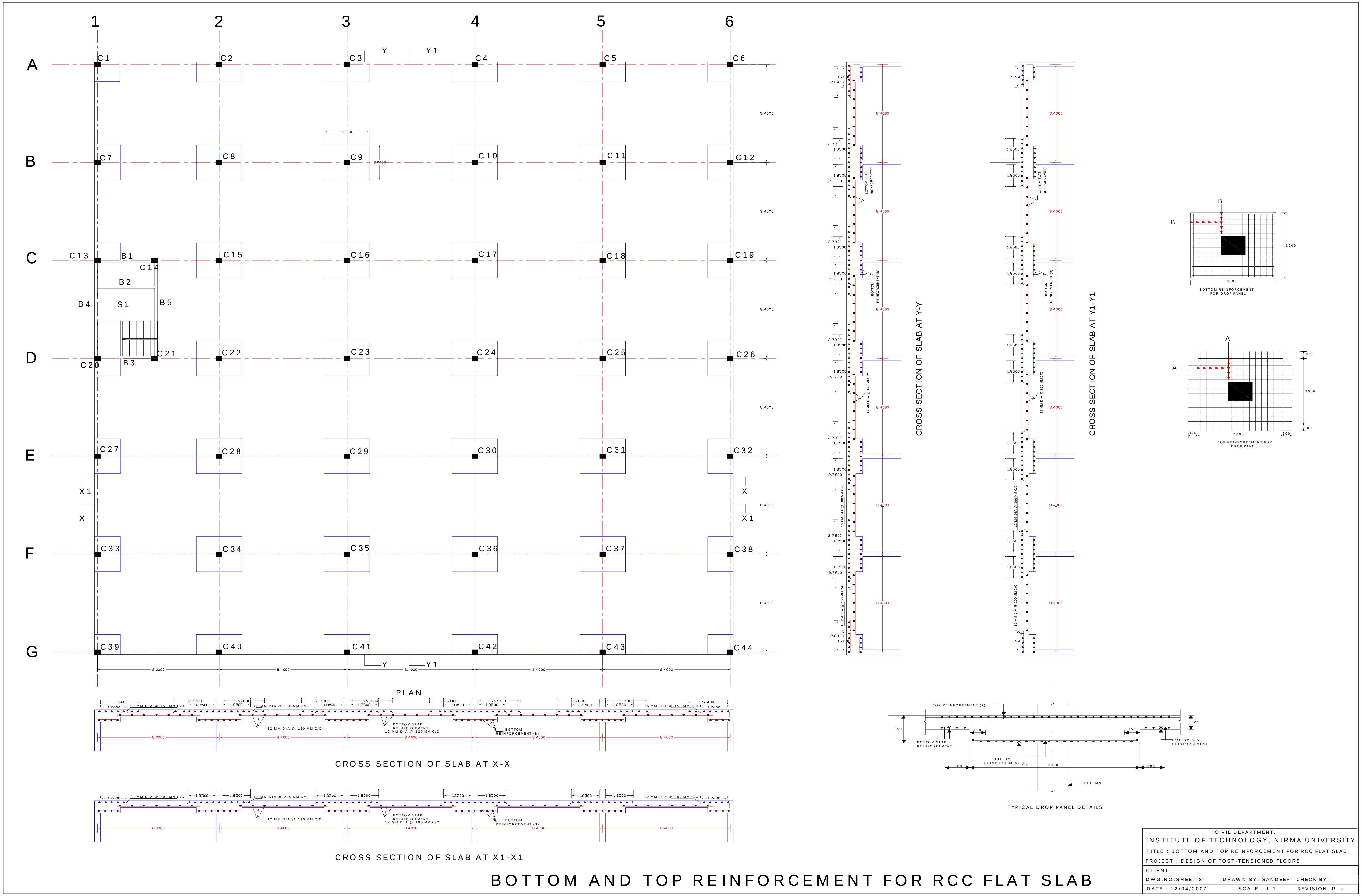
TYPICAL DETAIL OF BEAM REINFORCEMENT



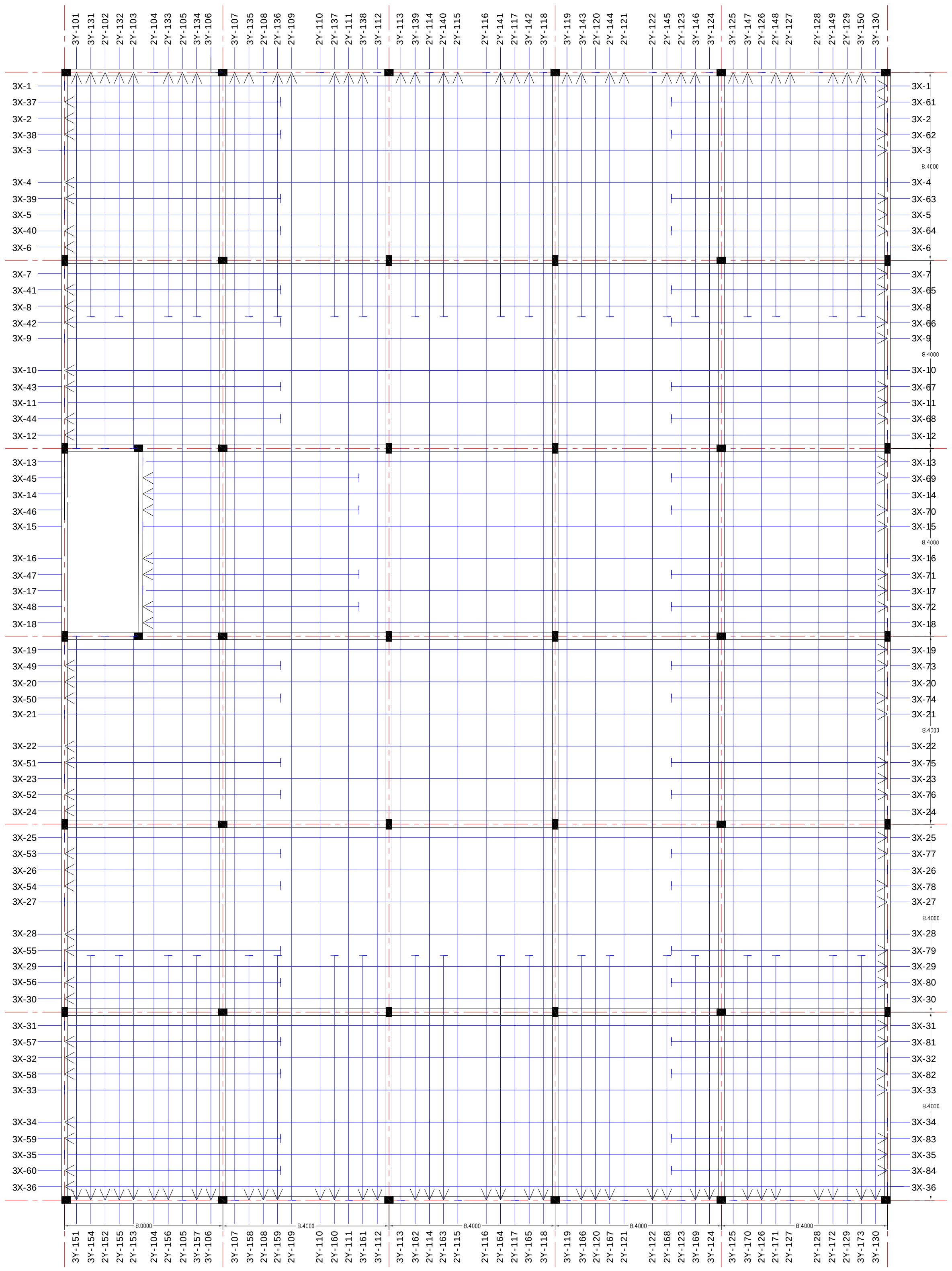
TYPICAL REINFORCEMENT DETAIL OF BEAM

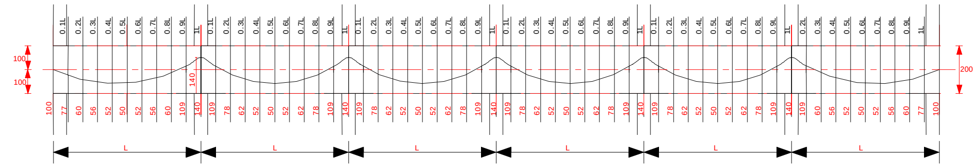
REINFORCEMENT DETAILS FOR RCC SLAB WITH RCC BEAMS

CIVIL DEPARTMENT, INSTITUTE OF TECHNOLOGY, NIRMA UNIVERSITY		
TITLE : REINFORCEMENT DETAILS FOR RCC SLAB WITH RCC BEAMS		
PROJECT : DESIGN OF POST-TENSIONED FLOORS		
CLIENT : -		
DWG.NO:SHEET 6	DRAWN BY: SANDEEP	CHECK BY :
DATE : 12/04/2007	SCALE : 1:1	REVISION: R0

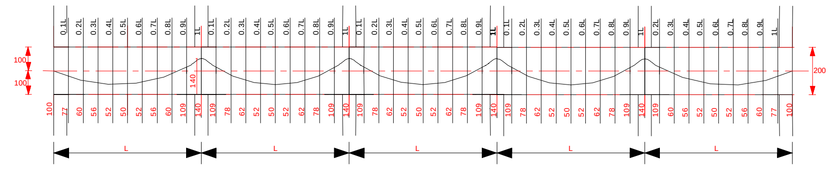

PLAN
CROSS SECTION OF SLAB AT X-X
CROSS SECTION OF SLAB AT X1-X1
CROSS SECTION OF SLAB AT Y-Y
CROSS SECTION OF SLAB AT Y1-Y1
BOTTOM REINFORCEMENT FOR DROP PANEL
TOP REINFORCEMENT FOR DROP PANEL
TYPICAL DROP PANEL DETAILS
TOP REINFORCEMENT (A)
BOTTOM SLAB REINFORCEMENT
BOTTOM REINFORCEMENT (B)
COLUMN
CIVIL DEPARTMENT,
INSTITUTE OF TECHNOLOGY, NIRMA UNIVERSITY
TITLE : BOTTOM AND TOP REINFORCEMENT FOR RCC FLAT SLAB
PROJECT : DESIGN OF POST-TENSIONED FLOORS
CLIENT : -
DWG.NO:SHEET 3 DRAWN BY: SANDEEP CHECK BY :
DATE : 12/04/2007 SCALE : 1:1 REVISION: R 0
BOTTOM AND TOP REINFORCEMENT FOR RCC FLAT SLAB

SHEET 3

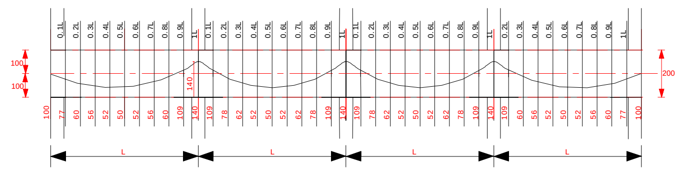




FOR TENDON Y-104 TO Y-130



FOR TENDON X-1 TO X-12,
X-19 TO X-36



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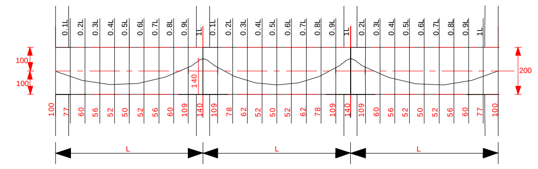

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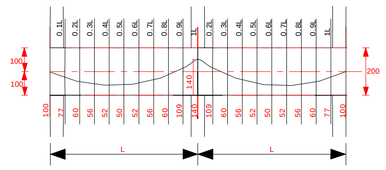

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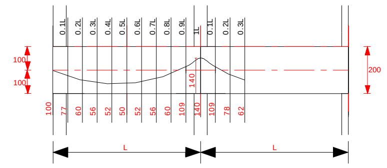


FOR TENDON Y-131 TO Y-153,Y-150,
Y-154 TO Y-173, X-37 TO X-82

CABLE LAYOUT AND CALBE PROFILE FOR PT SLAB WITH RCC BEAMS

CIVIL DEPARTMENT,
INSTITUTE OF TECHNOLOGY, NIRMA UNIVERSITY

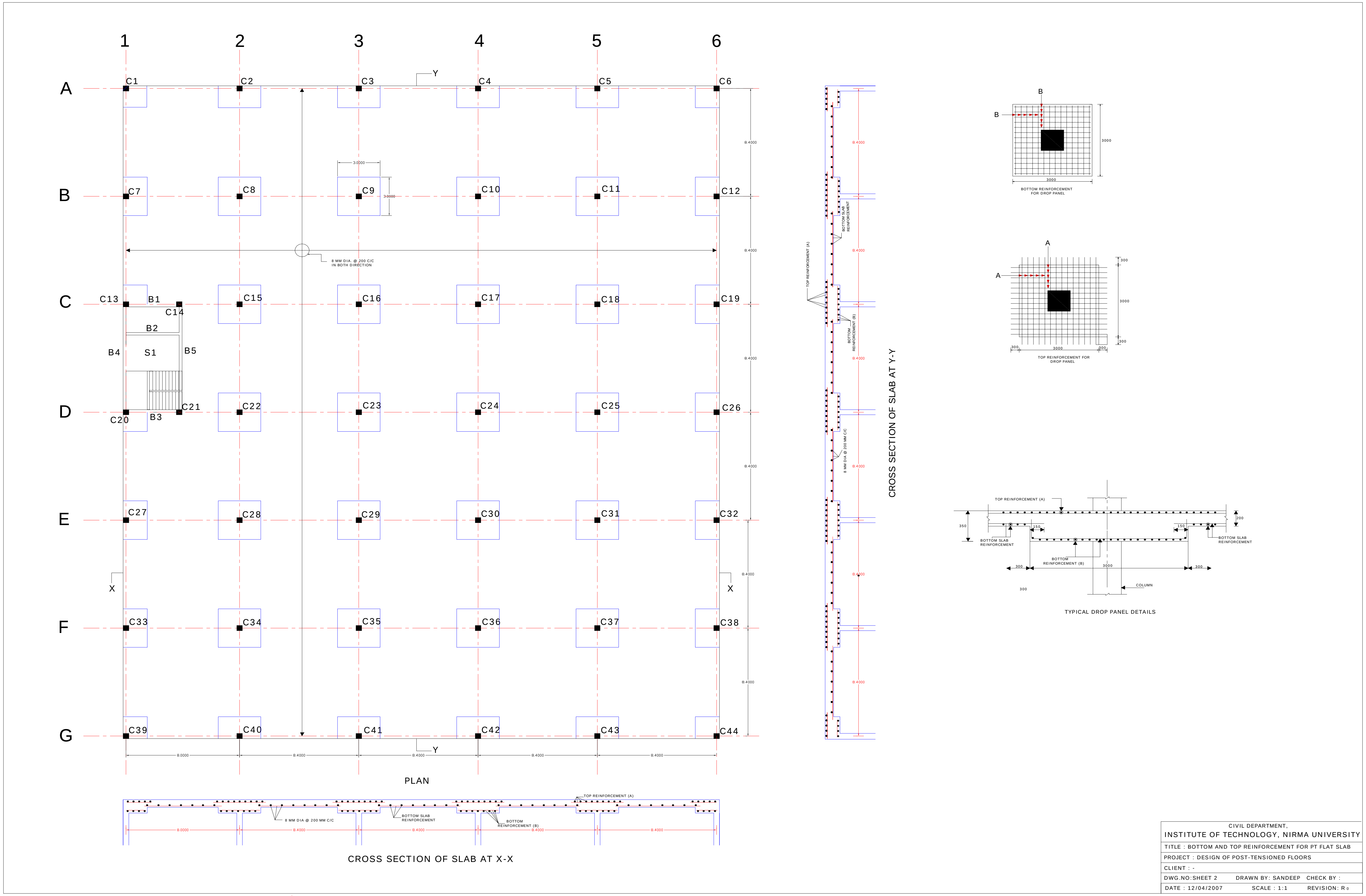
TITLE : CABLE LAYOUT AND CALBE PROFILE FOR PT SLAB WITH RCC BEAMS

PROJECT : DESIGN OF POST-TENSIONED FLOORS

CLIENT : -

DWG.NO:SHEET 4 DRAWN BY: SANDEEP CHECK BY :

DATE : 12/04/2007 SCALE : 1:1 REVISION: R 0



1
2
3
4
5
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A
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C
D
E
F
G
C1
C2
C3
C4
C5
C6
C7
C8
C9
C10
C11
C12
C13
B1
C14
C15
C16
C17
C18
C19
B2
B4
S1
B5
C20
B3
C21
C22
C23
C24
C25
C26
C27
C28
C29
C30
C31
C32
X
X
C33
C34
C35
C36
C37
C38
C39
C40
C41
C42
C43
C44
Y
Y
8 MM DIA. @ 200 C/C
IN BOTH DIRECTION
PLAN
TOP REINFORCEMENT (A)
8 MM DIA @ 200 MM C/C
BOTTOM SLAB REINFORCEMENT
BOTTOM REINFORCEMENT (B)
CROSS SECTION OF SLAB AT X-X
CROSS SECTION OF SLAB AT Y-Y
B
B
3000
3000
BOTTOM REINFORCEMENT FOR DROP PANEL
A
A
300
3000
300
TOP REINFORCEMENT FOR DROP PANEL
TOP REINFORCEMENT (A)
350
200
150
150
BOTTOM SLAB REINFORCEMENT
BOTTOM REINFORCEMENT (B)
BOTTOM SLAB REINFORCEMENT
300
3000
300
COLUMN
300
TYPICAL DROP PANEL DETAILS
CIVIL DEPARTMENT,
INSTITUTE OF TECHNOLOGY, NIRMA UNIVERSITY
TITLE : BOTTOM AND TOP REINFORCEMENT FOR PT FLAT SLAB
PROJECT : DESIGN OF POST-TENSIONED FLOORS
CLIENT : -
DWG.NO:SHEET 2
DRAWN BY: SANDEEP
CHECK BY :
DATE : 12/04/2007
SCALE : 1:1
REVISION: R0

SHEET 2

8. CONSTRUCTION PROCEDURE

8.1 GENERAL

The construction of a post-tensioned slab is broadly similar to that for an ordinarily reinforced slab. Differences arise in the placing of the reinforcement, the stressing of the tendons and in respect of the rate of construction.

The placing work (placing of reinforcement, tendons) consists of three phases. First, the bottom ordinary reinforcement of the slab and the edge reinforcement are placed. Then the ducts or tendons are positioned, fitted with supports and fixed in place. Third is the placing of the top ordinary reinforcement. The stressing of the tendons and the grouting in the case of bonded tendons represent additional construction operations as compared with a normally reinforced slab. Since, however, these operations are usually carried out by the prestressing firm; the main contractor can continue his work without interruption. A feature of great importance is the short stripping times that can be achieved with post-tensioned slabs. The minimum period between concreting and stripping of formwork (after stressing the tendons) is 48 to 72 hours, depending upon concrete quality and ambient temperature.

When the required concrete strength is reached, the full prestressing force can usually be applied and the formwork stripped immediately afterwards. Depending upon the total size, the construction of the slabs is carried out in a number of sections. The number of section n which the slab is constructed is depend upon the geometry of the structure, the dimensions, the planning, the construction procedure, the utilization of formwork material etc. The construction joints that do occur are subsequently subjected to permanent compression by the prestressing, so that, finally the behavior of the entire slab is the same throughout the total dimensions of the slab. The weight of a newly concreted slab must be transmitted to the slabs below the newly constructed slab through the formwork provided. As compared to the reinforced concrete slab the weight of the post-tensioned slab is usually less sothat the supporting system required to transfer this weight is not much and ultimately the cost of the supporting structure is also less.

8.2 FABRICATION OF THE TENDONS

There are two possible methods of fabricating cables

- Fabrication at the works of the prestressing firm and
- Fabrication by the prestressing firm on the site

The method chosen will depend upon the local conditions. The detail drawing are available for the post-tensioned slab to be constructed and from that drawings the strands are cut to the desired length, placed in the duct and, if appropriate, equipped with dead-end anchorages at the work place. The finished cables are then coiled up and transported to the site and then these ducts are placed at the position according to the drawings. In fabrication on the site, the cables can either be fabricated in exactly the same manner as at works, or they can be pushed first through the ducts and then these tendons are cut out. In the latter method, the ducts are initially placed empty and the strands are pushed through them subsequently during the placing of the tendons. If the cables have stressing anchorages at both ends, the pushing of the tendon can even be carried out after concreting (except for the cables with flat ducts).

In case of the unbonded tendons, the fabrication of monostrands tendons is usually carried out at the works of the prestressing firm but if required, can also be carried out on site. The monostrands are cut to length and if necessary fitted with the dead-end anchorages. They are then coiled up and transported to site. The stressing anchorages are fixed to the formwork. During placing, the monostrands are then threaded through the anchorages.

8.3 CONSTRUCTION PROCEDURE FOR POST-TENSIONING

When the slab is constructed by using the post-tensioning method, the operation that are normally carried out, are as follows

- Erection of supporting formwork to the slab to be constructed.
- Fitting of end formwork and placing of stressing anchorages.
- Marking of the position and the eccentricity of the duct or tendon on the formwork in both X and Y directions as per drawings.
- Placing of bottom and edge reinforcement as per the drawings.

- Placing of tendons or, if applicable, empty ducts according to the tendon layout drawing.
- Supporting of tendons or empty ducts with supporting chairs according to support drawing i.e. to provide the calculated eccentricity to the tendon to form a required curve of the tendon.
- Placing of top reinforcement.
- After placing the bottom reinforcement, tendon and top reinforcement the concreting of the section of the slab is carried out.
- Removal of end formwork and forms for the stressing block-outs.
- Stressing of cables according to stressing programme given by the designer. The elongation of each tendon is given in the drawing. According to the elongation and the stressing force the stressing of the tendon is carried out.
- Stripping of slab supporting formwork.
- Grouting of cables and concreting of block-outs.

If the bonded tendons are used then the above procedure is adopted directly, otherwise for the unbonded tendons only the last step i.e. grouting of cable should be omitted from the above procedure. Experience has shown that those activities that are specific to prestressing (post-tensioning) i.e. placing of cable, providing eccentricity, stressing of tendons, etc. should be carried out with more attention by the prestressing firm. While doing the post-tensioning work the following aspects should be kept in mind.

Placing and supporting of tendons

The placing sequence and the supporting of the tendons are carried out in accordance with the placing and support drawings (sheet 1 and 3). Placing of the tendons is the additional operation for the post-tensioned slab as compared to the reinforced concrete slab, and it is of more importance. The drawing for the tendon layout, tendon profile and the ordinary reinforcement (bottom and top reinforcement) should be prepared separately for the post-tensioned slab. Sothat for a post-tensioned slab there is always two drawings. The sequence in which the tendons are to be placed must be carefully considered, so that the operation can take place smoothly. Normally a sequence allowing the tendons can be found without any difficulty. To assure the accuracy in the construction of post-tensioned slab, good coordination is required between all the installation contractors (electrical, heating, plumbing etc.) and the organization responsible for the tendon layout i.e. prestressing firm. The eccentricity provided for each tendon should be maintained sothat corresponding care is also necessary in concreting.

Stressing of tendons

For stressing the tendons, a properly secured scaffolding 0.50 m wide and of 2 KN/m^2 load-bearing capacity is required at the edge of the slab. For the jacks used there is a space requirement behind the anchorage of 1 m along the axis and 120 mm radius about it. All stressing operations are recorded for each tendon. The primary objective is to stress the tendon to the required load and the elongation is measured at the applied load. Elongation measured is compared with the calculated value.

Following are the some of the pictures showing the construction procedure of the post-tensioned slab, anchorage length, guide ring, balloon provided at the dead end of the cable, reinforcement at the drop panel, stressing end, vent pipe for the grouting after stressing the tendons, chairs provided to the duct for obtaining the eccentricity etc.

Fig. 8.1 Stressing end for the cables

Fig. 8.2 Guide ring, balloon and anchorage for the cables

Fig. 8.3 Laying out tendon and the drop panel reinforcement

Fig. 8.4 Placing of tendon and slab reinforcement

Fig. 8.5 Tendon duct and chair provided for eccentricity

Fig. 8.6 Dead end and beam reinforcement

Fig. 8.7 Reinforcement at the drop panel

Fig. 8.8 Vent pipe provided for grouting

9. SUMMARY AND FUTURE SCOPE OF WORK

9.1 SUMMARY

The detail design of the post-tensioned flat slab is done here. To understand the concept of the post-tensioned flat slab and effect of the different parameters while design of it, a parametric study of the post-tensioned flat slab of span 7m to 12m by varying the span at an interval of 0.5m is done. While doing this study the flat slab with drop and without drop is considered. In this the bending moment at midspan of flat slab with drop is less than the flat slab without drop panel and at the support the bending moment is greater in the slab with drop panel. This results in the reduction of the prestressing steel at the mid span and increase in the normal reinforcement at the support of the span.

For the design of the post-tensioned flat slab the equivalent frame method and the load balancing method are used. The design is carried out by both the methods and results obtained are almost similar by both the methods. The shear check and deflection check is of great importance in case of the post-tensioned flat slab. The cable profile used here is the parabolic profile and it is continuous through out the total span. The secondary moments due to prestressing are induced in the post-tensioned slab at the supports for which care should be taken while design of the slab.

If we consider the construction procedure for the post-tensioned flat slab it is similar to that of the reinforced concrete flat slab except that the layout, placing of tendon and the stressing of the tendons. The stressing of the tendon is done with the help of jack by applying the force calculated by the designer. Once the 70 to 80% stress is gained by the concrete the stressing is done and after the stressing the formwork can be removed. Generally the stressing operation is done on the third or fourth day from the casting of slab.

For the application of the post-tensioned design in the construction field, the detail design of the office building (G+4) is carried out by considering four different floor systems. The same building is designed by post-tensioned flat slab, reinforced concrete flat slab, post-tensioned slab with reinforced concrete beams and reinforced concrete slab and beams. These four cases are analyzed, designed and the total quantities required for the building are calculated. The summary of the total quantities and rate per square meter for the typical floor of each case are as per the following table.

Table 9.1 Summary of the quantity and rate for different cases

Case	Concrete (m^3)	Reinforcing steel (Kg)	Prestressing steel (Kg)	Rate per sq.m (Rs.)
PT flat slab	507.52	31659.56	8400	2800
RCC flat slab	549.69	85550.78	-	3600
PT slab + RCC beams	641.33	42271	6720	3200
RCC slab + RCC beam	626.31	86701.2	-	3800

Note: The quantities and the rate include only slab, beam, column and the formwork.

9.2 CONCLUSION

1. For the construction of slab the floor system provided with post-tensioned flat slab is the more economical than the other floor systems.
2. Post-tensioned flat slab allows more span to depth ratio so that the thickness of the slab is small resulting in the more floor to floor to height or it reduces the total height of the whole building. This is of great importance in the case of high rise building.
3. In the post-tensioned flat slab earlier stripping of formwork increases the rate of the construction due to which total construction period of the building decrease and finally resulting in the saving of the overall cost of the structure.
4. The quantity of prestressing steel is 4 Kg/m^2 and the reinforcing steel is 16 Kg/m^2 for the post-tensioned flat slab while it is more in case of the other floor system (reinforced concrete flat slab, post-tensioned slab with reinforced concrete beams and reinforced concrete slab and beams).
5. The quantity of the prestressing steel is less is case of post-tensioned slab with reinforced concrete beams than the post-tensioned flat slab as more load is

taken by the beams. But for the normal reinforcing steel the case is inverse and it results in the economy of the post-tensioned flat slab.

6. During the placing of the tendon for the post-tensioned slab the care must be taken for the sequence of the tendon given in the design drawing.
7. The eccentricity of the cable given in the drawing should be maintained throughout the slab and for that the attention should be given at the time of casting of the slab. This is the responsibility of the prestressing firm. If the eccentricity of the cable is not maintained the profile of the cable change and it may lead to the failure of slab constructed.
8. Where the large span are required, the post-tensioned flat slab can be used effectively and economically, because there is no limit for the construction of wall anywhere and this gives the flexibility to the user.

9.3 FUTURE SCOPE OF WORK

- In the present report the parametric study is done by considering the square panel of the slab, further it is extended for the slab panel by varying the L/B ratio. The lateral load is not consider while design of post-tensioned flat slab for the parametric study, so this study is carried out by considering the lateral loads for the flat slab.
- The study of the post-tensioned beams for the different span with continuous beams and for the simply supported beams by considering the different parameters of design.

The detail design is carried out here so it is extended for the preparation of computer programme for the design procedure. The study of the post-tensioned flat slab by considering the finite element method and preparation of computer programming by considering different factors such as with drop panel, without drop panel, with and without column head, loading to be considered i.e. for live load the pattern loading should be used or not and the other design parameters.

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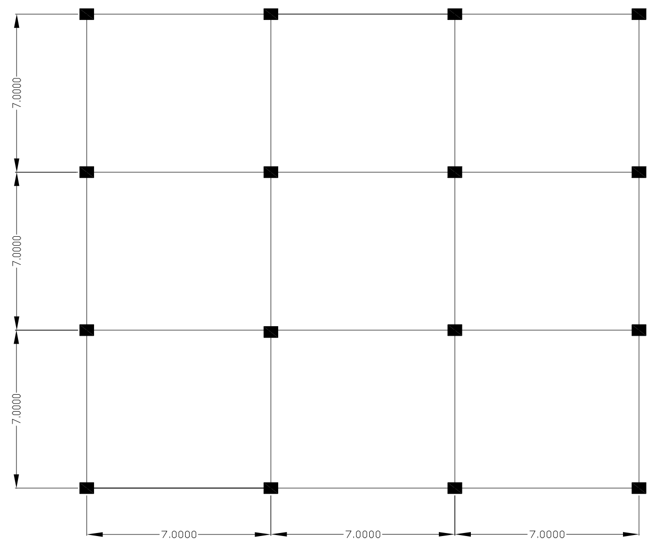
APPENDIX A

DESIGN OF POST-TENSIONED SLAB

A.1 DESIGN OF POST-TENSIONED FLAT SLAB

Data Given



PLAN

Span 1 (X-Dir) = 7 m

Span 2 (X-Dir) = 7 m

Span 1 (Y-Dir) = 7 m

Span 2 (Y-Dir) = 7 m

Tendon diameter = 12.70 mm

Area of tendon = A_p = 98.71 mm^2

F_{pu} = 1862 N/mm^2

Long term losses = 207 N/mm^2

E_s = 195000 N/mm^2

E_c = 29580.40 N/mm^2

μ = 0.35

k = 0.0015

Creep coefficient Φ = 1.60

Column sizes -

Exterior column = 350 mm x 300 mm

Interior column = 350 mm x 500 mm

Height of the floor = 3 m

F_c = 35 N/mm^2

F_y = 415 N/mm^2

Loads -

Dead Load = Self weight of the Slab

Superimposed dead load = 1.00 KN/m^2

Live Load = 2.00 KN/m^2

Slab Thickness -

Let us consider slab thickness @ L/45

Ratio of the span = 45

Longitudinal direction = 155.56 mm

Transverse Direction = 155.56 mm

Take the slab thickness = 170 mm

A = 1190000 mm^2

Z = 33716666.67 mm^3

I = 2865916667 mm^4

Load Calculation -

Dead Load due to self weight of slab = 4.25 KN/m^2

Superimposed dead load = 1.00 KN/m^2

Live Load = 2.00 KN/m^2

Total Service Load = DL + LL = 7.25 KN/m^2

Total Ultimate Load = 1.5 DL + 1.5 LL = 10.875 KN/m^2

Service Load Design -

A force corresponding to an average compression stress of 1.03N/mm2 with maximum parabolic tendon profile will be used for the initial estimate of balanced load. Then ,

Fe = 175.1 KN/m

Force = 1759.61 KN

Calculation of losses -

(a) Loss due to elastic deformation of concrete -

Loss of stress = $\alpha_e \times f_c$

Where, $\alpha_e = E_s / E_c$

α_e = 6.59

f_c = P / A + P x e x y / I

f_c = 4.65 N/mm^2

Loss of stress = 30.65 N/mm^2

(b) Loss due to shrinkage of concrete -

Loss of stress = ε_{cs} x E_s

Where, ε_{cs} = 200 x 10^{-6} / log10(t+2)

t = Age of concrete at transfer in days

t = 7 day

ε_{cs} = 0.0002096

Loss of stress = 40.87 N/mm^2

(c) Loss due to creep of concrete -

Loss of stress = Φ * f_c *αe

Loss of stress = 49.04 N/mm^2

(d) Loss of stress due to friction -

Loss of stress = f_{pu} * e-(μα + k_x)

For the parabola the equation is given by

y = 4e/L^2 * x(L - x)

Slope at the ends (at x = 0) = dy/dx = 4e / L

Slope = 0.034

Cumulative angle between tangents,

α = 0.0685

For the small values of (μα + k_x),

Loss of stress = f_{pu} x (μα + k_x) With reference to IS 1343: 1980

Loss of stress = 64.24 N/mm^2

(e) Loss due to relaxation -

According to IS 1343;1980, Table 4,

Relaxation loss = 70 N/mm^2

Total loss of stress = 254.81 N/mm^2

Effective force per tendon = 103574.188 N

For the bay in longitudinal direction the number of tendons required

= 16.98

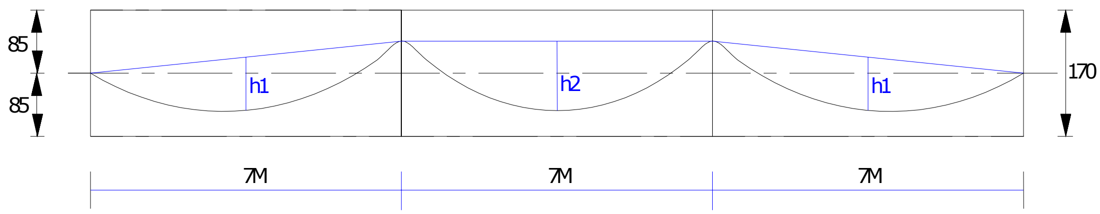
Say 17 Tendons

Then,

F_e = 251.53 KN/m

F/A =1.04 N/mm^2

Tendon Profile -



Span 1 &3,

Bottom cover = 44.00 mm

Top cover = 25.00 mm

Span 2,

Bottom cover = 25.00 mm

Top cover = 25.00mm

For Span 1 & 3,

h_1 = 71.00 mm

w_{bal} = 8Fh / L^2 = 2.92 KN/m^2

Net load causing bending,

w_{net} = 4.33 KN/m^2

For span 2,

h_1 = 120.00 mm

w_{bal} = 8Fh / L^2 = 4.93 KN/m^2

Net load causing bending,

w_{net} = 2.32 KN/m^2

Equivalent Frame Properties -

(a) Equivalent Columns -

The stiffness of column,

K_c = 4EI / (L - 2h)

Where h is the slab thickness.

For Exterior column -

$I = 787500000 \text{ mm}^4$

$E_{col} / E_{slab} = 1$

$K_c = 1280487.805$

Stiffness of a joint = 2560975.61

Torsional stiffness of column line K_t, is calculated as

$C = (1 - 0.63x / y) * x^3 * y /3$

Where,

$x = 170.00\text{mm}$ and $y = 300.00\text{mm}$

$C = 315905900$

$K_t = \Sigma 9*C*E / [L^2* (1 - c^2/L^2)3]$

$K^2 = 947461.096$

Equivalent column stiffness is obtained as

$1/K_{ec} = 1/K_c + 1/K_t$

$K_{ec} = 691597.1304$

For Interior column -

$I = 3645833333 \text{ mm}^4$

$E_{col} / E_{slab} = 1$

$K_c = 5928184.282$

Stiffness of a joint = 11856368.56

Torsional stiffness of column line K_t, is calculated as

$C = (1 - 0.63x / y) * x^3 * y /3$

Where,

$x = 170.00 \text{ mm}$ and $y = 500.00 \text{ mm}$

$C = 643439233.3$

$K_t = \Sigma 9*C*E / [L^2* (1 - c^2/L^2)3]$

$K_t = 1929795.047$

Equivalent column stiffness is obtained as

$1/K_{ec} = 1/K_c + 1/K_t$

$K_{ec} = 1659661.236$

(b) Slab Stiffness -

Slab stiffness is given by,

$K_s = 4*E*I / (L_1 - c_1/2)$

Where, L_1 is centre line span

c_1 is column depth

At Exterior column -

K_s = 1673527.981

At Interior column - span 1 &3

K_s = 1698320.988

At Interior column - span 2

K_s = 1698320.988

(c) Distribution Factor -

Distribution factor for analysis by moment distribution at exterior joint is

Slab distribution factor = $K_s / (K_s + K_{ce})$

Slab distribution factor = 0.71

At interior joints for span 1 & 3,

Slab distribution factor = 0.33

At interior joints for span 2,

Slab distribution factor = 0.33

Moment distribution -

For span 1 & 3, net load FEM = $wL^2 / 12$ = 17.70 KNm

For span 2, net load FEM = $wL^2 / 12$ = 9.48 KNm

Table A1 Moment distribution along the span

Moment distribution			
0.71	0.33	0.33	DF
-17.70	-17.70	-9.48	FEM
12.58	2.74	-2.74	DIST.
-1.37	-6.29	1.37	CO
0.97	2.55	-2.55	DIST.
-1.28	-0.49	1.28	CO
0.91	0.59	-0.59	DIST.
-5.89	-18.59	-12.71	

Check net tensile stresses at the face of column -

Moment at column face is centre line moment

M_{max} = -8.20 KNm

Z = 5250000 mm^3

Then, f_t = M / Z - F / A

f_t = 0.51 N/mm^2

< 2.66 N/mm^2 HENCE OK

Check midspan tensile stresses -

M_{max} = wL^2 / 8 - M = 1.51

f_t = M / Z - F / A

f_t = -0.76 N/mm^2

< 0.99 N/mm^2

By the requirements of ACI code, section 18.9, when stress of $2\sqrt{F_c}$ is exceeded the entire tensile force must be replaced by mild reinforcing at a stress of $F_y/2$

f_c = M / Z + F / A

f_c = -1.34 N/mm^2

< 15.75 N/mm^2

Then,

y = -225.67 mm

T = 25800.85 N

A_s = 124.3414679

Ultimate Flexural capacity -

(a) Calculation of design moment -

Design moments for statically indeterminate post-tensioned members are determined by combining frame moments due to factored dead load and live load with secondary moments induced into the frame by tendons.

Balanced load moments for span 1 & 3,

Balanced load FEM = 11.91 KNm

Span 2,

Balanced load FEM = 20.12 KNm

Table A2 Moment distribution along the span

Moment distribution			
0.71	0.33	0.33	DF
-11.91	-11.91	-20.12	FEM
8.46	-2.74	2.74	DIST.
1.37	-4.23	-1.37	CO
-0.97	0.95	-0.95	DIST.
-0.48	0.49	0.48	CO
0.34	0.00	0.00	DIST.
-3.19	-17.44	-19.23	

Since load balancing accounts for both primary and secondary moments directly. Secondary moments can be found as

$$M_{bal} = M_1 + M_2$$

Sothat, $M_2 = M_{bal} - M_1$

And, $M_1 = F * e$

Where e is the eccentricity of the tendon.

At exterior column, e = 0

$M_1 = 0$

Secondary moment $M_2 = M_{bal} = 3.19$ KNm

At interior column span 1 & 3,

e = 60.00 mm

$M_1 = 15.09$ KNm

$M_2 = 2.35$ KNm

Span 2,

e = 60.00mm

$M_1 = 15.09$ KNm

$M_2 = 4.14$ KNm

Factored load moments -

For Span 1 & 3,

FEM = 44.41 KNm

For Span 2,

FEM = 44.41 KNm

Table A3 Moment distribution of factored Load along the span

Moment distribution Factored Load			
0.71	0.33	0.33	DF
-44.406	-44.406	-44.406	FEM
31.556	0.000	0.000	DIST.
0.000	-15.778	0.000	CO
0.000	5.259	-5.259	DIST.
-2.630	0.000	2.630	CO
1.869	0.877	-0.877	DIST.
-0.438	-0.934	0.438	CO
0.311	0.458	-0.458	DIST.
-13.738	-54.525	-47.932	

Combine factored load and secondary moments to obtain design moments are as follows –

Table A4 Combined factored and secondary moments

	Span 1		Span 2
Factored load Moments	-13.738	-54.525	-47.932
Secondary Moments	3.187	2.346	4.141
Design column Moments	-10.551	-52.179	-43.790
Moment reduction to face V_c / 3	1.3	4.5	5.4
Design moments at the critical section	-9.251	-47.679	-38.390

Calculate design Midspan moments -

Span 1,

Shear at the external point -

V_{ext} = wL / 2 - M/L = 32.236 KN/m

Shear at the internal point -

V_{int} = wL / 2 + M/L = 43.889 KN/m

Point of zero shear and maximum moment

x = 2.96m

End span moments

Moment at the point of zero shear-

M = -47.77 KNm

End moment = -13.738 KNm

34.039 KNm

Secondary moment, M_2 = 3.187 KNm

M_{max} = 37.226 KNm/m

Span 2,

V = 38.063 KN/m

Positive Moment = 66.609 KNm/m

End moment = -47.932 KNm

18.678 KNm

Secondary moment, M_2 = 4.14 KNm

M_{max} = 22.819 KNm/m

(b) Calculation of flexural strength -

The minimum amount of mild steel at the immediate column zone regardless of service load stress conditions or strength, unless more than the minimum is required for ultimate flexural capacity, is given by

A_s = 0.00075 x h x L

The initial check of flexural strength will be made by considering this steel.

A_s = 892.5 mm^2

Provide 8-12 mm Φ

Spacing should be at 200.00mm on centre sothat bars are placed within a width of column + 1.5 slab thickness on each side of column.

For average 1m strip,

A_s = 127.50 mm^2

Calculate the stress in tendon at ultimate,

f_{ps} = f_{se} + $f_c/100\rho_p$ + 10Ksi   Ref. - ACI code equation 18-4

ρ_p = A_{ps} / bd

Where, A_{ps} is area of tendons, b is width of slab and d is effective depth of slab.

ρ_p = 0.00165

f_{se} = 1096.4 N/mm^2

f_{ps} = 1378.40 N/mm^2

F_{su} = f_{ps} x A_{ps} / L = 330.438 KN/m

F_u = 53.568 KN/m

Total force F = F_{su} + F_u

F = 384.006 KN/m

Depth of compression block,

a = F / (0.85*b*f_c) = 12.91 mm

(d - a/2) = 138.55 mm

Moment capacity at the column centre line -

M_u = 0.9 * (d - a/2) * F

M_u = 47.882 KNm/m

Calculate the available inelastic moment redistribution at column

Allowable redistribution = 20 % (1-w_p/0.3)

$\Sigma w = 0.094 < 0.2$ Ref. - ACI code equation 18-10

R = 13.72%

$M_R = 6.570$ KNm

Since the midspan 2 requires 4-16mm bars from service load consideration, the flexural strength is also 47.882 KNm. The required moment capacity is 22.819 KNm which leaves a 25.063 KNm available to accommodate moment redistribution from support section. If a moment of 7.50 KNm is redistributed, then

(- M) = -30.890 < 47.882 KNm

(+ M) = 30.319 < 47.882 KNm HENCE OK

Thus minimum rebar and tendons are adequate for the strength.

Ultimate Shear Strength -

(a) Shear at Exterior column -

Vertical shear at the exterior column calculated as above is

V = 32.236 KN/m

Total shear at the exterior column,

V = 225.65 KN

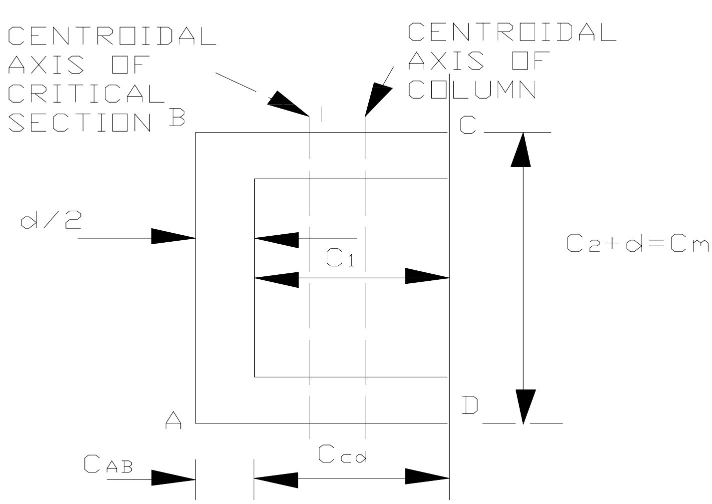
Assume exterior skin is masonry & glass averaging 6.00 KN/m

V = 63.00 KN

Total shear = Vu = 288.65 KN

(b) Moment transfer -

The critical section properties are calculated as follows. The critical section for shear is taken at d/2 from the face of column.



$d = 145.00$ mm

$C_1 = 300$ mm $C_2 = 350$ mm

$C_m = 495.00$ mm

$C_t = 372.50$ mm

$A_c = d\,(C_m + 2C_t) = 179800\ mm^2$

$C^2 = Ct^2 * d / Ac = 111.90$ mm

$C_{CD} = C_t - C_{AB} = 260.60$ mm

$g = C_{CD} - C_1/2 = 110.60$ mm

$\alpha = 1 - [1/(1 + 2/3(C_m/C_t)1/2)]$

$\alpha = 0.435$

$Jc = d*C_t^3/6 + C_t*d^3/6 + C_m*d* C_{AB}^2 + 2*C_t*d*(C_t/2 - C_{AB})2$

$Jc = 2934256513\ mm^4$

Total bay moment at column centre line -

$Mu = 21.91$ KNm

Moment transferred by eccentricity of shear reaction -

$Vg = 24.96$ KNm

Net moment to be transferred,

$Mt = Mu- Vg = -3.05$ KNm

Amount of moment to be transferred by shear $= \alpha * M_t = -1.32$ KNm

Now,

$V_c = V_u / A_c + \alpha M_t*C_{AB} / J_c$

$V_c = 1.829\ N/mm^2$

Allowable shear stress -

$V_c = 3.5*\sqrt{\ } f_c + 0.3f_{se}$

$V_c = 2.150\ N/mm^2$

$> 1.829\ N/mm_2$ HENCE OK

Check the flexural moment transfer -

For this joint the area of rebar is calculated as

$A_s = 0.00075 \times h \times L$

$A_s = 892.50\ mm^2$

Provide 6-12 mm Φ.

Calculate the stress in strand tendon -

b = 860.00 mm

f_{pe} = 1296.30 N/mm2

F_p = 260.96 KN

F_y = 272.16 KN

$T_u = F_p + F_y$ = 533.12 KN

$a = Tu / (0.85*b*f_c)$

a = 20.84

Tendon j_ud = 74.58 mm

Rebar j_ud = 134.58 mm

$\Phi M_t = 0.9*(j_ud*F_p + j_ud*F_y)$

ΦM_t = 50.48 KNm

> -3.05 KNm HENCE OK

This moment is greater than the moment transferred required.

(c) Shear at Interior column -

The direct shear left and right of interior column is calculated above.

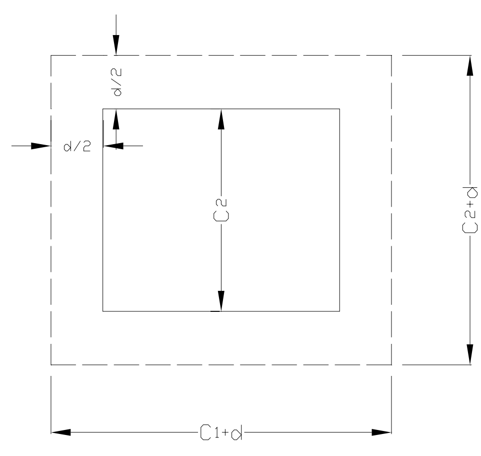
Total direct shear -

V = 573.66 KN

Moment transferred

Mt = 41.236 KNm

Shear section properties -



d = 145.00 mm

C_1 = 500.00 mm, C_2 = 350.00 mm

$d + C_1 = 645.00$ mm

$d + C_2 = 495.00$ mm

$b_0d = 330600$

Polar moment of inertia -

$J = d*(C_1+d)^3/12 + (C_1+d)*d^3/12 + (C_2+d)*d*((C_1+d)/2)^2$

$J = 21742614063$ mm^4

$Z = J / ((C_1 + d)/2)$

$Z = 67418958.33$ mm^3

Portion of moment to be transferred by torsional shear -

$\alpha = 0.432$

$M_{ut} = 17.82$ KNm

$M_{uf} = M_t - M_{ut} = 23.42$ KNm

Shear stresses -

Direct shear stress = V/b_0d = 1.735 N/mm^2

Torsional shear stress = M / Z = 0.264 N/mm^2

Total shear stress = 2.000 N/mm^2

Calculate allowable shear stress -

$V_c = 3.5*\sqrt{f_c} + 0.3F/A$

$Vc = 2.05$ N/mm^2

> 2.00 N/mm^2 HENCE OK

Moment transferred by flexure -

$M_{uf} = 23.42$ KNm

$b = 860.00$ mm

Say , $F_p = 260.96$ KN

$A_s = 0.00075 \times h \times L$

$A_s = 914.81$ mm2

Use 6-12 mm Φ Bars

$A_s = 678.00$ mm^2

$F_y = 285.59$ KN

$T_u = 546.56$ KN

Depth of compression block -

$a = 21.36$ mm

d - a/2 = 134.32 mm

ΦM_t = 66.07 KNm

> 23.42 KNm HENCE OK

Check for Deflection -

L_x = 7.00 m

L_y = 7.00 m

F_c = 35.00 N/mm^2

F_y = 415.00 N/mm^2

D = 170.00mm

Clear cover = 20.00 mm

E_s = 195000 N/mm^2

E_c = 29580.40 N/mm^2

Loads -

Dead Load = 4.25 KN/m^2

Live Load = 2.00 KN/m^2

Superimposed dead load = 1.00 KN/m^2

Let us consider permanent load = DL + 0.25LL

Permanent Load = 5.75 KN/m^2

Temporary Load = 1.50 KN/m^2

W_{perm}. = 5.75 KN/m Per meter width

f_{cr} = 4.14 N/mm^2

y_t = 85.00 mm

I_{gr} = 409416666.7 mm^4

Moment of resistance -

$M_r = f_{cr} * I_{gr} / y_t$

M_r = 19947049 Nmm

M_r = 19.947049 KNm

Deflection in X- Direction -

Effective depth d = 144.00 mm

A_{st} = 892.50 mm

Deflections for dead load and live load -

(a) Short term deflection -

M_o perm. = $_{Wperm.}$ * L^2 / 8

M_o perm. = 35.21875 KNm

m = E_s / E_c

m = 6.59

Depth of neutral axis -

$bx^2/2$ = m * A_{st} * (d - x)

500 x2 = 847229.9536 - 5883.54 x

x = 35.70 mm

Lever arm z = d - x/3

z = 132.10 mm

I_r = bx3/3 + m*Ast*(d-x)2

I_r = 84173840.16

Total Load = 7.25 KN/m

M_{total} = 44.41 KNm

For total load -

C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b

C = 0.89

I_{eff} = I_r / C

I_{eff} = 94568718.93

> I_r < I_{gr}

a_{itotal} = K*w*L3 / Ec*Ieff

$a_{i\ total}$ = 11.57 mm

For permanent load -

C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b

C = 0.809

I_{eff} = I_r / C

I_{eff} = 104016837.3

$a_{i\ perm.}$ = K*w*L^3 / E_c*I_{eff}

$a_{i\ perm.}$ = 8.35 mm

$a_{i\ temp.}$ = $a_{i\ total}$ - $a_{i\ perm.}$

$a_{i\ temp.}$ = 3.23 mm

(b) Long term deflection due to shrinkage -

$a_{cs} = k_3 * \psi_{cs} * L^2$

Where, $k_3 = 0.125$

$\psi_{cs} = k4 * \varepsilon_{cs}/D$

Percentage of steel - $P_t = 0.62$ %

For $P_t < 1$ and $P_c = 0$

$k_4 = 0.72 * \sqrt{Pt}$

$k_4 = 0.566$

$\varepsilon_{cs} = 0.0003$

$\psi_{cs} = 1.1809E\text{-}06$

$a_{cs} = 7.23$ mm

(c) Long term deflection due to creep -

$E_{ce} = E_c / (1 + \theta)$

$E_{ce} = 11377.07$

$m = E_s / E_{ce}$

$m = 17.14$

Depth of neutral axis -

$bx^2/2 = m * A_{st} * (d - x)$

$500x^2 = 2202797.879 - 15297.21\ x$

$x = 52.82$ mm

$z = 126.3941986$

$I_r = bx^3/3 + m*A_{st}*(d-x)^2$

$I_r = 176299568.9$

For creep, $M = M_{o\ perm}. = 35.21875$

$C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b$

$C = 0.885$

$I_{eff} = I_r / C$

$I_{eff} = 199160951$

$a_{iccperm}. = K*w*L3 / E_{ce}*I_{eff}$

$a_{iccperm}. = 11.33$ mm

$a_{ccperm} = a_{iccperm} - a_{iperm}$

$a_{ccperm} = 2.99$ mm

Total deflection due to dead and live load -

$a_T = a_{iccperm} + a_{cs} + a_{iperm}$

$a_T = 26.91$ mm

Total upward deflection due to balance load -

(a) Due to short term -

Balanced load = 2.92

$M_{bal} = 17.86$

$C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b$

$C = 0.429$

$I_{eff} = I_r / C$

$I_{eff} = 196028448.3$

$a_{ibal} = K*w*L3 / E_c*I_{eff}$

$a_{ibal} = 2.25$ mm

(b) Due to shrinkage -

$a_{cs\ bal} = 7.23$ mm

(c) Due to creep

$C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b$

$C = 0.579$

$I_{eff} = I_r / C$

$I_{eff} = 304369807.6$

$a_{icc\ ba}l = K*w*L^3 / E_c*I_{eff}$

$a_{icc\ bal} = 3.76$ mm

Total deflection due to balance load -

$a_{T\ bal} = a_{ibal} + a_{cs\ bal} + a_{icc\ bal}$

$a_{T\ bal} = 13.24$ mm

Therefore total deflection in x-direction -

$a_{Tx} = a_T - a_{Tbal}$.

$a_{Tx} = 13.67$ mm

Deflection in Y- Direction -

Effective depth d = 132.00 mm

$A_{st} = 892.50$ mm^2

Deflections for dead load and live load -

(a) Short term deflection -

$M_{o\ perm.} = W_{perm.} * L^2 / 8$

$M_{o\ perm.} = 35.21875$

Depth of neutral axis -

$bx^2/2 = m * A_{st} * (d - x)$

500 x2 = 776627.4574 - 5883.54 x

x = 33.96 mm

z = d - x/3

z = 120.6784809

$I_r = bx^3/3 + m*A_{st}*(d-x)^2$

I_r = 69606814.33

Total Load = 7.25

M_{total} = 44.40625

$M_{perm.}$ = 35.21875

For total load -

$C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b$

C = 0.895000139

$I_{eff} = I_r / C$

I_{eff} = 77772964.8

$a_{i\ total} = K*w*L^3 / E_c*I_{eff}$

$a_{i\ total}$ = 14.07 mm

For permanent load -

$C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b$

C = 0. Ieff =Ir / C

I_{eff} = 85361577.43

$a_{i\ perm.} = K*w*L^3 / E_c*I_{eff}$

$a_{i\ perm.}$ = 10.17 mm

$a_{i\ temp.} = a_{i\ total} - a_{i\ perm.}$

$a_{i\ temp.}$ = 3.90 mm

(b) Long term deflection due to shrinkage -

$a_{cs} = k_3 * \psi_{cs} * L^2$

Where, $k_3 = 0.125$

$\psi_{cs} = k_4 * \varepsilon_{cs}/D$

Percentage of steel - P_t = 0.6761

For $P_t < 1$ and $P_c = 0$

$k_4 = 0.72 * \sqrt{P_t}$

$k_4 = 0.592$

$\varepsilon_{cs} = 0.0003$

$\psi_{cs} = 1.34554E-06$

a_{cs} = 8.24 mm

(c) Long term deflection due to creep -

$E_{ce} = E_c / (1 + \theta)$

$E_{ce} = 11377.076$

$m = E_s / E_{ce}$

$m = 17.139$

Depth of neutral axis -

$bx^2/2 = m * A_{st} * (d - x)$

$500x^2 = 2019231.389 \quad -15297.21\ x$

x = 50.07 mm

z = 115.3110313

$I_r = bx^3/3 + m*A_{st}*(d-x)^2$

$I_r = 144524797.4$

For creep, $M = M_{o\ perm.} = 35.21875$

$C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b$

$C = 0.892$

$I_{eff} = I_r / C$

$I_{eff} = 161860933.1$

$a_{iccperm.} = K*w*L3 / E_{ce}*I_{eff}$

$a_{iccperm.}$ = 13.95 mm

$a_{ccperm.} = a_{iccperm.} - a_{iperm.}$

$a_{ccperm.}$ = 3.77 mm

Total deflection due to dead and live load -

$a_T = a_{iccperm.} + a_{cs} + a_{iperm.}$

a_T = 32.36 mm

Total upward deflection due to balance load -

(a) Due to short term -

Balanced load = 2.92

$M_{bal.}$ = 17.85

$C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b$

$C = 0.441$

$I_{eff} = I_r / C$

$I_{eff} = 157614558.8$

$a_{i\ bal.} = K*w*L^3 / E_c*I_{eff}$

$a_{i\ bal.} = 2.79$ mm

(b) Due to shrinkage -

$a_{cs\ bal.} = 8.24$ mm

(c) Due to creep

$C = 1.2 - M_r/M_T * z/d * (1 - x/d) * b_w/b$

$C = 0.594$

$I_{eff} = I_r / C$

$I_{eff} = 243152203.5$

$a_{iccbal.} = K*w*L^3 / E_{ce}*I_{eff}$

$a_{iccbal.} = 4.71$ mm

Total deflection due to balance load -

$a_{T\ bal.} = a_{ibal.} + a_{cs\ bal.} + a_{icc\ bal.}$

$a_{T\ bal.} = 15.74$ mm

Therefore total deflection in Y-direction -

$a_{Ty} = a_T - a_{Tbal.}$

$a_{Ty} = 16.62$ mm

To calculate the final deflection -

We have,

$\delta Ax = \delta C_x = 16.62$ mm

$\delta D_y = \delta E_y = 13.67$ mm

$\delta B_x = 16.62$ mm

$\delta B_y = 13.67$ mm

Now, $\delta 1_B = 1/2\ (\delta A_x + \delta C_x) + \delta B_y$

$\delta 1_B = 16.62$ mm

$\delta 2_B = 1/2\ (\delta D_y + \delta E_y) + \delta B_x$

$\delta 2_B = 13.67$ mm

Total deflection -

$\delta = 1/2\ (\delta 1_B + \delta 2_B)$

$\delta = 15.14$ mm

Allowable deflection -

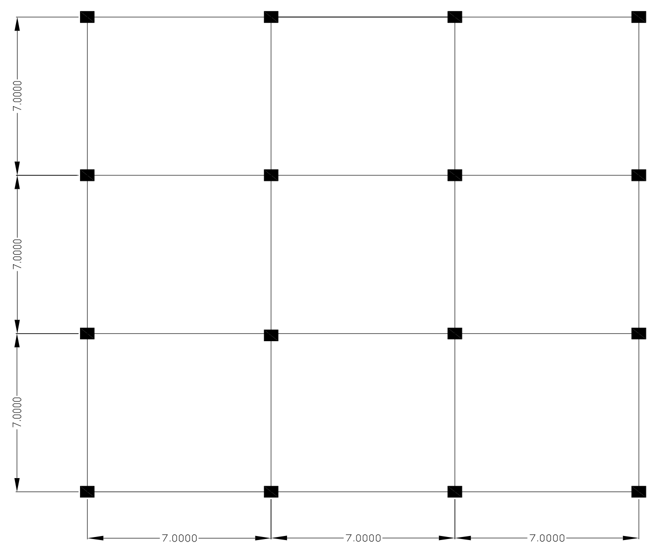
$\delta = L / 325$

$\delta = 21.54$ mm

> 15.14 mm HENCE OK.

A.2 DESIGN OF POST-TENSIONED FLAT SLAB BY LOAD BALANCING METHOD

Data Given



PLAN

Span 1 (X-Dir) = 7 m

Span 2 (X-Dir) = 7 m

Span 1 (Y-Dir) = 7 m

Span 2 (Y-Dir) = 7 m

Tendon diameter = 12.70 mm

Area of tendon = A_p = 98.71 mm^2

F_{pu} = 1862 N/mm^2

E_s = 195000 N/mm^2

E_c = 29580.40 N/mm^2

μ = 0.35

k = 0.0015

Creep coefficient Φ = 1.60

Column sizes -

Exterior column = 350 mm x 300 mm

Interior column = 350 mm x 500 mm

Height of the floor = 3 m

F_c = 35 N/mm^2

F_y = 415 N/mm^2

Loads -

Dead Load = Self weight of the Slab

Superimposed dead load = 1 KN/m^2

Live Load = 2.00 KN/m^2

Slab Thickness -

Let us consider slab thickness @ L/45

Ratio of the span = 45

Longitudinal direction = 155.56 mm

Transverse Direction = 155.56 mm

Take the slab thickness = 170 mm

Load Calculation -

Dead Load due to self weight of slab = 4.25KN/m^2

Superimposed dead load = 1 KN/m2

Live Load = 2.00 KN/m^2

f_{se} = 1048.59

P_{eff} = Effective force per tendon = A * f_{se}

P_{eff} = 103.50 KN

Design of East-West Frame Interior frame

Total bay width between centre lines = 7 m

LL / DL = 0.47

< 0.75

No pattern loading is required.

Section Properties -

A = 1190000 mm^2

Z = 33716666.67 mm^3

I = 2865916667 mm^4

Design parameters -

Allowable stresses -

At the time of jacking -

f_{ci} = 21

Compression = 0.6 * f_{ci} = 12.6 N/mm^2

Tension = 3 * $\sqrt{f_{ci}}$ = 1.15 N/mm^2

At Service Load -

f_c = 35 N/mm^2

Compression = 0.45 * f_c = 15.75 N/mm^2

Tension = 6 * $\sqrt{f_c}$ = 2.98 N/mm^2

Average precompression limit -

P / A = 0.87 N/mm^2

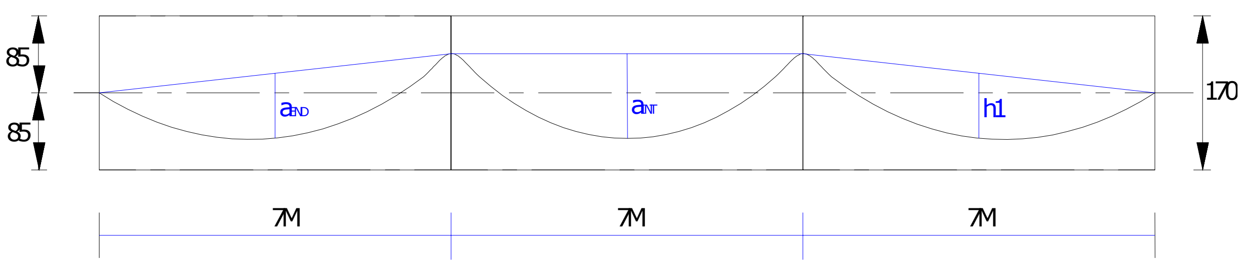
2.1 N/mm^2

Target load balances -

60 to 80% of dead load for slabs (only self weight). Let us assume 75%.

$0.75W_{DL}$= 3.1875 KN/m^2

Tendon Profile - Parabolic shape



For a layout with span of similar length, the tendons will be typically located at the highest allowable point at the interior columns, the lowest possible point at the midspan and the neutral axis at the anchor locations. This provides the maximum drape for load balancing.

Cover Provided -

Exterior span - At bottom = 38 mm

At top = 25 mm

Interior span - At bottom = 25 mm

At top = 25 mm

a_{INT} = 120 mm

a_{END} = 77 mm

Center of gravity of tendons measured from bottom slab -

Table A5 Tendon coordinate and location

Tendon ordinate	Tendon location
Exterior support - anchor	85.00 mm
Interior support - top	145.00 mm
Interior support - bottom	25.00 mm
End span - bottom	38.00 mm

Eccentricity e, is the distance from the centre to tendon to the neutral axis, varies along the span.

Prestress force required to balance 75% of dead load (self weight) - Since the span are of similar length, the span will typically govern the maximum required post-tensioning force. This is due to significantly reduced tendon drape, a_{END}

W_b = 0.75 x W_{DL}

W_b = 22.312 KN/m

Force required in the tendon to counteract the load in end bay -

$P = w_b \times L^2 / 8a_{END}$

P = 1774.86 KN

Calculation of losses -

(a) Loss due to elastic deformation of concrete -

Loss of stress = $\alpha_e \times f_c$

Where, $\alpha_e = E_s / E_c$

α_e = 6.59

f_c = P / A + P x e x y / I

f_c = 4.65 N/mm^2

Loss of stress = 30.65 N/mm^2

(b) Loss due to shrinkage of concrete -

Loss of stress = ε_{cs} x E_s

Where, ε_{cs} = 200 x 10^{-6} / log10(t+2)

t = Age of concrete at transfer in days

t = 7 day

ε_{cs} = 0.0002096

Loss of stress = 40.87 N/mm^2

(c) Loss due to creep of concrete -

Loss of stress = Φ * f_c *αe

Loss of stress = 49.04 N/mm^2

(d) Loss of stress due to friction -

Loss of stress = f_{pu} * e-(μα + k_x)

For the parabola the equation is given by

y = 4e/L^2 * x(L - x)

Slope at the ends (at x = 0) = dy/dx = 4e / L

Slope = 0.034

Cumulative angle between tangents,

α = 0.0685

For the small values of (μα + k_x),

Loss of stress = f_{pu} x (μα + k_x) With referance to IS1343: 1980

Loss of stress = 64.24 N/mm^2

(e) Loss due to relaxation -

According to IS 1343;1980, Table 4,

Relaxation loss = 70 N/mm^2

Total loss of stress = 254.81 N/mm^2

Check precompression allowance -

Determine number of tendons to achieve P -

No. of tendons = 17.15

Say 17 No.

$P_{act.}$ = 1759.61 KN

The balanced load for the end span is slightly adjusted

W_b = 22.12 KN/m

Determine actual precompression stresses -

$P_{act.}/A$ = 1.48 N/mm^2

> 0.87 N/mm^2

< 2.1 N/mm^2 HENCE OK.

Check the interior span forces -

$P = W_b * L^2 / 8a_{INT}$

P = 1138.87 KN

By taking $P_{act.}$ -

$W_b = 8*Pact.*a_{INT} / L^2$

W_b = 34.47 KN/m

W_b/W_{DL} = 115.88 %

Effective prestressing force P_{eff} = 1759.61 KN

Check slab stresses -

Total dead load = 5.05 KN/m^2

Total dead load = 35.35 KN/m

Total live load = 14.00 KN/m

Total balancing load= -22.12 KN/m

The bending moments for each load (dead, live, balancing load) are as follows

Bending moment due to dead load -

At support = 172.25 KNm

At middle span

Interior span = 44.26 KNm

Exterior span = 138.60 KNm

Bending moment due to live load -

At support = 68.22 KNm

At middle span

Interior span = 17.54 KNm

Exterior span = 55.00 KNm

Bending moment due to balancing load -

At support = 107.78 KNm

At middle span

Interior span = 27.75 KNm

Exterior span = 86.90 KNm

Stage 1 - Stresses immediately after jacking (DL + PT)

Midspan stresses -

f_{top} = $(-M_{DL} + M_{BAL}) / Z - P/A$

f_{bot} = $(M_{DL} - M_{BAL}) / Z - P/A$

Interior span -

f_{top} = -1.97 N/mm^2

< 12.60 N/mm^2

f_{bot} = -0.99 N/mm^2

< 12.60 N/mm^2

End span -

f_{top} = -3.01 N/mm^2

< 12.60 N/mm^2

f_{bot} = 0.05 N/mm^2

< 1.15 N/mm^2

Support stresses -

f_{top} = $(M_{DL} - M_{BAL}) / Z - P/A$

f_{bot} = $(-M_{DL} + M_{BAL}) / Z - P/A$

f_{top} = 0.43 N/mm^2

< 1.15 N/mm^2

f_{bot} = -3.39 N/mm^2

< 12.60 N/mm^2 HENCE OK

Stage 2 - Stresses at service load (DL + PT + LL)

Midspan stresses -

f_{top} = $(-M_{DL} - M_{LL} + M_{BAL}) / Z - P/A$

f_{bot} = $(M_{DL} + M_{LL} - M_{BAL}) / Z - P/A$

Interior span -

f_{top} = -2.49 N/mm^2

$< 15.75\ N/mm^2$

$f_{bot} = -0.47\ N/mm^2$

$< 15.75\ N/mm^2$

End span -

$f_{top} = -4.64\ N/mm^2$

$< 15.75\ N/mm^2$

$f_{bot} = 1.69\ N/mm^2$

$< 2.98\ N/mm^2$

Support stresses -

$f_{top} = (M_{DL} + M_{LL} - M_{BAL}) / Z - P/A$

$f_{bot} = (-M_{DL} - M_{LL} + M_{BAL}) / Z - P/A$

$f_{top} = 2.46\ N/mm^2$

$< 2.98\ N/mm^2$

$f_{bot} = -5.41\ N/mm^2$

$< 15.75\ N/mm^2$ HENCE OK

All the stresses are within permissible limit.

Ultimate Moment -

Determine the factored moments. The primary post-tensioning moment M_1, vary along the length of the span.

$M_1 = P \times e$

At exterior support $e = 0$, sothat

$M_1 = 0$ at exterior supports.

At interior supports,

$e = 60.00\ mm$

$M_1 = 105.58\ KNm$

The secondary post-tensioning moments M_2, vary linearly between the supports,

$M_2 = M_{bal} - M_1$

$M_2 = 2.20\ KNm$ at interior supports.

Typical load combination for ultimate strength design is

$M_u = 1.2\ M_{DL} + 1.6\ M_{LL} + M_2$

At mid span,

$M_u = 256.52\ KNm$

At support,

M_u = -318.06 KNm

Determination of minimum bonded reinforcement -

Positive moment region - Interior span

$f_t = (M_{DL} + M_{LL} - M_{BAL}) / Z - P/A$

$f_t = -0.46$

$< 1\ N/mm^2$

No positive reinforcement is required.

Exterior span -

$f_t = (M_{DL} + M_{LL} - M_{BAL}) / Z - P/A$

$f_t = 1.69\ N/mm^2$

$> 1\ N/mm^2$

Therefore minimum positive reinforcement is required.

$y = f_t / (f_t + f_c) \times h$

$y = 45.28$ mm

$N_c = M_{DL+LL} \times 0.5y \times L_2 / Z$

$N_c = 910.06$ KN

$A_{stmin} = N_c / 0.5f_y$

$A_{stmin} = 4385.83\ mm^2$

Distribute the positive moment reinforcement uniformly across the slab beam width and as close as possible to the extreme fiber.

$A_{stmin} = 626.55\ mm^2/m$

Diameter of bar =10 mm

Area of bar= $78.53\ mm^2$

Spacing = 125.35 mm

Provide 10 mm Φ bars @ 120 mm c/c

$A_{stprov.} = 654.50\ mm^2/m$

Minimum length shall be 1/3 clear span and centered in a positive moment region.

Negative moment region - Interior span

$A_{smin} = 0.00075 * A_{cf}$

$A_{smin} = 892.50\ mm^2$

Diameter of bar = 12 mm

Area of bar = 113.09 mm^2

No. of bars required = 7.89

Provide 8 No. 12 mm Φ bars.

Exterior span -

A_{smin} = 0.00075 * A_{cf}

A_{smin} = 892.50 mm^2

Diameter of bar = 12 mm

Area of bar = 113.09 mm^2

No. of bars required = 7.89

Provide 8 No. 12 mm Φ bars.

Span must minimum of 1/6 the clear span on each support.

At least 4 bars are required in each direction. Place top bars within 1.5h from the face of the support on each side 255 mm

Keep Bar spacing of 250 mm

Check minimum reinforcement for ultimate strength -

$M_n = (A_s*f_y + A_{ps} x\ f_{ps})\ x\ (d - a/2)$

Where, d = effective depth

At support -

d = 144 mm

A_{ps} = 1678.07 mm^2

$f_{ps} = f_{se} + f_c/100\rho_p + 10Ksi$

ρ_p = Aps / bd

Where, A_{ps} is area of tendons, b is width of slab and d is effective depth of slab.

ρ_p = 0.0016

f_{ps} = 1188.98

$a = (A_s \times f_y + A_{ps} \times f_{ps})/(0.85 \times f_c \times b)$

a = 11.38 mm

$\Phi M_n = 0.9 \times (A_s \times f_y + A_{ps} \times f_{ps}) \times (d - a/2)$

ΦM_n = 295095308.5

ΦM_n = 295.10 KNm

< 318.06 KNm

Reinforcement for ultimate strength governs.

Astreqd. = 1349.24mm^2

Diameter of bar = 12 mm

Area of bar = 113.09 mm^2

No. of bars required = 11.92

Provide 12 No. 12 mm Φ bars at interior support.

Provide 8 No. 12mm Φ bars at exterior support.

At Mid span -

$d = 124$ mm

$\rho_p = 0.0014$

$f_{ps} = 1201.63$ N/ mm^2

$a = 18.42$ mm

$\Phi M_n = 0.9*(A_s \times f_y + A_{ps} \times f_{ps})x(d - a/2)$

$\Phi M_n = 396352680.3$

$\Phi M_n = 396.35$ KNm

> 256.52 KNm

Provide 10 mm Φ bars @ 120 mm c/c HENCE OK

The check for the shear and the deflection is same as in case of design of post-tensioned flat slab by equivalent frame method.

A.3 DESIGN RESULT OF POST-TENSIONED FLAT SLAB BY ADAPT SOFTWARE

```
 ----------------------------------------------------------------------------
                             ADAPT CORPORATION
                    STRUCTURAL CONCRETE SOFTWARE SYSTEM
       1733 Woodside Road, Suite 220, Redwood City, California 94061
 ----------------------------------------------------------------------------
                  ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN
                  Version 7.10  AMERICAN (ACI 318-02/IBC-03)
             ADAPT CORPORATION - Structural Concrete Software System
       1733 Woodside Road, Suite 220, Redwood City, California 94061
                   Phone: (650)306-2400,  Fax: (650)364-4678
      Email: Support@AdaptSoft.com, Web site: http://www.AdaptSoft.com
 ----------------------------------------------------------------------------
--
 DATE AND TIME OF PROGRAM EXECUTION:  Mar 1,2007   At Time: 19:34
 PROJECT FILE: sdtypg4
 P R O J E C T   T I T L E: pt slab typ fl sd grid 4

 1 - USER SPECIFIED   G E N E R A L   D E S I G N   P A R A M E T E R S

=============================================================================
```

```
 CONCRETE:
 STRENGTH at 28 days, for BEAMS/SLABS ............      28.00 N/mm^2
                      for COLUMNS ................      28.00 N/mm^2

 MODULUS OF ELASTICITY for BEAMS/SLABS ...........   24870.00 N/mm^2
                       for COLUMNS ...............   24870.00 N/mm^2

 CREEP factor for deflections for BEAMS/SLABS .....       2.00
 CONCRETE WEIGHT .................................      NORMAL

 SELF WEIGHT .....................................    2400.00 Kg/m^3


 TENSION STRESS limits (multiple of (f'c)1/2)
 At Top  .........................................        .620
 At Bottom  ......................................        .620

 COMPRESSION STRESS limits (multiple of (f'c))
 At all locations ................................        .450

 REINFORCEMENT:
 YIELD Strength ..................................     415.00 N/mm^2
 Minimum Cover at TOP ............................      20.00 mm
 Minimum Cover at BOTTOM .........................      20.00 mm

 POST-TENSIONING:
 SYSTEM ..........................................       BONDED
 Ultimate strength of strand .....................    1860.00 N/mm^2
 Average effective stress in strand (final) .......    1200.00 N/mm^2
 Strand area......................................     99.000 mm^2

 Page  2                      (sdtypg4)                  ADAPT-PT  V- 7.10
ACI-02
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 Min CGS of tendon from TOP.......................      60.00 mm
 Min CGS of tendon from BOTTOM for INTERIOR spans..      50.00 mm
 Min CGS of tendon from BOTTOM for EXTERIOR spans..      50.00 mm
 Min average precompression ......................        .85 N/mm^2
 Max spacing between strands (factor of slab depth)       8.00
 Tendon profile type and support widths...........  (see section 9)

 ANALYSIS OPTIONS USED:
 Structural system ....(using EQUIVALENT FRAME)....     TWO-WAY
 Moments REDUCED to face of support ...............       YES

 2 - I N P U T   G E O M E T R Y

============================================================================
=

                2.1.1  PRINCIPAL SPAN DATA OF UNIFORM SPANS
 ---------------------------------------------------------------------------
--
 S     F|       |              |     TOP      |BOTTOM/MIDDLE|      |
```

```
 P     O|       |              |    FLANGE    |    FLANGE   |  REF |
MULTIPLIER
 A     R| LENGTH| WIDTH   DEPTH|  width thick.| width thick.|HEIGHT| left
right
 N     M|   m   |   mm      mm |    mm     mm |   mm     mm |  mm  |
-1-----3----4-------5-------6-------7------8------9------10----11-----12----
13-
 1     1    8.40   8400     200                                  0   .50
.50
 2     1    8.40   8400     200                                  0   .50
.50
 3     1    8.40   8400     200                                  0   .50
.50
 4     1    8.40   8400     200                                  0   .50
.50
 5     1    8.40   8400     200                                  0   .50
.50
 6     1    8.40   8400     200                                  0   .50
.50
 ----------------------------------------------------------------------------
--
 LEGEND:
 1 - SPAN                                3 - FORM
       C = Cantilever                          1 = Rectangular section
                                               2 = T or Inverted L section
                                               3 = I section
                                               4 = Extended T or L section
                                               7 = Joist
                                               8 = Waffle
                                        11 - Top surface to reference line

       2.1.5 -   D R O P   C A P   A N D   D R O P   P A N E L   D A T A

=============================================================================
=
        CAPT    CAPB   CAPDL   CAPDR    DROPTL  DROPTR   DROPB   DROPL
DROPR
 JOINT    mm      mm      mm      mm        mm      mm      mm      mm
mm
 --1------2-------3-------4-------5---------6-------7-------8-------9-------
10-
   1      0       0       0       0         0     350    2800       0
1400
   2      0       0       0       0       350     350    2800    1400
1400

 Page  3                     (sdtypg4)                 ADAPT-PT  V- 7.10
ACI-02
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--

   3      0       0       0       0       350     350    2800    1400
1400
   4      0       0       0       0       350     350    2800    1400
1400
   5      0       0       0       0       350     350    2800    1400
1400
```

```
   6       0       0       0       0       350     350    2800    1400
1400
   7       0       0       0       0       350       0    2800    1400
0
 ---------------------------------------------------------------------------
--
 LEGEND:
 DROP CAP DIMENSIONS:                         DROP PANEL DIMENSIONS:
 CAPT = Total depth of cap                  DROPTL = Total depth left of joint
 CAPB   = Transverse Width                    DROPTR = Total depth right of joint
 CAPDL  = Extension left of joint             DROPB  = Transverse Width
 CAPDR  = Extension right of joint            DROPL  = Extension left of joint
                                              DROPR  = Extension right of joint
 ---------------------------------------------------------------------------
--

      2.2 - S U P P O R T   W I D T H   A N D   C O L U M N   D A T A

        SUPPORT   <------- LOWER COLUMN ------>   <------ UPPER COLUMN ------
>
         WIDTH    LENGTH   B(DIA)    D    CBC*    LENGTH   B(DIA)    D   CBC*
 JOINT    mm        m       mm      mm              m       mm      mm
 --1-------2---------3-------4-------5-----6---------7-------8-------9----10-
--
    1       800      3.00     800     800  (1)       3.00     800     800
(1)
    2       800      3.00     800     800  (1)       3.00     800     800
(1)
    3       800      3.00     800     800  (1)       3.00     800     800
(1)
    4       800      3.00     800     800  (1)       3.00     800     800
(1)
    5       800      3.00     800     800  (1)       3.00     800     800
(1)
    6       800      3.00     800     800  (1)       3.00     800     800
(1)
    7       800      3.00     800     800  (1)       3.00     800     800
(1)

 *THE COLUMN BOUNDARY CONDITION CODES (CBC)
 Fixed at both ends ...(STANDARD) ............................ = 1
 Hinged at near end, fixed at far end ........................ = 2
 Fixed at near end, hinged at far end ........................ = 3
 Fixed at near end, roller with rotational fixity at far end .. = 4

 3 - I N P U T   A P P L I E D   L O A D I N G

============================================================================
=

 <---CLASS--->               <--------------TYPE------------------->
 D = DEAD LOAD               U = UNIFORM         P = PARTIAL UNIFORM
 L = LIVE LOAD               C = CONCENTRATED    M = APPLIED MOMENT
 Page  4                       (sdtypg4)                  ADAPT-PT  V- 7.10
ACI-02
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```
                               Li= LINE LOAD
 SW= SELF WEIGHT Computed from geometry input and treated as dead loading
     Unit selfweight  W = 2400.0 Kg/m^3

                    Intensity  ( From ...  To ) ( M   or C ...At) Total on
Trib
 SPAN  CLASS  TYPE    kN/m^2   (  m        m  ) (kN-m or kN...m )      kN/m
 -1-----2------3--------4----------5--------6---------7-------8---------9----
--

  1     L      U        4.000      .00     1.40
33.600
  1     L      U        4.000     1.40     7.00
33.600
  1     L      U        4.000     7.00     8.40
33.600
  1     D      U        2.000      .00     1.40
16.800
  1     D      U        2.000     1.40     7.00
16.800
  1     D      U        2.000     7.00     8.40
16.800
  1     D      C                                   110.00     .10
  1     SW     P                   .00     1.40
49.442
  1     SW     P                  1.40     7.00
39.554
  1     SW     P                  7.00     8.40
49.442

  2     L      U        4.000      .00     1.40
33.600
  2     L      U        4.000     1.40     7.00
33.600
  2     L      U        4.000     7.00     8.40
33.600
  2     D      U        2.000      .00     1.40
16.800
  2     D      U        2.000     1.40     7.00
16.800
  2     D      U        2.000     7.00     8.40
16.800
  2     SW     P                   .00     1.40
49.442
  2     SW     P                  1.40     7.00
39.554
  2     SW     P                  7.00     8.40
49.442

  3     L      U        4.000      .00     1.40
33.600
  3     L      U        4.000     1.40     7.00
33.600
  3     L      U        4.000     7.00     8.40
33.600
```

```
  3     D       U         2.000      .00     1.40
16.800
  3     D       U         2.000     1.40     7.00
16.800
  3     D       U         2.000     7.00     8.40
16.800
  3     SW      P                    .00     1.40
49.442
  3     SW      P                   1.40     7.00
39.554
  3     SW      P                   7.00     8.40
49.442

  4     L       U         4.000      .00     1.40
33.600
  4     L       U         4.000     1.40     7.00
33.600
  4     L       U         4.000     7.00     8.40
33.600
  4     D       U         2.000      .00     1.40
16.800
  4     D       U         2.000     1.40     7.00
16.800
  4     D       U         2.000     7.00     8.40
16.800
  4     SW      P                    .00     1.40
49.442
  4     SW      P                   1.40     7.00
39.554
  4     SW      P                   7.00     8.40
49.442

  5     L       U         4.000      .00     1.40
33.600
 Page  5                 (sdtypg4)                ADAPT-PT  V- 7.10
ACI-02
 ----------------------------------------------------------------------------
--

  5     L       U         4.000     1.40     7.00
33.600
  5     L       U         4.000     7.00     8.40
33.600
  5     D       U         2.000      .00     1.40
16.800
  5     D       U         2.000     1.40     7.00
16.800
  5     D       U         2.000     7.00     8.40
16.800
  5     SW      P                    .00     1.40
49.442
  5     SW      P                   1.40     7.00
39.554
  5     SW      P                   7.00     8.40
49.442
```

```
  6     L      U        4.000      .00     1.40
33.600
  6     L      U        4.000     1.40     7.00
33.600
  6     L      U        4.000     7.00     8.40
33.600
  6     D      U        2.000      .00     1.40
16.800
  6     D      U        2.000     1.40     7.00
16.800
  6     D      U        2.000     7.00     8.40
16.800
  6     D      C                                     110.00    8.30
  6     SW     P                   .00     1.40
49.442
  6     SW     P                  1.40     7.00
39.554
  6     SW     P                  7.00     8.40
49.442
 3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING

==============================================================================
=

                     UNIFORM
                     (kN/m^2),    ( CON. or PART. )    ( M O M E N T )
 SPAN  CLASS  TYPE   LINE(kN/m)   ( kN@m or  m-m  )    ( kN-m  @  m  )
 -1-----2------3---------4------------5-------6-----------7-------8----------
--

  1     L      U        4.000
  1     D      U        2.000
  1     D      C                   110.00     .10
  2     L      U        4.000
  2     D      U        2.000
  3     L      U        4.000
  3     D      U        2.000
  4     L      U        4.000
  4     D      U        2.000
  5     L      U        4.000
  5     D      U        2.000
  6     L      U        4.000
  6     D      U        2.000
  6     D      C                   110.00    8.30
  NOTE: SELFWEIGHT INCLUSION REQUIRED

 4 - C A L C U L A T E D   S E C T I O N   P R O P E R T I E S

 Page  6                       (sdtypg4)                 ADAPT-PT  V- 7.10
ACI-02
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--

==============================================================================
=

 4.2 - Computed Section Properties for Segments of Nonprismatic Spans
```

Section properties are listed for all segments of each span
A= cross-sectional geometry   Yt= centroidal distance to top fiber
I= gross moment of inertia   Yb= centroidal distance to bottom fiber

SPAN (SEGMENT)	AREA mm^2	I mm^4	Yb mm	Yt mm
---------------1	---------------2	---------------3	---------------4	------------5-----
SPAN 1				
1	2100000.00	.1668E+11	215.00	135.00
2	1680000.00	.5600E+10	100.00	100.00
3	2100000.00	.1668E+11	215.00	135.00
SPAN 2				
1	2100000.00	.1668E+11	215.00	135.00
2	1680000.00	.5600E+10	100.00	100.00
3	2100000.00	.1668E+11	215.00	135.00
SPAN 3				
1	2100000.00	.1668E+11	215.00	135.00
2	1680000.00	.5600E+10	100.00	100.00
3	2100000.00	.1668E+11	215.00	135.00
SPAN 4				
1	2100000.00	.1668E+11	215.00	135.00
2	1680000.00	.5600E+10	100.00	100.00
3	2100000.00	.1668E+11	215.00	135.00
SPAN 5				
1	2100000.00	.1668E+11	215.00	135.00
2	1680000.00	.5600E+10	100.00	100.00
3	2100000.00	.1668E+11	215.00	135.00
SPAN 6				
1	2100000.00	.1668E+11	215.00	135.00
2	1680000.00	.5600E+10	100.00	100.00
3	2100000.00	.1668E+11	215.00	135.00

5 - D E A D   L O A D   M O M E N T S,   S H E A R S & R E A C T I O N S

===

SPAN	< 5.1 S P A N M O M E N T S (kNm) > M(l)*	Midspan	M(r)*	< 5.2 SPAN SHEARS (kN) > SH(l)	SH(r)
--1	---------2	--------------3	---------------4	---------------5	-----------6-------
1	-295.98	142.18	-444.13	-341.58	269.48
2	-402.95	114.84	-380.84	-253.16	247.90
3	-386.44	118.55	-389.93	-250.12	250.95
4	-389.92	118.55	-386.45	-250.94	250.12
5	-380.84	114.84	-402.95	-247.90	253.16

Page 7   (sdtypg4)   ADAPT-PT V- 7.10 ACI-02

SPAN	M(l)*	Midspan	M(r)*	SH(l)	SH(r)
6	-444.13	142.18	-295.99	-269.48	341.59

Note:
  * = Centerline moments

```
 JOINT   < 5.3 REACTIONS (kN)  >   <- 5.4 COLUMN MOMENTS (kNm)   ->
 --1---------------2----------------Lower columns----Upper columns-----
   1             341.58                -159.29          -136.69
   2             522.64                  22.17            19.02
   3             498.01                  -3.01            -2.58
   4             501.89                    .00              .00
   5             498.01                   3.02             2.59
   6             522.64                 -22.16           -19.02
   7             341.59                 159.29           136.70

 6 -  L I V E   L O A D   M O M E N T S,   S H E A R S  &  R E A C T I O N S

=============================================================================
=

     <-- 6.1  L I V E   L O A D  SPAN MOMENTS (kNm)  and SHEAR FORCES (kN) --
>

    <----- left* -----> <--- midspan ---> <---- right* -----> <--SHEAR FORCE-
->
 SPAN    max        min       max      min       max       min      left
right
 -1-------2---------3--------4--------5---------6---------7--------8--------
9--
  1   -167.81   -167.81    83.62    83.62   -257.65   -257.65  -130.43
151.81
  2   -234.38   -234.38    68.21    68.21   -221.90   -221.90  -142.61
139.63
  3   -225.06   -225.06    70.31    70.31   -227.03   -227.03  -140.89
141.35
  4   -227.03   -227.03    70.31    70.31   -225.06   -225.06  -141.35
140.89
  5   -221.89   -221.89    68.21    68.21   -234.39   -234.39  -139.63
142.61
  6   -257.64   -257.64    83.62    83.62   -167.82   -167.82  -151.81
130.43

 Note:
    * = Centerline moments


       <- 6.2 REACTIONS (kN) -> <-------- 6.3  COLUMN MOMENTS (kNm)  --------
>
                                <--- LOWER COLUMN --->  <--- UPPER COLUMN ---
>
 JOINT        max        min          max        min          max        min
 --1-----------2----------3------------4----------5------------6----------7--
--
   1        130.43        .00          .00     -90.31          .00     -77.50
   2        294.42        .00        12.52        .00        10.74        .00
   3        280.52        .00          .00      -1.70          .00      -1.46
   4        282.71        .00          .00        .00          .00        .00
   5        280.52        .00         1.71        .00         1.46        .00
   6        294.42        .00          .00     -12.52          .00     -10.74
   7        130.43        .00        90.32        .00        77.50        .00
```

```
 Note:  Block 6.1 through 6.3 values are maxima of all skipped loading cases
 Page  8                       (sdtypg4)                     ADAPT-PT  V- 7.10
ACI-02
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 7 -    M O M E N T S   REDUCED TO FACE-OF-SUPPORT

=============================================================================
=

          7.1  R E D U C E D   DEAD LOAD MOMENTS (kNm)

  SPAN        <- left* ->  <- midspan ->   <- right* ->
 --1---------------2-------------3-------------4-----------------------------
--
   1           -197.60        142.20       -341.60
   2           -307.00        114.80       -287.00
   3           -291.70        118.50       -294.80
   4           -294.80        118.50       -291.70
   5           -287.00        114.80       -307.00
   6           -341.60        142.20       -197.70

 Note:
    * = face-of-support

          7.2  R E D U C E D   LIVE LOAD MOMENTS (kNm)

       <----- left* ------>   <---- midspan  ---->   <----- right* ----->
 SPAN       max        min          max        min          max        min
 -1----------2----------3-----------4----------5-----------6----------7-----
  1      -118.30    -118.30       83.62      83.62     -199.60    -199.60
  2      -180.00    -180.00       68.21      68.21     -168.70    -168.70
  3      -171.40    -171.40       70.31      70.31     -173.20    -173.20
  4      -173.20    -173.20       70.31      70.31     -171.40    -171.40
  5      -168.70    -168.70       68.21      68.21     -180.00    -180.00
  6      -199.60    -199.60       83.62      83.62     -118.30    -118.30

 Note:
    * = face-of-support
 8 -   SUM OF DEAD AND LIVE MOMENTS (kNm)

=============================================================================
=
 Maxima of dead load and live load span moments combined
 for serviceability checks   (  1.00DL +  1.00LL )

       <----- left* ------>   <---- midspan  ---->   <----- right* ----->
 SPAN       max        min          max        min          max        min
 -1----------2----------3-----------4----------5-----------6----------7-----
  1      -315.90    -315.90      225.82     225.82     -541.20    -541.20
  2      -487.00    -487.00      183.01     183.01     -455.70    -455.70
  3      -463.10    -463.10      188.81     188.81     -468.00    -468.00
 Page  9                       (sdtypg4)                     ADAPT-PT  V- 7.10
ACI-02
 ----------------------------------------------------------------------------
--
```

4	-468.00	-468.00	188.81	188.81	-463.10	-463.10
5	-455.70	-455.70	183.01	183.01	-487.00	-487.00
6	-541.20	-541.20	225.82	225.82	-316.00	-316.00

Note:
 * = face-of-support

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES

==
=

9.1 PROFILE TYPES AND PARAMETERS

LEGEND:

For Span:
1 = reversed parabola
2 = simple parabola with straight portion over support
3 = harped tendon

For Cantilever:
1 = simple parabola
2 = partial parabola
3 = harped tendon

9.2 T E N D O N P R O F I L E

	TYPE	X1/L	X2/L	X3/L	A/L
	1	2	3	4	5
1	1	.100	.500	.100	.000
2	1	.100	.500	.100	.000
3	1	.100	.500	.100	.000
4	1	.100	.500	.100	.000
5	1	.100	.500	.100	.000
6	1	.100	.500	.100	.000

9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE

==
=

Tendon editing mode selected: TENDON SELECTION

	<-------- SELECTED VALUES			-------->	<--- CALCULATED VALUES		--->
	FORCE	<- DISTANCE OF CGS (mm) ->			P/A	Wbal	Wbal
SPAN	(kN/-)	Left	Center	Right	(N/mm^2)	(kN/-)	(%DL)
1	2	3	4	5	6	7	8
1	2361.938	-100.00	-150.00	-60.00	1.41	18.746	26
2	1570.648	-60.00	-150.00	-60.00	.93	16.027	27
3	1501.165	-60.00	-150.00	-60.00	.89	15.318	26

Page 10 (sdtypg4) ADAPT-PT V- 7.10
ACI-02

--

4	1501.165	-60.00	-150.00	-60.00	.89	15.318	26
5	1570.648	-60.00	-150.00	-60.00	.93	16.027	27
6	2361.938	-60.00	-150.00	-100.00	1.41	18.746	26

Approximate weight of strand .... 600.0 Kg

9.35 - TENDON SELECTION DATA:

```
 TYPE  SEL. FORCE  <------------------ TENDON EXTENTS ----------------------
>
           (kN)          <1>   <2>   <3>   <4>   <5>   <6>
 --1----2-----3---|-----|-----|-----|-----|-----|-----|-----|-----|-----|----
-|
   A   12  126.21      <====================================>
   B    7  123.38      <=====|
   C    7  123.38                                  |=====>
```

9.5 R E Q U I R E D MINIMUM P O S T - T E N S I O N I N G FORCES (kN)

	<- BASED ON STRESS CONDITIONS ->			<- BASED ON MINIMUM P/A ->		
SPAN	LEFT*	CENTER	RIGHT*	LEFT	CENTER	RIGHT
--1--	--2--	--3--	--4--	--5--	--6--	--7--
1	.00	691.67	946.69	1785.00	1428.00	1785.00
2	596.31	.00	329.39	1785.00	1428.00	1785.00
3	384.70	80.53	417.90	1785.00	1428.00	1785.00
4	417.90	80.53	384.69	1785.00	1428.00	1785.00
5	329.39	.00	596.31	1785.00	1428.00	1785.00
6	946.70	691.67	.00	1785.00	1428.00	1785.00

Note:
  * = face-of-support

9.6 S E R V I C E S T R E S S E S (N/mm^2) (tension shown positive)

	L E F T *		C E N T E R		R I G H T *	
SPAN	TOP	BOTTOM	TOP	BOTTOM	TOP	BOTTOM
-1-	--2--	--3--	--4--	--5--	--6--	--7--
1	.27	-3.28	-4.28	1.47	1.57	-5.49
2	1.24	-4.98	-3.28	1.41	1.78	-4.74
3	1.88	-4.89	-3.45	1.66	2.00	-5.00
4	2.00	-5.00	-3.45	1.66	1.88	-4.89
5	1.78	-4.74	-3.28	1.41	1.24	-4.98
6	1.57	-5.49	-4.28	1.47	.27	-3.28

Note:
  * = face-of-support

```
 Page 11                    (sdtypg4)                  ADAPT-PT  V- 7.10
ACI-02
 ----------------------------------------------------------------------------
--
```

9.7 POST-TENSIONING B A L A N C E D M O M E N T S, SHEARS & REACTIONS

	<-- S P A N M O M E N T S (kNm) -->			<-- SPAN SHEARS (kN) -->	
SPAN	left*	midspan	right*	SH(l)	SH(r)
--1--	--2--	--3--	--4--	--5--	--6--
1	147.00	-64.97	204.80	4.85	4.85
2	190.30	-51.71	145.30	-2.12	-2.12
3	140.30	-45.80	134.20	.13	.13
4	134.20	-45.80	140.30	-.13	-.13
5	145.30	-51.71	190.30	2.12	2.12
6	204.80	-64.97	147.00	-4.86	-4.86

Note:
* = face-of-support

joint	REACTIONS (kN)	COLUMN MOMENTS (kNm) Lower columns	Upper columns
1	-4.854	38.520	33.060
2	6.975	-8.016	-6.879
3	-2.251	-3.181	-2.730
4	.260	-.001	-.001
5	-2.250	3.178	2.727
6	6.975	8.013	6.877
7	-4.855	-38.520	-33.060

10 - F A C T O R E D   M O M E N T S   &   R E A C T I O N S

===

Calculated as (1.40D + 1.70L + 1.00 secondary moment effects)

10.1 FACTORED DESIGN MOMENTS (kNm)

SPAN	left* max	left* min	midspan max	midspan min	right* max	right* min
1	-408.11	-408.11	392.40	392.40	-784.81	-784.81
2	-719.04	-719.04	301.55	301.55	-655.71	-655.71
3	-672.00	-672.00	312.76	312.76	-680.39	-680.39
4	-680.39	-680.39	312.76	312.76	-672.00	-672.00
5	-655.72	-655.72	301.55	301.55	-719.04	-719.04
6	-784.82	-784.82	392.39	392.39	-408.25	-408.25

Note:
* = face-of-support

10.2 SECONDARY MOMENTS (kNm)

SPAN	left*	midspan	right*
1	69.64	51.19	32.75
2	16.76	24.82	32.88
3	27.76	27.27	26.77
4	26.77	27.27	27.76
5	32.87	24.81	16.76
6	32.74	51.19	69.64

Note:
* = face-of-support

10.3 FACTORED REACTIONS (kN)   10.4 FACTORED COLUMN MOMENTS (kNm)

JOINT	max	min	LOWER column max	LOWER column min	UPPER column max	UPPER column min

```
  1       695.07     473.39     -184.50    -338.03     -158.32    -290.07
  2      1239.09     738.61       44.31      23.02       38.01      19.75
  3      1171.80     694.95       -7.40     -10.29       -6.35      -8.83
  4      1183.51     702.92         .01        .00         .01        .00
  5      1171.80     694.95       10.30       7.40        8.84       6.35
  6      1239.09     738.61      -23.01     -44.29      -19.75     -38.01
  7       695.07     473.39      338.04     184.50      290.07     158.32

 11 - M I L D   S T E E L

=============================================================================
=

 Support cut-off length for minimum steel(length/span) ...    .17
 Span    cut-off length for minimum steel(length/span) ...    .33
 Top     bar extension beyond where required ............. 300.00 mm
 Bottom  bar extension beyond where required ............. 300.00 mm


       REINFORCEMENT based on    NO   REDISTRIBUTION  of factored moments
 ----------------------------------------------------------------------------
--
 11.1     TOTAL WEIGHT OF REBAR =       137.6  Kg     AVERAGE =       .3
Kg/m^2
          TOTAL AREA COVERED    =      423.36   m^2


 11.2.1     S T E E L    A T    M I D - S P A N
                     T O P                                 B O T T O M
          As    DIFFERENT REBAR CRITERIA        As    DIFFERENT REBAR
CRITERIA
  SPAN  (mm^2) <---ULT-----TENS-------->      (mm^2) <---ULT-----TENS--------
>
 --1------2---------3-------4-------5-----------6---------7-------8-------9--
--
 Page 13                       (sdtypg4)                ADAPT-PT  V- 7.10
ACI-02
 ----------------------------------------------------------------------------
--

   1        0  (      0       0       0)         0  (      0       0
0)
   2        0  (      0       0       0)       375  (    375       0
0)
   3        0  (      0       0       0)       563  (    563       0
0)
   4        0  (      0       0       0)       563  (    563       0
0)
   5        0  (      0       0       0)       375  (    375       0
0)
   6        0  (      0       0       0)         0  (      0       0
0)


 11.3.1     S T E E L    A T    S U P P O R T S
                     T O P                                 B O T T O M
```

JOINT	As (mm^2)	DIFFERENT REBAR CRITERIA <---ULT	-----MIN	--------->	As (mm^2)	DIFFERENT REBAR CRITERIA <---ULT	-----MIN	--------->
--1	------2	---------3	-------4	-------5	-----------6	---------7	-------8	-------9----
1	0	(0	0	0)	0	(0	0	0)
2	242	(242	0	0)	0	(0	0	0)
3	1570	(1570	0	0)	0	(0	0	0)
4	1654	(1654	0	0)	0	(0	0	0)
5	1571	(1571	0	0)	0	(0	0	0)
6	242	(242	0	0)	0	(0	0	0)
7	0	(0	0	0)	0	(0	0	0)

12 - P U N C H I N G   S H E A R   C H E C K

===

```
 LEGEND:
 CONDITION... 1 = INTERIOR COLUMN
              2 = END COLUMN
              3 = CORNER COLUMN
              4 = EDGE COLUMN (PARALLEL TO SPAN)
              5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
                  PERFORM SHEAR CHECK MANUALLY
              6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR

 CASE........ 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
              2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)
```

JNT	COND.	FACTORED ACTIONS shear kN	FACTORED ACTIONS moment kN-m	<- PUNCHING SHEAR STRESSES IN N/mm^2 -> due to shear	due to moment	TOTAL	allow-able	STRESS RATIO	CASE
-1	----2	-------3	-------4	---------5	---------6	--------7	---------8	-------9	-----10-
1	2	695.07	628.10	.73	.51	1.24	1.32	.94	1
2	1	1239.10	82.31	.64	.02	.65	1.00	.66	2
3	1	1171.80	19.12	.60	.00	.61	.89	.68	2
4	1	1183.51	.01	.61	.00	.61	.88	.69	2
5	1	1171.80	19.14	.60	.00	.61	.89	.68	2
6	1	1239.10	82.30	.64	.02	.65	1.00	.66	2
7	2	695.07	628.11	.73	.51	1.24	1.32	.94	1

Page 14   (sdtypg4)   ADAPT-PT V- 7.10 ACI-02

```
 PUNCHING SHEAR CHECK SATISFACTORY
 NO ADDITIONAL REBAR OR CHANGE IN SECTION IS NECESSARY

  13 - MAXIMUM  S P A N   D E F L E C T I O N S

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  Concrete`s modulus of elasticity ............. Ec =    24870 N/mm^2
  Creep factor ................................. K  =     2.00
  Ieffective/Igross...(due to cracking).......... K  =     1.00

  Where stresses exceed 0.5(fc`)^1/2 cracking of section is allowed for.
  Values in parentheses are (span/max deflection) ratios

         <.......DEFLECTION ARE ALL IN  mm   , DOWNWARD POSITIVE.......>
 SPAN     DL      DL+PT    DL+PT+CREEP         LL        DL+PT+LL+CREEP
 -1--------2--------3-----------4--------------5--------------6------
  1       4.6      2.6       7.8( 1080)      2.7( 3160)     10.4(  805)
  2       3.0      1.5       4.5( 1853)      1.8( 4790)      6.3( 1336)
  3       3.2      1.9       5.8( 1449)      1.9( 4484)      7.7( 1095)
  4       3.2      1.9       5.8( 1450)      1.9( 4484)      7.7( 1095)
  5       3.0      1.5       4.5( 1853)      1.8( 4790)      6.3( 1336)
  6       4.6      2.6       7.8( 1080)      2.7( 3161)     10.4(  805)

 Page 15                       (sdtypg4)                  ADAPT-PT  V- 7.10
ACI-02
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 16 - FRICTION, ELONGATION AND LONG TERM STRESS LOSSES

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 16.6 LONG TERM STRESS LOSS CALCULATIONS

 16.6.1  INPUT PARAMETERS :
 Type of strand ..............................................  LOW LAX
 Modulus of elasticity of strand ...........................  200000.00
N/mm^2
 Average weight of concrete ................................  NORMAL
 Estimate age of concrete at stressing .....................       5     days
 Modulus of elasticity of concrete at stressing ...........   20000.00
N/mm^2
 Modulus of elasticity of concrete at 28 days .............   24870.00
N/mm^2
 Estimate of average relative humidity .....................      80.00  %
 Volume to surface ratio of member .........................     100.00  mm


 16.6.2 CALCULATED LONG-TERM STRESS LOSS(average of all tendons) :

          <----- STRESS (N/mm^2) ----->
 SPAN      start      center      right
 -1----------2-----------3-----------4----
  1        60.05       63.46       58.24
```

```
  2        60.43       62.51       53.04
  3        52.85       53.74       47.48
  4        47.48       53.74       52.85
  5        53.04       62.51       60.43
  6        58.24       63.46       60.05


 16.7 FRICTION AND ELONGATION CALCULATIONS

 16.7.1  INPUT PARAMETERS :
 Coefficient of angular friction (meu) ....................        .070  /rad
 Coefficient of wobble friction (K) .......................       .0046  /m
 Ultimate strength of strand ..............................      1860.0
N/mm^2
 Ratio of jacking stress to strand's ultimate strength ....        .800
 Anchor set ...............................................       6.000  mm
 Cross-sectional area of strand ...........................      99.000  mm^2


 16.7.2  CALCULATED STRESSES(average of all tendons) :

      LENGTH     <TENDON HEIGHT(mm )>  Horizontal ratios  <-- STRESS(N/mm^2)-
->
 SPAN   m    P   start  center  right   X1/L  X2/L  X3/L  start  center
right
 -1-----2----3-----4------5-------6------7-----8-----9------10-----11------
12--
  1    8.40  1   -100.  -150.   -60.    .10   .50   .10 1226.28 1255.68
1295.78
  2    8.40  1    -60.  -150.   -60.    .10   .50   .10 1298.02 1322.09
1298.28
  3    8.40  1    -60.  -150.   -60.    .10   .50   .10 1298.47 1263.61
1238.19
  4    8.40  1    -60.  -150.   -60.    .10   .50   .10 1238.19 1263.61
1298.47
 Page 16                       (sdtypg4)                 ADAPT-PT  V- 7.10
ACI-02
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  5    8.40  1    -60.  -150.   -60.    .10   .50   .10 1298.28 1322.09
1298.02
  6    8.40  1    -60.  -150.  -100.    .10   .50   .10 1295.78 1255.68
1226.28
 ----------------------------------------------------------------------------
--
 Note: P= tendon profile (refer to legend of data block 9)
       Stresses at each location are the average of strands after anchor set,
       and after long-term losses


 16.8    TENDON SELECTION AND DATA:

                   <------ TENDON EXTENTS ------>    ELONGATION   Stress
ratios
 TYPE OFF FORCE    CAN<------ S P A N S ----->CAN    LEFT  RIGHT   Anch.
Max.
```

```
                     <1><2><3><4><5><6>            (mm)   (mm)
 -1----2----3---------------------4-------------------5------6-------7-----8-
--
  A   12  126.21      <==================>           319.    18.    .73
.75
  B    7  123.38      <===|                           55.     0.    .72
.72
  C    7  123.38                     |===>             0.    55.    .69
.72
 Note: Force is the average value per strand (kN)
       Stress ratios are at anchorage (7) and maximum along tendon (8)
```

APPENDIX B

DESIGN PROCEDURE

B.1 DESIGN OF POST-TENSIONED FLAT SLAB

Design parameters

Loads -

Dead Load = Self weight of the Slab.

Superimposed dead Load = 2.00 KN/m^2

Live Load = 4.00 KN/m^2

Tendon diameter = 12.70mm

Area of tendon = A_p = 99.00 mm^2

F_{pu} = 1862.0 N/mm^2

E_s = 195000 N/mm^2

E_c = 29580.4 N/mm^2

μ = 0.35

k = 0.0015

Creep coefficient Φ = 1.60

Column sizes -

Exterior column = 600.00mm x 600.00mm

Interior column = 600.00mm x 600.00mm

Height of the floor = 3 m

f_c = 35.00N/mm^2

f_y = 415.00N/mm^2

Thickness of slab= 200 mm

Thickness of drop panel = 350 mm

Thickness of wall = 200 mm

Estimated losses = 65.00 N/mm^2

Area = 1050000

Section modulus = 83390000

Moment of inertia= 8339000000

Load Calculation -

Dead Load due to self weight of slab = 5.00 KN/m^2

Superimposed dead Load = 2.00 KN/m^2

Live Load = 4.00 KN/m^2

Wall Load = 12.00 KN/m

f_{se} = 1238.40

P_{eff} = Effective force per tendon = A * f_{se}

P_{eff} = 122.60 KN

Allowable stresses -

At the time of jacking -

f_{ci} = 21

Compression = 0.6 * f_{ci} = 12.6 N/mm^2

Tension = 3 * $\sqrt{f_{ci}}$ = 1.15 N/mm^2

At Service Load -

f_c = 35 N/mm^2

Compression = 0.45 * f_c = 15.75 N/mm_2

Tension = 6 * $\sqrt{f_c}$ = 2.98 N/mm^2

Average precompression limit -

P / A = 0.87 N/mm^2

2.1 N/mm^2

Target load balances -

60 to 80% of dead load for slabs (only self weight). Let us assume 75%.

Moments in the different spans

Table B1 Moments in the slab

Span	Dead Load Moment			Live Load Moment		
	Left	Mid	Right	Left	Mid	Right
1	-280.34	87.54	-286.14	-110.48	35.55	-114.78
2	-281.94	86.17	-281.77	-113.18	35.03	-113.18
3	-281.84	86.19	-281.83	-113.18	35.03	-113.18
4	-281.84	86.19	-281.83	-113.18	35.03	-113.18
5	-281.78	86.17	-281.93	-113.18	35.03	-113.18
6	-286.15	87.54	-280.32	-114.78	35.55	-110.48

Table B1 continued

Span	Lateral Load Moment			Final moments		
	Left	Mid	Right	Left	Mid	Right
1	-8.57	0	-17.01	-399.39	123.09	-417.93
2	-17.01	0	-17.24	-412.13	121.2	-412.19
3	-17.24	0	-17.76	-412.26	121.22	-412.77
4	-17.76	0	-18.61	-412.78	121.22	-413.62
5	-18.61	0	-19.91	-413.57	121.2	-415.02
6	-19.91	0	-10.63	-420.84	123.09	-401.43

Shear forces in the span

Table B2 Shear force in the span of the slab

Span	Dead Load Shear		Live Load Shear		Total Shear	
	Left	Right	Left	Right	Left	Right
1	-233.52	181.21	-70.05	71.07	-303.57	252.28
2	-179.89	179.84	-70.57	70.55	-250.46	250.39
3	-179.87	179.86	-70.57	70.55	-250.44	250.41
4	-179.87	179.86	-70.57	70.55	-250.44	250.41
5	-179.85	179.88	-70.57	70.55	-250.42	250.43
6	-181.21	233.52	-71.07	70.05	-252.28	303.57

Reaction at joints

Table B3 Reaction at each joint

Joint	**Dead Load**	**Live Load**	**Total**
1	233.52	70.05	303.57
2	361.1	141.64	502.74
3	359.71	141.11	500.82
4	359.73	141.11	500.84
5	359.71	141.11	500.82
6	361.1	141.64	502.74
7	233.52	70.05	303.57

Reduced Moments at the face of column

Table B4 Reduced moments in the slab

Span	Dead Load Moment			Live Load Moment			Total Moment		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
1	-207	87.5	-217	-83.81	35.55	-87.69	-290.91	123.09	-304.99
2	-214	86.2	-214	-86.29	35.03	-86.25	-299.99	121.2	-299.95
3	-214	86.2	-214	-86.29	35.03	-86.25	-299.99	121.2	-299.95
4	-214	86.2	-214	-86.29	35.03	-86.25	-299.99	121.2	-299.95
5	-214	86.2	-214	-86.29	35.03	-86.25	-299.99	121.2	-299.95
6	-217	87.5	-207	-87.7	35.55	-83.8	-305.1	123.09	-290.9

Tendon Profile - Parabolic shape

For a layout with span of similar length, the tendons will be typically located at the highest allowable point at the interior columns, the lowest possible point at the midspan and the neutral axis at the anchor locations. This provides the maximum drape for load balancing.

Cover Provided -

Exterior span - At bottom = 50.00 mm

At top = 60.00 mm

Interior span - At bottom = 50.00 mm

At top = 60.00 mm

a_{INT} = 90.00 mm

a_{END} = 70.00 mm

Force needed in the tendon to counteract the load

Table B5 Tendon forces and number of tendons in span

Span	Length	Force	Tendon No
1	8.4	1313.1	17.71
2	8.4	1005.32	11.19
3	8.4	1005.55	10.20
4	8.4	1005.55	10.20
5	8.4	1005.32	11.19
6	8.4	1313.1	17.71

Check precompression allowance -

Determine number of tendons to achieve P -

No. of tendons = 17.71

Say 18 No.

$P_{act.}$ = 2206.83 KN

Determine actual precompression stresses -

$P_{act.}/A = 1.91\ N/mm^2$

$> 0.87\ N/mm^2$

$< 2.1\ N/mm^2$ HENCE OK.

Check net tensile stress at the face of column

$f_t = 1.39\ N/mm^2$

$< 2.98\ N/mm^2$ HENCE OK.

Check Midspan tensile stress

$f_t = -0.63\ N/mm^2$

$< 1.00\ N/mm^2$ HENCE OK.

Calculation of flexural strength

Minimum amount of steel required is given by

$A_s = 0.00075 \times A_{cf} = 787.50\ mm^2$

For average 1m strip

$A_s = 93.75\ mm^2$

Provide 8mm bar @ 200mm c/c on the both directions of slabs.

Reinforcement in the drop panel,

At support 1 and 7,

M = 276.3 KNm

Area of steel required -

$A_s = 0.5 x f_{ck}/f_y \times [1 - \sqrt{(1 - 4.6M/f_{ck}bd^2)}] \times b \times d$

$A_s = 1398.45\ mm^2$

Provide 16mm diameter bars @ 140 mm c/c at top and bottom of the drop panel.

At support 2 to 6,

M = 293.82 KNm

$A_s = 0.5 x f_{ck}/f_y \times [1 - \sqrt{(1 - 4.6M/f_{ck}bd^2)}] \times b \times d$

$A_s = 499.43\ mm^2$

Provide 12mm diameter bars @ 150 mm c/c at top and bottom of the drop panel.

Ultimate Shear Strength -

(a) Shear at Exterior column -

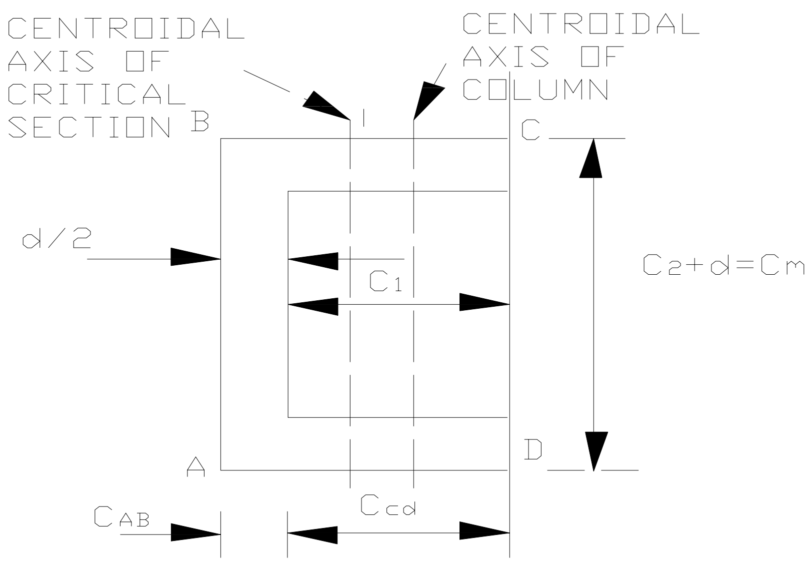
Vertical shear at the exterior column calculated as above is

V = 303.57 KN

Total shear = Vu = 455.36 KN

(b) Moment transfer -

The critical section properties are calculated as follows. The critical section for shear is taken at d/2 from the face of column drop.



$d = 200$ mm

$C_1 = 600$ mm, $C_2 = 600$ mm

$C_m = 800$ mm

$C_t = 700$ mm

$A_c = d\,(C_m + 2C_t) = 440000\ mm^2$

$C_{AB} = Ct_2 \times d / A_c = 222.73$ mm

$C_{CD} = C_t - C_{AB} = 477.27$ mm

$g = C_{CD} - C_1/2 = 177.27$ mm

$\alpha = 1 - [1/(1 + 2/3(C_m/C_t)1/2)]$

$\alpha = 0.416$

$J_c = d \times C_t^3/6 + C_t \times d^3/6 + C_m \times d \times C_{AB}^2 + 2 \times C_t \times d \times (C_t/2 - C_{AB})^2$

$Jc = 24839393939\ mm^4$

Total moment at column centre line -

M_u = -399.39 KNm

Moment transferred by eccentricity of shear reaction -

V_g = 80.72 KNm

Net moment to be transferred,

$M_t = M_u - V_g$ = -480.11 KNm

Amount of moment to be transferred by shear = $\alpha \times M_t$ = -199.79 KNm

Now,

$V_c = V_u / A_c + \alpha M_t \times C_{AB} / J_c$

V_c = 0.89 N/mm^2

Permissible shear stress -

The shear strength of concrete in slab is given by,

$\tau_c' = K_s * \tau_c$ Ref. IS 456:2000 Cl. 31.6.3.1

Where, $K_s = 0.5 + \beta \leq 1$

β = Ratio of short side to long side of column

β = 1.000

$K_s = 1.500 \leq 1$

$K_s = 1$

$\tau_c = 0.25 \sqrt{f_c}$

τ_c = 1.479

τ_c' = 1.479

> 0.89 N/mm^2 HENCE SAFE

(c) Shear at Interior column -

The direct shear left and right of interior column is calculated above.

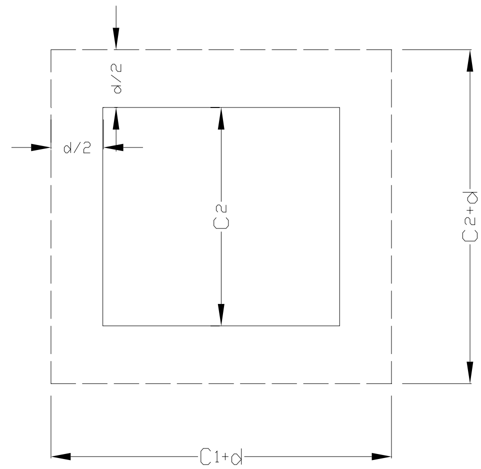
Total direct shear -

V = 502.74 KN

Moment transferred,

M_t = 41.236 KNm

Shear section properties -



$d = 200.00$

$C_1 = 600$ mm $\quad C_2 = 600$ mm

$d + C_1 = 800.00$

$d + C_2 = 800.00$

$b_0 d = 640000$

Polar moment of inertia -

$J = d \times (C_1+d)^3/12 + (C_1+d) \times d^3/12 + (C_2+d) \times d \times ((C_1+d)/2)^2$

$J = 69333333333$ mm^4

$Z = J / ((C_1 + d)/2)$

$Z = 173333333.3$ mm^3

Portion of moment to be transferred by tensional shear -

$\alpha = 0.400$

$M_{ut} = 16.49$ KNm

$M_{uf} = M_t - M_{ut} = 24.74$ KNm

Shear stresses -

Direct shear stress = V/b_0d = 0.786 N/mm^2

Torsional shear stress = M / Z = 0.095 N/mm^2

Total shear stress = 0.881 N/mm^2

Permissible shear stress -

The shear strength of concrete in slab is given by,

$\tau_c' = K_s * \tau_c$ Ref. IS 456:2000 Cl. 31.6.3.1

Where, $K_s = 0.5 + \beta \leq 1$

β = Ratio of short side to long side of column

β = 1.000

K_s = 1.500 ≤ 1

K_s = 1

τ_c = $0.25 \sqrt{f_c}$

τ_c = 1.479

τ_c' = 1.479

> 0.88 N/mm^2 HENCE SAFE

B.2 DESIGN OF REINFORCED CONCRETE FLAT SLAB

Data

Size of panel = 8.4m x 8.4m

Column size = 500mm x 500mm

Height of floor = 3m

f_c = 30 N/mm^2

f_y = 415 N/mm^2

Let us assume span/depth = 45

Depth of slab = 250 mm

Depth of drop = 350 mm

Load calculation:-

Dead load = 0.25 x 25 = 6.25 KN/m^2

Superimposed load = 2 KN/m^2

Live load = 4 KN/m^2

Total load = 12.25 KN/m^2

Calculation of bending moment:-

Clear span between column head

L_n = 8.4 - 3 = 5.4 m

=> 0.65 L= 5.46 m

The sum of positive and negative bending moment in a panel

$M_0 = wL_2L_n^2/ 8$

$= 12.25 \times 8.4 \times 5.46^2/8$

= 384 KNm

To calculate α_c

Clear height of column = height of floor – depth of drop panel

= 3- 0.35

= 2.65 m

Effective height of column = 2.65 x 0.8

L_c = 2.12 m

Stiffness of column = $I_c/L_c = bd^3/12L_c$

$= 600 \times 600^3/(12 \times 2120)$

$= 5.09 \times 10^6 \text{ mm}^4$

Stiffness of slab = $I_s/L_s = bd^3/12L_s$

$K_s = 8400 \times 225^3/(12 \times 8400)$

$= 1.30 \times 10^6$

$\alpha_c = \Sigma K_c/K_s$

= 2 x 5.09/1.30

= 7.83

$> \alpha_{cmin}$ HENCE OK.

a) Exterior panel

Negative bending moment at exterior support

$= 0.65M_o/(1+1/\alpha_c)$

= 0.65 x 384/(1+1/7.83)

= -221.33 KNm

Positive bending moment near midspan

$= [0.63-0.28/(1+1/\alpha_c)]\ M_0$

= [0.63 - 0.28/(1+1/7.83)] 384

= 146.58 KNm

Negative bending moment at interior support

$= [0.75-0.1/(1+1/\alpha_c)]\ M_0$

= [0.75-0.1/(1+1/7.83)] 384

= 253.95 KNm

b) Interior panel

Negative bending moment at interior panel

$= 0.65\ M_0 = 0.65 \times 384$

= 249.6 KNm

Positive bending moment near midspan

= $0.35M_0$ = 0.35x384

= 134.4 KNm

Negative bending moment at other exterior panel

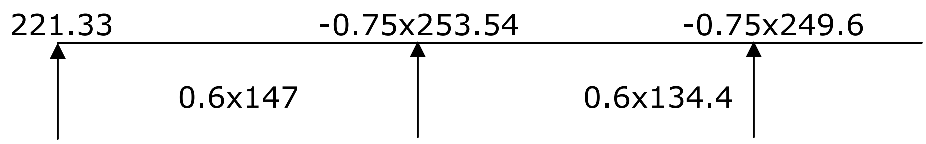
= -0.65 M_0 = -0.65x384

= -249.6 KNm

Distribution of bending moment across the width of panel

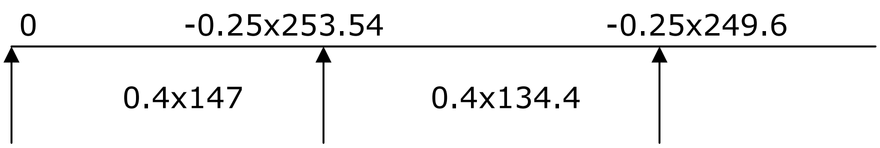
Column strip

(half width =0.25x8.4 = 2.1m)



Middle strip

(Width = 3m)



The absolute maximum bending moment occurs at the first exterior support in the column strip

= 221.33 KNm

Using M_{30} Grade concrete and Fe 415 Steel

$$M_u = 0.138\, f_{ck}bd^2$$

$$d = \sqrt{[M_u/(0.138 f_{ck} b)]}$$

$$d = \sqrt{[221.33\text{x}10^6/(0.138\text{x}30\text{x}4200)]}$$

$$d = 195.41\text{mm}$$

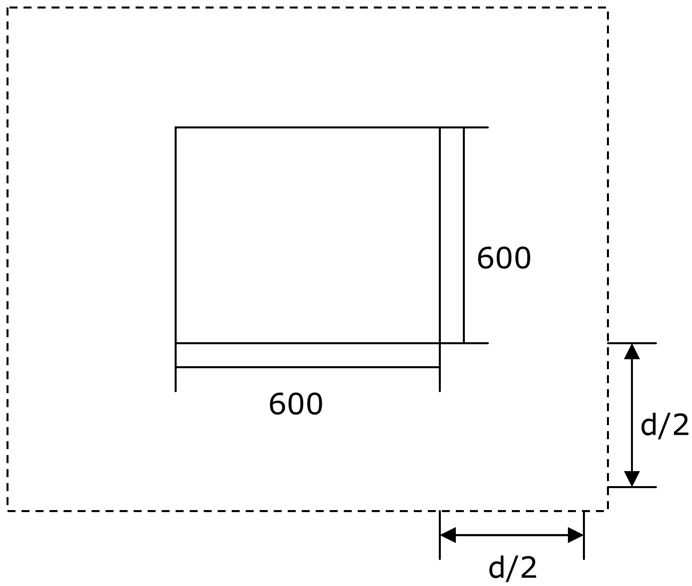
Therefore provide 225mm overall depth

Effective depth d = 200mm.

Design of shear

Let us check for shear at interior column. The critical section for shear occurs at d/2 from column head.

Effective depth = 200mm



Perimeter of critical section = 4 (600+200)

= 3200 mm

Total factored shear force = $1.5 \times 12.25 \times (8.4^2 - 0.825^2)$

V_o = 1284 KN

Nominal shear stress $\tau_v = V_0/b_0 d = 1284 \times 10^3 / 3200 \times 200$

$= 1.37 N/mm^2$

The shear strength of concrete = $0.25\sqrt{30}$

$\tau_c = 2 N/mm^2$

$> \tau_v$ HENCE OK

Design of reinforcement

a) Column strip

i) Negative steel at the exterior support

$$M_u = 0.87 f_y A_t (d - f_y A_t / f_{ck} b)$$

$$1.5 \times 221.33 \times 10^3 = 0.87 \times 415 \times A_f \times (200 - 415 \times A_f / 30 \times 4200)$$

$$A_t^2 - 60175 A_t + 276.67 \times 10^6 = 0$$

$$A_t = 5015.6 mm^2$$

Provide 16mm & @ 160mm c/c

ii) Positive steel near midspan of exterior panel.

$$1.5x0.6x147x10^6 = 0.87x415xA_t[200-(415A_t/4200x30)]$$

$$A_t^2-60175A_t + 110.25x10^6 = 0$$

$$A_t = 1891.62 \text{ mm}$$

Provide 12mm & @ 120mm c/c.

iii) Negative steel at first interior support

$$1.5x0.75x253.54 = 0.87x415xA_t[200-(415A_t/4200x30)]$$

$$A_t^2 - 60175A_t + 237.69x10^6 = 0$$

$$A_t = 4250 \text{ mm}^2$$

Provide 16 mm & @ 100mm c/c.

iv) Positive steel near midspan

$$A_t^2 - 60175A_t + 100.8x10^6 = 0$$

$$A_t = 1725 \text{ mm}^2$$

Provide 12 mm & @ 130mm c/c.

v) Negative steel at the other interior support

$$A_t^2 - 60175A_t + 280.8x10^6 = 0$$

$$A_t = 5100 \text{ mm}^2$$

Provide 16mm & @ 175mm c/c

b) Middle strip

i) Positive steel near the midspan of exterior panel

$$1.5x0.4x147x10^6 = 0.87x415xA_t [200-(415A_t/4200x30)]$$

$$A_t^2 - 60175 A_t+73.5x10^6 = 0$$

$$A_t = 2495\text{mm}^2$$

Provide 12 mm & @ 190mm c/c.

ii) Negative steel at interior support

$$A_t^2 - 60175 A_t + 79.29x10^6 = 0$$

$$A_t = 1347.84 \text{ mm}^2$$

Provide 12 mm & @ 200mm c/c.

Min reinforcement in slab = 0.12 x 4200 x 200/100

= 1008 $\text{mm}^2 < 1347 \text{ mm}^2$ HENCE OK.

Provide 12 mm & @ 175mm c/c.

B.3 DESIGN OF TWO WAY SLAB

Data

Length in short direction,

L_x = 4.00 m

Length in long direction,

L_y = 4.20 m

Grade of concrete -

f_{ck} = 30.00 N/mm^2

Grade of steel -

f_y = 415.00 N/mm^2

Loads

Dead load = self weight of slab

Floor finish = 1.50 KN/m^2

Live Load = 4.00 KN/m^2

Thickness of slab -

Span / depth = 35

Depth, d = 114.29 mm

Let us consider 10 mm diameter bar with 15 mm cover to the reinforcement.

Overall depth D = 134.29 mm

Say 140.00 mm

Effective depth d = 120.00 mm

Load calculation -

Dead Load = 3.36 KN/m^2

Floor finish = 1.50 KN/m^2

Live Load = 4.00 KN/m^2

Total = 8.86 KN/m^2

Factored Load = 13.29 KN/m^2

Maximum factored moment -

Ly / Lx =1.05

Therefore, α_x = 0.066

α_y = 0.0615

$M_x = \alpha_x wL_x^2$

M_x = 14.03 KNm

M_y = αy wLx^2

M_y = 13.07 KNm

Check for depth -

M_{max} = $0.138 f_{ck} bd^2$

d = 58.21 mm

< 120.00 mm HENCE OK.

Area of steel required

a) In short direction -

M/bd^2 = 0.974

P_t = 0.289 %

Area of steel,

A_{st} = 346.80 mm^2/m

Provide 8 mm ϕ @ 140mm c/c spacing in short direction.

b) In long direction -

M/bd^2 = 0.907

P_t = 0.259 %

Area of steel,

A_{st} = 310.80 mm^2/m

Provide 8 mm ϕ @ 160mm c/c spacing in short direction.

Spacing < 3d = 360.00 mm

> 160.00 mm HENCE OK.

Hence provide 8 mm ϕ @ 140 mm c/c spacing in both the directions.

Check for shear -

Maximum design shear = wL_x / 2

V = 26.57 KN

τ_v = V / bd

τ_v = 0.22 N/mm^2

Percentage of steel provided = 0.262 %

τ_c = 0.40 N/mm^2

> 0.22 N/mm^2 HENCE OK.

Hence slab is safe in shear, no shear reinforcement is required.

Check for cracking

a) Percentage of steel provided = 0.262 %

> 0.12 % (minimum)

b) Spacing < 3d = 360.00 mm

> 140.00 mm

c) Diameter of bar used < 16.78

8 < 16.78 HENCE OK.

Tortional reinforcement

For the negative moment at the corner of the slab we provide the reinforcement at each corner of the slab. So provide 8 mm ϕ Bars @ 150 mm c/c at top and bottom of each corner.

B.4 DESIGN OF ONE WAY SLAB

DATA

Dimensions of slab = 4.25 m x 8.4m

Live load = 4 KN/m^2

Superimposed dead load = 2 KN/m^2

Grade of concrete = M30

Grade of steel = Fe415

To calculate factored load

Span = 4.25 m

Span/ d = 35 -------------for simply supported

Therefore

d= 4250/35 = 121.42mm $\approx$ 125mm

Assuming 10 mm& bars & 20 mm cover

Total depth (D) = 125+5+20 = 150mm

Self weight of slab = 0.15 x 25

= 3.75 KN/m^2

Superimposed dead load =_2 KN/m^2

Total dead load = 4.75 KN/m^2

Live load = 4 KN/m^2

Factored (design) load = 1.5 (4.75+4)

$= 13.125\ KN/m^2$

Span length of slab

Span = effective span + d = 4.25 + .125

= 4.375 m

Total load on span per meter width

W = 4.375 x13.125 = 57.421 KN

Ultimate moment and shear

M = WL/8 = 57.421x4.375/8

= 31.4 KNm

V = W/2 = 57.421/2

= 28.71 KN

Check depth for bending

For Fe415, $M = 0.138\ f_{ck}bd^2$

$d = \sqrt{(M/(0.138 f_{ck}\ b))}$

$= \sqrt{(31.4x10^6/(0.138x25x1000))}$

= 95.4 mm < 125mm HENCE OK

Hence provide the depth of 140 mm.

Calculation for shear.

$\tau_v = V/bd = 28.71x10^3/1000x125$

$= 0.229\ N/mm^2$

$\tau_c = 0.36$ (min) for M25 grade

Therefore $\tau_c > \tau_v$ HENCE OK

Calculation for steel areas

i) From formula of neutral axis depth

$x/d = 1.2 - \sqrt{[(1.2^2) - 6.6M/f_{ck}\ bd^2]}$

$= 1.2 - \sqrt{[1.44 - 6.6 \text{ x } 31.4x10^6/(25x1000x125^2)}$

x/d = 0.25 < 0.5, HENCE OK

Z = d(1-0.416 x/d)

= 125 (1- 0.416x0.25)

Z = 112 mm

$A_{st} = M/(0.87f_yZ) = 31.4x10^6/(0.87x415x112)$

$= 776.5\ mm^2$

ii) By SP-16 procedure

$M/bd^2 = 31.4x10^6/ 1000x125^2 = 2$

$P_t = 0.618$ %

$A_{st} = 0.618x1000x125/100$

$= 772.5\ mm^2$

iii) Main steel

Spacing = 78.54x1000/772.5 = 101.67 mm

Provide 10 mm & @ 100mm c/c.

$A_{stprov.} = 78.54x1000/100$

$= 785.4\ mm^2$

iv) Distribution steel / secondary steel

$A_s = 0.12x1000x150/100 = 180\ mm^2$

Spacing for 8mm &

$S_v = 51x1000/180 = 250$ mm

Provide 8mm & @ 250mm c/c.

Spacing < 5d = 5x125 = 625 mm or

< 450 mm HENCE OK

Check for control of crack

Min $P_t = 0.12\%$

$A_s = 180\ mm^2$

$< 772.5\ mm^2$ HENCE OK

$\phi = 10 < 125/8$ (15.625mm)

Max spacing = 3d = 3x 125 = 375 mm or

= 300 mm

> 100 mm HENCE OK

Recheck for shear

P = 785.4x100/1000x125 = 0.628%

$f_{ck} = 25\ N/mm^2$

$\tau_c = 0.53\ N/mm^2$

$> 0.229 N/mm^2$

Therefore slab is safe in shear.

Check for deflection

Basic span depth ratio = 35

Factor F_1 for P_t (= 0.628) = 1.2

For f_s = 240

f_s= 0.58f_y A_{streqd}./ $A_{stprovd}$.= 236.74

= 237

Allowable L/d = 1.2 x 35 = 42

Assumed L/d = 35

< 42.

Hence safe in deflection.

B.5 DESIGN OF THE COLUMN

DATA

Size of column = 600 mm x 600 mm

b = 600 mm

D = 600 mm

Concrete mix = f_{ck} = 25 N/mm^2

Characteristic strength of steel = f_y = 415 N/mm^2

Factored load = P_u = 2128.14 KN

Factored moments -

M_{ux} = 132.22 KNm

M_{uy} = 115.20 KNm

Cover to the reinforcement = 40 mm

Diameter of bar to be used = 25 mm

Reinforcement is equally distributed on four sides.

For the first trial assume the percentage of reinforcement

p = 1.65

p/f_{ck} = 0.066

Uniaxial moment capacity of the section about xx-axis

d' = 52.5

d'/D = 0.0875

Chart for d'/D = 0.1 will be used.

P_u/f_{ck}.b.D = 0.23646

Referring to chart 44,

$M_u/f_{ck}.b.D^2 = 0.11$

$M_{ux1} = 594.00$ KNm

Uniaxial moment capacity of the section about y-y axis

d'/D = 0.0875

Chart for d'/D = 0.1 will be used.

Referring to chart 44,

$M_u/f_{ck}.b.D^2 = 0.11$

$M_{uy1} = 594.00$ KNm

Calculation of P_{uz}

Referring to chart 63 and corresponding to

p = 1.65

$f_y = 415$ N/mm^2

$f_{ck} = 25$ N/mm^2

$P_{uz}/A_g = 14$ N/mm^2

$P_{uz} = 5040.00$ KN

$P_u/P_{uz} = 0.422$

$M_{ux}/M_{ux1} = 0.223$

$M_{uy}/M_{uy1} = 0.194$

Referring to chart 64 the permissible value of M_{ux}/M_{ux1} corresponding to the above M_{uy}/M_{uy1} and P_u/P_{uz} is equal to 0.219. The actual value is only slightly higher than the value read from the chart. This can be made up by slight increase in reinforcement.

A_s = 5940 mm2

Provide 10 - 16 mm ϕ and 8 – 25 mm ϕ.

The area provided is,

A_{sp} = 5940 mm^2

Reinforcement percentage provided,

p = 1.65

With this percentage the section may be rechecked.

$p/f_{ck} = 0.066$

Referring to chart 44,

$M_u/f_{ck}.b.D^2$ = 0.11

M_{ux1} = 594.00 KNm

Referring to chart 44,

$M_u/f_{ck}.b.D^2$ = 0.11

M_{uy1} = 594.00 KNm

Referring to chart 63,

P_{uz}/A_g = 14

P_{uz} = 5040.00 KN

P_u/P_{uz} = 0.422

M_{ux}/M_{ux1} = 0.223

M_{uy}/M_{uy1} = 0.194

Referring to chart 64 corresponding to the above values of M_{uy}/M_{uy1} and P_u/P_{uz}, the permissible value of M_{ux}/M_{ux1} is 0.219.

Hence the design is safe.