STUDIES IN BEAMLESS RC STRUCTURAL FLOOR SYSTEMS FOR MEDIUM FLOOR-PLANS

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled **"STUDIES IN BEAMLESS RC STRUCTURAL FLOOR SYSTEMS FOR MEDIUM FLOOR-PLANS"** in partial fulfilment of the requirements for the award of the Degree of Doctor of Philosophy and submitted in the Department of Civil Engineering of the Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out during a period from January, 2003 to May, 2006 under the supervision of **Dr. Jagdish Prasad** and **Dr. A. K. Ahuja**.

The matter presented in this thesis has not been submitted by me for the award of any other degree of this or any other Institute.

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ABSTRACT

The control of deflection and punching shear govern the design of RC beamless floors unlike the other two way slabs supported by beams. Since the thin slab is supported directly by columns, the time dependent measured deflections are more when compared with the calculated deflections. Though the building may be safe against the strength aspects, the excessive deflections lead to lack of feeling of safety in the occupants of the building that may even question the proficiency of the designer. Another main problem in these beamless slabs is the brittle punching shear failure.

Existing research on flat plates has shown extensive study on either eliminating the punching shear failure or to change the mode of brittle failure to flexural failure mode, by using different arrangements of reinforcement. Prefabricated systems are also available to address the punching shear problems. But in many of those systems, either the depth of slab is to be increased or skilled labour is needed to fix them in position. Hence the main aim of this research was concentrated in improving the stiffness of slab without increasing the thickness of slab.

To address the above problems effectively, the concept of providing the concealed beam in the slab was considered for this research. The concealed beam is an arrangement of reinforcement having equal number of longitudinal reinforcement in both tension and compression face of the slab for a fixed width tied together by means of shear stirrups along the transverse direction. Since the column strip plays a major role in transmitting the load, the stiffness of the column strip must be improved.

To utilise the benefits of beamless slabs without much compromise in safety and serviceability, the concealed beam can best be placed within the column strip. This study was aimed to give a theoretical model for the concealed beam and to study the effect of the same in terms of deflection, moment and shear carrying capacity of slab. The modelling of the concealed beam was made, by using the normal reinforced concrete theory for transformed section for flexure. For torsion, the concealed beam was modelled as a thin equivalent concrete box.

The Equivalent Frame method was used to study the stiffness of the flat plate and flat slab with column head using concealed beam. Since the depth of the slab is the main object, the required depth based on long-term deflection was to be found. For this, the provisions in the codes ACI: 318-2002, BS: 8110-1997, EC: 2-2002 and IS: 456-2000 were studied and the available empirical beam formulas have been used with suitable modifications by applying the Branson approach.

A parametric study was conducted by using the software developed for the modified empirical formulas as per the above codes. A modification to the multiplying factor in the Branson formula was proposed based on this study. The results using the proposed formula was verified with the experimental results available in the literature and found that there was good agreement between them.

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NOTATIONS

A limited number of notations used commonly in this thesis are given here. The other symbols are explained in the relevant section itself.

A_{sc1cx} = area of compression reinforcement in column strip along x-x axis

A_{sc1cy} = area of compression reinforcement in column strip along y-y axis

A_{st1cx} = area of tension reinforcement in column strip along x-x axis

A_{st1cy} = area of tension reinforcement in column strip along y-y axis

A_{st1mx} = area of tension reinforcement in middle strip along x-x axis

A_{st1my} = area of tension reinforcement in middle strip along y-y axis

b_{cx} = width of column strip along x-x axis

b_{cy} = width of column strip along y-y axis

b_{mx} = width of middle strip along x-x axis

b_{my} = width of middle strip along y-y axis

b_{fx} = width of frame along x-x axis

b_{fy} = width of frame along y-y axis

C_1 = width of column along the direction of moment considered

C_2 = width of column along the transverse direction

C_b = torsional constant of the concealed beam

C_x = torsional constant of the column along x-x axis

cc = clear cover to tension reinforcement

d = effective depth of slab

d_1 = depth concealed beam in equivalent area of concrete

d_s = depth of stirrups provided in concealed beam

D = total depth of slab

D_{px} = distribution factor for positive moment in exterior span along x-x axis

D_{nex} = distribution factor for negative moment in exterior span along x-x axis

D_{nix} = distribution factor for negative moment in interior span along x-x axis

D_{py} = distribution factor for positive moment in exterior span along y-y axis

D_{ney} = distribution factor for negative moment in exterior span along y-y axis

D_{niy} = distribution factor for negative moment in interior span along y-y axis

E_c = modulus of elasticity of concrete

E_{ce} = modified modulus of elasticity of concrete

E_s = modulus of elasticity of steel

FF = floor finish

f_{ck} = characteristic compressive strength of concrete

f_{ctm} = tensile strength of concrete

f_{cr} = modulus of rupture of concrete

I = moment of inertia

I_1 = moment of inertia in the column strip for flat slab

I_2 = moment of inertia in the middle strip for flat slab

I_{cx} = moment of inertia of column strip along x-x axis

I_{cy} = moment of inertia of column strip along y-y axis

I_{c1x} = moment of inertia of column along x-x axis

I_{c1y} = moment of inertia of column along y-y axis

I_{eff} = effective moment of inertia

I_{efx} = effective moment of inertia of frame along x-x axis

I_{efy} = effective moment of inertia of frame along y-y axis

I_{fx} = moment of inertia of frame along x-x axis

I_{fy} = moment of inertia of frame along y-y axis

I_{grcx} = gross moment of inertia of column strip along x-x axis

I_{grcy} = gross moment of inertia of column strip along y-y axis

I_{grmx} = gross moment of inertia of middle strip along x-x axis

I_{grmy} = gross moment of inertia of middle strip along y-y axis

I_{mx} = moment of inertia of middle strip along x-x axis

I_{my} = moment of inertia of middle strip along y-y axis

K_{s1x} = slab stiffness along x-x axis

K_{s1y} = slab stiffness along y-y axis

K_{tx} = torsional stiffness of the column along x-x axis

K_{ty} = torsional stiffness of the column along y-y axis

l_1 = width of concealed beam

L_c = storey height

L_x = span along x-x axis

L_y = span along y-y axis

LDF_c = lateral moment distribution factor for column strip

LDF_m = lateral moment distribution factor for middle strip

m = modular ratio

m_{ce} = modified modular ratio

M_{crx} = cracked moment along x-x axis

M_{cry} = cracked moment along y-y axis

n = number of longitudinal bars provided concealed beam

x = depth of neutral axis

x_{1cxs} = depth of neutral axis (short term) in column strip along x-x axis

x_{1cxl} = depth of neutral axis (long term) in column strip along x-x axis

x_{1mxs} = depth of neutral axis (short term) in middle strip along x-x axis

x_{1mxl} = depth of neutral axis (long term) in middle strip along x-x axis

w = uniformly distributed panel load

W_d = sustained load

W_D = factored sustained load

W_l = superimposed load

W_L = factored superimposed load

W_t = total load

W_T = factored total load

$\frac{1}{r_{cs1}}$ = shrinkage curvature in uncracked stage

$\frac{1}{r_{cs2}}$ = shrinkage curvature in cracked stage

$\frac{1}{r_1}$ = creep curvature in uncracked stage

$\frac{1}{r_2}$ = creep curvature in cracked stage

ψ_{cs} = shrinkage curvature

ε_{cs} = shrinkage strain

ξ = factor considering the steel stress in both uncracked and cracked stage

δ = deflection at mid panel

δ_{tx} = deflection due to rotation for total load along x-x axis

δ_{ty} = deflection due to rotation for total load along y-y axis

δ_{dx} = deflection due to rotation for sustained load along x-x axis

δ_{dy} = deflection due to rotation for sustained load along y-y axis

δ_{t1x} = deflection due to rotation for total load (short term effects) along x-x axis

δ_{t1y} = deflection due to rotation for total load (short term effects) along y-y axis

δ_{d1x} = deflection due to rotation for sustained load (short term effects) along x-x axis

δ_{d1y} = deflection due to rotation for sustained load (short term effects) along y-y axis

δ_{t2x} = deflection due to rotation for total load (long term effects) along x-x axis

δ_{t2y} = deflection due to rotation for total load (long term effects) along y-y axis

δ_{d2x} = deflection due to rotation for sustained load (long term effects) along x-x axis

δ_{d2y} = deflection due to rotation for sustained load (long term effects) along y-y axis

δ_{DL} = deflection at mid panel due to dead load

δ_{LL} = deflection at mid panel due to live load

δ_{cs} = deflection due to shrinkage

$\delta_{i,cc(perm)}$ = instantaneous deflection due to creep for permanent loads

$\delta_{i,(temp)}$ = instantaneous deflection due to loads of temporary nature

δ_{cx} = deflection of column strip along x-x axis

δ_{mx} = deflection of middle strip along x-x axis

δ_{cy} = deflection of column strip along y-y axis

δ_{my} = deflection of middle strip along y-y axis

δ_{cxitl} = deflection of column strip (interior panel) along x-x axis for total load

δ_{cxidl} = deflection of column strip (interior panel) along x-x axis for sustained load

δ_{mxitl} = deflection of middle strip (interior panel) along x-x axis for total load

δ_{mxidl} = deflection of middle strip (interior panel) along x-x axis for sustained load

δ_{cyitl} = deflection of column strip (interior panel) along y-y axis for total load

δ_{cyidl} = deflection of column strip (interior panel) along y-y axis for sustained load

δ_{myitl} = deflection of middle strip (interior panel) along y-y axis for total load

δ_{myidl} = deflection of middle strip (interior panel) along y-y axis for sustained load

δ_{cxctl} = deflection of column strip (corner panel) along x-x axis for total load

δ_{cxcdl} = deflection of column strip (corner panel) along x-x axis for sustained load

δ_{mxctl} = deflection of middle strip (corner panel) along x-x axis for total load

δ_{mxcdl} = deflection of middle strip (corner panel) along x-x axis for sustained load

δ_{cyctl} = deflection of column strip (corner panel) along y-y axis for total load

δ_{cycdl} = deflection of column strip (corner panel) along y-y axis for sustained load

δ_{myctl} = deflection of middle strip (corner panel) along y-y axis for total load

δ_{mycdl} = deflection of middle strip (corner panel) along y-y axis for sustained load

δ_{cxstl} = deflection of column strip (side panel) along x-x axis for total load

δ_{cxsdl} = deflection of column strip (side panel) along x-x axis for sustained load

δ_{mxstl} = deflection of middle strip (side panel) along x-x axis for total load

δ_{mxsdl} = deflection of middle strip (side panel) along x-x axis for sustained load

δ_{cystl} = deflection of column strip (side panel) along y-y axis for total load

δ_{cysdl} = deflection of column strip (side panel) along y-y axis for sustained load

δ_{mystl} = deflection of middle strip (side panel) along y-y axis for total load

δ_{mysdl} = deflection of middle strip (side panel) along y-y axis for sustained load

θ_{tx} = rotation due to total load along x-x axis

θ_{ty} = rotation due to total load along y-y axis

θ_{dx} = rotation due to sustained load along x-x axis

θ_{dy} = rotation due to sustained load along y-y axis

θ_{t1x} = rotation due to total load (short term effects) along x-x axis

θ_{t1y} = rotation due to total load (short term effects) along y-y axis

θ_{d1x} = rotation due to sustained load (short term effects) along x-x axis

θ_{d1y} = rotation due to sustained load (short term effects) along y-y axis

θ_{t2x} = rotation due to total load (long term effects) along x-x axis

θ_{t2y} = rotation due to total load (long term effects) along y-y axis

θ_{d2x} = rotation due to sustained load (long term effects) along x-x axis

θ_{d2y} = rotation due to sustained load (long term effects) along y-y axis

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Reinforced concrete (RC) beamless floors lead to architecturally pleasing buildings as well as simplifying and accelerating site operations. They allow easy and flexible partitioning of space and reduce the overall height of tall buildings. Beamless ceiling facilitates minimum structural depth, and hence, allows for maximum flexibility in the arrangement of air-conditioning ducts and light fixtures. Flat plates and flat slabs are two-way beamless floors that are supported directly by columns.

1.1.1 Flat Plates

Flat plates are two-way slabs, used to transfer vertical loads directly to columns without the use of beams (Fig. 1.1). This floor is suitable for hotels, short span office and residential buildings. They need the minimum overall storey heights (less than 3 m) to provide specific headroom requirements. They also give more flexibility in the column layouts and partitions.

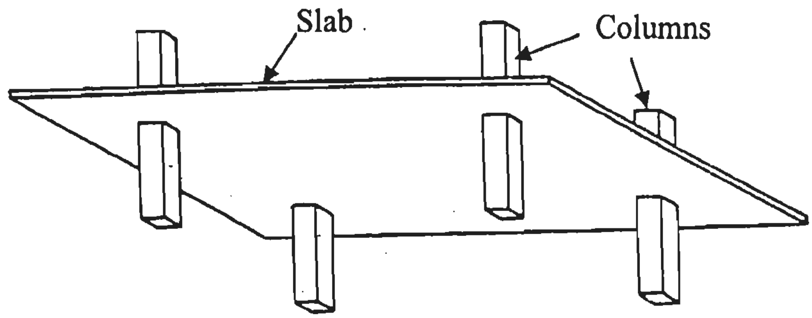


Fig. 1.1 Flat plate floor

1.1.2 Flat Slabs

Flat slabs are also two-way systems of beamless construction. But they incorporate a thickened slab in the region of columns (Fig. 1.2). The thickened parts at the slab column junction are referred to as drop panels and column capitals. These reduce the shear and negative bending stresses around the columns. They are also effective in controlling the floor deflection. The flow chart (Fig. 1.3) explains the advantages of beamless floors.

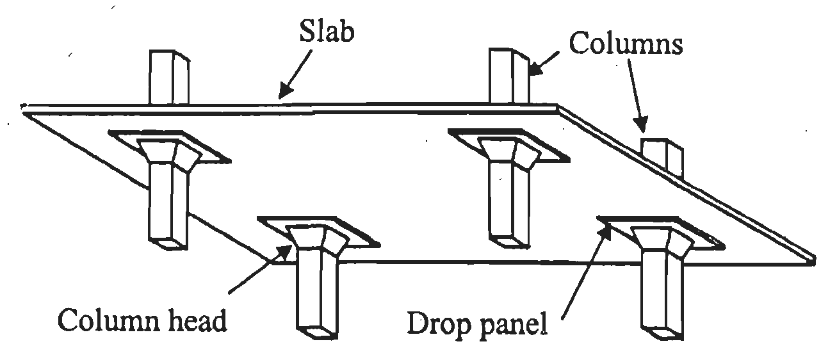


Fig. 1.2 Flat slab floor

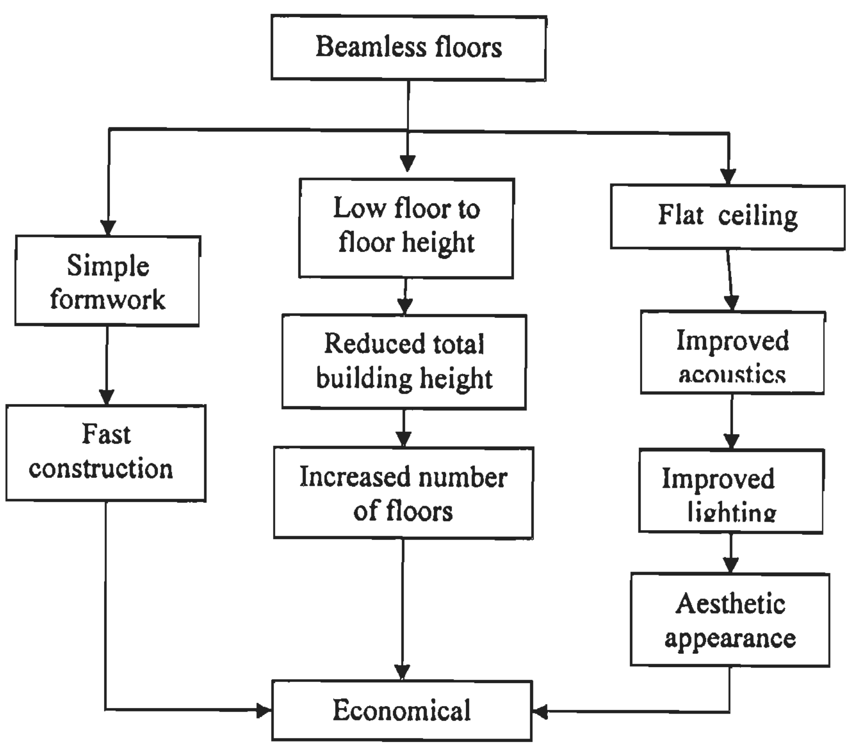


Fig. 1.3 Advantages of beamless floors

1.2 MOTIVATION OF THE PROPOSED RESEARCH

1.2.1 Shear

In the case of beam supported slabs, the load is distributed uniformly to the beams. But in beamless floors the load is transferred from both the directions to the column that covers only a small area (Fig.1.4). When shear forces are concentrated over a small area, they are referred to as punching shear. The shear forces and moment transfer produce vertical shear stresses at the slab column junction. The critical section extends around the column

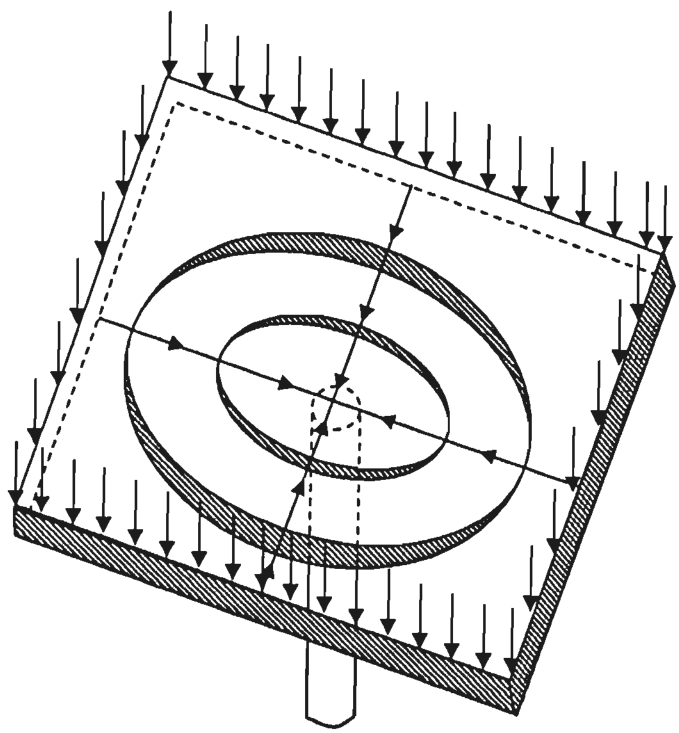


Fig. 1.4 Load transfer from slab to column

The high shear forces and unbalanced moments around the supporting columns (corner and edge columns) render the flat plate structure susceptible to brittle punching shear failure. A punching shear failure occurs along a truncated cone or pyramid caused by the critical diagonal tension crack around the column (Fig. 1.5).

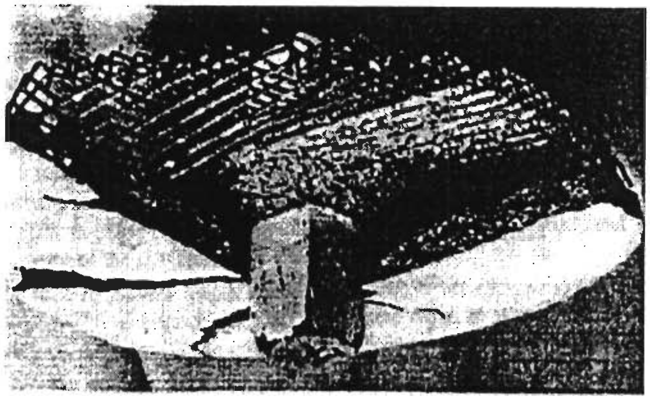
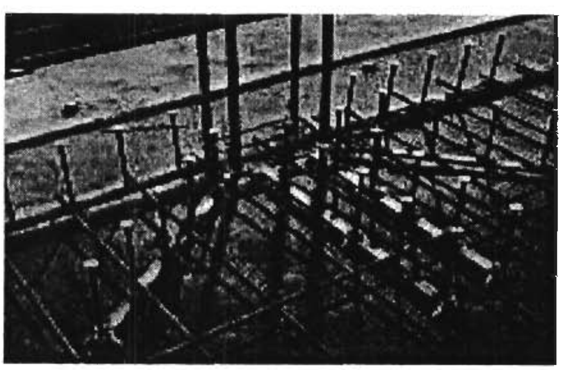


Fig. 1.5 Punching shear failure [Park & Gamble, 2000]

Shear failure in slab column connection can result in progressive failures of adjacent connections to become more heavily loaded. Also the lower floors may fail progressively as they become unable to support the impact of material dropping from above. Hence the punching shear becomes critical and governs the design in this case.

Gomes et al. (1999) conducted experiments for flat plates with and without shear reinforcement. They found that the punching failure load was doubled by using the shear reinforcement. Olivera et al. (1999) studied the strength of flat plates with vertical and inclined stirrups. They observed that the performance of slab was better with inclined stirrups. Stud rails, ACI shear stirrups, shear ladders, shear hoops and structural steel shear heads [Best practice guide, 2000] are some of the prefabricated systems (Fig. 1.5) used to eliminate the punching shear failure in flat plates.



a. Stud rails

b. Shear ladder

Fig. 1.6 Prefabricated reinforcement systems [Best practice guide, 2000]

Binici et al. (2003) used Carbon fibre reinforced polymers in the form of strips. Chen et al. (2005) strengthened the slab column connection by Glass fibre-reinforced polymer (GFRP) laminates. These stems need skilled labour to fix them in position. Some of the prefabricated systems required increase in the depth of the slab to have the proper anchorage.

1.2.2 Deflection

The magnitude of the deflection for RC floors is important because excessive deflections may cause sagging floors. It may damage the appearance of the building and create a lack of feeling of safety in the occupants of the building. Hence the structure used by people should be quite rigid so as to provide a sense of security.

Codes suggest that this limit state of serviceability can be satisfied by adopting suitable maximum allowable deflections. The codes give a simpler method to provide thickness in the form of span to depth ratio. Little guidance is given in the codes for calculation of the long term deflection of flat plates and flat slabs. Literatures have reported that certain buildings have collapsed due to excessive deflection [Beeby, 2002].

For deflection calculation, Rangan (1976) had developed an expression for flat plates and flat slabs. Chang et al. (1996) proposed an algebraic equation for determining the deflection of two way slab systems. Improvements are to be made in these equations by considering the existing material properties.

The designer has to go for elaborate calculation based on the need of the user. This calculation is lengthy and time consuming. Hence it is necessary to have a simplified approach considering the difficulties faced by the designer.

At the design stage, there are several ways of avoiding punching shear failure and deflection such as

- Reducing the applied loads;
- Reducing the effective length of slabs.

- Increasing the overall thickness of the slab.
- Providing some kind of shear reinforcement to avoid punching shear failure.

The dimensioning of the members in the preliminary stage is the essential design requirement, which must satisfy both the limit state of strength and serviceability. The thickness can very well be increased to avoid the problems due to deflection and punching shear failure. But this increase will result in increased dead load moment. Since the flat slab is vulnerable to deflection due to long-term effects, there should be an optimum thickness that satisfies both the problems.

In order to make the effective use of the advantages of beamless floor systems, it is necessary to improve its stiffness. Amongst the various methods, provision of the concealed beam along the column strip of the slab appears to be the best method to improve the stiffness of the beamless floor systems. The theory and the standard design procedures are not available for the concealed beam. In the present study, an attempt is made to develop an analytical for the concealed beam.

1.3 OBJECTIVE OF THE PROPOSED RESEARCH

To improve the stiffness of the flat plate and flat slabs, concealed beam is provided. It would give better serviceability (deflection control including long term effects) and improved strength in terms of shear and moment carrying capacity without increasing the thickness of the slab. The objective of the present study, therefore, is to compare the performance of flat plates and flat slabs with and without concealed beams under service loads.

1.4 METHODOLOGY

To achieve the above objective of the research, the methodology planned to be adopted are explained in the following steps.

i. To carry out the literature survey on the control of deflection including the effects of creep & shrinkage and punching shear of flat plates and flat slabs.

ii. To compare the available provisions in the four codes of practice namely (i) ACI: 318-2002 [1] (ii) BS: 8110-1997 [3] (iii) EC: 2-2002 [4] and (iv) IS: 456-2000 [6] for the prediction of long term deflections in flat plates and flat slabs.

iii. To determine the mid panel deflection for interior, corner and side panel of flat plates

- The available beam formulae as per the above listed codes are to be used with suitable modifications to consider the two way behaviour of the slab.
- To conduct an extensive parametric study a computer program is to be developed as per the above model.
- To assess the influence of the parameters such as span, load, grade of concrete, area of steel, clear cover to reinforcement, in total mid panel deflection of flat plate.
- To study the contribution of creep and shrinkage effects in the total deflection.

iv. To find the equation that predicts the total deflection that represents the actual deflection among those suggested by the four codes.

v. To arrive at a simplified approach that can be used for deflection control of flat plates and flat slabs at the design stage.

vi. To propose the span up to which the flat plate, flat slab with column head, drop panel, column head & drop panel are suitable.

vii. To determine the moment and shear carrying capacity of the slab due to gravity loads by using the Equivalent Frame Method.

viii. To model the concealed beam suitably in flexure, shear and torsion to analyse the stiffened flat plate and flat slab with column head by using the Equivalent Frame Method.

ix. To find the deflection, moment and shear carrying capacity of flat plates and flat slab with column head by using the proposed model for concealed beam based on the above study.

1.5 ORGANISATION OF THESIS

The achievement of these objectives is presented in the following chapters of the thesis. In total a number of seven chapters are contained in the present work. Their content is outlined in the following paragraphs.

The second chapter presents a comprehensive literature review on the deflection, punching shear and other aspects of the flat plates and flat slab.

The third chapter describes the structural modelling of the slab and concealed beam, and the theoretical aspects, adopted for this research. The fourth chapter presents the details of the empirical formulas studied and modified by using the proposed structural modelling to predict the long term deflection as per ACI: 318-2000, BS: 8110-1997, EC2: 2002 and IS: 456-2000.

The fifth chapter gives briefly about the developed computer program for the equations furnished in the previous chapter and the parametric study conducted using this program. And also the effect of each parameter in the total deflection of flat plates is explained in detail based on the parametric study. A modification in the present multiplier approach is proposed in this section and validated with the existing results reported in the literature.

The sixth chapter describes the procedure involved in the calculation of deflection (using the proposed approach), moment and shear carrying capacity of beamless floors with and without concealed beam by solving numerical examples. The comparisons of results obtained are discussed at the end of this chapter.

By using the developed program for the procedure explained in the previous chapter, the detailed study was carried out to find the stiffness of flat plates and flat slab with column head using the concealed beam. The results obtained for the different parameters are presented in the seventh chapter. The improved stiffness values in terms of reduced deflection, increased moment and shear strength are compared and discussed in this chapter.

The thesis concludes with the eighth chapter in which general conclusions from the present thesis are discussed. Recommendations for future research are also made at the end.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The beamless slabs have the advantage of plain ceiling, which gives aesthetic appearance and other benefits. But the long term deflection and the brittle punching shear failure are the major problems which need to be solved. For deflection at present the provisions suggested in the codes are not sufficient to meet the actual requirement. The stiffness of the slabs is less compared to the conventional slabs supported by the beams. This chapter tries to analyze the problems faced in the beamless floors. These are divided into deflection, punching shear and the other aspects.

2.2 DEFLECTION

Rangan [1976] developed an approximate method to compute the long term deflection of flat plates and flat slabs. In this method crossing beam analogy was used to find the mid panel deflection. This method uses only unit width of slab at the mid span. Rangan and Mcmullen [1978] criticized the codes ACI 318-71 and AS 1480-74 that the minimum thicknesses suggested in the codes have resulted in a long term deflection greater than $l_l/240$.The expressions developed by them are still used to predict the long term deflection of flat plates and flat slabs. This expression was evaluated by them with the existing building deflection. But improvements are to be made in this expression considering the existing material properties. Also for the edge column and corner column the torsional stiffness is not considered in the crossing beam analogy.

Chang and Hwang [1996] proposed an algebraic equation for determining the elastic deflection of two-way slab systems subjected to uniform gravity loads. They derived the equation from the differential equation of plate theory and then calibrated using the results of finite element analysis. This expression was common for all the two way slabs. This predicts the deflection of interior panel accurately. But it could not predict the deflection of edge panel accurately.

Boyce [2002] presented a case study about a flat slab floor building, which was a subject of court action. The paper discussed the legal proceedings, deflection issues, the defects and rectification work carried out. The deflections at mid panel exceeded 80mm and it was highlighted that the deflection limits set by the concrete code of the time were not capable of being related to the actual deflections in the finished structure. Also it was highlighted that the effective moment of inertia based on the Branson formula underestimated the deflection in service. This paper highlights the real situation faced by the occupants of the building as well as the designers.

Beeby [2002] conducted experiments on 44 prisms subjected to pure tension and 8 large full-scale slabs subjected to bending. Based on the results they concluded that the tension stiffening decayed within less than 20 days from loading. This results shows that the early reduction in tension stiffening will increase the deflection at the early age at loading of concrete.

Gilbert and Guo [2005] conducted experiments on seven continuous flat slab specimens for long-term deflection for periods up to 750 days. The measured long-term deflection was found to be many times the initial short-term deflection. This was due to the loss of stiffness associated with time-dependent cracking under the combined influences of transverse load and drying shrinkage. The data given in this paper are useful for analytical study of long term deflection. This is used for the comparison of the results with the proposed approach in this research.

Scott and Beeby [2005] conducted laboratory tests on beams and slabs to investigate tension-stiffening effects. Loads were sustained for periods upto 4 months. The results indicated that tension stiffening decayed much more rapidly by 20 days or sooner after loading, it had reduced to its long-term value. This shows that the deflection at the early age of concrete is more. They highlighted that the long term creep strain is less at a later date.

Das [2004] reviewed the design approach for RC slabs given in IS 456-2000 and presented a method for direct design of singly reinforced concrete slabs, to simultaneously satisfy the condition of bending and the serviceability. Furnished design charts for the design engineer to apply this for the adjustment of steel reinforcement and depth. Also charts are provided to find out effective depth when the area of steel is adequate for deflection criteria.

Nayak and Menon [2004] conducted test on six one-way slab specimens and proposed an improved procedure for estimating the short-term deflection of RC slabs. Compared the codes IS 456-2000, BS 8110-1997, ACI 318-2002 and Euro code 2-1996 for the existing procedures for the estimation of cracking moment, moment-curvature and load- deflection behaviour. They highlighted the disparities in the prediction of deflection among the codes

Manaseer and Lam [2005] evaluated four shrinkage and creep prediction models namely the ACI 209 model, the CEB 90 model, the B3 model, and the GL 2000 model. The shrinkage and creep values determined by the models were compared against the RILEM experimental data bank. Five statistical methods were used to determine the order of performance of each model. Based on the study they concluded that the B3 and the GL 2000 are best performing models for shrinkage strain prediction while the CEB 90, B3 and GL 2000 models perform best for creep prediction.

2.3 PUNCHING SHEAR

The significant achievement in this flat slab with special type of reinforcement called mushroom head were introduced by C.A.P. Turner (Gasparini,[2002]) in 1905. This reinforcement extended upto half the width of span along with four way reinforcement. After this lot of developments have taken place in this area.

Pilakoutas and Li [2003] carried out on 1:1.5-scaled models of full-scale slabs. They tested the slabs with a new system of shear reinforcement called shear band. They found that the slabs with shear band exhibited ductile behavior. This system needs the flexural reinforcement at both top and bottom to anchor the shear band.

Oliveira [2000] conducted experiments on 1.8 m x 1.8 m flat plate of thickness 130 mm. It was observed that with the use of shear stirrups the failure zone was away from the stirrups. This shows that providing shear stirrups showed less brittle failure. But the brittle failure was not completely eliminated. Also the failure zone beyond the shear stirrups shows that providing the stirrups up to certain perimeter is not sufficient.

Broms [2000] did experiment on flat plates to eliminate punching shear failure. The test was conducted on seven specimens all of which had the same dimensions and approximately the flexural capacity, but with different reinforcement arrangements. The remaining slabs exhibited a very ductile behavior similar to that of ordinary reinforced concrete slabs supported by beams or walls. But the thickness of slab was 180 mm for a span of 2.6 m.

Gomes and Regan [1999] proposed a model for analyzing the punching resistance of reinforced concrete flat slabs with shear reinforcement for concentric loading. The most important conclusion from the experimental investigation was the punching failure load was doubled with the use of suitable shear reinforcement. But the failure pattern was punching shear only.

Rationalisation of flat slab reinforcement (Best practice guide [2000]) was to reduce the overall cost of reinforced concrete flat slab by way of rationalizing the reinforcements. They evaluated the time and cost benefit of the various methods of reinforcing flat slabs at the European Concrete Building Project at Cardington. Different flexural reinforcements and shear stirrups used in the project were analyzed with regard to time and cost and they recommend they elastically designed, rationalized loose bar flexural reinforcement should be used for small buildings and for large buildings two-way mats should be used. For shear reinforcement proprietary shear systems such as stud rails and shear ladders are to be used. These systems need skilled labour for fixing them in position.

Prefabricated punching shear reinforcement for reinforced concrete flat slabs (Best practice guide [2000]) this guide provides information about the different types of proprietary systems for punching shear reinforcement to be used for flat plates. These systems are three to ten times quicker to fix than traditional loose links. In this guide they describe about the stud rails, ACI shear stirrups, Shear ladders, Shear hoops and structural steel shear heads. The structural steel shear heads are heavy. Since these are prefabricated systems the conveyance cost is more. To fix them in the slab needs skilled labour. For certain systems the depth of slab has to be increased.

Vollum Tay [2003] reviewed the limited test data available for ACI shear stirrups used in Cardington and showed that the EC2 guidelines are conservative. They suggested an improved method for calculating the maximum shear strength for the slabs with ACI shear stirrups and a series of 6 testes were conducted to investigate the strength of slabs.

Guan and Loo [2003] used a nonlinear-layered finite element method to find out the effectiveness of stud shear reinforcement in the corner column-slab connection in flat plates. The accuracy and effectiveness of the method was verified by two case studies comprising a total number of 14 slab specimens.

El-Ghandour et al. [2003] made experiments on fibre reinforced polymer reinforced concrete flat plates with and without carbon fiber reinforced polymer shear reinforcement to investigate the punching shear strength of flat plates. They proposed a model that predicts the punching shear strength accurately without shear reinforcement and for slabs with shear reinforcement they reduced the concrete shear resistance and limited the strain in steel as 0.0045. Also comparison with ACI 318-95, ACI 440-98 and BS 8110 codes were made.

Binici and Bayrak [2003] presented a strengthening technique for increasing the punching shear resistance in reinforced concrete flat plates using carbon fiber reinforced polymers in the form of strips as shear reinforcement in the vertical direction around the concentrated load area. Four strengthened specimens and two reference specimens were tested and the results showed that the failure surface can be shifted away from the column in the specimens. The pre- and post punching behavior of the specimens with the highest increase in load carrying capacity was better than that of the reference specimens. The punching shear occurred outside the column.

Chen and Li [2005], conducted experiments on 18 specimens to study the punching shear strength and failure behaviour of reinforced concrete slabs strengthened with glass fiber-reinforced polymer (GFRP) laminates. The parameters studied were the concrete compressive strength, steel reinforcement ratio, and the number of GFRP laminate layers. They observed that the punching shear capacity was markedly increased when the slab column connection was strengthened by the GFRP laminates. The capacity was increased but the failure was due to punching shear.

Desayi and Seshadri [1996] conducted experiments on corner column. The punching shear is critical at the corner column connections. The edge and corner column connections are vulnerable to punching shear.

Deodhar and Dubey [2004] conducted experiments on two way slabs with hidden beam provided at the mid-span. The width of the beam was 230 mm. The experiments were conducted without applying any theoretical modelling. They concluded that the same can be used in flat slabs also. But the width provided by them will not be sufficient for flat slab. Also the reinforcements were fixed without any theory.

2.4 OTHER ASPECTS

Salim and Sebastian [2003] made an experimental study on four slabs with one control specimen. The application of plasticity theory to predict punching shear failure loads were presented and a method was proposed by taking into account the effect of compressive membrane action.

Balendra et al. [1981] proposed an iterative analysis for building frames subjected to lateral load analysis. Walker and Regan [1987] made experiments on 11 slabs to investigate the behavior of corner bays of flat plate floors. The performance of each specimen was described and related to the details of the slab-column joints. Comparisons were made between the moment distributions measured in these and other tests and those derived from the equivalent frame analyses of the ACI and British Codes.

The softening and stiffening of concrete flexural members have become the main area of research in the concrete flexural members. Some of the significant researches are given here. Mendis [2000] studied the softening of Reinforced Concrete Structures. It was insisted that the softening must be considered for rational analysis. Since the softening is the material instability which causes the decrease in moment carrying capacity. This effect is not considered in the existing provisions given in the codes for the slabs. Neelamegam et al. [2005] studied the effect of Mineral Admixtures and Mixture composition on early age stiffening characteristics of concrete.

Pendyala et al. [1996] studied the full-range behaviour of high-strength concrete flexural members; comparison of ductility parameters of high and normal-strength concrete members. Regan and Hussaini [1993] studied the behaviour of high strength concrete slabs. Hussaini and Ramdane [1992] studied the properties and creep characteristics of high strength concrete and by using the high strength concrete in slabs [1993], they studied the behaviour.

Albrecht [2002] compared the various codes for the design of flat plate for punching. The provisions given in the various codes for punching were compared. In this the punching shear capacity of concrete, the punching shear resistance with shear reinforcement and the relevant detailing of the reinforcement were compared.

Yankelevsky and Leibowitz [1999] presented a new model based on rigid post-fractured behavior of flat plates. This paper has made analysis based on the theory of plasticity. It presents a new model, based on rigid post-fractured behavior, utilizing the post fracture properties of concrete at the rough crack interfaces that are developed.

Nagpal et al. [1993] proposed an efficient procedure for free vibration characteristics of framed tube buildings and studied the negative shear lag in framed tube buildings using the equivalent frame.

Megally and Ghali [2000] studied theoretically on punching of flat plates due to column moment transfer. They referred ACI 318-95 in which a fraction of the unbalanced moment is transferred between the slab and the column by flexure; the remaining moment fraction is to be transferred by shear.

Ibrahim and Shehata [1990] recommended a simpler rational method for designing RC Slabs (Flat Plate) to resist punching. This paper presents simple model to estimate the punching strength of the slab-column. The proposed method has shown to correlate very well with test results and has the great advantage of being simple to be used by designers. For this reason, this method is considered to be a good alternative for codes of practice recommendations that, although are safe in their present formulation, lack a rational base and do not correlate well with test results in terms of the effect of the different parameters on punching failure.

Natarajan [1998] studied the defects and the rehabilitation in RCC structures. Jaafar [2002] studied the strength and durability characteristics of high strength autoclaved stone dust concrete. Waleed [2005], conducted experiments on repair and structural performance of initially cracked reinforced concrete slabs.

Pavic and Reynolds [2003] conducted modal test on full-scale 15m x 15m floor made of in-situ high-strength concrete of 115 MPa. The concrete floor was tested and updated for both cracked and uncracked states, and also they did FE model updating, which provided a valuable insight in the change of floor stiffness between the uncracked and cracked states. It was found that even a heavily cracked HSC floor remains linear and does not have significantly increased damping under low-level excitation. Also, bending of in situ cast HSC columns provided considerable stiffness to the floor even when heavily and visibly cracked. Finally, FE model updating confirmed that dynamic modulus of elasticity of the as-built HSC was about 47 GPa and, therefore, considerably increased compared with normal strength concrete.

Murray et al. [2003] proposed a modification to the ACI 318-02 Equivalent Frame Analysis of Reinforced concrete Flat Plate exterior panels. In this they compared two code methods such as ACI 318 and BS 8110 [1997]. Experiments were conducted on 1/3-scale models of Flat Plate exterior panels. Strains were measured in reinforcing bars and were used to calculate curvature, bending moments to assess the accuracy of the method.

Gilbert et al. [2000] conducted experimental study and compared the results with the equivalent frame analysis methods. The distribution of bending moment between column and middle strips were studied. They observed that at the edge columns the distribution of moment was closer to the existing provision given in BS 8110. This suggestion is used in this study for the edge column.

Natarajan [1997] proposed an object oriented programming for structural design. Kant [1982], proposed a numerical analysis of thick plates by using higher order differential equations. Mehmood [1990] studied the aerodynamic interference in tall buildings. Sateesh Kumar and Deb [2002] used fuzzy logic to assess the seismic damage of reinforced concrete buildings.

Park and Kim [1999] made a numerical study using nonlinear finite-element analysis was made to investigate the behavior of reinforced –concrete flat plates subject to combined in-plane compressive and out-of-plane floor loads. A computer program for material and geometric nonlinearities is developed. This paper provides a rational design rules for the moment magnifier method.

Hwang and Moehle [2000] made experiment on 0.4-scale model of nine-panel model was tested. A portion of slab was designed for gravity and wind load as per ACI 318-83. The remaining was designed for moment redistribution. Gravity load tests provide data on structural responses at the service load and lateral load tests provide data on behavior for loadings ranging from the service load level to the ultimate load level.

Theodorakopoulos and Swamy [2002] proposed a simple analytical model based on the physical behavior of the slab-column connections under load, which was applicable to both lightweight and normal weight concrete. The model assumed that punching was a form of combined shearing and splitting, occurring without concrete crushing, but under complex three-dimensional stresses. Failure was assumed to occur when the tensile splitting strength of the concrete was exceeded. The proposed theory was applied to predict the ultimate punching shear strength of 60 slab–column connections reported in literature, and designed to fail in shear, involving a large number of variables, such as type of concrete, concrete strength, tension steel ratio, compression reinforcement and loaded area which showed very good agreement between the predicted and experimental values.

Choi et al. [2003] proposed an improved moment magnifier method based on nonlinear finite element analysis, which is applicable to the long-term behavior and the load carrying capacities of flat plates. A computer program considering creep and shrinkage as well as geometrical and material nonlinearity was developed.

2.5 FLAT SLAB

Sherif and Dilger [2000] tested two full-scale 5m continuous flat slabs to failure after it had been repaired twice. Square drop panels were provided and shear failure was occurred at the edge and interior column-slab connections. The test results were compared with the moments predicted by equivalent frame method (EFM) and linear finite element analysis (FEM).

Paultre and Moisan [2002] made an analytical study on flat slab with continuous drop panels between column lines. This makes long spans and possible in addition to increase in punching shear resistance around supports. The drop panels were provided along the x-axis. The width of drop panels were less than or equal to the column strip and of variable thickness. Providing drop panel for width equal to column strip will be uneconomical. Also this will not be acceptable in places where strict restriction in height is required.

Megally and Ghali [2000] proposed design requirements for earthquake-resistant slab-column connections. It includes the value of the unbalanced moment to be used in punching shear design. Also it shows that the use of shear reinforcement particularly shear studs with mechanical anchors significantly enhances the ductility of slab-column connections under reversed cycle of loading.

Nanni and Mettemeyer [2001] have used diagnostic load testing and finite element modeling of a two-way post tensioned flat slab to verify load carrying capacity of structure. Combination of concentrated (hydraulic jacks) and gravity loads (pallets loaded with rolls of paper or sand bags) were used. FEM was used to determine the bending moments by the load tests. They concluded that the diagnostic load testing could be used for testing the existing structures.

Furst and Marti [1997] compared the Robert Maillart's dimensioning procedure for orthogonally reinforced flat slabs with the elastic plate theory and limit analysis. While considering the elastic analysis Maillart underestimated the flexural moments, whereas the design had reasonable safety margin when compared to limit analysis.

The lateral load analysis for seismic forces are the other main areas of research in which the significant works are carried out by Durrani et al. [1994], Farhey et al. [1993], Hosahalli and Aktan [1994] and Mosalam and Naito [2002].

The fire resistance of concrete buildings were studied by Huang et al.[2003]. They developed the theoretical model using the membrane action.

2.6 SUMMARY

From the literatures it is observed that some of the buildings had excessive deflection. The empirical equations developed by some of the researchers considered the material properties existing at that period. These equations need improvements considering the present conditions. Also the equation should be simple to be used by the designer. To avoid the punching shear failure many types of reinforcements were used and studied for their contribution in changing the mode of failure. These were summarized in this chapter. Also the methods used for modeling of the slab in punching shear were given in this chapter. The main objective of this research is to have simple structural model for the slab with the concealed beam to improve the stiffness. This structural modeling and the theories used are explained in the next chapter.

CHAPTER 3

STRUCTURAL MODELLING

3.1 GENERAL

The main aim of this study was to fix the depth of slab to meet the serviceability and strength requirement. To meet the serviceability requirement an empirical relation that would predict the total deflection of beamless slabs accurately is required. Hence the existing provisions in the codes for long term effects including shrinkage and creep are studied and presented in this chapter. Also this chapter describes the derivation of the equation for flat slab using the conjugate beam method. The theories used for this research and the theoretical modelling of concealed beam are furnished at the end of this chapter.

3.2 DEPTH REQUIREMENT BASED ON CONTROL OF DEFLECTION

The principal dimension of the member deciding its stiffness is the depth because moment of inertia of the section forming flexural rigidity EI varies with cube of the depth. Codes set the limit state of serviceability can be attended to by imposing suitable maximum allowable deflections. When deflection becomes a criterion governing the cross-section, the calculations for deflection prove to be quite lengthy and tedious that is not suitable for practical design. The simpler method suggested in the codes is to provide thickness for the members in the form of span to depth ratio.

3.2.1 Provisions in Codes

Table 3.1 furnish the span to depth ratio to be followed for beamless slabs without edge beams as per the four codes.

Table 3.1 Span to depth ratio as per codes

S.No.	ACI 318-2002		BS: 8110-1997	EC: 2-1997	IS: 456-2000
	Interior Panels	Exterior Panels			
1.	$l_n/33$	$l_n/30$	26	30	40

The code IS: 456-2000 gives the value for two-way slab of span up to 3.5m and load class up to $3KN/m^2$. In all this the limit of span up to which this slab can be used is not given. Also the concrete grade, the load class are not considered. The beam formula for short term, shrinkage and creep deflection, as per the codes is furnished in Table 3.2. The modelling of the slab as per this method and the mathematical expression used with the above modification are explained below.

3.2.2 Short-term Deflection

Short-term deflection is due to initial elastic deformation of the member due to load and permanent imposed load under service conditions. At working loads the reinforced concrete member behaves elastically and hence the short-term deflection can be calculated by using the elastic theory as the basis (Table. 3.2).

$$\delta = \frac{wl^4}{384 E_c I} \tag{3.1}$$

where δ - the deflection

w - the uniformly distributed load

l - the span

E_c - the modulus of elasticity of concrete

I - moment of inertia

3.2.3 Shrinkage Deflection

Shrinkage is defined as the reduction in volume of an unloaded concrete at constant temperature. Its primary cause is the loss of water during a drying process. The magnitude of this deformation is described by the shrinkage strain. If the shrinkage is

Table 3.2 Basic equations as per codes for deflection computation

S. No.	Description	Formulae			
		ACI: 318-2002	BS: 8110-1997 Curvature	EC: 2-1992 Curvature	IS: 456-2000
1.	Short-term	$\delta = \frac{wl^4}{384E_cI}$	$\delta = \frac{wl^4}{384E_cI}$	$\delta = \frac{wl^4}{384E_cI}$	$\delta = \frac{wl^4}{384E_cI}$
2.	Shrinkage	Multiplier approach	$\frac{\varepsilon_{cs} * \alpha_e * \delta_s}{I_x}$	$\xi(1/r_{cs2}) + (1-\xi)(1/r_{cs1})$	$\delta_{cs} = k_3 * \psi_{cs} * l^2$
3.	Creep	Multiplier approach	$\frac{M_{r2}}{E_{eff} * I_x}$	$\xi(1/r_2) + (1-\xi)(1/r_1)$	$\delta_{i,cc(perm)} = \frac{wl^4}{384E_{ce}I_{eff}}$
4.	Total	$\delta_{tot} = 3 * \delta_{DL} + \delta_{LL}$	1+2+3	1+2+3	$\delta_T = \delta_{i,cc(perm)} + \delta_{cs} + \delta_{i(temp)}$

restrained in a non-uniform manner over the depth of a section then the section will tend to warp, leading to deflection. The major factor causing such restraint in the flexural member is the reinforcement. In a singly reinforced section, therefore, the tension face is restrained while the compression face is not restrained. This will lead to a shortening of the compressive face relative to the tension face and hence a deflection will be induced. The suggestion given in the four codes to calculate deflection due to shrinkage are given below.

ACI 318 - 2002

The multiplier approach (Branson approach) is used for the total deflection including shrinkage and creep.

BS: 8110 - 1997

Deflection is calculated by using the moment curvature relationship. In the absence of experimental data the shrinkage strain for inside atmosphere is adopted as given in the code as 300×10^{-6}. The empirical equations are furnished in the Chapter 4 (section 4.3).

EC: 2-2002

The code gives a separate formula to calculate the shrinkage strain. This code suggests the curvature method and the empirical relations used are explained in Chapter 4 (section 4.4).

IS: 456-2000

Deflection is calculated by using the moment curvature relationship. This method depends on support condition and area of steel provided. In the absence of experimental data the shrinkage strain for inside atmosphere is adopted as given in the code as 300×10^{-6}. The empirical relations used are explained in section 4.5.

3.2.4 Creep Deflection

Creep is a plastic deformation under sustained load, and its effect is to increase the strain in concrete at constant stress. With the increase in strain, but stress remaining constant, there is a decrease in the modulus of elasticity of concrete. The modified or reduced modulus of elasticity of concrete is known as effective modulus of concrete. The effect of creep is thus reduction in modulus of elasticity of concrete, increase in depth of neutral axis and increase in curvature.

The effect of creep on stiffness can be understood by noting its effect on each of the two quantities E and I constituting the stiffness. But simultaneous increase in depth of neutral axis increases the moment of inertia I_{eff} of the section on occurrence of creep. However the reduction in E is greater than increase in I, with the result that final value of the stiffness $E_{ce}I_{eff2}$ is less than its initial value E_cI_{eff1} and thus, there is increase in curvature and, hence, the deflection due to creep. Theoretically, it is very difficult to determine deflection due to creep because it requires creep-time history of the R.C. member. The method followed to predict the creep deflection as per the four codes are given below.

ACI: 318-2002

The final deflection is calculated using the formula (multiplier approach) below

$$\delta_T = 3 \times \delta_D + \delta_l \tag{3.2}$$

where δ_D - deflection due to sustained deflection

δ_l - deflection due to varying part of live load

BS: 8110 -1997

Assuming the humidity as 45% and indoor atmosphere the creep coefficient is obtained from the graph provided in the part 2 of this code. Creep coefficient for 28 days is 3.1 and for one year 1.6. The empirical relations used are explained in section 4.3.

EC: 2 - 2002

The code gives formula to calculate creep coefficient. By using the formula and based on the same assumption made for other codes, the creep coefficient is calculated as 1.31 for 28 days. The empirical relations used are explained in section 4.4.

IS: 456-2000

In the absence of experimental data the code gives the creep coefficient as 1.6 for 28 days and 1.1 for one-year age at loading. The empirical relations used are explained in section 4.5.

3.3 MODELLING OF BEAMLESS FLOORS

The aim of this study is to give a simplified approach to find the total deflection of beamless floors by considering the parameters, which affect the total deflection. The formulas available in the codes for the determination of total deflection including long-term effects are based on beam behaviour, which are used extensively by the designers.

Since the flat plates and slabs are two-way slabs, the deflection formulas as given in the codes for beams can not be used as such. These formulas have to be used suitably by incorporating the two-way behaviour in the empirical relations. To have little change in the available formulas, it was planned to model the slab using the Equivalent Frame method (Varghese [2002]). This method divides the slab into two strips. These strips are called column strip and middle strip. The strips are also called wide beams. In this the width of the column strips are equal to half of the shorter span and for middle strip the remaining span becomes the width. This width can be substituted in place of the width of the beam in the beam formula.

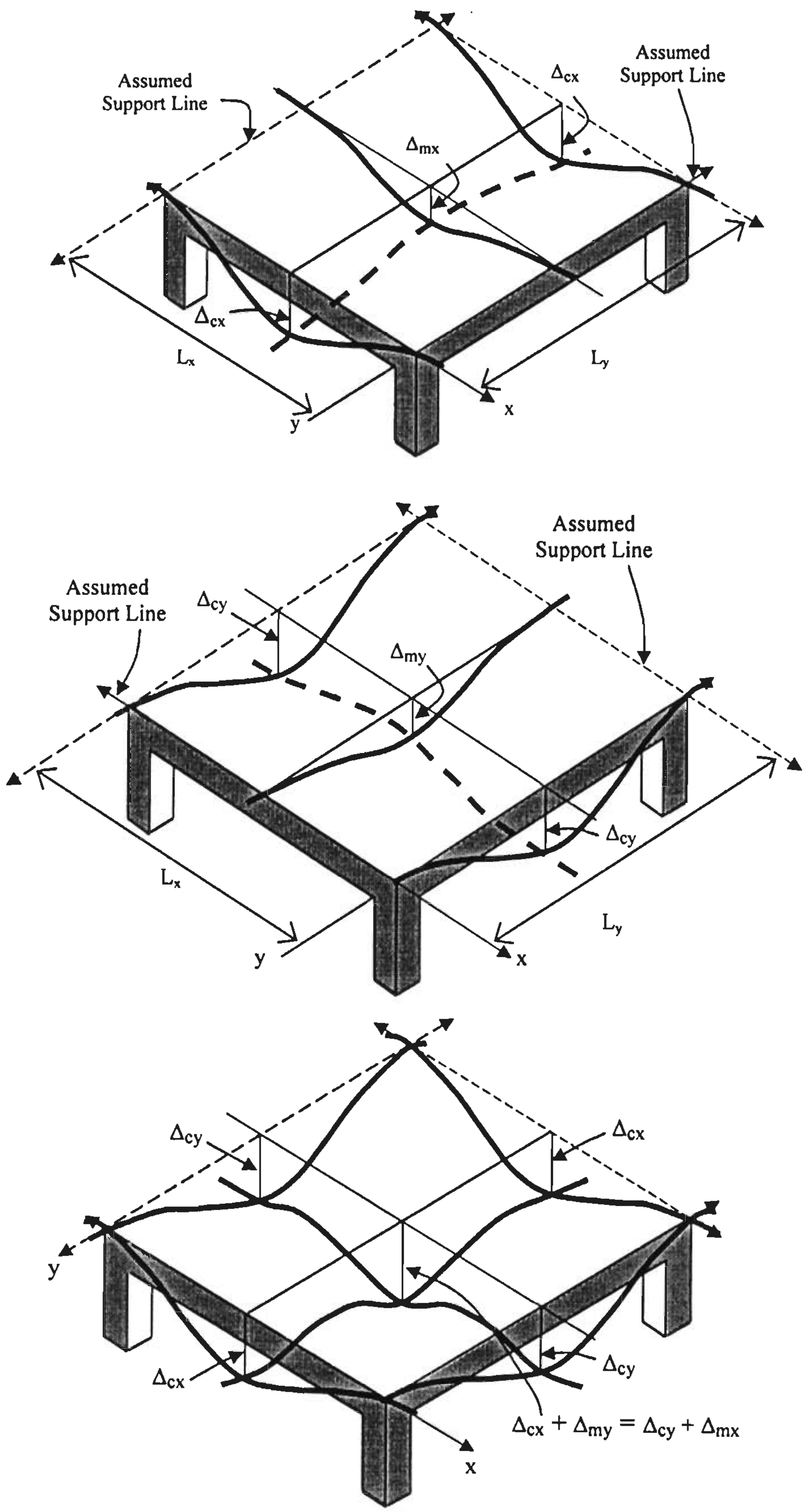


Fig. 3.1 Structural modelling of the slab

3.3.1 Equivalent Frame Method

Since the column strip and middle strip are considered as wide beams, the available beam formula for creep and shrinkage deflection in the codes BS: 8110-1997, IS: 456-2000 and EC: 2-2002 are used by substituting the width of the relevant column or middle strip in place of width of beam (Table 3.2), by using the following structural modelling.

Determination of total deflection of flat plates and flat slabs is obtained by adding the short-term deflection, deflection due to creep and shrinkage. The basic equations which are explained in the previous clauses are used by applying the following beam model.

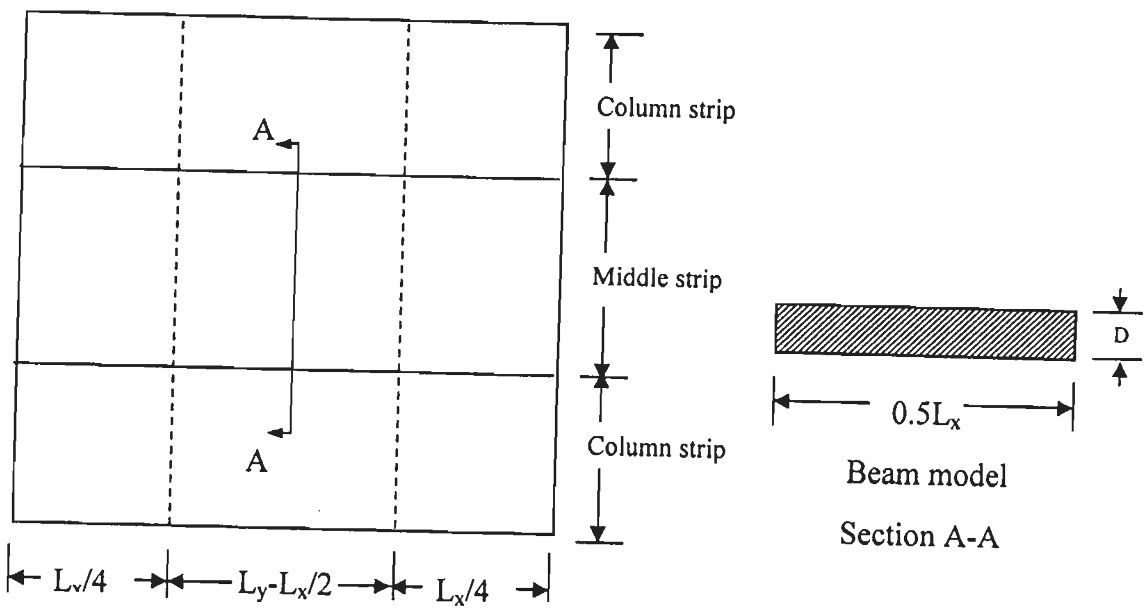


Fig. 3.2 Structural modelling of the flat plate

Basic mid-span deflection of the interior panel [Branson, 1976], assumed as fixed at both ends given by

$$\delta = \frac{wl^4}{384 E_c I} \tag{3.1}$$

This has to be proportioned to separate deflection of the column strip and middle strip, such that;

The midspan deflection of Flat plate (Eqn.3.3) is determined by the formula

$$(\delta) = \delta_{cx} + \delta_{my} = \delta_{mx} + \delta_{cy} \tag{3.3}$$

where, δ_{cx}, δ_{cy} = deflection of column strip along x and y direction respectively.

δ_{mx}, δ_{my} = deflection of middle strip along x and y direction respectively.

The column strip has width equal to $L_x/2$ along both directions. The middle strip has $L_x/2$ along X direction and L_y- $L_x/2$ along Y direction. In Eq. (3.1), the 'w' is the uniformly distributed panel load. E_c is the modulus of elasticity of concrete and I is the moment of inertia of the column strip or middle strip.

3.3.2 Flat Slab

The cross section of a flat slab is not uniform and it changes at the slab column connection. Since the cross section is varying along the span, the conjugate beam method is suitable to determine the deflection of flat slab. Hence it was decided to use this method for finding the deflection of flat slab.

3.3.2.1 Conjugate beam method

The concept of the conjugate beam provides a simple method for the determination of slope and deflection at any point of a beam. The expressions for shear force, bending moment, slope and deflection in a beam can be derived by successive integration of the expression for the intensity of load. If the load acting on a beam is replaced by the M/EI diagram, this beam is known as conjugate beam. The shear force at any point in the conjugate beam is equal to the slope at the same point in the actual beam. Similarly, the bending moment at any point in the conjugate beam is equal to the deflection at the same point in the actual beam.

As the slope and deflection in the actual beam are analogous to the shear force and bending moment in the conjugate beam, it is necessary to select the support conditions for the conjugate beam so as to maintain the analogy between the two beams. The slope and deflection at a fixed support is zero. Hence there should be no shear force and bending moment at that point in the conjugate beam. It follows that a fixed end in the actual beam should be replaced by a free end, in the conjugate beam. Similarly a free end in the actual beam should be replaced by a fixed end in the conjugate beam. A simple end support continues to remain so in the conjugate beam.

The conjugate beam has the advantage of having a simple and unambiguous sign convention. If the bending moment in the actual beam is positive (sagging), the corresponding elastic load M/EI in the conjugate beam should be considered to be acting downward. On the other hand, if the shear force and bending moment in the conjugate beam are found to be negative, the analogous rotation and deflection in the actual beam are counter clockwise and upward respectively. By using the above concepts, the equations for predicting the final deflection of flat slab is derived and the important steps are given below.

3.3.2.2 Flat slab with drop panel

The column strip in this slab has varying cross section as shown in Fig. 3.3. The drop panel thickness is assumed as 1.25 times the total depth of slab. The width is assumed as $L_x/3$. The dead load at this location includes the load of the drop panel also. Since the cross section is varying the moment of inertia is different at the column strip and at the middle strip.

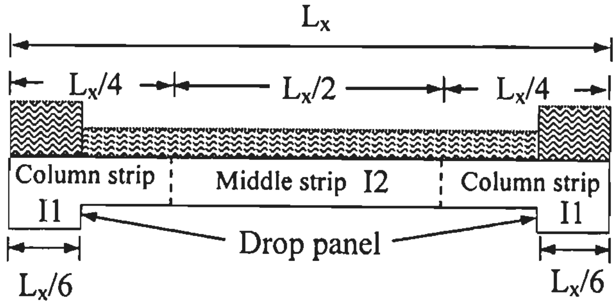
For the column strip the deflection equations are derived using the conjugate beam method. Figure 3.3 shows the varying cross section of the flat slab along the centre of column strip. The equations derived using the conjugate beam methods are for the critical locations are given below. The details are not furnished here.

Reaction at A and B is calculated by the following equation

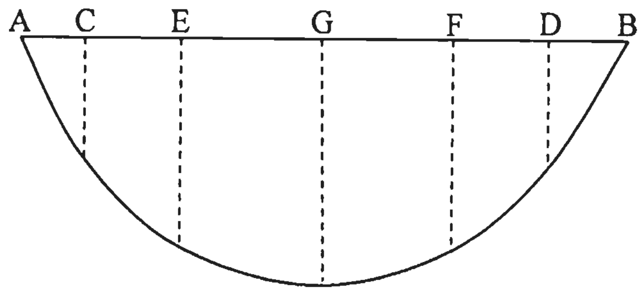
$$R = \frac{L}{6}(4.5 + 3.25w) \tag{3.4}$$

where w - uniformly distributed load due to self weight of slab

L - span of the column strip ($L_x/2$)



a. Details of column strip cross section along the column centreline



b. M/EI diagram

Fig. 3.3 Structural modelling of flat slab with drop panel

At C

$$= \frac{L^2}{36}(3.75 + 2.625w) \tag{3.5}$$

At E

$$= \frac{L^2}{12}(1.6875 + 1.167w) \tag{3.6}$$

At G

$$= \frac{L^2}{12}(2.25 + 1.541w) \tag{3.7}$$

The reaction at the conjugate beam is

$$R_{A'} = \frac{L^3}{EI_1} \times \frac{1}{432}(6.6875 + 4.667w) + \frac{L^3}{EI_2} \times \frac{1}{48}(2.0625 + 1.417w) \tag{3.8}$$

The final expression for the mid panel deflections for dead load and live load for the column strip are as below;

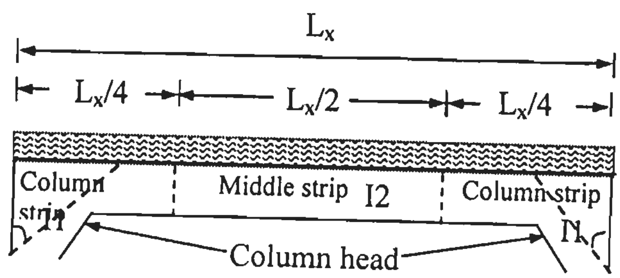
$$\delta_T = \frac{L^4}{864EI_1}(5.6745 + 3.974w) + \frac{L^4}{96EI_2}(1.5703 + 1.0785w) \tag{3.9}$$

The final expression for middle strip is as below:

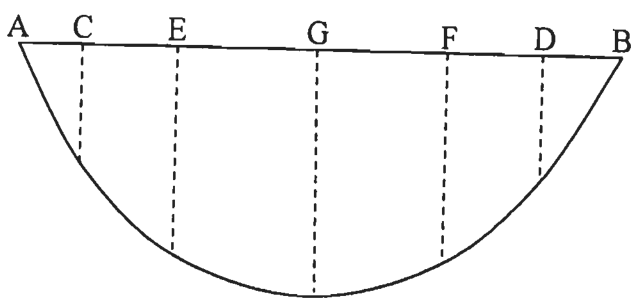
$$\delta_T = \frac{LDF_m \times 0.013 \times wl^4}{EI_m} \tag{3.10}$$

3.3.2.3 Flat slab with column head

The self weight of the column head is not included in the self weight of the slab. Hence the panel load is same along the column strip. But the moment of inertia is different at the slab column junction and at middle strip. The Fig. 3.4 shows the modelling of this slab.



a. Details of column strip cross section along the column centreline



b. M/EI diagram

Fig. 3.4 Structural modelling of flat slab with column head

Reaction at A and B is calculated by the following equation

$$R = \frac{L}{2}(1.5 + w) \tag{3.11}$$

At C

$$= \frac{9L^2}{200}(1.5 + w) \quad (3.12)$$

At E

$$= \frac{3L^2}{32}(1.5 + w) \quad (3.13)$$

At G

$$= \frac{L^2}{8}(1.5 + w) \quad (3.14)$$

$$R_{A'} = \frac{L^3}{EI_1} \times \frac{28.43}{800}(1.5 + w) + \frac{L^3}{EI_2} \times \frac{13}{32 \times 12}(1.5 + w) \quad (3.15)$$

The final expression for the mid panel deflections for dead load and live load for the column strip are as below;

$$\delta_T = \frac{0.0055 \times L^4}{EI_1}(1.5 + w) + \frac{0.013 \times L^4}{EI_2}(1.5 + w) \quad (3.16)$$

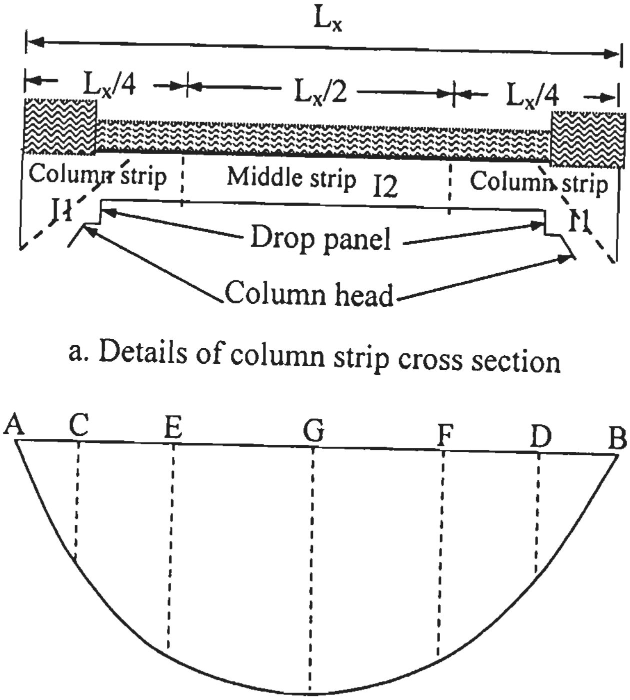
The final expression for mid panel deflection of column strip due to dead load and live load are as below:

$$\delta_{cd} = \frac{(1.5 + w) \times L^4}{E}(\frac{0.0055}{I_1} + \frac{0.013}{I_2}) \quad (3.17)$$

The expression for middle strip is the same for all the types.

3.3.2.4 Flat slab with drop panel and column head

The drop panel thickness is assumed as 1.25 times the total depth of slab. The width is assumed as $L_x/3$. Since the column head will not increase the dead load the same expression derived for deflection of flat slab with drop panel can be used for this slab also. But the moment of inertia at the slab column junction will be different because it considers both drop panel and column head.



a. Details of column strip cross section

b. M/EI diagram

Fig. 3.5 Structural modelling of flat slab with drop panel & column head

$$\delta_T = \frac{L^4}{864EI_1}(5.6745 + 3.974w) + \frac{L^4}{96EI_2}(1.5703 + 1.0785w) \tag{3.9}$$

The final expression for middle strip is as below:

$$\delta_T = \frac{LDF_m \times 0.013 \times wl^4}{EI_m} \tag{3.10}$$

3.4 THEORIES USED

To model the concealed beam and to use the empirical equations suggested in the codes the following theories are used.

3.4.1 Calculation of the Neutral axis

To find the moment of inertia the neutral axis is to be found. The equilibrium condition is used to find the neutral axis. The normal reinforced concrete theory is used.

$$\frac{bx^2}{2} + (m - 1)A_{sc}(x - d')^2 = m\ A_{st}(d - x)^2 \tag{3.11}$$

In the above equation the width denoted by 'b' has to be substituted by the width of column strip and middle strip separately.

Solving the quadratic equation will give the neutral axis value. This is used for the uncracked stage. The dominative effect of movement of neutral axis is downward due to creep (Branson, [1978]). Here it is assumed that the neutral axis depth will move downwards by 0.2 times compared to the original position in (Pillai and Menon, [2002]) uncracked stage. Hence for finding the neutral axis depth in the cracked stage, it was assumed that the position was 1.2 times the original position in the uncracked stage.

Width of column strip is

$$b_{cx} = b_{cy} = \frac{L_x}{2}$$

Width of middle strip along x-x direction is

$$b_{mx} = \frac{L_x}{2}$$

Width of middle strip along y-y direction is

$$b_{my} = (L_y - \frac{L_x}{2})$$

3.4.2 Calculation of the Moment of Inertia

The gross moment of inertia is determined by using the following relation for uncracked stage considering the concrete as homogeneous material. The column strip is considered as doubly reinforced beam, since longitudinal reinforcement will be provided at both top and bottom face.

1. The following assumptions are made for calculating the moment of inertia: Uncracked Stage
   - Both column strip and middle strip are considered uncracked and transformed area method is used.

2. Cracked stage

- The column strip is considered partially cracked. Hence effective moment of inertia is used.
- The middle strip is considered uncracked and hence the gross moment of inertia is used. .

For uncracked stage (Branson [1978]), the transformed area method is used for modifying the available beam formula. This method transforms the area of steel into equivalent area of concrete and the second moment area is calculated by using the following formula.

$$I_{cgr} = \frac{b_c D^3}{12} + b_c D(\frac{D}{2} - x)^2 + (m-1)A_{st}(d-x)^2 + (m-1)A_{sc}(x-d')^2 \qquad (3.12)$$

For the cracked stage the cracked moment of inertia is to be used. In this stage it is assumed that the concrete in tension side is completely cracked and the effect of that concrete is not considered.

$$I_{ccr} = \frac{b_c x^3}{3} + m_{ce} A_{st}(d-x)^2 + (m_{ce}-1)A_{sc}(x-d')^2 \qquad (3.13)$$

3.4.3 Material Properties

The concrete grade 20, 25 and 30 are considered. For this the young's modulus of concrete is calculated as per the provisions given in each code. Fe 415 grade steel is considered.

Modulus of elasticity of concrete

The modulus of elasticity of concrete is calculated as per the provisions given in the code.

ACI: 318-2002

$E_c = 5375\sqrt{f'_c}$ where $f'c$ is the cylinder strength of concrete.

IS: 456-2000

$E_c = 5000\sqrt{f_{ck}}$ where f_{ck} is the cube strength of concrete.

The nominal values given in the code are used as such for BS: 8110-1997 and EC: 2-2002.

3.5 MODELLING OF CONCEALED BEAM

Concealed beam is an arrangement of reinforcement that is placed within the slab. This beam does not increase the depth of the slab. Since this is embedded within the concrete, this can be called as concealed beam. The main aim of this study is to have a simple analytical model for this beam and to study the strength and serviceability of flat plate and flat slab with this beam. The following clauses explain how the reinforcements are decided to fix the dimensions of concealed beam.

3.5.1 Arrangement of Reinforcement

The concealed beam is provided with equal number of longitudinal reinforcements in both top and bottom faces. These are tied together by means of 8 mm diameter shear stirrups (Fig.3.6). The width of the beam is decided to cover the critical section on either side of column face. The stirrups can be provided for the critical section at d/2 (IS: 456-2000) to 2d (EC: 2-2002) on either side of column face. Many trial studies were conducted to find the appropriate dimension and the details are not furnished here. Finally based on the trial study it was decided to select the width of beam as 1000 mm. Because this width covers the critical section for the maximum size of column to be provided at the ground floor (assumed as 600mm x 600mm). The horizontal distance between the longitudinal reinforcement is fixed as 50 mm as per the minimum spacing suggested by IS: 456- 2000. The floor plan of flat plate with concealed beam (Fig. 3.7) is shown. The minimum spacing is provided between the stirrups.

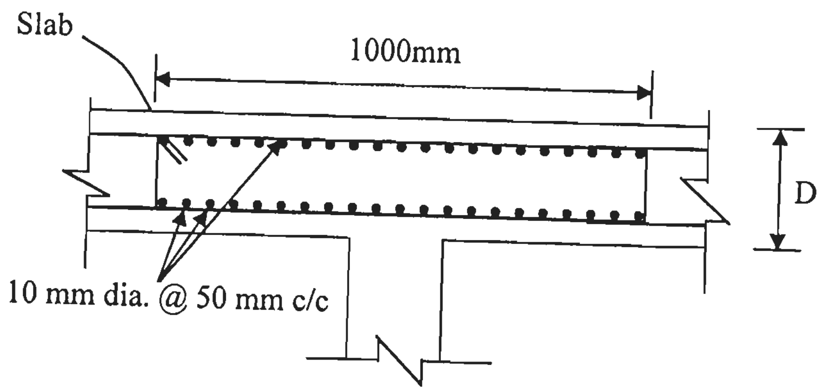
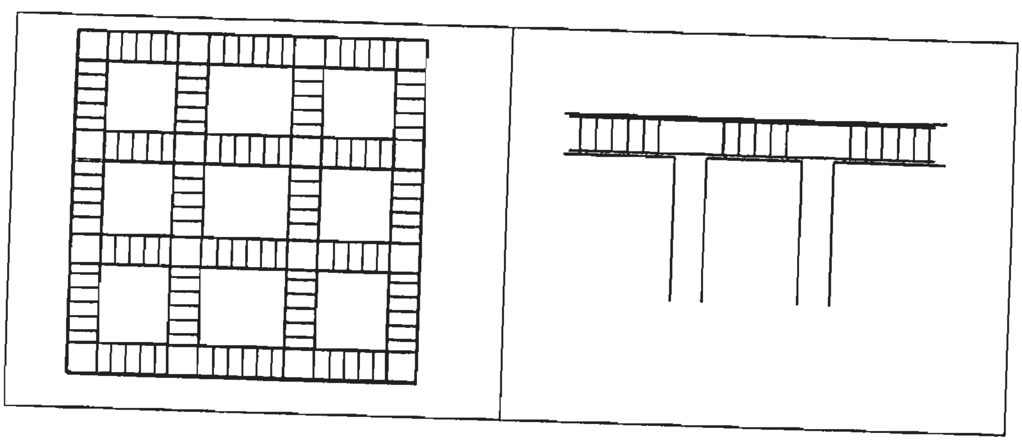


Fig. 3.6 Cross section of concealed beam

The shear stirrups are provided at a spacing of 50mm distance centre to centre.

3.5.2 Proposed Modelling for Flexure

For flexure the transformed section method is used for finding stiffness. In this method the cross section containing the steel and concrete are transformed into a homogeneous section of one material all having the modulus of elasticity of concrete. This requires the replacement of the actual steel area by an equivalent area of concrete.



(a). Floor plan **(b). Longitudinal section**

Fig. 3.7 Floor plan with concealed beam

3.5.3 Proposed Modelling for Torsion

For torsion the concealed beam is modelled as a thin box of equivalent concrete section (Fig. 3.9).

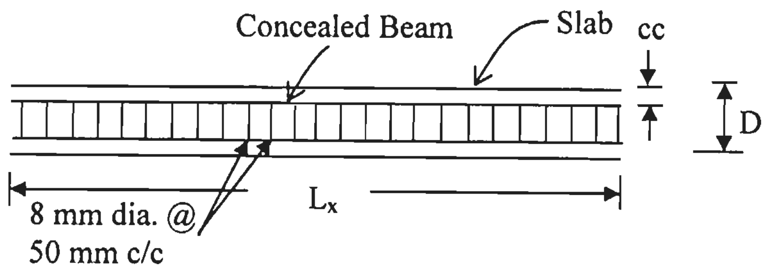


Fig. 3.8 Longitudinal section of the concealed beam

The spacing of stirrups (s) in the concealed beam is assumed as 50 mm. because

$$s = 50 \text{ mm}$$

The diameter (d_s) of the stirrup

$$d_s = 8 \text{ mm}$$

The number of stirrups (n_{sx}) along the x-x direction (Fig.3.8)

$$n_{sx} = \left(\frac{L_y - C1}{s} \right) + 1$$

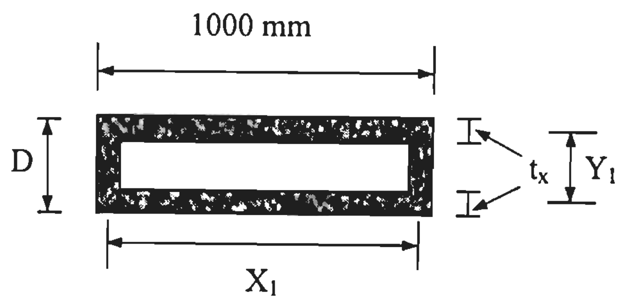


Fig. 3.9 Equivalent thin concrete box for torsion

The length of the stirrups (L_s) to be provided in the x-x direction

$$L_s = L_y$$

Equivalent area of concealed beam (E_a) in concrete

$$E_a = \frac{n_{sx} \times m \times \pi \times d_s^{\ 2}}{4}$$

The thickness (t_x) of the concealed beam (Fig.3.9)

$$t_x = \frac{E_a}{L_y}$$

$$X_1 = 1000 - t_x$$

$$Y_1 = D - t_x$$

The torsional constant (C_b) of the concealed beam (Hsu, 1984)

$$C_b = \left(\frac{2 \times X_1^2 \times Y_1^2 \times D}{X_1 + Y_1} \right)$$

3.6 SUMMARY

This chapter explained the proposed structural modelling of flat plates and flat slabs. The theoretical modelling of the concealed beam was given at the end of this section. By using the above the empirical relations studied with suitable modifications as per the four codes ACI: 318-2002, BS: 8110-1997, EC: 2-2002 and IS: 456-2000 are explained in the next chapter.

CHAPTER 4

STUDIES ON EMPIRICAL EQUATIONS

4.1 GENERAL

The comparative study on the available provisions in the four codes shows that the empirical equations are based on beam behaviour. It was planned to use the available formulas with little modification. For this it was considered that the Equivalent Frame method would give the required modification. This method and the modelling considered for this study were explained in the previous chapter. This chapter presents the empirical equations which were studied with the suitable substitution as per the above modelling.

4.2 AS PER ACI: 318-2002

The total deflection due to combined effect of creep and shrinkage is obtained by using the Branson approach, which adopts the equivalent frame method (Fig. 4.1). A multiplier of 3 is used to find out the total deflection.

The formulas used are as furnished below. The modulus of elasticity of concrete is calculated for normal density concrete. Since cylinder strength is used in all the formulas of ACI, for comparison of codal provisions 20% reduction in cube compressive strength is considered for concrete.

$$E_c = 5375\sqrt{f'_c} \tag{4.1}$$

where f'_c is the cylinder strength of concrete.

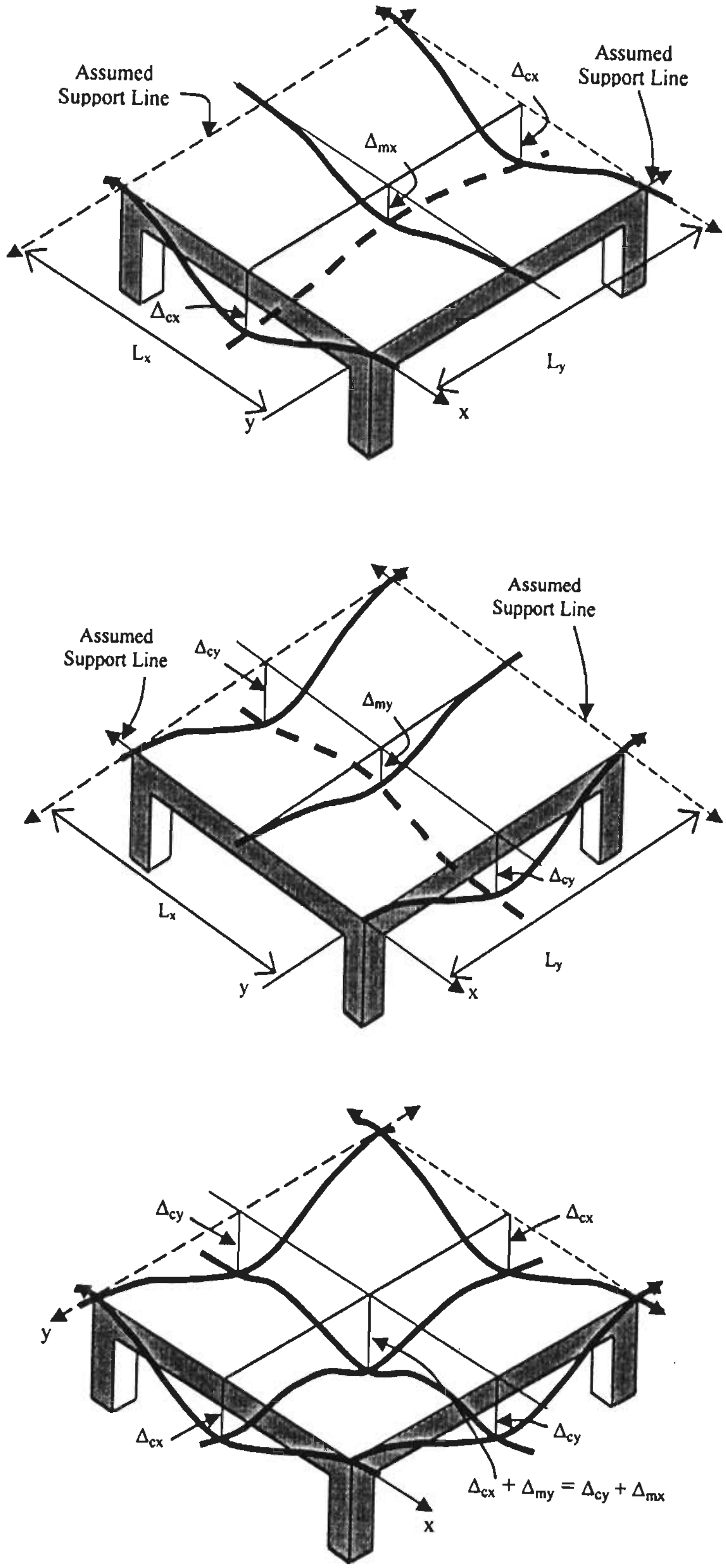


Fig. 4.1 Structural modelling of slab

The dead or sustained load is determined by using the Eqn.4.2 which includes the weight of the slab, floor finish and a part of live load that may act as permanent in nature. For this study 25% of live load (Wl) is considered to act as sustained load.

$$W_d = 25D + FF + 0.25Wl \tag{4.2}$$

The total load (W_t) is computed by adding the live load with the sustained load (W_d) as in Eqn. (4.3)

$$W_t = W_d + Wl \tag{4.3}$$

The moment is calculated using the panel load separately for short direction (M_{t1x}) and long direction (M_{t1y})

$$M_{tlx} = \frac{0.5L_y W_t L_x^2}{8} \tag{4.4}$$

$$M_{tly} = \frac{0.5L_x W_t L_y^2}{8} \tag{4.5}$$

The coefficients for distribution among the column and middle strip are calculated as per Branson method. The distribution for interior panel is always as given below. The relative

$$LDF_{ic} = 0.675$$

$$LDF_{im} = 0.325$$

For the corner and side panel based on the distribution of hogging and sagging moment between column and middle strips.

$$LDF_c = 0.738$$

$$LDF_m = 0.262$$

The modular ratio is $m = \frac{E_s}{E_c}$;

The effective depth (d) to the reinforcement from the top of the concrete surface is determined by deducting the clear cover (cc) and half the diameter (dia) of tension reinforcement.

$$d = D - cc - (dia/2)$$

The width of frame (b_{fx}) is taken as equal to the span along that direction. For plates the effect of column is neglected.

Frame

$$b_{fx} = L_y \tag{4.6}$$

The moment of inertia of frame (I_{fx}) is calculated by considering the plate as a rectangular beam.

$$I_{fx} = \frac{b_{fx} D^3}{12} \tag{4.7}$$

$$b_{fy} = L_x \tag{4.8}$$

$$I_{fy} = \frac{b_{fy} D^3}{12} \tag{4.9}$$

The panel is considered to be continuous and hence the following formula is used for instantaneous deflection due to sustained load (d_{fdx}, d_{fdy}) and total load (d_{ftx}, d_{fty}) separately.

$$df_{dx} = \frac{W_d L_y L_x^4}{384 E_c I_{fx}} \tag{4.10}$$

$$df_{dy} = \frac{W_d L_x L_y^4}{384 E_c I_{fy}} \tag{4.11}$$

$$df_{tx} = \frac{W_t L_y L_x^4}{384 E_c I_{fx}} \tag{4.12}$$

$$df_{ty} = \frac{W_t L_x L_y^4}{384 E_c I_{fy}} \tag{4.13}$$

The section properties for the column strip along short as well as long directions are computed by using the following formulas.

Column Strip

The width of column strip (b_{cx}) in both the direction is the width on each side of the column centre line equal to one fourth the smaller of the two panel dimensions.

$$b_{cx} = b_{cy} = \frac{L_x}{2} \tag{4.14}$$

The area of reinforcement (A_{st1cx}) is calculated as percentage of the cross sectional area of column strip.

$$A_{st1cx} = \frac{0.5 \times b_{cx} \times D}{100}$$

$$I_{cx} = \frac{b_{cx} D^3}{12} \tag{4.15}$$

$$I_{cy} = \frac{b_{cy} D^3}{12} \tag{4.16}$$

Middle Strip

The width of middle strip (b_{mx} ,b_{my}) and the moment of inertia (I_{mx} ,I_{my}) are calculated as below:

$$b_{mx} = L_y - \frac{L_x}{2} \tag{4.17}$$

$$I_{mx} = \frac{b_{mx} D^3}{12} \tag{4.18}$$

$$b_{my} = \frac{L_x}{2} \tag{4.19}$$

$$I_{my} = \frac{b_{my} D^3}{12} \tag{4.20}$$

Flexural stiffness of equivalent column

The stiffness of column is computed based on equivalent column stiffness.

Factored dead load (W_D)

$$W_D = 1.4 \times (25 \times D + FF) \tag{4.21}$$

Factored live load (W_L)

$$W_L = 1.7 \times Wl \tag{4.22}$$

Total factored load (W_T)

$$W_T = W_D + W_L \tag{4.23}$$

Uniformly distributed load (w)

$$w = W_T \times L_y \tag{4.24}$$

Area of column required (A_c)

$$A_c = \frac{2 \times 3 \times W_T \times L_x \times L_y/2}{0.85 \times \sqrt{f_{ck}}} \tag{4.25}$$

Edge column size

The column size (C_{1x}, C_{1y}) is calculated using the following formula.

$$C_{1x} = \frac{((A_c/d) - 2d)}{3} \tag{4.26}$$

$$C_{1y} = \frac{((A_c/d) - 2d)}{3} \tag{4.27}$$

Exterior column rotations for corner and side panel

Short term deflection

Along the shorter direction (x-x axis):

The sectional properties of column and stiffness computations are made by using the following equations.

$$I_{c1x} = \frac{C_{1x} C_{1y}^3}{12} \tag{4.28}$$

$$K_{cx} = \frac{4 E_c I_{c1x}}{L_c} \tag{4.29}$$

$$C_x = \frac{\left(1 - \left(0.63D/C_{1x}\right)\right)\left(D^3 C_{1x}\right)}{3} \tag{4.30}$$

$$L_{nx} = L_x - C_{1x}$$

The model proposed by Gilbert [27] is used here for torsional stiffness (K_{tx}) of edge column.

$$K_{tx} = \frac{2\times 4.5\times E_c \times C_x}{\left(L_y\left(1-\left(\frac{C_{1y}}{L_y}\right)\right)^3\right)} \tag{4.31}$$

Equivalent column stiffness (K_{ecx}) is

$$K_{ecx} = \left(\frac{1}{2\times K_{cx} + K_{tx}}\right) \tag{4.32}$$

$$\alpha_x = \frac{K_{ecx}}{2K_{sx}} \tag{4.33}$$

Distribution factor for positive moment at the exterior Span

$$D_{px} = 0.63 - \frac{0.28}{\left(1+ \frac{1}{\alpha_x}\right)} \tag{4.34}$$

Distribution factor for negative moment at the exterior span

$$D_{nex} = \frac{0.65}{\left(1+ \frac{1}{\alpha_x}\right)} \tag{4.35}$$

Distribution factor for negative moment at the interior span

$$D_{nix} = 0.75 - \frac{0.10}{\left(1+ \frac{1}{\alpha_x}\right)} \tag{4.36}$$

The cracked moment

$$M_{cr} = 6.71\sqrt{f_{ck}} \tag{4.37}$$

$$M_{at1px} = D_{px} M_{t1x} \tag{4.38}$$

If the M_{at1px} is less than the M_{cr} then the value of later is considered.

$$M_{atnex} = D_{nex} M_{t1x} \tag{4.39}$$

If the M_{at1nex} is less than the M_{cr} then the value of later is considered.

$$M_{atnix} = D_{nix} M_{t1x} \tag{4.40}$$

If the M_{at1nix} is less than the M_{cr} then the value of later is considered.

The cracked moment of inertia (I_{crx})

$$I_{crx} = b_{cx}\frac{x^3}{3} + (m)(A_{st1cx})(d-x)^2 \tag{4.41}$$

Effective moment of inertia for positive cracked moment (I_{ecpx}) in column strip

$$I_{ecpx} = \left(\frac{M_{cr}}{M_{at1px}}\right)^3 (I_{cx}) + \left(1 - \left(\frac{M_{cr}}{M_{at1px}}\right)^3\right)(I_{crx}) \tag{4.42}$$

Effective moment of inertia for negative cracked moment (I_{ecnex}) in column strip in exterior span

$$I_{ecnex} = \left(\frac{M_{cr}}{M_{at1nex}}\right)^3 (I_{cx}) + \left(1 - \left(\frac{M_{cr}}{M_{at1nex}}\right)^3\right)(I_{crx}) \tag{4.43}$$

Effective moment of inertia for negative cracked moment in column strip (I_{ecnix}) in interior span

$$I_{ecnix} = \left(\frac{M_{cr}}{M_{at1nix}}\right)^3 (I_{cx}) + \left(1 - \left(\frac{M_{cr}}{M_{at1nix}}\right)^3\right)(I_{crx}) \tag{4.44}$$

Effective moment of inertia (I_{ecx}) for column strip

$$I_{ecx} = 0.7I_{ecpx} + 0.15I_{ecnex} + 0.15I_{ecnix} \tag{4.45}$$

Effective moment of inertia (I_{efx}) for frame

$$I_{efx} = I_{ecx} + I_{mx} \tag{4.46}$$

The moment due to total load (M_{ox})

$$M_{ox} = \frac{W_t \times L_y \times (L_{nx})^2}{8} \tag{4.47}$$

$$M_{ntx} = \frac{0.65 \times M_{ox}}{\left(1 + \frac{1}{\alpha_x}\right)} \tag{4.47}$$

Moment due to sustained load (M_{ndx})

$$M_{ndx} = \left(W_d \big/ W_t \right) M_{ntx} \tag{4.49}$$

The rotation due to total load (θ_{tx})

$$\theta_{tx} = \frac{M_{ntx}}{K_{ecx}} \tag{4.50}$$

The rotation due to sustained load (θ_{dx})

$$\theta_{dx} = \frac{M_{ndx}}{K_{ecx}} \tag{4.51}$$

The deflection due to total load (δ_{tx})

$$\delta_{tx} = \left(\theta_{tx} \times L_x \big/ 8 \right) \left(\frac{I_{fx}}{I_{efx}} \right) \tag{4.52}$$

The deflection due to sustained load (δ_{dx})

$$\delta_{dx} = \frac{\theta_{dx} L_x}{8} \tag{4.53}$$

Short term deflection

Along the longer direction (y-y axis)

The moment of inertia of column (I_{c1y}) along the longer direction

$$I_{c1y} = \frac{C_{1y} C_{1x}^3}{12} \tag{4.54}$$

The relative stiffness of column (K_{cy}) along the longer direction

$$K_{cy} = \frac{4 E_c I_{c1y}}{L_c} \tag{4.55}$$

$$C_y = \frac{\left(1 - \left(0.63 D / C_{1y}\right)\right)\left(D^3 C_{1y}\right)}{3}$$

The clear distance from face to face of column (L_{ny}) along the longer direction

$$L_{ny} = L_y - C_{1y}$$

$$K_{ty} = \frac{2 \times 4.5 \times E_c \times C_y}{\left(L_x\left(1-\left(\frac{C_{1x}}{L_x}\right)\right)^3\right)} \tag{4.56}$$

The relative stiffness of frame (K_{ecy})

$$K_{ecy} = \left(\frac{1}{2 \times K_{cy} + K_{ty}}\right) \tag{4.57}$$

$$\alpha_y = \frac{K_{ecy}}{2K_{sy}} \tag{4.58}$$

Distribution factor for positive moment at the exterior span

$$D_{py} = 0.63 - \frac{0.28}{\left(1 + \frac{1}{\alpha_y}\right)} \tag{4.59}$$

Distribution factor for negative moment at the exterior span

$$D_{ney} = \frac{0.65}{\left(1 + \frac{1}{\alpha_y}\right)} \tag{4.60}$$

Distribution factor for negative moment at the interior span

$$D_{niy} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{\alpha_y}\right)} \tag{4.61}$$

The cracked moment

$$M_{cr} = 6.71\sqrt{f_{ck}} \tag{4.62}$$

$$M_{at1py} = D_{py} M_{t1y} \tag{4.63}$$

If the M_{at1py} is less than the M_{cr} then the value of later is considered.

$$M_{atney} = D_{ney} M_{t1y} \tag{4.64}$$

If the M_{at1ney} is less than the M_{cr} then the value of later is considered.

$$M_{atniy} = D_{niy} M_{t1y} \tag{4.65}$$

If the M_{at1niy} is less than the M_{cr} then the value of later is considered.

The cracked moment of inertia (I_{cry})

$$I_{cry} = b_{cy}\frac{x^3}{3} + (m)(A_{st1cy})(d-x)^2 \tag{4.66}$$

Effective moment of inertia for positive cracked moment (I_{ecpy}) in column strip

$$I_{ecpy} = \left(\frac{M_{cr}}{M_{at1py}}\right)^3 (I_{cy}) + \left(1-\left(\frac{M_{cr}}{M_{at1py}}\right)^3\right)(I_{cry}) \tag{4.67}$$

Effective moment of inertia for negative cracked moment (I_{ecney}) in column strip at the exterior span

$$I_{ecney} = \left(\frac{M_{cr}}{M_{at1ney}}\right)^3 (I_{cy}) + \left(1-\left(\frac{M_{cr}}{M_{at1ney}}\right)^3\right)(I_{cry}) \tag{4.68}$$

Effective moment of inertia for negative cracked moment (I_{ecniy}) in column strip at the interior span

$$I_{ecniy} = \left(\frac{M_{cr}}{M_{at1niy}}\right)^3 (I_{cy}) + \left(1-\left(\frac{M_{cr}}{M_{at1niy}}\right)^3\right)(I_{cry}) \tag{4.69}$$

Effective moment of inertia for column strip (I_{ecy})

$$I_{ecy} = 0.7I_{ecpy} + 0.15I_{ecney} + 0.15I_{ecniy} \tag{4.70}$$

Effective moment of inertia for frame (I_{efy})

$$I_{efy} = I_{ecy} + I_{my} \tag{4.71}$$

The moment due to total load (M_{oy})

$$M_{oy} = \frac{W_t \times L_x \times (L_{ny})^2}{8} \tag{4.72}$$

$$M_{nty} = \frac{0.65 \times M_{oy}}{\left(1 + \frac{1}{\alpha_y}\right)} \tag{4.73}$$

Moment due to sustained load (M_{ndy})

$$M_{ndy} = \left(W_d \big/ W_t \right) M_{nty} \tag{4.74}$$

The rotation due to total load (θ_{ty})

$$\theta_{ty} = \frac{M_{nty}}{K_{ecy}} \tag{4.75}$$

The rotation due to sustained load (θ_{dy})

$$\theta_{dy} = \frac{M_{ndy}}{K_{ecy}} \tag{4.76}$$

The deflection due to total load (δ_{ty})

$$\delta_{ty} = \left(\theta_{ty} \times L_y \big/ 8 \right) \left(\frac{I_{fy}}{I_{efy}} \right) \tag{4.77}$$

The deflection due to sustained load (δ_{dy})

$$\delta_{dy} = \frac{\theta_{dy} L_y}{8} \tag{4.78}$$

Deflection of column strip along shorter direction (x-x axis)

Interior Panel

Deflection due to sustained load (δ_{cxid})

$$\delta_{cxid} = \frac{(LDF_{ci})(\delta_{fdx})(I_{fx})}{I_{cx}} \tag{4.79}$$

Deflection due to total load (δ_{cxit})

$$\delta_{cxit} = \frac{(LDF_{ci})(\delta_{ftx})(I_{fx})}{I_{cx}} \tag{4.80}$$

Deflection due to varying part of live load (δ_{cxil})

$$\delta_{cxil} = \delta_{cxit} - \delta_{cxid} \tag{4.81}$$

Corner Panel

Deflection due to sustained load (δ_{cxcd})

$$\delta_{cxcd} = \frac{(LDF_c)(\delta_{fdx})(I_{fx})}{I_{cx}} + \delta_{dx} \tag{4.82}$$

Deflection due to total load (δ_{cxct})

$$\delta_{cxct} = \frac{(LDF_c)(\delta_{ftx})(I_{fx})}{I_{efy}} + \delta_{tx} \tag{4.83}$$

Deflection due to varying part of live load (δ_{cxcl})

$$\delta_{cxcl} = \delta_{cxct} - \delta_{cxcd} \tag{4.84}$$

Side Panel

Deflection due to sustained load (δ_{cxsd})

$$\delta_{cxsd} = \frac{(LDF_c)(\delta_{fdx})(I_{fx})}{I_{cx}} \tag{4.85}$$

Deflection due to total load (δ_{cxst})

$$\delta_{cxst} = \frac{(LDF_c)(\delta_{ftx})(I_{fx})}{I_{efy}} \tag{4.86}$$

Deflection due to varying part of live load (δ_{cxsl})

$$\delta_{cxsl} = \delta_{cxst} - \delta_{cxsd} \tag{4.87}$$

Deflection of column strip along longer direction (y-y axis)

Interior Panel

Deflection due to sustained load (δ_{cyid})

$$\delta_{cyid} = \frac{(LDF_{ci})(\delta_{fdy})(I_{fx})}{I_{cy}} \tag{4.88}$$

Deflection due to total load (δ_{cyit})

$$\delta_{cyit} = \frac{(LDF_{ci})(\delta_{fty})(I_{fx})}{I_{cy}} \tag{4.89}$$

Deflection due to varying part of live load (δ_{cyil})

$$\delta_{cyil} = \delta_{cyit} - \delta_{cyid} \tag{4.90}$$

Corner Panel

Deflection due to sustained load (δ_{cycd})

$$\delta_{cycd} = \frac{(LDF_c)(\delta_{fdy})(I_{fy})}{I_{cy}} + \delta_{dy} \tag{4.91}$$

Deflection due to total load (δ_{cyct})

$$\delta_{cyct} = \frac{(LDF_c)(\delta_{fly})(I_{fy})}{I_{cy}} + \delta_{dy} \tag{4.92}$$

Deflection due to varying part of live load (δ_{cycl})

$$\delta_{cycl} = \delta_{cyct} - \delta_{cycd} \tag{4.93}$$

Side Panel

Deflection due to sustained load (δ_{cysd})

$$\delta_{cysd} = \frac{(LDF_c)(\delta_{fdy})(I_{fy})}{I_{cy}} + \delta_{dy} \tag{4.94}$$

Deflection due to total load (δ_{cyst})

$$\delta_{cyst} = \frac{(LDF_c)(\delta_{fly})(I_{fy})}{I_{efy}} + \delta_{ty} \tag{4.95}$$

Deflection due to varying part of live load (δ_{cysl})

$$\delta_{cysl} = \delta_{cyst} - \delta_{cysd} \tag{4.96}$$

Deflection of middle strip along shorter direction (x-x axis)

Interior Panel

Deflection due to sustained load (δ_{mxid})

$$\delta_{mxid} = \frac{(LDF_{mi})(\delta_{fdx})(I_{fx})}{I_{mx}} \tag{4.97}$$

Deflection due to total load (δ_{mxit})

$$\delta_{mxit} = \frac{(LDF_{mi})(\delta_{ftx})(I_{fx})}{I_{mx}} \tag{4.98}$$

Deflection due to varying part of live load (δ_{mxil})

$$\delta_{mxil} = \delta_{mxit} - \delta_{mxid} \tag{4.99}$$

Corner Panel

Deflection due to sustained load (δ_{mxcd})

$$\delta_{mxcd} = \frac{(LDF_{m})(\delta_{fdx})(I_{fx})}{I_{mx}} + \delta_{dx} \tag{4.100}$$

Deflection due to total load (δ_{mxct})

$$\delta_{mxct} = \frac{(LDF_{m})(\delta_{ftx})(I_{fx})}{I_{mx}} + \delta_{tx} \tag{4.101}$$

Deflection due to varying part of live load (δ_{mxcl})

$$\delta_{mxcl} = \delta_{mxct} - \delta_{mxcd} \tag{4.102}$$

Side Panel

Deflection due to sustained load (δ_{mxsd})

$$\delta_{mxsd} = \frac{(LDF_{m})(\delta_{fdx})(I_{fx})}{I_{mx}} \tag{4.103}$$

Deflection due to total load (δ_{mxst})

$$\delta_{mxst} = \frac{(LDF_{m})(\delta_{ftx})(I_{fx})}{I_{mx}} \tag{4.104}$$

Deflection due to varying part of live load (δ_{mxsl})

$$\delta_{mxsl} = \delta_{mxst} - \delta_{mxsd} \tag{4.105}$$

Deflection of middle strip along longer direction (y-y axis)

Interior panel

Deflection due to sustained load (δ_{myid})

$$\delta_{myid} = \frac{(LDF_{mi})(\delta_{fdy})(I_{fy})}{I_{my}} \tag{4.106}$$

Deflection due to total load (δ_{myit})

$$\delta_{myit} = \frac{(LDF_{mi})(\delta_{fty})(I_{fy})}{I_{my}} \tag{4.107}$$

Deflection due to varying part of live load (δ_{myil})

$$\delta_{myil} = \delta_{myit} - \delta_{myid} \tag{4.108}$$

Corner panel

Deflection due to sustained load (δ_{mycd})

$$\delta_{mycd} = \frac{(LDF_{m})(\delta_{fdy})(I_{fy})}{I_{my}} + \delta_{dy} \tag{4.109}$$

Deflection due to total load (δ_{myct})

$$\delta_{myct} = \frac{(LDF_{m})(\delta_{fty})(I_{fy})}{I_{my}} + \delta_{ty} \tag{4.110}$$

Deflection due to varying part of live load (δ_{mycl})

$$\delta_{mycl} = \delta_{myct} - \delta_{mycd} \tag{4.111}$$

Side panel

Deflection due to sustained load (δ_{mysd})

$$\delta_{mysd} = \frac{(LDF_{m})(\delta_{fdy})(I_{fy})}{I_{my}} + \delta_{dy} \tag{4.112}$$

Deflection due to total load (δ_{myst})

$$\delta_{myst} = \frac{(LDF_{m})(\delta_{fty})(I_{fy})}{I_{my}} + \delta_{ty} \tag{4.113}$$

Deflection due to varying part of live load (δ_{mysl})

$$\delta_{mysl} = \delta_{myst} - \delta_{mysd} \tag{4.114}$$

Total mid panel deflection due to sustained load

Interior Panel

For two way action, the deflection along shorter direction is computed by adding the deflection of column strip along x-x direction and deflection of middle strip along y-y direction.

Deflection due to sustained load (δ_{idx}) along x-x direction

$$\delta_{idx} = \delta_{cxid} + \delta_{myid} \tag{4.115}$$

Deflection due to sustained load (δ_{idy}) along y-y direction

$$\delta_{idy} = \delta_{cyid} + \delta_{mxid} \tag{4.116}$$

The total mid panel deflection of interior panel (δ_{id}) for sustained load, is the average of the above two deflections.

$$\delta_{id} = \frac{(\delta_{idx} + \delta_{idy})}{2} \tag{4.117}$$

The above procedure is followed for the corner as well as side panels.

Corner Panel

Deflection due to sustained load (δ_{cdx}) along x-x direction

$$\delta_{cdx} = \delta_{cxcd} + \delta_{mycd} \tag{4.118}$$

Deflection due to sustained load (δ_{cdy}) along y-y direction

$$\delta_{cdy} = \delta_{cycd} + \delta_{mxcd} \tag{4.119}$$

Total mid panel deflection of corner panel (δ_{cd}) for sustained load

$$\delta_{cd} = \frac{(\delta_{cdx} + \delta_{cdy})}{2} \tag{4.120}$$

Side Panel

Deflection due to sustained load (δ_{sdx}) along x-x direction

$$\delta_{sdx} = \delta_{cxsd} + \delta_{mysd} \tag{4.121}$$

Deflection due to sustained load (δ_{sdy}) along y-y direction

$$\delta_{sdy} = \delta_{cysd} + \delta_{mxsd} \tag{4.122}$$

Total mid panel deflection of side panel (δ_{sd}) for sustained load

$$\delta_{sd} = \frac{(\delta_{sdx} + \delta_{sdy})}{2} \tag{4.123}$$

Total mid panel deflection due to varying part of live load

Interior Panel

Deflection due to varying part of live load (δ_{ilx}) along x-x direction

$$\delta_{ilx} = \delta_{cxil} + \delta_{myil} \tag{4.124}$$

Deflection due to varying part of live load (δ_{ily}) along y-y direction

$$\delta_{ily} = \delta_{cyil} + \delta_{mxil} \tag{4.125}$$

The total mid panel deflection of interior panel (δ_{il}) for varying part of live load

$$\delta_{il} = \frac{(\delta_{ilx} + \delta_{ily})}{2} \tag{4.126}$$

Corner Panel

Deflection due to varying part of live load (δ_{clx}) along x-x direction

$$\delta_{clx} = \delta_{cxcl} + \delta_{mycl} \tag{4.127}$$

Deflection due to varying part of live load (δ_{cly}) along y-y direction

$$\delta_{cly} = \delta_{cycl} + \delta_{mxcl} \tag{4.128}$$

The total mid panel deflection of corner panel (δ_{cl}) for varying part of live load

$$\delta_{cl} = \frac{(\delta_{clx} + \delta_{cly})}{2} \tag{4.129}$$

Side Panel

Deflection due to varying part of live load (δ_{slx}) along x-x direction

$$\delta_{slx} = \delta_{cxsl} + \delta_{mysl} \quad (4.130)$$

Deflection due to varying part of live load (δ_{sly}) along y-y direction

$$\delta_{sly} = \delta_{cysl} + \delta_{mxsl} \quad (4.131)$$

The total mid panel deflection of side panel (δ_{sl}) for varying part of live load

$$\delta_{sl} = \frac{(\delta_{slx} + \delta_{sly})}{2} \quad (4.132)$$

Total deflection

The total deflection including long-term effects is computed by multiplying the deflection due to sustained load by 3 and the deflection due to live load is added with this.

Interior Panel

$$\delta_i = 3 \times \delta_{idl} + \delta_{ill} \quad (4.133)$$

Corner Panel

$$\delta_c = 3 \times \delta_{cdl} + \delta_{cll} \quad (4.134)$$

Side Panel

$$\delta_s = 3 \times \delta_{sdl} + \delta_{sll} \quad (4.135)$$

4.3 AS PER BS: 8110-1997

Following the same procedure adopted in 4.2 the equations given in this code for creep and shrinkage effects are studied and modified.

Moments

$$W_d = 25D + FF + 0.25Wl \quad (4.2)$$

$$W_t = W_d + Wl \quad (4.3)$$

The moment due to sustained load is calculated using the panel load separately for short direction (M_{d1x}) and long direction (M_{d1y})

$$M_{dlx} = \frac{0.5 L_y W_d L_x^2}{8} \tag{4.136}$$

$$M_{dly} = \frac{0.5 L_x W_d L_y^2}{8} \tag{4.137}$$

The moment due to total load is calculated using the panel load separately for short direction (M_{tlx}) and long direction (M_{tly})

$$M_{tlx} = \frac{0.5 L_y W_t L_x^2}{8} \tag{4.4}$$

$$M_{tly} = \frac{0.5 L_x W_t L_y^2}{8} \tag{4.5}$$

Short-term deflection (instantaneous curvatures for cracked section)

$$m = \frac{E_s}{E_c}$$

$$d = D - cc - (dia/2)$$

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

$$m_{ce} = \frac{E_s}{E_{ce}}$$

$LDF_c = 0.65$ (Lateral moment distribution factor in column strip for interior panel)

$LDF_m = 0.35$ (Lateral moment distribution factor in middle strip for interior panel)

$LDF_{1c} = 0.55$ (Lateral moment distribution factor in column strip for corner & side panel)

$LDF_{1m} = 0.45$ (Lateral moment distribution factor in middle strip for corner & side panel)

Column Strip

$$b_{cx} = b_{cy} = \frac{L_x}{2}$$

$$A_{st1cx} = \frac{0.5 \times b_{cx} \times D}{100}$$

Solving the following equation the x_{1cx} may be found

$$0.5 \times b_{cx} \times x_{1cx}^2 + m \times A_{st1cx} \times x_{1cx} = m \times A_{st1cx} \times (d) \quad (4.138)$$

$$A_{st2cx} = \frac{0.5 \times b_{cx} \times D}{100} \quad (4.139)$$

$$0.5 \times L_x \times x_{2cx}^2 + m \times A_{st2cx} \times x_{2cx} = m \times A_{st2cx} \times (d) \quad (4.140)$$

$$I_{cx} = \left(\frac{0.5 \times L_x \times D^3}{12}\right) + 0.5 \times L_x \times D \times \left(D/2 - x_{1cx}\right)^2 + (m-1) \times A_{st1cx} \times (d - x_{1cx})^2$$

(4.141)

$$A_{st1cy} = \frac{0.5 \times b_{cy} \times D}{100} \quad (4.142)$$

$$0.5 \times 0.5 \times L_x \times x_{1cy}^2 + m \times A_{st1cy} \times x_{1cy} = m \times A_{st1cy} \times (d) \quad (4.143)$$

$$A_{st2cy} = \frac{0.5 \times b_{cy} \times D}{100} \quad (4.144)$$

$$0.5 \times L_x \times x_{2cy}^2 + m \times A_{st2cy} \times x_{2cy} = m \times A_{st2cy} \times (d) \quad (4.145)$$

$$I_{cy} = \left(\frac{0.5 \times L_y \times D^3}{12}\right) + 0.5 \times L_y \times D \times \left(D/2 - x_{1cy}\right)^2 + (m-1) \times A_{st1cy} \times (d - x_{1cy})^2$$

(4.146)

Middle Strip

$$b_{mx} = L_y - \frac{L_x}{2}$$

$$A_{st1mx} = \frac{0.5 \times b_{mx} \times D}{100} \quad (4.147)$$

$$0.5 \times b_{mx} \times x_{1mx}^2 + m \times A_{st1mx} \times x_{1mx} = m \times A_{st1mx} \times (d) \quad (4.148)$$

$$I_{mx} = \left(\frac{0.5 \times L_x \times D^3}{12}\right) + 0.5 \times L_x \times D \times \left(D/2 - x_{1mx}\right)^2 + (m-1) \times A_{st1mx} \times (d - x_{1mx})^2$$

(4.149)

$$b_{my} = \frac{L_x}{2}$$

$$A_{st1my} = \frac{0.5 \times b_{my} \times D}{100} \tag{4.150}$$

$$0.5 \times b_{my} \times x_{1my}^2 + m \times A_{st1my} \times x_{1my} = m \times A_{st1my} \times (d) \tag{4.151}$$

$$I_{my} = \left(\frac{0.5 \times L_y \times D^3}{12} \right) + 0.5 \times L_y \times D \times \left(D/2 - x_{1my} \right)^2 + (m-1) \times A_{st1my} \times \left(d - x_{1my} \right)^2 \tag{4.152}$$

Flexural stiffness of equivalent column

Factored dead load (W_D)

$$W_D = 1.4 \times 25 \times D + FF \tag{4.21}$$

Factored live load (W_L)

$$W_L = 1.7 \times Wl \tag{4.22}$$

Total load (W_T)

$$W_T = W_D + W_L \tag{4.23}$$

Uniformly distributed load (w)

$$w = W_T \times L_y \tag{4.24}$$

Short term deflection along x-x axis

Moment of inertia of column (I_{c1x})

$$I_{c1x} = \frac{C_{1x} C_{1y}^3}{12} \tag{4.28}$$

The stiffness of column (K_{cx})

$$K_{cx} = \frac{4 E_c I_{c1x}}{L_c} \tag{4.29}$$

$$C_x = \frac{\left(1 - (0.63 \times D / C_{1x})\right)\left(D^3 C_{1x}\right)}{3} \tag{4.30}$$

Clear span (L_{nx})

$$L_{nx} = L_x - C_{1x}$$

Stiffness of equivalent column (K_{ecx})

$$K_{ecx} = \left(\frac{1}{2 \times K_{cx} + K_{tx}} \right) \tag{4.32}$$

$$\alpha_{1x} = \frac{K_{ecx}}{2K_{s1x}} \tag{4.153}$$

Stiffness of equivalent column (K_{ecy})

$$K_{ecy} = \left(\frac{1}{2 \times K_{cy} + K_{ty}} \right) \tag{4.154}$$

$$\alpha_{1y} = \frac{K_{ecy}}{2K_{sy}} \tag{4.155}$$

Distribution factor (D_{px}) for positive moment in exterior span

$$D_{px} = 0.63 - \frac{0.28}{\left(1 + \frac{1}{\alpha_{1x}}\right)} \tag{4.59}$$

Distribution factor (D_{nex}) for negative moment in exterior span

$$D_{nex} = \frac{0.65}{\left(1 + \frac{1}{\alpha_{1x}}\right)} \tag{4.60}$$

Distribution factor (D_{nix}) for negative moment in interior span

$$D_{nix} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{\alpha_{1x}}\right)} \tag{4.61}$$

The cracked moment

$$M_{cr} = 6.71\sqrt{f_{ck}} \tag{4.37}$$

$$M_{atpx} = D_{px} \times M_{t1x} \tag{4.63}$$

$$M_{atnex} = D_{nex} \times M_{t1x} \tag{4.64}$$

$$M_{atnix} = D_{nix} \times M_{t1x} \tag{4.65}$$

The cracked moment of inertia

$$I_{cry} = b_{cy}\frac{x^3}{3} + (m)(A_{st1cy})(d-x)^2 \tag{4.66}$$

Effective moment of inertia for positive cracked moment in column strip

$$I_{ecpy} = \left(\frac{M_{cr}}{M_{at1py}}\right)^3 (I_{cy}) + \left(1-\left(\frac{M_{cr}}{M_{at1py}}\right)^3\right)(I_{cry}) \tag{4.67}$$

Effective moment of inertia for negative cracked moment in column strip in exterior span

$$I_{ecney} = \left(\frac{M_{cr}}{M_{at1ney}}\right)^3 (I_{cy}) + \left(1-\left(\frac{M_{cr}}{M_{at1ney}}\right)^3\right)(I_{cry}) \tag{4.68}$$

Effective moment of inertia for negative cracked moment in column strip in interior span

$$I_{ecniy} = \left(\frac{M_{cr}}{M_{at1niy}}\right)^3 (I_{cy}) + \left(1-\left(\frac{M_{cr}}{M_{at1niy}}\right)^3\right)(I_{cry}) \tag{4.69}$$

Effective moment of inertia for column strip

$$I_{ecy} = 0.7 I_{ecpy} + 0.15 I_{ecney} + 0.15 I_{ecniy} \tag{4.70}$$

Effective moment of inertia for frame

$$I_{efy} = I_{ecy} + I_{my} \tag{4.71}$$

The moment due to total load (M_{oy})

$$M_{oy} = \frac{W \times_t L_x \times (L_{ny})^2}{8} \tag{4.72}$$

Negative moment (M_{ney})

$$M_{ney} = \frac{0.65 \times M_{oy}}{\left(1 + \frac{1}{\alpha_y}\right)} \tag{4.73}$$

Moment due to sustained load (M_{ndy})

$$M_{ndy} = \left(\frac{W_d}{W_t}\right) M_{nty} \tag{4.74}$$

The rotation (θ_{ty}) due to total load

$$\theta_{ty} = \frac{M_{nty}}{K_{ecy}} \tag{4.75}$$

The rotation (θ_{dy}) due to sustained load

$$\theta_{dy} = \frac{M_{ndy}}{K_{ecy}} \tag{4.76}$$

The deflection (δ_{ty}) due to total load

$$\delta_{ty} = \left(\theta_{ty} \times L_y \big/ 8\right)\left(\frac{I_{fy}}{I_{efy}}\right) \tag{4.77}$$

The deflection (δ_{dy}) due to sustained load

$$\delta_{dy} = \frac{\theta_{dy} \times L_y}{8} \tag{4.78}$$

The moment (M_{ox}) due to total load

$$M_{ox} = \frac{W_t \times L_y \times (L_{nx})^2}{8} \tag{4.47}$$

The moment (M_{nt1x}) due to total load

$$M_{nt1x} = \frac{0.65 \times M_{ox}}{\left(1 + 1 \big/ \alpha_{1x}\right)} \tag{4.153}$$

The moment (M_{nd1x}) due to sustained load

$$M_{nd1x} = \frac{W_d}{W_t} \times M_{nt1x} \tag{4.154}$$

The rotation (θ_{t1x}) due to total load

$$\theta_{t1x} = \frac{M_{nt1x}}{K_{ecx}} \tag{4.155}$$

The rotation (θ_{d1x}) due to sustained load

$$\theta_{d1x} = \frac{M_{nd1x}}{K_{ecx}} \tag{4.156}$$

The deflection (δ_{tx}) due to total load

$$\delta_{tx} = \left(\theta_{t1x} \times L_x / 8\right)\left(\frac{I_{gx}}{I_{efx}}\right) \tag{4.157}$$

The deflection (δ_{dx}) due to total load

$$\delta_{dx} = \left(\theta_{d1x} \times L_x / 8\right) \tag{4.158}$$

Long term deflection along the shorter x-x direction

The relative stiffness (K_{s2x}) for the long term effects

$$K_{s2x} = \frac{4 \times E_c \times L_y \times D^3}{3} + \frac{m \times A_{st2cy} \times \left(d - x_{2cy}\right)^2}{L_x} \tag{4.159}$$

$$\alpha_{2x} = \frac{K_{ecx}}{2K_{s2x}} \tag{4.160}$$

The moment (M_{nt2x}) due to total load

$$M_{nt2x} = \frac{0.5 \times L_y \times W_t \times L_x^2}{8} \tag{4.161}$$

The moment (M_{nd2x}) due to sustained load

$$M_{nd2x} = \frac{W_d}{W_t} \times M_{nt2x} \tag{4.162}$$

The rotation (θ_{d2x}) due to sustained load

$$\theta_{d2x} = \frac{M_{nd2x}}{K_{ecx}} \tag{4.163}$$

The deflection (δ_{dx}) due to sustained load

$$\delta_{dx} = \left(\theta_{d2x} \times L_x / 8\right) \tag{4.164}$$

The moment (M_{oy}) due to total load

$$M_{oy} = \frac{W_t \times L_x \times \left(L_{ny}\right)^2}{8} \tag{4.72}$$

Long term deflection along the longer (y-y) direction

The relative stiffness (K_{s2y}) for the long term effects

$$K_{s2y} = \frac{4 \times E_c \times L_x \times D^3}{3} + \frac{m \times A_{st2cx} \times (d - x_{2cx})^2}{L_y} \tag{4.165}$$

$$\alpha_{2y} = \frac{K_{ecy}}{2K_{s2y}} \tag{4.166}$$

The moment (M_{nt2y}) due to total load

$$M_{nt2y} = \frac{0.5 \times L_x \times W_t \times L_y^2}{8} \tag{4.167}$$

The moment (M_{nd2y}) due to sustained load

$$M_{nd2y} = \frac{W_d}{W_t} \times M_{nt2y} \tag{4.168}$$

The rotation (θ_{d2y}) due to sustained load

$$\theta_{d2y} = \frac{M_{nd2y}}{K_{ecy}} \tag{4.169}$$

The deflection (δ_{dy}) due to sustained load

$$\delta_{dy} = \left(\theta_{d2y} \times L_y / 8 \right) \tag{4.170}$$

The force in concrete in tension in the column strip (T_{ccx}) along the shorter direction for short term load

$$T_{ccx} = \left(\frac{0.5 \times f_{ct} \times (D - x_{c1x})^2}{(d - x_{c1x})} \right) \tag{4.171}$$

The force in concrete in tension in the column strip (T_{c1cx}) along the shorter direction for long term load

$$T_{c1cx} = \left(\frac{0.5 \times f_{ct1} \times (D - x_{c1x})^2}{(d - x_{c1x})} \right) \tag{4.172}$$

The moment of resistance of the concrete in tension in the column strip (M_{ccx}) for short term load

$$M_{ccx} = \frac{2 \times T_{ccx} \times (D - x_{c1x})}{3} \tag{4.173}$$

The moment of resistance of the concrete in tension in the column strip (M_{clcx}) for long term load

$$M_{clcx} = \frac{2 \times T_{clcx} \times (D - x_{c1x})}{3} \tag{4.174}$$

The force in concrete in tension in the column strip (T_{ccy}) along the longer direction for short term load

$$T_{ccy} = \left(\frac{0.5 \times f_{ct} \times (D - x_{c1y})^2}{(d - x_{c1y})} \right) \tag{4.175}$$

The force in concrete in tension in the column strip (T_{clcy}) along the longer direction for long term load

$$T_{clcy} = \left(\frac{0.5 \times f_{ctl} \times (D - x_{c1y})^2}{(d - x_{c1y})} \right) \tag{4.176}$$

The moment of resistance of the concrete in tension in the column strip (M_{ccy}) for short term load

$$M_{ccy} = \frac{2 \times T_{ccy} \times (D - x_{c1y})}{3} \tag{4.177}$$

The moment of resistance of the concrete in tension in the column strip (M_{clcy}) for long term load

$$M_{clcy} = \frac{2 \times T_{clcy} \times (D - x_{c1y})}{3} \tag{4.178}$$

The force in concrete in tension in the middle strip (T_{cmx}) along the shorter direction for short term load

$$T_{cmx} = \left(\frac{0.5 \times f_{ct} \times (D - x_{m1x})^2}{(d - x_{m1x})} \right) \tag{4.179}$$

The force in concrete in tension in the middle strip (T_{c1mx}) along the shorter direction for long term load

$$T_{c1mx} = \left(\frac{0.5 \times f_{ct1} \times (D - x_{m1x})^2}{(d - x_{m1x})} \right) \tag{4.180}$$

The moment of resistance of the concrete in tension in the middle strip (M_{cmx}) for short term load

$$M_{cmx} = \frac{2 \times T_{cmx} \times (D - x_{m1x})}{3} \tag{4.181}$$

The moment of resistance of the concrete in tension in the middle strip (M_{c1mx}) for short term load

$$M_{c1mx} = \frac{2 \times T_{c1mx} \times (D - x_{m1x})}{3} \tag{4.183}$$

The force in concrete in tension in the middle strip (T_{cmy}) along the longer direction for short term load

$$T_{cmy} = \left(\frac{0.5 \times f_{ct} \times (D - x_{m1y})^2}{(d - x_{m1y})} \right) \tag{4.184}$$

The force in concrete in tension in the middle strip (T_{c1my}) along the longer direction for long term load

$$T_{c1my} = \left(\frac{0.5 \times f_{ct1} \times (D - x_{m1y})^2}{(d - x_{m1y})} \right) \tag{4.185}$$

The moment of resistance of the concrete in tension in the middle strip (M_{cmy}) for short term load

$$M_{cmy} = \frac{2 \times T_{cmy} \times (D - x_{m1y})}{3} \tag{4.186}$$

The moment of resistance of the concrete in tension in the middle strip (M_{c1my}) for long term load

$$M_{c1my} = \frac{2 \times T_{c1my} \times (D - x_{m1y})}{3} \tag{4.187}$$

For total load

Column Strip along x-x direction

Interior Panel

The moment (M_{r1icx}) due to total load

$$M_{r1icx} = M_{t1x} - M_{ccx} \tag{4.188}$$

Curvature due to total load for column strip (ψ_{1icx}) along the shorter span

$$\psi_{1icx} = LDF_c \times \left(\frac{M_{r1icx}}{E_c \times I_{cx}} \right) \tag{4.189}$$

Corner Panel

Curvature due to total load for column strip (ψ_{1ccx}) along the shorter span

$$\psi_{1ccx} = LDF_{1c} \times \left(\frac{M_{r1icx}}{E_c \times I_{ecx}} \right) \tag{4.190}$$

Side Panel

Curvature due to total load for column strip (ψ_{1scx}) along the shorter span

$$\psi_{1scx} = LDF_c \times \left(\frac{M_{r1icx}}{E_c \times I_{cx}} \right) \tag{4.191}$$

Column Strip along y-y direction

Interior Panel

The moment (M_{r1icy}) due to total load

$$M_{r1icy} = M_{t1y} - M_{ccy} \tag{4.192}$$

Curvature due to total load for column strip (ψ_{1icy}) along the longer span

$$\psi_{1icy} = LDF_c \times \left(\frac{M_{r1icy}}{E_c \times I_{cy}} \right) \tag{4.193}$$

Corner Panel

Curvature due to total load for column strip (ψ_{1ccy}) along the longer span

$$\psi_{1ccy} = LDF_{1c} \times \left(\frac{M_{r1icy}}{E_c \times I_{ecy}} \right) \tag{4.194}$$

Side Panel

Curvature due to total load for column strip (ψ_{1scy}) along the longer span

$$\psi_{1scy} = LDF_c \times \left(\frac{M_{r1icy}}{E_c \times I_{cy}} \right) \tag{4.195}$$

Middle strip along x-x direction

Interior Panel

The moment (M_{r1imx}) due to total load

$$M_{r1imx} = M_{t1x} - M_{cmx} \tag{4.196}$$

Curvature due to total load for middle strip (ψ_{1imx}) along the shorter span

$$\psi_{1imx} = LDF_m \times \left(\frac{M_{r1imx}}{E_c \times I_{mx}} \right) \tag{4.197}$$

Corner Panel

Curvature due to total load for middle strip (ψ_{1cmx}) along the shorter span

$$\psi_{1cmx} = LDF_{1m} \times \left(\frac{M_{r1imx}}{E_c \times I_{mx}} \right) \tag{4.198}$$

Side Panel

Curvature due to total load for middle strip (ψ_{1smx}) along the shorter span

$$\psi_{1smx} = LDF_m \times \left(\frac{M_{r1imx}}{E_c \times I_{mx}} \right) \tag{4.199}$$

Middle strip along y-y direction

Interior Panel

The moment (M_{r1imy}) due to total load

$$M_{r1imy} = M_{t1y} - M_{cmy} \tag{4.200}$$

Curvature due to total load for middle strip (ψ_{1imy}) along the longer span

$$\psi_{1imy} = LDF_m \times \left(\frac{M_{r1imy}}{E_c \times I_{my}} \right) \tag{4.201}$$

Corner Panel

Curvature due to total load for middle strip (ψ_{1cmy}) along the longer span

$$\psi_{1cmy} = LDF_{1m} \times \left(\frac{M_{r1imy}}{E_c \times I_{my}} \right) \tag{4.202}$$

Side Panel

Curvature due to total load for middle strip (ψ_{1smy}) along the longer span

$$\psi_{1smy} = LDF_m \times \left(\frac{M_{r1imy}}{E_c \times I_{my}} \right) \tag{4.203}$$

Curvature1x

Interior Panel

Instantaneous curvature (ψ_{i1x}) in the shorter direction

$$\psi_{i1x} = \psi_{1icx} + \psi_{1imy} \tag{4.204}$$

Corner Panel

Instantaneous curvature (ψ_{c1x}) in the shorter direction

$$\psi_{c1x} = \psi_{1ccx} + \psi_{1cmy} \tag{4.205}$$

Side Panel

Instantaneous curvature (ψ_{s1x}) in the shorter direction

$$\psi_{s1x} = \psi_{1scx} + \psi_{1smy} \tag{4.206}$$

Curvature1y

Interior Panel

Instantaneous curvature (ψ_{i1y}) in the longer direction

$$\psi_{i1y} = \psi_{1icy} + \psi_{1imx} \tag{4.207}$$

Corner Panel

Instantaneous curvature (ψ_{c1y}) in the longer direction

$$\psi_{c1y} = \psi_{1ccy} + \psi_{1cmx} \tag{4.208}$$

Side Panel

Instantaneous curvature (ψ_{s1y}) in the longer direction

$$\psi_{s1y} = \psi_{1scy} + \psi_{1smx} \tag{4.209}$$

For sustained load

Column Strip along x-x direction

Interior Panel

The moment (M_{r2icx}) due to sustained load

$$M_{r2icx} = M_{operx} - M_{ccx} \tag{4.210}$$

Curvature due to sustained load for column strip (ψ_{2icx}) along the shorter span

$$\psi_{2icx} = LDF_c \times \left(\frac{M_{r2icx}}{E_c \times I_{cx}} \right) \tag{4.211}$$

Corner Panel

Curvature due to sustained load for column strip (ψ_{2ccx}) along the shorter span

$$\psi_{2ccx} = LDF_{1c} \times \left(\frac{M_{r2ccx}}{E_c \times I_{ecx}} \right) \tag{4.212}$$

Side Panel

Curvature due to sustained load for column strip (ψ_{2scx}) along the shorter span

$$\psi_{2scx} = LDF_c \times \left(\frac{M_{r2scx}}{E_c \times I_{cx}} \right) \tag{4.213}$$

Column Strip along y-y direction

Interior Panel

The moment (M_{r2icy}) due to sustained load is

$$M_{r2icy} = M_{opery} - M_{ccy} \tag{4.214}$$

Curvature due to sustained load for column strip (ψ_{2icy}) along the longer span is calculated by

$$\psi_{2icy} = LDF_c \times \left(\frac{M_{r2icy}}{E_c \times I_{cy}} \right) \tag{4.215}$$

Corner Panel

Curvature due to sustained load for column strip (ψ_{2ccy}) along the longer span is given by

$$\psi_{2ccy} = LDF_{1c} \times \left(\frac{M_{r2ccy}}{E_c \times I_{ecy}} \right) \tag{4.216}$$

Side Panel

Curvature due to sustained load for column strip (ψ_{2scy}) along the longer span is

$$\psi_{2scy} = LDF_c \times \left(\frac{M_{r2scy}}{E_c \times I_{cy}} \right) \tag{4.217}$$

Middle Strip along x-x direction

Interior Panel

The moment (M_{r2imx}) due to sustained load

$$M_{r2imx} = M_{operx} - M_{cmx} \tag{4.218}$$

Curvature due to sustained load for column strip (ψ_{2imx}) along the shorter span

$$\psi_{2imx} = LDF_c \times \left(\frac{M_{r2imx}}{E_c \times I_{mx}} \right) \tag{4.219}$$

Corner Panel

Curvature due to sustained load for column strip (ψ_{2cmx}) along the shorter span

$$\psi_{2cmx} = LDF_{1c} \times \left(\frac{M_{r2cmx}}{E_c \times I_{mx}} \right) \quad (4.220)$$

Side Panel

Curvature due to sustained load for column strip (ψ_{2smx}) along the shorter span

$$\psi_{2smx} = LDF_c \times \left(\frac{M_{r2smx}}{E_c \times I_{mx}} \right) \quad (4.221)$$

Middle Strip along y-y direction

Interior Panel

The moment (M_{r2imy}) due to sustained load

$$M_{r2imy} = M_{opery} - M_{cmy} \quad (4.222)$$

Curvature due to sustained load for middle strip (ψ_{2imy}) along the longer span

$$\psi_{2imy} = LDF_c \times \left(\frac{M_{r2imy}}{E_c \times I_{my}} \right) \quad (4.223)$$

Corner Panel

Curvature due to sustained load for middle strip (ψ_{2cmy}) along the longer span

$$\psi_{2cmy} = LDF_{1c} \times \left(\frac{M_{r2cmy}}{E_c \times I_{my}} \right) \quad (4.224)$$

Side Panel

Curvature due to sustained load for middle strip (ψ_{2smy}) along the longer span

$$\psi_{2smy} = LDF_c \times \left(\frac{M_{r2smy}}{E_c \times I_{my}} \right) \quad (4.225)$$

Curvature2x

Shorter direction

The curvature for sustained load (ψ_{i2x}) for interior panel

$$\psi_{i2x} = \psi_{2icx} + \psi_{2imy} \tag{4.226}$$

Corner Panel

The curvature for sustained load (ψ_{c2x}) for corner panel

$$\psi_{c2x} = \psi_{2ccx} + \psi_{2cmy} \tag{4.227}$$

Side Panel

The curvature for sustained load (ψ_{s2x}) for side panel

$$\psi_{s2x} = \psi_{2scx} + \psi_{2smy} \tag{4.228}$$

Curvature2y

Longer direction

Interior Panel

The curvature for sustained load (ψ_{i2y}) for interior panel

$$\psi_{i2y} = \psi_{2icy} + \psi_{2imx} \tag{4.229}$$

Corner Panel

The curvature for sustained load (ψ_{c2y}) for corner panel

$$\psi_{c2y} = \psi_{2ccy} + \psi_{2cmx} \tag{4.230}$$

Side Panel

The curvature for sustained load (ψ_{s2y}) for side panel

$$\psi_{s2y} = \psi_{2scy} + \psi_{2smx} \tag{4.231}$$

Long term deflection due to shrinkage along x-x direction

$$E_{cs} = 300 \times 10^{-6}$$

Column strip

Interior Panel

The first moment of area of the reinforcement (S_{sicx}) about the centroid of the cracked or gross section

$$S_{sicx} = A_{st1cx} \times (d - x_{1cx}) \tag{4.232}$$

The shrinkage curvature for the column strip (ψ_{3icx}) along shorter direction

$$\psi_{3icx} = LDF_c \times \left(\frac{E_{cs} \times m_{ce} \times S_{sicx}}{I_{cx}} \right) \tag{4.233}$$

Corner Panel

The shrinkage curvature for the column strip (ψ_{3ccx}) along shorter direction

$$\psi_{3ccx} = LDF_{1c} \times \left(\frac{E_{cs} \times m_{ce} \times S_{sicx}}{I_{cx}} \right) \tag{4.234}$$

Side Panel

The shrinkage curvature for the column strip (ψ_{3scx}) along shorter direction

$$\psi_{3scx} = LDF_c \times \left(\frac{E_{cs} \times m_{ce} \times S_{sicx}}{I_{cx}} \right) \tag{4.235}$$

Column strip along y-y direction

Interior Panel

The first moment of area of the reinforcement (S_{sicy}) about the centroid of the cracked or gross section

$$S_{sicy} = A_{st1cy} \times (d - x_{1cy}) \tag{4.236}$$

The shrinkage curvature for the column strip (ψ_{3icy}) along longer direction

$$\psi_{3icy} = LDF_c \times \left(\frac{E_{cs} \times m_{ce} \times S_{sicy}}{I_{cy}} \right) \tag{4.237}$$

Corner Panel

The shrinkage curvature for the column strip (ψ_{3ccy}) along longer direction

$$\psi_{3ccy} = LDF_{1c} \times \left(\frac{E_{cs} \times m_{ce} \times S_{sicy}}{I_{cy}} \right) \tag{4.238}$$

Side Panel

The shrinkage curvature for the column strip (ψ_{3scy}) along longer direction

$$\psi_{3scy} = LDF_{1c} \times \left(\frac{E_{cs} \times m_{ce} \times S_{sicy}}{I_{cy}} \right) \tag{4.239}$$

Middle strip along x-x direction

Interior Panel

The first moment of area of the reinforcement (S_{simx}) about the centroid of the cracked or gross section

$$S_{simx} = A_{st1mx} \times (d - x_{1mx}) \tag{4.240}$$

The shrinkage curvature for the middle strip (ψ_{3imx}) along longer direction

$$\psi_{3imx} = LDF_{m} \times \left(\frac{E_{cs} \times m_{ce} \times S_{simx}}{I_{mx}} \right) \tag{4.241}$$

Corner Panel

The shrinkage curvature for the middle strip (ψ_{3cmx}) along longer direction

$$\psi_{3cmx} = LDF_{1m} \times \left(\frac{E_{cs} \times m_{ce} \times S_{simx}}{I_{mx}} \right) \tag{4.242}$$

Side Panel

The shrinkage curvature for the middle strip (ψ_{3smx}) along longer direction

$$\psi_{3smx} = LDF_{m} \times \left(\frac{E_{cs} \times m_{ce} \times S_{simx}}{I_{mx}} \right) \tag{4.243}$$

Middle strip along y-y direction

Interior Panel

The first moment of area of the reinforcement (S_{simy}) about the centroid of the cracked or gross section

$$S_{simy} = A_{st1my} \times (d - x_{1my}) \tag{4.244}$$

The shrinkage curvature for the middle strip (ψ_{3imy}) along longer direction

$$\psi_{3imy} = LDF_m \times \left(\frac{E_{cs} \times m_{ce} \times S_{simy}}{I_{my}} \right) \tag{4.245}$$

Corner Panel

The shrinkage curvature for the middle strip (ψ_{3cmy}) along longer direction

$$\psi_{3cmy} = LDF_{1m} \times \left(\frac{E_{cs} \times m_{ce} \times S_{simy}}{I_{my}} \right) \tag{4.246}$$

Side Panel

The shrinkage curvature for the middle strip (ψ_{3smy}) along longer direction

$$\psi_{3smy} = LDF_{1m} \times \left(\frac{E_{cs} \times m_{ce} \times S_{simy}}{I_{my}} \right) \tag{4.247}$$

Curvature3x

Interior Panel

The shrinkage curvature for the interior panel (ψ_{i3x}) along shorter direction

$$\psi_{i3x} = \psi_{3icx} + \psi_{3imy} \tag{4.248}$$

Corner Panel

The shrinkage curvature for the corner panel (ψ_{c3x}) along shorter direction

$$\psi_{c3x} = \psi_{3ccx} + \psi_{3cmy} \tag{4.249}$$

Side Panel

The shrinkage curvature for the side panel (ψ_{s3x}) along shorter direction

$$\psi_{s3x} = \psi_{3scx} + \psi_{3smy} \tag{4.250}$$

Curvature3y

Interior Panel

The shrinkage curvature for the interior panel (ψ_{i3y}) along longer direction

$$\psi_{i3y} = \psi_{3icy} + \psi_{3imx} \tag{4.251}$$

Corner Panel

The shrinkage curvature for the corner panel (ψ_{c3y}) along longer direction

$$\psi_{c3y} = \psi_{3ccy} + \psi_{3cmx} \tag{4.252}$$

Side Panel

The shrinkage curvature for the side panel (ψ_{s3y}) along longer direction

$$\psi_{s3y} = \psi_{3scy} + \psi_{3smx} \tag{4.253}$$

Long term deflection due to creep

Column Strip along x-x direction

Interior Panel

The moment of resistance of concrete for the column strip (M_{r3icx}) due to creep along shorter direction

$$M_{r3icx} = M_{operx} - M_{c1cx} \tag{4.254}$$

The creep curvature of column strip (ψ_{4icx}) along shorter direction

$$\psi_{4icx} = LDF_c \times \left(\frac{M_{r3icx}}{E_{ce} \times I_{cx}} \right) \tag{4.255}$$

Corner Panel

The creep curvature of column strip (ψ_{4ccx}) along shorter direction

$$\psi_{4ccx} = LDF_{1c} \times \left(\frac{M_{r3icx}}{E_{ce} \times I_{cx}} \right) \tag{4.256}$$

Side Panel

The creep curvature of column strip (ψ_{4scx}) along shorter direction

$$\psi_{4scx} = LDF_c \times \left(\frac{M_{r3icx}}{E_{ce} \times I_{cx}} \right) \tag{4.257}$$

Column Strip along y-y direction

Interior Panel

The moment of resistance of concrete for the column strip (M_{r3icy}) due to creep along longer direction

$$M_{r3icy} = M_{opery} - M_{c1cy} \tag{4.258}$$

The creep curvature of column strip (ψ_{4icy}) along longer direction

$$\psi_{4icy} = LDF_c \times \left(\frac{M_{r3icy}}{E_{ce} \times I_{cy}} \right) \tag{4.259}$$

Corner Panel

The creep curvature of column strip (ψ_{4ccy}) along longer direction

$$\psi_{4ccy} = LDF_{1c} \times \left(\frac{M_{r3icy}}{E_{ce} \times I_{cy}} \right) \tag{4.260}$$

Side Panel

The creep curvature of column strip (ψ_{4scy}) along longer direction

$$\psi_{4scy} = LDF_c \times \left(\frac{M_{r3icy}}{E_{ce} \times I_{cy}} \right) \tag{4.261}$$

Middle strip along x-x direction

Interior Panel

The moment of resistance of concrete for the middle strip (M_{r3imx}) due to creep along shorter direction

$$M_{r3imx} = M_{operx} - M_{c1mx} \tag{4.262}$$

The creep curvature of middle strip (ψ_{4imx}) along shorter direction

$$\psi_{4imx} = LDF_c \times \left(\frac{M_{r3imx}}{E_{ce} \times I_{mx}} \right) \tag{4.263}$$

Corner Panel

The creep curvature of middle strip (ψ_{4cmx}) along shorter direction

$$\psi_{4cmx} = LDF_{1c} \times \left(\frac{M_{r3imx}}{E_{ce} \times I_{mx}} \right) \tag{4.264}$$

Side Panel

The creep curvature of middle strip (ψ_{4smx}) along shorter direction

$$\psi_{4smx} = LDF_c \times \left(\frac{M_{r3imx}}{E_{ce} \times I_{mx}} \right) \tag{4.265}$$

Middle strip along y-y direction

Interior Panel

The moment of resistance of concrete for the middle strip (M_{r3imy}) due to creep along longer direction

$$M_{r3imy} = M_{opery} - M_{c1my} \tag{4.266}$$

The creep curvature of middle strip (ψ_{4imy}) along longer direction

$$\psi_{4imy} = LDF_c \times \left(\frac{M_{r3imy}}{E_{ce} \times I_{my}} \right) \tag{4.267}$$

Corner Panel

The creep curvature of middle strip (ψ_{4cmy}) along longer direction

$$\psi_{4cmy} = LDF_{1c} \times \left(\frac{M_{r3imy}}{E_{ce} \times I_{my}} \right) \tag{4.268}$$

Side Panel

The creep curvature of middle strip (ψ_{4smy}) along longer direction

$$\psi_{4smy} = LDF_c \times \left(\frac{M_{r3imy}}{E_{ce} \times I_{my}} \right) \tag{4.269}$$

Curvature4x

Interior Panel

The creep curvature for the interior panel (ψ_{i4x}) along shorter direction

$$\psi_{i4x} = \psi_{4icx} + \psi_{4imy} \tag{4.270}$$

Corner Panel

The creep curvature for the corner panel (ψ_{c4x}) along shorter direction

$$\psi_{c4x} = \psi_{4ccx} + \psi_{4cmy} \tag{4.271}$$

Side Panel

The creep curvature for the side panel (ψ_{s4x}) along shorter direction

$$\psi_{s4x} = \psi_{4scx} + \psi_{4smy} \tag{4.272}$$

Curvature4y

Interior Panel

The creep curvature for the interior panel (ψ_{i4y}) along longer direction

$$\psi_{i4y} = \psi_{4icy} + \psi_{4imx} \tag{4.273}$$

Corner Panel

The creep curvature for the corner panel (ψ_{c4y}) along longer direction

$$\psi_{c4y} = \psi_{4ccy} + \psi_{4cmx} \tag{4.274}$$

Side Panel

The creep curvature for the side panel (ψ_{s4y}) along longer direction

$$\psi_{s4y} = \psi_{4scy} + \psi_{4smx} \tag{4.275}$$

Curvature x-x direction

Interior Panel

The total curvature for interior panel (ψ_{ix}) along shorter direction

$$\psi_{ix} = \psi_{i1x} - \psi_{i2x} + \psi_{i3x} + \psi_{i4x} \tag{4.276}$$

Corner Panel

The total curvature for corner panel (ψ_{cx}) along shorter direction

$$\psi_{cx} = \psi_{c1x} - \psi_{c2x} + \psi_{c3x} + \psi_{c4x} \tag{4.277}$$

Side Panel

The total curvature for side panel (ψ_{sx}) along shorter direction

$$\psi_{sx} = \psi_{s1x} - \psi_{s2x} + \psi_{s3x} + \psi_{s4x} \tag{4.278}$$

Curvature y-y

Interior Panel

The total curvature for interior panel (ψ_{iy}) along longer direction

$$\psi_{iy} = \psi_{i1y} - \psi_{i2y} + \psi_{i3y} + \psi_{i4y} \tag{4.279}$$

Corner Panel

The total curvature for corner panel (ψ_{cy}) along longer direction

$$\psi_{cy} = \psi_{c1y} - \psi_{c2y} + \psi_{c3y} + \psi_{c4y} \tag{4.280}$$

Side Panel

The total curvature for side panel (ψ_{sy}) along longer direction

$$\psi_{sy} = \psi_{s1y} - \psi_{s2y} + \psi_{s3y} + \psi_{s4y} \tag{4.281}$$

Total deflection

Interior Panel

The deflection of interior panel (δ_{ix}) along shorter direction

$$\delta_{ix} = \left(\psi_{ix} \times 0.104 \times \left(1 - 0.75/10\right) \times L_x^2\right) \tag{4.282}$$

The deflection of interior panel (δ_{iy}) along longer direction

$$\delta_{iy} = \left(\psi_{iy} \times 0.104 \times \left(1 - 0.75/10\right) \times L_y^2\right) \tag{4.283}$$

The deflection (δ_i) of interior panel

$$\delta_i = \frac{\delta_{ix} + \delta_{iy}}{2} \tag{4.284}$$

Corner Panel

The deflection of interior panel (δ_{cx}) along shorter direction

$$\delta_{cx} = \left(\psi_{cx} \times 0.104 \times \left(1 - 0.75/_{10}\right) \times L_x^2\right) \tag{4.285}$$

The deflection of interior panel (δ_{cy}) along longer direction

$$\delta_{cy} = \left(\psi_{cy} \times 0.104 \times \left(1 - 0.75/_{10}\right) \times L_y^2\right) \tag{4.286}$$

The deflection (δ_c) of corner panel

$$\delta_c = \left(\frac{\delta_{cx} + \delta_{cy}}{2}\right) + \left(\frac{(\delta_{t1x} - \delta_{d1x}) + (\delta_{t1y} - \delta_{d1y})}{2}\right) + \left(\frac{\delta_{d2x} + \delta_{d2y}}{2}\right) \tag{4.287}$$

Side Panel

The deflection of interior panel (δ_{sx}) along shorter direction

$$\delta_{sx} = \left(\psi_{sx} \times 0.104 \times \left(1 - 0.75/_{10}\right) \times L_x^2\right) \tag{4.288}$$

The deflection of interior panel (δ_{sx}) along longer direction

$$\delta_{sy} = \left(\psi_{sy} \times 0.104 \times \left(1 - 0.75/_{10}\right) \times L_y^2\right) \tag{4.289}$$

The deflection (δ_s) of side panel

$$\delta_s = \left(\frac{\delta_{sx} + \delta_{sy}}{2}\right) + \left(\frac{(\delta_{t1x} - \delta_{d1x}) + (\delta_{t1y} - \delta_{d1y})}{2}\right) + \left(\frac{\delta_{d2x} + \delta_{d2y}}{2}\right) \tag{4.290}$$

Short term deflection

Interior Panel

Short term deflection (δ_{ix}) along x-x direction

$$\delta_{ix} = \left((\psi_{i1x} - \psi_{i2x}) \times 0.104 \times \left(1 - 0.75/_{10}\right) \times L_x^2\right) \tag{4.291}$$

Short term deflection (δ_{iy}) along y-y direction

$$\delta_{iy} = \left(\left(\psi_{i1y} - \psi_{i2y}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_y^2\right) \tag{4.292}$$

Short term deflection (δ_{is})

$$\delta_{is} = \left(\frac{\delta_{ix} + \delta_{iy}}{2}\right) \tag{4.293}$$

Corner Panel

Short term deflection (δ_{cx}) along x-x direction

$$\delta_{cx} = \left(\left(\psi_{c1x} - \psi_{c2x}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_x^2\right) \tag{4.294}$$

Short term deflection (δ_{cy}) along y-y direction

$$\delta_{cy} = \left(\left(\psi_{c1y} - \psi_{c2y}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_y^2\right) \tag{4.295}$$

Short term deflection (δ_{cs})

$$\delta_{cs} = \left(\frac{\delta_{cx} + \delta_{cy}}{2}\right) \tag{4.296}$$

Side Panel

Short term deflection (δ_{sx}) along x-x direction

$$\delta_{sx} = \left(\left(\psi_{s1x} - \psi_{s2x}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_x^2\right) \tag{4.297}$$

Short term deflection (δ_{sy}) along y-y direction

$$\delta_{sy} = \left(\left(\psi_{s1y} - \psi_{s2y}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_y^2\right) \tag{4.298}$$

Short term deflection (δ_{ss})

$$\delta_{ss} = \left(\frac{\delta_{sx} + \delta_{sy}}{2}\right) \tag{4.299}$$

Shrinkage deflection

Interior Panel

Shrinkage deflection (δ_{shix}) along x-x direction

$$\delta_{shix} = \left(\left(\psi_{i3x}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_x^2\right) \tag{4.300}$$

Shrinkage deflection (δ_{shiy}) along y-y direction

$$\delta_{shiy} = \left(\left(\psi_{i3y}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_y^2\right) \tag{4.301}$$

Shrinkage deflection (δ_{ish})

$$\delta_{ish} = \left(\frac{\delta_{shix} + \delta_{shiy}}{2}\right) \tag{4.302}$$

Corner Panel

Shrinkage deflection (δ_{shcx}) along x-x direction

$$\delta_{shcx} = \left(\left(\psi_{c3x}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_x^2\right) \tag{4.303}$$

Shrinkage deflection (δ_{shcy}) along y-y direction

$$\delta_{shcy} = \left(\left(\psi_{c3y}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_y^2\right) \tag{4.304}$$

Shrinkage deflection (δ_{csh})

$$\delta_{csh} = \left(\frac{\delta_{shcx} + \delta_{shcy}}{2}\right) \tag{4.305}$$

Side Panel

Shrinkage deflection (δ_{shsx}) along x-x direction

$$\delta_{shsx} = \left(\left(\psi_{s3x}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_x^2\right) \tag{4.306}$$

Shrinkage deflection (δ_{shsy}) along y-y direction

$$\delta_{shsy} = \left(\left(\psi_{s3y}\right) \times 0.104 \times \left(1 - 0.75/10\right) \times L_y^2\right) \tag{4.307}$$

Shrinkage deflection (δ_{ssh})

$$\delta_{ssh} = \left(\frac{\delta_{shsx} + \delta_{shsy}}{2}\right) \tag{4.308}$$

Creep deflection

Interior Panel

Creep deflection (δ_{icx}) along x-x direction

$$\delta_{icx} = \left((\psi_{i4x}) \times 0.104 \times \left(1 - {0.75}/{10}\right) \times L_x^2\right) \tag{4.309}$$

Creep deflection (δ_{icy}) along y-y direction

$$\delta_{icy} = \left((\psi_{i4y}) \times 0.104 \times \left(1 - {0.75}/{10}\right) \times L_y^2\right) \tag{4.310}$$

Creep deflection (δ_{ic})

$$\delta_{ic} = \left(\frac{\delta_{icx} + \delta_{icy}}{2}\right) \tag{4.311}$$

Corner Panel

Creep deflection (δ_{ccx}) along x-x direction

$$\delta_{ccx} = \left((\psi_{c4x}) \times 0.104 \times \left(1 - {0.75}/{10}\right) \times L_x^2\right) \tag{4.312}$$

Creep deflection (δ_{ccy}) along y-y direction

$$\delta_{ccy} = \left((\psi_{c4y}) \times 0.104 \times \left(1 - {0.75}/{10}\right) \times L_y^2\right) \tag{4.313}$$

Creep deflection (δ_{cc})

$$\delta_{cc} = \left(\frac{\delta_{ccx} + \delta_{ccy}}{2}\right) \tag{4.314}$$

Side Panel

Creep deflection (δ_{scx}) along x-x direction

$$\delta_{scx} = \left((\psi_{s4x}) \times 0.104 \times \left(1 - {0.75}/{10}\right) \times L_x^2\right) \tag{4.315}$$

Creep deflection (δ_{scy}) along y-y direction

$$\delta_{scy} = \left((\psi_{s4y}) \times 0.104 \times \left(1 - {0.75}/{10}\right) \times L_y^2\right) \tag{4.316}$$

Creep deflection (δ_{sc})

$$\delta_{sc} = \left(\frac{\delta_{scx} + \delta_{scy}}{2} \right) \tag{4.317}$$

Percentage contribution

Interior Panel

The percentage contribution of short-term deflection in total deflection

$$\%\delta_{ishort} = \left(\frac{\delta_{is}}{\delta_i} \right) \times 100 \tag{4.318}$$

The percentage contribution of shrinkage deflection in total deflection

$$\%\delta_{ishr} = \left(\frac{\delta_{ish}}{\delta_i} \right) \times 100 \tag{4.319}$$

The percentage contribution of creep deflection in total deflection

$$\%\delta_{icr} = \left(\frac{\delta_{ic}}{\delta_i} \right) \times 100 \tag{4.320}$$

The percentage contribution of creep and shrinkage deflection in total deflection

$$\%\delta_{il} = \%\delta_{ishr} + \%\delta_{icr} \tag{4.321}$$

Corner Panel

The percentage contribution of short-term deflection in total deflection

$$\%\delta_{cshort} = \left(\frac{\delta_{cs}}{\delta_c} \right) \times 100 \tag{4.322}$$

The percentage contribution of shrinkage deflection in total deflection

$$\%\delta_{cshr} = \left(\frac{\delta_{csh}}{\delta_c} \right) \times 100 \tag{4.323}$$

The percentage contribution of creep deflection in total deflection

$$\%\delta_{ccr} = \left(\frac{\delta_{cc}}{\delta_c} \right) \times 100 \tag{4.324}$$

The percentage contribution of creep and shrinkage deflection in total deflection

$$\%\delta_{cl} = \%\delta_{cshr} + \%\delta_{ccr} \tag{4.325}$$

Side Panel

The percentage contribution of short-term deflection in total deflection

$$\%\delta_{sshort} = \left(\frac{\delta_{ss}}{\delta_{s}}\right) \times 100 \tag{4.326}$$

The percentage contribution of shrinkage deflection in total deflection

$$\%\delta_{sshr} = \left(\frac{\delta_{ssh}}{\delta_{s}}\right) \times 100 \tag{4.327}$$

The percentage contribution of creep deflection in total deflection

$$\%\delta_{scr} = \left(\frac{\delta_{sc}}{\delta_{s}}\right) \times 100 \tag{4.328}$$

The percentage contribution of creep and shrinkage deflection in total deflection

$$\%\delta_{sl} = \%\delta_{sshr} + \%\delta_{scr} \tag{4.329}$$

4.4 AS PER EC: 2-2002

This code gives detailed procedure for predicting the long term deflection due to creep and shrinkage. The tension stiffening effect of concrete is considered.

Moments

The sustained load

$$W_{d} = 25D + FF + 0.25Wl \tag{4.2}$$

The moment due to sustained load along shorter direction

$$M_{operx} = \frac{W_{d} \times 0.5 \times L_{y} \times L_{x}^{2}}{8} \tag{4.136}$$

The total load

$$TL = 25D + FF + Wl \tag{4.330}$$

The moment due to total load along shorter direction

$$M_{tlx} = \frac{L_y \times TL \times L_x^2}{8} \tag{4.331}$$

The moment due to sustained load along longer direction

$$M_{opery} = \frac{W_d \times 0.5 \times L_x \times L_y^2}{8} \tag{4.332}$$

The moment due to total load along longer direction

$$M_{tly} = \frac{L_x \times TL \times L_y^2}{8} \tag{4.333}$$

$$W_t = W_d + Wl \tag{4.3}$$

Short term deflection

The modular ratio

$$m = \frac{E_s}{E_c}$$

The modified modulus of elasticity of concrete

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

The modified modular ratio

$$m_{ce} = \frac{E_s}{E_{ce}}$$

The effective depth of tension reinforcement from the extreme fibre of slab

$$d = D - cc - (dia/2)$$

$$d_c = cc + (dia1/2)$$

$$LDF_c = 0.65$$

$$LDF_m = 0.35$$

$$LDF_{1c} = 0.75$$

$$LDF_{1m} = 0.25$$

Frame

x-x axis

$$b_{fx} = L_y$$

$$A_{st2cx} = \frac{0.5 \times b_{fx} \times D}{100} \tag{4.334}$$

Short term

$$A_{sc2cx} = \frac{0.5 \times 0.5 \times b_{fx} \times D}{100} \tag{4.335}$$

$$0.5 \times b_{fx} \times x_{2cxs}^2 + \left(m \times A_{st2cx} + (m-1) \times A_{sc2cx}\right) = \left(m \times A_{st2cx} \times d\right) + (m-1) \times A_{sc2cx} \times d \tag{4.336}$$

$$I_{fxs} = \frac{b_{fx} D^3}{12} + b_{fx} \times D \times \left(\frac{D}{2} - x_{2cxs}\right)^2 + (m-1) \times A_{sc2cx} \times \left(x_{2cxs} - d_c\right)^2 + (m-1) \times A_{st2cx} \times \left(d - x_{2cxs}\right)^2 \tag{4.337}$$

Long term

$$0.5 \times b_{fx} \times x_{2cxl}^2 + \left(m_{ce} \times A_{st2cx} + (m_{ce}-1) \times A_{sc2cx}\right) = \left(m_{ce} \times A_{st2cx} \times d\right) + (m_{ce}-1) \times A_{sc2cx} \times d \tag{4.338}$$

$$I_{fxl} = \frac{b_{fx} x_{2cxl}^3}{3} + b_{fx} \times D \times \left(\frac{D}{2} - x_{2cxl}\right)^2 + (m_{ce}-1) \times A_{sc2cx} \times \left(x_{2cxl} - d_c\right)^2 + (m_{ce}-1) \times A_{st2cx} \times \left(d - x_{2cxl}\right)^2 \tag{4.339}$$

y-y axis

$$b_{fy} = L_x$$

$$A_{st2cy} = \frac{0.5 \times b_{fy} \times D}{100} \tag{4.340}$$

Short term

$$A_{sc2cy} = \frac{0.5 \times 0.5 \times b_{fy} \times D}{100} \tag{4.341}$$

$$0.5 \times b_{fy} \times x_{2cys}^2 + \left(m \times A_{st2cy} + (m-1) \times A_{sc2cy}\right) = \left(m \times A_{st2cy} \times d\right) + (m-1) \times A_{sc2cy} \times d \tag{4.342}$$

$$I_{fys} = \frac{b_{fy}D^3}{12} + b_{fy} \times D \times \left(\frac{D}{2} - x_{2cys}\right)^2 + (m-1) \times A_{sc2cy} \times \left(x_{2cys} - d_c\right)^2 + (m-1) \times A_{st2cy} \times \left(d - x_{2cys}\right)^2 \quad (4.343)$$

Long term

$$0.5 \times b_{fy} \times x_{2cyl}^2 + \left(m_{ce} \times A_{st2cy} + (m_{ce} - 1) \times A_{sc2cy}\right) = \left(m_{ce} \times A_{st2cy} \times d\right) + (m_{ce} - 1) \times A_{sc2cy} \times d \quad (4.344)$$

$$I_{fyl} = \frac{b_{fy}x_{2cyl}^3}{3} + b_{fy} \times D \times \left(\frac{D}{2} - x_{2cyl}\right)^2 + (m_{ce} - 1) \times A_{sc2cy} \times \left(x_{2cyl} - d_c\right)^2 + (m_{ce} - 1) \times A_{st2cy} \times \left(d - x_{2cyl}\right)^2 \quad (4.345)$$

$$df_{dx} = \frac{W_d \times L_y \times L_x^4}{384 \times E_c \times I_{fxs}} \quad (4.346)$$

$$df_{tx} = \frac{TL \times L_y \times L_x^4}{384 \times E_c \times I_{fxs}} \quad (4.347)$$

$$df_{dy} = \frac{W_d \times L_x \times L_y^4}{384 \times E_c \times I_{fys}} \quad (4.348)$$

$$df_{ty} = \frac{TL \times L_x \times L_y^4}{384 \times E_c \times I_{fys}} \quad (4.349)$$

Column Strip

x-x axis

The width of column strip along shorter direction

$$b_{cx} = 0.5 \times L_y \quad (4.14)$$

The area of tension reinforcement provided in the column strip along shorter direction

$$A_{st1cx} = \frac{0.5 \times b_{cx} \times D}{100} \quad (4.350)$$

The area of compression reinforcement provided in the column strip along shorter direction

$$A_{sc1cx} = \frac{0.5 \times 0.5 \times b_{cx} \times D}{100} \quad (4.351)$$

Short term

$$0.5\times b_{cx}\times x_{1cxs}^2+\left(m\times A_{st1cx}+\left(m-1\right)\times A_{sc1cx}\right)$$
$$=\left(m\times A_{st1cx}\times d\right)+\left(m-1\right)\times A_{sc1cx}\times d \tag{4.352}$$

The moment of inertia of column strip along shorter direction for short term loads

$$I_{cxs}=\frac{b_{cx}D^3}{12}+b_{cx}\times D\times\left(\frac{D}{2}-x_{1cxs}\right)^2+\left(m-1\right)\times A_{sc1cx}\times\left(x_{1cxs}-d_c\right)^2$$
$$+\left(m-1\right)\times A_{st1cx}\times\left(d-x_{1cxs}\right)^2 \tag{4.353}$$

Long term

$$0.5\times b_{cx}\times x_{1cxl}^2+\left(m_{ce}\times A_{st1cx}+\left(m_{ce}-1\right)\times A_{sc1cx}\right)$$
$$=\left(m_{ce}\times A_{st1cx}\times d\right)+\left(m_{ce}-1\right)\times A_{sc1cx}\times d \tag{4.354}$$

The moment of inertia of column strip along shorter direction for long term loads

$$I_{cxl}=\frac{b_{cx}x_{1cxl}^3}{3}+b_{cx}\times x_{1cxl}\times\left(\frac{D}{2}-x_{1cxl}\right)^2+\left(m_{ce}-1\right)\times A_{sc1cx}\times\left(x_{1cxl}-d_c\right)^2$$
$$+\left(m_{ce}-1\right)\times A_{st1cx}\times\left(d-x_{1cxl}\right)^2 \tag{4.355}$$

y-y axis

The width of column strip along longer direction

$$b_{cy}=0.5\times L_x \tag{4.14}$$

The area of tension reinforcement provided in the column strip along longer direction

$$A_{st1cy}=\frac{0.5\times b_{cy}\ \times D}{100} \tag{4.356}$$

The area of compression reinforcement provided in the column strip along longer direction

$$A_{sc1cy}=\frac{0.5\times 0.5\times b_{cy}\ \times D}{100} \tag{4.357}$$

Short term

$$0.5\times b_{cx}\times x_{1cys}^2+\left(m\times A_{st1cy}+\left(m-1\right)\times A_{sc1cy}\right)$$
$$=\left(m\times A_{st1cy}\times d\right)+\left(m-1\right)\times A_{sc1cy}\times d \tag{4.358}$$

The moment of inertia of column strip along longer direction for short term loads

$$I_{cys} = \frac{b_{cy}D^3}{12} + b_{cy} \times D \times \left(\frac{D}{2} - x_{1cys}\right)^2 + (m-1) \times A_{sc1cy} \times \left(x_{1cys} - d_c\right)^{22}$$
$$+(m-1) \times A_{st1cy} \times \left(d - x_{1cys}\right) \quad (4.359)$$

Long term

$$0.5 \times b_{cy} \times x_{1cyl}^2 + \left(m_{ce} \times A_{st1cy}\left(m_{ce} - 1\right) \times A_{sc1cy}\right)$$
$$= \left(m_{ce} \times A_{st1cy} \times d\right) + \left(m_{ce} - 1\right) \times A_{sc1cy} \times d \quad (4.360)$$

The moment of inertia of column strip along longer direction for long term loads

$$I_{cyl} = \frac{b_{cy}x_{1cyl}^3}{3} + b_{cy} \times x_{1cyl} \times \left(\frac{D}{2} - x_{1cyl}\right)^2 + \left(m_{ce} - 1\right) \times A_{sc1cy} \times \left(x_{1cyl} - d_c\right)^2$$
$$+\left(m_{ce} - 1\right) \times A_{st1cy} \times \left(d - x_{1cyl}\right)^2 \quad (4.361)$$

Middle Strip

The width of middle strip along shorter direction

$$b_{mx} = \left(L_y - {}^{L_x}\!/_{2}\right) \quad (4.17)$$

The area of tension reinforcement provided in the middle strip along shorter direction

$$A_{st1mx} = \frac{0.5 \times b_{mx} \times D}{100} \quad (4.147)$$

$$0.5 \times b_{cx} \times x_{1mx}^2 + \left(m \times A_{st1mx}\right) = \left(m \times A_{st1mx} \times d\right) \quad (4.362)$$

The moment of inertia of middle strip along shorter direction for short term loads

$$I_{mx} = \frac{b_{mx}D^3}{12} + b_{mx} \times D \times \left(\frac{D}{2} - x_{mx}\right)^2 + (m-1) \times A_{st1mx} \times \left(d - x_{mx}\right)^2 \quad (4.363)$$

The width of middle strip along longer direction

$$b_{my} = \frac{L_x}{2} \quad (4.364)$$

The area of tension reinforcement provided in the middle strip along longer direction

$$A_{st1my} = \frac{0.5 \times b_{my} \times D}{100} \quad (4.148)$$

$$0.5\times b_{cx}\times x_{1my}^2+\left(m\times A_{st1my}\right)=\left(m\times A_{st1my}\times d\right) \tag{4.365}$$

The moment of inertia of middle strip along longer direction

$$I_{my}=\frac{b_{my}D^3}{12}+b_{my}\times D\times\left(\frac{D}{2}-x_{my}\right)^2+(m-1)\times A_{st1my}\times\left(d-x_{my}\right)^2 \tag{4.366}$$

Flexural stiffness of equivalent column

Factored dead load

$$W_D=1.35\times\left(D\times 25+FF\right) \tag{4.367}$$

Factored live load

$$W_L=1.5\times Wl \tag{4.368}$$

Total load

$$W_T=W_D+W_L \tag{4.23}$$

Uniformly distributed load

$$w=W_T\times L_y \tag{4.24}$$

Area of column required

$$A_c=\left(\frac{2\times 3\times W_T\times L_x\times {}^{L_y}\!/_2}{0.85\times\sqrt{f_{ck}}}\right) \tag{4.25}$$

$$d=D-cc-{}^{dial}\!/_2$$

Edge column size

$$C_{1x}=\frac{((A_c/d)-2d)}{3} \tag{4.26}$$

$$C_{1y}=\frac{((A_c/d)-2d)}{3} \tag{4.27}$$

Short term deflection along x-x axis

$$I_{c1x}=\frac{C_{1x}C_{1y}^3}{12} \tag{4.28}$$

$$K_{cx} = \frac{4E_c I_{c1x}}{L_c} \tag{4.29}$$

$$C_x = \frac{(1-(0.63D/C_{1x}))(D^3 C_{1x})}{3} \tag{4.30}$$

$$L_{nx} = L_x - C_{1x}$$

$$K_{tx} = \frac{2 \times 4.5 \times E_c \times C_x}{\left(L_y \left(1 - \left(\frac{C_{1y}}{L_y} \right) \right)^3 \right)} \tag{4.31}$$

$$K_{s1x} = \frac{4E_c I_{fxs}}{L_x} \tag{4.369}$$

$$K_{ecx} = \left(\frac{1}{2 \times K_{cx} + K_{tx}} \right) \tag{4.32}$$

$$\alpha_{1x} = \frac{K_{ecx}}{2K_{s1x}} \tag{4.153}$$

Distribution factor for positive moment in exterior span

$$D_{px} = 0.63 - \frac{0.28}{\left(1 + \frac{1}{\alpha_{1x}} \right)} \tag{4.59}$$

Distribution factor for negative moment in exterior span

$$D_{nex} = \frac{0.65}{\left(1 + \frac{1}{\alpha_{1x}} \right)} \tag{4.60}$$

Distribution factor for negative moment in interior span

$$D_{nix} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{\alpha_{1x}} \right)} \tag{4.61}$$

The cracked moment for column strip along shorter direction

$$M_{crcx} = \frac{f_{etm} \times b_{cx} \times D^2}{6} \tag{4.370}$$

$$M_{at1px} = D_p \times M_{t1x} \tag{4.371}$$

If the M_{at1px} is less than the M_{cr} then the value of later is considered.

$$M_{at1nex} = D_{ne} \times M_{t1x} \tag{4.372}$$

If the M_{at1nex} is less than the M_{cr} then the value of later is considered.

$$M_{at1nix} = D_{ni} \times M_{t1x} \tag{4.373}$$

If the M_{at1nix} is less than the M_{cr} then the value of later is considered.

Effective moment of inertia for positive cracked moment in column strip

$$I_{ecpx} = \left(\frac{M_{crcx}}{M_{at1px}}\right)^3 (I_{cxs}) + \left(1 - \left(\frac{M_{crcx}}{M_{at1px}}\right)^3\right)(I_{cxl}) \tag{4.374}$$

Effective moment of inertia for negative cracked moment in column strip in exterior span

$$I_{ecnex} = \left(\frac{M_{crcx}}{M_{at1nex}}\right)^3 (I_{cxs}) + \left(1 - \left(\frac{M_{crcx}}{M_{at1nex}}\right)^3\right)(I_{cxl}) \tag{4.375}$$

Effective moment of inertia for negative cracked moment in column strip in interior span

$$I_{ecnix} = \left(\frac{M_{crcx}}{M_{at1nix}}\right)^3 (I_{cxs}) + \left(1 - \left(\frac{M_{crcx}}{M_{at1nix}}\right)^3\right)(I_{cxl}) \tag{4.376}$$

Effective moment of inertia for column strip

$$I_{ecx} = 0.7 I_{ecpx} + 0.15 I_{ecnex} + 0.15 I_{ecnix} \tag{4.377}$$

Effective moment of inertia for frame

$$I_{efx} = I_{ecx} + I_{mx} \tag{4.378}$$

The moment due to total load

$$M_{ox} = \frac{W_T L_y (L_{nx})^2}{8} \tag{4.379}$$

Moment due to total load

$$M_{nt1x} = \frac{0.65 \times M_{ox}}{\left(1 + \frac{1}{\alpha_{1x}}\right)} \tag{4.380}$$

Moment due to sustained load

$$M_{nd1x} = \frac{W_d \times M_{nt1x}}{TL} \tag{4.381}$$

The rotation due to total load

$$\theta_{t1x} = \frac{M_{nt1x}}{K_{ecx}} \tag{4.382}$$

The rotation due to sustained load

$$\theta_{d1x} = \frac{M_{nd1x}}{K_{ecx}} \tag{4.383}$$

The deflection due to total load

$$\delta_{t1x} = \left(\theta_{t1x} \times L_x/8\right)\left(\frac{I_{fxs}}{I_{efx}}\right) \tag{4.384}$$

The deflection due to sustained load

$$\delta_{d1x} = \frac{\theta_{d1x} L_x}{8} \tag{4.385}$$

Long term

The relative stiffness for the long term effects

$$K_{s2x} = \frac{4E_c I_{fxl}}{L_x} \tag{4.386}$$

$$\alpha_{2x} = \frac{K_{ecx}}{2K_{s2x}} \tag{4.387}$$

The moment due to total load

$$M_{nt2x} = \frac{0.65 \times M_{ox}}{\left(1 + 1/\alpha_{2x}\right)} \tag{4.388}$$

The moment due to sustained load

$$M_{nd2x} = \frac{W_d \times M_{nt2x}}{TL} \tag{4.389}$$

The rotation due to sustained load

$$\theta_{d2x} = \frac{M_{nd2x}}{K_{ecx}} \tag{4.390}$$

The deflection due to sustained load

$$\delta_{d2x} = \frac{\theta_{d2x} L_x}{8} \tag{4.391}$$

Short term deflection along y-y axis

The moment of inertia of column

$$I_{c1y} = \frac{C_{1y} C_{1x}^3}{12} \tag{4.392}$$

The relative stiffness of column

$$K_{cy} = \frac{4 E_c I_{c1y}}{L_c} \tag{4.393}$$

$$C_y = \frac{\left(1 - \left(0.63 D / C_{1y}\right)\right)\left(D^3 C_{1y}\right)}{3} \tag{4.394}$$

Clear span

$$L_{ny} = L_y - C_{1y}$$

$$K_{ty} = \frac{2 \times 4.5 \times E_c \times C_y}{\left(L_x \left(1 - \left(\frac{C_{1x}}{L_x}\right)\right)^3\right)} \tag{4.395}$$

$$K_{s1y} = \frac{4 E_c I_{fys}}{L_y} \tag{4.396}$$

$$K_{ecy} = \left(\frac{1}{2 \times K_{cy} + K_{ty}}\right)$$

$$\alpha_{1y} = \frac{K_{ecy}}{2 K_{s1y}} \tag{4.397}$$

Distribution factor for positive moment in exterior span

$$D_{py} = 0.63 - \frac{0.28}{\left(1 + \frac{1}{\alpha_{1y}}\right)} \tag{4.398}$$

Distribution factor for negative moment in exterior span

$$D_{ney} = \frac{0.65}{\left(1 + \frac{1}{\alpha_{1y}}\right)} \tag{4.399}$$

Distribution factor for negative moment in interior span

$$D_{niy} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{\alpha_{1y}}\right)} \tag{4.400}$$

$$M_{crcy} = \frac{f_{etm} \times b_{cy} \times D^2}{6} \tag{4.401}$$

$$M_{at1py} = D_p \times M_{t1y} \tag{4.402}$$

If the M_{at1py} is less than the M_{cr} then the value of later is considered.

$$M_{at1ney} = D_{ne} \times M_{t1y} \tag{4.403}$$

If the M_{at1ney} is less than the M_{cr} then the value of later is considered.

$$M_{at1niy} = D_{ni} M_{t1y} \tag{4.404}$$

If the M_{at1niy} is less than the M_{cr} then the value of later is considered.

Effective moment of inertia for positive cracked moment in column strip

$$I_{ecpy} = \left(\frac{M_{crcy}}{M_{at1py}}\right)^3 (I_{cys}) + \left(1 - \left(\frac{M_{crcy}}{M_{at1py}}\right)^3\right)(I_{cyl}) \tag{4.405}$$

Effective moment of inertia for negative cracked moment in column strip in exterior span

$$I_{ecney} = \left(\frac{M_{crcy}}{M_{at1ney}}\right)^3 (I_{cys}) + \left(1 - \left(\frac{M_{crcy}}{M_{at1ney}}\right)^3\right)(I_{cyl}) \tag{4.406}$$

Effective moment of inertia for negative cracked moment in column strip in interior span

$$I_{ecniy} = \left(\frac{M_{crcy}}{M_{at1niy}}\right)^3 (I_{cys}) + \left(1 - \left(\frac{M_{crcy}}{M_{at1niy}}\right)^3\right)(I_{cyl}) \tag{4.407}$$

Effective moment of inertia for column strip

$$I_{ecy} = 0.7 I_{ecpy} + 0.15 I_{ecney} + 0.15 I_{ecniy} \tag{4.408}$$

Effective moment of inertia for frame

$$I_{efy} = I_{ecy} + I_{my} \tag{4.409}$$

The moment due to total load

$$M_{oy} = \frac{W_T L_x (L_{ny})^2}{8} \tag{4.410}$$

Moment due to total load

$$M_{nt1y} = \frac{0.65 \times M_{oy}}{\left(1 + \frac{1}{\alpha_{1y}}\right)} \tag{4.411}$$

The moment due to total load

$$M_{nd1y} = \frac{W_d \times M_{nt1y}}{TL} \tag{4.412}$$

The rotation due to total load

$$\theta_{t1y} = \frac{M_{nt1y}}{K_{ecy}} \tag{4.413}$$

The rotation due to sustained load

$$\theta_{d1y} = \frac{M_{nd1y}}{K_{ecy}} \tag{4.414}$$

The deflection due to total load

$$\delta_{t1y} = \left(\theta_{t1y} \times \frac{L_y}{8}\right)\left(\frac{I_{fys}}{I_{efy}}\right) \tag{4.415}$$

The deflection due to sustained load

$$\delta_{d1y} = \frac{\theta_{d1y} L_y}{8} \tag{4.416}$$

Long term deflection

$$K_{s2y} = \frac{4 E_c I_{fyl}}{L_y} \tag{4.417}$$

$$\alpha_{2y} = \frac{K_{ecy}}{2K_{s2y}} \tag{4.418}$$

The moment due to total load

$$M_{nt2y} = \frac{0.65 \times M_{oy}}{\left(1 + \frac{1}{\alpha_{2y}}\right)} \tag{4.419}$$

The moment due to sustained load

$$M_{nd2y} = \frac{W_d \times M_{nt2y}}{TL} \tag{4.420}$$

The rotation due to sustained load

$$\theta_{d2y} = \frac{M_{nd2y}}{K_{ecy}} \tag{4.421}$$

The deflection due to sustained load

$$\delta_{d2y} = \frac{\theta_{d2y} L_y}{8} \tag{4.422}$$

Short term deflection along x-x direction

Column strip

Dead load

Interior Panel

Short term deflection δ_{scidx} due to sustained load for column strip

$$\delta_{scidx} = \left(\frac{LDF_c \times \delta_{fdx} \times I_{fxs}}{I_{cxs}}\right) \tag{4.423}$$

Corner Panel

Short term deflection δ_{sccdx} due to sustained load for column strip

$$\delta_{sccdx} = \left(\frac{LDF_{1c} \times \delta_{fdx} \times I_{fxs}}{I_{cxs}}\right) \tag{4.424}$$

Side Panel

Short term deflection δ_{scsdx} due to sustained load for column strip

$$\delta_{scsdx} = \left(\frac{LDF_{1c} \times \delta_{fdx} \times I_{fxs}}{I_{cxs}} \right) \tag{4.425}$$

Total load

Interior Panel

Short term deflection δ_{scitx} due to total load for column strip

$$\delta_{scitx} = \left(\frac{LDF_{c} \times \delta_{ftx} \times I_{fxs}}{I_{cxs}} \right) \tag{4.426}$$

Corner Panel

Short term deflection δ_{scctx} due to total load for column strip

$$\delta_{scctx} = \left(\frac{LDF_{1c} \times \delta_{ftx} \times I_{fxs}}{I_{cxs}} \right) \tag{4.427}$$

Side Panel

Short term deflection δ_{scstx} due to total load for column strip

$$\delta_{scstx} = \left(\frac{LDF_{1c} \times \delta_{ftx} \times I_{fxs}}{I_{cxs}} \right) \tag{4.428}$$

Dead load

y-y axis

Interior Panel

Short term deflection δ_{scidy} due to total load for column strip

$$\delta_{scidy} = \left(\frac{LDF_{c} \times \delta_{fdy} \times I_{fys}}{I_{cys}} \right) \tag{4.429}$$

Corner Panel

Short term deflection δ_{sccdy} due to total load for column strip

$$\delta_{sccdy} = \left(\frac{LDF_{1c} \times \delta_{fdy} \times I_{fys}}{I_{cys}} \right) \tag{4.430}$$

Side Panel

Short term deflection δ_{scsdy} due to total load for column strip

$$\delta_{scsdy} = \left(\frac{LDF_{1c} \times \delta_{fdy} \times I_{fys}}{I_{cys}} \right) \tag{4.431}$$

Total load

Interior Panel

Longer direction

Short term deflection δ_{scity} due to total load for column strip

$$\delta_{scity} = \left(\frac{LDF_{c} \times \delta_{fty} \times I_{fys}}{I_{cys}} \right) \tag{4.432}$$

Corner Panel

Short term deflection δ_{sccty} due to total load for column strip

$$\delta_{sccty} = \left(\frac{LDF_{1c} \times \delta_{fty} \times I_{fys}}{I_{cys}} \right) \tag{4.433}$$

Side Panel

Short term deflection δ_{scsty} due to total load for column strip

$$\delta_{scsty} = \left(\frac{LDF_{1c} \times \delta_{fty} \times I_{fys}}{I_{cys}} \right) \tag{4.434}$$

Middle strip along x-x axis

Dead load

Interior Panel

Short term deflection δ_{smidx} due to sustained load for middle strip

$$\delta_{smidx} = \left(\frac{LDF_{m} \times \delta_{fdx} \times I_{fxs}}{I_{mx}} \right) \tag{4.435}$$

Corner Panel

Short term deflection δ_{smcdx} due to sustained load for middle strip

$$\delta_{smcdx} = \left(\frac{LDF_{1m} \times \delta_{fdx} \times I_{fxs}}{I_{mx}} \right) \tag{4.436}$$

Side Panel

Short term deflection δ_{smsdx} due to sustained load for middle strip

$$\delta_{smsdx} = \left(\frac{LDF_{1c} \times \delta_{fdx} \times I_{fxs}}{I_{mx}} \right) \tag{4.437}$$

Total load

Interior Panel

Short term deflection δ_{smitx} due to total load for middle strip

$$\delta_{smitx} = \left(\frac{LDF_{c} \times \delta_{ftx} \times I_{fxs}}{I_{mx}} \right) \tag{4.438}$$

Corner Panel

Short term deflection δ_{smctx} due to total load for middle strip

$$\delta_{smctx} = \left(\frac{LDF_{1c} \times \delta_{ftx} \times I_{fxs}}{I_{mx}} \right) \tag{4.439}$$

Side Panel

Short term deflection δ_{smstx} due to total load for middle strip

$$\delta_{smstx} = \left(\frac{LDF_{1c} \times \delta_{ftx} \times I_{fxs}}{I_{mx}} \right) \tag{4.440}$$

y-y axis

Dead load

Interior Panel

Short term deflection δ_{smidy} due to sustained load for middle strip

$$\delta_{smidy} = \left(\frac{LDF_{c} \times \delta_{fdy} \times I_{fys}}{I_{my}} \right) \tag{4.441}$$

Corner Panel

Short term deflection δ_{smcdy} due to sustained load for middle strip

$$\delta_{smcdy} = \left(\frac{LDF_{1c} \times \delta_{fdy} \times I_{fys}}{I_{my}} \right) \tag{4.442}$$

Side Panel

Short term deflection δ_{smsdy} due to sustained load for middle strip

$$\delta_{smsdy} = \left(\frac{LDF_{1c} \times \delta_{fdy} \times I_{fys}}{I_{my}} \right) \tag{4.443}$$

Total load

Interior Panel

Short term deflection δ_{smity} due to total load for middle strip

$$\delta_{smity} = \left(\frac{LDF_{c} \times \delta_{fty} \times I_{fys}}{I_{my}} \right) \tag{4.444}$$

Corner Panel

Short term deflection δ_{smcty} due to total load for middle strip

$$\delta_{smcty} = \left(\frac{LDF_{1c} \times \delta_{fty} \times I_{fys}}{I_{my}} \right) \tag{4.445}$$

Side Panel

Short term deflection δ_{smsty} due to total load for middle strip

$$\delta_{smsty} = \left(\frac{LDF_{1c} \times \delta_{fty} \times I_{fys}}{I_{my}} \right) \tag{4.446}$$

Elastic deflection due to varying part of live load

Column strip

Interior Panel

Short term deflection of column strip δ_{sciix} due to varying part of load along shorter direction

$$\delta_{sciix} = \delta_{scitx} - \delta_{scidx} \tag{4.447}$$

Short term deflection of column strip δ_{sciy} due to varying part of load along longer direction

$$\delta_{sciy} = \delta_{scity} - \delta_{scidy} \tag{4.448}$$

Corner Panel

Short term deflection of column strip δ_{sccx} due to varying part of load along shorter direction

$$\delta_{sccx} = \delta_{scctx} - \delta_{sccdx} \tag{4.449}$$

Short term deflection of column strip δ_{sccy} due to varying part of load along longer direction

$$\delta_{sccy} = \delta_{sccty} - \delta_{sccdy} \tag{4.450}$$

Side Panel

Short term deflection of column strip δ_{sccx} due to varying part of load along shorter direction

$$\delta_{scsx} = \delta_{scstx} - \delta_{scsdx} \tag{4.451}$$

Short term deflection of column strip δ_{scsy} due to varying part of load along longer direction

$$\delta_{scsy} = \delta_{scsty} - \delta_{scsdy} \tag{4.452}$$

Middle strip

Interior Panel

Short term deflection of middle strip δ_{smix} due to varying part of load along shorter direction

$$\delta_{smix} = \delta_{smitx} - \delta_{smidx} \tag{4.453}$$

Short term deflection of middle strip δ_{smiy} due to varying part of load along longer direction

$$\delta_{smiy} = \delta_{smity} - \delta_{smidy} \tag{4.454}$$

Corner Panel

Short term deflection of middle strip δ_{smcx} due to varying part of load along shorter direction

$$\delta_{smcx} = \delta_{smctx} - \delta_{smcdx} \tag{4.455}$$

Short term deflection of middle strip δ_{smcy} due to varying part of load along longer direction

$$\delta_{smcy} = \delta_{smcty} - \delta_{smcdy} \tag{4.456}$$

Side Panel

Short term deflection of middle strip δ_{smsx} due to varying part of load along shorter direction

$$\delta_{smsx} = \delta_{smstx} - \delta_{smsdx} \tag{4.457}$$

Short term deflection of middle strip δ_{smsy} due to varying part of load along longer direction

$$\delta_{smsy} = \delta_{smsty} - \delta_{smsdy} \tag{4.458}$$

Short term deflection

Interior Panel

Short term deflection δ_{is} of interior panel

$$\delta_{is} = \left(\frac{(\delta_{scix} + \delta_{smiy}) + (\delta_{sciy} + \delta_{smix})}{2} \right) \tag{4.459}$$

Corner Panel

Short term deflection δ_{cs} of corner panel

$$\delta_{cs} = \left(\frac{(\delta_{sccx} + \delta_{smcy}) + (\delta_{sccy} + \delta_{smcx})}{2} \right) + \left(\frac{(\delta_{t1x} - \delta_{d1x}) + (\delta_{t1y} - \delta_{d1y})}{2} \right) \tag{4.460}$$

Side Panel

Short term deflection δ_{ss} of side panel

$$\delta_{ss} = \left(\frac{(\delta_{scsx} + \delta_{smsy}) + (\delta_{scsy} + \delta_{smsx})}{2} \right) + \left(\frac{(\delta_{t1y} - \delta_{d1y})}{2} \right) \tag{4.461}$$

Long term deflection due to creep

Cracking moment

Column strip

Cracked moment of column strip M_{crcx} along shorter direction

$$M_{crcx} = \frac{f_{etm} \times b_{cx} \times D^2}{6} \tag{4.462}$$

Cracked moment of column strip M_{crcy} along longer direction

$$M_{crcy} = \frac{f_{etm} \times b_{cy} \times D^2}{6} \tag{4.463}$$

Long term curvature of column strip ψ_{4cxl} along shorter direction

$$\psi_{4cxl} = \frac{M_{operx}}{E_{ce} \times I_{cxl}} \tag{4.464}$$

Long-term curvature of column strip ψ_{4cyl} along longer direction

$$\psi_{4cyl} = \frac{M_{opery}}{E_{ce} \times I_{cyl}} \tag{4.465}$$

Middle strip

Cracked moment of middle strip M_{crmx} along shorter direction (x-x)

$$M_{crmx} = \frac{f_{etm} \times b_{mx} \times D^2}{6} \tag{4.466}$$

Cracked moment of middle strip M_{crmy} along longer direction (y-y)

$$M_{crmy} = \frac{f_{etm} \times b_{my} \times D^2}{6} \tag{4.467}$$

Long-term curvature of middle strip ψ_{4mxl} along shorter direction

$$\psi_{4mxl} = \frac{M_{operx}}{E_{ce} \times I_{mx}} \tag{4.468}$$

Long-term curvature of middle strip ψ_{4myl} along longer direction

$$\psi_{4myl} = \frac{M_{opery}}{E_{ce} \times I_{my}} \tag{4.469}$$

Steel stress

Column strip

The steel stress of the column strip along the shorter direction S_{tcx} is calculated in Eq. 4.470. Here the neutral axis in cracked stage is assumed as $1.2x_{cxs}$ [Branson], 20% shift downwards from the uncracked stage.

$$S_{tcx} = \frac{M_{operx}}{\left(A_{st1cx} \times \left(\frac{d - (1.2 \times x_{cxs})}{3}\right)\right)} \tag{4.470}$$

The steel stress of the column strip along the longer direction S_{tcy} is calculated in Eq. 4.471. Same assumption as above is made for neutral axis.

$$S_{tcy} = \frac{M_{opery}}{\left(A_{st1cy} \times \left(\frac{d - (1.2 \times x_{cys})}{3}\right)\right)} \tag{4.471}$$

Middle strip

The steel stress of middle strip S_{tmx} along shorter direction

$$S_{tmx} = \frac{M_{operx}}{\left(A_{st1mx} \times \left(\frac{d-(1.2 \times x_{mxs})}{3}\right)\right)} \tag{4.472}$$

The steel stress of middle strip S_{tmy} along longer direction

$$S_{tmy} = \frac{M_{opery}}{\left(A_{st1my} \times \left(\frac{d-(1.2 \times x_{mys})}{3}\right)\right)} \tag{4.473}$$

For the cracked section

Column strip

Cracked moment of inertia for column strip I_{c1x} along shorter direction

$$I_{c1x} = \frac{(I_{cxs} + I_{cxl})}{2} \tag{4.474}$$

Cracked moment of inertia for column strip I_{c1y} along longer direction

$$I_{c1y} = \frac{(I_{cys} + I_{cyl})}{2} \tag{4.475}$$

The curvature of column strip along ψ_{4cx2} shorter direction

$$\psi_{4cx2} = \frac{M_{operx}}{E_{ce} \times I_{c1x}} \tag{4.476}$$

The curvature of column strip along ψ_{4cy2} longer direction

$$\psi_{4cy2} = \frac{M_{opery}}{E_{ce} \times I_{c1y}} \tag{4.477}$$

Middle strip

The cracked moment of inertia I_{m1x} along shorter direction

$$I_{m1x} = I_{mx} + b_{mx} \times \left(1.2 \times x_{mx}^3/3\right) + m \times A_{st1mx} \times \left(d - (1.2 \times x_{mx})\right)^2/2 \tag{4.478}$$

The cracked moment of inertia I_{m1y} along longer direction

$$I_{m1y} = I_{my} + b_{my} \times \left(1.2 \times x_{my}^3/3\right) + m \times A_{st1my} \times \left(d - \left(1.2 \times x_{my}\right)\right)^2/2 \tag{4.479}$$

The curvature of middle strip ψ_{4mx2} along shorter direction

$$\psi_{4mx2} = \frac{M_{operx}}{E_{ce} \times I_{m1x}} \tag{4.480}$$

The curvature of middle strip ψ_{4my2} along longer direction

$$\psi_{4my2} = \frac{M_{operx}}{E_{ce} \times I_{m1y}} \tag{4.481}$$

Distribution factor

Steel stress at cracking moment

Column strip

If M_{crcx} is less than the M_{operx} then the value of later is considered

The steel stress at cracking moment for column strip S_{cx} along shorter direction

$$S_{cx} = \frac{S_{tcx} \times M_{crcx}}{M_{operx}} \tag{4.482}$$

If M_{crcy} is less than the M_{opery} then the value of later is considered

The steel stress at cracking moment for column strip S_{cy} along longer direction

$$S_{cy} = \frac{S_{tcy} \times M_{crcy}}{M_{opery}} \tag{4.483}$$

Middle strip

If M_{crmx} is less than the M_{operx} then the value of later is considered

The steel stress at cracking moment for middle strip S_{mx} along shorter direction

$$S_{mx} = \frac{S_{tmx} \times M_{crmx}}{M_{operx}} \tag{4.484}$$

If M_{crmy} is less than the M_{opery} then the value of later is considered.

The steel stress at cracking moment for middle strip S_{mx} along longer direction

$$S_{my} = \frac{S_{tmy} \times M_{crmy}}{M_{opery}} \tag{4.485}$$

For deformed bars and long term loads

Column strip

The factor for column strip along e_{cx} shorter direction

$$e_{cx} = 1 - 0.5 \times \left(\frac{S_{cx}}{S_{tcx}} \right)^2 \tag{4.486}$$

The factor for column strip e_{cy} along longer direction

$$e_{cy} = 1 - 0.5 \times \left(\frac{S_{cy}}{S_{tcy}} \right)^2 \tag{4.487}$$

Middle strip

The factor for middle strip e_{mx} along shorter direction

$$e_{mx} = 1 - 0.5 \times \left(\frac{S_{mx}}{S_{tmx}} \right)^2 \tag{4.488}$$

The factor for middle strip e_{my} along longer direction

$$e_{my} = 1 - 0.5 \times \left(\frac{S_{my}}{S_{tmy}} \right)^2 \tag{4.489}$$

Column strip

The curvature due to creep for column strip ψ_{4cx} along shorter direction

$$\psi_{4cx} = e_{cx} \times \psi_{4cx2} + (1 - e_{cx}) \times \psi_{4cx1} \tag{4.490}$$

The curvature due to creep for column strip ψ_{4cy} along longer direction

$$\psi_{4cy} = e_{cy} \times \psi_{4cy2} + (1 - e_{cy}) \times \psi_{4cy1} \tag{4.491}$$

Middle strip

The curvature due to creep for middle strip ψ_{4mx} along shorter direction

$$\psi_{4mx} = e_{mx} \times \psi_{4mx2} + (1 - e_{mx}) \times \psi_{4mx1} \tag{4.492}$$

The curvature due to creep for middle strip ψ_{4my} along longer direction

$$\psi_{4my} = e_{my} \times \psi_{4my2} + (1 - e_{my}) \times \psi_{4my1} \tag{4.493}$$

Long term deflection due to shrinkage

$$E_{cs} = 9.181 \times 10^{-4}$$

Column strip

The steel stress of column strip due to shrinkage S_{scx1} for uncracked section along shorter direction

$$S_{scx1} = \left(\frac{A_{st1cx} \times (D/2 - (cc + dia1/2))}{I_{cxs}} \right) \tag{4.494}$$

The curvature due to creep for column strip ψ_{3cx1} for uncracked section along shorter direction

$$\psi_{3cx1} = E_{cs} \times m_{ce} \times S_{scx1} \tag{4.495}$$

The steel stress of column strip due to shrinkage S_{scy1} for uncracked section along longer direction

$$S_{scy1} = \left(\frac{A_{st1cy} \times (D/2 - (cc + dia1/2))}{I_{cys}} \right) \tag{4.496}$$

The curvature due to shrinkage for column strip ψ_{3cy1} for uncracked section along longer direction

$$\psi_{3cy1} = E_{cs} \times m_{ce} \times S_{scy1} \tag{4.497}$$

Middle strip

The steel stress of middle strip due to shrinkage S_{smx1} for uncracked section along shorter direction

$$S_{smx1} = \left(\frac{A_{st1mx} \times (D/2 - (cc + dia1/2))}{I_{mxs}} \right) \tag{4.498}$$

The curvature due to shrinkage for middle strip ψ_{3mx1} for uncracked section along shorter direction

$$\psi_{3mx1} = E_{cs} \times m_{ce} \times S_{smx1} \tag{4.499}$$

The steel stress of middle strip due to shrinkage S_{smy1} for uncracked section along longer direction

$$S_{smy1} = \left(\frac{A_{st1my} \times (D/2 - (cc + dia1/2))}{I_{mys}} \right) \tag{4.500}$$

The curvature due to shrinkage for middle strip ψ_{3my1} for uncracked section along longer direction

$$\psi_{3my1} = E_{cs} \times m_{ce} \times S_{smy1} \tag{4.501}$$

For the cracked section

Column strip

The steel stress of column strip S_{scx2} for cracked condition along shorter direction

$$S_{scx2} = \left(\frac{A_{st1cx} \times (d - x_{cx1})}{I_{ecx}} \right) \tag{4.502}$$

The curvature due to shrinkage ψ_{3cx2} for column strip for cracked section along shorter direction

$$\psi_{3cx2} = E_{cs} \times m_{ce} \times S_{scx2} \tag{4.503}$$

The steel stress of column strip S_{scy2} for cracked condition along shorter direction

$$S_{scy2} = \left(\frac{A_{st1cy} \times (d - x_{cy1})}{I_{ecy}} \right) \tag{4.504}$$

The curvature due to shrinkage ψ_{3cy2} for column strip for cracked section along longer direction

$$\psi_{3cy2} = E_{cs} \times m_{ce} \times S_{scy2} \tag{4.505}$$

The actual shrinkage curvature ψ_{3cx} for column strip along shorter direction

$$\psi_{3cx} = e_{cx} \times \psi_{3cx1} + (1 - e_{cx}) \times \psi_{3cx2} \tag{4.506}$$

The actual shrinkage curvature ψ_{3cy} for column strip along longer direction

$$\psi_{3cy} = e_{cy} \times \psi_{3cy1} + (1 - e_{cy}) \times \psi_{3cy2} \quad (4.507)$$

Middle strip

The steel stress of middle strip S_{smx2} due to shrinkage for cracked section along shorter direction

$$S_{smx2} = \left(\frac{A_{st1mx} \times (d - x_{mx1})}{I_{mx}} \right) \quad (4.508)$$

The steel stress of middle strip ψ_{3mx2} due to shrinkage for cracked section along shorter direction

$$\psi_{3mx2} = E_{cs} \times m_{ce} \times S_{smx2} \quad (4.509)$$

The steel stress of middle strip S_{smy2} due to shrinkage for cracked section along longer direction

$$S_{smy2} = \left(\frac{A_{st1my} \times (d - x_{my2})}{I_{my}} \right) \quad (4.60)$$

The steel stress of middle strip ψ_{3my2} due to shrinkage for cracked section along longer direction

$$\psi_{3my2} = E_{cs} \times m_{ce} \times S_{smy2} \quad (4.61)$$

The actual shrinkage curvature ψ_{3mx} for middle strip along shorter direction

$$\psi_{3mx} = e_{mx} \times \psi_{3mx1} + (1 - e_{mx}) \times \psi_{3mx2} \quad (4.62)$$

The actual shrinkage curvature ψ_{3my} for middle strip along longer direction

$$\psi_{3my} = e_{my} \times \psi_{3my1} + (1 - e_{my}) \times \psi_{3my2} \quad (4.63)$$

Shrinkage deflection

Along x-x axis

Interior Panel

Shrinkage deflection of column strip δ_{shcix} for interior panel along shorter direction

$$\delta_{shcix} = LDF_c \times \psi_{3cx} \times 0.104 \times (1 - (0.75/10)) \times L_x^2 \quad (4.64)$$

Shrinkage deflection of middle strip δ_{shmix} for interior panel along shorter direction

$$\delta_{shmix} = LDF_m \times \psi_{3mx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\;2} \quad (4.65)$$

Corner Panel

Shrinkage deflection of column strip δ_{shccx} for corner panel along shorter direction

$$\delta_{shccx} = LDF_{1c} \times \psi_{3cx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\;2} \quad (4.66)$$

Shrinkage deflection of middle strip δ_{shmcx} for corner panel along shorter direction

$$\delta_{shmcx} = LDF_{1m} \times \psi_{3mx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\;2} \quad (4.67)$$

Side Panel

Shrinkage deflection of middle strip δ_{shcsx} for side panel along shorter direction

$$\delta_{shcsx} = LDF_{1c} \times \psi_{3cx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\;2} \quad (4.68)$$

Shrinkage deflection of middle strip δ_{shmsx} for side panel along shorter direction

$$\delta_{shmsx} = LDF_{1m} \times \psi_{3mx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\;2} \quad (4.69)$$

Along y-y axis

Interior Panel

Shrinkage deflection of column strip δ_{shciy} for interior panel along longer direction

$$\delta_{shciy} = LDF_c \times \psi_{3cy} \times 0.104 \times (1-(0.75/10)) \times L_y^{\;2} \quad (4.520)$$

Shrinkage deflection of middle strip δ_{shmiy} for interior panel along longer direction

$$\delta_{shmiy} = LDF_m \times \psi_{3my} \times 0.104 \times (1-(0.75/10)) \times L_y^{\;2} \quad (4.521)$$

Corner Panel

Shrinkage deflection of column strip δ_{shccy} for corner panel along longer direction

$$\delta_{shccy} = LDF_{1c} \times \psi_{3cy} \times 0.104 \times (1-(0.75/10)) \times L_y^{\;2} \quad (4.522)$$

Shrinkage deflection of middle strip δ_{shmcy} for corner panel along longer direction

$$\delta_{shmcy} = LDF_{1m} \times \psi_{3my} \times 0.104 \times (1-(0.75/10)) \times L_y^{\;2} \quad (4.523)$$

Side Panel

Shrinkage deflection of column strip δ_{shcsy} for side panel along longer direction

$$\delta_{shcsy} = LDF_{1c} \times \psi_{3cy} \times 0.104 \times (1-(0.75/10)) \times L_y^{\,2} \tag{4.524}$$

Shrinkage deflection of middle strip δ_{shmsy} for side panel along longer direction

$$\delta_{shmsy} = LDF_{1m} \times \psi_{3my} \times 0.104 \times (1-(0.75/10)) \times L_y^{\,2} \tag{4.525}$$

Shrinkage deflection

Interior Panel

Shrinkage deflection δ_{ish} of interior panel

$$\delta_{ish} = \left(\frac{(\delta_{shcix} + \delta_{shmiy}) + (\delta_{shciy} + \delta_{shmix})}{2} \right) \tag{4.526}$$

Corner Panel

Shrinkage deflection δ_{csh} of corner panel

$$\delta_{csh} = \left(\frac{(\delta_{shccx} + \delta_{shmcy}) + (\delta_{shccy} + \delta_{shmcx})}{2} \right) \tag{4.527}$$

Side Panel

Shrinkage deflection δ_{ssh} of side panel

$$\delta_{ssh} = \left(\frac{(\delta_{shcsx} + \delta_{shmsy}) + (\delta_{shcsy} + \delta_{shmsx})}{2} \right) \tag{4.528}$$

Creep deflection

x-x axis

Interior Panel

Creep deflection of column strip δ_{crcix} for interior panel

$$\delta_{crcix} = LDF_c \times \psi_{4cx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\,2} \tag{4.529}$$

Creep deflection of middle strip δ_{crmix} for interior panel

$$\delta_{crmix} = LDF_m \times \psi_{4mx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\,2} \tag{4.530}$$

Corner Panel

Creep deflection of column strip δ_{crccx} for corner panel

$$\delta_{crccx} = LDF_{1c} \times \psi_{4cx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\ 2} \tag{4.531}$$

Creep deflection of middle strip δ_{crmcx} for corner panel

$$\delta_{crmcx} = LDF_{1m} \times \psi_{4mx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\ 2} \tag{4.532}$$

Side Panel

Creep deflection of column strip δ_{crcsx} for side panel

$$\delta_{crcsx} = LDF_{1c} \times \psi_{4cx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\ 2} \tag{4.533}$$

Creep deflection of middle strip δ_{crmsx} for side panel

$$\delta_{crmsx} = LDF_{1m} \times \psi_{4mx} \times 0.104 \times (1-(0.75/10)) \times L_x^{\ 2} \tag{4.534}$$

y-y axis

Interior Panel

Creep deflection of column strip δ_{crciy} for interior panel

$$\delta_{crciy} = LDF_{c} \times \psi_{4cy} \times 0.104 \times (1-(0.75/10)) \times L_y^{\ 2} \tag{4.535}$$

Creep deflection of middle strip δ_{crmiy} for interior panel

$$\delta_{crmiy} = LDF_{m} \times \psi_{4my} \times 0.104 \times (1-(0.75/10)) \times L_y^{\ 2} \tag{4.536}$$

Corner Panel

Creep deflection of column strip δ_{crccy} for corner panel

$$\delta_{crccy} = LDF_{1c} \times \psi_{4cy} \times 0.104 \times (1-(0.75/10)) \times L_y^{\ 2} \tag{4.537}$$

Creep deflection of middle strip δ_{crmcy} for corner panel

$$\delta_{crmcy} = LDF_{1m} \times \psi_{4my} \times 0.104 \times (1-(0.75/10)) \times L_y^{\ 2} \tag{4.538}$$

Side Panel

Creep deflection of middle strip δ_{crcsy} for side panel

$$\delta_{crcsy} = LDF_{1c} \times \psi_{4cy} \times 0.104 \times (1-(0.75/10)) \times L_y^{\ 2} \tag{4.539}$$

Creep deflection of middle strip δ_{crmsy} for side panel

$$\delta_{crmsy} = LDF_{1m} \times \psi_{4my} \times 0.104 \times (1-(0.75/10)) \times L_y^{\,2} \tag{4.540}$$

Creep deflection

Interior Panel

Creep deflection δ_{ic} for interior panel

$$\delta_{ic} = \left(\frac{(\delta_{crcix} + \delta_{crmiy}) + (\delta_{crciy} + \delta_{crmix})}{2} \right) \tag{4.541}$$

Corner Panel

Creep deflection δ_{cc} for corner panel

$$\delta_{cc} = \left(\frac{(\delta_{crccx} + \delta_{crmcy}) + (\delta_{crccy} + \delta_{crmcx}) + (\delta_{d2x} + \delta_{d2y})}{2} \right) \tag{4.542}$$

Side Panel

Creep deflection δ_{sc} for side panel

$$\delta_{sc} = \left(\frac{(\delta_{crcsx} + \delta_{crmsy}) + (\delta_{crcsy} + \delta_{crmsx}) + \delta_{d2y}}{2} \right) \tag{4.543}$$

Total Deflection

Interior Panel

Total deflection δ_{ia} of interior panel

$$\delta_{ia} = \delta_{is} + \delta_{ish} + \delta_{ic} \tag{4.544}$$

Corner Panel

Total deflection δ_{ca} of corner panel

$$\delta_{ca} = \delta_{cs} + \delta_{csh} + \delta_{cc} \tag{4.545}$$

Side Panel

Total deflection δ_{sa} of side panel

$$\delta_{sa} = \delta_{ss} + \delta_{ssh} + \delta_{sc} \tag{4.546}$$

Percentage contribution

Interior Panel

The contribution of short term deflection of interior panel

$$\%\delta_{ishort} = \left(\frac{\delta_{is}}{\delta_i}\right) \times 100 \tag{4.317}$$

The contribution of shrinkage deflection of interior panel

$$\%\delta_{ishr} = \left(\frac{\delta_{ish}}{\delta_i}\right) \times 100 \tag{4.318}$$

The contribution of creep deflection of interior panel

$$\%\delta_{icr} = \left(\frac{\delta_{ic}}{\delta_i}\right) \times 100 \tag{4.319}$$

The total contribution of creep and shrinkage effects

$$\%\delta_{il} = \%\delta_{ishr} + \%\delta_{icr} \tag{4.320}$$

Corner Panel

The contribution of short term deflection of corner panel

$$\%\delta_{cshort} = \left(\frac{\delta_{cs}}{\delta_c}\right) \times 100 \tag{4.321}$$

The contribution of shrinkage deflection of corner panel

$$\%\delta_{cshr} = \left(\frac{\delta_{csh}}{\delta_c}\right) \times 100 \tag{4.322}$$

The contribution of creep deflection of corner panel

$$\%\delta_{ccr} = \left(\frac{\delta_{cc}}{\delta_c}\right) \times 100 \tag{4.323}$$

The total contribution of creep and shrinkage effects

$$\%\delta_{cl} = \%\delta_{cshr} + \%\delta_{ccr} \tag{4.324}$$

Side Panel

The contribution of short term deflection of side panel

$$\%\delta_{sshort} = \left(\frac{\delta_{ss}}{\delta_s}\right) \times 100 \tag{4.325}$$

The contribution of shrinkage deflection of side panel

$$\%\delta_{sshr} = \left(\frac{\delta_{ssh}}{\delta_s}\right) \times 100 \tag{4.326}$$

The contribution of creep deflection of side panel

$$\%\delta_{scr} = \left(\frac{\delta_{sc}}{\delta_s}\right) \times 100 \tag{4.327}$$

The total contribution of creep and shrinkage effects

$$\%\delta_{sl} = \%\delta_{sshr} + \%\delta_{scr} \tag{4.328}$$

4.5 AS PER IS: 456-2000

This code uses the moment curvature method for shrinkage deflection. The creep coefficient method is used for deflection due to creep. The tension stiffening effect of concrete is not considered.

The sustained load

$$W_d = 25D + FF + 0.25Wl \tag{4.2}$$

The moment due to sustained load M_{operx} along shorter direction

$$M_{operx} = \frac{W_d \times 0.5 \times L_y \times L_x^2}{8} \tag{4.136}$$

The moment due to sustained load M_{opery} along longer direction

$$M_{opery} = \frac{W_d \times 0.5 \times L_x \times L_y^2}{8} \tag{4.331}$$

Modulus of rupture f_{cr} of concrete

$$f_{cr} = 0.7 \times \sqrt{f_{ck}} \tag{4.547}$$

The distance of extreme fibre

$$y_t = \frac{D}{2}$$

The total load

$$TL = 25D + FF + Wl \tag{4.329}$$

The moment due to total load M_{tlx} along shorter direction

$$M_{tlx} = \frac{L_y \times TL \times L_x^2}{8} \tag{4.330}$$

The moment due to total load M_{tly} along longer direction

$$M_{tly} = \frac{L_x \times TL \times L_y^2}{8} \tag{4.332}$$

$$LDF_c = 0.60$$

$$LDF_m = 0.40$$

Based on Murray et al. (2000), the edge column moment distribution is considered as 55 : 45.

$$LDF_{1c} = 0.75$$

$$LDF_{1m} = 0.25$$

$$W_t = W_d + Wl \tag{4.3}$$

Column Strip

The gross moment of inertia of column strip I_{grcx} along shorter direction

$$I_{grcx} = \frac{0.5 \times L_x \times D^3}{12} \tag{4.548}$$

The gross moment of inertia of column strip I_{grcy} along longer direction

$$I_{grcy} = \frac{0.5 \times L_y \times D^3}{12} \tag{4.550}$$

Cracked moment of column strip M_{crcx} along shorter direction

$$M_{crcx} = \frac{f_{cr} \times I_{grcx}}{y_t} \tag{4.551}$$

Cracked moment of column strip M_{crcy} along longer direction

$$M_{crcy} = \frac{f_{cr} \times I_{grcy}}{y_t} \tag{4.552}$$

Middle Strip

The gross moment of inertia of middle strip I_{grmx} along shorter direction

$$I_{grmx} = \frac{0.5 \times L_x \times D^3}{12} \tag{4.553}$$

The gross moment of inertia of middle strip I_{grmy} along longer direction

$$I_{grmy} = \frac{0.5 \times L_y \times D^3}{12} \tag{4.554}$$

The gross moment of inertia of middle strip M_{rmx} along shorter direction

$$M_{rmx} = \frac{f_{cr} \times I_{grmx}}{y_t} \tag{4.555}$$

The gross moment of inertia of middle strip M_{rmy} along longer direction

$$M_{rmy} = \frac{f_{cr} \times I_{grmy}}{y_t} \tag{4.556}$$

Short term deflection

Modulus of elasticity of concrete

$$E_c = 5000 \times \sqrt{f_{ck}} \tag{4.557}$$

The modular ratio

$$m = \frac{E_s}{E_c}$$

The effective depth of tension reinforcement

$$d = D - cc - (dia/2) \tag{4.558}$$

Column strip

The area of tension reinforcement provided in the column strip A_{stcx} along shorter direction

$$A_{stcx} = \frac{0.5 \times 0.5 \times L_x \times D}{100} \tag{4.559}$$

The area of tension reinforcement provided in the column strip A_{stcy} along longer direction

$$A_{stcy} = \frac{0.5 \times 0.5 \times L_y \times D}{100} \tag{4.560}$$

$$0.5 \times b_{cx} \times x_{cx}^2 + (m \times A_{stcx}) = (m \times A_{stcx} \times d) \tag{4.561}$$

$$0.5 \times b_{cy} \times x_{cy}^2 + (m \times A_{stcy}) = (m \times A_{stcy} \times d) \tag{4.562}$$

The lever arm of column strip Z_{cx} along shorter direction

$$Z_{cx} = d - (x_{cx}/3) \tag{4.563}$$

The lever arm of column strip Z_{cy} along longer direction

$$Z_{cy} = d - (x_{cy}/3) \tag{4.564}$$

The area of tension reinforcement provided in the frame A_{st2cx} along shorter direction

$$A_{st2cx} = \frac{0.5 \times L_x \times D}{100} \tag{4.565}$$

$$0.5 \times L_x \times x_{2cx}^2 + (m \times A_{st2cx}) = (m \times A_{st2cx} \times d) \tag{4.566}$$

The area of tension reinforcement provided in the frame A_{st2cy} along longer direction

$$A_{st2cy} = \frac{0.5 \times L_y \times D}{100} \tag{4.567}$$

$$0.5 \times L_y \times x_{2cy}^2 + (m \times A_{st2cy}) = (m \times A_{st2cy} \times d) \tag{4.568}$$

The moment of inertia of column strip I_{r1cx} along shorter direction

$$I_{r1cx} = \frac{0.5 \times L_x \times x_{cx1}{}^3}{3} + \left(m \times A_{stcx} \times (d - x_{cx1})^2\right) \tag{4.569}$$

The moment of inertia of frame I_{r2cx} along shorter direction

$$I_{r2cx} = \frac{0.5 \times L_x \times x_{cx2}{}^3}{3} + \left(m \times A_{stcx} \times (d - x_{cx2})^2\right) \tag{4.570}$$

The moment of inertia of column strip I_{r1cy} along longer direction

$$I_{r1cy} = \frac{0.5 \times L_y \times x_{cy1}{}^3}{3} + \left(m \times A_{stcy} \times (d - x_{cy1})^2\right) \tag{4.571}$$

The moment of inertia of frame I_{r2cy} along longer direction

$$I_{r2cy} = \frac{0.5 \times L_y \times x_{cy2}{}^3}{3} + \left(m \times A_{stcy} \times \left(d - x_{cy2}\right)^2\right) \tag{4.572}$$

Middle strip

The area of tension reinforcement provided in the middle strip A_{stmx} along shorter direction

$$A_{stmx} = \frac{0.5 \times 0.5 \times L_x \times D}{100} \tag{4.573}$$

The area of tension reinforcement provided in the middle strip A_{stmy} along shorter direction

$$A_{stmy} = \frac{0.5 \times 0.5 \times L_y \times D}{100} \tag{4.574}$$

$$0.5 \times b_{mx} \times x_{mx}^2 + \left(m \times A_{stmx}\right) = \left(m \times A_{stmx} \times d\right) \tag{4.575}$$

$$0.5 \times b_{my} \times x_{my}^2 + \left(m \times A_{stmy}\right) = \left(m \times A_{stmy} \times d\right) \tag{4.576}$$

The lever arm of middle strip Z_{mx} along shorter direction

$$Z_{mx} = d - \left(x_{mx}/3\right) \tag{4.577}$$

The lever arm of middle strip Z_{my} along longer direction

$$Z_{my} = d - \left(x_{my}/3\right) \tag{4.578}$$

The moment of inertia of middle strip I_{r1mx} along shorter direction

$$I_{r1mx} = \frac{0.5 \times L_x \times x_{mx1}{}^3}{3} + \left(m \times A_{stmx} \times \left(d - x_{mx1}\right)^2\right) \tag{4.579}$$

The moment of inertia of frame I_{r2mx} along shorter direction

$$I_{r2mx} = \frac{0.5 \times L_x \times x_{mx2}{}^3}{3} + \left(m \times A_{stmx} \times \left(d - x_{mx2}\right)^2\right) \tag{4.580}$$

The moment of inertia of middle strip I_{r1my} along longer direction

$$I_{r1my} = \frac{0.5 \times L_y \times x_{my1}{}^3}{3} + \left(m \times A_{stmy} \times \left(d - x_{my1}\right)^2\right) \tag{4.581}$$

The moment of inertia of frame I_{r2my} along longer direction

$$I_{r2my} = \frac{0.5 \times L_y \times x_{my2}^{\;3}}{3} + \left(m \times A_{stmy} \times \left(d - x_{my2}\right)^2\right) \tag{4.582}$$

Flexural stiffness of equivalent column

Lc = the storey height of the frame.

Factored dead load W_D

$$W_D = 1.5 \times \left(D \times 25 + FF\right) \tag{4.583}$$

Factored live load W_L

$$W_L = 1.5 \times Wl \tag{4.584}$$

Total load W_T

$$W_T = W_D + W_L \tag{4.23}$$

Uniformly distributed load w

$$w = W_T \times L_y \tag{4.24}$$

Area of column required A_c

$$A_c = \left(\frac{2 \times 3 \times W_T \times L_x \times {}^{L_y}\!/_{2}}{0.85 \times \sqrt{f_{ck}}}\right) \tag{4.25}$$

$$d = D - cc - {}^{dia}\!/_{2}$$

Edge column size ($C_{1x} \times C_{1y}$)

$$C_{1x} = \frac{((A_c/d) - 2d)}{3} \tag{4.26}$$

$$C_{1y} = \frac{((A_c/d) - 2d)}{3} \tag{4.27}$$

Short term deflection along x-x axis

$$I_{c1x} = \frac{C_{1x} C_{1y}^3}{12} \tag{4.28}$$

$$K_{cx} = \frac{4E_c I_{c1x}}{L_c} \tag{4.29}$$

$$C_x = \frac{(1-(0.63D/C_{1x}))(D^3 C_{1x})}{3} \tag{4.30}$$

$$L_{nx} = L_x - C_{1x}$$

$$K_{tx} = \frac{2\times 4.5\times E_c \times C_x}{\left(L_y\left(1-\left(\frac{C_{1y}}{L_y}\right)\right)^3\right)} \tag{4.31}$$

The moment of inertia of frame along shorter direction for short term loads

$$I_{fxs} = \frac{b_{fx}D^3}{12} + b_{fx}\times D\times\left(\frac{D}{2} - x_{2cxs}\right)^2 + (m-1)\times A_{st2cx}\times(d - x_{2cxs})^2 \tag{4.585}$$

$$K_{s1x} = \frac{4E_c I_{fxs}}{L_x} \tag{4.586}$$

$$K_{ecx} = \left(\frac{1}{2\times K_{cx} + K_{tx}}\right) \tag{4.587}$$

$$\alpha_{1x} = \frac{K_{ecx}}{2K_{s1x}} \tag{4.588}$$

The moment due to total load

$$M_{operx} = \frac{TL\times L_y \times L_{nx}^2}{8} \tag{4.589}$$

Moment due to total load M_{nt1x}

$$M_{nt1x} = \frac{0.65\times M_{ox}}{\left(1+\frac{1}{\alpha_{1x}}\right)} \tag{4.590}$$

Moment due to sustained load M_{nd1x}

$$M_{nd1x} = \frac{W_d \times M_{nt1x}}{TL} \tag{4.591}$$

The rotation due to total load θ_{t1x}

$$\theta_{t1x} = \frac{M_{nt1x}}{K_{ecx}} \tag{4.379}$$

The rotation due to sustained load θ_{d1x}

$$\theta_{d1x} = \frac{M_{nd1x}}{K_{ecx}} \tag{4.380}$$

The deflection due to total load δ_{t1x}

$$\delta_{t1x} = \left(\theta_{t1x} \times L_x / 8\right) \tag{4.592}$$

The deflection due to sustained load δ_{d1x}

$$\delta_{d1x} = \frac{\theta_{d1x} L_x}{8} \tag{4.593}$$

Long term

The moment of inertia of frame I_{fxl} along shorter direction for long term loads

$$I_{fxl} = \frac{b_{fx} D^3}{3} + m \times A_{st2cx} \times \left(d - x_{2cx}\right)^2 \tag{4.594}$$

$$K_{s2x} = \frac{4 E_c I_{fxl}}{L_x} \tag{4.595}$$

$$\alpha_{2x} = \frac{K_{ecx}}{2 K_{s2x}} \tag{4.596}$$

The moment due to total load M_{nt2x}

$$M_{nt2x} = \frac{0.65 \times M_{ox}}{\left(1 + 1/\alpha_{2x}\right)} \tag{4.597}$$

The moment due to sustained load M_{nd2x}

$$M_{nd2x} = \frac{W_d \times M_{nt2x}}{TL} \tag{4.598}$$

The rotation θ_{d2x} due to sustained load

$$\theta_{d2x} = \frac{M_{nd2x}}{K_{ecx}} \tag{4.599}$$

The deflection δ_{d2x} due to sustained load

$$\delta_{d2x} = \frac{\theta_{d2x} L_x}{8} \tag{4.600}$$

Short term deflection along y-y-axis

$$I_{c1y} = \frac{C_{1y} C_{1x}^3}{12} \tag{4.601}$$

The relative stiffness K_{cy} of column

$$K_{cy} = \frac{4E_c I_{c1y}}{L_c} \tag{4.602}$$

$$C_y = \frac{\left(1 - \left(0.63D/C_{1y}\right)\right)\left(D^3 C_{1y}\right)}{3} \tag{4.603}$$

Clear span L_{ny} along longer direction

$$L_{ny} = L_y - C_{1y}$$

$$K_{ty} = \frac{2 \times 4.5 \times E_c \times C_y}{\left(L_x \left(1 - \left(\frac{C_{1x}}{L_x}\right)\right)^3\right)} \tag{4.604}$$

The moment of inertia of frame I_{fys} along shorter direction for short term loads

$$I_{fys} = \frac{b_{fy} D^3}{12} + b_{fy} \times D \times \left(\frac{D}{2} - x_{2cys}\right)^2 + (m-1) \times A_{st2cy} \times \left(d - x_{2cys}\right)^2 \tag{4.605}$$

$$K_{s1y} = \frac{4E_c I_{fys}}{L_y} \tag{4.606}$$

$$K_{ecy} = \left(\frac{1}{2 \times K_{cy} + K_{ty}}\right) \tag{4.607}$$

$$\alpha_{1y} = \frac{K_{ecy}}{2K_{s1y}} \tag{4.608}$$

The moment due to total load

$$M_{opery} = \frac{TL \times L_x \times L_{ny}^2}{8} \tag{4.609}$$

$$M_{nt1y} = \frac{0.65 \times M_{oy}}{\left(1 + \frac{1}{\alpha_{1y}}\right)} \tag{4.610}$$

$$M_{nd1y} = \frac{W_d \times M_{nt1y}}{TL} \tag{4.611}$$

The rotation θ_{t1y} due to total load

$$\theta_{t1y} = \frac{M_{nt1y}}{K_{ecy}} \tag{4.612}$$

The rotation θ_{d1y} due to sustained load

$$\theta_{d1y} = \frac{M_{nd1y}}{K_{ecy}} \tag{4.613}$$

The deflection δ_{t1y} due to total load

$$\delta_{t1y} = \left(\theta_{t1y} \times L_y / 8\right) \tag{4.614}$$

The deflection δ_{d1y} due to sustained load

$$\delta_{d1y} = \frac{\theta_{d1y} L_y}{8} \tag{4.615}$$

Long term

The moment of inertia of frame I_{fyl} along longer direction for long term loads

$$I_{fyl} = \frac{b_{fy} D^3}{3} + m \times A_{st2cy} \times \left(d - x_{2cy}\right)^2 \tag{4.616}$$

$$K_{s2y} = \frac{4 E_c I_{fyl}}{L_y} \tag{4.617}$$

$$\alpha_{2y} = \frac{K_{ecy}}{2 K_{s2y}} \tag{4.618}$$

$$M_{nt2y} = \frac{0.65 \times M_{oy}}{\left(1 + 1/\alpha_{2y}\right)} \tag{4.619}$$

$$M_{nd2y} = \frac{W_d \times M_{nt2y}}{TL} \tag{4.620}$$

The rotation θ_{d2y} due to sustained load

$$\theta_{d2y} = \frac{M_{nd2y}}{K_{ecy}} \tag{4.621}$$

The deflection δ_{d2y} due to sustained load

$$\delta_{d2y} = \frac{\theta_{d2y} L_y}{8} \tag{4.622}$$

For total load

Column Strip

$$C_{cx} = 1.2 - \left(\frac{M_{rcx}}{M_{t1x}}\right) \times \left(\frac{Z_{cx}}{d}\right) \times \left(1 - \frac{x_{cx}}{d}\right) \tag{4.623}$$

$$C_{cy} = 1.2 - \left(\frac{M_{rcy}}{M_{t1y}}\right) \times \left(\frac{Z_{cy}}{d}\right) \times \left(1 - \frac{x_{cy}}{d}\right) \tag{4.624}$$

$$I_{cx} = \frac{I_{r2cx}}{C_{cx}} \tag{4.625}$$

If I_{cx} is less than the I_{grcx} then the value of later is considered

$$I_{cy} = \frac{I_{r2cy}}{C_{cy}} \tag{4.626}$$

If I_{cy} is less than the I_{grcy} then the value of later is considered

x-x axis

Interior Panel

Deflection of column strip for total load δ_{t1cix} for interior panel along shorter direction

$$\delta_{t1cix} = \left(\frac{LDF_c \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{cx}}\right) \tag{4.627}$$

Corner Panel

Deflection of column strip for total load δ_{t1ccx} for corner panel along shorter direction

$$\delta_{t1ccx} = \left(\frac{LDF_{1c} \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{cx}}\right) \tag{4.628}$$

Side Panel

Deflection of column strip for total load δ_{t1csx} for side panel along shorter direction

$$\delta_{t1csx} = \left(\frac{LDF_{1c} \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{cx}}\right) \tag{4.629}$$

y-y axis

Interior Panel

Deflection of column strip for total load δ_{t1ciy} for interior panel along longer direction

$$\delta_{t1ciy} = \left(\frac{LDF_c \times TL \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} \right) \tag{4.630}$$

Corner Panel

Deflection of column strip for total load δ_{t1ccy} for corner panel along longer direction

$$\delta_{t1ccy} = \left(\frac{LDF_{1c} \times TL \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} \right) \tag{4.631}$$

Side Panel

Deflection of column strip for total load δ_{t1csy} for side panel along longer direction

$$\delta_{t1csy} = \left(\frac{LDF_{1c} \times TL \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} \right) \tag{4.632}$$

Middle Strip

$$C_{mx} = 1.2 - \left(\frac{M_{rmx}}{M_{t1x}} \right) \times \left(\frac{Z_{mx}}{d} \right) \times \left(1 - \frac{x_{mx}}{d} \right) \tag{4.633}$$

$$C_{my} = 1.2 - \left(\frac{M_{rmy}}{M_{t1y}} \right) \times \left(\frac{Z_{my}}{d} \right) \times \left(1 - \frac{x_{my}}{d} \right) \tag{4.634}$$

$$I_{mx} = \frac{I_{r2mx}}{C_{mx}} \tag{4.635}$$

If I_{mx} is less than the I_{grmx} then the value of later is considered

$$I_{my} = \frac{I_{r2my}}{C_{my}} \tag{4.636}$$

If I_{my} is less than the I_{grmy} then the value of later is considered

x-x axis

Interior Panel

Deflection of middle strip for total load δ_{t1mix} for interior panel along shorter direction

$$\delta_{t1mix} = \left(\frac{LDF_c \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} \right) \tag{4.637}$$

Corner Panel

Deflection of middle strip for total load δ_{t1mcx} for corner panel along shorter direction

$$\delta_{t1mcx} = \left(\frac{LDF_{1c} \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} \right) \tag{4.638}$$

Side Panel

Deflection of middle strip for total load δ_{t1msx} for side panel along shorter direction

$$\delta_{t1msx} = \left(\frac{LDF_{1c} \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} \right) \tag{4.639}$$

y-y axis

Interior Panel

Deflection of middle strip for total load δ_{t1miy} for interior panel along longer direction

$$\delta_{t1miy} = \left(\frac{LDF_c \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{my}} \right) \tag{4.640}$$

Corner Panel

Deflection of middle strip for total load δ_{t1mcy} for corner panel along longer direction

$$\delta_{t1mcy} = \left(\frac{LDF_{1c} \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{my}} \right) \tag{4.641}$$

Side Panel

Deflection of middle strip for total load δ_{t1msy} for side panel along longer direction

$$\delta_{t1msy} = \left(\frac{LDF_{1c} \times TL \times L_y \times L_x^4}{384 \times E_c \times I_{my}} \right) \tag{4.642}$$

For permanent load

Column Strip

$$C_{1cx} = 1.2 - \left(\frac{M_{rcx}}{M_{operx}}\right) \times \left(\frac{Z_{cx}}{d}\right) \times \left(1 - \frac{x_{cx}}{d}\right) \tag{4.643}$$

$$C_{1cy} = 1.2 - \left(\frac{M_{rcy}}{M_{opery}}\right) \times \left(\frac{Z_{cy}}{d}\right) \times \left(1 - \frac{x_{cy}}{d}\right) \tag{4.644}$$

$$I_{1cx} = \frac{I_{r2cx}}{C_{1cx}} \tag{4.645}$$

If I_{1cx} is less than the I_{grcx} then the value of later is considered

$$I_{1cy} = \frac{I_{r2cy}}{C_{1cy}} \tag{4.646}$$

If I_{1cy} is less than the I_{grcy} then the value of later is considered

x-x axis

Interior Panel

Deflection of column strip for sustained load $\delta_{permcix}$ for interior panel along shorter direction

$$\delta_{permcix} = \left(\frac{LDF_c \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{1cx}}\right) \tag{4.647}$$

Corner Panel

Deflection of column strip for sustained load $\delta_{permccx}$ for corner panel along shorter direction

$$\delta_{permccx} = \left(\frac{LDF_{1c} \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{1cx}}\right) \tag{4.648}$$

Side Panel

Deflection of column strip for sustained load $\delta_{permcsx}$ for side panel along shorter direction

$$\delta_{permcsx} = \left(\frac{LDF_{1c} \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{1cx}}\right) \tag{4.649}$$

y-y axis

Interior Panel

Deflection of column strip for sustained load $\delta_{permciy}$ for interior panel along longer direction

$$\delta_{permciy} = \left(\frac{LDF_c \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{1cy}} \right) \tag{4.650}$$

Corner Panel

Deflection of column strip for sustained load $\delta_{permccy}$ for corner panel along longer direction

$$\delta_{permccy} = \left(\frac{LDF_{1c} \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{1cy}} \right) \tag{4.651}$$

Side Panel

Deflection of column strip for sustained load $\delta_{permcsy}$ for side panel along longer direction

$$\delta_{permcsy} = \left(\frac{LDF_{1c} \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{1cy}} \right) \tag{4.652}$$

Middle Strip

$$C_{1mx} = 1.2 - \left(\frac{M_{rmx}}{M_{operx}} \right) \times \left(\frac{Z_{mx}}{d} \right) \times \left(1 - \frac{x_{mx}}{d} \right) \tag{4.653}$$

$$C_{1my} = 1.2 - \left(\frac{M_{rmy}}{M_{opery}} \right) \times \left(\frac{Z_{my}}{d} \right) \times \left(1 - \frac{x_{my}}{d} \right) \tag{4.654}$$

$$I_{1mx} = \frac{I_{r2mx}}{C_{1mx}} \tag{4.655}$$

If I_{1mx} is less than the I_{grmx} then the value of later is considered

$$I_{1my} = \frac{I_{r2my}}{C_{1my}} \tag{4.656}$$

If I_{1my} is less than the I_{grmy} then the value of later is considered

x-x axis

Interior Panel

Deflection of middle strip for sustained load $\delta_{permmix}$ for interior panel along shorter direction

$$\delta_{permmix} = \left(\frac{LDF_c \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{1mx}} \right) \tag{4.657}$$

Corner Panel

Deflection of middle strip for sustained load $\delta_{permmcx}$ for corner panel along shorter direction

$$\delta_{permmcx} = \left(\frac{LDF_{1c} \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{1mx}} \right) \tag{4.658}$$

Side Panel

Deflection of middle strip for sustained load $\delta_{permmsx}$ for side panel along shorter direction

$$\delta_{permmsx} = \left(\frac{LDF_{1c} \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{1mx}} \right) \tag{4.659}$$

y-y axis

Interior Panel

Deflection of middle strip for sustained load $\delta_{permmiy}$ for interior panel along longer direction

$$\delta_{permmiy} = \left(\frac{LDF_c \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{1my}} \right) \tag{4.660}$$

Corner Panel

Deflection of middle strip for sustained load $\delta_{permmcy}$ for corner panel along longer direction

$$\delta_{permmcy} = \left(\frac{LDF_{1c} \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{1my}} \right) \tag{4.661}$$

Side Panel

Deflection of middle strip for sustained load $\delta_{permmsy}$ for side panel along longer direction

$$\delta_{permmsy} = \left(\frac{LDF_{1c} \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{1my}} \right) \tag{4.662}$$

x-x axis

Deflection of Interior Panel δ_{ix} along shorter direction

$$\delta_{ix} = \delta_{t1cix} + \delta_{t1miy} \tag{4.663}$$

Corner Panel

Deflection of Corner Panel δ_{cx} along shorter direction

$$\delta_{cx} = \delta_{t1ccx} + \delta_{t1mcy} \tag{4.664}$$

Side Panel

Deflection of Side Panel δ_{sx} along shorter direction

$$\delta_{sx} = \delta_{t1csx} + \delta_{t1msy} \tag{4.665}$$

y-y axis

Interior Panel

Deflection of Interior Panel δ_{iy} along longer direction

$$\delta_{iy} = \delta_{t1ciy} + \delta_{t1mix} \tag{4.666}$$

Corner Panel

Deflection of Corner Panel δ_{cy} along longer direction

$$\delta_{cy} = \delta_{t1ccy} + \delta_{t1mcx} \tag{4.667}$$

Side Panel

Deflection of Side Panel δ_{sy} along longer direction

$$\delta_{sy} = \delta_{t1csy} + \delta_{t1msx} \tag{4.668}$$

Long term deflection due to shrinkage

Column Strip

The percentage of tension reinforcement P_{tcx} for column strip along shorter direction

$$P_{tcx} = \frac{A_{stcx}}{0.5 \times L_x \times d} \times 100 \tag{4.669}$$

$$P_{cc} = 0$$

$$K_{4cx} = 0.72 \times \sqrt{P}_{tcx} \tag{4.670}$$

The percentage of tension reinforcement P_{tcy} for column strip along longer direction

$$P_{tcy} = \frac{A_{stcy}}{0.5 \times L_y \times d} \times 100 \tag{4.671}$$

$$P_{cc} = 0$$

$$K_{4cy} = 0.72 \times \sqrt{P}_{tcy} \tag{4.672}$$

Shrinkage strain

$$E_{cs} = 0.0003$$

Shrinkage curvature of column strip SI_{cscx} along shorter direction

$$SI_{cscx} = \frac{K_{4cx} \times E_{cs}}{D} \tag{4.673}$$

Shrinkage curvature of column strip SI_{cscy} along longer direction

$$SI_{cscy} = \frac{K_{4cy} \times E_{cs}}{D} \tag{4.674}$$

Middle Strip

The percentage of tension reinforcement P_{tmx} for middle strip along shorter direction

$$P_{tmx} = \frac{A_{stmx}}{0.5 \times L_x \times d} \times 100 \tag{4.675}$$

$$P_{cc} = 0$$

$$K_{4mx} = 0.72 \times \sqrt{P}_{tmx} \tag{4.676}$$

The percentage of tension reinforcement P_{tmy} for middle strip along longer direction

$$P_{tmy} = \frac{A_{stmy}}{0.5 \times L_y \times d} \times 100 \tag{4.677}$$

$$P_{cc} = 0$$

$$K_{4my} = 0.72 \times \sqrt{P}_{tmy} \tag{4.678}$$

As per IS: 456 – 2000, the shrinkage strain is assumed as 0.0003.

$$E_{cs} = 0.0003$$

Shrinkage curvature of middle strip SI_{csmx} along shorter direction

$$SI_{csmx} = \frac{K_{4mx} \times E_{cs}}{D} \tag{4.679}$$

Shrinkage curvature of middle strip SI_{csmy} along longer direction

$$SI_{csmy} = \frac{K_{4my} \times E_{cs}}{D} \tag{4.680}$$

x-x axis

Interior Panel

Deflection due to shrinkage δ_{cscix} for column strip is given by

$$\delta_{cscix} = LDF_c \times K3 \times SI_{cscx} \times L_x^2 \tag{4.681}$$

Deflection due to shrinkage δ_{csmix} for middle strip is given by

$$\delta_{csmix} = LDF_m \times K3 \times SI_{csmx} \times L_x^2 \tag{4.682}$$

Corner panel

Deflection due to shrinkage δ_{csccx} for column strip is given by

$$\delta_{csccx} = LDF_{1c} \times K3 \times SI_{cscx} \times L_x^2 \tag{4.683}$$

Deflection due to shrinkage δ_{csmcx} for middle strip is given by

$$\delta_{csmcx} = LDF_{1m} \times K3 \times SI_{csmx} \times L_x^2 \tag{4.684}$$

Side panel

Deflection due to shrinkage δ_{cscsx} for column strip is given by

$$\delta_{cscsx} = LDF_{1c} \times K3 \times SI_{cscx} \times L_x^2 \quad (4.685)$$

Deflection due to shrinkage δ_{csmsx} for column strip is given by

$$\delta_{csmsx} = LDF_{1m} \times K3 \times SI_{csmx} \times L_x^2 \quad (4.686)$$

y-y axis

Interior panel

Deflection due to shrinkage δ_{csciy} for column strip is given by

$$\delta_{csciy} = LDF_c \times K3 \times SI_{cscy} \times L_y^2 \quad (4.687)$$

Deflection due to shrinkage δ_{csmiy} for middle strip is given by

$$\delta_{csmiy} = LDF_m \times K3 \times SI_{csmy} \times L_y^2 \quad (4.688)$$

Corner panel

Deflection due to shrinkage δ_{csccy} for column strip is given by

$$\delta_{csccy} = LDF_{1c} \times K3 \times SI_{cscy} \times L_y^2 \quad (4.689)$$

Deflection due to shrinkage δ_{csmcy} for middle strip is given by

$$\delta_{csmcy} = LDF_{1m} \times K3 \times SI_{csmy} \times L_y^2 \quad (4.690)$$

Side panel

Deflection due to shrinkage δ_{cscsy} for column strip is given by

$$\delta_{cscsy} = LDF_{1c} \times K3 \times SI_{cscy} \times L_y^2 \quad (4.691)$$

Deflection due to shrinkage δ_{csmsy} for column strip is given by

$$\delta_{csmsy} = LDF_{1m} \times K3 \times SI_{csmy} \times L_y^2 \quad (4.692)$$

x-x axis

Interior panel

Deflection due to shrinkage δ_{csix} along shorter direction is given by

$$\delta_{csix} = \delta_{csc\,ix} + \delta_{csmiy} \tag{4.693}$$

Corner panel

Deflection due to shrinkage $\delta_{csc\,x}$ along shorter direction is given by

$$\delta_{csc\,x} = \delta_{csc\,cx} + \delta_{csmcy} \tag{4.694}$$

Side panel

Deflection due to shrinkage δ_{cssx} along shorter direction is given by

$$\delta_{cssx} = \delta_{csc\,sx} + \delta_{csmsy} \tag{4.695}$$

y-y axis

Interior panel

Deflection due to shrinkage δ_{csiy} along longer direction is given by

$$\delta_{csiy} = \delta_{csc\,iy} + \delta_{csmix} \tag{4.696}$$

Corner panel

Deflection due to shrinkage $\delta_{csc\,y}$ along longer direction is given by

$$\delta_{csc\,y} = \delta_{csc\,cy} + \delta_{csmcx} \tag{4.697}$$

Side panel

Deflection due to shrinkage δ_{cssy} along longer direction is given by

$$\delta_{cssy} = \delta_{csc\,sy} + \delta_{csmsx} \tag{4.698}$$

Long term deflection due to creep

Column strip

The modified young's modulus of concrete

$$E_{ce} = \frac{E_c}{(1+\theta)} \tag{4.699}$$

The modified modular ratio

$$m_{ce} = \frac{E_s}{E_{ce}}$$

The neutral axis depth for cracked stage x_{1cx} along shorter direction is computed as

$$0.5 \times 0.5 \times L_x \times x_{1cx}^2 + (m_{ce} \times A_{stcx}) = (m_{ce} \times A_{stcx} \times d) \tag{4.700}$$

The neutral axis depth for cracked stage x_{1cy} along longer direction is computed as

$$0.5 \times 0.5 \times L_y \times x_{1cy}^2 + (m_{ce} \times A_{stcy}) = (m_{ce} \times A_{stcy} \times d) \tag{4.701}$$

The lever arm for column strip Z_{1cx} along shorter direction

$$Z_{1cx} = d - (x_{1cx}/3) \tag{4.702}$$

The lever arm for column strip Z_{1cy} along longer direction

$$Z_{1cy} = d - (x_{1cy}/3) \tag{4.703}$$

The cracked moment of inertia

$$I_{r3cx} = \frac{0.5 \times L_x \times x_{1cx1}^3}{3} + \left(m_{ce} \times A_{stcx} \times (d - x_{1cx1})^2\right) \tag{4.704}$$

$$I_{r4cx} = \frac{0.5 \times L_x \times x_{1cx2}^3}{3} + \left(m_{ce} \times A_{stcx} \times (d - x_{1cx2})^2\right) \tag{4.705}$$

$$I_{r3cy} = \frac{0.5 \times L_y \times x_{1cy1}^3}{3} + \left(m_{ce} \times A_{stcy} \times (d - x_{1cy1})^2\right) \tag{4.706}$$

$$I_{r4cy} = \frac{0.5 \times L_y \times x_{1cy2}^3}{3} + \left(m_{ce} \times A_{stcy} \times (d - x_{1cy2})^2\right) \tag{4.707}$$

$$C_{2cx} = 1.2 - \left(\frac{M_{rcx}}{M_{operx}}\right) \times \left(\frac{Z_{1cx}}{d}\right) \times \left(1 - \frac{x_{1cx}}{d}\right) \tag{4.708}$$

$$C_{2cy} = 1.2 - \left(\frac{M_{rcy}}{M_{opery}}\right) \times \left(\frac{Z_{1cy}}{d}\right) \times \left(1 - \frac{x_{1cy}}{d}\right) \tag{4.709}$$

$$I_{2cx} = \frac{I_{r4cx}}{C_{2cx}} \tag{4.710}$$

If I_{2cx} is less than the I_{grcx} then the value of later is considered

$$I_{2cy} = \frac{I_{r4cy}}{C_{2cy}} \tag{4.711}$$

If I_{2cy} is less than the I_{grcy} then the value of later is considered

x-x axis

Interior panel

The creep deflection of column strip δ_{icccix}

$$\delta_{icccix} = \left(\frac{LDF_c \times W_d \times L_y \times L_x^4}{384 \times E_{ce} \times I_{2cx}}\right) \tag{4.712}$$

$$\delta_{ccpermcix} = \delta_{icccix} - \delta_{permcix} \tag{4.713}$$

Corner panel

$$\delta_{icccx} = \left(\frac{LDF_{1c} \times W_d \times L_y \times L_x^4}{384 \times E_{ce} \times I_{2cx}}\right) \tag{4.714}$$

$$\delta_{ccpermccx} = \delta_{icccx} - \delta_{permccx} \tag{4.715}$$

Side panel

$$\delta_{icccsx} = \left(\frac{LDF_{1c} \times W_d \times L_y \times L_x^4}{384 \times E_{ce} \times I_{2cx}}\right) \tag{4.716}$$

$$\delta_{ccpermcsx} = \delta_{icccsx} - \delta_{permcsx} \tag{4.717}$$

y-y axis

Interior panel

$$\delta_{iccciy} = \left(\frac{LDF_c \times W_d \times L_x \times L_y^4}{384 \times E_{ce} \times I_{2cy}} \right) \tag{4.718}$$

$$\delta_{ccpermciy} = \delta_{iccciy} - \delta_{permciy} \tag{4.719}$$

Corner panel

$$\delta_{icccy} = \left(\frac{LDF_{1c} \times W_d \times L_x \times L_y^4}{384 \times E_{ce} \times I_{2cy}} \right) \tag{4.720}$$

$$\delta_{ccpermccy} = \delta_{icccy} - \delta_{permccy} \tag{4.721}$$

Side panel

$$\delta_{icccsy} = \left(\frac{LDF_{1c} \times W_d \times L_x \times L_y^4}{384 \times E_{ce} \times I_{2cy}} \right) \tag{4.722}$$

$$\delta_{ccpermcsy} = \delta_{icccsy} - \delta_{permcsy} \tag{4.723}$$

Middle strip

$$0.5 \times 0.5 \times L_x \times x_{1mx}^2 + \left(m_{ce} \times A_{slmx}\right) = \left(m_{ce} \times A_{slmx} \times d\right) \tag{4.724}$$

$$0.5 \times 0.5 \times L_y \times x_{1my}^2 + \left(m_{ce} \times A_{slmy}\right) = \left(m_{ce} \times A_{slmy} \times d\right) \tag{4.725}$$

$$Z_{1mx} = d - \left(x_{1mx}/3\right) \tag{4.726}$$

$$Z_{1my} = d - \left(x_{1my}/3\right) \tag{4.727}$$

$$I_{r3mx} = \frac{0.5 \times L_x \times x_{1mx1}{}^3}{3} + \left(m_{ce} \times A_{slmx} \times \left(d - x_{1mx1}\right)^2\right) \tag{4.728}$$

$$I_{r4mx} = \frac{0.5 \times L_x \times x_{1mx2}{}^3}{3} + \left(m_{ce} \times A_{slmx} \times \left(d - x_{1mx2}\right)^2\right) \tag{4.729}$$

$$I_{r3my} = \frac{0.5 \times L_y \times x_{1my1}{}^3}{3} + \left(m_{ce} \times A_{slmy} \times \left(d - x_{1my1}\right)^2\right) \tag{4.730}$$

$$I_{r4my} = \frac{0.5 \times L_y \times x_{1my2}{}^3}{3} + \left(m_{ce} \times A_{slmy} \times \left(d - x_{1my2}\right)^2\right) \tag{4.731}$$

$$C_{2mx} = 1.2 - \left(\frac{M_{rmx}}{M_{operx}}\right) \times \left(\frac{Z_{1mx}}{d}\right) \times \left(1 - \frac{x_{1mx}}{d}\right) \tag{4.732}$$

$$C_{2my} = 1.2 - \left(\frac{M_{rmy}}{M_{opery}}\right) \times \left(\frac{Z_{1my}}{d}\right) \times \left(1 - \frac{x_{1my}}{d}\right) \tag{4.733}$$

$$I_{2mx} = \frac{I_{r4mx}}{C_{2mx}} \tag{4.734}$$

If I_{2mx} is less than the I_{grmx} then the value of later is considered

$$I_{2my} = \frac{I_{r4my}}{C_{2my}} \tag{4.735}$$

If I_{2my} is less than the I_{grmy} then the value of later is considered

Interior panel

$$\delta_{iccmix} = \left(\frac{LDF_m \times W_d \times L_y \times L_x^4}{384 \times E_{ce} \times I_{2mx}}\right) \tag{4.736}$$

$$\delta_{iccmiy} = \left(\frac{LDF_m \times W_d \times L_x \times L_y^4}{384 \times E_{ce} \times I_{2my}}\right) \tag{4.737}$$

Corner panel

$$\delta_{iccmcx} = \left(\frac{LDF_{1m} \times W_d \times L_y \times L_x^4}{384 \times E_{ce} \times I_{2mx}}\right) \tag{4.738}$$

$$\delta_{iccmcy} = \left(\frac{LDF_{1m} \times W_d \times L_x \times L_y^4}{384 \times E_{ce} \times I_{2my}}\right) \tag{4.739}$$

Side panel

$$\delta_{iccmsx} = \left(\frac{LDF_{1m} \times W_d \times L_y \times L_x^4}{384 \times E_{ce} \times I_{2mx}}\right) \tag{4.740}$$

$$\delta_{iccmsy} = \left(\frac{LDF_{1m} \times W_d \times L_x \times L_y^4}{384 \times E_{ce} \times I_{2my}}\right) \tag{4.741}$$

Interior panel

$$\delta_{ccpermmix} = \delta_{iccmix} - \delta_{permmix} \tag{4.742}$$

$$\delta_{ccpermmiy} = \delta_{iccmiy} - \delta_{permmiy} \tag{4.743}$$

Corner panel

$$\delta_{ccpermmcx} = \delta_{iccmcx} - \delta_{permmcx} \tag{4.744}$$

$$\delta_{ccpermmcy} = \delta_{iccmcy} - \delta_{permmcy} \tag{4.745}$$

Side panel

$$\delta_{ccpermmsx} = \delta_{iccmsx} - \delta_{permmsx} \tag{4.746}$$

$$\delta_{ccpermmsy} = \delta_{iccmsy} - \delta_{permmsy} \tag{4.747}$$

Interior panel

$$\delta_{ccpermix} = \delta_{ccpermcix} + \delta_{ccpermmiy} \tag{4.748}$$

$$\delta_{ccpermiy} = \delta_{ccpermciy} + \delta_{ccpermmix} \tag{4.749}$$

Corner panel

$$\delta_{ccpermcx} = \delta_{ccpermccx} + \delta_{ccpermmcy} \tag{4.750}$$

$$\delta_{ccpermcy} = \delta_{ccpermccy} + \delta_{ccpermmcx} \tag{4.751}$$

Side panel

$$\delta_{ccpermsx} = \delta_{ccpermcsx} + \delta_{ccpermmsy} \tag{4.752}$$

$$\delta_{ccpermsy} = \delta_{ccpermcsy} + \delta_{ccpermmsx} \tag{4.753}$$

Interior panel

$$\delta_{TLix} = \delta_{ix} + \delta_{ccpermix} + \delta_{csix} \tag{4.754}$$

$$\delta_{TLiy} = \delta_{iy} + \delta_{ccpermiy} + \delta_{csiy} \tag{4.755}$$

Corner panel

$$\delta_{TLcx} = \delta_{cx} + \delta_{ccpermcx} + \delta_{cscx} \quad (4.756)$$

$$\delta_{TLcy} = \delta_{cy} + \delta_{ccpermcy} + \delta_{cscy} \quad (4.757)$$

Side panel

$$\delta_{TLsx} = \delta_{sx} + \delta_{ccpermsx} + \delta_{cssx} \quad (4.758)$$

$$\delta_{TLsy} = \delta_{sy} + \delta_{ccpermsy} + \delta_{cssy} \quad (4.759)$$

Deflection

Interior panel

$$\delta_i = \frac{\delta_{TLix} + \delta_{TLiy}}{2} \quad (4.760)$$

$$\delta_{is} = \frac{\delta_{ix} + \delta_{iy}}{2} \quad (4.761)$$

$$\delta_{ish} = \frac{\delta_{csix} + \delta_{csiy}}{2} \quad (4.762)$$

$$\delta_{ic} = \frac{\delta_{ccpermix} + \delta_{ccpermiy}}{2} \quad (4.763)$$

Corner panel

$$\delta_c = \frac{(\delta_{TLcx} + \delta_{TLcy})}{2} + \left(\frac{(\delta_{t1x} - \delta_{d1x}) + (\delta_{t1y} - \delta_{d1y})}{2}\right) + \left(\frac{\delta_{d2x} + \delta_{d2y}}{2}\right) \quad (4.764)$$

$$\delta_{cs} = \frac{(\delta_{cx} + \delta_{cy})}{2} + \left(\frac{(\delta_{t1x} - \delta_{d1x}) + (\delta_{t1y} - \delta_{d1y})}{2}\right) \quad (4.765)$$

$$\delta_{csh} = \frac{(\delta_{cscx} + \delta_{cscy})}{2} \quad (4.766)$$

$$\delta_{cc} = \frac{(\delta_{ccpermcx} + \delta_{ccpermcy})}{2} + \left(\frac{\delta_{d2x} + \delta_{d2y}}{2}\right) \quad (4.767)$$

Side panel

$$\delta_s = \frac{(\delta_{TLsx} + \delta_{TLsy})}{2} + \left(\frac{(\delta_{t1y} - \delta_{d1y})}{2}\right) + \left(\frac{\delta_{d2y}}{2}\right) \quad (4.768)$$

$$\delta_{ss} = \frac{(\delta_{sx} + \delta_{sy})}{2} + \left(\frac{(\delta_{t1y} - \delta_{d1y})}{2}\right) \tag{4.769}$$

$$\delta_{ssh} = \frac{(\delta_{cssx} + \delta_{cssy})}{2} \tag{4.770}$$

$$\delta_{sc} = \frac{(\delta_{ccpermsx} + \delta_{ccpermsy})}{2} + \left(\frac{\delta_{d2y}}{2}\right) \tag{4.771}$$

Percentage contribution

Interior Panel

The contribution of short term deflection of interior panel

$$\%\delta_{ishort} = \left(\frac{\delta_{is}}{\delta_i}\right) \times 100 \tag{4.317}$$

The contribution of shrinkage deflection of interior panel

$$\%\delta_{ishr} = \left(\frac{\delta_{ish}}{\delta_i}\right) \times 100 \tag{4.318}$$

The contribution of creep deflection of interior panel

$$\%\delta_{icr} = \left(\frac{\delta_{ic}}{\delta_i}\right) \times 100 \tag{4.319}$$

The total contribution of creep and shrinkage effects

$$\%\delta_{il} = \%\delta_{ishr} + \%\delta_{icr} \tag{4.320}$$

Corner Panel

The contribution of short term deflection of corner panel

$$\%\delta_{cshort} = \left(\frac{\delta_{cs}}{\delta_c}\right) \times 100 \tag{4.321}$$

The contribution of shrinkage deflection of corner panel

$$\%\delta_{cshr} = \left(\frac{\delta_{csh}}{\delta_c}\right) \times 100 \tag{4.322}$$

The contribution of creep deflection of corner panel

$$\%\delta_{ccr} = \left(\frac{\delta_{cc}}{\delta_c}\right)\times 100 \tag{4.323}$$

The total contribution of creep and shrinkage effects

$$\%\delta_{cl} = \%\delta_{cshr} + \%\delta_{ccr} \tag{4.324}$$

Side Panel

The contribution of short term deflection of side panel

$$\%\delta_{sshort} = \left(\frac{\delta_{ss}}{\delta_s}\right)\times 100 \tag{4.325}$$

The contribution of shrinkage deflection of side panel

$$\%\delta_{sshr} = \left(\frac{\delta_{ssh}}{\delta_s}\right)\times 100 \tag{4.326}$$

The contribution of creep deflection of side panel

$$\%\delta_{scr} = \left(\frac{\delta_{sc}}{\delta_s}\right)\times 100 \tag{4.327}$$

The total contribution of creep and shrinkage effects

$$\%\delta_{sl} = \%\delta_{sshr} + \%\delta_{scr} \tag{4.328}$$

4.6 SUMMARY

To find the required depth of slab to meet the serviceability requirements, the available empirical formulas as per the four codes such as ACI: 318, BS: 8110, EC: 2-2002 and IS: 456-2000 were studied with suitable modification by using the modelling explained in Chapter 3. Since hand calculation is difficult by using theses equations a computer program in MATLAB was developed. An extensive parametric study was conducted using the developed programme. The development of the program and the parametric study made using the program are described in the next chapter.

CHAPTER 5

PROPOSED APPROACHES FOR DEFLECTION CALCULATION

5.1 GENERAL

The fourth chapter explained the empirical relations studied as per the four codes ACI: 318-2002, BS: 8110-1997, EC: 2-2002 and IS: 456-2000, to calculate the deflection including the creep and shrinkage effects, with the suitable modification incorporating the two way behavior of the slab. It can be seen that the number of equations involved and the assumptions made then and there makes the hand calculations difficult. Also it consumes lot of time. Since the main aim of this study was to assess the influence of each parameter in total deflection of flat plates, a computer program was required.

This chapter briefly explains the development of computer programs and the parametric study conducted using these programs. The influence of each parameter in the total deflection is explained in detail in this chapter.

5.2 DEVELOPMENT OF THE PROGRAM

A program for the empirical equations studied (with suitable modifications) explained in the Chapter 4 was developed. For this the MATLAB 6.5 version was used. The following sections explain briefly about the developed program.

5.2.1 ACI: 318-2002

The multiplier approach is followed in this code. A program in MATLAB 6.5 (Rudrapratap, [2002], Chapman [2002]) was developed for the equations 4.1 to 4.135. The input parameters of the program were the span (m), live load (kN/m^2) and grade of concrete (N/mm^2). Depth of the slab was varied with an increment of 5 mm from

125 mm to 300 mm. The output of this program gives the total mid-panel deflection of the flat plate, the deflection due to sustained load and the deflection due to varying part of live load for the interior panel, corner panel & side panel (mm). Also the output will give the percentage contribution of sustained load and varying part of live load in the total deflection.

5.2.2 BS: 8110-1997

This code gives a detailed method for predicting the deflection due to creep and shrinkage. This uses the moment curvature relationship for shrinkage and creep deflection. The tension stiffening effect of concrete is also considered. The equations from 4.3.1 to 4.3.232 were used for the program in MATLAB. The input parameters of the program were span (m), grade of concrete (N/mm^2), modulus of elasticity of concrete (N/mm^2), grade of steel (N/mm^2), clear cover to reinforcement (mm) and depth (125 mm to 300 mm with an increment of 5 mm). The program will give the total mid panel deflection of the interior panel, corner panel & side panel (mm) as output. In this the total deflection (mm), short term deflection (mm), shrinkage deflection (mm), creep deflection (mm) and the percentage contribution of each deflection in total deflection are obtained as output.

5.2.3 EC: 2-2002

This code gives an elaborate method for predicting the deflection due to creep and shrinkage. It is based on the moment curvature relationship. A factor is introduced to consider the effect of cracked stage of concrete. Also the tension stiffening effect of concrete is taken into account. A program was developed for the modified equations from 4.4.1 to 4.4.253. The input parameters of the program were span (m), grade of concrete (N/mm^2), modulus of elasticity of concrete (N/mm^2), grade of steel (N/mm^2), clear cover to reinforcement (mm) and depth (125 mm to 300 mm with an increment of 5 mm). This program will give the total mid panel deflection of interior panel, corner panel & side panel (mm) as output. In this the total deflection (mm), short term deflection (mm), shrinkage deflection (mm), creep deflection (mm) and the percentage contribution of the each deflection in total deflection are obtained as output.

5.2.4 IS: 456-2000

This code also suggests a detailed procedure for deflection due to creep and shrinkage. A program was developed in MATLAB for the equations 4.5.1 to 4.5.256 explained in Chapter 4. The input parameters of the program were span (m), grade of concrete (N/mm^2), grade of steel (N/mm^2), clear cover to reinforcement (mm) and depth (125 mm to 300 mm with an increment of 5 mm). The program will give the mid panel deflection of interior panel, corner panel & side panel (mm). In this the total deflection (mm), short term deflection (mm), shrinkage deflection (mm), creep deflection (mm) and the percentage contribution of the above deflections in total deflection were obtained as output.

5.3 PARAMETRIC STUDY

An extensive parametric study was conducted by using the above developed programs. The following sections explain the effect of different parameters in total deflection based on the study made. The floor plan considered for this study is shown in Fig. 5.1.

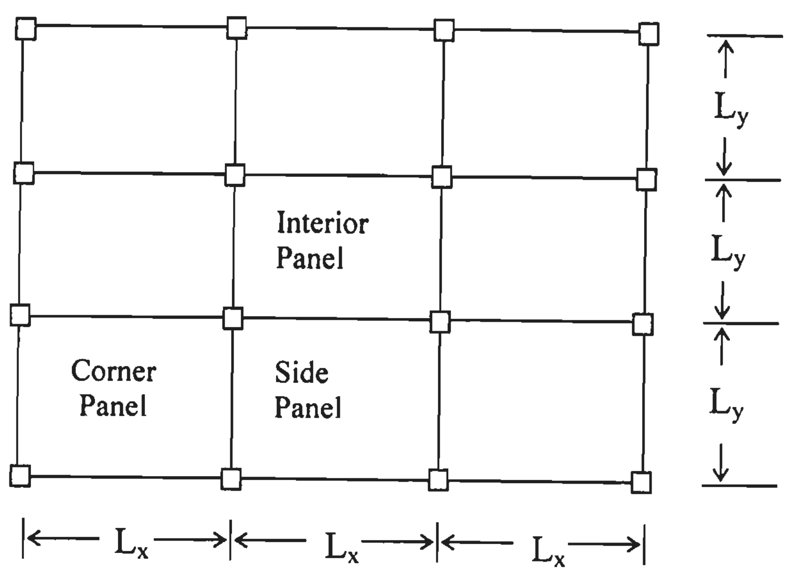


Fig. 5.1 Typical RC Floor plan

The following parameters were considered.

Span	:	6 m to 12 m (Medium floor plans - mainly for apartments and short span office buildings)
Materials	:	Concrete M 20, M 25 & M 30
	:	Steel Fe 415 (normally used in India)
Area of steel	:	0.5% & 1%
Clear cover	:	20 mm, 25 mm & 30 mm. (Based on durability and fire resistance as per IS: 456-2000)
Superimposed load	:	3 kN/m^2, 4 kN/m^2 & 5 kN/m^2 (The maximum gravity loading of which is used in passages as per IS: 875).
Panels	:	Interior panel, corner panel & side panel.
Codes	:	ACI: 318-2002, BS: 8110-1997, EC: 2-2002 & IS: 456-2000.

By using the developed programs for the above parameters, the total midpanel deflections of flat plates for interior panel, corner panel and side panel were computed as per the above codes. The influence of each parameter in total deflection was determined and discussed in the following sections.

5.4 ACI: 318-2002

In this code method the multiplier approach is used in which the parameters considered are only the span, load and grade of concrete. The gross moment of inertia is taken into account for flexural rigidity. Tables 5.1 to 5.3 give the deflection of flat plate for the span of 6 m x 7 m. The influence of the above three parameters were studied and discussed below.

Effect of Span

The increase in span, by 1 square meter increases the deflection by 65.3%.

The span has major influence in increasing the total deflection of flat plates. Figure 5.2 shows the influence of the deflection for different floor sizes.

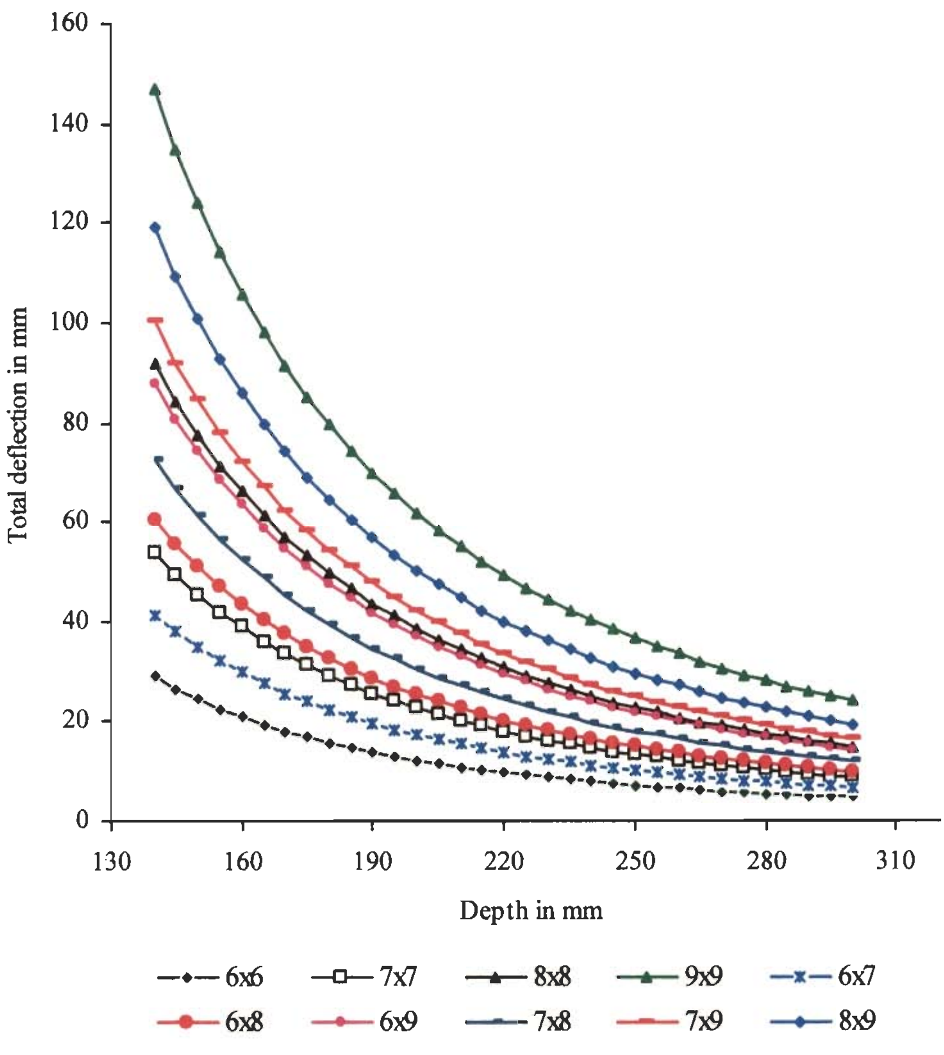


Fig. 5.2 Effect of span on total deflection (ACI: 318-2002)

Effect of grade of concrete

The influence of concrete grade was studied for three grades of concrete such as f_{ck} = 20 N/mm^2, f_{ck} = 25 N/mm^2 and f_{ck} = 30 N/mm^2 by keeping the other parameters constant.

The deflection of flat plate (Fig. 5.3) for f_{ck} = 20 N/mm^2 is 19.42 mm and for f_{ck} = 25 N/mm^2 is 17.37 mm which is 10.56% less than that for f_{ck} = 20 N/mm^2. If the concrete grade of f_{ck} = 30 N/mm^2 is used the deflection is reduced by 8.69% compared to the deflection for f_{ck} = 25 N/mm^2. The grade of concrete has much influence in reducing the deflection.

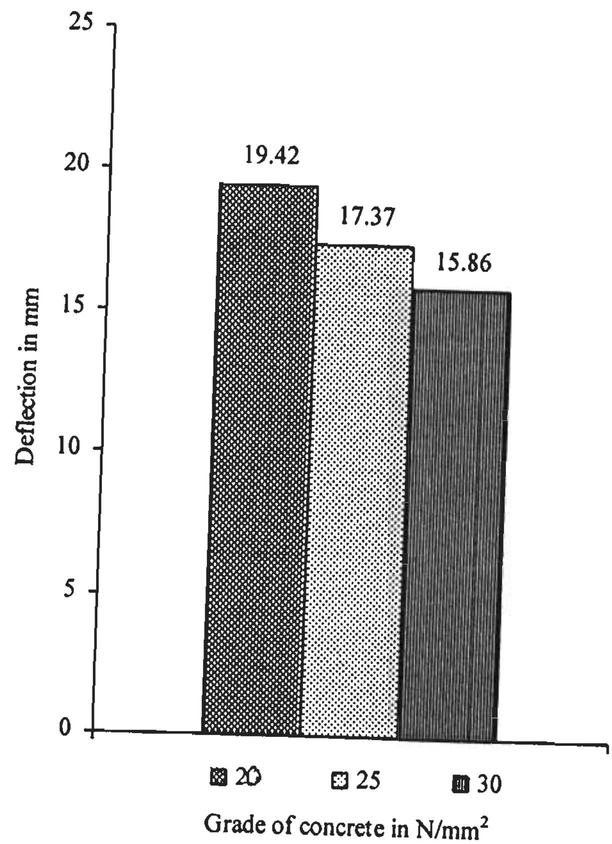


Fig. 5.3 Effect of grade of concrete on total deflection (ACI: 318-2002)

Effect of live load

The influence of live load in the deflection was studied and the total deflection is plotted in the Fig. 5.4. It is observed that the deflection increases by 7.13% when the live load is increased from 3 kN/m^2 to 4 kN/m^2. And if the load is increased from 4 kN/m^2 to 5 kN/m^2, the increase in deflection is 6.65%.

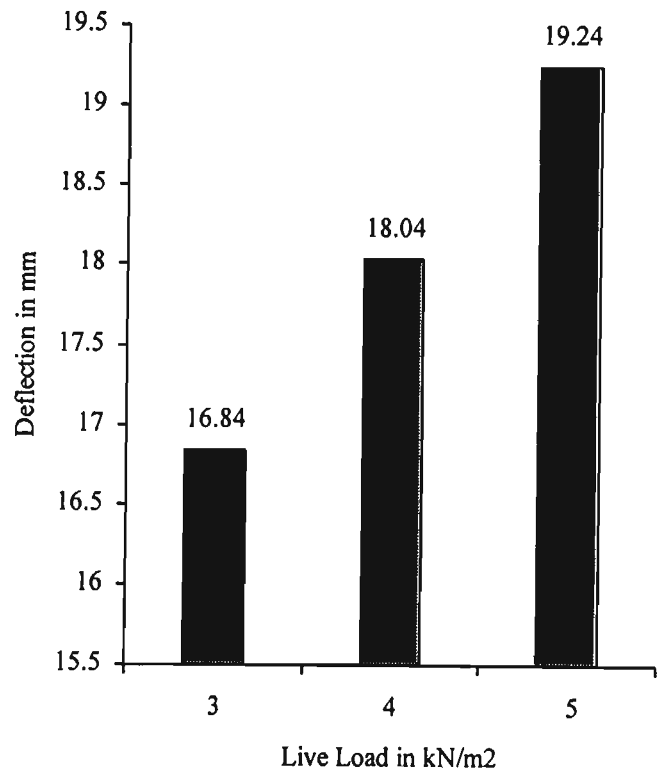
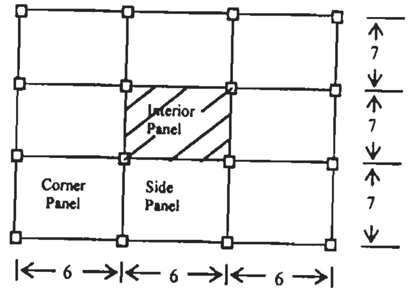


Fig. 5.4 Effect of live load on total deflection (ACI: 318-2002)

Table 5.1 Deflection of plate (Interior panel)
(6 m x 7 m – as per ACI: 318-2002)



Structural Details

Floor Plan Size (m) : 6 x 7

Column Size (mm) : 300 x 300

Storey Height (m) : 3.5

Materials Used : M 20 concrete
Fe 415 steel

Load (kN/m^2): (i) Live Load : 3
(ii) Floor Finish: 1.5

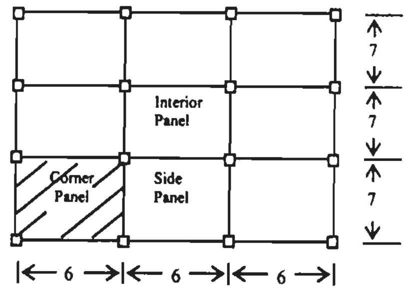
Permissible deflection : Span/240 or 20 mm whichever is less
(ACI: 318-2002)

Total thickness in mm	Total deflection in mm	Deflection due to dead load in mm	Deflection due to live load in mm	Percentage contribution of dead load in Total Deflection	Percentage contribution of live load in Total Deflection	Percentage of balanced capacity
195	18.11	15.88	2.23	87.69	12.31	90.55
200	17.04	14.98	2.07	87.88	12.12	85.20
205	16.07	14.15	1.92	88.06	11.94	80.35
210	15.17	13.38	1.78	88.24	11.76	75.85
215	14.34	12.68	1.66	88.41	11.59	71.70
220	13.58	12.03	1.55	88.57	11.43	67.90
225	12.88	11.43	1.45	88.73	11.27	64.40
230	12.22	10.87	1.36	88.89	11.11	61.10
235	11.62	10.35	1.27	89.04	10.96	58.10
240	11.06	9.86	1.20	89.19	10.81	55.30
245	10.54	9.41	1.12	89.33	10.67	52.70
250	10.05	8.99	1.06	89.47	10.53	50.25
255	9.59	8.60	1.00	89.61	10.39	47.95
260	9.17	8.23	0.94	89.74	10.26	45.85
265	8.77	7.88	0.89	89.87	10.13	43.85
270	8.40	7.56	0.84	90.00	10.00	42.00
275	8.05	7.25	0.79	90.12	9.88	40.25
280	7.72	6.96	0.75	90.24	9.76	38.60
285	7.41	6.69	0.71	90.36	9.64	37.05
290	7.12	6.44	0.68	90.48	9.52	35.60
295	6.84	6.20	0.64	90.59	9.41	34.20
300	6.58	5.97	0.61	90.70	9.30	32.90

Table 5.2 Deflection of plate (Corner panel)
(6 m x 7 m – as per ACI: 318-2002)

Structural Details

Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5



Materials Used : M 20 concrete
Fe 415 steel

Load (kN/m^2): (i) Live Load: 3
(ii) Floor Finish: 1.5

Permissible deflection : Span/240 or 20 mm whichever is less (ACI: 318-2002)

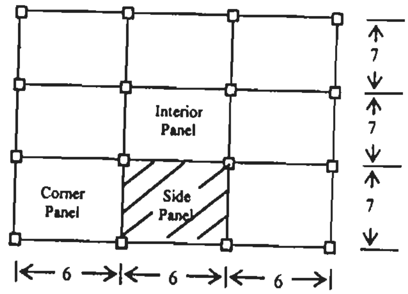
Total thickness in mm	Total deflection in mm	Deflection due to dead load in mm	Deflection due to live load in mm	Percentage contribution of dead load in Total Deflection	Percentage contribution of live load in Total Deflection	Percentage of balanced capacity
200	19.11	17.07	2.04	89.33	10.67	95.55
205	18.00	16.11	1.89	89.48	10.52	90.00
210	16.98	15.22	1.76	89.62	10.38	84.90
215	16.05	14.40	1.64	89.77	10.23	80.25
220	15.18	13.65	1.53	89.90	10.10	75.90
225	14.39	12.95	1.43	90.04	9.96	71.95
230	13.65	12.31	1.34	90.17	9.83	68.25
235	12.96	11.71	1.26	90.30	9.70	64.80
240	12.33	11.15	1.18	90.42	9.58	61.65
245	11.74	10.63	1.11	90.55	9.45	58.70
250	11.19	10.14	1.04	90.66	9.34	55.95
255	10.67	9.69	0.98	90.78	9.22	53.35
260	10.20	9.27	0.93	90.89	9.11	51.00
265	9.75	8.87	0.88	91.00	9.00	48.75
270	9.33	8.50	0.83	91.11	8.89	46.65
275	8.93	8.15	0.78	91.21	8.79	44.65
280	8.56	7.82	0.74	91.32	8.68	42.80
285	8.21	7.51	0.71	91.42	8.58	41.05
290	7.89	7.22	0.67	91.51	8.49	39.45
295	7.58	6.94	0.64	91.61	8.39	37.90
300	7.29	6.68	0.60	91.70	8.30	36.45

Table 5.3 Deflection of plate (Side panel)
(6 m x 7 m – as per ACI: 318-2002)

Structural Details

Floor Plan Size (m) : 6 x 7
Column Size (mm) : 300 x 300
Storey Height (m) : 3.5

Materials Used : M 20 concrete
Fe 415 steel



Load (kN/m^2): (i) Live Load : 3
(ii) Floor Finish: 1.5

Permissible deflection: Span/240 or 20 mm whichever is less (ACI: 318-2002)

Total thickness in mm	Total deflection in mm	Deflection due to dead load in mm	Deflection due to live load in mm	Percentage contribution of dead load in total deflection	Percentage contribution of live load in total deflection	Percentage of balanced capacity
195	19.32	17.11	2.20	88.60	11.40	96.60
200	18.17	16.13	2.04	88.77	11.23	90.85
205	17.12	15.22	1.89	88.93	11.07	85.60
210	16.15	14.39	1.76	89.09	10.91	80.75
215	15.27	13.63	1.64	89.24	10.76	76.35
220	14.45	12.92	1.53	89.39	10.61	72.25
225	13.69	12.26	1.43	89.54	10.46	68.45
230	13.00	11.65	1.34	89.68	10.32	65.00
235	12.35	11.09	1.26	89.82	10.18	61.75
240	11.75	10.56	1.18	89.95	10.05	58.75
245	11.19	10.08	1.11	90.08	9.92	55.95
250	10.66	9.62	1.04	90.21	9.79	53.30
255	10.18	9.19	0.98	90.33	9.67	50.90
260	9.72	8.79	0.93	90.45	9.55	48.60
265	9.30	8.42	0.88	90.57	9.43	46.50
270	8.90	8.07	0.83	90.68	9.32	44.50
275	8.52	7.74	0.78	90.79	9.21	42.60
280	8.17	7.43	0.74	90.90	9.10	40.85
285	7.84	7.14	0.71	91.01	8.99	39.20
290	7.53	6.86	0.67	91.11	8.89	37.65
295	7.24	6.60	0.64	91.21	8.79	36.20
300	6.96	6.35	0.60	91.31	8.69	34.80

5.5 BS: 8110-1997

In this code method the moment curvature method is used in which all the parameters are considered. Tables 5.4 to 5.6 give the deflection of flat plate for the span of 6 m x 7 m. The influence of the parameters were studied and discussed below.

Grade of concrete

By keeping the other parameters constant, by changing the grade of concrete, the deflection of the slab meeting the limit on deflection for the 170 mm depth is 18.84 mm, 18.14 mm and 17.84 mm (Figure 5.5) for 20, 25 and 30 grades concrete.

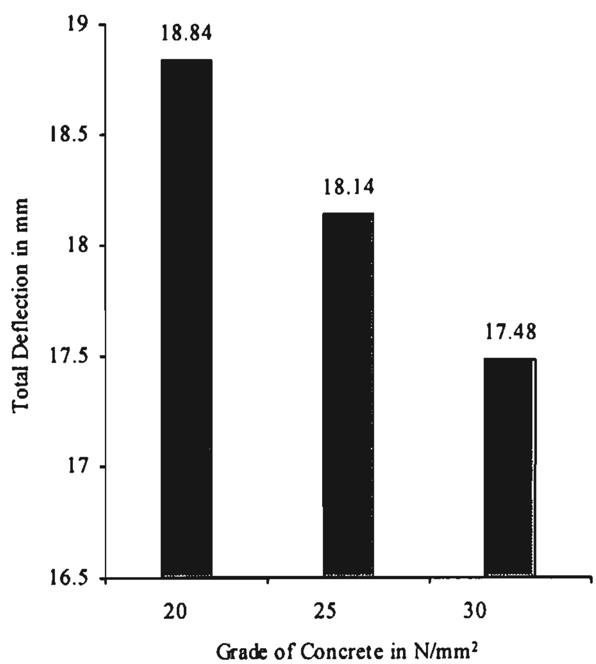


Fig. 5.5 Effect of grade of concrete on total deflection (BS: 8110-1997)

The percentage reduction of deflection is 3.72% and 7.22% for 25 grade and 30 grade concrete over 20 grade concrete. The percentage contribution of creep and shrinkage in total deflection is 82.37%, 82.36% and 82.35%. Hence the influence of grade of concrete in deflection is approximately 3.7% by increasing the grade by 5 N/mm^2.

Clear cover to reinforcement

Increase in clear cover by 5 mm increases the total deflection on an average of 0.75% (Fig. 5.6). This increase in deflection is negligible. Hence this parameter does not have significant influence in deflection of flat plates as per this code.

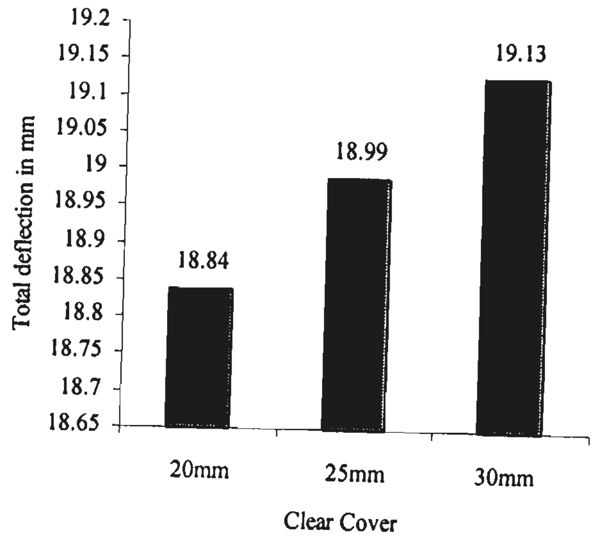


Fig. 5.6 Effect of clear cover on total deflection (BS: 8110-1997)

Live load

The increase in live load by 1 kN/m^2 increases the deflection by approximately 7 to 8.2% (Figure 5.7). This has significant influence in total deflection. But the code does not give limitation based on live load.

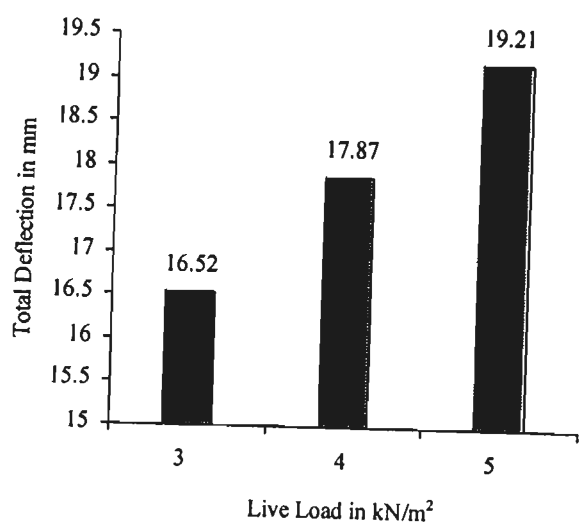


Fig. 5.7 Effect of live load on total deflection (BS: 8110-1997)

Percentage of tension reinforcement

The increase in percentage of tension reinforcement from 0.5% to 1.0% increases the total deflection by 4.35% (Fig. 5.8). For example the deflection for 170mm depth of slab for 0.5% to 1.0% tension reinforcement the total deflection is 18.84 mm and 19.66 mm respectively. But the increase in total deflection is due to the shrinkage effect. By increasing the percentage of reinforcement reduces the short term and creep deflection. Since the shrinkage deflection is increasing the contribution of creep and shrinkage increases from 82.37% to 84.49%. This shrinkage effect increase is due to the fact that the slab strip is considered as the singly reinforced section and due to the large area exposed to the atmosphere. Hence the early drying shrinkage will be more.

Creep coefficient

The creep coefficient method is adopted for the determination of creep deflection. The creep coefficient for 28 days age at loading and 1 year age at loading as per BS: 8110-1997 was considered. The creep coefficient for the above age at loading is 3.1 and 1.6 respectively. For 170 mm depth of slab the total deflection for 28 days age at loading is 18.84 mm and 14.35 mm for 1-year age at loading (Figure 5.9). The percentage reduction in deflection is 23.83%. The contribution of creep and shrinkage in total deflection is 76.84% as against the 86.25% for 28 days. Here the deflection is less at one year against the deflection at 28 days. This may be due to the fact that the concrete gains its full strength at one year when compared to the early stage. This is true that during the early stage of construction and during the construction the floor is more vulnerable for deflection.

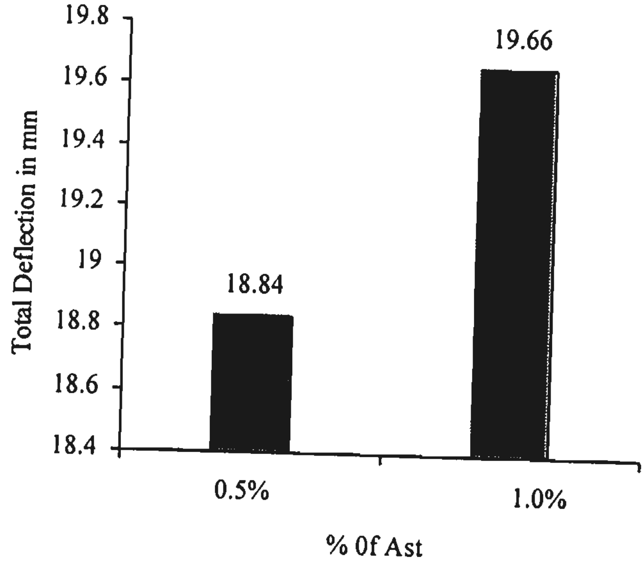


Fig. 5.8 Effect of percentage of reinforcement on total deflection (BS: 8110-1997)

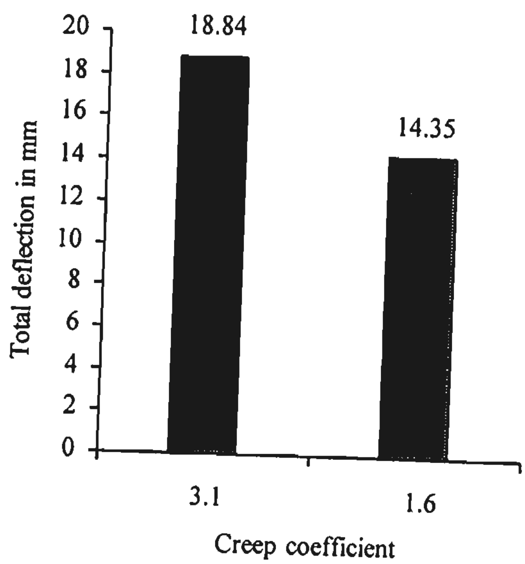
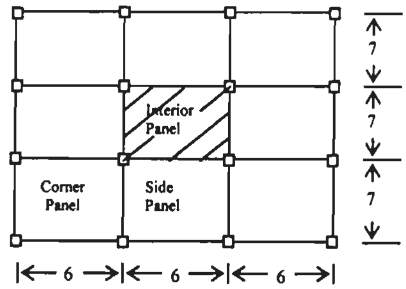


Fig. 5.9 Effect of creep coefficient on total deflection (BS: 8110-1997)

Table 5.4 Deflection of plate (Interior panel)
(6 m x 7 m – as per BS: 8110-1997)



Structural Details

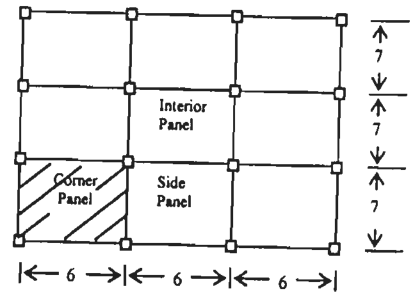
Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5
Materials Used		:	M 20 concrete Fe 415 steel
Load	(kN/m^2):		(i) Live Load : 3 (ii) Floor Finish: 1.5
Permissible deflection		:	Span/250 or 20 mm whichever is less (BS: 8110-1997)

Total thick ness in mm	Total deflection in mm	Short term deflection in mm	Shrinkage deflection in mm	Creep deflection in mm	% contribution of short term in total deflection	% contribution of shrinkage in total deflection	% contribution of creep in total deflection	% contribution of creep & shrinkage	% of balanced capacity
185	19.29	2.35	2.58	14.36	12.20	13.36	74.44	87.80	96.45
190	18.19	2.17	2.52	13.49	11.95	13.86	74.19	88.05	90.95
195	17.18	2.01	2.47	12.70	11.71	14.36	73.93	88.29	85.90
200	16.26	1.87	2.42	11.98	11.48	14.86	73.66	88.52	81.30
205	15.42	1.73	2.37	11.31	11.25	15.36	73.39	88.75	77.10
210	14.64	1.61	2.32	10.70	11.03	15.85	73.12	88.97	73.20
215	13.92	1.51	2.27	10.14	10.82	16.34	72.84	89.18	69.60
220	13.25	1.41	2.23	9.62	10.61	16.83	72.56	89.39	66.25
225	12.64	1.31	2.19	9.13	10.41	17.32	72.27	89.59	63.20
230	12.06	1.23	2.15	8.68	10.21	17.80	71.99	89.79	60.30
235	11.53	1.16	2.11	8.27	10.02	18.28	71.70	89.98	57.65
240	11.03	1.08	2.07	7.88	9.83	18.76	71.41	90.17	55.15
245	10.57	1.02	2.03	7.52	9.65	19.24	71.11	90.35	52.85
250	10.14	0.96	2.00	7.18	9.47	19.71	70.82	90.53	50.70
255	9.73	0.91	1.96	6.87	9.30	20.18	70.52	90.70	48.65
260	9.36	0.85	1.93	6.57	9.14	20.64	70.22	90.86	46.80
265	9.00	0.81	1.90	6.29	8.97	21.10	69.93	91.03	45.00
270	8.66	0.76	1.87	6.03	8.82	21.56	69.63	91.18	43.30
275	8.35	0.72	1.84	5.79	8.66	22.01	69.33	91.34	41.75
280	8.05	0.69	1.81	5.56	8.51	22.46	69.03	91.49	40.25
285	7.77	0.65	1.78	5.34	8.36	22.91	68.73	91.64	38.85
290	7.51	0.62	1.75	5.14	8.22	23.35	68.43	91.78	37.55
295	7.26	0.59	1.73	4.94	8.08	23.79	68.12	91.92	36.30
300	7.02	0.56	1.70	4.76	7.95	24.23	67.82	92.05	35.10

Table 5.5 Deflection of plate (Corner panel)
(6 m x 7 m – as per BS: 8110-1997)

Structural Details

Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5
Materials Used		:	M 20 concrete Fe 415 steel
Load	(kN/m^2):		(i) Live Load: 3 (ii) Floor Finish: 1.5
Permissible deflection		:	Span/250 or 20 mm whichever is less (BS: 8110-1997)

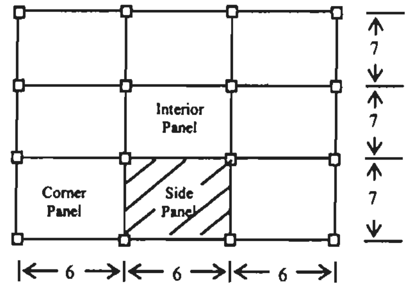


Total thick ness in mm	Total deflection in mm	Short term deflection in mm	Shrinkage deflection in mm	Creep deflection in mm	% contribution of short term in total deflection	% contribution of shrinkage in total deflection	% contribution of creep in total deflection	% contribution of creep & shrinkage	% as balanced capacity
195	18.95	2.21	2.76	13.98	11.65	14.58	73.78	88.35	94.75
200	17.94	2.05	2.71	13.19	11.41	15.08	73.51	88.59	89.70
205	17.01	1.90	2.65	12.46	11.17	15.59	73.24	88.83	85.05
210	16.15	1.77	2.60	11.79	10.94	16.09	72.97	89.06	80.75
215	15.36	1.65	2.55	11.17	10.72	16.59	72.69	89.28	76.80
220	14.63	1.54	2.50	10.59	10.51	17.08	72.41	89.49	73.15
225	13.95	1.44	2.45	10.06	10.30	17.57	72.13	89.70	69.75
230	13.32	1.34	2.41	9.57	10.09	18.06	71.84	89.91	66.60
235	12.74	1.26	2.36	9.11	9.90	18.55	71.55	90.10	63.70
240	12.19	1.18	2.32	8.69	9.70	19.03	71.26	90.30	60.95
245	11.68	1.11	2.28	8.29	9.52	19.51	70.97	90.48	58.40
250	11.20	1.05	2.24	7.92	9.33	19.99	70.67	90.67	56.00
255	10.76	0.99	2.20	7.57	9.16	20.47	70.38	90.84	53.80
260	10.34	0.93	2.16	7.25	8.98	20.94	70.08	91.02	51.70
265	9.95	0.88	2.13	6.94	8.82	21.40	69.78	91.18	49.75
270	9.58	0.83	2.09	6.66	8.65	21.87	69.48	91.35	47.90
275	9.23	0.78	2.06	6.39	8.49	22.33	69.18	91.51	46.15
280	8.90	0.74	2.03	6.13	8.34	22.78	68.88	91.66	44.50
285	8.60	0.70	2.00	5.89	8.19	23.23	68.58	91.81	43.00
290	8.30	0.67	1.97	5.67	8.04	23.68	68.28	91.96	41.50
295	8.03	0.63	1.94	5.46	7.90	24.13	67.98	92.10	40.15
300	7.77	0.60	1.91	5.26	7.76	24.57	67.68	92.24	38.85

Table 5.6 Deflection of plate (Side panel)
(6 m x 7 m – as per BS: 8110-1997)

Structural Details

Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5
Materials Used		:	M 20 concrete Fe 415 steel
Load	(kN/m^2):		(i) Live Load : 3 (ii) Floor Finish: 1.5
Permissible deflection:			Span/250 or 20 mm whichever is less (BS: 8110-1997)



Total thick ness in mm	Total deflection in mm	Short term deflection in mm	Shrinkage deflection in mm	Creep deflection in mm	% contribution of short term in total deflection	% contribution of shrinkage in total deflection	% contribution of creep in total deflection	% contribution of creep & shrinkage	% as balanced capacity
190	19.22	2.11	2.82	14.35	10.98	14.68	74.66	89.34	96.10
195	18.16	1.95	2.76	13.51	10.74	15.21	74.39	89.60	90.80
200	17.19	1.81	2.71	12.74	10.50	15.74	74.11	89.85	85.95
205	16.30	1.67	2.65	12.04	10.27	16.26	73.83	90.09	81.50
210	15.48	1.56	2.60	11.39	10.05	16.78	73.54	90.33	77.40
215	14.72	1.45	2.55	10.79	9.83	17.30	73.26	90.56	73.60
220	14.02	1.35	2.50	10.23	9.62	17.82	72.96	90.78	70.10
225	13.38	1.26	2.45	9.72	9.42	18.33	72.67	91.00	66.90
230	12.77	1.18	2.41	9.24	9.22	18.84	72.37	91.21	63.85
235	12.21	1.10	2.36	8.80	9.03	19.35	72.07	91.42	61.05
240	11.69	1.03	2.32	8.39	8.84	19.85	71.77	91.62	58.45
245	11.20	0.97	2.28	8.01	8.66	20.35	71.46	91.81	56.00
250	10.75	0.91	2.24	7.65	8.48	20.84	71.16	92.00	53.75
255	10.32	0.86	2.20	7.31	8.31	21.34	70.85	92.19	51.60
260	9.92	0.81	2.16	7.00	8.14	21.83	70.54	92.37	49.60
265	9.54	0.76	2.13	6.70	7.98	22.31	70.23	92.55	47.70
270	9.19	0.72	2.09	6.43	7.82	22.79	69.92	92.72	45.95
275	8.86	0.68	2.06	6.17	7.67	23.27	69.61	92.88	44.30
280	8.54	0.64	2.03	5.92	7.52	23.74	69.30	93.05	42.70
285	8.25	0.61	2.00	5.69	7.37	24.21	68.99	93.20	41.25
290	7.97	0.58	1.97	5.47	7.23	24.68	68.68	93.36	39.85
295	7.70	0.55	1.94	5.27	7.09	25.14	68.37	93.51	38.50
300	7.45	0.52	1.91	5.07	6.96	25.60	68.06	93.66	37.25

5.6 IS: 456-2000

In this code method the moment curvature method is used in which all the parameters are considered. Tables 5.8 to 5.10 give the deflection of flat plate for the span of 6 m x 7 m. The influence of the parameters were studied and discussed below.

Effect of span

As per this code the depth required for interior panels based on limit on deflection for the square and rectangular plan keeping the parameters f_{ck} = 20 kN/m^2, Ast = 0.5%, LL = 3 kN/m^2, cc = 20 mm and Φ = 1.6 and changing only the span for the interior panel are furnished in Table 5.7.

Table 5.7 Depth required for flat plate as per IS: 456-2000

S. No.	Span	Depth required in mm
1.	6x6	210
2.	7x7	265
3.	8x8	>300
4.	9x9	>300
5.	6x7	240
6.	6x8	265
7.	6x9	300
8.	7x8	300
9.	7x9	>300
10.	8x9	>300

Since the depth for the spans such as 8x8, 9x9, 6x9, 7x8 and 7x9 are equal to or more than 300 mm, those floor plans were not considered for the parametric study.

Effect of grade of concrete

As per this code, the depth required based on limit of deflection is minimum 210 mm. The deflections for the different grades of concrete are plotted in Fig. 5.10.

The total deflection is 19.89 mm, 19.05 mm and 18.37 mm for the f_{ck} = 20 N/mm^2, f_{ck} = 25 N/mm^2 and f_{ck} = 30 N/mm^2 respectively. The percentage reduction in deflection is 4.22% and 3.57% if the grade of concrete is increased from 20 to 25, 25 to 30 respectively. Here the reduction in deflection is considerable. The grade of concrete is effective in reducing the deflection.

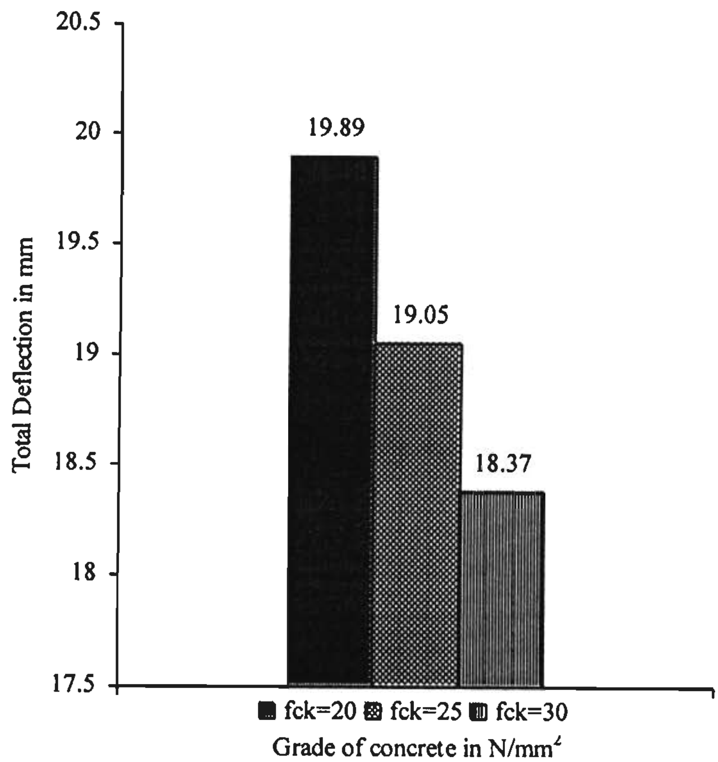


Fig. 5.10 Effect of grade of concrete on total deflection (IS: 456-2000)

Effect of clear cover

Increase in clear cover by 5 mm increases the deflection by 5.11% and 5.35% for 25 mm and 30 mm clear cover (Fig. 5.11) when the clear cover was increased from 20 mm and 25mm respectively. Hence increase in clear cover by 5 mm increases the deflection on an average of 5%.

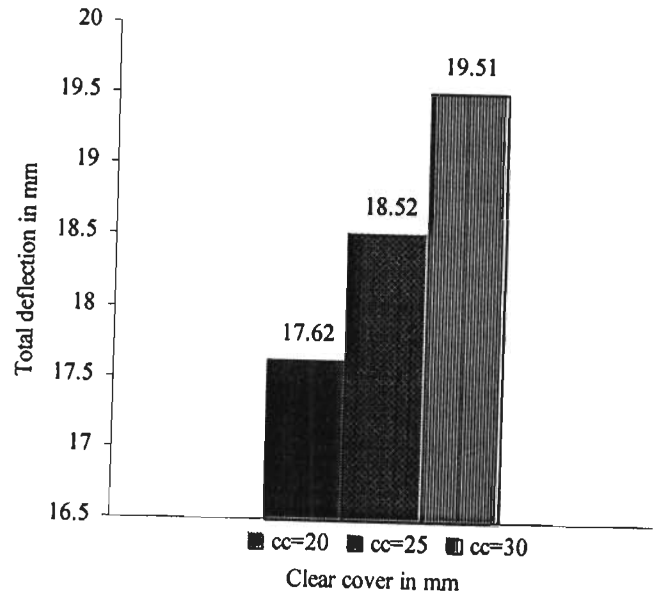


Fig. 5.11 Effect of clear cover on total deflection (IS: 456-2000)

Effect of live load

The influence of live load was studied for the live loads of 3 kN/m^2, 4 kN/m^2 and 5 kN/m^2 and the deflection for 225 mm depth are plotted in the Fig. 5.12. The deflections for the above loads are 16.62 mm, 18 mm and 19.38 mm respectively. The increase in deflection is 8.3% and 7.67% for the live load of 4 kN/m^2 and 5 kN/m^2, when increased from 3 kN/m^2 and 4 kN/m^2. The increase in load by 1 kN/m^2 increases the deflection considerably.

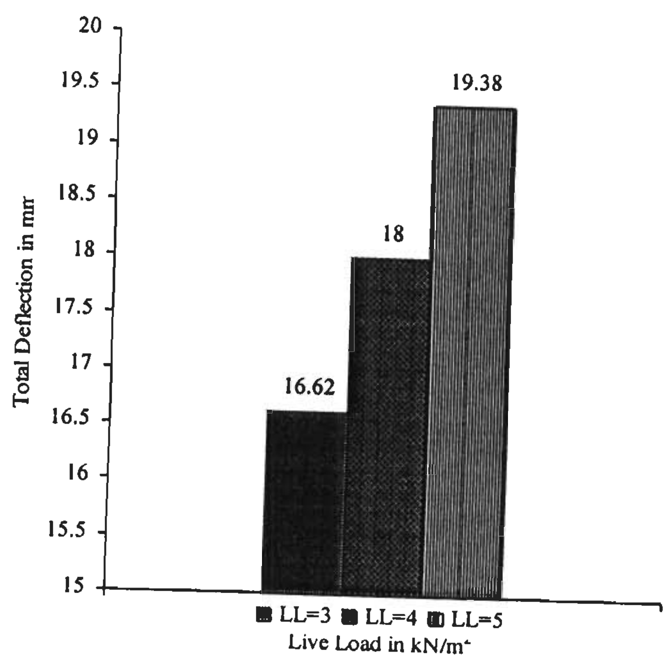


Fig. 5.12 Effect of live load on total deflection (IS: 456-2000)

Effect of percentage of steel

Increase in percentage of steel, decreases the deflection. The deflection for a depth of 210 mm for 0.5% of steel is 19.89 mm and for 1.0% steel is 15.99 mm. The decrease in deflection is 19.61% (Fig. 5.13). The formula uses the shrinkage curvature method that depends on the support conditions and percentage of steel.

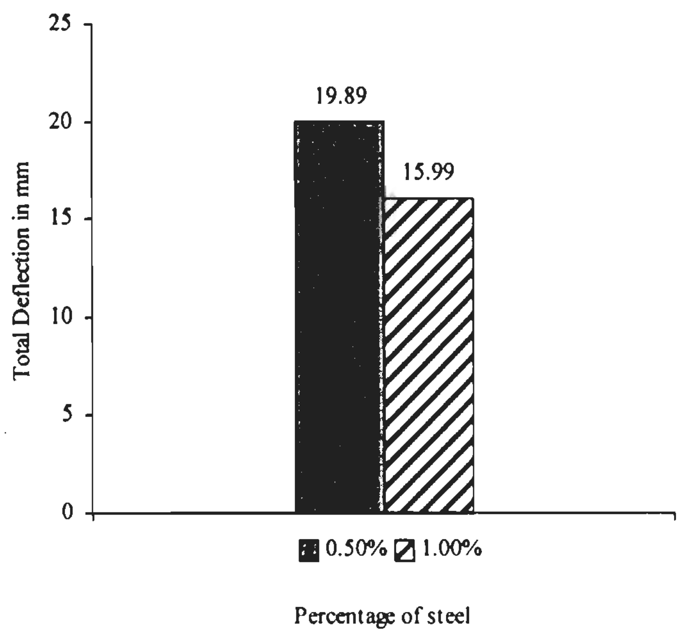


Fig. 5.13 Effect of percentage of steel on total deflection (IS: 456-2000)

Effect of creep coefficient

The deflection decreases when the creep coefficient is considered for one-year age at loading when compared to 28 days age at loading. The deflection for a depth of 210 mm for 28 days age at loading is 19.89 mm (Fig.5.14). whereas the deflection for one-year age at loading is 19.1 mm. The decrease in deflection is 3.97%. Here the deflection is more at the early age at loading of concrete.

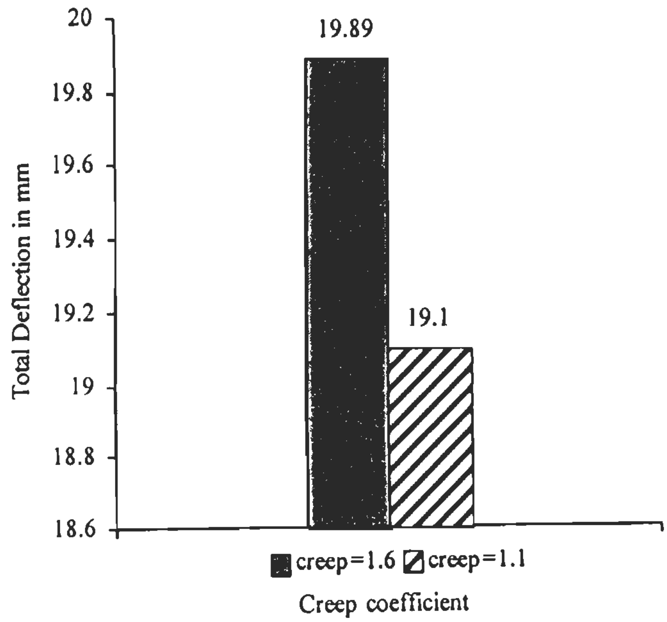
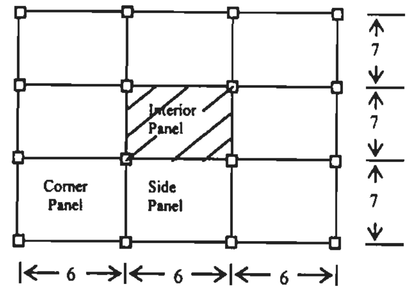


Fig. 5.14 Effect of creep coefficient on total deflection (IS: 456-2000)

Table 5.8 Deflection of plate (Interior panel)
(6 m x 7 m – as per IS: 456-2000)



Structural Details

Floor Plan Size (m) : 6 x 7

Column Size (mm) : 300 x 300

Storey Height (m) : 3.5

Materials Used : M 20 concrete
Fe 415 steel

Load (kN/m^2): (i) Live Load : 3
(ii) Floor Finish: 1.5

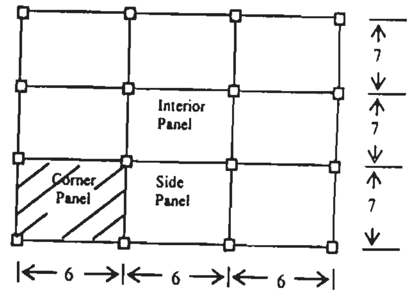
Permissible deflection : Span/250 or 20 mm whichever is less (IS: 456-2000)

Total thick ness in mm	Total deflection in mm	Short term deflection in mm	Shrinkage deflection in mm	Creep deflection in mm	% contribution of short term in total deflection	% contribution of shrinkage in total deflection	% contribution of creep in total deflection	% contribution of creep & shrinkage	% as balanced capacity
240	19.5	13.98	2.46	3.06	71.69	12.61	15.71	28.31	97.50
245	18.47	13.15	2.40	2.91	71.23	13.02	15.74	28.77	92.35
250	17.51	12.40	2.35	2.76	70.78	13.44	15.78	29.22	87.55
255	16.63	11.70	2.31	2.63	70.32	13.86	15.82	29.68	83.15
260	15.81	11.05	2.26	2.51	69.86	14.28	15.86	30.14	79.05
265	15.05	10.45	2.21	2.39	69.40	14.71	15.89	30.60	75.25
270	14.35	9.89	2.17	2.29	68.94	15.13	15.93	31.06	71.75
275	13.69	9.37	2.13	2.19	68.48	15.56	15.97	31.52	68.45
280	13.07	8.89	2.09	2.09	68.01	15.99	16.00	31.99	65.35
285	12.49	8.44	2.05	2.00	67.55	16.41	16.04	32.45	62.45
290	11.95	8.02	2.01	1.92	67.08	16.85	16.07	32.92	59.75
295	11.45	7.63	1.98	1.84	66.61	17.28	16.11	33.39	57.25
300	10.97	7.26	1.94	1.77	66.15	17.71	16.14	33.85	54.85

Table 5.9 Deflection of plate (Corner panel)
(6 m x 7 m – as per IS: 456-2000)

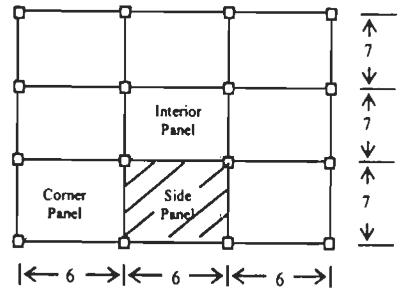
Structural Details

Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5
Materials Used		:	M 20 concrete Fe 415 steel
Load	(kN/m^2):		(i) Live Load : 3 (ii) Floor Finish: 1.5
Permissible deflection		:	Span/250 or 20 mm whichever is less (IS: 456-2000)



Total thick ness in mm	Total deflection in mm	Short term deflection in mm	Shrinkage deflection in mm	Creep deflection in mm	% contribution of short term in total deflection	% contribution of shrinkage in total deflection	% contribution of creep in total deflection	% contribution of creep & shrinkage	% as balanced capacity
240	19.69	14.05	2.46	3.19	71.33	12.48	16.19	28.67	98.45
245	18.65	13.22	2.40	3.03	70.88	12.89	16.23	29.12	93.25
250	17.69	12.46	2.35	2.88	70.42	13.30	16.27	29.58	88.45
255	16.80	11.75	2.31	2.74	69.96	13.72	16.32	30.04	84.00
260	15.97	11.1	2.26	2.61	69.51	14.14	16.36	30.49	79.85
265	15.21	10.5	2.21	2.49	69.05	14.56	16.40	30.95	76.05
270	14.49	9.94	2.17	2.38	68.58	14.98	16.44	31.42	72.45
275	13.83	9.42	2.13	2.28	68.12	15.40	16.48	31.88	69.15
280	13.20	8.93	2.09	2.18	67.66	15.82	16.52	32.34	66.00
285	12.62	8.48	2.05	2.09	67.20	16.25	16.56	32.80	63.10
290	12.08	8.06	2.01	2.00	66.73	16.67	16.60	33.27	60.40
295	11.57	7.67	1.98	1.92	66.27	17.10	16.64	33.73	57.85
300	11.09	7.30	1.94	1.85	65.80	17.53	16.67	34.20	55.45

Table 5.10 Deflection of plate (Side panel) (6 m x 7 m – as per IS: 456-2000)



Structural Details

Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5
Materials Used		:	M 20 concrete Fe 415 steel
Load	(kN/m^2):		(i) Live Load : 3 (ii) Floor Finish: 1.5
Permissible deflection		:	Span/250 or 20 mm whichever is less (IS: 456-2000)

Total thick ness in mm	Total deflection in mm	Short term deflection in mm	Shrinkage deflection in mm	Creep deflection in mm	% contribution of short term in total deflection	% contribution of shrinkage in total deflection	% contribution of creep in total deflection	% contribution of creep & shrinkage	% as balanced capacity
240	19.62	14.02	2.46	3.21	71.48	12.53	16.38	28.91	98.10
245	18.58	13.20	2.40	3.05	71.02	12.94	16.43	29.37	92.90
250	17.62	12.44	2.35	2.90	70.57	13.36	16.47	29.83	88.10
255	16.74	11.73	2.31	2.76	70.11	13.77	16.52	30.29	83.70
260	15.91	11.08	2.26	2.64	69.65	14.19	16.56	30.75	79.55
265	15.15	10.48	2.21	2.52	69.19	14.61	16.60	31.22	75.75
270	14.44	9.92	2.17	2.40	68.73	15.03	16.64	31.68	72.20
275	13.77	9.40	2.13	2.30	68.27	15.46	16.69	32.15	68.85
280	13.15	8.92	2.09	2.20	67.80	15.88	16.73	32.61	65.75
285	12.57	8.47	2.05	2.11	67.34	16.31	16.77	33.08	62.85
290	12.03	8.05	2.01	2.02	66.87	16.74	16.81	33.55	60.15
295	11.52	7.65	1.98	1.94	66.41	17.17	16.85	34.01	57.60
300	11.05	7.28	1.94	1.87	65.94	17.60	16.89	34.48	55.25

5.7 EC: 2-2002

In this code method the moment curvature method is used in which all the parameters are considered. The Tables 5.11 to 5.13 give the deflection of flat plate for the span of 6 m x 7 m. The influence of the parameters were studied and discussed below.

Effect of span

The increase in span by 1 square meter increases the deflection by 65%. The effect of span in total deflection for different floor sizes is plotted in Fig.5.15.

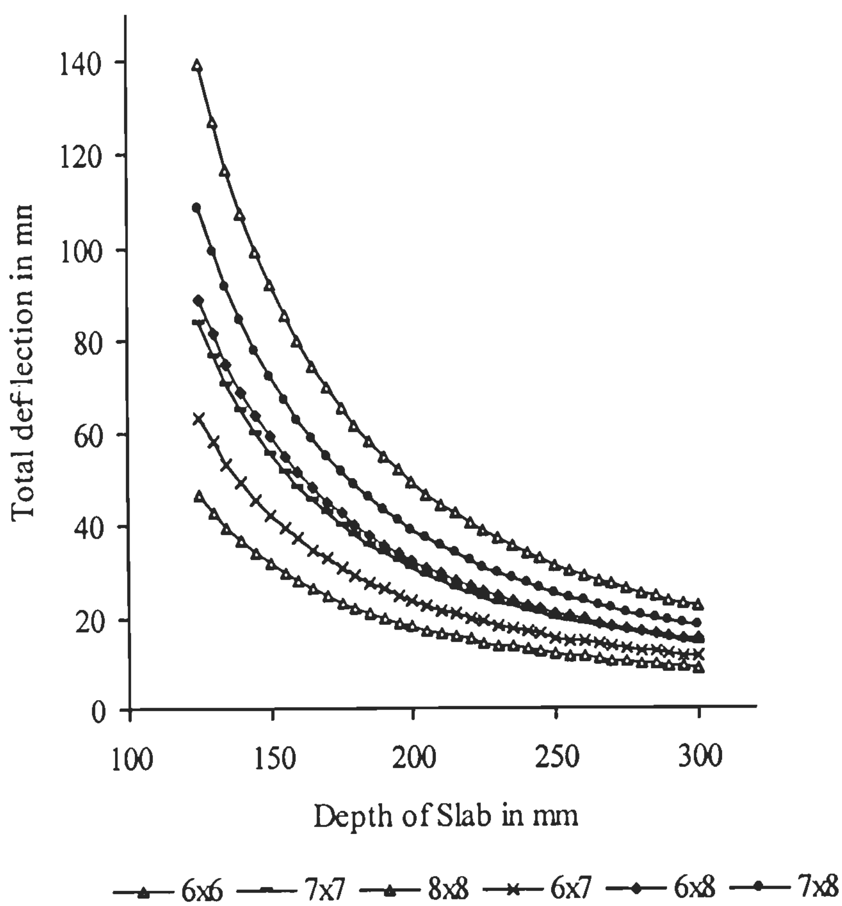


Fig. 5.15 Effect of span on total deflection (EC: 2-2002)

Effect of Grade of Concrete

As per this code, the depth required based on limit of deflection is minimum 210 mm. The deflection for the different grade of concrete is plotted in the Fig. 5.16.

The total deflection is 19.93 mm, 19.56 mm and 19.20 mm for the f_{ck} = 20 N/mm^2, f_{ck} = 25 N/mm^2 and f_{ck} = 30 N/mm^2 respectively. The percentage reduction in deflection is 1.86% and 1.84% if the grade of concrete is increased from 20 to 25, 25 to 30. The effect of grade of concrete is negligible in this case.

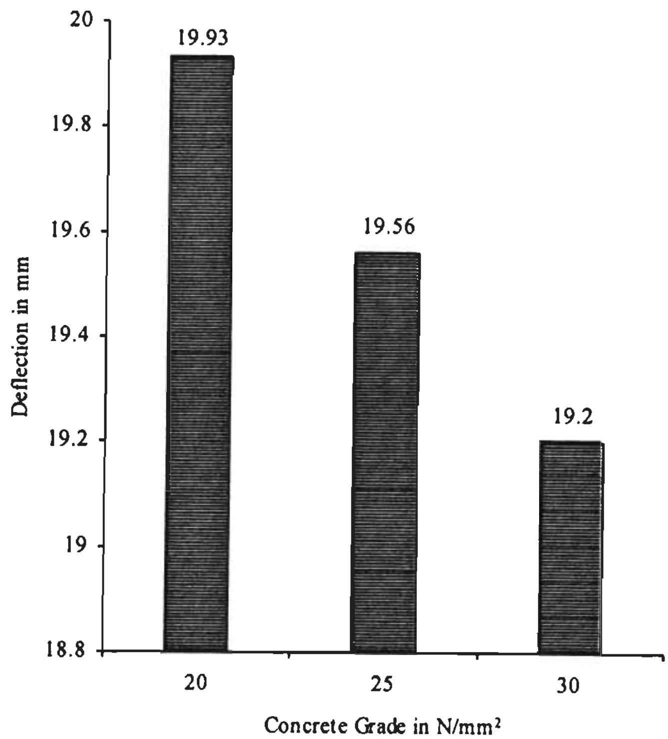


Fig. 5.16 Effect of grade of concrete on total deflection (EC: 2-2002)

Effect of Live Load

The influence of live load was studied for the live loads of 3 kN/m^2, 4 kN/m^2 and 5 kN/m^2 and the deflections for 225 mm depth are plotted in the figure 5.17. The deflections for the above loads are 18.09 mm, 18.65 mm and 19.21 mm respectively.

The increase in deflections was 3% and 5.52% for the live load 4 kN/m^2 and 5 kN/m^2, when increased from 3 kN/m^2 and 4 kN/m^2. The increase in live load by 1 kN/m^2 increases the deflection considerably.

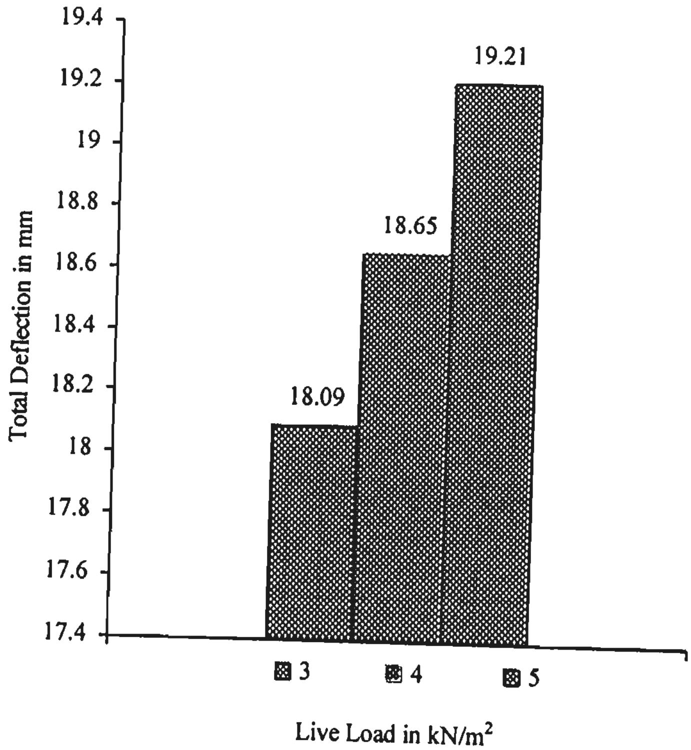
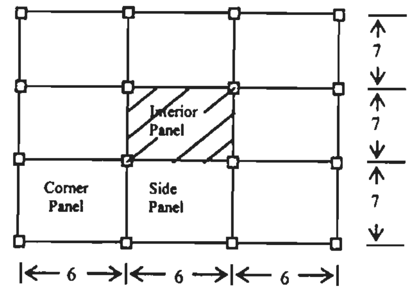


Fig. 5.17 Effect of live load on total deflection (EC: 2-2002)

Table 5.11 Deflection of plate (Interior panel)
(6 m x 7 m – as per EC: 2-2002)

Structural Details

Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5
Materials Used		:	M 20 concrete Fe 415 steel
Load	(kN/m^2):		(i) Live Load : 3 (ii) Floor Finish: 1.5
Permissible deflection		:	Span/250 or 20 mm whichever is less (EC: 2-2002)

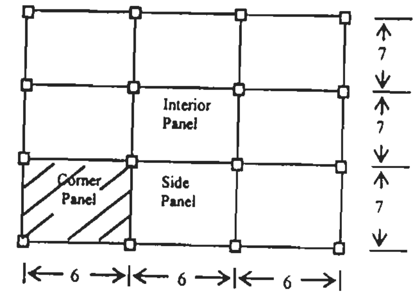


Total thick ness in mm	Total deflection in mm	Short term deflection in mm	Shrinkage deflection in mm	Creep deflection in mm	% contribution of short term in total deflection	% contribution of shrinkage in total deflection	% contribution of creep in total deflection	% contribution of creep & shrinkage	% as balanced capacity
220	19.85	0.47	4.74	14.64	2.38	23.88	73.74	97.62	99.25
225	19.03	0.44	4.67	13.92	2.32	24.54	73.14	97.68	95.15
230	18.27	0.41	4.60	13.26	2.27	25.19	72.54	97.73	91.35
235	17.57	0.39	4.54	12.64	2.21	25.83	71.96	97.79	87.85
240	16.90	0.36	4.47	12.07	2.16	26.46	71.38	97.84	84.50
245	16.28	0.34	4.41	11.53	2.11	27.09	70.81	97.89	81.40
250	15.70	0.32	4.35	11.03	2.06	27.70	70.24	97.94	78.50
255	15.16	0.30	4.29	10.56	2.01	28.31	69.68	97.99	75.80
260	14.65	0.29	4.23	10.13	1.96	28.90	69.13	98.04	73.25
265	14.17	0.27	4.18	9.72	1.92	29.49	68.59	98.08	70.85
270	13.71	0.26	4.12	9.33	1.87	30.07	68.06	98.13	68.55
275	13.28	0.24	4.07	8.97	1.83	30.64	67.53	98.17	66.40
280	12.88	0.23	4.02	8.63	1.79	31.20	67.02	98.21	64.40
285	12.50	0.22	3.97	8.31	1.75	31.75	66.51	98.25	62.50
290	12.14	0.21	3.92	8.01	1.71	32.29	66.00	98.29	60.70
295	11.79	0.20	3.87	7.73	1.67	32.82	65.51	98.33	58.95
300	11.47	0.19	3.82	7.46	1.64	33.35	65.02	98.36	57.35

Table 5.12 Deflection of plate (Corner panel)
(6 m x 7 m – as per EC: 2-2002)

Structural Details

Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5



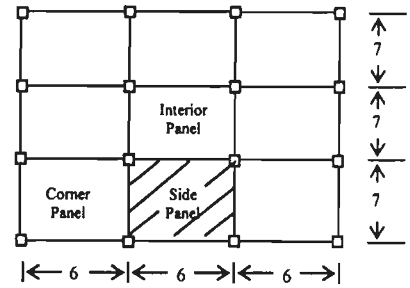
Materials Used : M 20 concrete
Fe 415 steel

Load (kN/m^2): (i) Live Load : 3
(ii) Floor Finish: 1.5

Permissible deflection : Span/250 or 20 mm whichever is less
(EC: 2-2002)

Total thick ness in mm	**Total deflection in mm**	**Short term deflection in mm**	**Shrinkage deflection in mm**	**Creep deflection in mm**	**% contribution of short term in total deflection**	**% contribution of shrinkage in total deflection**	**% contribution of creep in total deflection**	**% contribution of creep & shrinkage**	**% as balanced capacity**
230	19.24	0.45	5.08	13.72	2.32	26.40	71.28	97.68	96.20
235	18.50	0.42	4.99	13.09	2.26	26.99	70.75	97.74	92.50
240	17.81	0.39	4.91	12.51	2.20	27.57	70.23	97.80	89.05
245	17.16	0.37	4.83	11.96	2.15	28.14	69.71	97.85	85.80
250	16.55	0.35	4.75	11.46	2.09	28.70	69.20	97.91	82.75
255	15.98	0.33	4.68	10.98	2.04	29.25	68.71	97.96	79.90
260	15.45	0.31	4.60	10.54	1.99	29.80	68.21	98.01	77.25
265	14.94	0.29	4.53	10.12	1.94	30.33	67.73	98.06	74.70
270	14.47	0.27	4.46	9.73	1.90	30.85	67.26	98.10	72.35
275	14.02	0.26	4.40	9.36	1.85	31.36	66.79	98.15	70.10
280	13.59	0.25	4.33	9.02	1.81	31.86	66.33	98.19	67.95
285	13.19	0.23	4.27	8.69	1.76	32.36	65.88	98.24	65.95
290	12.81	0.22	4.21	8.38	1.72	32.84	65.44	98.28	64.05
295	12.45	0.21	4.15	8.09	1.68	33.32	65.00	98.32	62.25
300	12.11	0.20	4.09	7.82	1.64	33.78	64.57	98.36	60.55

Table 5.13 Deflection of plate (Side panel)
(6 m x 7 m – as per EC: 2-2002)



Structural Details

Floor Plan Size	(m)	:	6 x 7
Column Size	(mm)	:	300 x 300
Storey Height	(m)	:	3.5
Materials Used		:	M 20 concrete Fe 415 steel
Load	(kN/m^2)	:	(i) Live Load : 3 (ii) Floor Finish: 1.5
Permissible deflection		:	Span/250 or 20 mm whichever is less (EC: 2-2002)

Total thick ness in mm	Total deflection in mm	Short term deflection in mm	Shrinkage deflection in mm	Creep deflection in mm	% contribution of short term in total deflection	% contribution of shrinkage in total deflection	% contribution of creep in total deflection	% contribution of creep & shrinkage	% as balanced capacity
225	19.61	0.42	4.98	14.21	2.14	25.41	72.45	97.62	98.05
230	18.83	0.39	4.90	13.54	2.08	26.03	71.89	97.68	94.15
235	18.11	0.37	4.82	12.92	2.03	26.63	71.34	97.74	90.55
240	17.43	0.34	4.75	12.34	1.97	27.23	70.80	97.80	87.15
245	16.80	0.32	4.67	11.81	1.92	27.81	70.27	97.85	84.00
250	16.21	0.30	4.60	11.31	1.87	28.38	69.74	97.91	81.05
255	15.65	0.29	4.53	10.84	1.82	28.95	69.23	97.96	78.25
260	15.13	0.27	4.46	10.40	1.78	29.50	68.72	98.01	75.65
265	14.64	0.25	4.40	9.99	1.73	30.05	68.22	98.06	73.20
270	14.18	0.24	4.33	9.60	1.69	30.58	67.73	98.10	70.90
275	13.74	0.23	4.27	9.24	1.65	31.10	67.25	98.15	68.70
280	13.32	0.21	4.21	8.90	1.60	31.62	66.78	98.19	66.60
285	12.93	0.20	4.15	8.58	1.56	32.12	66.31	98.24	64.65
290	12.56	0.19	4.10	8.27	1.53	32.62	65.86	98.28	62.80
295	12.21	0.18	4.04	7.99	1.49	33.10	65.41	98.32	61.05
300	11.88	0.17	3.99	7.72	1.45	33.58	64.97	98.36	59.40

Among all the parameters studied, the span, load, depth and grade of concrete have the major influence in total deflection as per the four codes studied. This is summarized in Table 5.14.

Table 5.14 Percentage variation in total deflection with different parameters

Parameters	Change in deflection	ACI: 318-2002	BS: 8110-1997	EC: 2-2002	IS: 456-2000
Span (Increase in 1 m^2)	Increase	+85.30	+75.00	+65.00	+82.50
Depth (with 5 mm increase)	Decrease	-13.92	-14.00	-10.20	-12.16
Grade of concrete (with 5 N/mm^2 increase)	Decrease	-9.00	-3.70	-1.80	-4.50
Live Load (with 1 kN/mm^2 increase)	Increase	+6.60	+8.00	+3.10	+8.30

5.8 SIGNIFICANCE OF THE PARAMETRIC STUDY

From the observations furnished in table 5.8, the span has major role in increasing the deflection. Increase in span by 1 square meter increases the deflection by minimum 65% (EC: 2-2002) and maximum 85% (ACI: 318-2002). The parameter that has significant influence in reducing the deflection is the depth of the slab. Even 5 mm increase in depth of slab, reduces the deflection by minimum 10% (EC: 2-2002) and maximum 14% (ACI: 318-2002 & BS: 8110-1997).

This may be the reason for adopting the span/depth ratio to fix the preliminary dimension of slab in all the codes. But the increase in live load by 1 kN/m^2 increases the deflection by minimum 3.1% (EC: 2-2002) and maximum 8.30% (IS: 456-2002). Also the increase in grade of concrete by 5 N/mm^2 decreases the deflection by minimum 1.8% (EC: 2-2002) and maximum 9% (ACI: 318-2002). These are not considered in all the codes. Hence fixing the depth of slab based on span/depth ratio alone will not be sufficient to meet the actual requirement.

The span to depth ratio that meets the serviceability limit for the lower load case will not be suitable for the same span with higher load case. Also for the same span and loading condition if the grade of concrete is higher, the span to depth ratio provided for the lower grade concrete will become uneconomical. Hence it is necessary to have an approach considering all the parameters.

Among all the empirical relations studied suggested by the four codes, the EC: 2-2002, considers all the parameters including tension stiffening effect of concrete. The deflections obtained using the developed program for these equations (Chapter 4), gives good prediction of actual deflection. This is evaluated with the experimental results reported in the literature (Gilbert and Guo,[2005]).

The experimental data given in the above literature were substituted in the equations and the deflections were computed. The comparison of computed deflections with the experimental deflections is furnished in the Tables 5.17, 5.20 and 5.23.

The experimental data of the slabs furnished in the literature are summarized in the Tables 5.15, 5.16, 5.18, 5.19, 5.21 and 5.22 below:

Slab S1:

Total depth	= 100 mm
f_{ck}	= 45.48 N/mm^2
f_{ctm}	= 3.87 N/mm^2
E_c	= 30,020 N/mm^2
Tension reinforcement	= 550 mm^2
Compression reinforcement	= 510 mm^2

Table 5.15 Data for the slab S1

Days Nos.	Creep coefficient	Shrinkage strain	Dead load KPa	Live load KPa	Total load KPa
80	1.64	313×10^{-6}	2.40	3.15	5.55
200	2.06	471×10^{-6}	2.40	6.26	8.66
300	2.29	504×10^{-6}	2.40	6.26	8.66

Slab S2:

Total depth	= 100 mm
f_{ck}	= 40.32 N/mm^2
f_{ctm}	= 3.57 N/mm^2
E_c	= 29600 N/mm^2
Tension reinforcement	= 550 mm^2
Compression reinforcement	= 510 mm^2

Table 5.16 Data for the slab S2

Days Nos.	Creep coefficient	Shrinkage strain	Dead load KPa	Live load KPa	Total load KPa
28	1.25	416×10^{-6}	2.40	3.32	5.72
200	2.74	744×10^{-6}	2.40	3.32	5.72
250	2.79	751×10^{-6}	2.40	3.32	5.72

Table 5.17 Comparison with the existing experimental results (Gilbert and Guo, [2005]) in the literature.

S. No	Days Nos.	Slab 1			Slab 2		
		δ_c mm	δ_e mm	δ_c/δ_e	δ_c mm	δ_e mm	δ_c/δ_e
1	28	-	-	-	5.8	5.4	1.07
2	80	6.07	5.73	1.06	-	-	-
3	200	9.46	8.52	1.11	8.88	8.79	1.01
4	250	-	-	-	8.97	10.5	0.85
5	300	9.96	9.86	1.01	-	-	-
6	400	-	-	-	-	-	-
			Mean	1.06			0.98

Slab S3:

Total depth	= 90 mm
f_{ck}	= 21.48 N/mm^2
f_{ctm}	= 2.34 N/mm^2
E_c	= 22620 N/mm^2
Tension reinforcement	= 550 mm^2
Compression reinforcement	= 510 mm^2

Table 5.18 Data for the slab S3

Days Nos.	Creep coefficient	Shrinkage strain	Dead load KPa	Live load KPa	Total load KPa
28	0.58	85×10^{-6}	2.16	3.10	5.26
300	2.05	735×10^{-6}	2.16	3.10	5.26
400	2.18	763×10^{-6}	2.16	3.10	5.26

Slab S4:

Total depth	= 90 mm
f_{ck}	= 28.08 N/mm^2
f_{ctm}	= 2.8 N/mm^2
E_c	= 23150 N/mm^2
Tension reinforcement	= 550 mm^2
Compression reinforcement	= 510 mm^2

Table 5.19 Data for the slab S4

Days Nos.	Creep coefficient	Shrinkage strain	Dead load KPa	Live load KPa	Total load KPa
28	1.2	208×10^{-6}	2.16	6.23	8.39
80	2.2	450×10^{-6}	2.16	6.23	8.39
200	2.55	670×10^{-6}	2.16	6.23	8.39
400	3.22	831×10^{-6}	2.16	6.23	8.39

Table 5.20 Comparison with the existing experimental results [Gilbert et al., 2005] in the literature

S. No	Days Nos.	Slab 3			Slab 4		
		δ_c mm	δ_e mm	δ_c / δ_e	δ_c mm	δ_e mm	δ_c / δ_e
1	28	5.14	2.84	1.8	10.04	9.19	1.09
2	80	-	-	-	13.75	16.6	0.83
3	200	-	-	-	15.54	19.7	0.79
4	300	11.03	9.86	1.01	-	-	-
5	400	11.44	11.8	0.97	18.14	23.3	0.78
			Mean	1.26			0.87

Slab S6:

Total depth = 90 mm

f_{ck} = 23.04 N/mm^2

f_{ctm} = 2.45 N/mm^2

E_c = 21670 N/mm^2

Tension reinforcement = 475 mm^2

Compression reinforcement = 395 mm^2

Table 5.21 Data for the slab S6

Days Nos.	Creep coefficient	Shrinkage strain	Dead load KPa	Live load KPa	Total load KPa
28	1.06	170×10^{-6}	2.16	3.10	5.26
80	2.48	632×10^{-6}	2.16	3.10	5.26

Slab S7:

Total depth	= 90 mm
f_{ck}	= 23.04 N/mm^2
f_{ctm}	= 2.45 N/mm^2
E_c	= 21670 N/mm^2
Tension reinforcement	= 475 mm^2
Compression reinforcement	= 395 mm^2

Table 5.22 Data for the slab S7

Days Nos.	Creep coefficient	Shrinkage strain	Dead load KPa	Live load KPa	Total load KPa
28	1.06	170×10^{-6}	2.16	3.41	5.57
80	2.48	632×10^{-6}	2.16	3.41	5.57

Table 5.23 Comparison with the existing experimental results [Gilbert et al., 2005] in the literature

S.No	Days Nos.	Slab 6			Slab 7		
		δ_c mm	δ_e mm	δ_c/δ_e	δ_c mm	δ_e mm	δ_c/δ_e
1	28	7.89	6.47	1.22	7.75	5.93	1.3
2	300	12.21	17.1	0.71	12.56	17.7	0.71
			Mean	0.96			1.005

5.9 PROPOSED RATIONAL APPROACH

Among the above four codes, EC: 2-2002 considers all the parameters including the tension stiffening effects of concrete. The deflection computed by using this approach with suitable modification as explained in Chapter 4, gives reliable solution for predicting the long term deflection. This approach is validated with the experimental results reported in the literature of Gilbert and Guo [2005]. The comparison of results obtained by using the proposed method and experimental results available in the literature are furnished in the tables 5.17, 5.20 and 5.23. The equations for curvatures are given below.

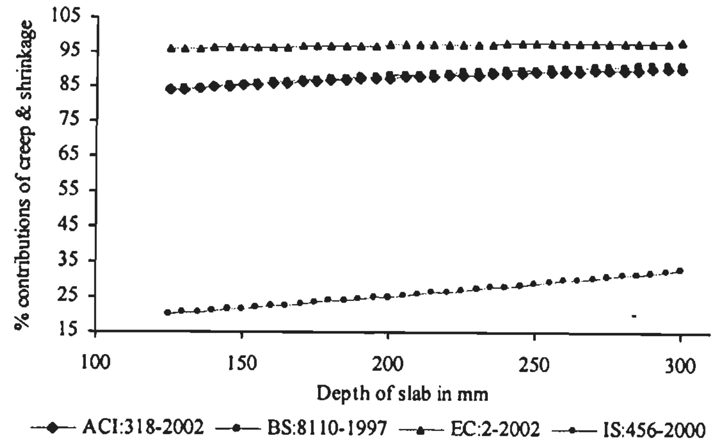
$$\text{Shrinkage curvature} = \xi(1/r_{cs2}) + [(1-\xi)(1/r_{cs1})]$$

$$\text{Creep curvature} = \xi(1/r_2) + (1-\xi)(1/r_1)$$

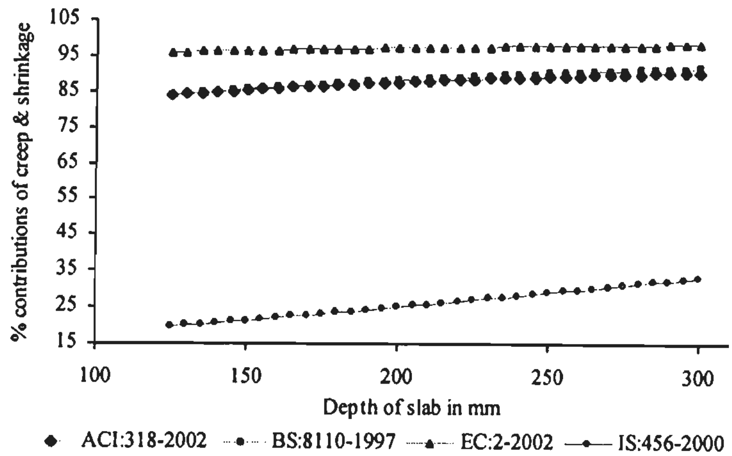
If elaborate calculations are essential, this method can be used effectively. But to use this method all the parameters must be known at the design stage itself, which is not normally possible. Hence a simplified approach is essential to find the depth of slab to meet the long term effects.

5.10 PROPOSED MODIFIED MULTIPLIER APPROACH

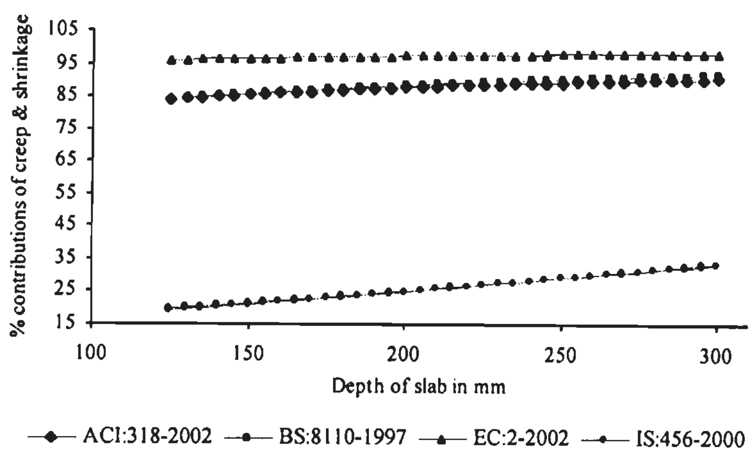
Among all the suggested equations in four codes, the multiplier approach as per ACI: 318-2002 is simple to use. But this formula, under estimates the actual deflection. Hence it needs modification in the multiplication factor for deflection due to sustained load. To modify this factor the contribution of creep and shrinkage in total deflection as per each code was studied. Since the IS: 456-2000 code under estimates the creep and shrinkage effects and EC: 2-2002 is on the higher side the same, those percentages were not considered for the modification. Hence to propose the modified multiplier approach the percentage contribution of creep and shrinkage effects as per ACI: 318-2002 and BS: 8110-1997 codes were considered. The Figs. 5.22 to 5.24 give the percentage effects for different parameters. The Tables 5.24 to 5.26 furnish the percentage contribution of creep and shrinkage effects in total deflection for interior panel, corner panel and side panel as per the four codes for different parameters.



(a) For M 20 grade concrete



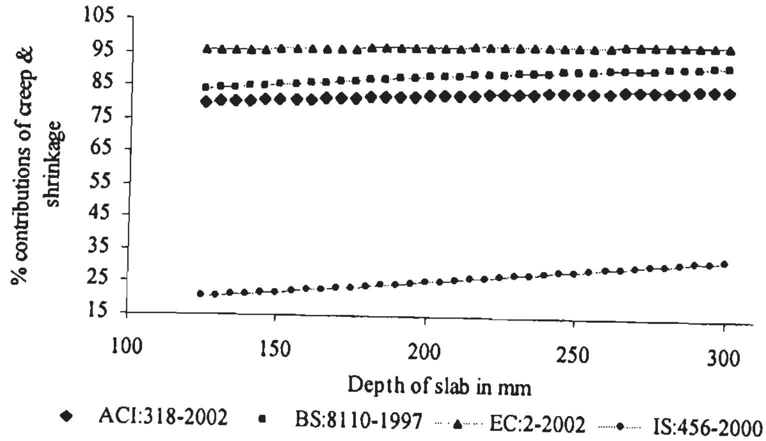
(b) For M 25 grade concrete



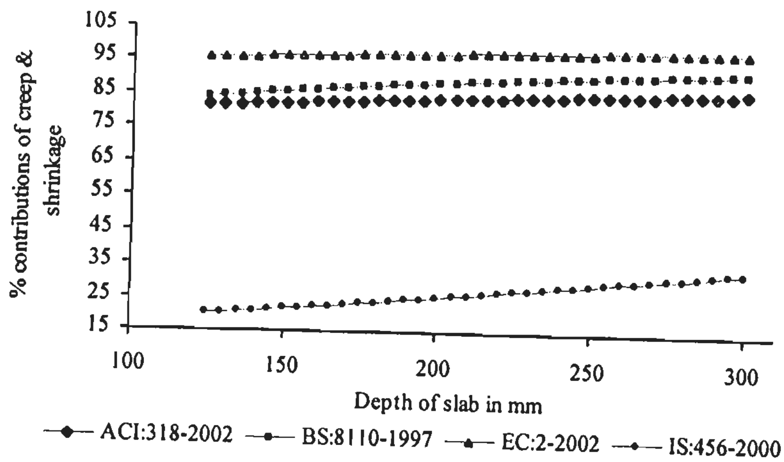
(c) For M 30 grade concrete

Fig. 5.18 Contributions of creep & shrinkage

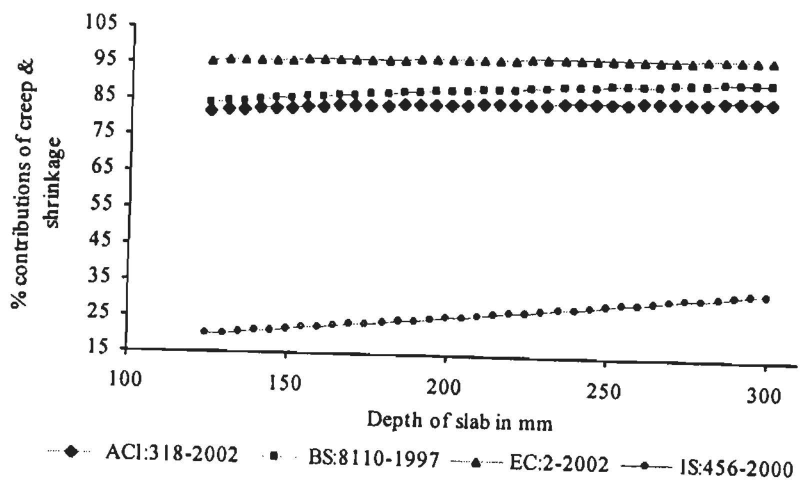
(7 m x 7 m span & live load 3 kN/m^2)-Interior panel



(a) For M 20 grade concrete



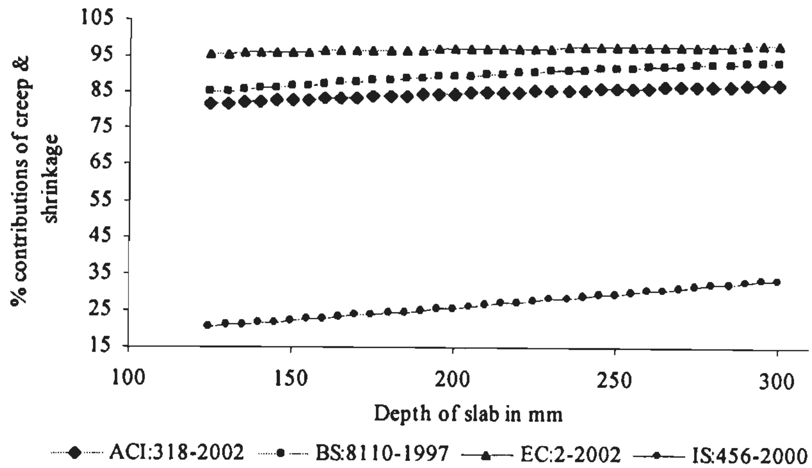
(b) For M 25 grade concrete



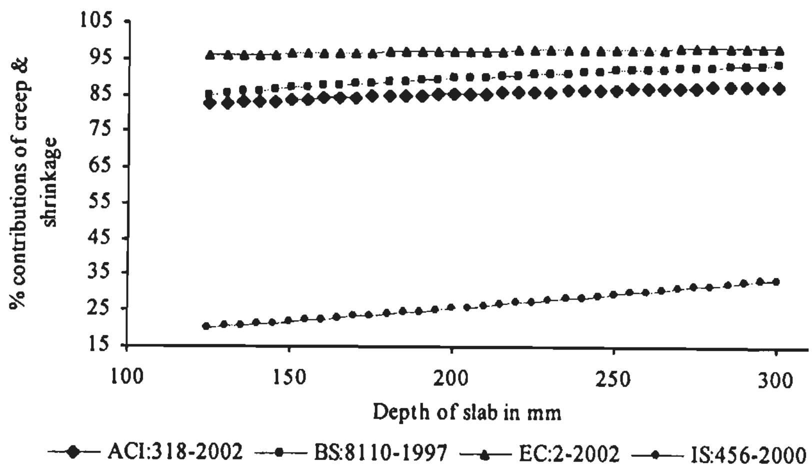
(c) For M 30 grade concrete

Fig. 5.19 Contributions of creep & shrinkage

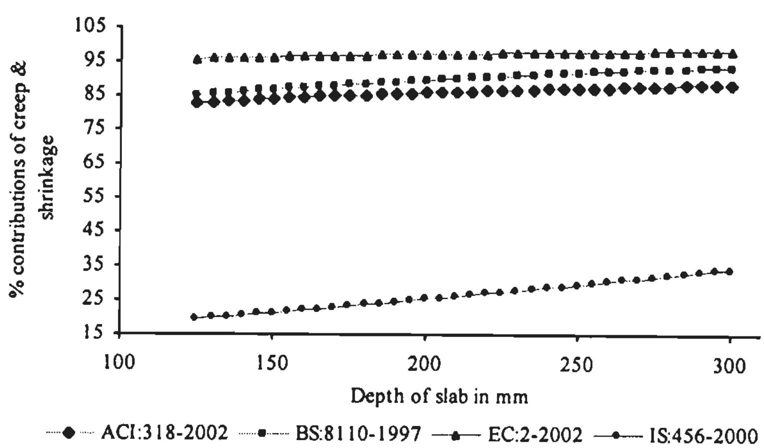
(7 m x 7 m span & live load 3 kN/m^2)-Corner panel



(a) For M 20 grade concrete



(b) For M 25 grade concrete



(c) For M 30 grade concrete

Fig. 5.20 Contributions of creep & shrinkage

(7 m x 7 m span & live load 3 kN/m^2)-Side panel

Table 5.24 Percentage contribution of creep & shrinkage in total deflection (Interior panel)

Span m	Grade N/mm²	Load kN/m²	ACI:318-2002 %	BS:8110-1997 %	EC:2-2002 %	IS:456-2000 %
6x6	20	3	87.82	88.97	97.55	29.55
		4	84.76	86.18	96.83	27.66
		5	82.08	83.70	96.16	26.05
	25	3	87.82	89.02	97.60	29.66
		4	84.76	86.24	96.90	27.67
		5	82.08	83.77	96.24	26.01
	30	3	87.82	89.07	97.65	29.98
		4	84.76	86.30	96.96	27.86
		5	82.08	83.84	96.32	26.10
6x7	20	3	87.82	88.74	97.41	26.00
		4	84.77	85.90	96.66	24.30
		5	82.08	83.39	95.96	22.86
	25	3	87.82	88.80	97.45	25.98
		4	84.77	85.97	96.90	24.22
		5	82.08	83.46	96.03	22.75
	30	3	87.82	88.85	97.49	26.04
		4	84.77	86.03	96.96	24.24
		5	82.08	83.54	96.08	22.71
7x7	20	3	87.82	88.48	97.33	26.27
		4	84.77	85.60	96.56	24.64
		5	82.08	83.06	95.85	23.26
	25	3	87.82	88.53	97.37	26.07
		4	84.77	85.66	96.61	24.41
		5	82.08	83.12	95.91	23.00
	30	3	87.82	88.56	97.41	25.97
		4	84.77	85.73	96.66	24.27
		5	82.08	83.20	95.96	22.84
		Mean	84.89	86.06	96.77	25.57

Table 5.25 Percentage contribution of creep & shrinkage in total deflection (Corner panel)

Span m	Grade N/mm^2	Load kN/m^2	ACI:318-2002 %	BS:8110-1997 %	EC:2-2002 %	IS:456-2000 %
6x6	20	3	86.11	88.91	97.42	29.89
		4	82.59	86.07	96.65	27.97
		5	78.73	83.55	95.94	26.35
	25	3	86.59	88.94	97.48	29.99
		4	83.02	86.11	96.73	28.02
		5	79.98	83.60	96.03	26.33
	30	3	87.12	88.96	97.54	30.28
		4	83.41	86.15	96.80	28.20
		5	80.27	83.64	96.11	26.41
6x7	20	3	87.21	88.82	97.33	26.34
		4	83.54	85.91	96.53	24.67
		5	80.15	83.34	95.78	23.20
	25	3	87.88	88.85	97.38	26.36
		4	84.15	85.96	96.73	24.57
		5	80.93	83.39	95.85	23.07
	30	3	88.50	88.88	97.42	26.40
		4	84.69	86.00	96.80	24.57
		5	81.40	83.44	95.91	23.02
7x7	20	3	82.80	88.44	97.18	26.66
		4	78.96	85.49	96.36	25.05
		5	75.75	82.88	95.59	23.65
	25	3	83.78	88.48	97.22	26.50
		4	79.70	85.54	96.41	24.80
		5	76.30	82.94	95.65	23.37
	30	3	84.71	88.51	97.26	26.37
		4	80.48	85.58	96.46	24.64
		5	76.91	83.00	95.71	23.19
		Mean	82.43	85.90	96.60	25.92

Table 5.26 Percentage contribution of creep & shrinkage in total deflection (Side panel)

Span m	Grade N/mm^2	Load kN/m^2	ACI:318-2002 %	BS:8110-1997 %	EC:2-2002 %	IS:456-2000 %
6x6	20	3	86.85	90.49	97.42	30.03
		4	83.54	87.63	96.65	28.12
		5	80.16	85.10	95.94	26.49
	25	3	87.12	90.52	97.48	30.13
		4	83.78	87.68	96.73	28.19
		5	80.88	85.15	96.03	26.50
	30	3	87.42	90.56	97.54	30.42
		4	83.99	87.72	96.80	28.36
		5	81.05	85.20	96.11	26.56
6x7	20	3	84.90	90.09	97.33	26.56
		4	81.39	87.19	96.53	24.93
		5	78.38	84.63	95.78	23.46
	25	3	85.48	90.12	97.38	26.62
		4	81.82	87.24	96.73	24.82
		5	78.71	84.68	95.85	23.31
	30	3	86.03	90.15	97.42	26.65
		4	82.29	87.28	96.80	24.80
		5	79.07	84.73	95.91	23.25
7x7	20	3	84.90	89.84	97.18	26.80
		4	81.39	86.89	96.36	25.22
		5	78.37	84.29	95.59	23.81
	25	3	85.48	89.88	97.22	26.65
		4	81.82	86.94	96.41	24.95
		5	78.71	84.34	95.65	23.51
	30	3	86.03	89.90	97.26	26.52
		4	82.29	86.98	96.46	24.78
		5	79.07	84.40	95.71	23.32
		Mean	82.63	87.39	96.60	26.10

From the parametric study conducted (Tables 5.24 to 5.26) the percentage contribution of creep and shrinkage in total deflection works out to approximately 84.88%. The factor 3 in Branson approach gives 75% in total deflection. This factor 3 is used for the deflection due to sustained load. Since the creep and shrinkage effects are due to sustained load, this can be modified based on the study. From the extensive parametric study the contribution of creep and shrinkage as per the three codes ACI: 318-2002, BS: 8110-1997 for the different parameters (average) works out to 84.9%.

If the factor 3 in the multiplier approach is modified to 6, the percentage contribution of sustained load due to long-term effects in total deflection works out to 85.7%. This is 0.8% in excess of what is obtained. Since the EC: 2-2002 approach gives 95% for creep and shrinkage effects the above increase will predict the realistic deflection. This approach avoids the elaborate calculation to be made for predicting the total deflection of the slab at a later stage. This can be easily used by the designers at the design stage itself in fixing the depth of slab. The following is the proposed modified multiplier approach.

$$\delta_T = 6\delta_D + \delta_L$$

The approach is validated with the actual building deflection reported in the literature (Rangan, [1976]). This can be used for flat slab also. For flat slab the proposed model as explained in Chapter 3 was used to find the deflection. Then applying the above proposed approach the deflection for flat slab was calculated. The above modification in the multiplier approach is validated with the existing results reported in the literatures.

5.11 VALIDATION OF PROPOSED MODIFIED MULTIPLIER APPROACH

The computed deflection using the above formula was compared with the actual deflection as well as the calculated deflection by Rangan [1976].The values obtained substituting the data available in the literature using the proposed approach are compared with the actual results in the literature and furnished in the Tables 5.27 and 5.28.

Table 5.27 Comparison with the results (Rangan, [1976]) and actual Building deflection

S. No	Structure	l_1 m	l_2 m	D mm	Deflection in mm			Ratio of computed to Rangan/measured deflection	
					Computed	Rangan	Measured	Rangan	Measured
1.	Taylor's Flat plate	5.08	6.35	203	18.38	18.30	18.29	1.004	1.005
2.	Heiman's Flat plate	7.24	7.54	241	19.87	26.16	20.83	0.76	0.95
3.	Taylor's Flat slab	7.92	8.61	241	22.57	26.16	22.35	0.76	1.01

Table 5.28 Comparison with Hwang and Chang [1996] and earlier test results

S. No	Structure	l_1 m	l_2 m	D mm	Deflection in mm			Ratio of computed to Hwang/Test deflection	
					Computed	Hwang	Test	Hwang	Test
1.	Hatcher et al.	1.524	1.524	44.5	2.19	1.74	1.96	1.26	1.12
2.	Hwang and Moehle	1.829	2.743	81.3	1.36	1.22	1.55	1.11	0.88

The actual deflection of flat slab building (Boyce, [2002]) that was the subject of legal proceedings was compared with the proposed approach. The building panel dimension and the comparison are given in the Table 5.29.

Table 5.29 Comparison with actual building deflection

S.No	l_1 m	l_2 m	D mm	Deflection in mm	
				Computed	actual
1.	8.4	8.4	220	65-75	49-82

5.12 SUMMARY

From the study made, it was observed that increase in depth of the slab even by 5 mm has significant influence in reducing the deflection. The suitability of depth for a particular span is to be fixed based on both serviceability and strength requirement. For this the slab has to be analysed for the moment carrying capacity and the shear stress induced. The main aim of this research is to analyse the stiffness of slab with concealed beam. The next chapter explains this in detail, by solving numerical examples to compute the deflection, moment carrying capacity and shear stress induced without and with concealed beam.

CHAPTER 6

CASE STUDIES

6.1 GENERAL

Concealed beam is an arrangement of reinforcement which is placed within the slab that does not increase the depth of the slab. The analytical modelling of concealed beam and the theories adopted for this study were explained in Chapter 3. To study the stiffness of the slab, the numerical examples are worked out based on the proposed modelling of the concealed beam. The deflection of the flat plate with and without concealed beam was calculated by using the proposed modified (Chapter 5) multiplier approach. This chapter gives the details of these calculations of deflection, moment carrying capacity and shear stress induced in the slab with and without concealed beam. Finally the results of these two types of slabs are compared with each other and presented at the end of this chapter.

6.2 DESIGN EXAMPLES

The step by step procedures followed for the numerical study are given in sections 6.2.1 and 6.2.2. The moment carrying capacity and shear stress induced in the flat plate with and without concealed beam was analyzed by applying the Equivalent frame method and furnished in sections 6.2.3 and 6.2.4. The results based on this study are compared and discussed in section 6.2.5.

6.2.1 Example 1: Deflection of Flat Plate Without Concealed Beam

The following multi-storey frame of three bays was considered for this numerical study. The dimensioning details of the frame are shown in Fig. 6.1 and Fig. 6.2.

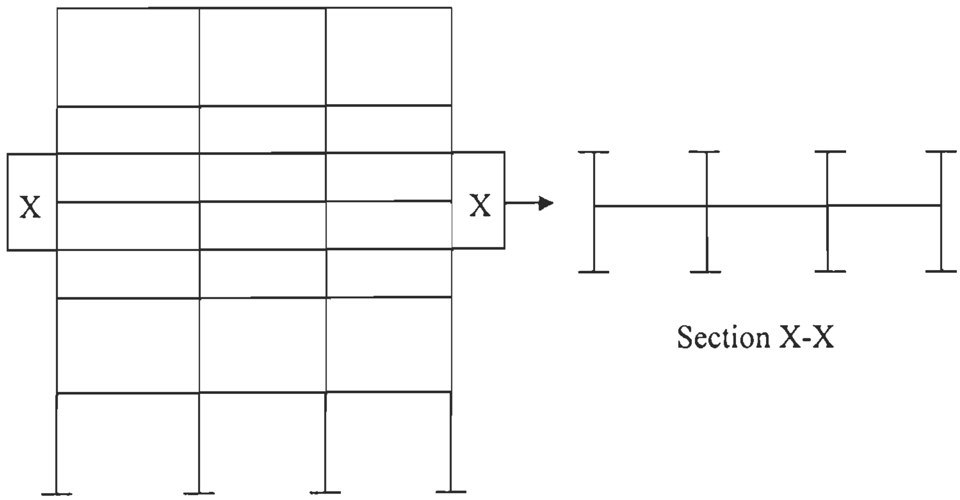


Fig. 6.1 Multistorey building frame

A flat plate with the panel dimension of 6mx6m is selected for this numerical example.

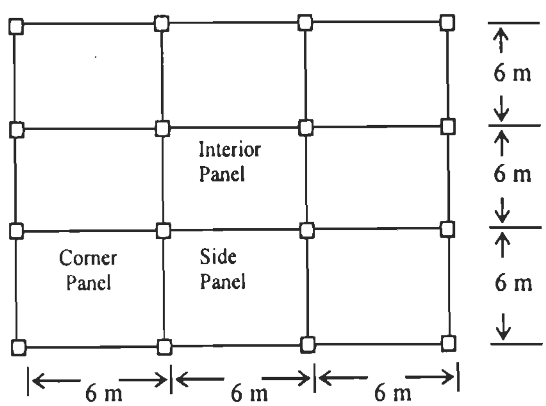


Fig. 6.2 Floor plan

The following parameters are used in the example below. The dimension of column is assumed as 300mm square.

Live load (Wl) = 3.0 kN/m^2

Floor finish (FF) = 1.5 kN/m^2

Clear cover to tension reinforcement = 25 mm

Grade of concrete = $f_{ck} = 20$ N/mm^2

Modulus of elasticity of steel E_s = 200000 N/mm^2

Diameter of rod = 10 mm

Column of size ($C_1 \times C_2$) = 300 mm x 300 mm

Total depth of slab = 260 mm

The modulus of elasticity of concrete E_c is

$$E_c = 5000 \times \sqrt{f_{ck}} = 5000 \times \sqrt{20} = 22360.68 = 22361 \text{ N/mm}^2$$

The sustained load (W_d) is

$$W_d = 25D + FF + 0.25Wl = 25 \text{ x } 0.26 + 1.5 + 0.25 \text{ x } 3 = 8.75 \text{ kN/m}^2$$

The total load (W_t) is

$$W_t = W_d + Wl = 8.75 + 3 = 11.75 \text{ kN/m}^2$$

The moment due to total load (M_{t1x}) along x-x direction

$$M_{tlx} = \frac{0.5 \times L_y \times W_t \times L_x^2}{8} = \frac{0.5 \times 6 \times 11.75 \times 6^2}{8} = 158.625 \text{ kNm}$$

The moment due to total load (M_{t1y}) along y-y direction

$$M_{tly} = \frac{0.5 \times L_x \times W_t \times L_y^2}{8} = \frac{0.5 \times 6 \times 11.75 \times 6^2}{8} = 158.625 \text{ kNm}$$

The lateral distribution factor for moments along the column strip (for interior panel)

$$LDF_c = 0.675$$

The lateral distribution factor for moments along the middle strip (for interior panel)

$$LDF_m = 0.325$$

The lateral distribution factor for moments along the column strip (for edge and corner panel)

$$LDF_{1c} = 0.55$$

The lateral distribution factor for moments along the middle strip (for edge and corner panel)

$$LDF_{1m} = 0.45$$

The modular ratio m

$$m = \frac{E_s}{E_c} = 8.94$$

The effective depth d of slab

$$d = D - cc - (dia/2) = 0.26 - 0.025 - 0.005 = 0.23 \text{ m}$$

The width of the column strip b_{cx} and b_{cy} along x-x and y-y direction respectively

$$b_{cx} = b_{cy} = 0.5 \times L_x = 0.5 \text{ x } 6000 = 3000 \text{ mm}$$

Factored dead load (W_D)

$$W_D = 1.5 \times (D \times 25 + FF) = 1.5\ (0.26 \text{ x } 25 + 1.5) = 12 \text{ kN/m}^2$$

Factored live load (W_L)

$$W_L = 1.5 \times Wl = 1.5 \text{ x } 3 = 4.5 \text{ kN/m}^2$$

Total load (W_T)

$$W_T = W_D + W_L = 12 + 4.5 = 16.5 \text{ kN/m}^2$$

Uniformly distributed load (w)

$$w = W_T \times L_y = 16.5 \text{ x } 6 = 99 \text{ kN/m}$$

Frame

The width of the frame along x-x direction (b_{fx}) is equal to the length along the span in the y-y direction.

$$b_{fx} = L_y = 6000 \text{ mm}$$

The gross moment of inertia of frame (I_{fx}) along x-x direction

$$I_{fx} = \frac{b_{fx} \times D^3}{12} = \frac{6000 \times 260^3}{12} = 8.7880 \times 10^9 \text{ mm}^4$$

The width of the frame along y-y direction (b_{fy}) is equal to the length along the span in the x-x direction.

$$b_{fy} = L_x = 6000 \text{ mm}$$

The gross moment of inertia of frame (I_{fy}) along y-y direction

$$I_{fy} = \frac{b_{fy} \times D^3}{12} = \frac{6000 \times 260^3}{12} = 8.7880 \times 10^9 \text{ mm}^4$$

Column Strip

The area of tension reinforcement provided in the column strip (A_{st1cx}) along the shorter x-x direction

$$A_{st1cx} = \frac{0.5 \times 0.5 \times b_{cx} \times D}{100} = \frac{0.5 \times 0.5 \times 3000 \times 260}{100} = 1950 \text{ mm}^2$$

The area of compression reinforcement provided in the column strip (A_{sc1cx}) along the shorter direction

$$A_{sc1cx} = \frac{0.5 \times b_{cx} \times D}{100} = \frac{0.5 \times 3000 \times 260}{100} = 3900 \text{ mm}^2$$

The gross moment of inertia of the column strip (I_{cx}) along the x-x direction

$$I_{cx} = \frac{b_{cx} \times D^3}{12} = \frac{3000 \times 260^3}{12} = 4.3940 \times 10^9 \text{ mm}^4$$

The area of tension reinforcement provided in the column strip (A_{st1cy}) along the longer y-y direction

$$A_{st1cy} = \frac{0.5 \times 0.5 \times b_{cy} \times D}{100} = \frac{0.5 \times 0.5 \times 3000 \times 260}{100} = 1950 \text{ mm}^2$$

The area of compression reinforcement provided in the column strip (A_{sc1cy}) along the longer y-y direction

$$A_{sc1cy} = \frac{0.5 \times b_{cy} \times D}{100} = \frac{0.5 \times 3000 \times 260}{100} = 3900 \text{ mm}^2$$

The gross moment of inertia of the column strip (I_{cy}) along y-y direction

$$I_{cy} = \frac{b_{cy} \times D^3}{12} = \frac{3000 \times 260^3}{12} = 4.3940 \times 10^9 \text{ mm}^4$$

Middle Strip

The width of middle strip (b_{mx}) along the x-x direction

$$b_{mx} = (L_y - L_x/2) = 6000 - 6000/2 = 3000 \text{ mm}$$

The area of tension reinforcement provided in the middle strip (A_{st1mx}) along the shorter x-x direction

$$A_{st1mx} = \frac{0.5 \times b_{mx} \times D}{100} = \frac{0.5 \times 3000 \times 260}{100} = 3900 \text{ mm}^2$$

The gross moment of inertia of middle strip (I_{mx}) along the x-x direction

$$I_{mx} = \frac{b_{mx} \times D^3}{12} = \frac{3000 \times 260^3}{12} = 4.3940 \times 10^9 \text{ mm}^4$$

The width of middle strip (b_{my}) along the y-y direction

$$b_{my} = L_x/2 = 3000 \text{ mm}$$

The area of tension reinforcement provided in the middle strip (A_{st1my}) along the longer y-y direction

$$A_{st1my} = \frac{0.5 \times b_{my} \times D}{100} = \frac{3000 \times 260^3}{12} = 4.3940 \times 10^9 \text{ mm}^4$$

The gross moment of inertia of the middle strip (I_{my}) along the y-y direction

$$I_{my} = \frac{b_{my} \times D^3}{12} = \frac{3000 \times 260^3}{12} = 4.3940 \times 10^9 \text{ mm}^4$$

Flexural stiffness of equivalent column

The storey height between the centre of floors is assumed as $L_c = 3.5$ m

X-X axis

The moment of inertia of the column (I_{c1x}) along x-x direction

$$I_{c1x} = \frac{C_{1x} C_{1y}^3}{12} = \frac{300 \times 300^3}{12} = 675000000 \text{ mm}^4$$

The relative stiffness of the column (K_{cx}) along x-x direction

$$K_{cx} = \frac{4E_c I_{c1x}}{L_c} = \frac{4 \times 22361 \times 675000000}{3500} = 1.7250 \times 10^{10}$$

The torsional constant (C_x) of the column

$$C_x = \frac{(1-(0.63D/C_{1x}))(D^3 C_{1x})}{3} = \frac{(1-(0.63 \times 260/300))(260^3 \times 300)}{3}$$

$$= 7.9795 \times 10^8$$

The clear span (L_{nx}) along the x-x direction

$$L_{nx} = L_x - C_{1x} = 6000 - 300 = 5700 \text{ mm}$$

The torsional stiffness (K_{tx}) of the column along x-x direction [Gilbert, 2000]

$$K_{tx} = \frac{2 \times 4.5 \times E_c \times C_x}{\left(L_y\left(1-\left(\frac{C_{1y}}{L_y}\right)\right)^3\right)} = \frac{2 \times 4.5 \times 22361 \times 7.9795 \times 10^8}{\left(6000\left(1-\left(\frac{300}{6000}\right)\right)^3\right)} = 3.1216 \times 10^{10}$$

The relative stiffness of the slab (K_{s1x}) along the x-x direction

$$K_{s1x} = \frac{4E_c I_{fxs}}{L_x} = \frac{4 \times 22361 \times 8.788 \times 10^9}{6000} = 1.3100 \times 10^{11}$$

$$\alpha_{1x} = \frac{K_{ecx}}{2K_{s1x}} = \frac{1.6388 \times 10^{10}}{2 \times 1.3100 \times 10^{11}} = 0.0625$$

The distribution factor for the positive moment (D_{px}) in the exterior span

$$D_{px} = 0.63 - \frac{0.28}{\left(1 + \frac{1}{\alpha_{1x}}\right)} = 0.63 - \frac{0.28}{\left(1 + \frac{1}{0.0625}\right)} = 0.6135$$

The distribution factor for the negative moment (D_{nex}) in the exterior span

$$D_{nex} = \frac{0.65}{\left(1 + \frac{1}{\alpha_{1x}}\right)} = \frac{0.65}{\left(1 + \frac{1}{0.0625}\right)} = 0.038$$

The distribution factor for the negative moment (D_{nix}) in the interior span

$$D_{nix} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{\alpha_{1x}}\right)} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{0.0625}\right)} = 0.7441$$

The cracked moment M_{cr}

$$M_{cr} = \frac{0.7\sqrt{f_{ck}} \times I_{cx}}{0.5 \times D} = \frac{0.7 \times \sqrt{20} \times 4.394 \times 10^{9}}{0.5 \times 260} = 1.0581 \times 10^{8}\,\text{Nmm}$$

The positive moment due to total load (M_{at1px}) at exterior span

$$M_{at1px} = D_{px} M_{t1x} = 0.6135 \times 158.625 = 97.316\,\text{kNm}$$

If $M_{at1px} < M_{cr}$ then

$$M_{at1px} = M_{cr}$$

97.316 kNm < 105.81 kNm

Hence $M_{at1px} = M_{cr} = 107.38$ kNm

The negative moment due to total load (M_{atnex}) at exterior span

$$M_{atnex} = D_{nex} M_{t1x} = 6.069\,\text{kNm}$$

If $M_{at1nex} < M_{cr}$ then

$$M_{at1nex} = M_{cr}$$

6.069 kNm < 105.81 kNm

Hence $M_{at1nex} = M_{cr} = 105.81$ kNm

The negative moment due to total load (M_{at1nix}) at interior span

$$M_{at1nix} = D_{nix} M_{t1x} = 0.7441 \times 158.025 = 117.586\,\text{kNm}$$

If $M_{at1nix} < M_{cr}$ then

$$M_{at1nix} = M_{cr}$$

Here 118.03 kNm > 105.81 kNm

Hence $M_{at1nix} = 118.03$ kNm

The cracked moment of inertia of the column strip (I_{crx}) along the x-x direction

$$I_{crx} = b_{cx}\frac{x^3}{3} + (m)(A_{st1cx})(d-x)^2$$

$$= 3000\times\frac{x^3}{3} + 8.94\times1950\times(230-x)^2 = 7.6614\times10^8\ \text{mm}^4$$

The effective moment of inertia for the positive moment (I_{ecpx}) at the exterior span along the x-x direction

$$I_{ecpx} = \left(\frac{M_{cr}}{M_{at1px}}\right)^3 (I_{cx}) + \left(1-\left(\frac{M_{cr}}{M_{at1px}}\right)^3\right)(I_{crx})$$

$$= \left(\frac{105.81}{105.81}\right)^3 (4.394\times10^9) + \left(1-\left(\frac{105.81}{105.81}\right)^3\right)(7.6614\times10^8)$$

$$= 4.3940\times10^9\,\text{mm}^4$$

The effective moment of inertia for the negative moment (I_{ecnex}) at the exterior span along the x-x direction

$$I_{ecnex} = \left(\frac{M_{cr}}{M_{at1nex}}\right)^3 (I_{cx}) + \left(1-\left(\frac{M_{cr}}{M_{at1nex}}\right)^3\right)(I_{crx})$$

$$I_{ecnex} = \left(\frac{105.81}{105.81}\right)^3 (4.394\times10^9) + \left(1-\left(\frac{105.81}{105.81}\right)^3\right)(7.6614\times10^8)$$

$$= 4.3940 \times 10^9\ \text{mm}^4$$

The effective moment of inertia for the negative moment (I_{ecnix}) at the interior span along the x-x direction

$$I_{ecnix} = \left(\frac{M_{cr}}{M_{at1nix}}\right)^3 (I_{cx}) + \left(1-\left(\frac{M_{cr}}{M_{at1nix}}\right)^3\right)(I_{crx}) = 4.3940\times10^9\,\text{mm}^4$$

$$I_{ecnix} = \left(\frac{105.81}{118.03}\right)^3 (4.394\times10^9) + \left(1-\left(\frac{105.81}{118.03}\right)^3\right)(7.6614\times10^8)$$

$$= 3.3795\times10^9\ \text{mm}^4$$

The effective moment of inertia for the column strip (I_{ecx}) along the x-x direction

$$I_{ecx} = 0.7I_{ecpx} + 0.15I_{ecnex} + 0.15I_{ecnix}$$

$$= 0.7 \times 4.3940 \times 10^9 + 0.15 \times 4.394 \times 10^9 + 0.15 \times 3.3795 \times 10^9$$

$$= 4.2418 \text{ x } 10^9 \text{mm}^4$$

The effective moment of inertia for the frame (I_{efx}) along the x-x direction

$$I_{efx} = I_{ecx} + I_{mx} = 4.2418 \text{ x } 10^9 + 4.394 \text{ x } 10^9 = 8.6358 \text{ x } 10^9 \text{ mm}^4$$

$$M_{ox} = \frac{W_t \times L_y \times (L_{nx})^2}{8} = \frac{11.75 \times 6 \times 5.7^2}{8} = 286.318 \text{ kNm}$$

$$M_{nt1x} = \left(\frac{0.65 \times M_{ox}}{1 + \frac{1}{\alpha_{1x}}}\right) = \left(\frac{0.65 \times 286.318}{1 + \frac{1}{0.0625}}\right) = 1.095 \text{ x } 10^7 \text{Nmm}$$

$$M_{nd1x} = \left(\frac{W_d}{W_t}\right) M_{nt1x} = \left(\frac{8.75}{11.75}\right) \times 1.0955 \times 10^7 = 8.1582 \times 10^6 \text{ Nmm}$$

The rotation due to total load (θ_{t1x}) along the shorter x-x direction

$$\theta_{t1x} = \frac{M_{nt1x}}{K_{ecx}} = \frac{1.0955 \times 10^7}{1.6388 \times 10^{10}} = 6.69 \times 10^{-4}$$

The rotation due to sustained load (θ_{d1x}) along the shorter x-x direction

$$\theta_{d1x} = \frac{M_{nd1x}}{K_{ecx}} = \frac{8.1582 \times 10^6}{1.6388 \times 10^{10}} = 4.98 \times 10^{-4}$$

The deflection due to total load (δ_{t1x}) along the shorter x-x direction

$$\delta_{t1x} = \left(\frac{\theta_{t1x} \times L_x}{8}\right) = \left(\frac{6.69 \times 10^{-4} \times 6000}{8}\right) = 0.50 \text{ mm}$$

The deflection due to sustained load (δ_{d1x}) along the shorter x-x direction

$$\delta_{d1x} = \left(\frac{\theta_{d1x} \times L_x}{8}\right) = \left(\frac{4.98 \times 10^{-4} \times 6000}{8}\right) = 0.37 \text{ mm}$$

Y-Y axis

The moment of inertia of the column (I_{c1y}) along the y-y direction

$$I_{c1y} = \frac{C_{1y} C_{1x}^3}{12} = \frac{300 \times 300^3}{12} = 675000000 \text{ mm}^4$$

The relative stiffness of the column (K_{cy}) along the y-y direction

$$K_{cy} = \frac{4E_c I_{cly}}{L_c} = \frac{4\times 22361\times 675000000}{3500} = 1.7250\times 10^{10}$$

The torsional constant of the column along the y-y direction

$$C_y = \frac{\left(1-\left(0.63D/C_{1y}\right)\right)\left(D^3 C_{1y}\right)}{3} = \frac{\left(1-\left(0.63\times 260/300\right)\right)\left(260^3\times 300\right)}{3}$$

$$= 7.9795\times 10^8$$

The clear span (L_{ny}) along the y-y direction

$$L_{ny} = L_y - C_{1y} = 6000\text{-}300 = 5700 \text{ mm}$$

The torsional stiffness of the column (K_{ty}) along the y-y direction

$$K_{ty} = \frac{2\times 4.5\times E_c\times C_y}{\left(L_x\left(1-\left(\frac{C_{1x}}{L_x}\right)\right)^3\right)} = \frac{2\times 4.5\times 22361\times 7.9795\times 10^8}{\left(6000\left(1-\left(\frac{300}{6000}\right)\right)^3\right)} = 3.1216\times 10^{10}$$

The relative stiffness of the slab (K_{sly}) along the y-y direction

$$K_{sly} = \frac{4E_c I_{fys}}{L_y} = \frac{4\times 22361\times 8.788\times 10^9}{6000} = 1.3100\times 10^{11}$$

$$\alpha_{1y} = \frac{K_{ecy}}{2K_{sly}} = \frac{1.6388\times 10^{10}}{2\times 1.3100\times 10^{11}} = 0.0625$$

Exterior span

The distribution factor for the positive moment (D_{py}) in the exterior span along the y-y direction

$$D_{py} = 0.63 - \frac{0.28}{\left(1+ 1/\alpha_{1y}\right)} = \frac{0.65}{\left(1+ 1/0.0625\right)} = 0.6135$$

The distribution factor for the negative moment (D_{ney}) in the exterior span along the y-y direction

$$D_{ney} = \frac{0.65}{\left(1+ 1/\alpha_{1y}\right)} = \frac{0.65}{\left(1+ 1/0.0625\right)} = 0.0383$$

The distribution factor for the negative moment (D_{nly}) in the interior span along the y-y direction

$$D_{niy} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{\alpha_y}\right)} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{0.0625}\right)} = 0.7441$$

The positive moment due to total load (M_{at1py}) at exterior span along y-y direction

$$M_{at1py} = D_{py} M_{t1y} = 0.6135 \times 158.625 = 97.319 \text{ kNm}$$

If $M_{at1py} < M_{cr}$ then

$$M_{at1py} = M_{cr}$$

97.319 kNm < 105.81 kNm

Hence $M_{at1py} = M_{cr} = 105.81$ kNm

The negative moment due to total load (M_{at1ney}) at the exterior span along the y-y direction

$$M_{at1ney} = D_{ney} M_{t1y} = 6.069 \text{ kNm}$$

If $M_{at1ney} < M_{cr}$, then

$$M_{at1ney} = M_{cr}$$

6.069 kNm < 105.81 kNm

The negative moment due to total load (M_{at1niy}) at the interior span along the y-y direction

$$M_{atniy} = D_{niy} M_{t1y} = 118.035 \text{ kNm}$$

If $M_{at1niy} < M_{cr}$ then

$$M_{at1niy} = M_{cr}$$

The cracked moment of inertia of the column strip (I_{cry}) along the y-y direction

$$I_{cry} = b_{cy} \frac{x^3}{3} + (m)(A_{st1cy})(d - x)^2 \text{ mm}^4$$

$$= 3000 \times \frac{x^3}{3} + 8.94 \times 1950 \times (230 - x)^2 = 7.6614 \times 10^8 \text{ mm}^4$$

The effective moment of inertia for the positive moment (I_{ecpy}) at the exterior span along the y-y direction

$$I_{ecpy} = \left(\frac{M_{cr}}{M_{at1py}}\right)^3 (I_{cy}) + \left(1 - \left(\frac{M_{cr}}{M_{at1py}}\right)^3\right)(I_{cry})$$

$$= \left(\frac{105.81}{105.81}\right)^3 (4.394\times10^9) + \left(1-\left(\frac{105.81}{105.81}\right)^3\right)(7.6614\times10^8)$$

$$= 4.3940\times10^9 \text{ mm}^4$$

The effective moment of inertia for the negative moment (I_{ecney}) at the exterior span along the y-y direction

$$I_{ecney} = \left(\frac{M_{cr}}{M_{at1ney}}\right)^3 (I_{cy}) + \left(1 - \left(\frac{M_{cr}}{M_{at1ney}}\right)^3\right)(I_{cry})$$

$$= \left(\frac{105.81}{105.81}\right)^3 (4.394\times10^9) + \left(1-\left(\frac{105.81}{105.81}\right)^3\right)(7.6614\times10^8)$$

$$= 4.3940\times10^9 \text{mm}^4$$

The effective moment of inertia for the negative moment (I_{ecniy}) at the interior span along the y-y direction

$$I_{ecniy} = \left(\frac{M_{cr}}{M_{at1niy}}\right)^3 (I_{cy}) + \left(1 - \left(\frac{M_{cr}}{M_{at1niy}}\right)^3\right)(I_{cry})$$

$$= \left(\frac{105.81}{117.586}\right)^3 (4.394\times10^9) + \left(1-\left(\frac{105.81}{117.586}\right)^3\right)(7.6614\times10^8)$$

$$= 4.3940\times10^9 \text{ mm}^4$$

The effective moment of inertia for the column strip (I_{ecy}) along the y-y direction

$$I_{ecy} = 0.7I_{ecpy} + 0.15I_{ecney} + 0.15I_{ecniy}$$

$$= 0.7\times4.3940\times10^9 + 0.15\times4.394\times10^9 + 0.15\times4.394\times10^9$$

$$= 4.3940\times10^9 \text{ mm}^4$$

The effective moment of inertia for the frame (I_{efy}) along the y-y direction

$$I_{efy} = I_{ecy} + I_{my}$$

$$= 4.394 \text{ x } 10^9 + 4.394 \text{ x } 10^9 = 8.7880 \times 10^9 \text{ mm}^4$$

$$M_{oy} = \frac{W_t L_x (L_{ny})^2}{8} = \frac{11.75 \times 6 \times 5.7^2}{8} = 286.3181 \text{ kNm}$$

$$M_{nt1y} = \left(\frac{0.65 \times M_{oy}}{1 + \frac{1}{\alpha_{1y}}} \right) = \left(\frac{0.65 \times 286.318}{1 + \frac{1}{0.0625}} \right) = 1.0955 \text{ x } 10^7 \text{ kNm}$$

$$M_{ndy} = \left(\frac{W_d}{W_t} \right) M_{nt1y} = \left(\frac{8.75}{11.75} \right) \times 1.0955 \times 10^7 = 8.1582 \times 10^6 \text{ Nmm}$$

The rotation due to total load (θ_{t1y}) along the y-y direction

$$\theta_{t1y} = \frac{M_{nt1y}}{K_{ecy}} = \frac{1.0955 \times 10^7}{1.6388 \times 10^{10}} = 6.6850 \times 10^{-4}$$

The rotation due to sustained load (θ_{d1y}) along the y-y direction

$$\theta_{d1y} = \frac{M_{nd1y}}{K_{ecy}} = \frac{8.1582 \times 10^6}{1.6388 \times 10^{10}} = 4.9782 \times 10^{-4}$$

The deflection due to total load (δ_{t1y}) along the y-y direction

$$\delta_{t1y} = \left(\theta_{t1y} \times \frac{L_y}{8} \right) = \left(6.69 \times 10^{-4} \times \frac{6000}{8} \right) = 0.50 \text{ mm}$$

The deflection due to sustained load (δ_{d1y}) along the y-y direction

$$\delta_{d1y} = \frac{\theta_{d1y} \times L_y}{8} = \left(4.98 \times 10^{-4} \times \frac{6000}{8} \right) = 0.37 \text{ mm}$$

Deflection of column strip due to total load along x-x direction

Interior panel

The deflection of the column strip (δ_{cxitl}) due to total load

$$\delta_{cxitl} = \frac{(LDF_{ci}) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} = \frac{0.675 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 1.63 \text{ mm}$$

The deflection of the column strip (δ_{cxidl}) due to sustained load

$$\delta_{cxidl} = \frac{(LDF_{ci}) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} = \frac{0.675 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 1.22 \text{ mm}$$

The deflection of the column strip (δ_{cxill}) due to varying part of live load

$$\delta_{cxill} = \delta_{cxitl} - \delta_{cxidl} = 1.63\text{-}1.22 = 0.41 \text{ mm}$$

Corner panel

The deflection of the column strip (δ_{cxctl}) due to total load

$$\delta_{cxctl} = \frac{(LDF_c) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} + (\delta_{t1x} - \delta_{d1x})$$

$$= \frac{0.55 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} + (0.5 - 0.37) = 1.46 \text{ mm}$$

The deflection of the column strip (δ_{cxcdl}) due to sustained load

$$\delta_{cxcdl} = \frac{(LDF_c) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} + (\delta_{t1x} - \delta_{d1x})$$

$$= \frac{0.55 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} + (0.5 - 0.37) = 1.12 \text{ mm}$$

The deflection of the column strip (δ_{cxcll}) due to varying part of live load

$$\delta_{cxcll} = \delta_{cxctl} - \delta_{cxcdl} = 1.46\text{-}1.12 = 0.34 \text{ mm}$$

Side panel

The deflection of the column strip (δ_{cxstl}) due to total load

$$\delta_{cxstl} = \frac{(LDF_c) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} = \frac{0.55 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 1.33 \text{ mm}$$

The deflection of the column strip (δ_{cxsdl}) due to sustained load

$$\delta_{cxsdl} = \frac{(LDF_c) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} = \frac{0.55 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 0.99 \text{ mm}$$

The deflection of the column strip (δ_{cxsll}) due to varying part of live load

$$\delta_{cxsll} = \delta_{cxstl} - \delta_{cxsdl} = 1.33\text{-}0.99 = 0.34 \text{ mm}$$

Deflection of column strip due to total load along y-y direction

Interior panel

The deflection of the column strip (δ_{cyitl}) due to total load

$$\delta_{cyitl} = \frac{(LDF_{ci}) \times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} = \frac{0.675 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 1.63 \text{ mm}$$

The deflection of the column strip (δ_{cyidl}) due to sustained load

$$\delta_{cyidl} = \frac{(LDF_{ci}) \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} = \frac{0.675 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = = 1.22 \text{ mm}$$

The deflection of the column strip (δ_{cyill}) due to varying part of live load

$$\delta_{cyill} = \delta_{cyitl} - \delta_{cyidl} = 1.63\text{-}1.22 = 0.41 \text{ mm}$$

Corner panel

The deflection of the column strip (δ_{cyctl}) due to total load

$$\delta_{cyctl} = \frac{(LDF_c) \times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.55 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} + (0.5 - 0.37) = 1.46 \text{ mm}$$

The deflection of the column strip (δ_{cycdl}) due to sustained load

$$\delta_{cycdl} = \frac{(LDF_c) \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.55 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} + (0.5 - 0.37) = 1.12 \text{mm}$$

The deflection of the column strip (δ_{cycll}) due to varying part of live load

$$\delta_{cycll} = \delta_{cyctl} - \delta_{cycdl} = 1.46\text{-}1.12 = 0.34 \text{ mm}$$

Side panel

The deflection of column strip (δ_{cystl}) due to total load

$$\delta_{cystl} = \frac{(LDF_c)\times W_t \times L_x \times L_y^4}{384\times E_c \times I_{cy}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.55\times 11.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} + (0.5 - 0.37) = 1.46 \text{ mm}$$

The deflection of the column strip (δ_{cysdl}) due to sustained load

$$\delta_{cysdl} = \frac{(LDF_c)\times W_d \times L_x \times L_y^4}{384\times E_c \times I_{cy}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.55\times 8.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} + (0.5 - 0.37) = 1.12\text{mm}$$

The deflection of the column strip (δ_{cysll}) due to varying part of live load

$$\delta_{cysll} = \delta_{cystl} - \delta_{cysdl} = 1.46\text{-}1.12 = 0.34 \text{ mm}$$

Deflection of middle strip due to total load along x-x direction

Interior panel

The deflection of the middle strip (δ_{mxitl}) due to total load

$$\delta_{mxitl} = \frac{(LDF_{mi})\times W_t \times L_y \times L_x^4}{384\times E_c \times I_{mx}} = \frac{0.325\times 11.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} = 0.79 \text{ mm}$$

The deflection of the middle strip (δ_{mxidl}) due to sustained load

$$\delta_{mxidl} = \frac{(LDF_{mi})\times W_d \times L_y \times L_x^4}{384\times E_c \times I_{mx}} = \frac{0.325\times 8.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} = 0.58 \text{ mm}$$

The deflection of the middle strip (δ_{mxill}) due to varying part of live load

$$\delta_{mxill} = \delta_{mxitl} - \delta_{mxidl} = 0.79\text{-}0.58 = 0.20 \text{ mm}$$

Corner panel

The deflection of the middle strip (δ_{mxctl}) due to total load

$$\delta_{mxctl} = \frac{(LDF_m)\times W_t \times L_y \times L_x^4}{384\times E_c \times I_{mx}} + (\delta_{t1x} - \delta_{d1x})$$

$$= \frac{0.45\times 11.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} + (0.5 - 0.37) = 1.22 \text{ mm}$$

The deflection of the middle strip (δ_{mxcdl}) due to sustained load

$$\delta_{mxcdl} = \frac{(LDF_m) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} + (\delta_{t1x} - \delta_{d1x})$$

$$= \frac{0.45 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} + (0.5 - 0.37) = 0.94 \text{ mm}$$

The deflection of the middle strip (δ_{mxcll}) due to varying part of live load

$$\delta_{mxcll} = \delta_{mxctl} - \delta_{mxcdl} = 1.22\text{-}0.94 = 0.28 \text{ mm}$$

Side panel

The deflection of the middle strip (δ_{mxstl}) due to total load

$$\delta_{mxstl} = \frac{(LDF_m) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} = \frac{0.45 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 1.09 \text{ mm}$$

The deflection of the middle strip (δ_{mxsdl}) due to sustained load

$$\delta_{mxsdl} = \frac{(LDF_m) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} = \frac{0.45 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 0.81 \text{ mm}$$

The deflection of the middle strip (δ_{mxsll}) due to varying part of live load

$$\delta_{mxsll} = \delta_{mxstl} - \delta_{mxsdl} = 1.09\text{-}0.81 = 0.28 \text{ mm}$$

Deflection of middle strip due to total load along y-y direction

Interior panel

The deflection of the middle strip (δ_{myitl}) due to total load

$$\delta_{myitl} = \frac{(LDF_{mi}) \times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{my}} = \frac{0.325 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 0.79 \text{ mm}$$

The deflection of the middle strip (δ_{myidl}) due to sustained load

$$\delta_{myidl} = \frac{(LDF_{mi}) \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{my}} = \frac{0.325 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.394 \times 10^9} = 0.59 \text{ mm}$$

The deflection of the middle strip (δ_{myill}) due to varying part of live load

$$\delta_{myill} = \delta_{myitl} - \delta_{myidl} = 0.79\text{-}0.58 = 0.20 \text{ mm}$$

Corner panel

The deflection of the middle strip (δ_{myctl}) due to total load

$$\delta_{myctl} = \frac{(LDF_m)\times W_t \times L_x \times L_y^4}{384\times E_c \times I_{my}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.45\times 11.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} + (0.5 - 0.37) = 1.22\text{mm}$$

The deflection of the middle strip (δ_{mycdl}) due to sustained load

$$\delta_{mycdl} = \frac{(LDF_m)\times W_d \times L_x \times L_y^4}{384\times E_c \times I_{my}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.45\times 8.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} + (0.5 - 0.37) = 0.94\text{mm}$$

The deflection of the middle strip (δ_{mycll}) due to varying part of live load

$$\delta_{mycll} = \delta_{myctl} - \delta_{mycdl} = 1.22\text{-}0.94 = 0.28 \text{ mm}$$

Side panel

The deflection of the middle strip (δ_{mystl}) due to total load

$$\delta_{mystl} = \frac{(LDF_m)\times W_t \times L_x \times L_y^4}{384\times E_c \times I_{my}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.45\times 11.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} + (0.5 - 0.37) = 1.22\text{mm}$$

The deflection of the middle strip (δ_{mysdl}) due to sustained load

$$\delta_{mysdl} = \frac{(LDF_m)\times W_d \times L_x \times L_y^4}{384\times E_c \times I_{my}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.45\times 8.75\times 6\times 6000^4}{384\times 22361\times 4.394\times 10^9} + (0.5 - 0.37) = 0.94\text{mm}$$

The deflection of the middle strip (δ_{mysll}) due to varying part of live load

$$\delta_{mysll} = \delta_{mystl} - \delta_{mysdl} = 1.22\text{-}0.94 = 0.28 \text{ mm}$$

Total deflection

Interior panel

The deflection (δ_{idl}) due to sustained load

$$\delta_{idl} = \left(\frac{\delta_{cxidl} + \delta_{myidl} + \delta_{cyidl} + \delta_{mxidl}}{2} \right)$$

$$\delta_{idl} = \left(\frac{1.21 + 0.59 + 1.21 + 0.59}{2} \right) = 1.80 \text{ mm}$$

The deflection (δ_i) of the interior panel

$$\delta_i = 6 \times \delta_{idl} + \delta_{ill}$$

$$\delta_i = 6 \times 1.80 + 0.61 = 11.41 \text{ mm}$$

Corner panel

The deflection (δ_{cdl}) due to sustained load

$$\delta_{cdl} = \left(\frac{\delta_{cxcdl} + \delta_{mycdl} + \delta_{cycdl} + \delta_{mxcdl}}{2} \right)$$

$$\delta_{cdl} = \left(\frac{1.12 + 0.94 + 1.12 + 0.94}{2} \right) = 2.06 \text{ mm}$$

The deflection (δ_c) of the corner panel

$$\delta_c = 6 \times \delta_{cdl} + \delta_{cll}$$

$$\delta_c = 6 \times 2.06 + 0.62 = 12.98 \text{ mm}$$

Side panel

The deflection (δ_{sdl}) due to sustained load

$$\delta_{sdl} = \left(\frac{\delta_{cxsdl} + \delta_{mysdl} + \delta_{cysdl} + \delta_{mxsdl}}{2} \right)$$

$$\delta_{sdl} = \left(\frac{1.12 + 0.81 + 1.12 + 0.81}{2} \right)$$

The deflection (δ_s) of the side panel

$$\delta_s = 6 \times \delta_{sdl} + \delta_{sll}$$

$$\delta_s = 6 \times 1.93 + 0.62 = 12.21 \text{ mm}$$

6.2.2 Example 2: Deflection of Flat Plate with concealed beam

The floor plan of floor with concealed beam is given in Fig. 6.3.

Live load (Wl) = 3.0 kN/m^2

Floor finish (FF) = 1.5 kN/m^2

Clear cover to tension reinforcement = 25 mm

Grade of concrete = $f_{ck} = 20$ N/mm^2

Young's modulus of steel E_s = 200000 N/mm^2

Diameter of rod = 10 mm

Square column of size ($C1 \times C2$) = 300 mm x 300 mm

Total depth of slab = 260 mm

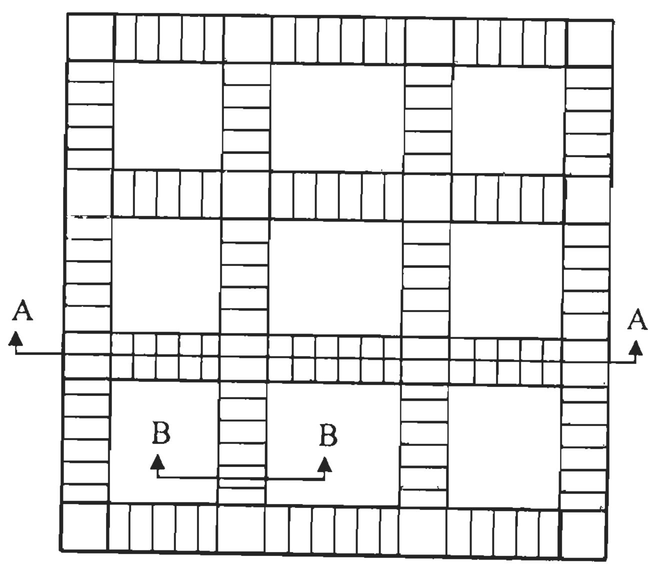


Fig. 6.3 Floor plan with concealed beam

Concealed beam

The modular ratio

$$m = \frac{E_s}{E_c} = 8.94$$

The width (l_1) of the concealed beam (Fig. 6.4)

$$l_1 = 1000 \text{ mm}$$

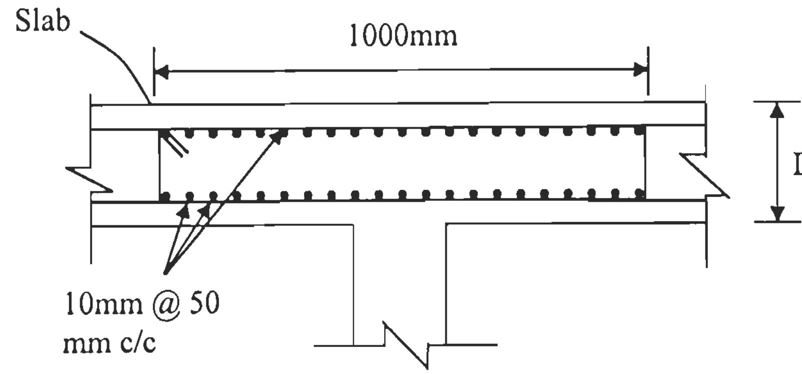


Fig. 6.4 Cross section of the concealed beam (section B-B)

Number of the longitudinal reinforcements (n)

$$n = \left(l_1 / 50 \right) + 1$$

$$n = \left(1000 / 50 \right) + 1 = 21 \text{ Nos.}$$

Area of the reinforcements (A_s)

$$A_s = \left(n \times \pi \times dia^2 / 4 \right)$$

$$A_s = \left(21 \times \pi \times 10^2 / 4 \right) = 1.6493 \times 10^3 \text{ mm}^2$$

The depth (d_{con}) of the concealed beam

$$d_{con} = \left(A_s / l_1 \right)$$

$$d_{con} = \left(1.6493 \times 10^3 / 1000 \right) = 1.65 \text{ mm}$$

The depth (d_2) of the compression reinforcement

$$d_2 = \left(cc + dia / 2 \right)$$

$$d_2 = \left(25 + 10 / 2 \right) = 30\text{mm}$$

The width of the column strip b_{cx} and b_{cy} along x-x and y-y direction respectively

$$b_{cx} = b_{cy} = 0.5 \times L_x = 3000 \text{ mm}$$

The area of the tension reinforcement provided in the column strip (A_{st1cx}) along the shorter x-x direction

$$A_{st1cx} = \frac{0.5\times 0.5\times b_{cx}\times D}{100} = \frac{0.5\times 0.5\times 3000\times 260}{100} = 1950\ \text{mm}^2$$

The area of the compression reinforcement provided in the column strip (A_{sc1cx}) along the shorter x-x direction

$$A_{sc1cx} = \frac{0.5\times b_{cx}\times D}{100} = \frac{0.5\times 3000\times 260}{100} = 3900\ \text{mm}^2$$

The area of the tension reinforcement provided in the column strip (A_{st1cy}) along the longer y-y direction

$$A_{st1cy} = \frac{0.5\times 0.5\times b_{cy}\times D}{100} = \frac{0.5\times 0.5\times 3000\times 260}{100} = 1950\ \text{mm}^2$$

The gross moment of inertia of the column strip (I_{cx}) along the x-x direction

$$I_{cx} = \frac{b_{cx}\times D^3}{12} + 2\times(m-1)\times A_s\times\left(D/2 - cc\right)^2$$

$$= \frac{3000\times 260^3}{12} + 2\times(8.94-1)\times 1.6493\times 10^3\times\left(260/2 - 25\right)^2 = 4.6829\times 10^9\ \text{mm}^4$$

The gross moment of inertia of the column strip (I_{cy}) along the y-y direction

$$I_{cy} = \frac{b_{cy}\times D^3}{12} + 2\times(m-1)\times A_s\times\left(D/2 - cc\right)^2$$

$$= \frac{3000\times 260^3}{12} + 2\times(8.94-1)\times 1.6493\times 10^3\times\left(260/2 - 25\right)^2 = 4.6829\times 10^9\ \text{mm}^4$$

Frame

The width of the frame along the x-x direction (b_{fx}) is equal to the length (Fig. 6.5) along the span in the y-y direction.

$$b_{fx} = L_y = 6000\ \text{mm}$$

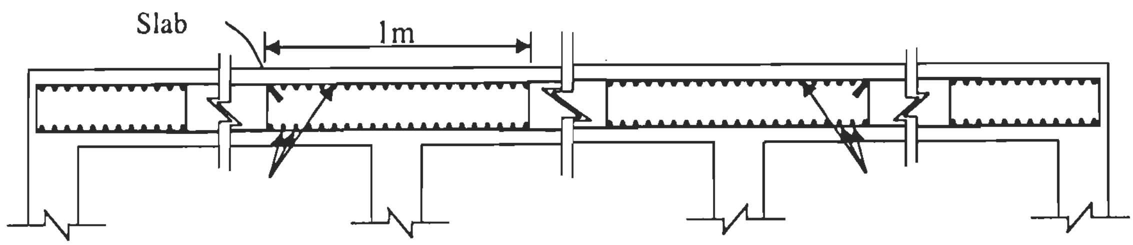


Fig. 6.5 Cross section of the concealed beam (section A-A)

The gross moment of inertia of the frame (I_{fx}) along the x-x direction

$$I_{fx} = \frac{b_{fx} \times D^3}{12} + 2 \times (m-1) \times A_s \times \left(D/_2 - cc\right)^2$$

$$I_{fx} = \frac{6000 \times 260^3}{12} + 2 \times (8.94 - 1) \times 1.6493 \times 10^3 \times \left(260/_2 - 25\right)^2 = 9.7069 \times 10^9 \text{ mm}^4$$

The width of the frame along the y-y direction (b_{fy}) is equal to the length along the span in the x-x direction.

$$b_{fy} = L_x = 6000\ mm$$

The gross moment of inertia of the frame (I_{fy}) along the y-y direction

$$I_{fy} = \frac{b_{fy} \times D^3}{12} + 2 \times (m-1) \times A_s \times \left(D/_2 - cc\right)^2$$

$$I_{fy} = \frac{6000 \times 260^3}{12} + 2 \times (8.94 - 1) \times 1.6493 \times 10^3 \times \left(260/_2 - 25\right)^2 = 9.7069 \times 10^9 \text{ mm}^4$$

Middle strip

The width of the middle strip (b_{mx}) along the x-x direction

$$b_{mx} = L_y - L_x / 2 = 6000 - 6000/2 = 3000 \text{ mm}$$

The gross moment of inertia of the middle strip (I_{mx}) along the x-x direction

$$I_{mx} = \frac{b_{mx} \times D^3}{12} = \frac{3000 \times 260^3}{12} = 4.3940 \times 10^9 \text{ mm}^4$$

The width of the middle strip (b_{my}) along the y-y direction

$$b_{my} = L_x / 2 = 6000/2 = 3000 \text{ mm}$$

The gross moment of inertia of the middle strip (I_{my}) along the y-y direction

$$I_{my} = \frac{b_{my} \times D^3}{12} = \frac{3000 \times 260^3}{12} = 4.3940 \times 10^9 \text{ mm}^4$$

Flexural stiffness of equivalent column

$$L_c = 3.5 \text{ m}$$

x-x axis

Short term deflection

The moment of inertia of the column (I_{c1x}) along the x-x direction

$$I_{c1x} = \frac{C_{1x} C_{1y}^3}{12} = \frac{300 \times 300^3}{12} = 675000000 \text{ mm}^4$$

The relative stiffness of the column (K_{cx}) along the x-x direction

$$K_{cx} = \frac{4 E_c I_{c1x}}{L_c} = \frac{4 \times 22361 \times 675000000}{3500} = 1.7250 \times 10^{10}$$

Concealed beam

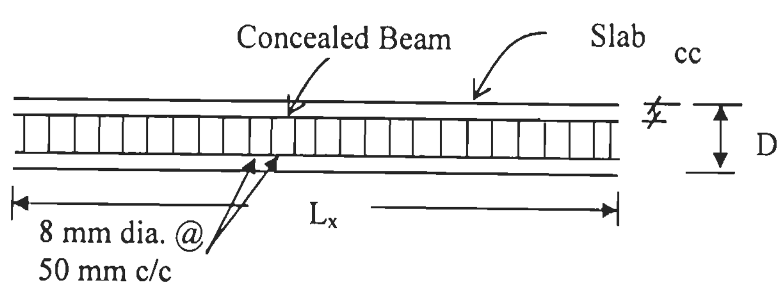


Fig. 6.6 Longitudinal section of concealed beam

The spacing of stirrups (s) in the concealed beam (Fig. 6.6)

$$s = 50 \text{ mm}$$

The diameter (d_s) of the stirrup

$$d_s = 8 \text{ mm}$$

The number of stirrups (n_{sx}) along the x-x direction

$$n_{sx} = \left(\frac{L_y - C_1}{s}\right) + 1 = \left(\frac{6000 - 300}{50}\right) + 1 = 115 \text{ Nos.}$$

The length of the stirrups (L_s) to be provided in the x-x direction

$$L_s = L_y = 6000 \text{ mm}$$

Equivalent area of concealed beam (E_a) in concrete

$$E_a = \frac{n_{sx} \times m \times \pi \times d_s^{\ 2}}{4} = \frac{115 \times 8.94 \times \pi \times 8^2}{4} = 5.17 \text{ x } 10^4 \text{ mm}^2$$

Equivalent area of concealed beam (E_a) in concrete

The thickness (t_x) of the concealed beam (Fig.6.7)

$$t_x = 9 \text{ mm}$$

$$X_1 = 1000 - t_x = 1000\text{-}8.6171 = 991 \text{ mm}$$

$$Y_1 = D - t_x = 260\text{-}9 = 251 \text{ mm}$$

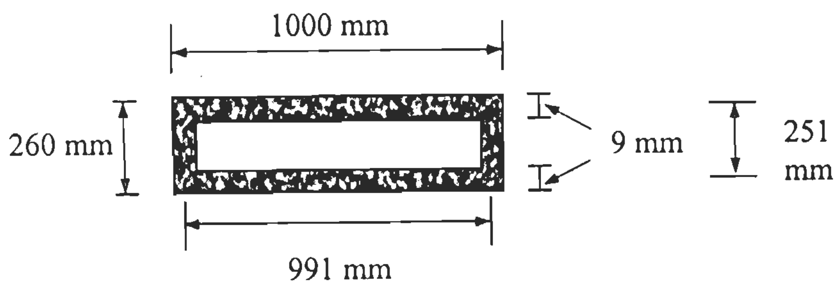


Fig. 6.7 Equivalent thin concrete box for torsion

The torsional constant (C_b) of the concealed beam (Hsu,[1984])

$$C_b = \left(\frac{2 \times X_1^2 \times Y_1^2 \times D}{X_1 + Y_1}\right) = \left(\frac{2 \times 991^2 \times 251^2 \times 260}{991 + 251}\right) = 2.5988 \text{ x } 10^{10}$$

The torsional constant of the column (C_x)

$$C_x = \frac{(1-(0.63D/C_{1x}))(D^3C_{1x})}{3} + C_b =$$

$$\frac{(1-(0.63\times 260/300))(260^3 \times 300)}{3} + 2.5988\times 10^{10} = 2.6786 \text{ x } 10^{10}$$

The clear span (L_{nx}) along the x-x direction

$$L_{nx} = L_x - C_{1x} = 5700 \text{ mm}$$

The torsional stiffness of the column (K_{tx}) along the x-x direction

$$K_{tx} = \frac{2\times 4.5\times E_c \times C_x}{\left(L_y\left(1-\left(\frac{C_{1y}}{L_y}\right)\right)^3\right)} = \frac{2\times 4.5\times 22361\times 2.6786\times 10^{10}}{\left(6000\left(1-\left(\frac{300}{6000}\right)\right)^3\right)} = 1.0479 \text{ x } 10^{12}$$

The relative stiffness of the slab (K_{s1x}) along the x-x direction

$$K_{s1x} = \frac{4E_c I_{fxs}}{L_x} = \frac{4\times 22361\times 9.7069\times 10^9}{6000} = 1.3531 \text{ x } 10^4$$

$$K_{ecx} = 3.34 \text{ x } 10^{10}$$

$$\alpha_{1x} = \frac{K_{ecx}}{2K_{s1x}} = \frac{3.34\times 10^{10}}{2\times 1.3531\times 10^4} = 0.1234$$

Exterior span

The distribution factor for the positive moment (D_{px}) in the exterior span along the x-x direction

$$D_{px} = 0.63 - \frac{0.28}{\left(1+\frac{1}{\alpha_x}\right)} = 0.63 - \frac{0.28}{\left(1+\frac{1}{0.1234}\right)} = 0.5992$$

The distribution factor for the negative moment (D_{nex}) in the exterior span along the x-x direction

$$D_{nex} = \frac{0.65}{\left(1+\frac{1}{\alpha_x}\right)} = \frac{0.65}{\left(1+\frac{1}{0.1234}\right)} = 0.0714$$

The distribution factor for the negative moment (D_{nix}) in the interior span along the x-x direction

$$D_{nix} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{\alpha_x}\right)} = 0.75 - \frac{0.10}{\left(1 + \frac{1}{0.1234}\right)} = 0.739$$

The cracked moment of inertia (M_{cr})

$$M_{cr} = \frac{0.7\sqrt{f_{ck}} \times I_{cx}}{0.5 \times D} = \frac{0.7 \times \sqrt{20} \times 4.6829 \times 10^9}{0.5 \times 260} = 1.0581 \times 10^8$$

The positive moment due to total load (M_{at1px}) at the exterior span along the x-x direction

$$M_{at1px} = D_{px} M_{t1x} = 0.5992 \times 158.625 = 95.05 \text{ kNm}$$

If $M_{at1px} < M_{cr}$ then

$$M_{at1px} = M_{cr}$$

Here 95.05 kNm < 105.81 kNm

The negative moment due to total load (M_{at1nex}) at the exterior span along the x-x direction

$$M_{atnex} = D_{nex} M_{t1x} = 0.0074 \times 158.625 = 11.327 \text{ kNm}$$

If $M_{at1nex} < M_{cr}$ then

$$M_{at1nex} = M_{cr}$$

The negative moment due to total load (M_{at1nix}) at the interior span along the x-x direction

$$M_{atnix} = D_{nix} M_{t1x} = 0.739 \text{ x } 158.625 = 118.03 \text{ kNm}$$

If $M_{at1nix} < M_{cr}$ then

$$M_{at1nix} = M_{cr}$$

The cracked moment of inertia (I_{crx}) of the column strip along the x-x direction

$$I_{crx} = b_{cx}\frac{x^3}{3} + (m)(A_{st1cx})(d-x)^2$$

$$+m \times A_s \times \left(0.5 \times D - \left(cc + \tfrac{dia}{2}\right)\right)^2 + (m-1) \times A_s \times \left(0.5 \times D - \left(cc + \tfrac{dia}{2}\right)\right)^2$$

$$= 2.4755 \times 10^8 mm^4$$

The effective moment of inertia for the positive moment (I_{ecpx}) at the exterior span along the x-x direction

$$I_{ecpx} = \left(\frac{M_{cr}}{M_{at1px}}\right)^3 (I_{cx}) + \left(1 - \left(\frac{M_{cr}}{M_{at1px}}\right)^3\right)(I_{crx})$$

$$= \left(\frac{105.81}{105.81}\right)^3 \left(4.6829 \times 10^9\right) + \left(1 - \left(\frac{105.81}{105.81}\right)^3\right)\left(2.4755 \times 10^8\right)$$

$$= 4.6829 \times 10^9 \text{ mm}^4$$

The effective moment of inertia for the negative moment (I_{ecnex}) at the exterior span along the x-x direction

$$I_{ecnex} = \left(\frac{M_{cr}}{M_{at1nex}}\right)^3 (I_{cx}) + \left(1 - \left(\frac{M_{cr}}{M_{at1nex}}\right)^3\right)(I_{crx})$$

$$= \left(\frac{105.81}{105.81}\right)^3 \left(4.6829 \times 10^9\right) + \left(1 - \left(\frac{105.81}{105.81}\right)^3\right)\left(2.4755 \times 10^8\right)$$

$$= 4.6829 \times 10^9 \text{ mm}^4$$

The effective moment of inertia for the negative moment (I_{ecnix}) at the interior span along the x-x direction

$$I_{ecnix} = \left(\frac{M_{cr}}{M_{at1nix}}\right)^3 (I_{cx}) + \left(1 - \left(\frac{M_{cr}}{M_{at1nix}}\right)^3\right)(I_{crx})$$

$$= \left(\frac{105.81}{105.81}\right)^3 \left(4.6829 \times 10^9\right) + \left(1 - \left(\frac{105.81}{105.81}\right)^3\right)\left(2.4755 \times 10^8\right)$$

$$= 4.6829 \times 10^9 \text{ mm}^4$$

The effective moment of inertia (I_{ecx}) for the column strip along the x-x direction

$$I_{ecx} = 0.7 I_{ecpx} + 0.15 I_{ecnex} + 0.15 I_{ecnix}$$

$$= 0.7 \text{ x } 4.6829 \text{ x } 10^9 + 0.15 \text{ x } 4.6829 \text{ x } 10^9 + 0.15 \text{ x } 4.6829 \text{ x } 10^9$$

$$= 4.6829 \times 10^9 \text{ mm}^4$$

The effective moment of inertia for the frame (I_{efx}) along the x-x direction

$$I_{efx} = I_{ecx} + I_{mz}$$

$$M_{ox} = \frac{W_t \times L_y \times (L_{nx})^2}{8} = \frac{11.75 \times 6 \times (5.7)^2}{8} = 286.318 \text{ kNm}$$

$$M_{nt1x} = 2.0446 \times 10^7 \text{ Nmm}$$

$$M_{nd1x} = \left(\frac{W_d}{W_t}\right) M_{nt1x} = \left(\frac{8.75}{11.75}\right) \times 2.0446 \times 10^7 = 1.5225 \times 10^7 \text{Nmm}$$

The rotation due to total load (θ_{t1x}) along the x-x direction

$$\theta_{t1x} = \frac{M_{nt1x}}{K_{ecx}} = \frac{2.0446 \times 10^7}{K_{ecx}} = 6.1215 \times 10^{-4}$$

The rotation due to sustained load (θ_{d1x}) along the x-x direction

$$\theta_{d1x} = \frac{M_{nd1x}}{K_{ecx}} = \frac{1.5225 \times 10^7}{K_{ecx}} = 4.5586 \text{ x } 10^{-4}$$

The deflection due to total load (δ_{t1x}) along the x-x direction

$$\delta_{t1x} = \left(\theta_{t1x} \times \frac{L_x}{8}\right) = \left(6.1215 \times 10^{-4} \times \frac{6000}{8}\right) = 0.46 \text{ mm}$$

The deflection due to sustained load (δ_{d1x}) along the x-x direction

$$\delta_{d1x} = \left(\theta_{d1x} \times \frac{L_x}{8}\right) = \left(4.5586 \times 10^{-4} \times \frac{6000}{8}\right) = 0.34 \text{ mm}$$

Y-Y axis

Short term

The moment of inertia of the column (I_{c1y}) along the y-y direction

$$I_{c1y} = \frac{C_{1y} C_{1x}^3}{12} = \frac{300 \times 300^3}{12} = 675000000 \text{ mm}^4$$

The relative stiffness of the column (K_{cy}) along the y-y direction

$$K_{cy} = \frac{4E_c I_{cly}}{L_c} = \frac{4 \times 22361 \times 675000000}{3500} = 1.7250 \text{ x } 10^{10}$$

Concealed beam

The spacing of the stirrups in the concealed beam

$$s = 50 \text{ mm}$$

The diameter of the stirrups in the concealed beam

$$d_s = 8 \text{ mm}$$

The number of the stirrups along the y-y direction

$$n_{sy} = \left(\frac{L_x - C_{1x}}{s}\right) + 1 = \left(\frac{6000 - 300}{50}\right) + 1 = 115 \text{ Nos}$$

The length of the stirrups to be provided in the y-y direction

$$L_{sy} = L_x = 6000 \text{ mm}$$

The area of concealed the beam

$$E_a = \frac{n_{sy} \times m \times \pi \times d_s^{\ 2}}{4} = \frac{115 \times 8.94 \times \pi \times 8^2}{4} = 5.17 \text{ x } 10^4 \text{ mm}^2$$

$$X_2 = 1000 - t_y = 1000\text{-}8.6171 = 991 \text{ mm}$$

$$Y_2 = D - t_y = 260\text{-}8.6171 = 251 \text{ mm}$$

The torsional constant of the concealed beam

$$C_b = \left(\frac{2 \times X_2^2 \times Y_2^2 \times D}{X_2 + Y_2}\right) = \left(\frac{2 \times 991^2 \times 251^2 \times 260}{991 + 251}\right) = 2.5988 \text{ x } 10^{10}$$

The torsional constant of the column along the y-y direction

$$C_y = \frac{\left(1 - \left(0.63D/C_{1y}\right)\right)\left(D^3 C_{1y}\right)}{3} + C_b$$

$$= \frac{\left(1 - \left(0.63 \times 260/300\right)\right)\left(260^3 \times 300\right)}{3} + 2.5988 \times 10^{10} = 2.6786 \text{ x } 10^{10}$$

The clear span (L_{ny}) along the y-y direction

$$L_{ny} = L_y - C_{1y} = 5700 \text{ mm}$$

The torsional stiffness of the column (K_{ty}) along the y-y direction

$$K_{ty} = \frac{2\times4.5\times E_c \times C_y}{\left(L_x\left(1-\left(\frac{C_{1x}}{L_x}\right)\right)^3\right)} = \frac{2\times4.5\times22361\times2.6786\times10^{10}}{\left(6000\left(1-\left(\frac{300}{6000}\right)\right)^3\right)} = 1.0479 \text{ x } 10^{12}$$

The relative stiffness of the slab (K_{s1y}) along the y-y direction

$$K_{s1y} = \frac{4E_c I_{fys}}{L_y} = \frac{4\times22361\times9.7069\times10^9}{6000} = 1.3531 \text{ x } 10^4$$

$$K_{ecy} = 3.34 \text{ x } 10^{10}$$

$$\alpha_{1y} = \frac{K_{ecy}}{2K_{s1y}} = \frac{3.34\times10^{10}}{2\times1.3531\times10^4} = 0.1234$$

Exterior span

The distribution factor for the positive moment (D_{py}) in the exterior span along the y-y direction

$$D_{py} = 0.63 - \frac{0.28}{\left(1+ \frac{1}{\alpha_{1y}}\right)} = 0.63 - \frac{0.28}{\left(1+ \frac{1}{0.1234}\right)} = 0.5992$$

The distribution factor for the negative moment (D_{ney}) in the exterior span along the y-y direction

$$D_{ney} = \frac{0.65}{\left(1+ \frac{1}{\alpha_{1y}}\right)} = \frac{0.65}{\left(1+ \frac{1}{0.1234}\right)} = 0.0714$$

The distribution factor for the negative moment (D_{niy}) in the interior span along the y-y direction

$$D_{niy} = 0.75 - \frac{0.10}{\left(1+ \frac{1}{\alpha_{1y}}\right)} = 0.75 - \frac{0.10}{\left(1+ \frac{1}{0.1234}\right)} = 0.739$$

The positive moment due to total load (M_{at1py}) at the exterior span along the y-y direction

$$M_{at1py} = D_{py} M_{t1y} = 0.5992 \times 158.625 = 95.05 \text{ kNm}$$

If $M_{at1py} < M_{cr}$ then

$$M_{at1py} = M_{cr}$$

The negative moment due to total load (M_{at1ney}) at the exterior span along the y-y direction

$$M_{at1ney} = D_{ney} M_{t1y} = 0.0074 \times 158.625 = 11.327 \text{ kNm}$$

If $M_{at1ney} < M_{cr}$ then

$$M_{at1ney} = M_{cr}$$

The negative moment due to total load (M_{at1niy}) at the interior span along the y-y direction

$$M_{at1niy} = D_{niy} M_{t1y} = 0.739 \text{ x } 158.625 = 117.22 \text{ kNm}$$

If $M_{at1niy} < M_{cr}$ then

$$M_{at1niy} = M_{cr}$$

The cracked moment of inertia of the column strip (I_{cry}) along the y-y direction

$$I_{cry} = b_{cy} \frac{x^3}{3} + (m)(A_{st1cy})(d-x)^2 + m \times A_s \times \left(0.5 \times D - \left(cc + \frac{dia}{2}\right)\right)^2 + (m-1) \times A_s \times \left(0.5 \times D - \left(cc + \frac{dia}{2}\right)\right)^2 = 2.4755 \times 10^8 \text{ mm}^4$$

The effective moment of inertia for the positive moment (I_{ecpy}) at the exterior span along the y-y direction

$$I_{ecpy} = \left(\frac{M_{cr}}{M_{at1py}}\right)^3 (I_{cy}) + \left(1 - \left(\frac{M_{cr}}{M_{at1py}}\right)^3\right)(I_{cry})$$

$$= \left(\frac{105.81}{105.81}\right)^3 \left(4.6829\times10^9\right) + \left(1-\left(\frac{105.81}{105.81}\right)^3\right)\left(2.4755\times10^8\right)$$

$$= 4.6829\times10^9 \text{ mm}^4$$

The effective moment of inertia for the negative moment (I_{ecney}) at the exterior span along the y-y direction

$$I_{ecney} = \left(\frac{M_{cr}}{M_{at1ney}}\right)^3 \left(I_{cy}\right) + \left(1-\left(\frac{M_{cr}}{M_{at1ney}}\right)^3\right)\left(I_{cry}\right)$$

$$= \left(\frac{105.81}{105.81}\right)^3 \left(4.6829\times10^9\right) + \left(1-\left(\frac{105.81}{105.81}\right)^3\right)\left(2.4755\times10^8\right)$$

$$= 4.6829\times10^9 \text{ mm}^4$$

The effective moment of inertia for the negative moment (I_{ecniy}) at the interior span along the y-y direction

$$I_{ecniy} = \left(\frac{M_{cr}}{M_{at1niy}}\right)^3 \left(I_{cy}\right) + \left(1-\left(\frac{M_{cr}}{M_{at1niy}}\right)^3\right)\left(I_{cry}\right)$$

$$= \left(\frac{105.81}{105.81}\right)^3 \left(4.6829\times10^9\right) + \left(1-\left(\frac{105.81}{105.81}\right)^3\right)\left(2.4755\times10^8\right)$$

$$= 4.6829\times10^9 \text{ mm}^4$$

The effective moment of inertia for the column strip (I_{ecy}) along the y-y direction

$$I_{ecy} = 0.7I_{ecpy} + 0.15I_{ecney} + 0.15I_{ecniy}$$

$$= 0.7 \text{ x } 4.6829 \text{ x } 10^9 + 0.15 \text{ x } 4.6829 \text{ x } 10^9 + 0.15 \text{ x } 4.6829 \text{ x } 10^9$$

$$= 4.6829\times10^9 \text{ mm}^4$$

The effective moment of inertia for the frame (I_{efy}) along the y-y direction

$$I_{efy} = I_{ecy} + I_{my} = 9.0769\times10^9 \text{ mm}^4$$

$$M_{oy} = \frac{W_t L_x \left(L_{ny}\right)^2}{8} = \frac{11.75\times6\times(5.7)^2}{8} = 286.318 \text{ kNm}$$

$$M_{ndy} = \left(W_d/W_t\right) M_{nt1y} = \left(8.75/11.75\right)\times2.0446\times10^7 = 1.5225\times10^7 \text{ Nmm}$$

The rotation due to total load (θ_{t1y}) along the y-y direction

$$\theta_{t1y} = \frac{M_{nt1y}}{K_{ecy}} = \frac{2.0446\times10^{7}}{K_{ecx}} = 6.1215\times10^{-4}$$

The rotation due to sustained load (θ_{d1y}) along the y-y direction

$$\theta_{d1y} = \frac{M_{nd1y}}{K_{ecy}} = \frac{1.5225\times10^{7}}{K_{ecx}} = 4.5586 \text{ x } 10^{-4}$$

The deflection due to total load (δ_{t1y}) along the y-y direction

$$\delta_{t1y} = \left(\theta_{t1y}\times L_y/8\right) = \left(6.1215\times10^{-4}\times6000/8\right) = 0.46 \text{ mm}$$

The deflection due to sustained load (δ_{d1y}) along the y-y direction

$$\delta_{d1y} = \frac{\theta_{d1y}\times L_y}{8} = \left(4.5586\times10^{-4}\times6000/8\right) = 0.34 \text{ mm}$$

Deflection of column strip due to total load along x-x direction

Interior panel

The deflection of the column strip (δ_{cxitl}) due to total load

$$\delta_{cxitl} = \frac{(LDF_{ci})\times W_t\times L_y\times L_x^4}{384\times E_c\times I_{cx}} = \frac{0.675\times11.75\times6\times6000^4}{384\times22361\times4.6829\times10^9} = 1.53 \text{ mm}$$

The deflection of the column strip (δ_{cxidl}) due to sustained load

$$\delta_{cxidl} = \frac{(LDF_{ci})\times W_d\times L_y\times L_x^4}{384\times E_c\times I_{cx}} = \frac{0.675\times8.75\times6\times6000^4}{384\times22361\times4.6829\times10^9} = 1.14 \text{ mm}$$

The deflection of the column strip (δ_{cxill}) due to varying part of live load

$$\delta_{cxill} = \delta_{cxitl} - \delta_{cxidl} = 1.53-1.14 = 0.39 \text{ mm}$$

Corner panel

x-x axis

The deflection of the column strip (δ_{cxctl}) due to total load

$$\delta_{cxctl} = \frac{(LDF_c)\times W_t\times L_y\times L_x^4}{384\times E_c\times I_{cx}} + (\delta_{t1x} - \delta_{d1x})$$

$$= \frac{0.55\times11.75\times6\times6000^4}{384\times22361\times4.6829\times10^9} + (0.46 - 0.34) = 1.37 \text{ mm}$$

The deflection of the column strip (δ_{cxcdl}) due to sustained load

$$\delta_{cxcdl} = \frac{(LDF_c) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} + (\delta_{t1x} - \delta_{d1x})$$

$$= \frac{0.55 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} + (0.46 - 0.34) = 1.05\text{mm}$$

The deflection of the column strip (δ_{cxcll}) due to varying part of live load

$$\delta_{cxcll} = \delta_{cxctl} - \delta_{cxcdl} = 1.37\text{-}1.05 = 0.32 \text{ mm}$$

Side panel

x-x axis

The deflection of the column strip (δ_{cxstl}) due to total load

$$\delta_{cxstl} = \frac{(LDF_c) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} = \frac{0.55 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 1.25 \text{ mm}$$

The deflection of the column strip (δ_{cxsdl}) due to sustained load

$$\delta_{cxsdl} = \frac{(LDF_c) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{cx}} = \frac{0.55 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 0.93 \text{ mm}$$

The deflection of the column strip (δ_{cxsll}) due to varying part of live load

$$\delta_{cxsll} = \delta_{cxstl} - \delta_{cxsdl} = 1.25\text{-} 0.93 = 0.32 \text{ mm}$$

Deflection of column strip

Interior panel

y-y axis

The deflection of the column strip δ_{cyitl} due to total load

$$\delta_{cyitl} = \frac{(LDF_{ci}) \times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} = \frac{0.675 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 1.53 \text{ mm}$$

The deflection of the column strip δ_{cyidl} due to sustained load

$$\delta_{cyidl} = \frac{(LDF_{ci}) \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} = \frac{0.675 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 1.14 \text{ mm}$$

The deflection of the column strip δ_{cyill} due to varying part of live load

$$\delta_{cyill} = \delta_{cyitl} - \delta_{cyidl} = 1.53\text{-}1.14 = 0.39 \text{ mm}$$

Corner panel

y-y axis

The deflection of the column strip δ_{cyctl} due to total load

$$\delta_{cyctl} = \frac{(LDF_c)\times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} + \left(\delta_{t1y} - \delta_{d1y}\right)$$

$$= \frac{0.55\times 11.75\times 6\times 6000^4}{384\times 22361\times 4.6829\times 10^9} + (0.46 - 0.34) = 1.37 \text{ mm}$$

The deflection of the column strip δ_{cycdl} due to sustained load

$$\delta_{cycdl} = \frac{(LDF_c)\times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} + \left(\delta_{t1y} - \delta_{d1y}\right)$$

$$= \frac{0.55\times 8.75\times 6\times 6000^4}{384\times 22361\times 4.6829\times 10^9} + (0.46 - 0.34) = 1.05\text{mm}$$

The deflection of the column strip δ_{cycll} due to varying part of live load

$$\delta_{cycll} = \delta_{cyctl} - \delta_{cycdl} = 1.37\text{-}1.05 = 0.32 \text{ mm}$$

Side panel

y-y axis

The deflection of the column strip δ_{cystl} due to total load

$$\delta_{cystl} = \frac{(LDF_c)\times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} + \left(\delta_{t1y} - \delta_{d1y}\right)$$

$$= \frac{0.55\times 11.75\times 6\times 6000^4}{384\times 22361\times 4.6829\times 10^9} + (0.46\text{-}0.34) = 1.37 \text{ mm}$$

The deflection of the column strip δ_{cysdl} due to sustained load

$$\delta_{cysdl} = \frac{(LDF_c) \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{cy}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.55 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} + (0.46 - 0.34) = 1.05\text{mm}$$

The deflection of the column strip δ_{cysll} due to varying part of live load

$$\delta_{cysll} = \delta_{cystl} - \delta_{cysdl} = 1.37 - 1.05 = 0.32 \text{ mm}$$

Deflection of middle strip

Interior panel

x-x axis

The deflection of the middle strip δ_{mxitl} due to total load

$$\delta_{mxitl} = \frac{(LDF_{mi}) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} = \frac{0.325 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 0.79 \text{ mm}$$

The deflection of the middle strip δ_{mxidl} due to sustained load

$$\delta_{mxidl} = \frac{(LDF_{mi}) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} = \frac{0.325 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 0.59 \text{ mm}$$

The deflection of the middle strip δ_{mxill} due to varying part of live load

$$\delta_{mxill} = \delta_{mxitl} - \delta_{mxidl} = 0.79 - 0.59 = 0.20 \text{ mm}$$

Corner panel

x-x axis

The deflection of the middle strip δ_{mxctl} due to total load

$$\delta_{mxctl} = \frac{(LDF_m) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} + (\delta_{t1x} - \delta_{d1x})$$

$$= \frac{0.45 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} + (0.46 - 0.34) = 1.20 \text{ mm}$$

The deflection of the middle strip δ_{mxcdl} due to sustained load

$$\delta_{mxcdl} = \frac{(LDF_m) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} + (\delta_{t1x} - \delta_{d1x})$$

$$= \frac{0.45 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} + (0.46 - 0.34) = 0.93$$

The deflection of the middle strip δ_{mxcll} due to varying part of live load

$$\delta_{mxcll} = \delta_{mxctl} - \delta_{mxcdl} = 1.20 - 0.93 = 0.27 \text{ mm}$$

Side panel

x-x axis

The deflection of the middle strip δ_{mxstl} due to total load

$$\delta_{mxstl} = \frac{(LDF_m) \times W_t \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} = \frac{0.45 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 1.09 \text{ mm}$$

The deflection of the middle strip δ_{mxsdl} due to sustained load

$$\delta_{mxsdl} = \frac{(LDF_m) \times W_d \times L_y \times L_x^4}{384 \times E_c \times I_{mx}} = \frac{0.45 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 0.81 \text{ mm}$$

The deflection of the middle strip δ_{mxsll} due to varying part of live load

$$\delta_{mxsll} = \delta_{mxstl} - \delta_{mxsdl} = 1.09 - 0.81 = 0.28 \text{ mm}$$

Deflection of middle strip

Interior panel

y-y axis

The deflection of the middle strip δ_{myitl} due to total load

$$\delta_{myitl} = \frac{(LDF_{mi}) \times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{my}} = \frac{0.325 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 0.79 \text{ mm}$$

The deflection of the middle strip δ_{myidl} due to sustained load

$$\delta_{myidl} = \frac{(LDF_{mi}) \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{my}} = \frac{0.325 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 0.59 \text{ mm}$$

The deflection of the middle strip δ_{myill} due to varying part of live load

$$\delta_{myill} = \delta_{myitl} - \delta_{myidl} = 0.79 - 0.59 = 0.20 \text{ mm}$$

Corner panel

y-y axis

The deflection of the middle strip δ_{myctl} due to total load

$$\delta_{myctl} = \frac{(LDF_m) \times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{my}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.45 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} + (0.46 - 0.34) = 1.20 \text{ mm}$$

The deflection of the middle strip δ_{mycdl} due to sustained load

$$\delta_{mycdl} = \frac{(LDF_m) \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{my}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.45 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} + (0.46 - 0.34) = 0.93 \text{ mm}$$

The deflection of the middle strip δ_{mycll} due to varying part of live load

$$\delta_{mycll} = \delta_{myctl} - \delta_{mycdl} = 1.20 - 0.93 = 0.27 \text{ mm}$$

Side panel

y-y axis

The deflection of the middle strip δ_{mystl} due to total load

$$\delta_{mystl} = \frac{(LDF_m) \times W_t \times L_x \times L_y^4}{384 \times E_c \times I_{my}} + (\delta_{t1y} - \delta_{d1y})$$

$$= \frac{0.45 \times 11.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 1.09 \text{ mm}$$

The deflection of the middle strip δ_{mysdl} due to sustained load

$$\delta_{mysdl} = \frac{(LDF_m) \times W_d \times L_x \times L_y^4}{384 \times E_c \times I_{my}} + (\delta_{tly} - \delta_{dly})$$

$$= \frac{0.45 \times 8.75 \times 6 \times 6000^4}{384 \times 22361 \times 4.6829 \times 10^9} = 0.81 \text{ mm}$$

The deflection of the middle strip δ_{mysll} due to varying part of live load

$$\delta_{mysll} = \delta_{mystl} - \delta_{mysdl} = 1.09 - 0.81 = 0.28 \text{ mm}$$

Total deflection

Interior panel

The deflection δ_{idl} due to sustained load

$$\delta_{idl} = \left(\frac{\delta_{cxidl} + \delta_{myidl} + \delta_{cyidl} + \delta_{mxidl}}{2} \right)$$

The deflection δ_i of interior panel

$$\delta_i = 6 \times \delta_{idl} + \delta_{ill}$$

$$\delta_i = 6 \times 1.73 + 0.59 = 10.96 \text{ mm}$$

Corner panel

The deflection δ_{cdl} due to sustained load

$$\delta_{cdl} = \left(\frac{\delta_{cxcdl} + \delta_{mycdl} + \delta_{cycdl} + \delta_{mxcdl}}{2} \right)$$

The deflection δ_c of corner panel

$$\delta_c = 6 \times \delta_{cdl} + \delta_{cll}$$

$$\delta_c = 6 \times 1.98 + 0.6 = 12.16 \text{ mm}$$

Side panel

The deflection δ_{sdl} due to sustained load

$$\delta_{sdl} = \left(\frac{\delta_{cxsdl} + \delta_{mysdl} + \delta_{cysdl} + \delta_{mxsdl}}{2} \right)$$

The deflection δ_s of side panel

$$\delta_s = 6 \times \delta_{sdl} + \delta_{sll}$$

$$\delta_s = 6 \times 1.86 + 0.6 = 11.75 \text{ mm}$$

6.2.3. Example 3 : Analysis of Frame using Equivalent Frame Method (Hoffman et al. [1998]) for Moment and Shear (Without Concealed Beam)

Step I Dimensions of the plate

Live load (W_l) = 3.0 kN/m^2

Floor finish (FF) = 1.5 kN/m^2

Clear cover to tension reinforcement = 25 mm

Grade of concrete = $f_{ck} = 20$ N/mm^2

Young's modulus of steel (E_s) = 200000 N/mm^2

Diameter of rod = 10 mm

Square column of size = 300 mm x 300 mm

Total depth of slab = 260 mm

Edge column size, C_1=300 mm & C_2=300 mm

Step II Column stiffness, K_c (Fig. 6.8)

The Young's modulus E_c of concrete

$$E_c = 5000 \times \sqrt{f_{ck}} = 22361 \text{ N/mm}^2$$

The moment of inertia I_c of the column

$$I_c = \frac{C2 \times C1^3}{12} = \frac{300 \times 300^3}{12} = 675000000 \text{ mm}^4$$

The stiffness of the column

$$\frac{K_c}{E_c} = \frac{I_c}{(L_c - D)}\left[1 + \frac{3 \times L_c^{\,2}}{(L_c - D)^2}\right] = \frac{675 \times 10^6}{(3500 - 260)}\left[1 + \frac{3 \times 3500^2}{(3500 - 260)^2}\right]$$

$$= 9.3767\text{x}10^5 \text{ mm}^3$$

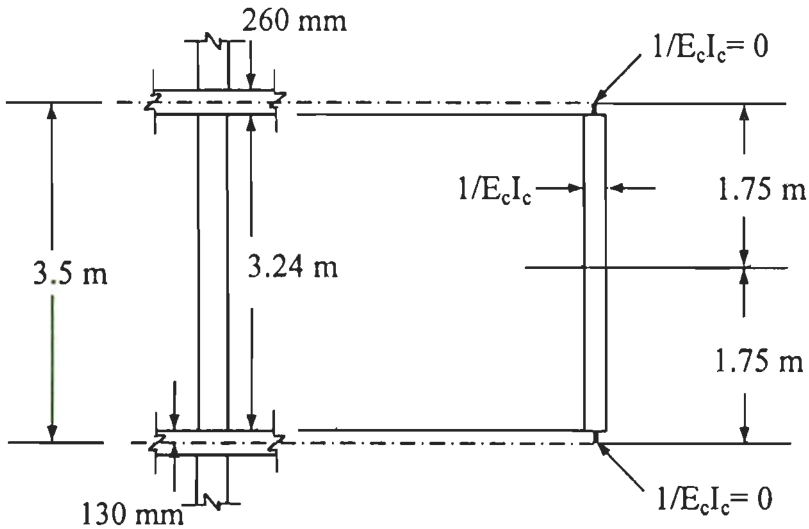


Fig. 6.8 Column stiffness by column analogy (without concealed beam)

Step III Slab stiffness, K_s (Fig. 6.9)

Slab between the (I_{g1}) columns

$$I_{g1} = \frac{L_1 \times D^3}{12} = \frac{6000 \times 260^3}{12} = 8.7880 \text{x} 10^9 \text{ mm}^4$$

Slab (I_{g2}) at the column

$$I_{g2} = \frac{I_{g1}}{\left(1-\left(\frac{C_2}{L_2}\right)\right)^2} = \frac{8.7880 \times 10^9}{\left(1-\left(\frac{300}{6000}\right)\right)^2} = 9.7374 \text{x} 10^9 \text{ mm}^4$$

$$I_1 = \frac{L1^3}{12 \times I_{g2}} = \frac{6000^3}{12 \times 9.3774 \times 10^9} = 1.8485 \text{ m}^4$$

$$I_2 = \frac{\left(\frac{1}{I_{g1}} - \frac{1}{I_{g2}}\right) \times (L_1 - C_1)^3}{12} = \frac{\left(\frac{1}{8.788 \times 10^9} - \frac{1}{9.7374 \times 10^9}\right) \times (6000-300)^3}{12}$$

$$= 0.1712 \text{ m}^4$$

Moment of inertia of the analogous column (I_{ac}) about the neutral axis

$$I_{ac} = I_1 + I_2 = 1.8485 + 0.1712 = 2.0198 \text{ m}^4$$

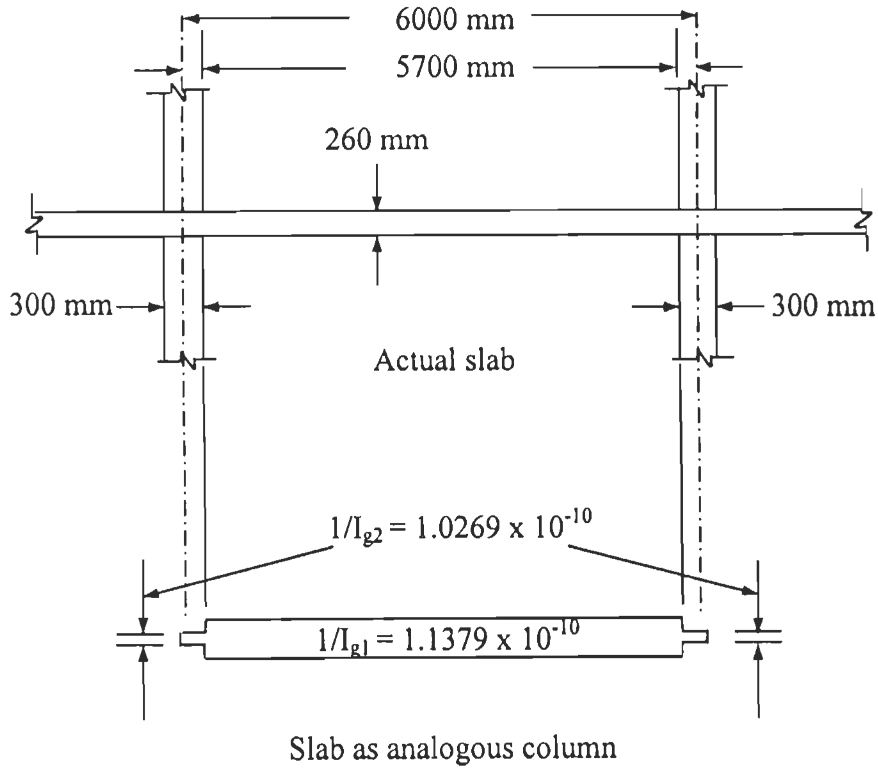


Fig. 6.9 Slab stiffness by column analogy (without concealed beam)

Area of the analogous (A_{ac}) column

Slab between (A_{ac1}) the columns

$$A_{ac1} = \frac{(L_1 - C1)}{I_{g1}} = \frac{(6000-300)}{8.788\times10^9} = 6.4861\text{x}10^{-7}\text{mm}^2$$

Slab (A_{ac2}) at the columns

$$A_{ac2} = \frac{2\times\left(\dfrac{C_1}{2}\right)}{I_{g2}} = \frac{2\times\left(\dfrac{300}{2}\right)}{9.7374\times10^9} = 3.0809\text{x}10^{-8}\ \text{mm}^2$$

$$A_{ac} = A_{ac1} + A_{ac2} = 6.4861\times10^{-7} + 3.0809\times10^{-8} = 6.7942\text{x}10^{-7}\ \text{mm}^2$$

The relative stiffness (K_s) of the slab

$$\frac{K_s}{E_s} = \left(\frac{1}{A_{ac}}\right) + \frac{(L_1/2)^2}{I_{ac}} = \left(\frac{1}{6.7942\times10^{-7}}\right) + \frac{(6000/2)^2}{2.0198\times1000^4} = 5.9278\text{x}10^6\ \text{mm}^3$$

Step IV Stiffness of exterior equivalent column

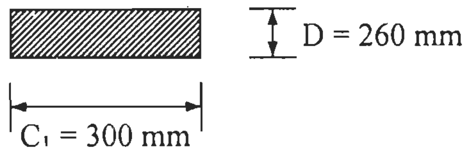


Fig. 6.10 Cross section at slab column connection

The torsional constant (C) of the equivalent column (Fig. 6.10)

$$C = \left(\left(1 - 0.63 \times \left(\frac{D}{C_1}\right)\right)\left(\frac{D^3 \times C_1}{3}\right)\right)$$

$$= \left(\left(1 - 0.63 \times \left(\frac{260}{300}\right)\right)\left(\frac{260^3 \times 300}{3}\right)\right) = 7.9795\text{x}10^8$$

The torsional stiffness (K_t) of the column

$$\frac{K_t}{E_s} = \left(\frac{(2 \times 4.5) \times C}{L_2 \times \left(1 - \frac{C_2}{L_2}\right)^3}\right) = \left(\frac{(2 \times 4.5) \times 7.9795 \times 10^8}{6000 \times \left(1 - \frac{300}{6000}\right)^3}\right) = 1.3960\text{x}10^6$$

The relative stiffness (K_{ec}) of the equivalent column

$$\frac{K_{ec}}{E_c} = \frac{\Sigma K_c}{1 + \left(\frac{\Sigma K_c}{K_t}\right)} = \frac{2 \times 9.3767 \times 10^5}{1 + \left(\frac{2 \times 9.3767 \times 10^5}{1.396 \times 10^6}\right)} = 8.0029\text{x}10^5 \text{ mm}^3$$

Step V Moment distribution factors

Exterior column

$$\text{DF} = \frac{K_s}{K_s + K_{ec}} = \frac{5.9278 \times 10^6}{5.9278 \times 10^6 + 8.0029 \times 10^5} = 0.88$$

Interior column

$$\text{DF} = \frac{K_s}{2 \times K_s + K_{ec}} = \frac{5.9278 \times 10^6}{2 \times 5.9278 \times 10^6 + 8.0029 \times 10^5} = 0.46$$

Step VI Factored loads for moment distribution

The dead load (W_d)

$$W_d = 1.5 \times ((0.26 \times 25) + 1.5) = 12 \text{ kN/m}^2$$

Live load (W_l)

$$Wl = 1.5 \times 3 = 4.5 \text{ kN/m}^2$$

Total load (W_t)

$$Wt = 12 + 4.5 = 16.50 \text{ kN/m}^2$$

Uniformly distributed load (w)

$$w = 99 \text{ kN/m}$$

Step VII Carry over factors for both interior as well as exterior columns (Fig. 11)

$$COF = \left(\frac{\left(\frac{1}{A_{ac}}\right) + \frac{(L_1/2)^2}{I_{ac}}}{\left(\frac{1}{A_{ac}}\right) - \frac{(L_1/2)^2}{I_{ac}}} \right) = \left(\frac{\left(\frac{1}{6.7942 \times 10^{-7}}\right) + \frac{(6000/2)^2}{2.0198}}{\left(\frac{1}{6.7942 \times 10^{-7}}\right) - \frac{(6000/2)^2}{2.0198}} \right) = -0.5034$$

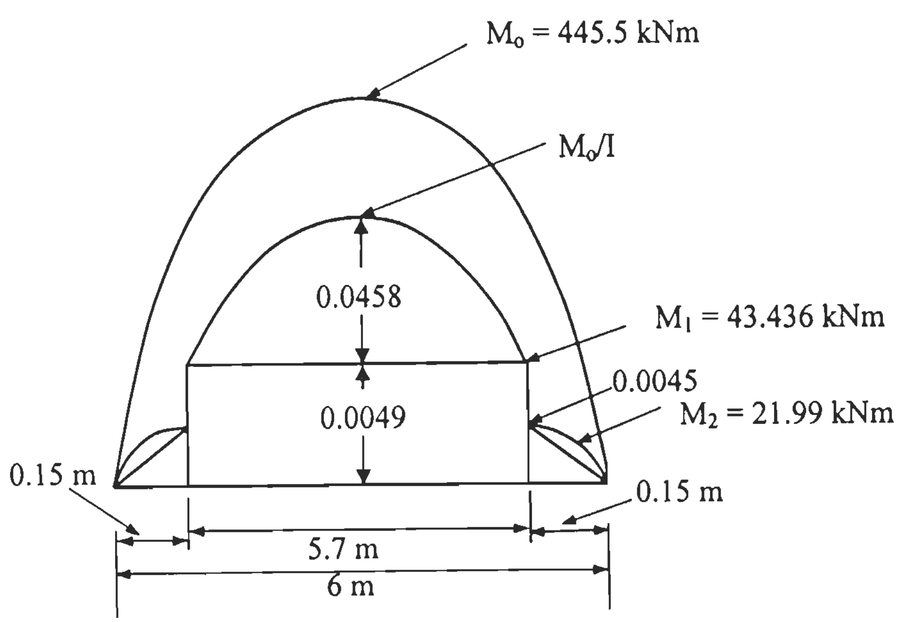
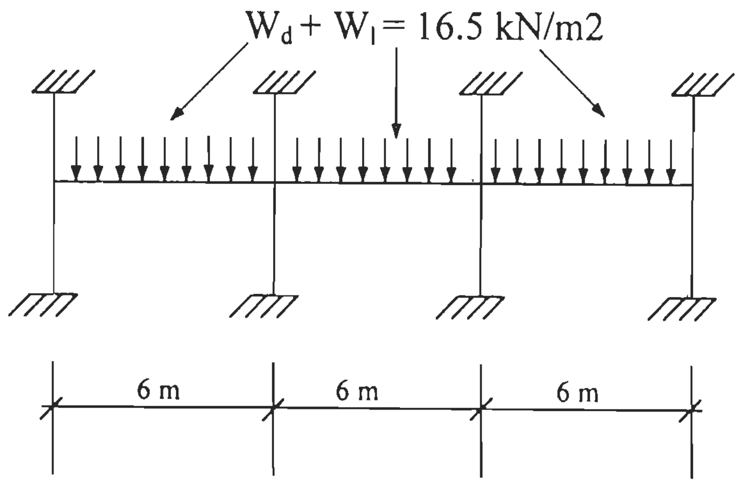


Fig. 6.11 Slab fixed end moments by column analogy (without concealed beam)

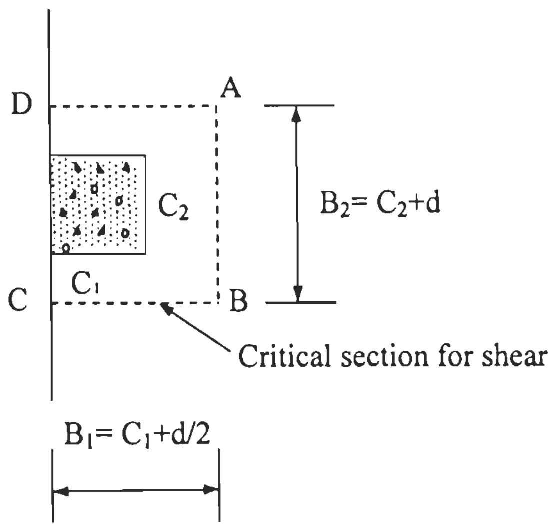
Step VII

Moment distribution analysis

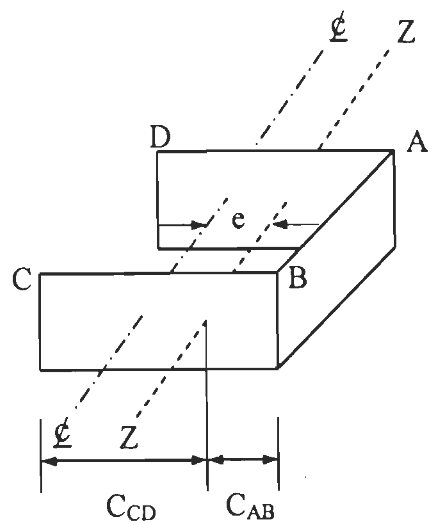


COF	-0.5034		-0.5034		-0.5034	
DF	0.88	0.46	0.46	0.46	0.46	0.88
FEM	-298.34	-298.34	-298.34	-298.34	-298.34	-298.34
D	+262.85	—	—	—	—	+262.85
CO	—	-132.32	—	—	-132.32	—
D	—	+61.98	-61.98	-61.98	+61.98	—
CO	-31.20	—	+31.20	+31.20	—	-31.20
D	+27.49	+14.61	-14.61	-14.61	+14.61	+27.49
CO	-13.83	-7.36	+7.36	+7.36	-7.36	-13.83
D	+12.19	—	—	—	—	+12.19
M_u	-40.84	-361.43	+336.37	-336.37	-361.43	-40.84
	297.00	297.00	297.00	297.00	297.00	297.00
	-53.43	53.43	-	-	53.43	-53.43
V_u	243.57	350.43	297.00	297.00	350.43	243.57

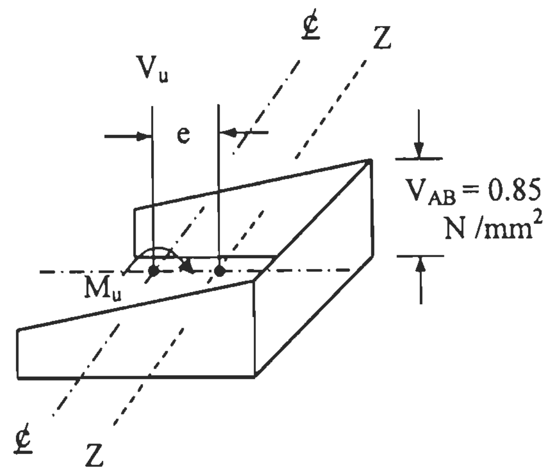
Step VIII. Shear at the exterior trial column (Fig. 6.12)



a. Plan



b. Shear section dimensions



c. V_u, M_u, V_{AB}

Fig. 6.12 Shear at exterior column (without concealed beam)

The critical section for the shear

$$B1 = C_1 + d/2 = 300+230/2 = 415 \text{ mm}$$

$$B2 = C_2 + d = 300+230 = 530 \text{ mm}$$

The shear (V_{cs}) is equal to the exterior column reaction

$$V_{cs} = 243.57 + 99 \times 300/2 = 258.42 \text{ kN}$$

The area (A_{s1}) of the critical shear section

$$A_{s1} = 2B1 \times d + B2 \times d = 2 \times 415 \times 230 + 530 \times 230 = 312800 \text{ mm}^2$$

$$C_{AB} = \left(\frac{2B1 \times d + B2 \times d}{A_{s1}}\right) = \left(\frac{2 \times 415 \times 230 + 530 \times 230}{312800}\right) = 126.6360$$

$$C_{CD} = B1 - C_{AB} = 415 - 126.636 = 288.36$$

Distance between the column centreline (e) and the centroid of the critical section

$$e = C_{CD} - C_2/2 = 288.36 - 300/2 = 138.36 \text{ mm}$$

$$J1 = \left(\frac{B1 \times d^3}{6}\right) + \left(\frac{d \times B1^3}{6}\right) + 2 \times d \times B1 \times \left(B1/2 - C_{AB}\right)^2 + d \times B2 \times C_{AB}{}^2$$

$$= \left(\frac{415 \times 230^3}{6}\right) + \left(\frac{230 \times 415^3}{6}\right) + 2 \times 230 \times 415 \times \left(\frac{415}{2} - 126.636\right)^2$$
$$+ 230 \times 530 \times 126.636^2$$

$$= 6.7845 \text{x} 10^9$$

$$J_c = J1$$

The fraction (γ_v) of the unbalanced moment to be transferred to the column by shear about the centroid of the shear area

$$\gamma_v = 1 - \left(\frac{1}{1 + \left(2\sqrt{B1/B2}/3\right)}\right) = 1 - \left(\frac{1}{1 + \left(2\sqrt{415/530}/3\right)}\right) = 0.371$$

$$M_{cs} = M_{u1} - w \times \left(C_1/2\right) \times \left(C_1/4\right) - V_{cs} \times e$$

$$= 40.84 \times 10^6 - 99 \times \left(300/2\right) \times \left(300/4\right) - 258.42 \times 10^3 \times 138.36 = 3.9741$$

The shear stresses (V_{uab}) at the exterior column

$$V_{uab} = \left(V_{cs}/A_{s1}\right) + \left(\gamma_v \times M_{cs} \times C_{AB}/J_c\right) =$$

$$V_{uab} = \left(258.42\times10^3/312800\right) + \left(0.3710\times3.9741\times10^6\times126.636/6.7845\times10^9\right)$$

$$= 0.8537$$

The permissible shear stress (V_s) without shear stirrups

$$V_s = 0.25\times\sqrt{f_{ck}}$$

$$= 0.25\times\sqrt{20} = 1.12 \text{ N/mm}^2$$

$V_{uab} < V_s$ Hence safe

$$V_{ucd} = \left(V_{cs}/A_{s1}\right) - \left(\gamma_v \times M_{cs} \times C_{CD}/J_c\right) =$$

$$= \left(258.42\times10^3/312800\right) - \left(0.3710\times3.9741\times10^6\times288.36/6.7845\times10^9\right)$$

$$= 0.7635 \text{ N/mm}^2$$

$V_{ucd} < V_s$ Hence safe

Step IX

Shear at interior column (Fig. 6.13)

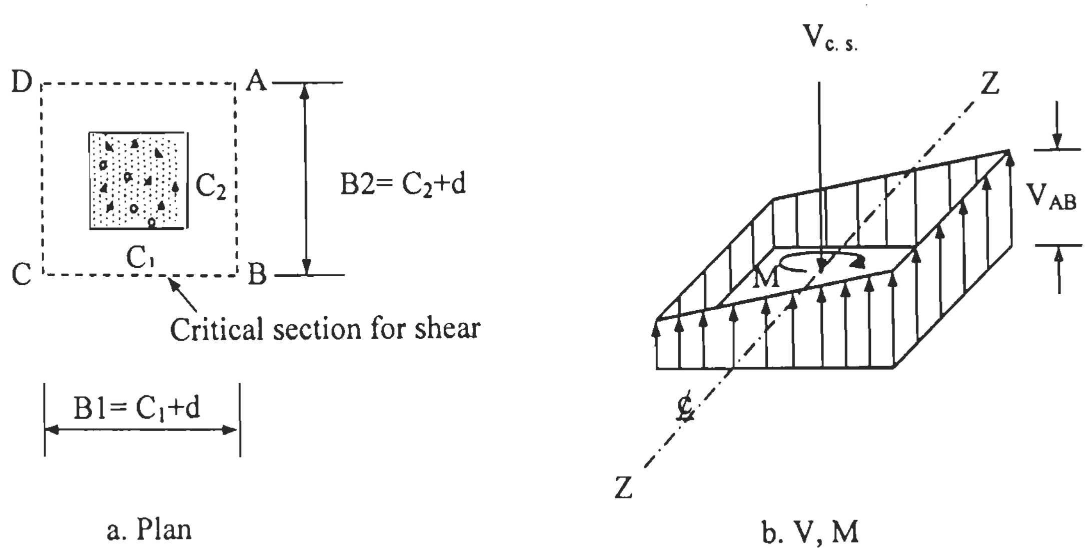


Fig. 6.13 Shear at interior column (without concealed beam)

$V_{csm} = 350.43 + 290.00 = 640.43$ kN

$M_v = -25.054$ kNm

$$J_{c1} = \left(d \times (C1+d)^3 / 6 \right) + \left((C1+d) \times d^3 / 6 \right) + \left((C2+d) \times d \times (C1+d)^2 / 2 \right)$$

$$= \left(\frac{230 \times (300+230)^3}{6} \right) + \left(\frac{(300+230) \times 230^3}{6} \right) + \left(\frac{(300+230) \times 230 \times (300+230)^2}{2} \right)$$

$$= 2.3903 \text{x} 10^{10}$$

The critical (A_{c1}) perimeter

$$A_{c1} = 4 \times B2 \times d \quad = \quad 4 \times 530 \times 230 = 487600 \text{ mm}^2$$

$$V_{u1} = \left(V_{csm} / A_{c1} \right) + \left(\gamma_v \times M_v \times B2 / 2 \times J_{c1} \right)$$

$$= \left(640.43 \times 10^3 / 487600 \right) + \left(0.371 \times 25.054 \times 10^6 \times 530 / 2 \times 2.3903 \times 10^{10} \right)$$

$$= 1.42 \text{ N/mm}^2$$

$V_{u1} < V_s$ Hence safe

Step X

The maximum beam shear (V_{u2}) for total factored load

$$V_{u2} = \frac{\left(V_{csm} / 2 - w \times d \right)}{L1 \times d}$$

$$= \frac{\left(640.43 \times 10^3 / 2 - 99 \times 230 \right)}{6000 \times 230} = 0.2181 \text{ N/mm}^2$$

The permissible shear stress (V_{s2}) without shear stress as per IS: 456-2000

$$V_{s1} = 0.25 \times \sqrt{f_{ck}} = 0.25 \times \sqrt{20} = 1.12 \text{ N/mm}^2$$

$V_{u2} < V_{s1}$ Hence safe

6.2.4 Design Example 4: Analysis of Frame Using Equivalent Frame Method for the Moment and Shear (With Concealed Beam)

Step I

Live load (W_l) = 3.0 kN/mm^2

Floor finish (FF) = 1.5 kN/mm^2

Clear cover to tension reinforcement = 25 mm

Grade of concrete = $f_{ck} = 20$ N/mm^2

Young's modulus of steel (E_s) = 200000 N/mm^2

Diameter of rod = 10 mm

Square column of size = 300 mm x 300 mm

Total depth of slab = 260 mm

Step II

Column stiffness, K_c (Fig. 6.14)

$$E_c = 5000 \times \sqrt{f_{ck}} = 22361 \text{ N/mm}^2$$

The moment of inertia (I_c) of the column

$$I_c = \frac{C2 \times C1^3}{12} = 675000000 \text{ mm}^4$$

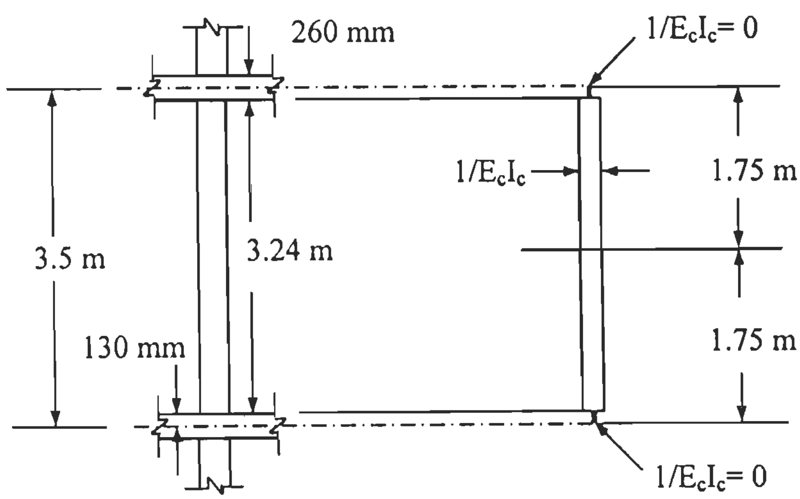


Fig. 6.14 Column stiffness by column analogy (with concealed beam)

$$\frac{K_c}{E_c} = \frac{I_c}{(L_c - D)}\left[1 + \frac{3 \times L_c^{\,2}}{(L_c - D)^2}\right] = \frac{675 \times 10^6}{(3500 - 260)}\left[1 + \frac{3 \times 3500^2}{(3500 - 260)^2}\right]$$

$= 9.3767 \text{x} 10^5 \text{ mm}^3$

Step III

Slab stiffness, K_s (Fig. 6.15)

d = 230 mm

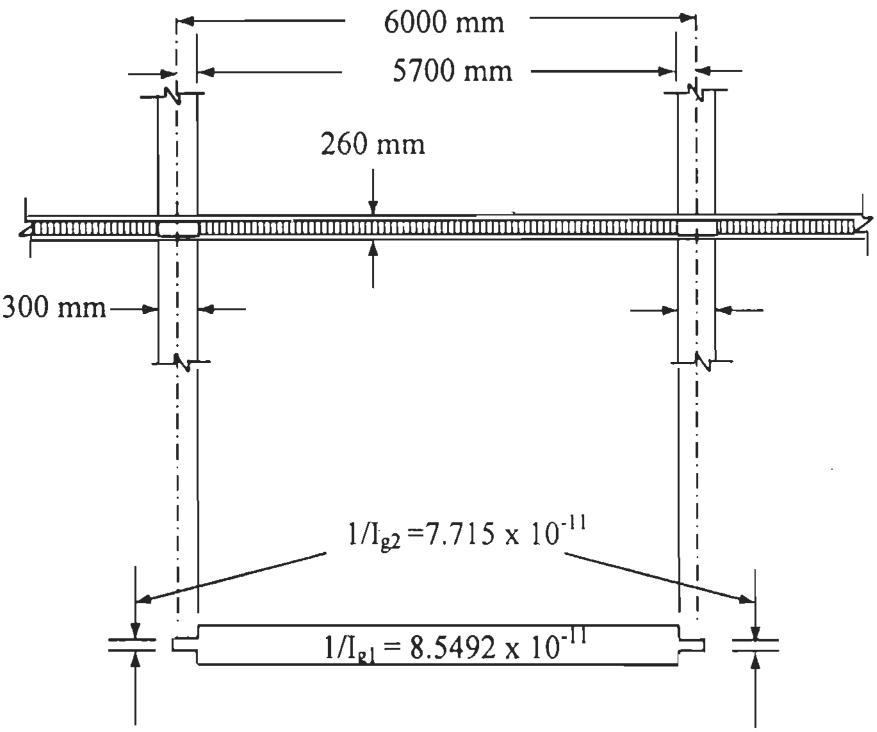


Fig. 6.15 Slab stiffness by column analogy (with concealed beam)

Slab between columns

Concealed beam

The width (l_1) of the concealed beam

l_1 = 1000 mm

The number of the longitudinal reinforcement (n) to be provided in the concealed beam

$$n = \left(\frac{l_1}{50}\right) + 1 = \left(\frac{1000}{50}\right) + 1 = 21 \text{ Nos.}$$

The equivalent concrete area (A_c) of the concealed beam

$$A_s = \left(\frac{m \times n \times \pi \times d_c^2}{4} \right)$$

$$= \left(\frac{8.394 \times 21 \times \pi \times 10^2}{4} \right) = 1.4752 \text{ x } 10^4 \text{ mm}^2$$

The depth of the concealed beam (d_1) in the equivalent concrete area (Fig. 6.16)

$$d_1 = \left(\frac{A_s}{l_1} \right) = \left(\frac{1.4752 \times 10^4}{1000} \right) = 14.8 \text{ mm}$$

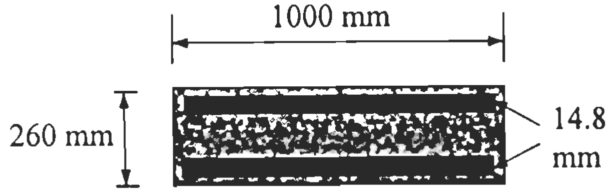


Fig. 6.16 Transformed section for flexural stiffness

The transformed moment of inertia (I_{gc}) of the concealed beam

$$I_{gc} = \frac{2 \times \left(L1 \times (2 \times d_1)^3 \right)}{12} + \left(L1 \times 2 \times d_1 \times \left(D/2 - (cc + d_1) \right)^2 \right)$$

$$= \frac{2 \times \left(6000 \times (2 \times 14.8)^3\right)}{12} + \left(6000 \times 2 \times 14.8 \times \left(260/2 - (25 + 14.8)\right)^2 \right)$$

$$= 2.9093 \text{x} 10^9 \text{ mm}^4$$

$$I_{g1} = \frac{\left(L_1 \times D^3\right)}{12} + I_{gc} = \frac{\left(6000 \times 260^3\right)}{12} + 2.9093 \times 10^9 = 1.1697 \text{x} 10^{10} \text{ mm}^4$$

Slab (I_{g2}) at the column

$$I_{g2} = \frac{I_{g1}}{\left(1 - \left(\frac{C_2}{L_2}\right)\right)^2} = \frac{1.1697 \times 10^{10}}{\left(1 - \left(\frac{300}{6000}\right)\right)^2} = 1.2961 \text{ x } 10^{10} \text{ mm}^4$$

$$I_1 = \frac{L_1^3}{12\times I_{g2}} = \frac{6000^3}{12\times 1.2961\times 10^{10}} = 1.3888\ \text{m}^4$$

$$I_2 = \frac{\left(\frac{1}{I_{g1}} - \frac{1}{I_{g2}}\right)\times (L_1 - C_1)^3}{12}$$

$$= \frac{\left(\frac{1}{1.1697\times 10^{10}} - \frac{1}{1.2961\times 10^{10}}\right)\times (6000-300)^3}{12} = 0.1286\ \text{m}^4$$

The moment of inertia (I_{ac}) of the analogous column

$$I_{ac} = I_1 + I_2 = 1.3888 + 0.1286 = 1.5174\ \text{m}^4$$

Area of (A_{ac}) the analogous column

Between the column

$$A_{ac1} = \frac{(L_1 - C_1)}{I_{g1}} = \frac{(6000-300)}{1.1697\times 10^{10}} = 4.8729\text{x}10^{-7}\ \text{mm}^2$$

At the column (A_{ac2})

$$A_{ac2} = \frac{2\times\left(\frac{C_1}{2}\right)}{I_{g2}} = \frac{2\times\left(\frac{300}{2}\right)}{1.2961\times 10^{10}} = 2.3146\text{x}10^{-8}\ \text{mm}^2$$

$$A_{ac} = A_{ac1} + A_{ac2} = 4.8729\times 10^{-7} + 2.3146\times 10^{-8} = 5.1044\text{x}10^{-7}\ \text{mm}^2$$

$$K_s = \left(\frac{1}{A_{ac}}\right) + \frac{(L_1/2)^2}{I_{ac}} = \left(\frac{1}{5.1044\times 10^{-7}}\right) + \frac{(6000/2)^2}{1.5174\times 1000^4} = 7.8902\text{x}10^6$$

Step IV Stiffness of exterior equivalent column

Concealed beam

The spacing of the stirrups (s) in the concealed beam

$$s = 50\,\text{mm}$$

The diameter of the stirrups (d_s) in the concealed beam

$$d_s = 8 \text{ mm}$$

The number of the stirrups (n_s)

$$n_s = \left(\frac{L_2 - C_2}{s}\right) + 1 = \left(\frac{6000 - 300}{50}\right) + 1 = 115 \text{ Nos.}$$

The length of the slab for the stirrups (L_s) to be provided

$$L_s = L_2 = 6000 \text{ mm}$$

The equivalent area of the concealed beam (E_a) in concrete

$$E_a = \frac{n_s \times m \times \left(\pi \times d_s^{\,2}\right)}{4} = \frac{115 \times 8.94 \times \left(\pi \times 8^2\right)}{4} = 5.1703 \text{ x } 10^4 \text{ mm}^2$$

The thickness of the concealed beam (t_y) in the equivalent concrete

$$t_y = \frac{E_a}{L_s} = \frac{5.1703 \times 10^4}{6000} = 8.62 \text{ mm}$$

$$X_1 = 1000 - t_y = 1000 - 8.62 = 991.38 \text{ mm}$$

$$Y_1 = D - t_y = 260 - 8.62 = \; 251.38\text{mm}$$

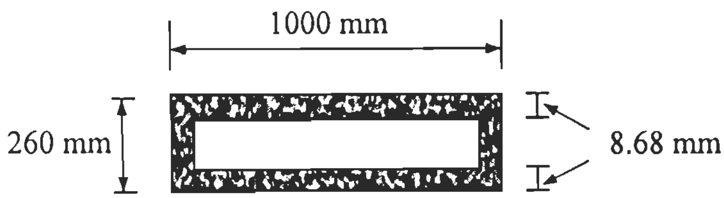


Fig. 6.17 Equivalent thin concrete box section for torsion

The torsional constant (C_b) of the concealed beam

$$C_b = \left(\frac{2 \times X_1^{\,2} \times 2 \times Y_1^2 \times D}{X_1 + Y_1}\right) = \left(\frac{2 \times 991.38^2 \times 2 \times 251.38^2 \times 260}{991.38 + 251.38}\right) = 2.5988\text{x}10^{10}$$

The torsional constant (C) of the column with concealed beam

$$C = \left(\left(1-0.63\times\left(\frac{D}{C_1}\right)\right)\left(\frac{D^3\times C_1}{3}\right)\right)+C_b$$

$$= \left(\left(1-0.63\times\left(\frac{260}{300}\right)\right)\left(\frac{260^3\times 300}{3}\right)\right)+2.5988\times10^{10} = 2.6786\text{x}10^{10}$$

The torsional stiffness (K_t) of the column

$$\frac{K_t}{E_s} = \left(\frac{(2\times4.5)\times C}{L_2\times\left(1-\frac{C_2}{L_2}\right)^3}\right) = \left(\frac{(2\times4.5)\times 2.6786\times10^{10}}{6000\times\left(1-\frac{300}{6000}\right)^3}\right) = 4.6862\text{x}10^7$$

The relative stiffness of the (K_{ec}) equivalent column

$$\frac{K_{ec}}{E_c} = \frac{\Sigma K_c}{1+\left(\frac{\Sigma K_c}{K_t}\right)} = \frac{2\times9.3767\times10^5}{1+\left(\frac{2\times9.3767\times10^5}{4.6862\times10^7}\right)} = 1.8032\text{x}10^6$$

Step V Moment distribution factors

Exterior column

$$\text{DF} = \frac{K_s}{K_s+K_{ec}} = \frac{7.8902\times10^6}{7.8902\times10^6+1.8032\times10^6} = 0.814$$

Interior column

$$\text{DF} = \frac{K_s}{2\times K_s+K_{ec}} = \frac{7.8902\times10^6}{2\times7.8902\times10^6+1.8032\times10^6} = 0.4487$$

Step VI Factored loads for moment distribution

The dead load (W_d)

$$W_d = 1.5\times((0.26\times25)+1.5) = 12 \text{ kN/m}^2$$

Live load (W_l)

$$W_l = 1.5\times3 = 4.5 \text{ kN/m}^2$$

The total load (W_t)

$$Wt = 12 + 4.5 = 16.50 \text{ kN/m}^2$$

Uniformly distributed load (w)

$$w = 99 \text{ kN/m}$$

Carry over factors (Fig. 6.18)

$$COF = \left(\frac{\left(\frac{1}{A_{ac}}\right) + \frac{(L_1/2)^2}{I_{ac}}}{\left(\frac{1}{A_{ac}}\right) - \frac{(L_1/2)^2}{I_{ac}}} \right) = \left(\frac{\left(\frac{1}{5.1044 \times 10^{-7}}\right) + \frac{(6000/2)^2}{1.5174 \times 1000^4}}{\left(\frac{1}{5.1044 \times 10^{-7}}\right) - \frac{(6000/2)^2}{1.5174 \times 1000^4}} \right) = -0.5034$$

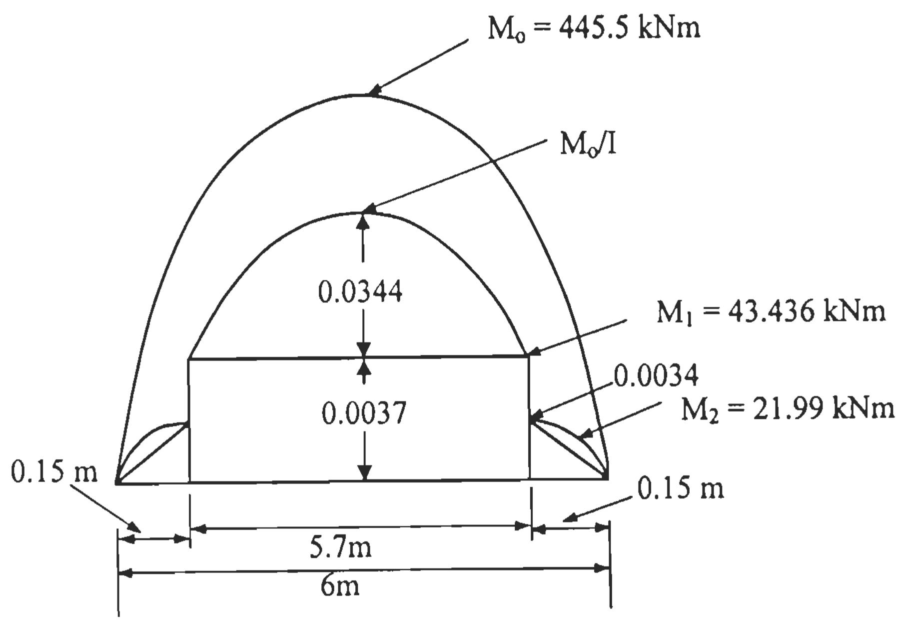
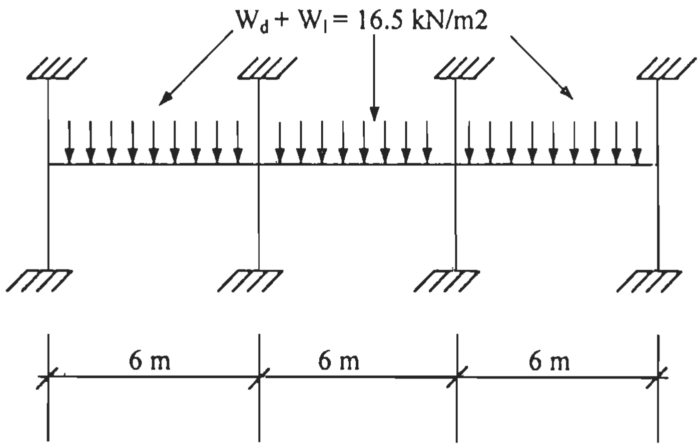


Fig. 6.18 Slab fixed end moments by column analogy (with concealed beam)

Step VII Moment distribution analysis



COF	-0.5034		-0.5034		-0.5034	
DF	-0.81	-0.45	-0.45	-0.45	-0.45	-0.81
FEM	-298.34	-298.34	-298.34	-298.34	-298.34	-298.34
D	+242.84	—	—	—	—	+242.84
CO	—	-122.25	—	—	-122.25	—
D	—	+54.86	-54.86	-54.86	+54.86	—
CO	-27.61	—	+27.61	+27.61	—	-27.61
D	+22.48	+12.39	-12.39	-12.39	+12.39	+22.48
CO	-11.31	-6.23	+6.23	+6.23	-6.23	-11.31
D	+9.21	—	—	—	—	+9.21
M_u	-62.74	-359.58	-331.73	-331.73	-359.58	-62.74
	297.00	297.00	297.00	297.00	297.00	297.00
	-49.47	49.47	-	-	49.47	-49.47
V_u	-247.53	-346.47	-297.00	-297.00	-346.47	-247.53

Step VIII Shear at the exterior trial column

$$B1 = C_1 + d/2 = 300+230/2 = 415 \text{ mm}$$

$$B2 = C_1 + d = 300+230 = 530 \text{ mm}$$

$$V_{cs} = 247.53 + 99 \times 300/2 = 262.38 \text{ kN}$$

Concealed beam

The area of the (A_c) concealed beam at one face (Fig. 6.19)

$$A_c = 2 \times l_1 \times d_1 = A_c = 2 \times 1000 \times 14.8 = 2.9504\text{x}10^4 \text{ mm}^2$$

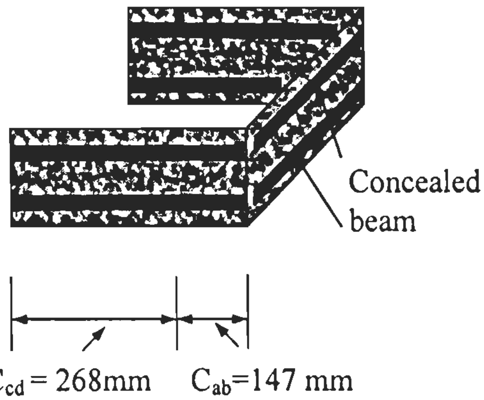


Fig. 6.19 Critical section for shear at exterior column (with concealed beam)

The actual area of the (A_{s1}) critical shear section

$$A_{s1} = 2B1 \times d + B2 \times d + A_c \times 3$$

$$= 2 \times 415 \times 230 + 530 \times 230 + 2.9054 \times 10^4 \times 3 = 3.5293\text{x}10^5 \text{ mm}^2$$

$$C_{ab} = \left(\frac{2B1 \times d \times {B1}/{2} + 2 \times A_c \times {l_1}/{2}}{A_{s1}} \right)$$

$$= \left(\frac{2 \times 415 \times 230 \times {415}/{2} + 2 \times 2.9504 \times 10^4 \times {1000}/{2}}{3.5293 \times 10^5} \right) = 147 \text{ mm}$$

$$C_{cd} = B1 - C_{ab} = 415 - 147 = 268 \text{ mm}$$

$$e = C_{cd} - {C_2}/{2} = 268 - {300}/{2} = 118 \text{ mm}$$

Shear coefficient for the moment transfer ($J1$) for concrete alone

$$J1 = \left(\frac{B1 \times d^3}{6} \right) + \left(\frac{d \times B1^3}{6} \right) + 2 \times d \times B1 \times \left({B1}/{2} - C_{ab} \right)^2 + d \times B2 \times C_{ab}{}^2$$

$$= \left(\frac{415\times530^3}{6}\right)+\left(\frac{230\times415^3}{6}\right)+2\times230\times415\times\left(415/2-147\right)^2$$

$$+\ 230\ \times\ 530\ \times\ 147^2$$

$= 6.9134\text{x}10^9\ \text{mm}^4$

Shear coefficient for the moment transfer ($J2$) for concealed beam

$$J2 = \left(\frac{2\times l_1\times d1^3}{6}\right)+\left(\frac{2\times d_1\times l_1^3}{6}\right)+2\times d_1\times l_1\times\left(l_1/2-C_{ab}\right)^2+d_1\times l_1\times C_{ab}^2$$

$$=\left(\frac{2\times1000\times14.8^3}{6}\right)+\left(\frac{2\times14.8\times1000^3}{6}\right)+2\times14.8\times1000\times\left(1000/2-147\right)^2$$

$$+\ 14.8\ \times\ 1000\times147^2$$

$= 5.6562\text{x}10^8\ \text{mm}^4$

Shear coefficient (J_c) for moment transfer

$$J_c = J_1 + J_2 = 6.9134\times10^9 + 5.6562\times10^8 = 7.479\text{x}10^9\ \text{mm}^4$$

$$\gamma_v = 1-\left(\frac{1}{1+\left(2\sqrt{B1/B2}/3\right)}\right)=1-\left(\frac{1}{1+\left(2\sqrt{415/530}/3\right)}\right)=0.6290$$

The moment at the centroid of the column section

$$M_{cs} = M_{u1}-w\times\left(C_1/2\right)\times\left(c1/4\right)-V_{cs}\times e$$

$$= 62.74\times10^6-99\times\left(300/2\right)\times\left(300/4\right)-262.38\times10^3\times118 = 30.64\ \text{kNm}$$

The shear stress (V_{uab}) at the exterior column face

$$V_{uab} = \left(V_{cs}/A_{s1}\right)+\left(\gamma_v\times M_{cs}\times C_{ab}/J_c\right)$$

$$=\left(\frac{262.38\times10^3}{3.5293\times10^5}\right)+\left(\frac{0.371\times30.64\times10^6\times147}{7.479\times10^9}\right) = 0.9668\ \text{N/mm}^2$$

The permissible (V_s) shear stress

$$V_s = 0.5\times\sqrt{f_{ck}} = 0.5\times\sqrt{20} = 2.4\ \text{N/mm}^2$$

$V_{uab} < V_s$ Hence safe

The shear stress (V_{ucd}) at the exterior column at the edge face

$$V_{ucd} = \left(V_{cs} / A_{s1} \right) - \left(\gamma_v \times M_{cs} \times C_{cd} / J_c \right)$$

$$= \left(\frac{262.38 \times 10^3}{3.5293 \times 10^5} \right) - \left(\frac{0.371 \times 30.64 \times 10^3 \times 268}{7.479 \times 10^9} \right) = 0.3359 \text{ N/mm}^2$$

$V_{ucd} < V_s$ Hence safe

Step IX Shear at interior trial column (Fig. 6.20)

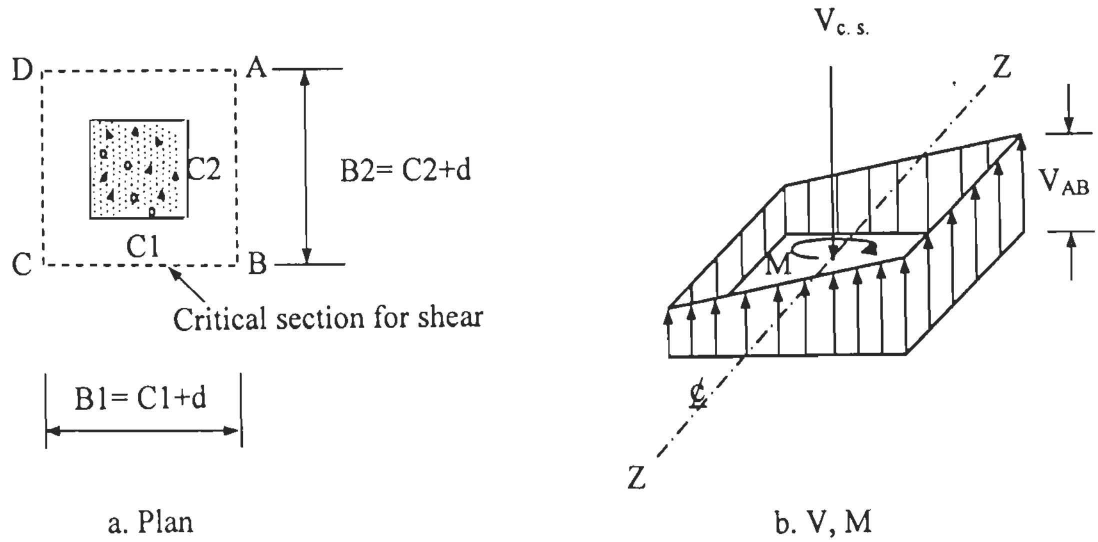


Fig. 6.20 Critical section for shear at interior column (with concealed beam)

$V_{csm} = 346.47 + 297.00 =$ 643.47 kN

$M_v =$ -27.84 kNm

Concealed beam

The critical shear section (A_{c1}) of the concealed beam

$$A_{c1} = (4 \times B2 \times d) + 4 \times A_c = (4 \times 530 \times 230) + 4 \times 2.9504 \times 10^4 = 5.5015\text{x}10^5 \text{ mm}^2$$

The shear coefficient (J_s) for moment transfer for the concealed beam

$$J_s = \left(2\times d_1\times(l_1)^3/6\right)+\left((l_1)^2\times d_1^3/6\right)+\left((l_1)\times d_1\times(l_1)^2/2\right)$$

$$= \left(\frac{2\times 14.8\times(1000)^3}{6}\right)+\left(\frac{(1000)^2\times 14.8^3}{6}\right)+\left(\frac{(1000)\times 14.8\times(1000)^2}{2}\right)$$

$= 1.8308\text{x}10^9\ \text{mm}^4$

The shear coefficient (J_{c1}) for moment transfer for concrete and the concealed beam

$$J_{c1} = \left(\frac{d\times(C1+d)^3}{6}\right)+\left(\frac{(C1+d)\times d^3}{6}\right)+\left(\frac{(C2+d)\times d\times(C1+d)^2}{2}\right)+J_s$$

$$= \left(\frac{230\times(300+230)^3}{6}\right)+\left(\frac{(300+230)\times 230^3}{6}\right)$$

$$+\left(\frac{(300+230)\times 230\times(300+230)^2}{2}\right)+1.8308\times 10^8$$

$= 2.5733\text{x}10^{10}\ \text{mm}^4$

The shear stress (V_{u1}) at the interior column

$$V_{u1} = \left(643.47\times 10^3/5.5015\times 10^5\right)+\left(0.371\times 27.84\times 10^3\times 530/2\times 2.5733\times 10^{10}\right)$$

$= 1.276\ \text{N/mm}^2$

$V_{u1} < V_s$ Hence safe

Step X

Beam shear (V_{u2})

$$V_{u2} = \frac{\left(V_{csm}/2 - w\times d\right)}{L1\times d} = \frac{\left(643.47\times 10^3/2 - 99\times 10^3\times 230\right)}{6000\times 230} = 0.2166\ \text{N/mm}^2$$

Permissible (V_{s1}) shear stress

$V_{s1} = 0.5\times\sqrt{f_{ck}} = V_{s1} = 0.5\times\sqrt{20} = 2.24\ \text{N/mm}^2$

$V_{u2} < V_{s1}$ Hence safe

Comparison of the results

The results obtained from the numerical examples are compared and summarised in the Tables 6.1 and 6.2.

Table 6.1 Deflection of flat plate for 260 mm depth

Panel	Deflection in mm		
	Without concealed beam	With concealed beam	% reduction in deflection
Interior	11.41	10.96	3.96
Corner	12.98	12.16	6.32
Side	12.21	11.75	3.77

The deflection of corner panel is getting reduced by 6.32%. For interior and side panel the deflection is reduced by 3.97 and 3.77% respectively. Hence the concealed beam is effective in reducing the deflection in the corner panel.

Table 6.2 Moment and shear stress of flat plate for 260 mm depth

<table>
<tr><th colspan="2" rowspan="2">Details</th><th colspan="2">Edge column</th><th rowspan="2">Interior column</th></tr>
<tr><th>Interior face</th><th>Exterior face</th></tr>
<tr><td rowspan="3">Without concealed beam</td><td>Shear stress in N/mm^2</td><td>0.85</td><td>0.76</td><td>1.42</td></tr>
<tr><td>Limiting stress in N/mm^2</td><td>1.12</td><td>2.24</td><td>1.12</td></tr>
<tr><td>% used</td><td>75.89</td><td>67.86</td><td>126.78*</td></tr>
<tr><td rowspan="3">With concealed beam</td><td>Shear stress in N/mm^2</td><td>0.97</td><td>0.34</td><td>1.28</td></tr>
<tr><td>Limiting stress in N/mm^2</td><td>2.24</td><td>2.24</td><td>2.24</td></tr>
<tr><td>% used</td><td>43.3</td><td>15.18</td><td>57.14</td></tr>
<tr><td rowspan="3">Without concealed beam</td><td>Moment in kNm</td><td colspan="2">40.84</td><td rowspan="6">-</td></tr>
<tr><td>Limiting moment in kNm</td><td colspan="2">60.59</td></tr>
<tr><td>% used</td><td colspan="2">67.40</td></tr>
<tr><td rowspan="3">With concealed beam</td><td>Moment in kNm</td><td colspan="2">62.74</td></tr>
<tr><td>Limiting moment in kNm</td><td colspan="2">60.59</td></tr>
<tr><td>% used</td><td colspan="2">103.55</td></tr>
</table>

* Shear stirrups required

Table 6.2 shows the edge panel moment carrying capacity of flat plate. The capacity is increased by 1.5 times by using the concealed beam. Also the shear stress in the exterior face of the edge column is reduced by 40%. In the interior column, the shear stress is reduced by 10%. From this study, it can be concluded that the concealed beam is effective in improving the stiffness of flat plates. The numerical examples are solved to show how the concealed beam is modelled and used in the analysis to show the improved stiffness of flat plates.

Since the above computation involves time consuming lengthy calculations, a program was developed in MATLAB for the above procedure.

6.3 SUMMARY

From the comparative study, it is observed that by providing the concealed beam, the deflection in the corner panel is reduced by 6.3%. The moment carrying capacity of flat plate at the edge column is increased by 1.5 times compared to the flat plate without the concealed beam. Also, the shear stress is reduced by 55% at the exterior column face. Hence the concealed beam improves the stiffness of the flat plate particularly in the edge column.

The examples worked out in this chapter are only for flat plate. The aim of this study was to find the effect of concealed beam in flat plates and flat slab with column head for different parameters. Hence a program was developed for the above procedure. The development of the program is given in the next chapter. Also the next chapter contains the comparative study made for different parameters and the discussion about the results obtained.

CHAPTER 7

RESULTS AND DISCUSSIONS

7.1 GENERAL

The effect of the concealed beam in flat plate was studied and presented in the previous chapter by solving numerical examples. The aim of this study was to find the improved stiffness of flat plate using concealed beam for different parameters. Hence a program was developed for the procedure explained in the previous chapter. The following sections explain the development of the program and the comparative study made.

7.2 FLAT PLATE

7.2.1 Development of the Program

A program was developed for the procedure involved in the numerical examples given in Chapter 6, for calculation of deflection, moment carrying capacity and shear stress induced for the flat plates. The input parameters of the program are span (mm), live load (kN/m^2) and grade of concrete (N/mm^2), depth of slab (mm). The output of this program will give the total mid panel deflection (mm) for interior, corner and side panels. For the moment carrying capacity and shear stress the procedure explained in the example was used for the development of the program. The same input parameters are used for this program also. The output of this program will give the moment carrying capacity (kNm) and shear stress (N/mm^2) at the edge column and the interior column.

7.2.2 Parameters Considered

The following parameters were considered for the study to find the deflection, moment carrying capacity and shear stress induced for the effect of concealed beam.

Span	=	6 m x 6 m, 7 m x 7 m, 6 m x 7 m
Depth of slab	=	125 mm to 300 mm with an increment of 5mm.
Live Load	=	3 kN/m^2 and 5 kN/m^2
Grade of Concrete	= f_{ck} =	20 N/mm^2 and 30 N/mm^2
Grade of steel	= f_y =	415 N/mm^2
Clear cover	=	25 mm
Diameter of rods	=	10 mm
Area of tension Reinforcement	=	0.5 %
Creep strain	=	1.6 for 28 days age at loading
Shrinkage strain	=	300 x 10^{-6}
Modulus of elasticity of Concrete	=	5000 x $\sqrt{f_{ck}}$ N/mm^2
Square column	=	300 mm x 300 mm
Storey height	=	3.5 m

7.2.3 Comparison of Deflection

Table 7.1 and Fig. 7.1 show the details about the comparative study conducted for a span of 6 m x 6 m, live load of 3 kN/m^2 and f_{ck} = 20 N/mm^2 . The table furnishes the total mid panel deflection of corner panel in mm for the plate without and with concealed beam. The limit on deflection was taken as 20 mm. The ratio of the obtained deflection to the limit 20 mm was calculated to check the capacity of the slab in percentage and furnished in the Table 7.1 for comparison.

The deflection is reduced at a maximum of 5% for plates with the concealed beam. Also the effect of concealed beam in reducing the deflection is less with the increase in thickness of slab. This pattern can be observed in Fig. 7.1. Hence it is concluded that the concealed beam is effective in reducing the deflection in lesser depth of slab. This pattern is followed for the different parameters and is shown in Figs. 7.2 to 7.12.

7.2.4 Comparison of Moment

The limiting moment was taken as equal to 0.138 x f_{ck} x b x d^2 where "b" is the width of the critical section around the column and "d" is the effective depth of the slab. The moments obtained for the depths that had deflection less than 20 mm are furnished in the table. The ratio of the obtained and the limiting moment were calculated to find the percentage used capacity.

The moment at the corner column from the analysis is furnished in Table 7.1. The moment carrying capacity of the plate with concealed beam is nearly 1.5 times higher than the plate without concealed beam. Here also the concealed beam is effective in lesser depth of slab. If the slab depth is 300 mm, the increase in moment carrying capacity is getting reduced. Figure 7.1 shows a pattern of decrease in moment carrying capacity of plates with and without concealed beam with increase in depth of slab. But this may be due to the fact that increase in depth of slab increases the dead load moment.

7.2.5 Comparison of Shear Stress

The shear stress around the exterior column was calculated. In the table shear stresses for the depths that were considered for deflection and moment were furnished. The limit on shear stress for plate without concealed beam was calculated as per the formula 0.25 x $\sqrt{f_{ck}}$ in the IS: 456- 2000. For the flat plate with concealed beam this formula becomes 0.5 x $\sqrt{f_{ck}}$ which is to be considered for slab with shear reinforcement. Here to find the used capacity available, the ratio of shear stress obtained to the limiting stress was calculated and furnished in the table.

The shear stress at the exterior column is the critical stress in the flat plates. Hence the shear stress at the exterior column was studied. From the results obtained that are furnished in Table 7.1 it is observed that the shear stress is reduced to a maximum of 60%.

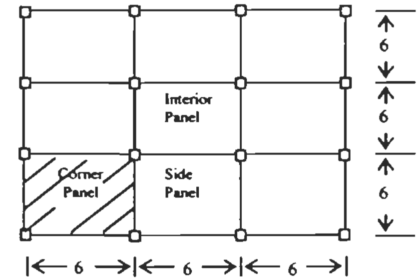


Table 7.1 Strength and serviceability of flat plate without and with concealed beam (6 m x 6 m) fck=20 N/mm² wl=3 kN/m²

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
230	17.37	86.85	16.58	82.90	45.61	44.16	103.28	74.61	168.95	0.65	58.04	0.04	1.79
240	15.69	78.45	15.01	75.05	44.03	49.29	89.33	70.37	142.77	0.70	62.50	0.16	7.14
250	14.24	71.20	13.65	68.25	42.44	54.77	77.49	66.41	121.25	0.73	65.18	0.25	11.16
260	12.97	64.85	12.46	62.30	40.84	60.59	67.40	62.74	103.55	0.76	67.86	0.34	15.18
270	11.87	59.35	11.41	57.05	39.26	66.77	58.80	59.33	88.86	0.79	70.54	0.40	17.86
280	10.89	54.45	10.49	52.45	37.68	73.31	51.40	56.16	76.61	0.81	72.32	0.46	20.54
290	10.03	50.15	9.67	48.35	36.12	80.23	45.02	53.23	66.35	0.82	73.21	0.51	22.77
300	9.27	46.35	8.95	44.75	34.58	87.52	39.51	50.50	57.70	0.83	74.11	0.55	24.55

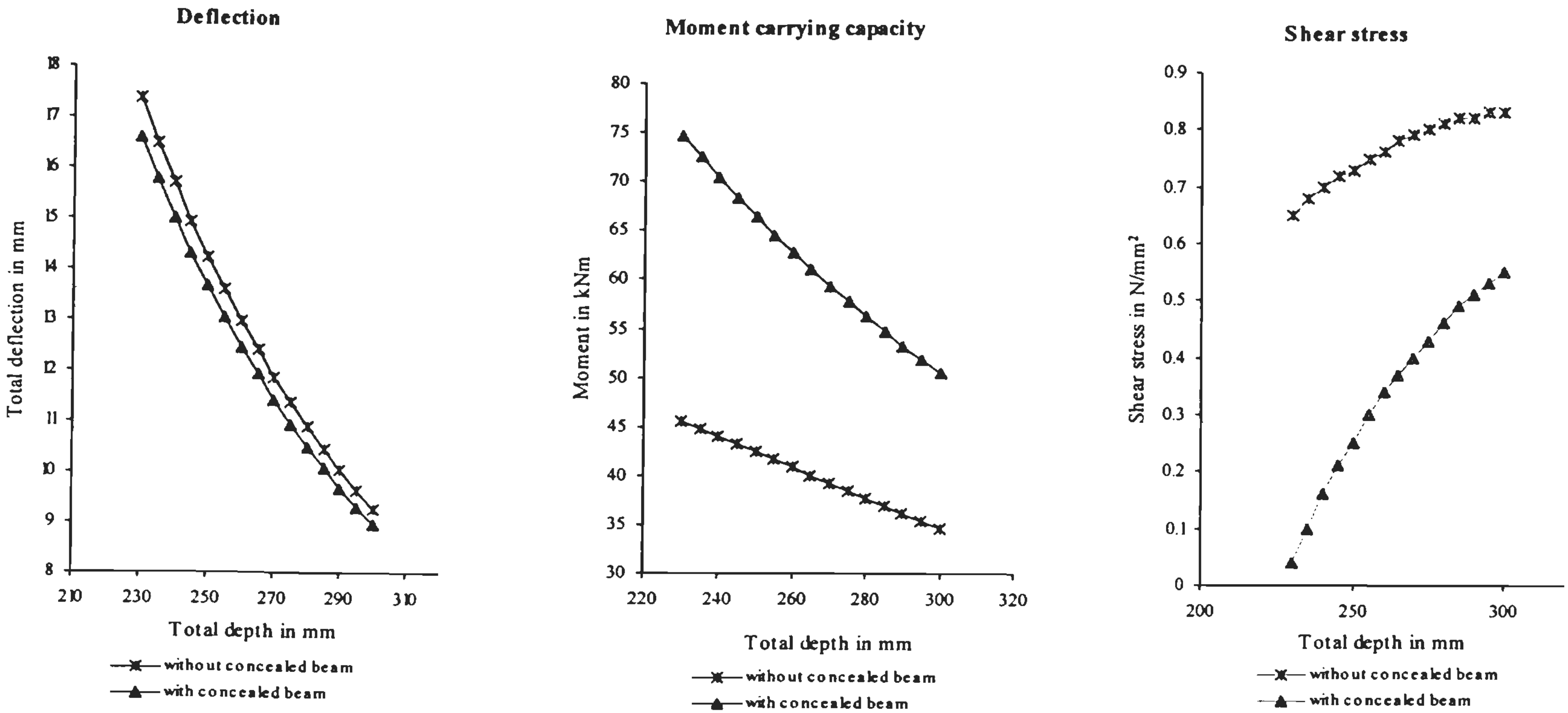


Fig. 7.1 Strength and serviceability of flat plate without and with concealed beam (6 m x 6 m) fck=20 N/mm^2 wl=3 kN/m^2

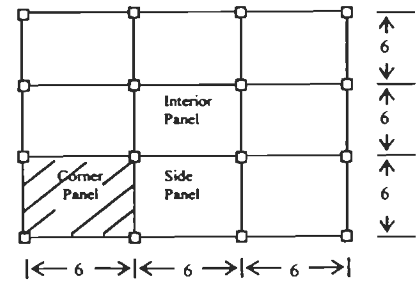


Table 7.2 Strength and serviceability of flat plate without and with concealed beam (6 m x 6 m) fck=30 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
200	19.85	99.25	18.92	94.60	50.14	30.71	163.27	90.85	295.83	0.46	41.07	-0.49	-21.88
210	17.65	88.25	16.87	84.35	48.68	34.88	139.56	85.76	245.87	0.54	48.21	-0.29	-12.95
220	15.78	78.90	15.13	75.65	47.16	39.36	119.82	80.94	205.64	0.60	53.57	-0.12	-5.36
230	14.18	70.90	13.63	68.15	45.61	44.16	103.28	76.40	173.01	0.65	58.04	0.02	0.89
240	12.81	64.05	12.34	61.70	44.03	49.29	89.33	72.14	146.36	0.70	62.50	0.14	6.25
250	11.63	58.15	11.22	56.10	42.44	54.77	77.49	68.17	124.47	0.73	65.18	0.24	10.71
260	10.59	52.95	10.24	51.20	40.84	60.59	67.40	64.46	106.39	0.76	67.86	0.33	14.73
270	9.69	48.45	9.38	46.90	39.26	66.77	58.80	61.01	91.37	0.79	70.54	0.40	17.86
280	8.89	44.45	8.62	43.10	37.68	73.31	51.40	57.80	78.84	0.81	72.32	0.46	20.54
290	8.19	40.95	7.95	39.75	36.12	80.23	45.02	54.81	68.32	0.82	73.21	0.51	22.77
300	7.57	37.85	7.35	36.75	34.58	87.52	39.51	52.04	59.46	0.83	74.11	0.55	24.55

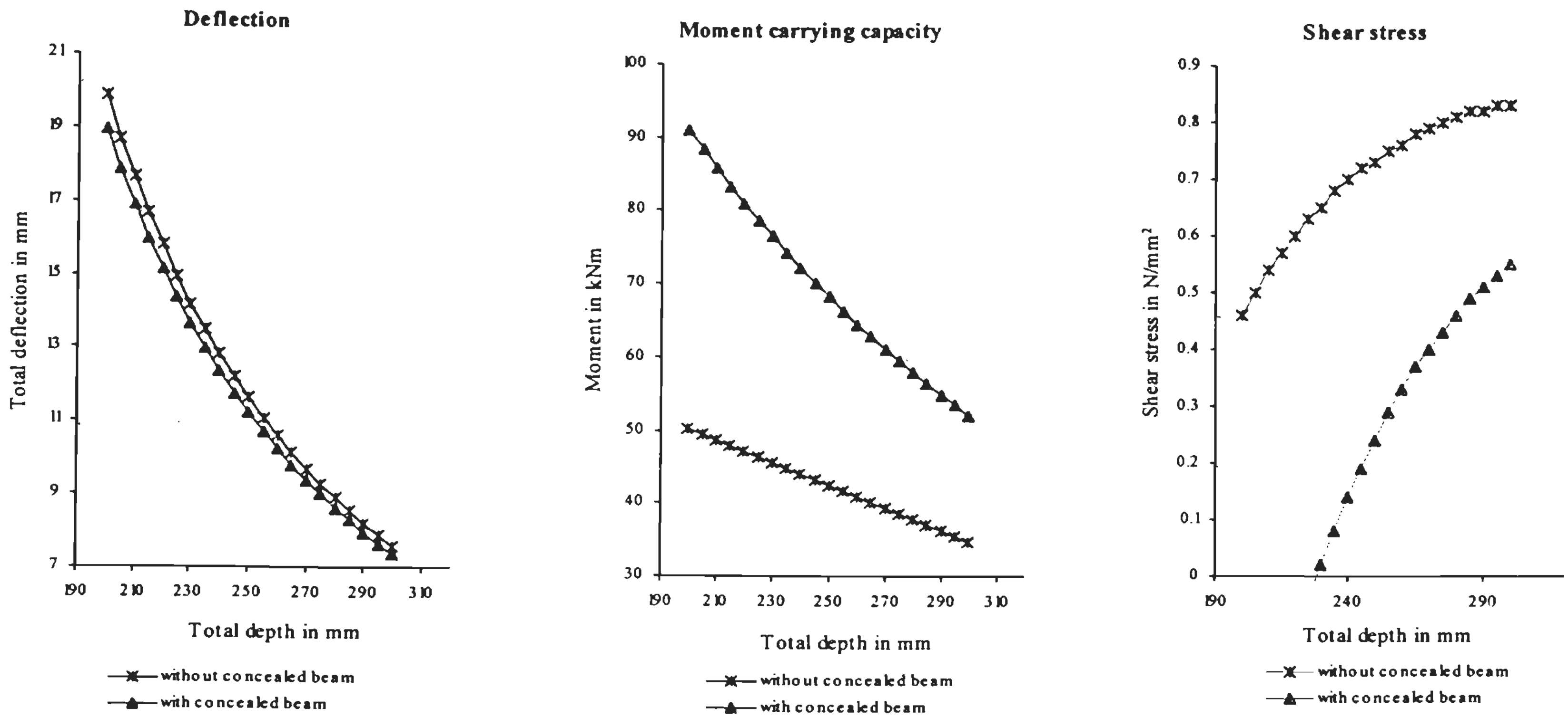


Fig. 7.2 Strength and serviceability of flat plate without and with concealed beam (6 m x 6 m) fck=30 N/mm^2 wl=3 kN/m^2

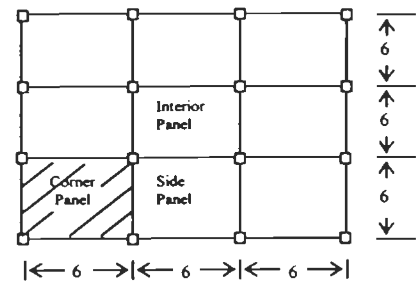


Table 7.3 Strength and serviceability of flat plate without and with concealed beam (6 m x 6 m) fck=20 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
230	20.31	101.55	19.30	96.50	54.51	44.16	123.44	89.17	201.92	0.78	69.64	0.05	2.23
240	18.29	91.45	17.43	87.15	52.42	49.29	106.35	83.77	169.95	0.83	74.11	0.19	8.48
250	16.54	82.70	15.81	79.05	50.33	54.77	91.89	78.77	143.82	0.87	77.68	0.30	13.39
260	15.03	75.15	14.39	71.95	48.27	60.59	79.67	74.15	122.38	0.90	80.36	0.40	17.86
270	13.71	68.55	13.15	65.75	46.23	66.77	69.24	69.87	104.64	0.93	83.04	0.48	21.43
280	12.55	62.75	12.06	60.30	44.23	73.31	60.33	65.93	89.93	0.95	84.82	0.54	24.11
290	11.52	57.60	11.09	55.45	42.27	80.23	52.69	62.29	77.64	0.96	85.71	0.59	26.34
300	10.62	53.10	10.23	51.15	40.35	87.52	46.10	58.92	67.32	0.97	86.61	0.64	28.57

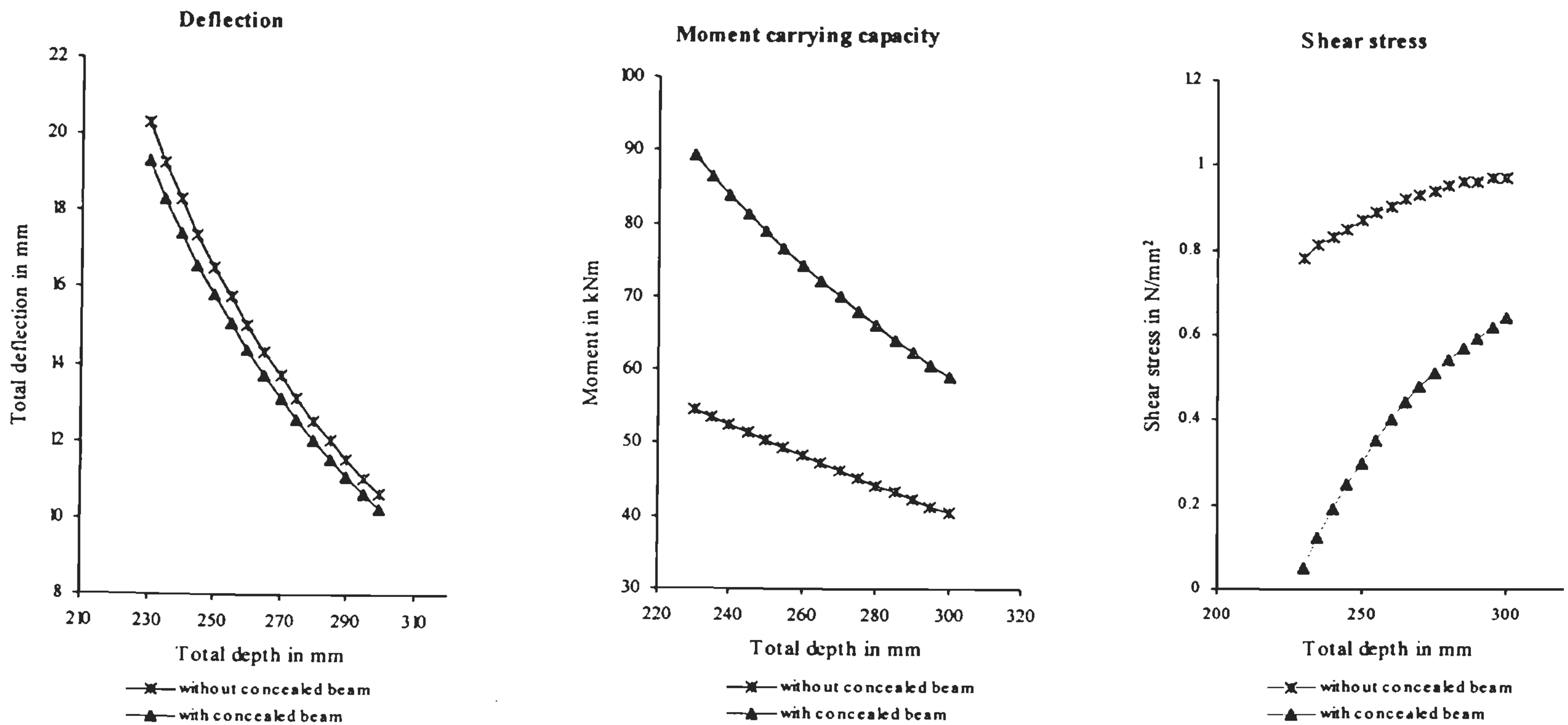


Fig. 7.3 Strength and serviceability of flat plate without and with concealed beam (6 m x 6 m) fck=20 N/mm^2 wl=5 kN/m^2

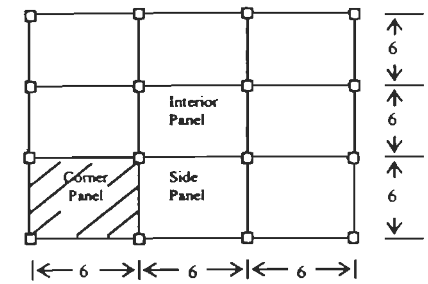


Table 7.4 Strength and serviceability of flat plate without and with concealed beam (6 m x 6 m) fck=30 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
220	18.52	92.60	17.66	88.30	56.60	39.36	143.80	56.60	143.80	0.72	64.29	-0.15	-6.70
230	16.59	82.95	15.87	79.35	54.51	44.16	123.44	54.51	123.44	0.78	69.64	0.02	0.89
240	14.93	74.65	14.33	71.65	52.42	49.29	106.35	52.42	106.35	0.83	74.11	0.17	7.59
250	13.51	67.55	12.99	64.95	50.33	54.77	91.89	50.33	91.89	0.87	77.68	0.29	12.95
260	12.27	61.35	11.83	59.15	48.27	60.59	79.67	48.27	79.67	0.90	80.36	0.39	17.41
270	11.19	55.95	10.81	54.05	46.23	66.77	69.24	46.23	69.24	0.93	83.04	0.47	20.98
280	10.24	51.20	9.91	49.55	44.23	73.31	60.33	44.23	60.33	0.95	84.82	0.54	24.11
290	9.41	47.05	9.11	45.55	42.27	80.23	52.69	42.27	52.69	0.96	85.71	0.60	26.79
300	8.67	43.35	8.41	42.05	40.35	87.52	46.10	40.35	46.10	0.97	86.61	0.64	28.57

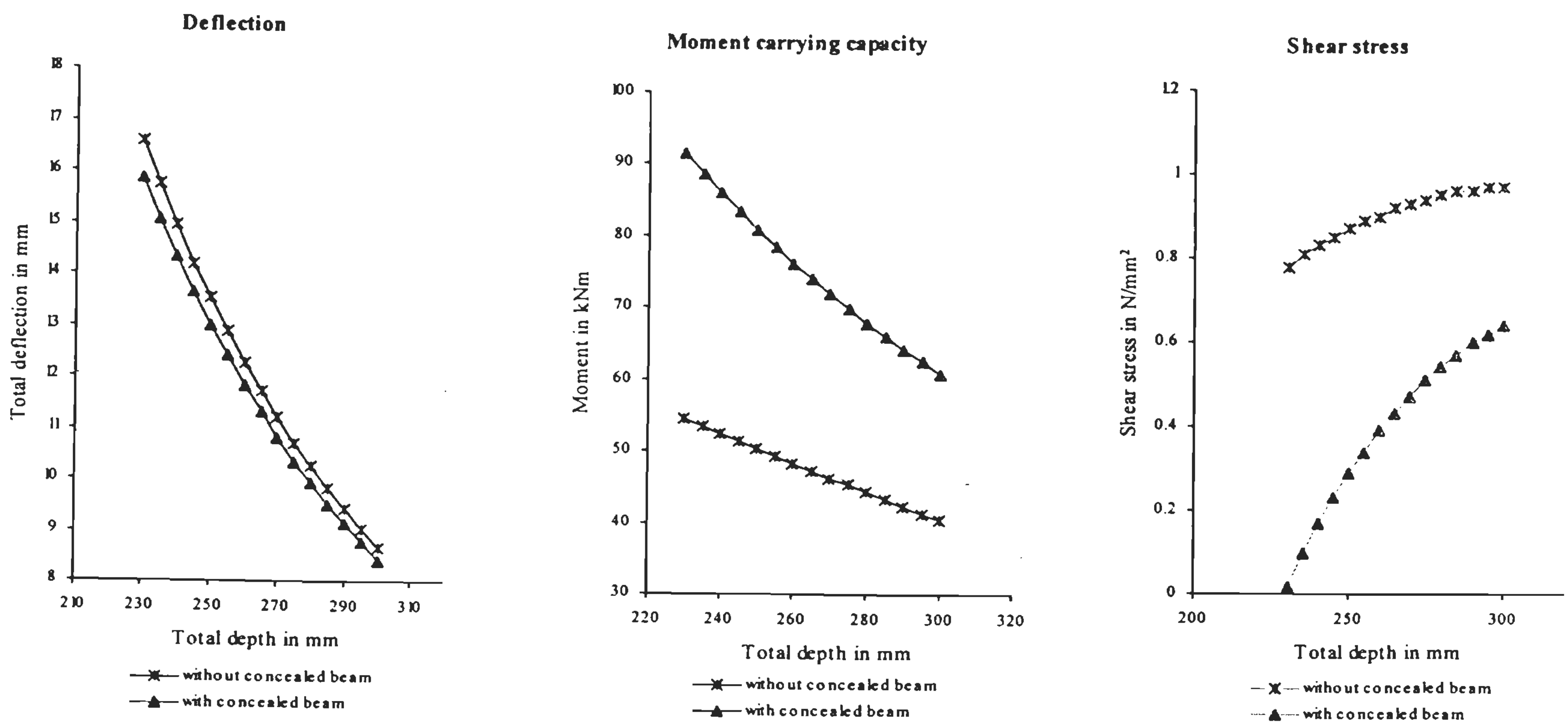


Fig. 7.4 Strength and serviceability of plate without and with concealed beam (6 m x 6 m) fck=30 N/mm^2 wl=5 kN/m^2

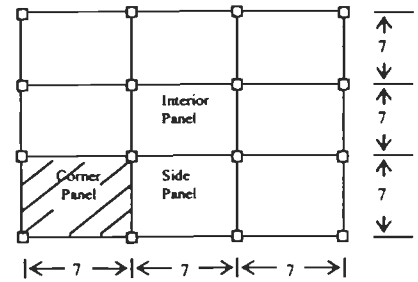


Table 7.5 Strength and serviceability of flat plate without and with concealed beam (7 m x 7 m) fck=20 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
280	20.23	101.15	19.56	97.80	54.86	73.31	74.83	88.81	121.14	1.03	91.96	0.47	20.98
285	19.40	97.00	18.77	93.85	53.75	76.72	70.06	86.46	112.70	1.04	92.86	0.51	22.77
290	18.63	93.15	18.03	90.15	52.64	80.23	65.61	84.20	104.95	1.06	94.64	0.55	24.55
295	17.89	89.45	17.33	86.65	51.54	83.83	61.48	82.02	97.84	1.07	95.54	0.58	25.89
300	17.20	86.00	16.67	83.35	50.44	87.52	57.63	79.92	91.32	1.08	96.43	0.62	27.68

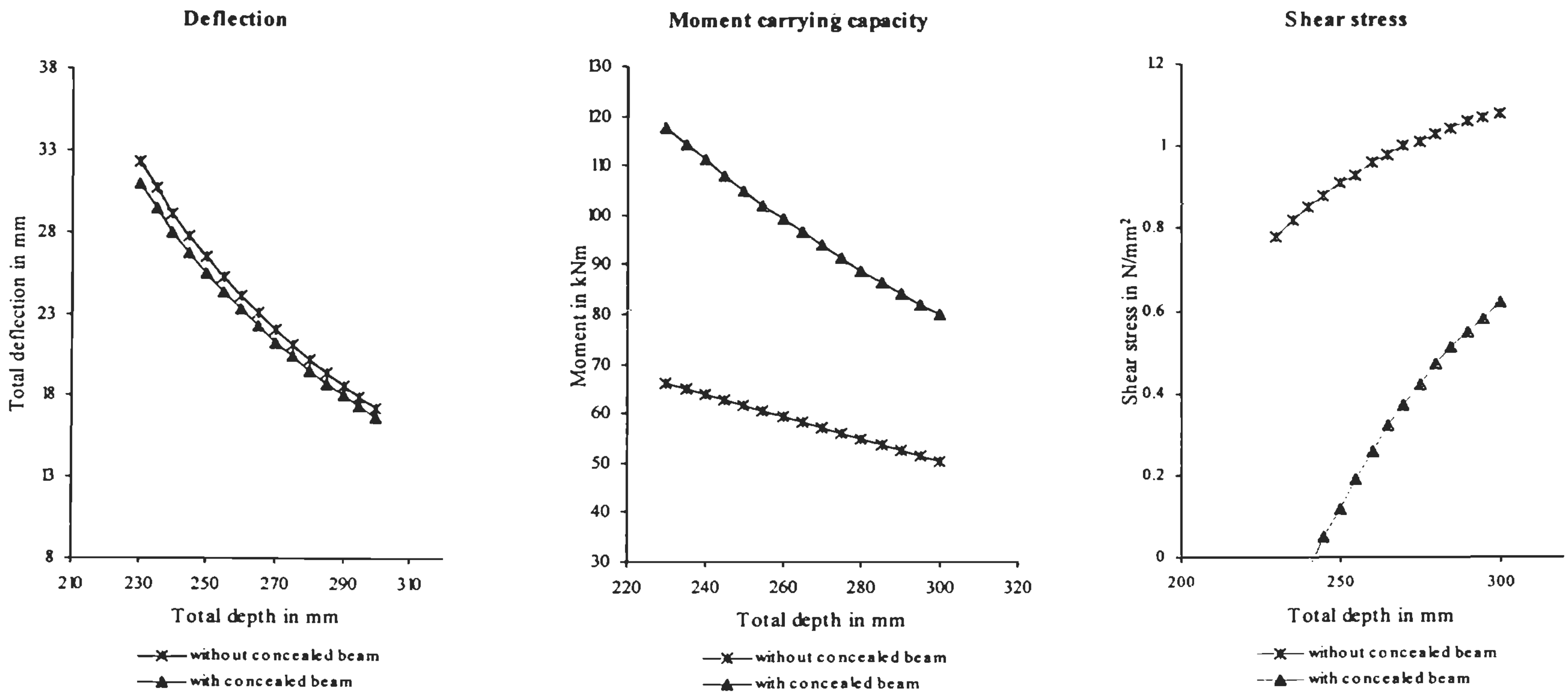


Fig. 7.5 Strength and serviceability of plate without and with concealed beam (7 m x 7 m) fck=20 N/mm^2 wl=3 kN/m^2

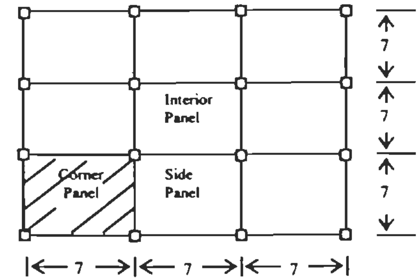


Table 7.6 Strength and serviceability of plate without and with concealed beam (7 m x 7 m) fck=30 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
260	19.67	98.35	19.08	95.40	59.32	60.59	97.90	59.32	97.90	0.96	85.71	0.24	10.71
265	18.81	94.05	18.25	91.25	58.20	63.64	91.45	58.20	91.45	0.98	87.50	0.30	13.39
270	17.99	89.95	17.47	87.35	57.09	66.77	85.50	57.09	85.50	1.00	89.29	0.36	16.07
275	17.23	86.15	16.74	83.70	55.97	70.00	79.96	55.97	79.96	1.01	90.18	0.41	18.30
280	16.52	82.60	16.06	80.30	54.86	73.31	74.83	54.86	74.83	1.03	91.96	0.46	20.54
285	15.84	79.20	15.41	77.05	53.75	76.72	70.06	53.75	70.06	1.04	92.86	0.50	22.32
290	15.21	76.05	14.80	74.00	52.64	80.23	65.61	52.64	65.61	1.06	94.64	0.54	24.11
295	14.61	73.05	14.23	71.15	51.54	83.83	61.48	51.54	61.48	1.07	95.54	0.58	25.89
300	14.05	70.25	13.69	68.45	50.44	87.52	57.63	50.44	57.63	1.08	96.43	0.61	27.23

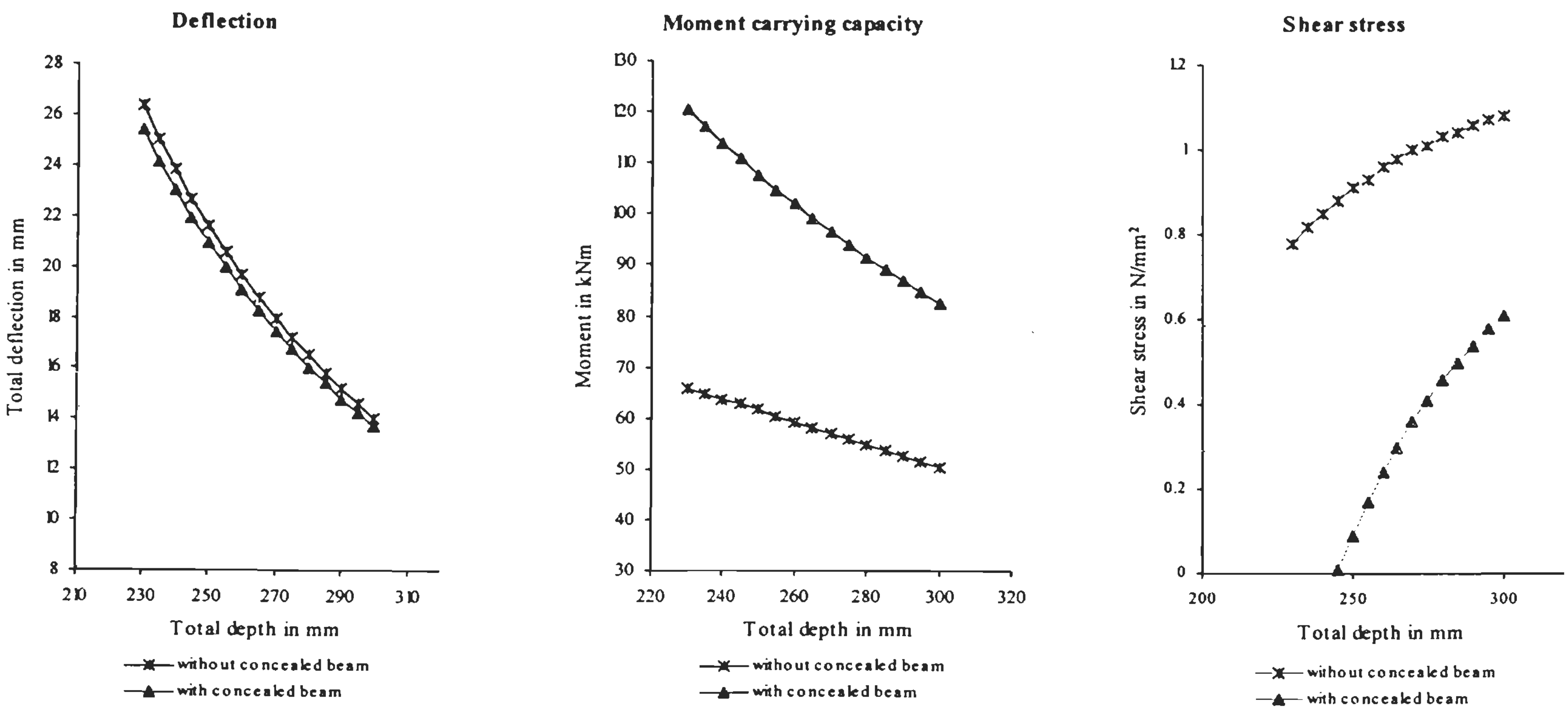


Fig. 7.6 Strength and serviceability of plate without and with concealed beam (7 m x 7 m) fck=30 N/mm^2 wl=3 kN/m^2

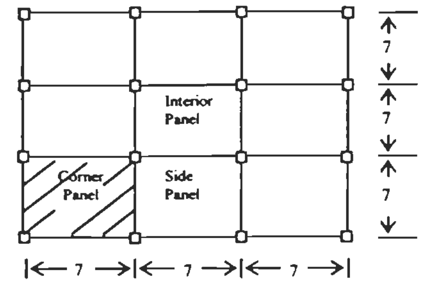


Table 7.7 Strength and serviceability of plate without and with concealed beam (7 m x 7 m) fck=20 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
280	23.32	116.60	22.49	112.45	64.40	73.31	87.85	104.26	142.22	1.21	108.04	1.21	54.02
285	22.34	111.70	21.56	107.80	63.00	76.72	82.12	101.34	132.09	1.22	108.93	1.22	54.46
290	21.41	107.05	20.69	103.45	61.60	80.23	76.78	98.53	122.81	1.24	110.71	1.24	55.36
295	20.54	102.70	19.86	99.30	60.22	83.83	71.84	95.84	114.33	1.25	111.61	1.25	55.80
300	19.73	98.65	19.08	95.40	58.85	87.52	67.24	93.24	106.54	1.26	112.50	1.26	56.25

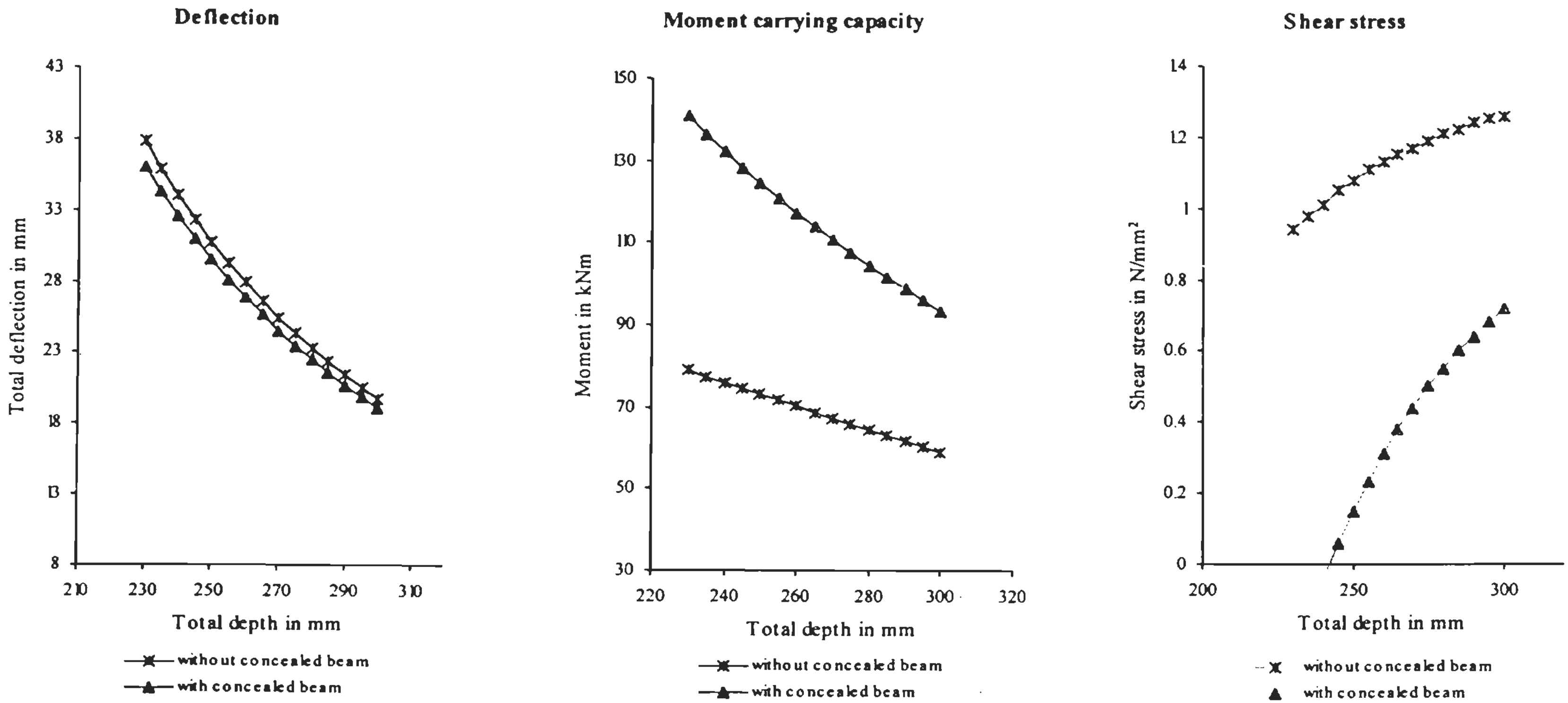


Fig. 7.7 Strength and serviceability of plate without and with concealed beam (7 m x 7 m) fck=20 N/mm^2 wl=5 kN/m^2

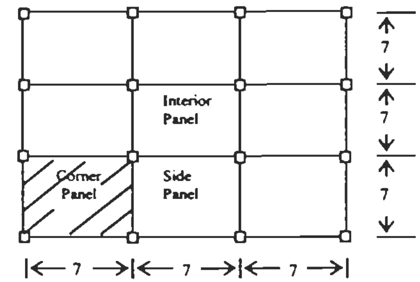


Table 7.8 Strength and serviceability of plate without and with concealed beam (7 m x 7 m) fck=30 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
260	22.81	114.05	18.32	91.60	70.11	60.59	115.71	120.38	198.68	1.13	100.89	0.28	12.50
265	21.78	108.90	17.52	87.60	68.67	63.64	107.90	116.92	183.72	1.15	102.68	0.35	15.63
270	20.80	104.00	16.77	83.85	67.24	66.77	100.70	113.59	170.12	1.17	104.46	0.42	18.75
275	19.89	99.45	16.07	80.35	65.81	70.00	94.01	110.39	157.70	1.19	106.25	0.48	21.43
280	19.04	95.20	15.41	77.05	64.40	73.31	87.85	107.30	146.36	1.21	108.04	0.54	24.11
285	18.24	91.20	14.79	73.95	63.00	76.72	82.12	104.34	136.00	1.22	108.93	0.59	26.34
290	17.48	87.40	14.21	71.05	61.60	80.23	76.78	101.48	126.49	1.24	110.71	0.64	28.57
295	16.77	83.85	13.65	68.25	60.22	83.83	71.84	98.73	117.77	1.25	111.61	0.68	30.36
300	16.11	80.55	13.13	65.65	58.85	87.52	67.24	96.09	109.79	1.26	112.50	0.72	32.14

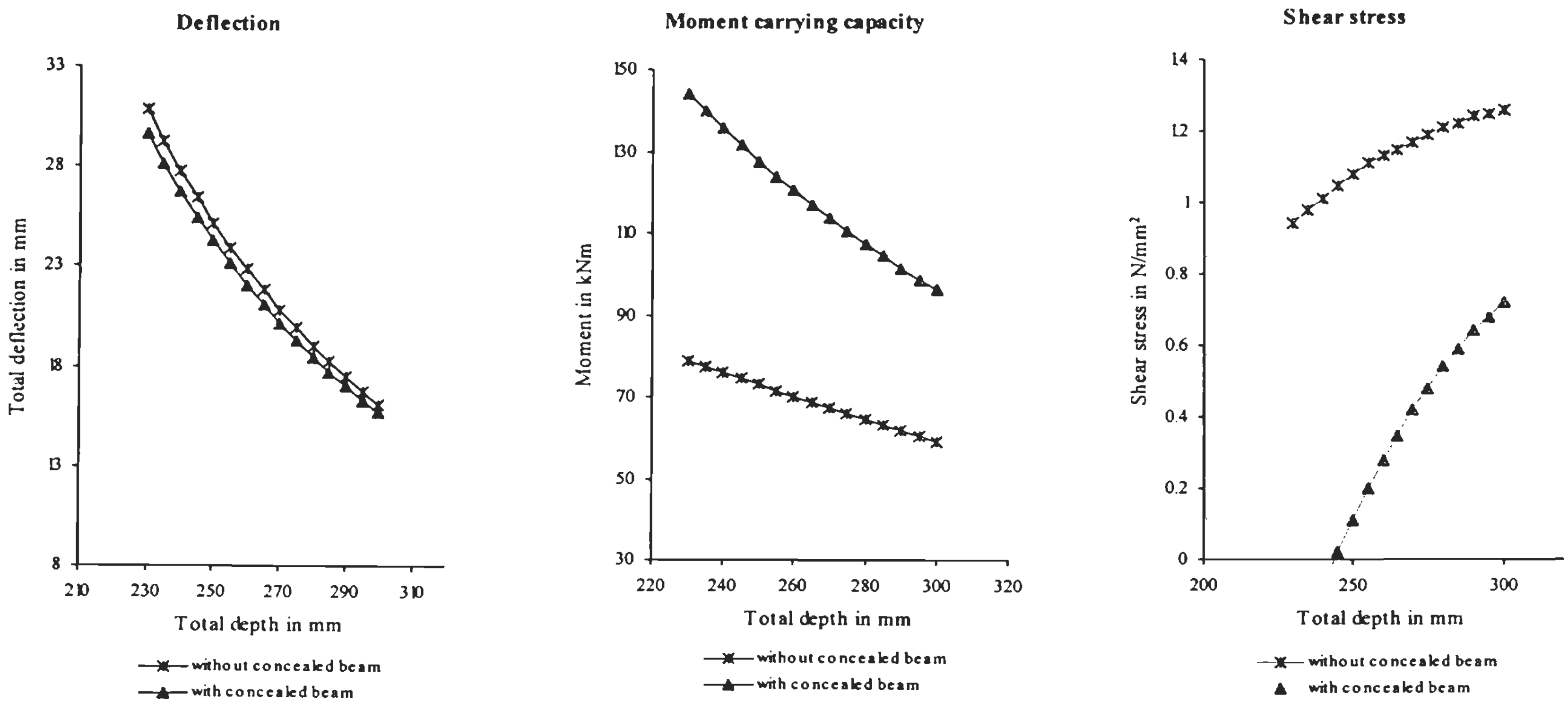


Fig. 7.8 Strength and serviceability of plate without and with concealed beam (7 m x 7 m) fck=30 N/mm^2 wl=5 kN/m^2

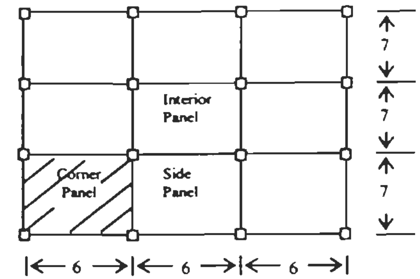


Table 7.9 Strength and serviceability of plate without and with concealed beam (6 m x 7 m) fck=20 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
230	24.97	124.85	23.80	119.00	48.42	44.16	109.65	86.45	195.77	0.85	75.89	0.06	2.68
235	23.72	118.60	22.64	113.20	47.63	46.69	102.01	83.97	179.85	0.87	77.68	0.13	5.80
240	22.56	112.80	21.55	107.75	46.82	49.29	94.99	81.58	165.51	0.89	79.46	0.19	8.48
245	21.48	107.40	20.54	102.70	46.01	51.99	88.50	79.27	152.47	0.91	81.25	0.25	11.16
250	20.47	102.35	19.60	98.00	45.20	54.77	82.53	77.03	140.64	0.93	83.04	0.30	13.39
255	19.53	97.65	18.72	93.60	44.38	57.64	77.00	74.88	129.91	0.94	83.93	0.35	15.63
260	18.66	93.30	17.89	89.45	43.56	60.59	71.89	72.80	120.15	0.95	84.82	0.40	17.86
265	17.84	89.20	17.12	85.60	42.74	63.64	67.16	70.80	111.25	0.96	85.71	0.44	19.64
270	17.07	85.35	16.39	81.95	41.92	66.77	62.78	68.87	103.15	0.97	86.61	0.48	21.43
275	16.34	81.70	15.71	78.55	41.10	70.00	58.71	67.02	95.74	0.98	87.50	0.51	22.77
280	15.67	78.35	15.07	75.35	40.29	73.31	54.96	65.22	88.96	0.99	88.39	0.54	24.11
285	15.03	75.15	14.46	72.30	39.47	76.72	51.45	63.50	82.77	1.00	89.29	0.57	25.45
290	14.43	72.15	13.90	69.50	38.66	80.23	48.19	61.84	77.08	1.00	89.29	0.60	26.79
295	13.86	69.30	13.36	66.80	37.85	83.83	45.15	60.24	71.86	1.01	90.18	0.62	27.68
300	13.33	66.65	12.85	64.25	37.04	87.52	42.32	58.69	67.06	1.01	90.18	0.64	28.57

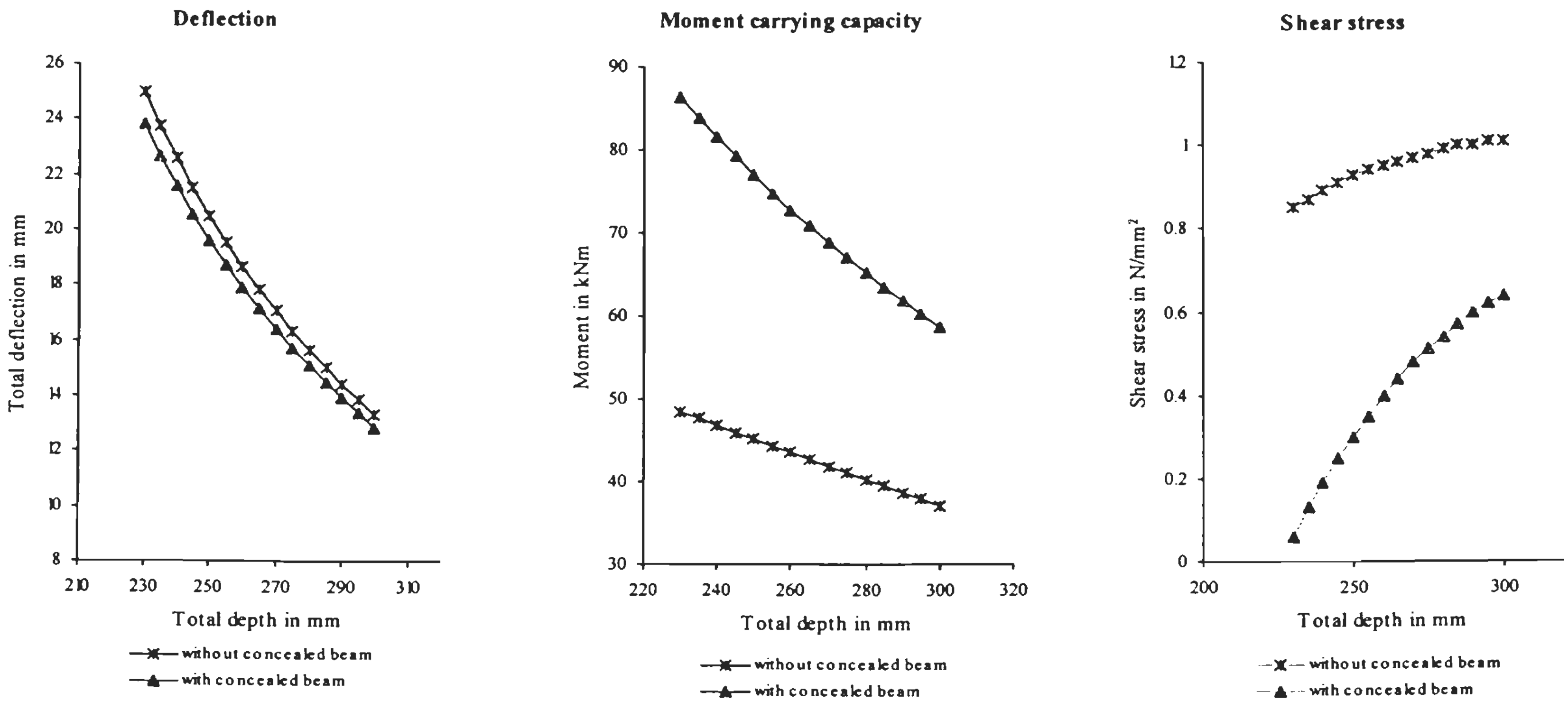


Fig. 7.9 Strength and serviceability of plate without and with concealed beam (6 m x 7 m) fck=20 N/mm^2 wl=3 kN/m^2

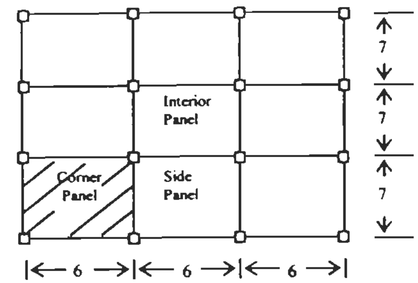


Table 7.10 Strength and serviceability of plate without and with concealed beam (6 m x 7 m) fck=30 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
230	20.39	101.95	19.58	97.90	48.42	44.16	109.65	88.53	200.48	0.85	75.89	0.03	1.34
235	19.37	96.85	18.62	93.10	47.63	46.69	102.01	86.05	184.30	0.87	77.68	0.11	4.91
240	18.42	92.10	17.73	88.65	46.82	49.29	94.99	83.65	169.71	0.89	79.46	0.17	7.59
245	17.54	87.70	16.89	84.45	46.01	51.99	88.50	81.32	156.41	0.91	81.25	0.23	10.27
250	16.72	83.60	16.12	80.60	45.20	54.77	82.53	79.08	144.39	0.93	83.04	0.29	12.95
255	15.95	79.75	15.39	76.95	44.38	57.64	77.00	76.90	133.41	0.94	83.93	0.34	15.18
260	15.23	76.15	14.71	73.55	43.56	60.59	71.89	74.81	123.47	0.95	84.82	0.39	17.41
265	14.56	72.80	14.07	70.35	42.74	63.64	67.16	72.78	114.36	0.96	85.71	0.43	19.20
270	13.93	69.65	13.48	67.40	41.92	66.77	62.78	70.83	106.08	0.97	86.61	0.47	20.98
275	13.35	66.75	12.91	64.55	41.10	70.00	58.71	68.95	98.50	0.98	87.50	0.51	22.77
280	12.79	63.95	12.39	61.95	40.29	73.31	54.96	67.13	91.57	0.99	88.39	0.54	24.11
285	12.27	61.35	11.89	59.45	39.47	76.72	51.45	65.38	85.22	1.00	89.29	0.57	25.45
290	11.78	58.90	11.42	57.10	38.66	80.23	48.19	63.69	79.38	1.00	89.29	0.60	26.79
295	11.32	56.60	10.98	54.90	37.85	83.83	45.15	62.06	74.03	1.01	90.18	0.62	27.68
300	10.88	54.40	10.56	52.80	37.04	87.52	42.32	60.48	69.10	1.01	90.18	0.64	28.57

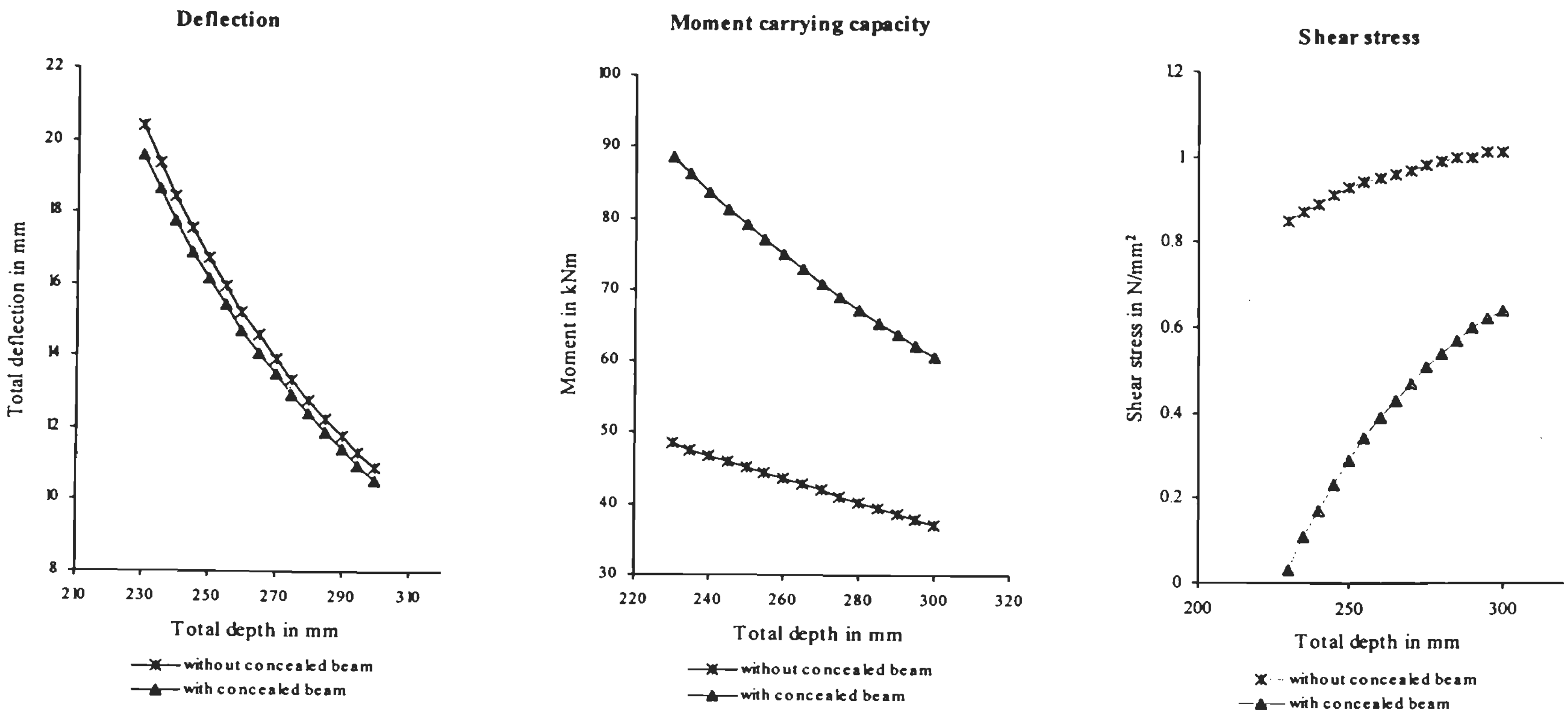


Fig. 7.10 Strength and serviceability of plate without and with concealed beam (6 m x 7 m) fck=30 N/mm^2 wl=3 kN/m^2

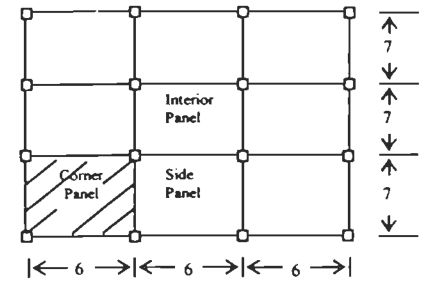


Table 7.11 Strength and serviceability of plate without and with concealed beam (6 m x 7 m) fck=20 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
230	29.15	145.75	27.67	138.35	57.87	44.16	131.05	103.32	233.97	1.02	91.07	0.07	3.13
235	27.64	138.20	26.28	131.40	56.81	46.69	121.67	100.16	214.52	1.04	92.86	0.15	6.70
240	26.25	131.25	24.99	124.95	55.74	49.29	113.09	97.12	197.04	1.06	94.64	0.23	10.27
245	24.95	124.75	23.78	118.90	54.67	51.99	105.15	94.19	181.17	1.08	96.43	0.30	13.39
250	23.74	118.70	22.66	113.30	53.61	54.77	97.88	91.37	166.82	1.10	98.21	0.36	16.07
255	22.62	113.10	21.61	108.05	52.54	57.64	91.15	88.65	153.80	1.11	99.11	0.42	18.75
260	21.57	107.85	20.63	103.15	51.48	60.59	84.96	86.04	142.00	1.12	100.00	0.47	20.98
265	20.59	102.95	19.71	98.55	50.43	63.64	79.24	83.53	131.25	1.14	101.79	0.52	23.21
270	19.67	98.35	18.85	94.25	49.38	66.77	73.96	81.12	121.49	1.15	102.68	0.56	25.00
275	18.81	94.05	18.05	90.25	48.33	70.00	69.04	78.80	112.57	1.15	102.68	0.60	26.79
280	18.01	90.05	17.29	86.45	47.29	73.31	64.51	76.57	104.45	1.16	103.57	0.64	28.57
285	17.25	86.25	16.58	82.90	46.26	76.72	60.30	74.42	97.00	1.17	104.46	0.67	29.91
290	16.54	82.70	15.90	79.50	45.24	80.23	56.39	72.36	90.19	1.17	104.46	0.70	31.25
295	15.87	79.35	15.27	76.35	44.22	83.83	52.75	70.38	83.96	1.17	104.46	0.72	32.14
300	15.24	76.20	14.67	73.35	43.21	87.52	49.37	68.48	78.24	1.18	105.36	0.75	33.48

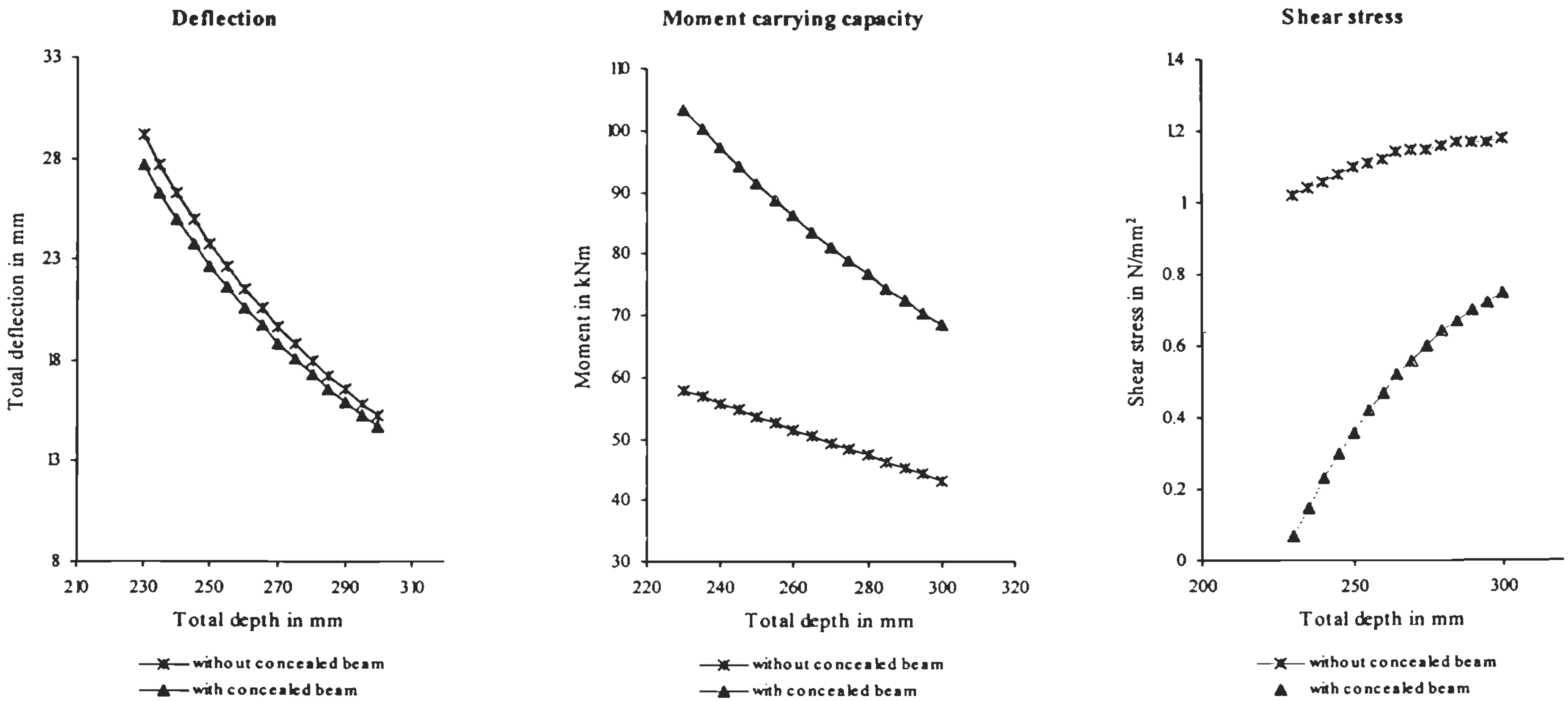


Fig. 7.11 Strength and serviceability of plate without and with concealed beam (6 m x 7 m) fck=20 N/mm^2 wl=5 kN/m^2

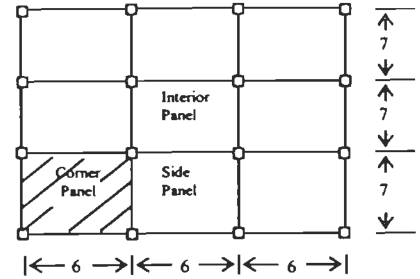


Table 7.12 Strength and serviceability of plate without and with concealed beam (6 m x 7 m) fck=30 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
230	23.80	119.00	22.76	113.80	57.87	44.16	131.05	105.81	239.61	1.02	91.07	0.04	1.79
235	22.57	112.85	21.61	108.05	56.81	46.69	121.67	102.64	219.83	1.04	92.86	0.13	5.80
240	21.43	107.15	20.55	102.75	55.74	49.29	113.09	99.58	202.03	1.06	94.64	0.21	9.38
245	20.37	101.85	19.56	97.80	54.67	51.99	105.15	96.63	185.86	1.08	96.43	0.28	12.50
250	19.39	96.95	18.63	93.15	53.61	54.77	97.88	93.79	171.24	1.10	98.21	0.34	15.18
255	18.47	92.35	17.77	88.85	52.54	57.64	91.15	91.05	157.96	1.11	99.11	0.40	17.86
260	17.61	88.05	16.96	84.80	51.48	60.59	84.96	88.41	145.92	1.12	100.00	0.46	20.54
265	16.81	84.05	16.21	81.05	50.43	63.64	79.24	85.87	134.93	1.14	101.79	0.51	22.77
270	16.06	80.30	15.50	77.50	49.38	66.77	73.96	83.42	124.94	1.15	102.68	0.55	24.55
275	15.36	76.80	14.84	74.20	48.33	70.00	69.04	81.07	115.81	1.15	102.68	0.60	26.79
280	14.70	73.50	14.21	71.05	47.29	73.31	64.51	78.80	107.49	1.16	103.57	0.63	28.13
285	14.09	70.45	13.62	68.10	46.26	76.72	60.30	76.62	99.87	1.17	104.46	0.67	29.91
290	13.51	67.55	13.07	65.35	45.24	80.23	56.39	74.53	92.90	1.17	104.46	0.70	31.25
295	12.96	64.80	12.55	62.75	44.22	83.83	52.75	72.51	86.50	1.17	104.46	0.73	32.59
300	12.44	62.20	12.06	60.30	43.21	87.52	49.37	70.56	80.62	1.18	105.36	0.75	33.48

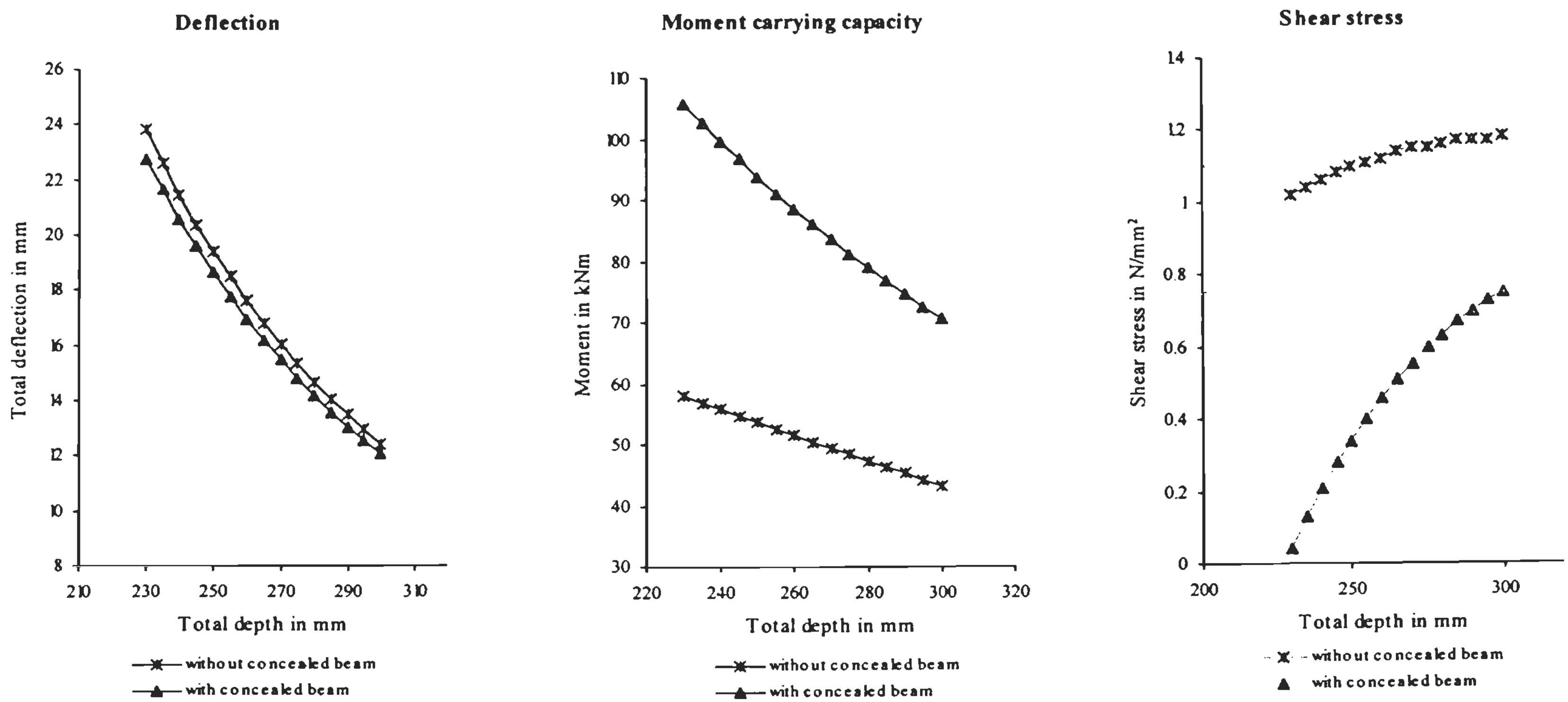


Fig. 7.12 Strength and serviceability of plate without and with concealed beam (6 m x 7 m) fck=30 N/mm^2 wl=5 kN/m^2

7.3 FLAT SLAB WITH COLUMN HEAD

7.3.1 Development of the Program

A program was developed for the proposed model given in Chapter 3, for calculation of deflection. For the moment carrying capacity and shear stress induced the same program developed for the flat plates (Chapter 6) with little modification in moment of inertia was used. The input parameters of the program are span (mm), live load (kN/m^2) and grade of concrete (N/mm^2), depth of slab (mm). The output of this program will give the total mid panel deflection (mm) for interior, corner and side panel. For the moment carrying the same input parameters given above are used for this program also. The output of this program will give the moment carrying capacity (kNm) and shear stress (N/mm^2) at the edge column and the interior column.

7.3.2 Parameters Considered

The following parameters were considered for the study to find the deflection, moment carrying capacity and shear stress induced for the effect of concealed beam.

Span	=	6 m x 8 m, 7 m x 8 m, 8 m x 8 m
Depth of slab	=	125 mm to 300 mm with an increment of 5mm.
Live Load	=	3 kN/m^2 and 5 kN/m^2
Grade of Concrete	= f_{ck}	= 20 N/mm^2 and 30 N/mm^2
Grade of steel	= f_y	= 415 N/mm^2
Clear cover		= 25 mm
Diameter of rods	=	10 mm
Area of tension Reinforcement	=	0.5 %
Creep strain	=	1.6 for 28 days age at loading
Shrinkage strain	=	300 x 10^{-6}
Modulus of elasticity of Concrete	=	5000 x $\sqrt{f_{ck}}$ N/mm^2
Square column	=	300 mm x 300 mm

Column head

Height = $L_x/10$ m

Storey height = 3.5 m

7.3.3 Comparison of Deflection

Table 7.13 and Fig. 7.13 show the details about the comparative study conducted for a span of 6 m x 6 m, live load of 3 kN/m^2 and f_{ck} = 20 N/mm^2 . The table furnishes the total mid panel deflection of corner panel in mm for the flat slab without and with concealed beam. The limit on deflection was taken as 20 mm. The ratio of the obtained deflection to the limit 20 mm was calculated to check the used capacity of the slab in percentage and furnished in the table for comparison.

The deflection is reduced at a maximum of 5% for plates with the concealed beam. Also the effect of concealed beam in reducing the deflection is less with the increase in thickness of slab. This pattern can be observed in Fig. 7.13 a. Hence it is concluded that the concealed beam is effective in reducing the deflection in lesser depth of slab.

7.3.4 Comparison of Moment

The limiting moment was calculated using the expression $0.138 \times f_{ck} \times b \times d^2$ where "b" is the width of the critical section around the column and "d" is the effective depth of slab. The moments obtained for the depths that had deflection less than 20 mm are furnished in the table. The ratio of the obtained and the limiting moments were calculated to find the percentage used capacity.

The moment at the corner column from the analysis is furnished in Table 7.13. The moment carrying capacity of the slab with concealed beam is nearly 10% more than the slab without concealed beam. The concealed beam is effective in lesser depth of slab. If the slab depth is increased, the increase in moment carrying capacity is getting reduced. At one stage the moment carrying of the slab without concealed beam is more than the slab with concealed beam. Figure 7.13 shows a pattern of decrease in moment carrying capacity of slab with and without concealed beam with increase in depth of slab.

7.3.5 Comparison of Shear Stress

The shear stress around the exterior column was calculated. Shear stresses for the depths that were considered for deflection and moment were furnished in the table. The limit on shear stress for plate without concealed beam was calculated as per the formula 0.25 x $\sqrt{f_{ck}}$ in the IS: 456- 2000. For the plate with concealed beam, this formula becomes 0.5 x $\sqrt{f_{ck}}$ which is to be considered for slab with shear reinforcement. Here to find the used capacity available, the ratio of shear stress obtained to the limiting stress was calculated and furnished in the table.

The shear stress at the exterior column is the critical stress in the flat slabs. Hence the shear stress at the exterior column was studied. From the results obtained that are furnished in Table 7.13, it can be seen that the shear stress is reduced to a maximum of 15 %.

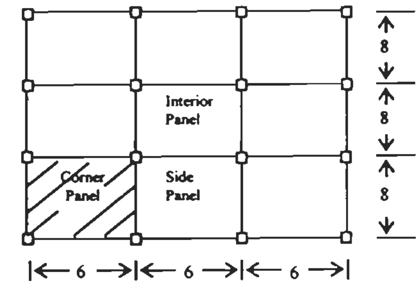


Table 7.13 Strength and serviceability of slab without and with concealed beam (6 m x 8 m) fck=20 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
190	19.52	97.60	18.45	92.25	116.47	90.44	128.78	134.40	148.61	0.95	84.82	0.80	35.71
200	17.13	85.65	16.34	81.70	113.83	102.50	111.05	127.28	124.18	0.92	82.14	0.80	35.71
210	15.14	75.70	14.55	72.75	110.93	115.36	96.16	120.45	104.41	0.90	80.36	0.79	35.27
220	13.46	67.30	13.02	65.10	107.83	129.03	83.57	113.95	88.31	0.88	78.57	0.78	34.82
230	12.03	60.15	11.71	58.55	104.59	143.52	72.87	107.79	75.10	0.87	77.68	0.78	34.82
240	10.81	54.05	10.57	52.85	101.25	158.84	63.74	101.99	64.21	0.85	75.89	0.77	34.38
250	9.76	48.80	9.58	47.90	97.86	175.00	55.92	96.54	55.17	0.83	74.11	0.76	33.93
260	8.85	44.25	8.72	43.60	94.47	192.00	49.20	91.44	47.63	0.82	73.21	0.75	33.48
270	8.06	40.30	7.96	39.80	91.09	209.85	43.41	86.67	41.30	0.81	72.32	0.75	33.48
280	7.37	36.85	7.29	36.45	87.76	228.56	38.40	82.23	35.98	0.80	71.43	0.74	33.04
290	6.75	33.75	6.70	33.50	84.50	248.15	34.05	78.08	31.46	0.78	69.64	0.73	32.59
300	6.21	31.05	6.17	30.85	81.33	268.61	30.28	74.22	27.63	0.77	68.75	0.72	32.14

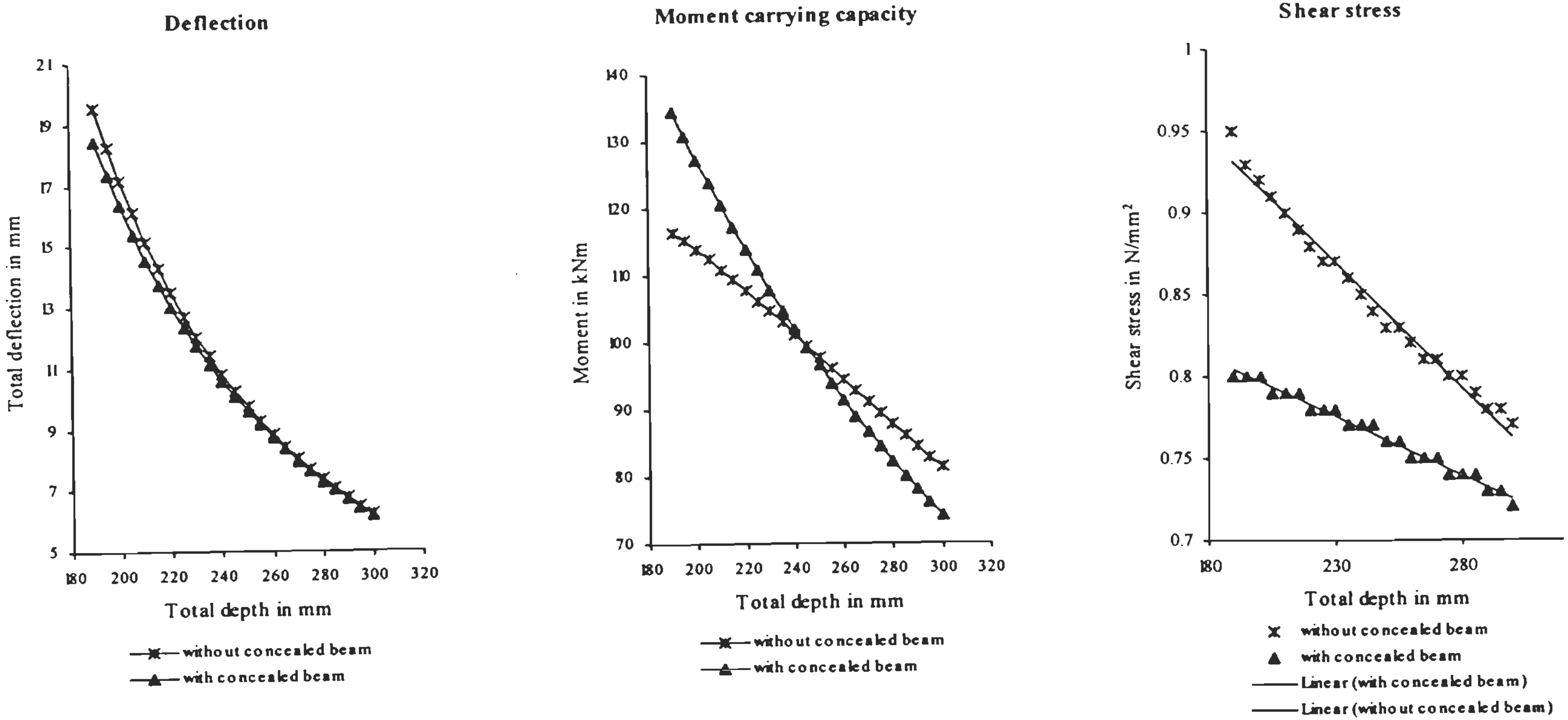


Fig. 7.13 Strength and serviceability of slab without and with concealed beam (6 m x 8 m) fck=20 N/mm² wl=3 kN/m²

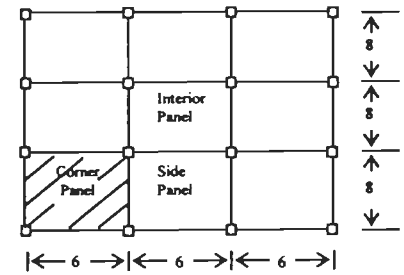


Table 7.14 Strength and serviceability of slab without and with concealed beam (6 m x 8 m) fck=30 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
180	18.31	91.55	17.14	85.70	118.79	118.77	100.02	143.82	121.09	0.97	70.80	0.82	36.61
190	15.94	79.70	15.09	75.45	116.47	135.66	85.85	136.64	100.72	0.95	69.34	0.81	36.16
200	13.99	69.95	13.36	66.80	113.83	153.75	74.04	129.64	84.32	0.92	67.15	0.81	36.16
210	12.36	61.80	11.90	59.50	110.93	173.04	64.11	122.89	71.02	0.90	65.69	0.80	35.71
220	10.99	54.95	10.65	53.25	107.83	193.54	55.71	116.43	60.16	0.88	64.23	0.79	35.27
230	9.82	49.10	9.57	47.85	104.59	215.28	48.58	110.29	51.23	0.87	63.50	0.79	35.27
240	8.83	44.15	8.64	43.20	101.25	238.26	42.50	104.49	43.86	0.85	62.04	0.78	34.82
250	7.97	39.85	7.83	39.15	97.86	262.49	37.28	99.02	37.72	0.83	60.58	0.77	34.38
260	7.23	36.15	7.13	35.65	94.47	287.99	32.80	93.89	32.60	0.82	59.85	0.76	33.93
270	6.58	32.90	6.51	32.55	91.09	314.77	28.94	89.07	28.30	0.81	59.12	0.76	33.93
280	6.01	30.05	5.96	29.80	87.76	342.84	25.60	84.57	24.67	0.80	58.39	0.75	33.48
290	5.51	27.55	5.48	27.40	84.50	372.22	22.70	80.37	21.59	0.78	56.93	0.74	33.04
300	5.07	25.35	5.05	25.25	81.33	402.91	20.19	76.44	18.97	0.77	56.20	0.73	32.59

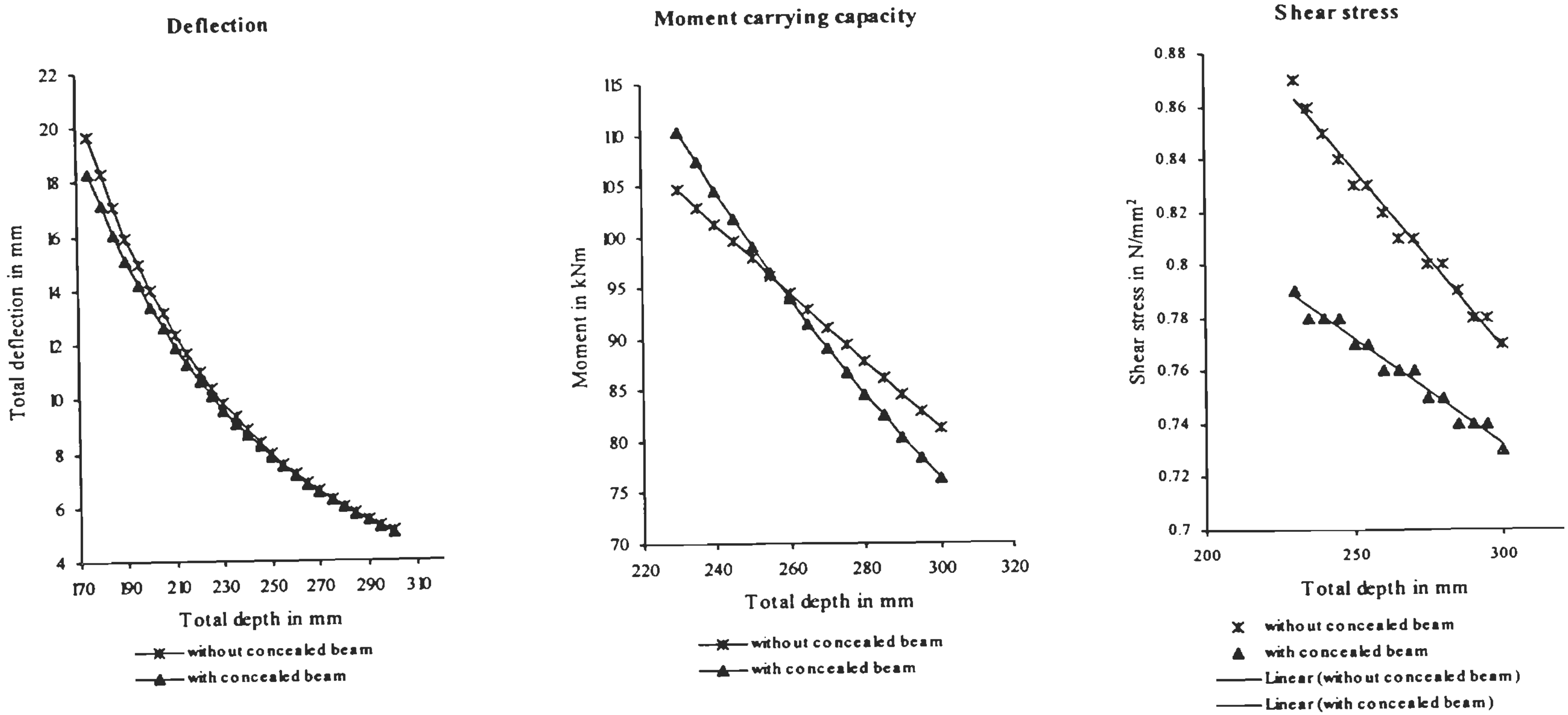


Fig. 7.14 Strength and serviceability of slab without and with concealed beam (6 m x 8 m) fck=30 N/mm^2 wl=3 kN/m^2

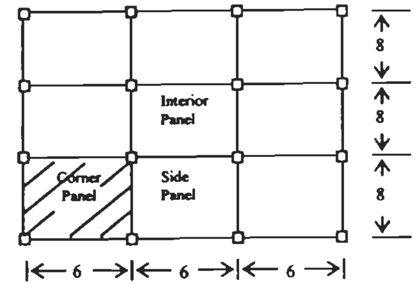


Table 7.15 Strength and serviceability of slab without and with concealed beam (6 m x 8 m) fck=20 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
210	29.15	145.75	18.69	93.45	133.68	115.36	115.88	145.16	125.83	1.09	97.32	0.95	42.41
220	26.25	131.25	16.68	83.40	129.39	129.03	100.28	136.74	105.98	1.06	94.64	0.94	41.96
230	23.74	118.70	14.96	74.80	124.99	143.52	87.09	128.82	89.76	1.03	91.96	0.93	41.52
240	21.57	107.85	13.47	67.35	120.54	158.84	75.89	121.41	76.44	1.01	90.18	0.92	41.07
250	19.67	98.35	12.18	60.90	116.07	175.00	66.33	114.50	65.43	0.99	88.39	0.90	40.18
260	18.01	90.05	11.05	55.25	111.64	192.00	58.15	108.07	56.29	0.97	86.61	0.89	39.73
270	16.54	82.70	10.06	50.30	107.29	209.85	51.13	102.08	48.64	0.95	84.82	0.88	39.29
280	15.24	76.20	9.19	45.95	103.03	228.56	45.08	96.53	42.23	0.93	83.04	0.87	38.84
290	15.24	76.20	8.42	42.10	98.89	248.15	39.85	91.37	36.82	0.92	82.14	0.86	38.39
300	15.24	76.20	7.74	38.70	94.88	268.61	35.32	86.59	32.24	0.90	80.36	0.85	37.95

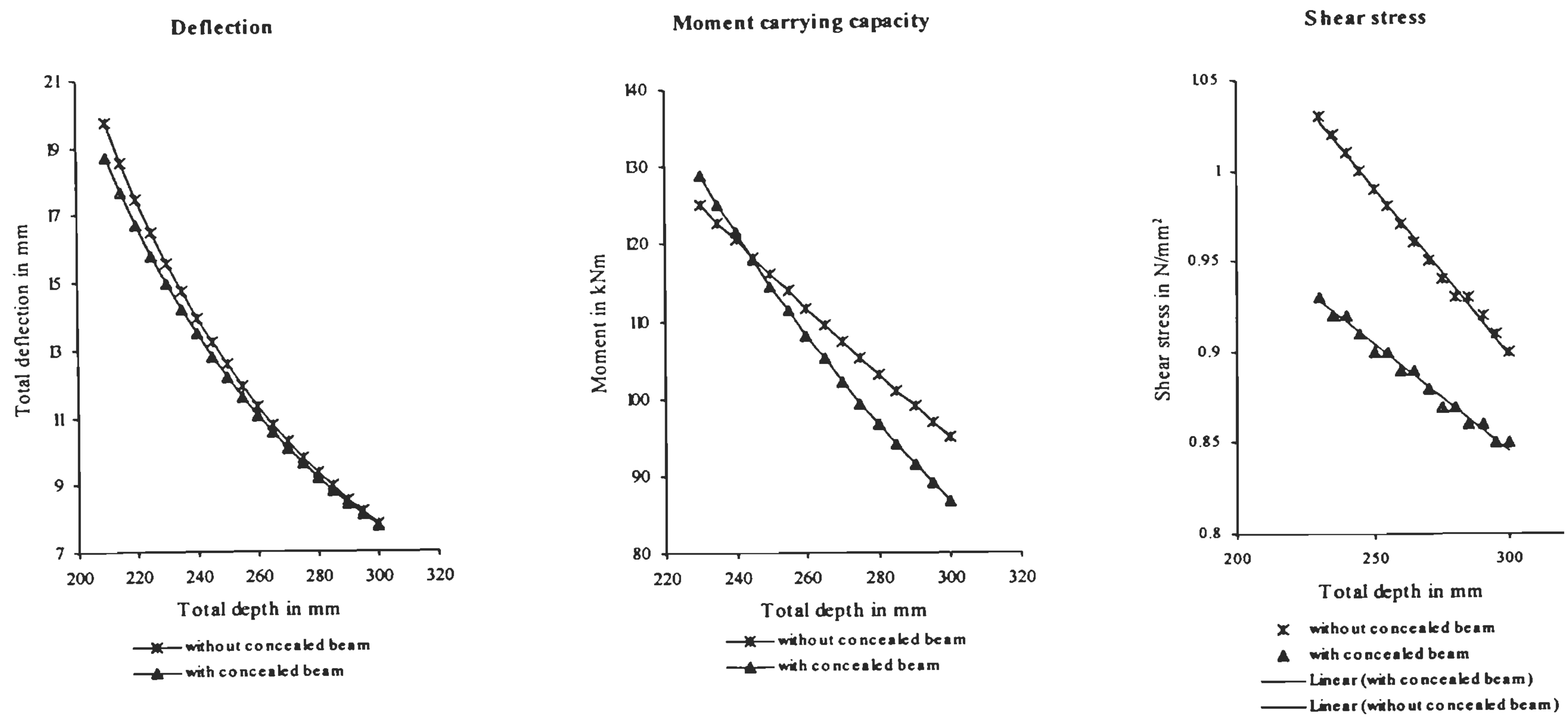


Fig. 7.15 Strength and serviceability of slab without and with concealed beam (6 m x 8 m) fck=20 N/mm^2 wl=5 kN/m^2

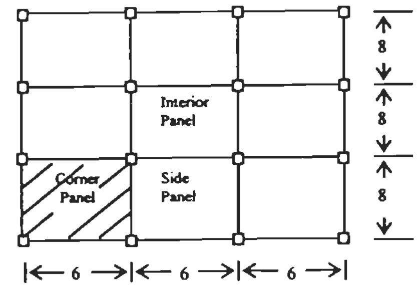


Table 7.16 Strength and serviceability of slab without and with concealed beam (6 m x 8 m) fck=30 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
200	18.31	91.55	17.21	86.05	68.63	153.75	44.64	154.90	100.75	1.12	81.75	0.98	35.77
210	16.11	80.55	15.29	76.45	66.71	173.04	38.55	146.43	84.62	1.09	79.56	0.97	35.40
220	14.27	71.35	13.65	68.25	64.71	193.54	33.43	138.39	71.50	1.06	77.37	0.95	34.67
230	12.71	63.55	12.24	61.20	62.67	215.28	29.11	130.78	60.75	1.03	75.18	0.94	34.31
240	11.38	56.90	11.02	55.10	60.58	238.26	25.43	123.62	51.88	1.01	73.72	0.93	33.94
250	10.23	51.15	9.97	49.85	58.47	262.49	22.28	116.90	44.54	0.99	72.26	0.92	33.58
260	9.25	46.25	9.04	45.20	56.34	287.99	19.56	110.61	38.41	0.97	70.80	0.90	32.85
270	8.39	41.95	8.23	41.15	54.20	314.77	17.22	104.74	33.28	0.95	69.34	0.89	32.48
280	7.64	38.20	7.52	37.60	52.06	342.84	15.18	99.26	28.95	0.93	67.88	0.88	32.12
290	6.98	34.90	6.89	34.45	49.93	372.22	13.41	94.16	25.30	0.92	67.15	0.87	31.75
300	6.40	32.00	6.33	31.65	47.80	402.91	11.86	89.41	22.19	0.90	65.69	0.85	31.02

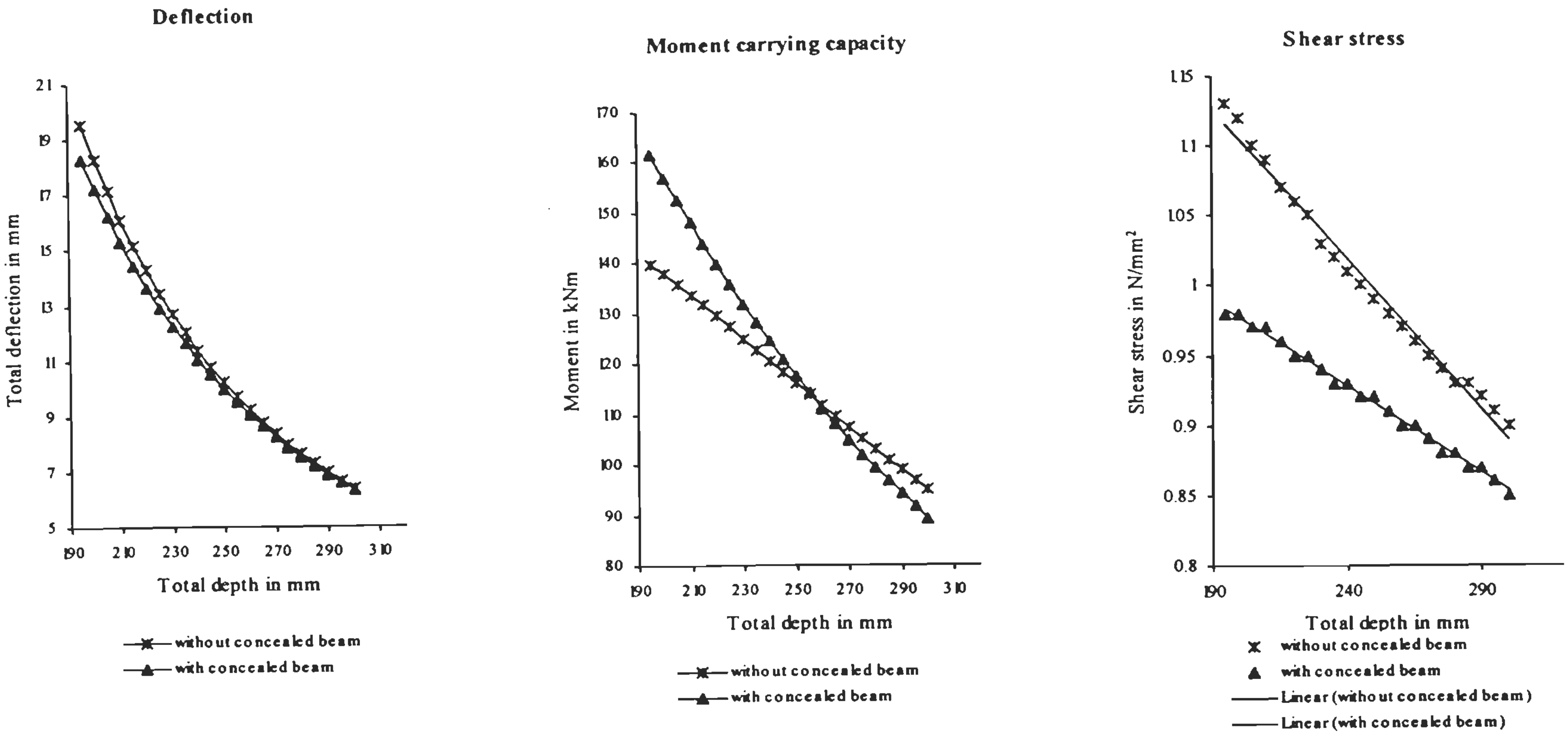


Fig. 7.16 Strength and serviceability of slab without and with concealed beam (6 m x 8 m) fck=30 N/mm^2 wl=5 kN/m^2

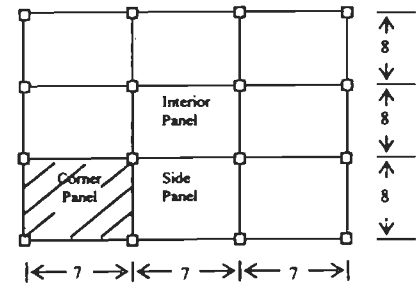


Table 7.17 Strength and serviceability of slab without and with concealed beam (7m x 8 m) fck=20 N/mm² wl=3 kN/m²

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
210	19.01	95.05	18.18	90.90	159.34	133.24	119.59	166.83	125.21	0.95	84.82	0.85	37.95
220	16.89	84.45	16.26	81.30	154.58	148.96	103.77	157.91	106.01	0.94	83.93	0.84	37.50
230	15.09	75.45	14.62	73.10	149.65	165.60	90.37	149.46	90.25	0.92	82.14	0.84	37.50
240	13.55	67.75	13.19	65.95	144.62	183.18	78.95	141.49	77.24	0.90	80.36	0.83	37.05
250	12.22	61.10	11.96	59.80	139.55	201.71	69.18	134.00	66.43	0.89	79.46	0.82	36.61
260	11.08	55.40	10.88	54.40	134.51	221.20	60.81	126.99	57.41	0.88	78.57	0.81	36.16
270	10.08	50.40	9.93	49.65	129.52	241.64	53.60	120.42	49.83	0.86	76.79	0.81	36.16
280	9.21	46.05	9.09	45.45	124.63	263.06	47.38	114.30	43.45	0.85	75.89	0.80	35.71
290	8.44	42.20	8.35	41.75	119.86	285.46	41.99	108.58	38.04	0.84	75.00	0.79	35.27
300	7.76	38.80	7.69	38.45	115.23	308.85	37.31	103.25	33.43	0.83	74.11	0.78	34.82

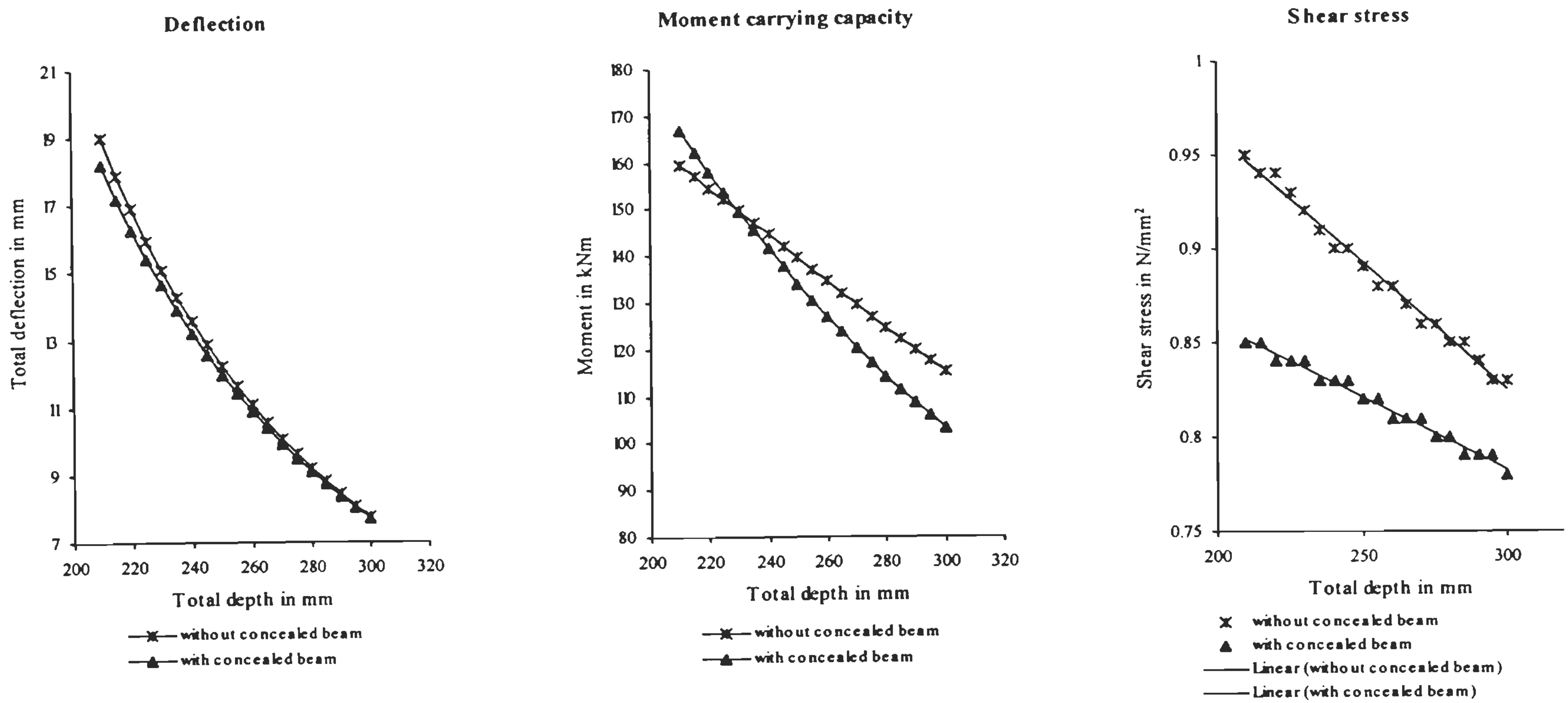


Fig. 7.17 Strength and serviceability of slab without and with concealed beam (7 m x 8 m) fck=20 N/mm^2 wl=3 kN/m^2

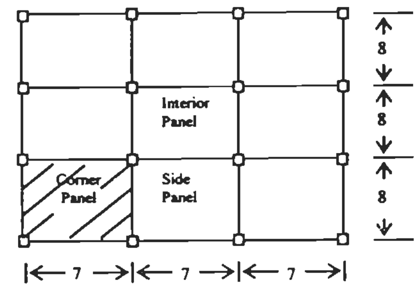


Table 7.18 Strength and serviceability of slab without and with concealed beam (7m x 8 m) fck=30 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
200	17.58	87.90	16.70	83.50	163.86	177.67	92.23	179.39	100.97	0.97	70.80	0.87	31.75
210	15.52	77.60	14.87	74.35	159.34	199.86	79.73	170.16	85.14	0.95	69.34	0.86	31.39
220	13.79	68.95	13.30	66.50	154.58	223.43	69.18	161.31	72.20	0.94	68.61	0.85	31.02
230	12.32	61.60	11.95	59.75	149.65	248.40	60.25	152.90	61.55	0.92	67.15	0.85	31.02
240	11.06	55.30	10.79	53.95	144.62	274.77	52.63	144.93	52.75	0.90	65.69	0.84	30.66
250	9.98	49.90	9.78	48.90	139.55	302.57	46.12	137.42	45.42	0.89	64.96	0.83	30.29
260	9.05	45.25	8.89	44.45	134.51	331.79	40.54	130.36	39.29	0.88	64.23	0.82	29.93
270	8.23	41.15	8.12	40.60	129.52	362.47	35.73	123.73	34.14	0.86	62.77	0.81	29.56
280	7.52	37.60	7.43	37.15	124.63	394.59	31.58	117.53	29.79	0.85	62.04	0.81	29.56
290	6.89	34.45	6.83	34.15	119.86	428.19	27.99	111.74	26.10	0.84	61.31	0.80	29.20
300	6.33	31.65	6.29	31.45	115.23	463.27	24.87	106.32	22.95	0.83	60.58	0.79	28.83

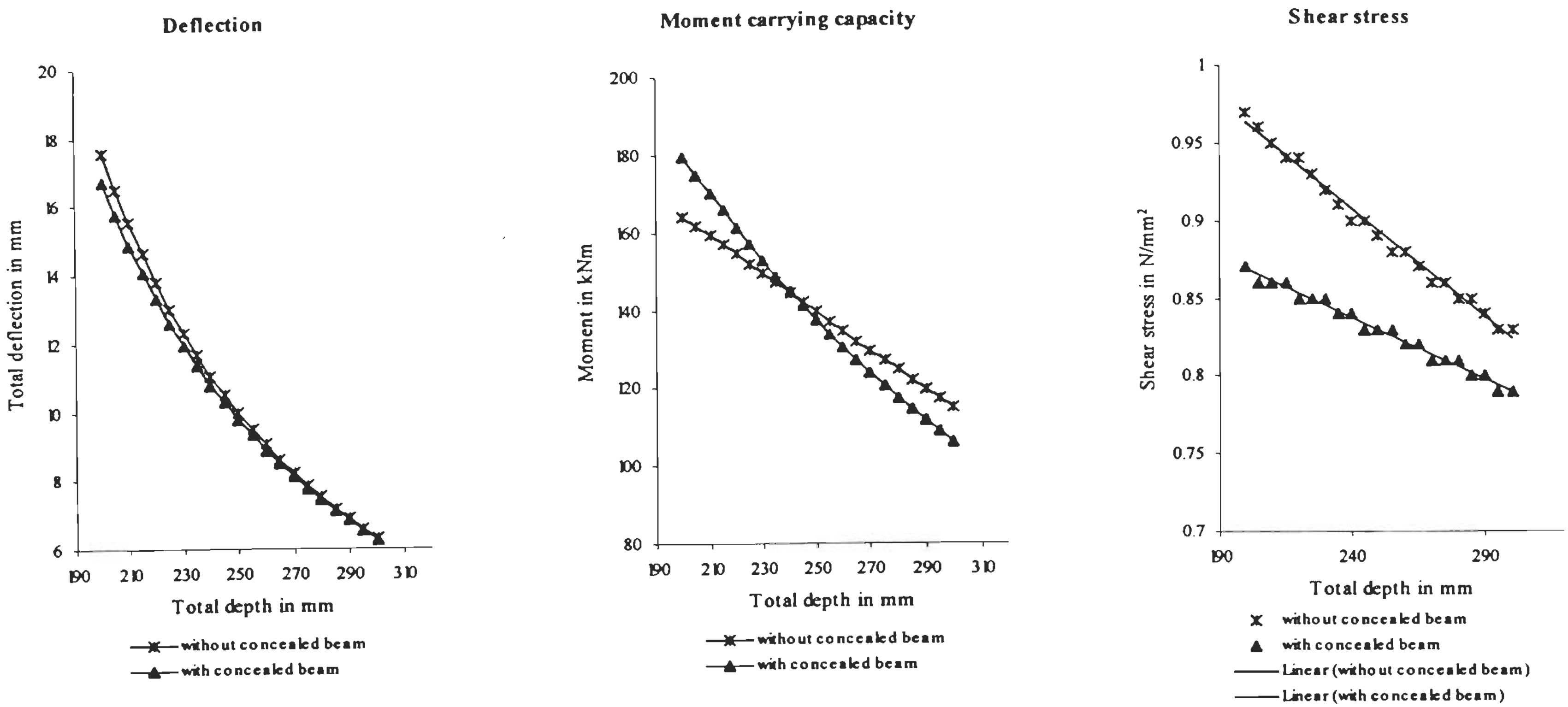


Fig. 7.18 Strength and serviceability of slab without and with concealed beam (7 m x 8 m) fck=30 N/mm^2 wl=3 kN/m^2

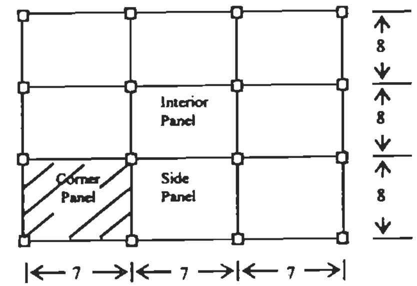


Table 7.19 Strength and serviceability of slab without and with concealed beam (7m x 8 m) fck=20 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
230	19.69	98.45	18.84	94.20	178.85	165.60	108.00	178.63	107.87	1.10	98.21	1.00	44.64
235	18.62	93.10	17.87	89.35	175.51	174.27	100.71	173.45	99.53	1.09	97.32	0.99	44.20
240	17.62	88.10	16.97	84.85	172.17	183.18	93.99	168.44	91.95	1.08	96.43	0.99	44.20
245	16.69	83.45	16.12	80.60	168.83	192.33	87.78	163.61	85.07	1.06	94.64	0.98	43.75
250	15.84	79.20	15.33	76.65	165.52	201.71	82.06	158.93	78.79	1.05	93.75	0.97	43.30
255	15.04	75.20	14.60	73.00	162.23	211.33	76.77	154.42	73.07	1.04	92.86	0.97	43.30
260	14.30	71.50	13.91	69.55	158.96	221.20	71.86	150.07	67.84	1.03	91.96	0.96	42.86
265	13.61	68.05	13.26	66.30	155.74	231.30	67.33	145.88	63.07	1.03	91.96	0.96	42.86
270	12.97	64.85	12.66	63.30	152.55	241.64	63.13	141.83	58.69	1.02	91.07	0.95	42.41
275	12.36	61.80	12.09	60.45	149.40	252.23	59.23	137.93	54.68	1.01	90.18	0.94	41.96
280	11.80	59.00	11.56	57.80	146.30	263.06	55.61	134.17	51.00	1.00	89.29	0.94	41.96
285	11.27	56.35	11.06	55.30	143.26	274.14	52.26	130.55	47.62	0.99	88.39	0.93	41.52
290	10.78	53.90	10.59	52.95	140.26	285.46	49.13	127.06	44.51	0.98	87.50	0.93	41.52
295	10.31	51.55	10.15	50.75	137.32	297.03	46.23	123.70	41.65	0.97	86.61	0.92	41.07
300	9.88	49.40	9.73	48.65	134.44	308.85	43.53	120.46	39.00	0.97	86.61	0.91	40.63

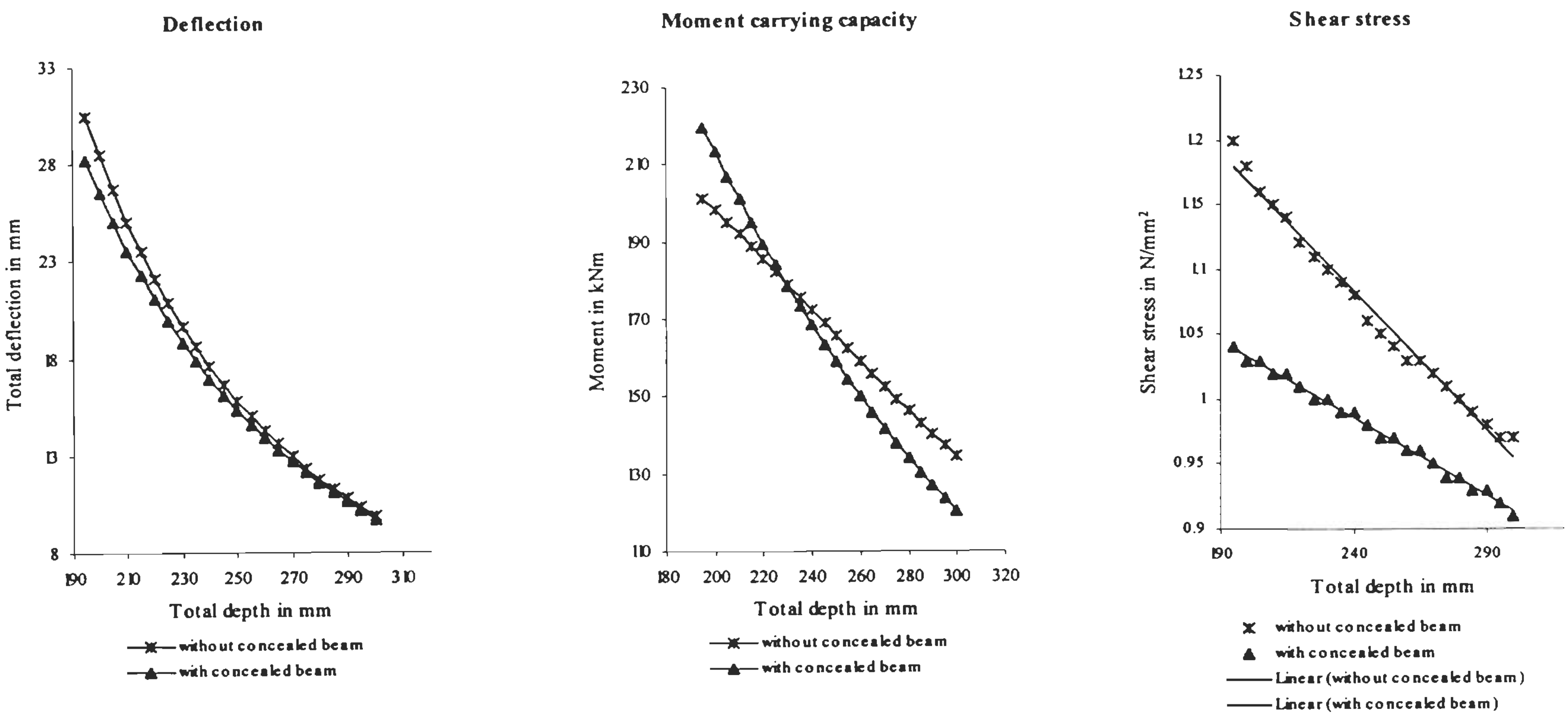


Fig. 7.19 Strength and serviceability of slab without and with concealed beam (7 m x 8 m) fck=20 N/mm^2 wl=5 kN/m^2

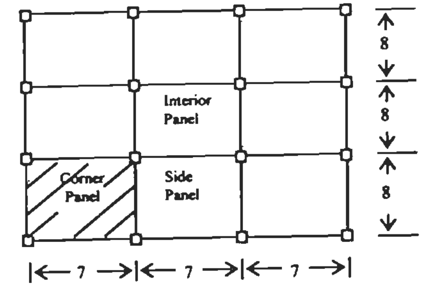


Table 7.20 Strength and serviceability of slab without and with concealed beam (7m x 8 m) fck=30 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
215	19.20	96.00	18.19	90.95	188.78	211.47	89.27	199.24	94.22	1.14	83.21	1.03	37.59
220	18.07	90.35	17.20	86.00	185.50	223.43	83.02	193.58	86.64	1.12	81.75	1.02	37.23
225	17.03	85.15	16.27	81.35	182.18	235.74	77.28	188.07	79.78	1.11	81.02	1.02	37.23
230	16.08	80.40	15.42	77.10	178.85	248.40	72.00	182.73	73.56	1.10	80.29	1.01	36.86
235	15.20	76.00	14.62	73.10	175.51	261.41	67.14	177.55	67.92	1.09	79.56	1.00	36.50
240	14.39	71.95	13.88	69.40	172.17	274.77	62.66	172.54	62.79	1.08	78.83	1.00	36.50
245	13.63	68.15	13.19	65.95	168.83	288.49	58.52	167.68	58.12	1.06	77.37	0.99	36.13
250	12.93	64.65	12.54	62.70	165.52	302.57	54.70	162.99	53.87	1.05	76.64	0.99	36.13
255	12.28	61.40	11.94	59.70	162.23	317.00	51.18	158.45	49.98	1.04	75.91	0.98	35.77
260	11.68	58.40	11.38	56.90	158.96	331.79	47.91	154.06	46.43	1.03	75.18	0.97	35.40
265	11.11	55.55	10.85	54.25	155.74	346.95	44.89	149.82	43.18	1.03	75.18	0.97	35.40
270	10.59	52.95	10.36	51.80	152.55	362.47	42.09	145.73	40.20	1.02	74.45	0.96	35.04
275	10.10	50.50	9.89	49.45	149.40	378.35	39.49	141.78	37.47	1.01	73.72	0.95	34.67
280	9.64	48.20	9.46	47.30	146.30	394.59	37.08	137.97	34.97	1.00	72.99	0.95	34.67
285	9.20	46.00	9.05	45.25	143.26	411.21	34.84	134.30	32.66	0.99	72.26	0.94	34.31
290	8.80	44.00	8.66	43.30	140.26	428.19	32.76	130.75	30.54	0.98	71.53	0.93	33.94
295	8.42	42.10	8.30	41.50	137.32	445.55	30.82	127.34	28.58	0.97	70.80	0.93	33.94
300	8.06	40.30	7.96	39.80	134.44	463.27	29.02	124.04	26.77	0.97	70.80	0.92	33.58

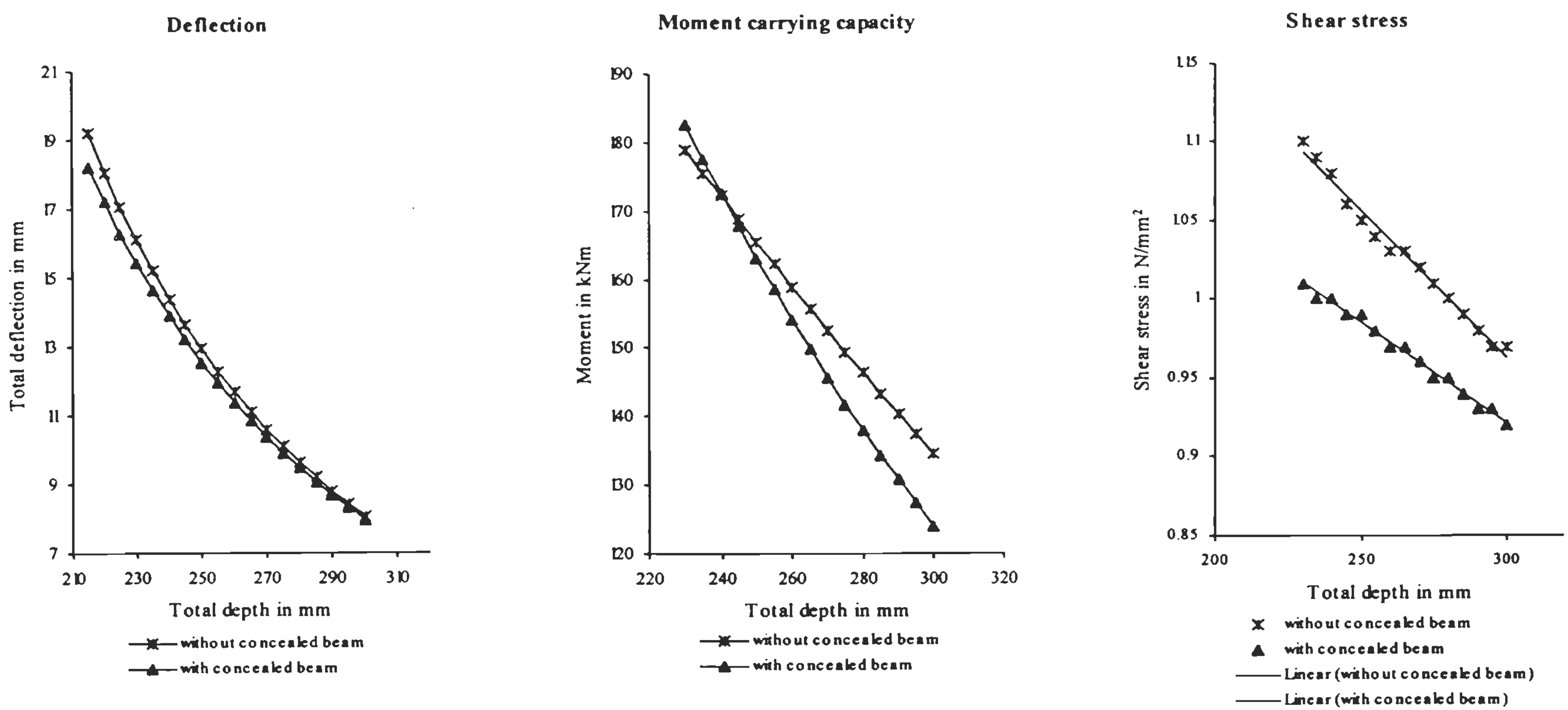


Fig. 7.20 Strength and serviceability of slab without and with concealed beam (7 m x 8 m) fck=30 N/mm^2 wl=5 kN/m^2

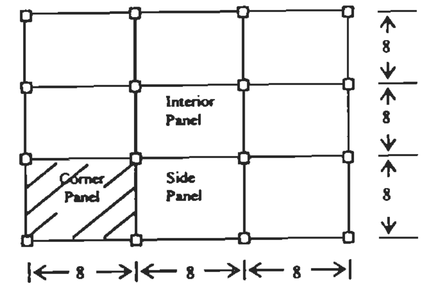


Table 7.21 Strength and serviceability of slab without and with concealed beam (8m x 8 m) fck=20 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
230	19.78	98.90	19.17	95.85	204.11	187.68	108.75	198.96	106.01	0.96	85.71	0.89	39.73
235	18.73	93.65	18.20	91.00	200.57	197.47	101.57	193.63	98.06	0.95	84.82	0.88	39.29
240	17.76	88.80	17.30	86.50	197.01	207.53	94.93	188.45	90.81	0.95	84.82	0.88	39.29
245	16.86	84.30	16.46	82.30	193.45	217.84	88.80	183.44	84.21	0.94	83.93	0.88	39.29
250	16.03	80.15	15.68	78.40	189.89	228.43	83.13	178.58	78.18	0.93	83.04	0.87	38.84
255	15.25	76.25	14.94	74.70	186.35	239.28	77.88	173.87	72.66	0.93	83.04	0.87	38.84
260	14.52	72.60	14.26	71.30	182.83	250.40	73.02	169.31	67.62	0.92	82.14	0.86	38.39
265	13.85	69.25	13.62	68.10	179.34	261.78	68.51	164.90	62.99	0.92	82.14	0.86	38.39
270	13.21	66.05	13.01	65.05	175.88	273.44	64.32	160.64	58.75	0.91	81.25	0.86	38.39
275	12.62	63.10	12.45	62.25	172.46	285.36	60.44	156.52	54.85	0.90	80.36	0.85	37.95
280	12.07	60.35	11.92	59.60	169.09	297.56	56.83	152.54	51.26	0.90	80.36	0.85	37.95
285	11.55	57.75	11.42	57.10	165.76	310.03	53.47	148.69	47.96	0.89	79.46	0.85	37.95
290	11.06	55.30	10.95	54.75	162.49	322.78	50.34	144.98	44.92	0.89	79.46	0.84	37.50
295	10.60	53.00	10.50	52.50	159.27	335.79	47.43	141.39	42.11	0.88	78.57	0.84	37.50
300	10.16	50.80	10.08	50.40	156.10	349.09	44.72	137.92	39.51	0.88	78.57	0.83	37.05

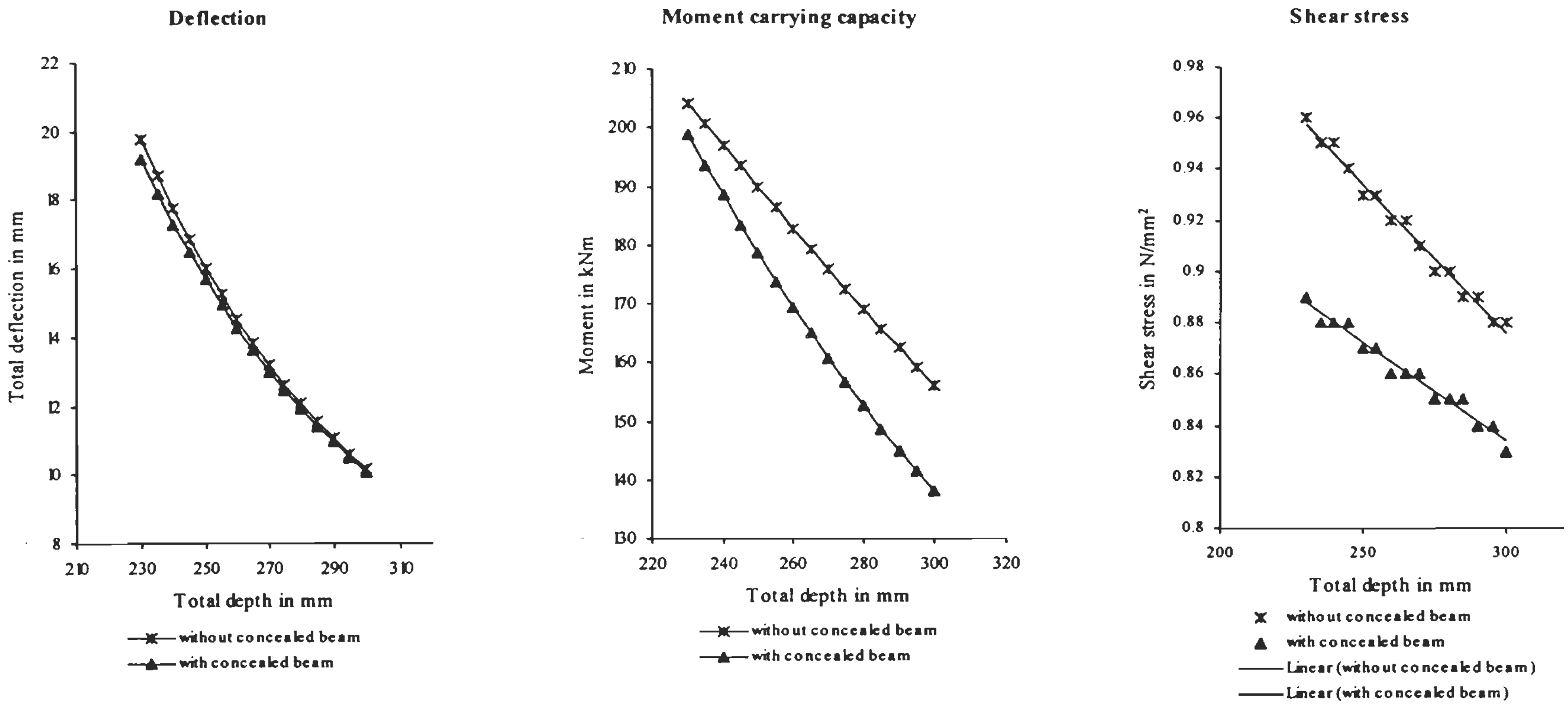


Fig. 7.21 Strength and serviceability of slab without and with concealed beam (8 m x 8 m) fck=20 N/mm^2 wl=3 kN/m^2

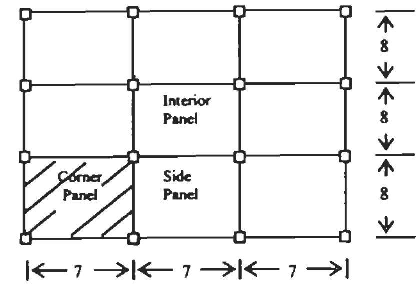


Table 7.22 Strength and serviceability of slab without and with concealed beam (7m x 8 m) fck=30 N/mm^2 wl=3 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
215	19.17	95.85	18.43	92.15	214.57	239.81	89.48	220.30	91.86	0.99	72.26	0.91	33.21
220	18.08	90.40	17.44	87.20	211.13	253.32	83.35	214.56	84.70	0.98	71.53	0.90	32.85
225	17.08	85.40	16.52	82.60	207.64	267.23	77.70	208.95	78.19	0.97	70.80	0.90	32.85
230	16.15	80.75	15.67	78.35	204.11	281.52	72.50	203.49	72.28	0.96	70.07	0.90	32.85
235	15.29	76.45	14.88	74.40	200.57	296.21	67.71	198.17	66.90	0.95	69.34	0.89	32.48
240	14.50	72.50	14.15	70.75	197.01	311.29	63.29	193.00	62.00	0.95	69.34	0.89	32.48
245	13.77	68.85	13.46	67.30	193.45	326.77	59.20	187.97	57.52	0.94	68.61	0.88	32.12
250	13.09	65.45	12.82	64.10	189.89	342.64	55.42	183.09	53.44	0.93	67.88	0.88	32.12
255	12.45	62.25	12.22	61.10	186.35	358.92	51.92	178.36	49.69	0.93	67.88	0.88	32.12
260	11.86	59.30	11.66	58.30	182.83	375.60	48.68	173.78	46.27	0.92	67.15	0.87	31.75
265	11.30	56.50	11.13	55.65	179.34	392.67	45.67	169.33	43.12	0.92	67.15	0.87	31.75
270	10.79	53.95	10.64	53.20	175.88	410.16	42.88	165.03	40.24	0.91	66.42	0.86	31.39
275	10.31	51.55	10.18	50.90	172.46	428.05	40.29	160.86	37.58	0.90	65.69	0.86	31.39
280	9.85	49.25	9.74	48.70	169.09	446.34	37.88	156.83	35.14	0.90	65.69	0.86	31.39
285	9.43	47.15	9.33	46.65	165.76	465.05	35.64	152.93	32.88	0.89	64.96	0.85	31.02
290	9.03	45.15	8.95	44.75	162.49	484.16	33.56	149.16	30.81	0.89	64.96	0.85	31.02
295	8.65	43.25	8.59	42.95	159.27	503.69	31.62	145.52	28.89	0.88	64.23	0.84	30.66
300	8.30	41.50	8.24	41.20	156.10	523.63	29.81	141.99	27.12	0.88	64.23	0.84	30.66

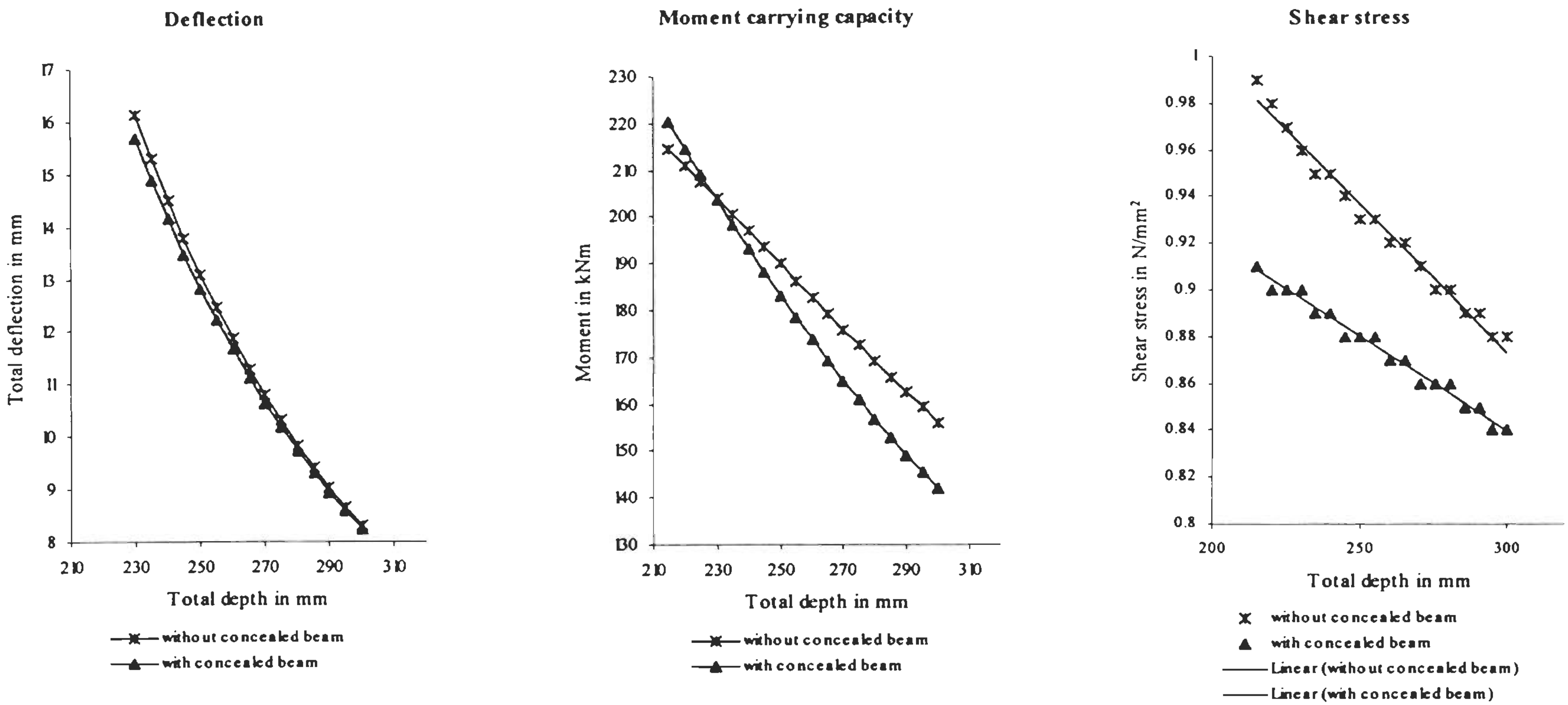


Fig. 7.22 Strength and serviceability of slab without and with concealed beam (8 m x 8 m) fck=30 N/mm^2 wl=3 kN/m^2

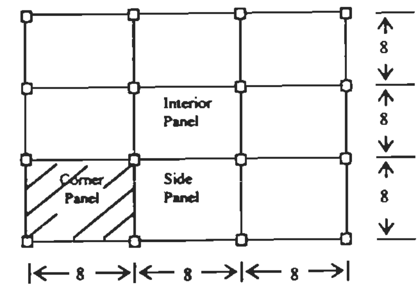


Table 7.23 Strength and serviceability of slab without and with concealed beam (8m x 8 m) fck=20 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
255	19.70	98.50	19.12	95.60	220.62	239.28	92.20	205.84	86.02	1.10	98.21	1.03	45.98
260	18.73	93.65	18.22	91.10	216.07	250.40	86.29	200.10	79.91	1.09	97.32	1.02	45.54
265	17.83	89.15	17.38	86.90	211.58	261.78	80.82	194.55	74.32	1.08	96.43	1.02	45.54
270	16.98	84.90	16.58	82.90	207.15	273.44	75.76	189.20	69.19	1.07	95.54	1.01	45.09
275	16.19	80.95	15.84	79.20	202.78	285.36	71.06	184.04	64.49	1.06	94.64	1.00	44.64
280	15.46	77.30	15.14	75.70	198.49	297.56	66.71	179.07	60.18	1.05	93.75	1.00	44.64
285	14.76	73.80	14.49	72.45	194.28	310.03	62.66	174.27	56.21	1.05	93.75	0.99	44.20
290	14.12	70.60	13.87	69.35	190.14	322.78	58.91	169.65	52.56	1.04	92.86	0.98	43.75
295	13.51	67.55	13.29	66.45	186.09	335.79	55.42	165.20	49.20	1.03	91.96	0.98	43.75
300	12.93	64.65	12.74	63.70	182.12	349.09	52.17	160.91	46.09	1.02	91.07	0.97	43.30

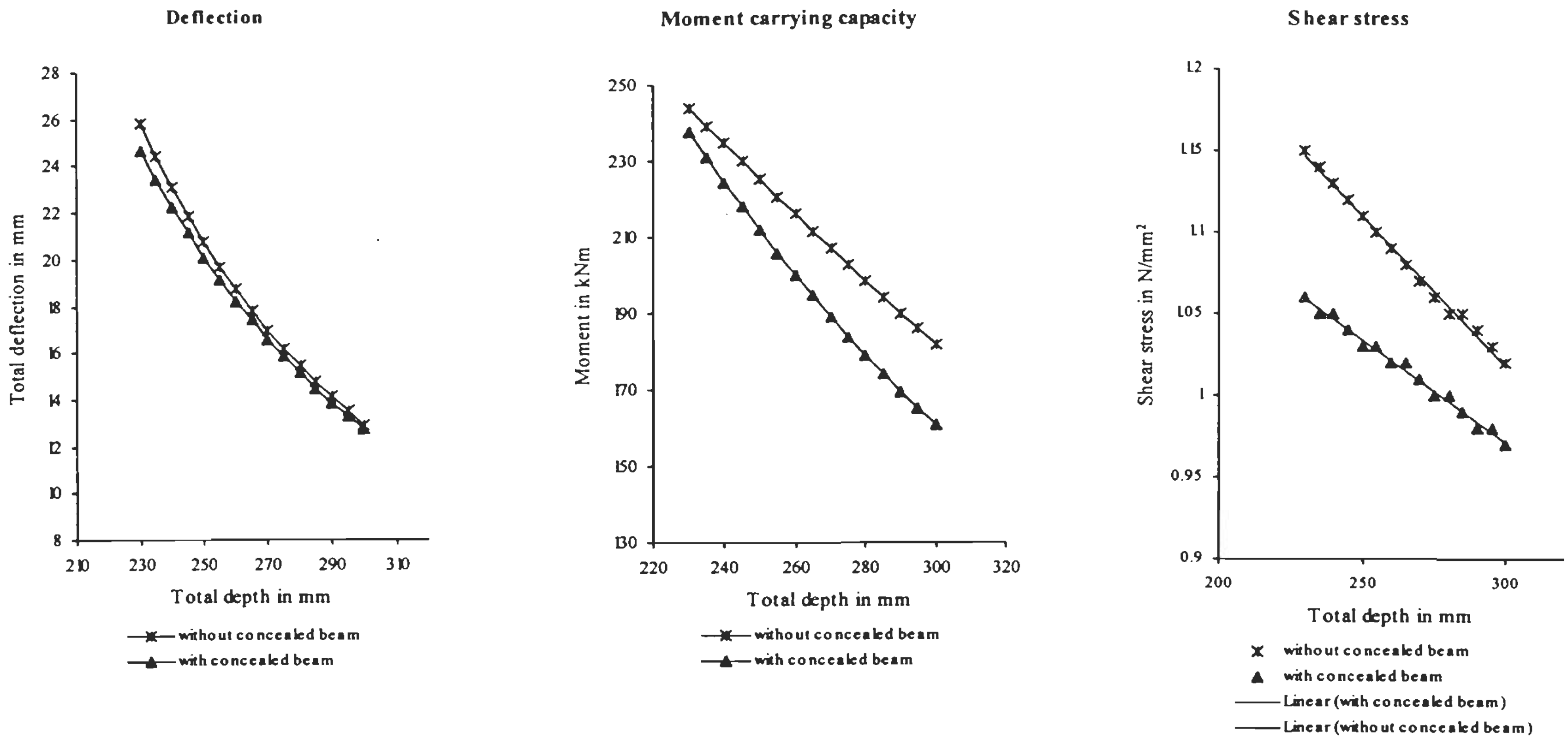


Fig. 7.23 Strength and serviceability of slab without and with concealed beam (8 m x 8 m) fck=20 N/mm^2 wl=5kN/m^2

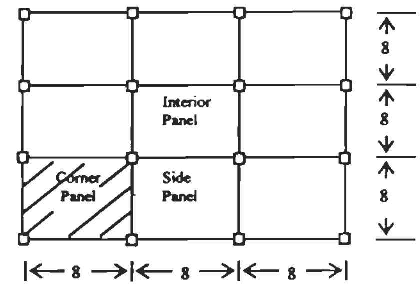


Table 7.24 Strength and serviceability of slab without and with concealed beam (8m x 8 m) fck=30 N/mm^2 wl=5 kN/m^2

Depth	Deflection				Moment					Shear stress			
	Without concealed beam		With concealed beam		Without concealed beam			With concealed beam		Without concealed beam		With concealed beam	
mm	mm	Used %	mm	Used %	kNm	limiting	Used %	kNm	Used %	N/mm^2	Used %	N/mm^2	Used %
235	19.91	99.55	19.15	95.75	239.23	296.21	80.76	236.37	79.80	0.90	65.69	1.06	38.69
240	18.85	94.25	18.18	90.90	234.53	311.29	75.34	229.76	73.81	0.89	64.96	1.06	38.69
245	17.86	89.30	17.28	86.40	229.86	326.77	70.34	223.35	68.35	0.87	63.50	1.05	38.32
250	16.94	84.70	16.43	82.15	225.22	342.64	65.73	217.16	63.38	0.86	62.77	1.04	37.96
255	16.09	80.45	15.64	78.20	220.62	358.92	61.47	211.16	58.83	0.85	62.04	1.04	37.96
260	15.29	76.45	14.90	74.50	216.07	375.60	57.53	205.37	54.68	0.83	60.58	1.03	37.59
265	14.56	72.80	14.21	71.05	211.58	392.67	53.88	199.77	50.87	0.82	59.85	1.02	37.23
270	13.87	69.35	13.57	67.85	207.15	410.16	50.50	194.37	47.39	0.81	59.12	1.02	37.23
275	13.22	66.10	12.96	64.80	202.78	428.05	47.37	189.15	44.19	0.80	58.39	1.01	36.86
280	12.62	63.10	12.39	61.95	198.49	446.34	44.47	184.11	41.25	0.79	57.66	1.01	36.86
285	12.05	60.25	11.85	59.25	194.28	465.05	41.78	179.24	38.54	0.78	56.93	1.00	36.50
290	11.52	57.60	11.35	56.75	190.14	484.16	39.27	174.55	36.05	0.77	56.20	0.99	36.13
295	11.03	55.15	10.87	54.35	186.09	503.69	36.95	170.02	33.75	0.76	55.47	0.99	36.13
300	10.56	52.80	10.42	52.10	182.12	523.63	34.78	165.66	31.64	0.75	54.74	0.98	35.77

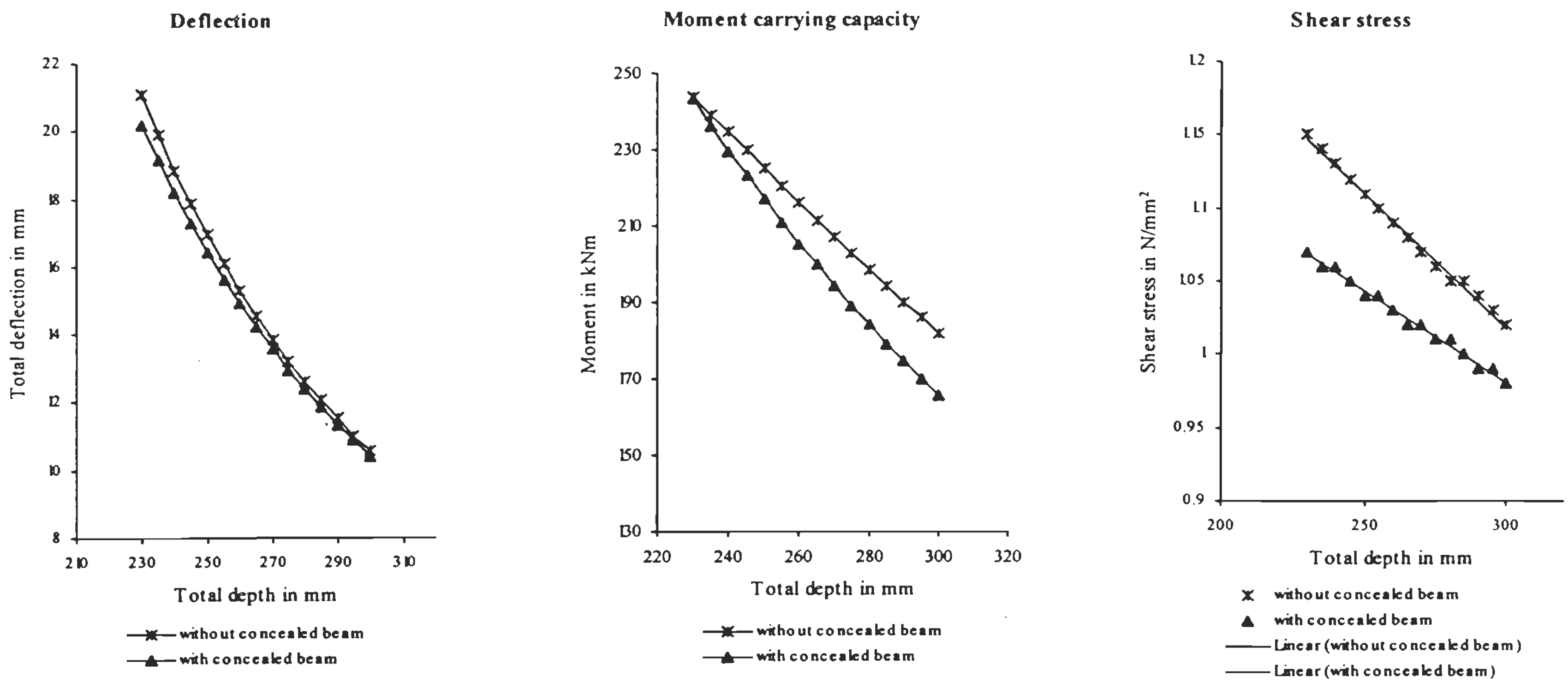


Fig. 7.24 Strength and serviceability of slab without and with concealed beam (8 m x 8 m) fck=30 N/mm^2 wl=5kN/m^2

7.3 SUMMARY

From the comparative study made, it is observed that the concealed beam is effective in reducing the deflection, increasing the moment carrying capacity and reducing the shear stress. The results obtained for flat plates and flat slab with column head are furnished and discussed in this chapter. The major conclusions drawn from the study are explained in the next chapter.

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 SUMMARY

Prefabricated reinforcement systems are available to address the punching shear problems in flat plates. But these systems need skilled labour, to fix them in position. In certain systems the minimum depth of slab required is 250mm. Therefore, the main aim of this research was kept as to find the ways to improve the stiffness of slab in terms of flexure, shear and torsion, without increasing the thickness of slab.

Hence the concept of providing the concealed beam in the column strip of slab was considered for this research. The concealed beam is an arrangement of reinforcement having equal number of longitudinal reinforcement in both tension and compression face of the slab tied together by means of shear stirrups along the transverse direction.

The main objective of this research was achieved by adopting the following methodology.

- The available provisions in four codes of practice namely (i) ACI: 318-2002 (ii) BS: 8110-1997 (iii) EC: 2-2002 and (iv) IS: 456-2000 were compared for the prediction of deflections in flat plates and flat slabs. The roles of the parameters, in the available empirical provisions suggested in the codes for prediction of deflection were studied.
- The available beam formulas as per the above listed codes were used with suitable modifications using the equivalent frame method. This method is chosen, since it considers the entire slab into two wide beam strips (column strip and middle strip). This will give very little modification in the beam formulas. The width of column strip and middle strip are to be substituted in place of width of the beam.

- A computer program in MATLAB was developed as per the above model to compute the mid panel deflection for interior panel, corner panel and side panel of flat plates.
- An extensive parametric study was conducted by using the developed program.
- The influence of the parameters such as span, load, grade of concrete, area of steel, clear cover to reinforcement, on total deflections were found out.
- The contributions of creep and shrinkage effects in the total deflection as per each code for different parameters were studied.
- From the parametric study conducted it was found that the EC:2-2002 gives the most reliable empirical equations to obtain the total deflection out of those suggested by four codes for flat plate including creep and shrinkage effects. This code considers the tension stiffening effect of concrete. This was compared with the experimental results in the literature of Gilbert (2005). There was good agreement between the calculated values and the experimental results.
- The above approach involves many factors that will not be available at the design stage. Hence it was essential to have a simplified approach like the multiplier approach followed by ACI: 318-2002. This underestimates the actual deflection.
- Hence based on the parametric study conducted, the contributions of creep and shrinkage effects as per each code were obtained using the developed program. Since the IS: 456-2000 underestimates the percentage contribution of creep and shrinkage effects (less than 40%), the results of other three codes were considered.
- Based on the above, a rational approach that can be used for deflection control of flat plates and flat slabs at the design stage by changing the factor in the multiplier approach was proposed.

- The 2-D frame was analysed using the Equivalent Frame Method and the moment and shear acting on the slab due to gravity loads were obtained in this analysis.
- The total mid panel deflection by using the above-developed common model for interior, corner and side panel of flat slabs with column head, drop panel, drop panel & column head was found for the spans upto which each slab can be used.
- The concealed beam was modelled suitably in flexure, shear and torsion to analyse the stiffened flat plate and flat slab with column head by Equivalent Frame Method. For flexure the normal reinforced concrete theory was used to model the concealed beam. For torsion the concealed beam was modelled as thin equivalent concrete box section.
- After studying the influence of the area of longitudinal reinforcements, spacing of stirrups in the concealed beam the dimensions were fixed for the concealed beam.
- The final mid panel deflection, moment and shear carrying capacity of flat plates and flat slab with column head after fixing the parameters for concealed beam based on the above study were obtained using the developed program.

8.2 CONCLUSIONS

This section highlights conclusions based on the detailed discussions and design studies presented in the earlier chapters. Based on these studies the following broad conclusions are drawn.

1. The concealed beam is effective in improving the stiffness of the flat plates and flat slab with column head without increasing the depth of the slab. The moment carrying capacity of slab at edge panel increased to 1.5 times than the slab without concealed beam. The mid panel deflection of beamless floor is reduced by 5%.

2. **When the concrete depth is increased, the effect of concealed beam is getting** reduced. In other words, it can be concluded that it is very much effective in lesser depth. This is beneficial, since the lesser depth result in reduced dead load moment.

3. In case of flat slab with drop panel or flat slab with column head and drop panel, the effect of concealed beam is negligible. The concrete depth at slab-column connection itself meets the required safety and serviceability aspects.

4. The influence of number of stirrups has very little effect in improving the stiffness of slab. The main reinforcement (10 mm) and stirrups (8 mm) to be used will be of the same diameter that may be used for the slabs and beams in the building; the concealed beam does not require any special type of material. Hence the concealed beam will be easy to fabricate and fix it in position in the slab. Skilled labour is not required for this.

5. The ACI: 318-2002 method, using the multiplier approach under estimates the actual total mid panel deflection of flat plates and slabs. IS: 456-2000 under estimates the creep and shrinkage effects in total deflection. Among all the four codes studied the Euro Code (EC: 2 -2002) considers all the parameters and gives most reliable solution to predict the total deflection including the long term effects.

6. The contribution of creep and shrinkage in total deflection varies from 80% to maximum 98%. As per the EC: 2-2002, the short term deflection is negligible.

7. To find the reliable depth required for the flat plate and flat slab to meet the long term effects, the factor in the multiplier approach can be modified to '6'. Hence the proposed modified multiplier approach is given below:

   $$\delta_T = 6 \times \delta_D + \delta_l$$

   The above proposed modified multiplier approach is validated with the experimental results available in the literature.

8. The empirical formula as per EC: 2-2002 method with the present modeling gives a more reliable solution to find the long-term deflection of flat plates. The curvatures can be computed by using the following expressions. If all the parameters are available at the design stage, this proposed rational approach can be used.

$$\text{Shrinkage curvature} = \xi(1/r_{cs2}) + [(1-\xi)(1/r_{cs1})]$$

$$\text{Creep curvature} = \xi(1/r_2) + (1-\xi)(1/r_1)$$

The above proposed rational method was validated with experimental results of Gilbert (2005) and a good correlation between the computed and the experimental results was found.

9. Substituting the material properties as per the relevant code, in the suitably modified formula, the depth required for the particular parameters are more or less same.

10. For a load of 3 kN/m^2 the flat plate can be used upto 7 m with concrete grade 20, whereas for increased load with this grade the flat slab with column head can be used. For load 3 kN/m^2, with higher concrete grade plate can be used upto 8 m. For load 5 kN/m^2, the plate can be used with higher grade of concrete. Beyond 7 m upto 8 m, the flat slab with column head can be used for 3 kN/m^2 with concrete grade 20. The flat slab with drop panel can be used safely for span upto 10 m. Beyond 10 m to 12 m the flat slab with column head and drop panel can be used.

11. The depth of slab, span and live load are the main parameters that have significant contribution in total deflection. The increase in span and live load increase the total deflection. Increase in depth and strength of concrete reduces the deflection. The increase in clear cover by 5 mm increases the deflection by 5.11% as per IS: 456-2000. Since the ACI: 318-2002 uses multiplier approach the parameter is not considered for the study. But this does not have much influence as per BS: 8110-1997 and EC: 2-2002.

8.3 RECOMMENDATIONS FOR FUTURE RESEARCH

The scope for future work in this area is as follows:

1. The behaviour of flat plate and flat slab with column head with concealed beam by conducting the experiments under construction loads and for service loads.
2. The behaviour of flat plate with concealed beam by conducting the experimental and analytical study for lateral loading.
3. The mode of failure of slab at ultimate load has to be studied.

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★ International Conference

4. Santhi A.S., Jagdish Prasad and A.K.Ahuja, "A Software Tool to Compute the Long Term Deflection of the Two-way RC Flat Plates", Indo-Australian Conference on IT Applications in Civil Engineering, IIT Roorkee, India, February 20-21, 2006.

★ International Journals

5. Santhi A.S., Jagdish Prasad and A.K.Ahuja, "Effect of Creep and Shrinkage on the Deflection of RCC Two-way Flat Plates", *Asian Journal of Civil Engineering (Building and Housing)* (Communicated).